Appendix D:
DRAFT Gold Ray Dam Project Rehabilitation Technical Memo

Gold Ray Dam Project

—DRAFT—

Rehabilitation Technical Memo

February 2010

Prepared by:

Table of Contents

List of Figures

List of Tables

List of Appendices

EXECUTIVE SUMMARY

Jackson County is evaluating the required components of rehabilitating the Gold Ray Dam complex to meet current dam, fish ladder, and fish screening design standards, thus addressing the issues of liability and substandard fish passage facilities posed by the existing dam. This evaluation also considers rehabilitation to reestablish hydropower generation at Gold Ray Dam. The purpose of this study is to determine the costs and structural modifications that will be required to leave the dam in place. In addition, there are substantial regulatory and legal challenges associated with its implementation. Although this work does not meet the purpose and need of the Environmental Assessment (EA) for improving fish passage and addressing Jackson County's liability at Gold Ray Dam, it is being analyzed because of stakeholder interest raised during scoping for the EA. The proposed elements of dam rehabilitation are described below. The project vicinity is shown in Figure 1.

Dam Rehabilitation

The design of Gold Ray Dam is know as a slab and buttress design, consisting of an inclined upstream slab supported between downstream buttresses (Figure 5). The majority of dam and water control structures are in a deteriorated state. There is evidence of significant concrete deterioration, cracking, and displacement throughout the site (HDR, 2010a).

Rehabilitation of Gold Ray Dam to meet current dam safety standards would be most efficiently accomplished by converting the existing slab and buttress design into a gravity dam. This would be accomplished by filling the voids beneath the slabs with concrete and tying the existing structure into the new interior concrete with dowels and reinforcing. To meet the required factors of safety for dam stability to prevent the structure from moving during an earthquake or severe flood, the new filled-in, solid portion of the dam would be secured into the bedrock with high-strength ground anchors.

Rehabilitation of the dam would also include extending the dam across the forebay opening. Removal of the existing forebay retaining walls and construction of new forebay retaining walls would be a component of the power generation alternative.

Work areas for rehabilitation of the dam would be isolated through the use of cofferdams, similar to what is described for removal of the existing dam.

Fish Passage Improvement

The Gold Ray Dam fish passage improvement would meet ODFW and NMFS' juvenile and adult fish passage criteria (ODFW, 2004; NMFS, 2008). A conceptual design for a pool-and-weir fish ladder was developed based on current fish passage criteria; the fish ladder consists of individual pools that are 12 feet long, 10 feet wide, and 10 feet deep with incremental steps of 6 inches. Five larger entrance pools located at the downstream end of the ladder would be plumbed with an auxiliary water system to meet attraction flow requirements (i.e., providing sufficient flow to attract fish into the ladder) (RDG, 2010).

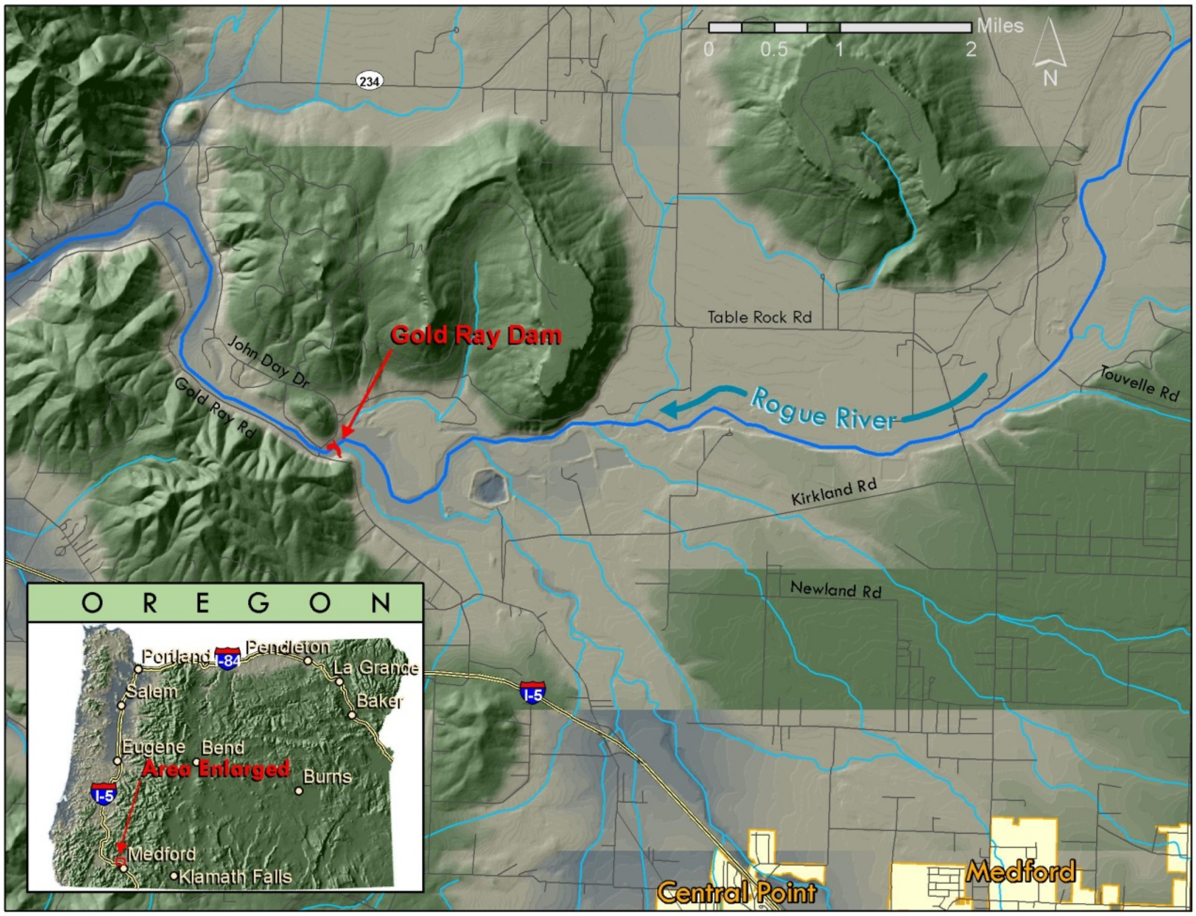


Figure 1: Project Vicinity Map for Gold Ray Dam Project on the Rogue River

Fish Screening

The existing fish screens and coarse trash racks, intended to prevent fish and debris from entering the forebay area, do not meet NMFS design criteria. These would have to be removed and replaced with screens and trash racks that meet the design criteria both at the upstream of the power canal and downstream of the tailrace.

Power Generation

The requirements for retrofitting the powerhouse facility to generate power again were evaluated in 1982 (Tudor, 1982). As part of Jackson County's dam rehabilitation study (HDR, 2010a, Appendix D) the findings of that study were reviewed and updated to assess including power generation as part of a dam rehabilitation alternative. The 1982 power generation study recommended modifying the forebay, reconstructing the powerhouse and generation facilities, modifying the tailrace, and installing a substation and power lines to link into the regional electrical power distribution system. The estimated construction cost (assuming the project would be bid in 1984) was approximately $12.9 million. The updated study concluded that, from an engineering perspective, while there have been some advances in hydropower turbine technology since the 1980s, application of newer technologies would have only a limited influence on the construction cost of upgrading the power generation system. Similarly, the energy production calculated in the 1982 study (31 million kilowatts per hour per year) is reasonable for a rehabilitation alternative. There have been improvements in turbine efficiency since the 1980s, but they are relatively minimal (likely one to two percent increased efficiencies over what was assumed in the 1982 study) (HDR, 2010a).

Table 1: Project Data for Power Generation

Component	
Head (with Downstream Streambed Modifications)	20 feet
Powerhouse Flow	3,000 cfs
Generator Capacity	4,500 kW
Annual Production	31,000 kW/hr

Restoring power generation would require Jackson County to acquire a new license from the Federal Energy Regulatory Commission (FERC). The FERC licensing procedure can be long and costly (generally taking 5 years or longer; licensing costs for other similarly sized facilities are approximately $2 million to $3 million). On the Rogue River, licensing and reestablishing hydroelectric power generation is complicated by a number of factors, including the presence of coho salmon, which is listed as an endangered species under the Endangered Species Act and Oregon water law that prohibits water withdrawals from the Rogue River for power generation (ORS 528.270).

Power generation would provide revenue for the County and this would, over time, offset some of the construction costs. Annual generation of 31 million kilowatts per hour per year would produce annual revenue of $1.8 million (in 2009 dollars). Estimated annual operation and maintenance costs would be approximately $300,000. Assuming a discount rate of 4.5 percent and a 40 year facility life, the present value of the revenue stream is approximately $25 million, which is significantly less than the construction costs estimated for rehabilitation (see Section 2.1.2.5) (HDR, 2010a).

Cost Estimate

Table 2 presents a summary of the estimated construction costs, including engineering and contingencies, for Alternative 2.

Table 2: Summary of Estimated Rehabilitation Costs – Gold Ray Dam

Component	Estimated Construction Cost ($ million)
Dam Rehabilitation	$11.9
Fish Passage Improvements	$4.0
Fish Screening	$24.8
Power Generation	$29.0
Total Estimated Cost	$69.7

1.0 INTRODUCTION

Gold Ray Dam is located on the Rogue River in Jackson County, Oregon, approximately six miles from Medford. The dam was originally constructed in 1904 as a log crib dam for the purpose of generating electricity. The existing concrete dam was constructed in 1941; it is 38 feet high and 360 feet long.

Gold Ray Dam was operated as a hydroelectric facility until 1972 when it was closed by PacifiCorp and ownership was transferred to Jackson County for the development of a park. Jackson County owns approximately 160 acres adjacent to the dam on the north side of the river. Existing facilities include the dam, the powerhouse with associated forebay and tailrace, fish ladders, and an Oregon Department of Fish and Wildlife (ODFW) fish counting station. The dam includes fish ladders, but they do not meet current design criteria established by ODFW and National Oceanic and Atmospheric Administration (NOAA). In addition, leaks in the dam and fish ladder can create false attraction flows that adversely upstream migrating. Further, the dam and associated facilities are in a deteriorated condition, posing a maintenance and liability concern for Jackson County.

In 2009 Jackson County applied for and received a NOAA Coastal and Marine Habitat Restoration Project Grant under the American Recovery and Reinvestment Act (ARRA) to evaluate and, if warranted, remove Gold Ray Dam for the purpose of improving fish habitat and passage. NOAA, as the administrator of the grant, must comply with the National Environmental Policy Act to assess the potential impacts of the proposed action on the natural and human environment (see following section). Jackson County, as the dam's owner, must assess whether dam removal is the best option for addressing concerns with the dam.

The design-build team of Slayden Construction Group, Inc. (SCG), River Design Group, Inc. (RDG) and HDR, Inc. was retained by Jackson County Roads and Parks to evaluate alternatives for Gold Ray Dam. The alternatives include: 1) dam removal, 2) rehabilitation of the existing dam to meet current criteria for structural stability and fish passage and 3) no action.

1.1 HISTORICAL CONTEXT

Gold Ray Dam was originally constructed in 1904 and consisted of a log crib structure with fish ladders on each side of the dam. In 1941 a concrete structure was completed just downstream of the log crib dam as shown in Figure 2. A fish ladder was blasted into the bedrock and built with concrete; the ladder remains in place in the current configuration as shown in Figure 3. The log crib dam the concrete structure replaced and which acted as a cofferdam during construction, was burned after dam completion.

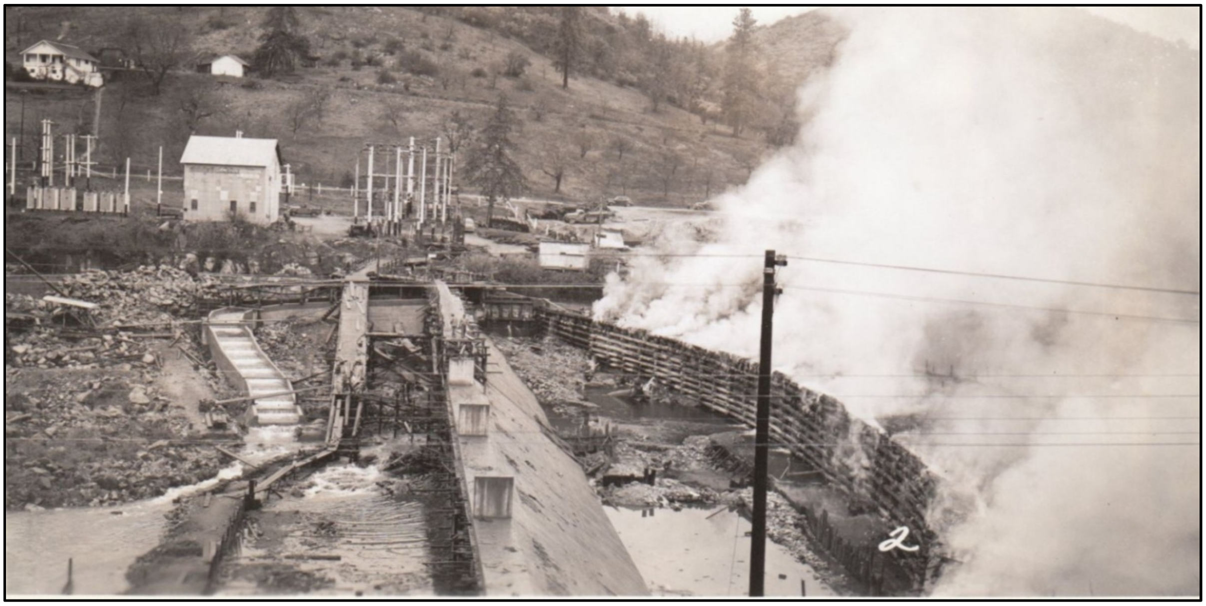


Figure 2: 1941 Photo of the Concrete Dam Being Built Along with Fish Ladder
(photo courtesy of PacifCorps historical records)

Figure 3: Existing View of Gold Ray Dam and Fish Ladder with Total Flow of 1,310 cfs

1.2 HYDROLOGY

The site has an operational river gage just downstream from Gold Ray Dam that has been operational since 1905. The Rogue River at Raygold gage (USGS 14359000) is located at 42º 26' 15" latitude and 122º 59' 10" longitude (NAD 27) and has a drainage area of 2,053 square miles at River Mile 125.8. The flow has been regulated at the gage since February 1977 due to construction of the U.S. Army Corps of Engineer's Lost Creek Dam on the Rogue River upstream of the Gold Ray Dam project area. To obtain regulated flow return intervals, a Log-Pearson Type III methodology, as outlined in Bulletin 17B

“Guidelines for Determining Flood Flow Frequency” was used on the yearly peak flows. The predicted peak discharges for the gage are summarized in Table 3.

Table 3: Predicted Stream Discharge for Rogue River at Gold Ray Dam Based on Regional Regression Equations from OWRD

Frequency	Flow (cfs)	Comments
2-yr	26,000	
5-yr	37,600	
10-yr	44,500	
25-yr	65,000	Restoration design stability flow
50-yr	74,600	
100-yr	98,000	Floodplain management flow
Est. Bankfull Discharge	16,000	~1% duration flow
Avg Daily Flow	2,850	Average daily flow

The Rogue River is a low gradient gravel-bed river that has a local reach slope of approximately 0.22 percent in the project area. General hydrologic patterns for the Rogue River are driven by rainfall and groundwater inflow. Peak flows normally occur November through May in response to rainfall, snow melt, and runoff as soils are often fully saturated through the rainy season. The Rogue River is regulated by Lost Creek Dam to reduce peak flows during high flow events. Figure 4 illustrates the daily average flows resulting from the last 30 years of regulation.

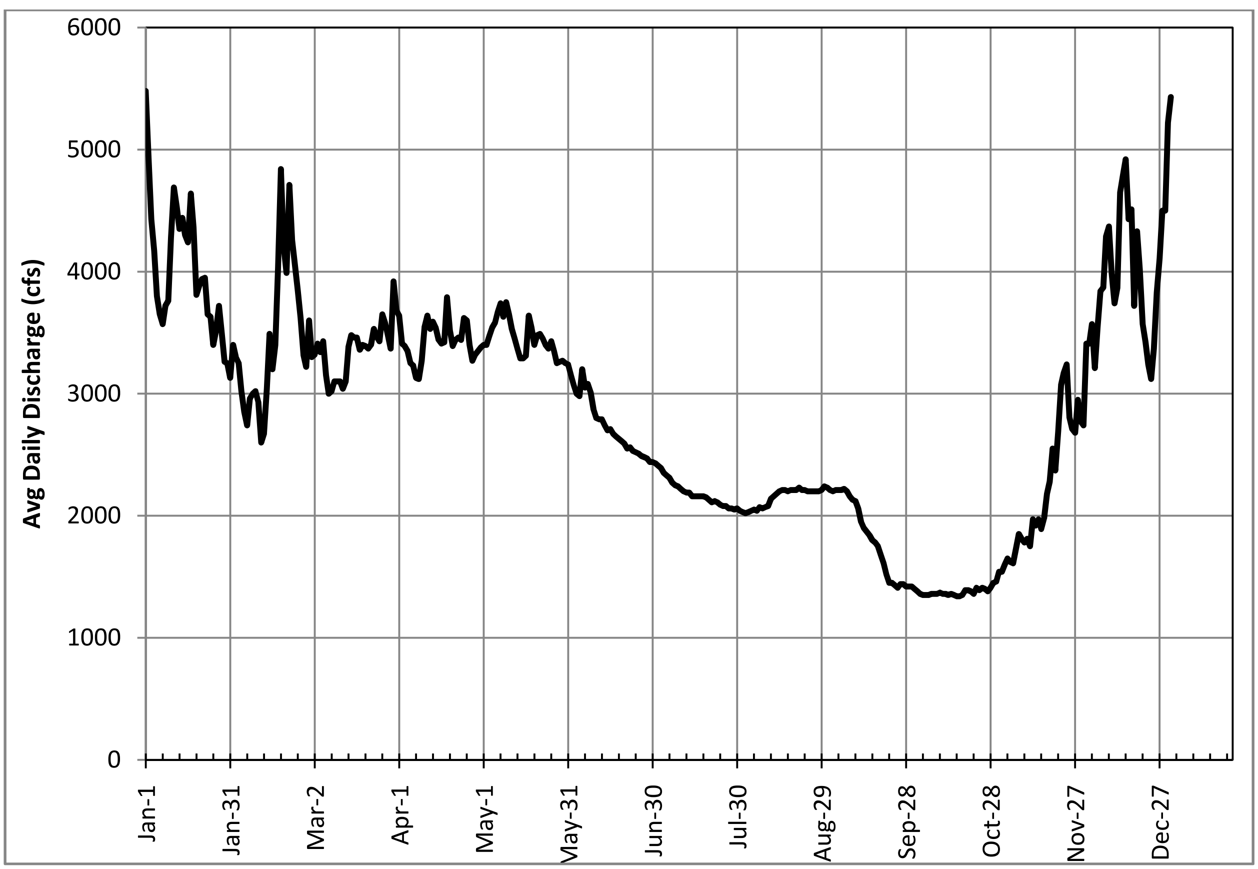


Figure 4: 30-year Daily Average Flows at Raygold Gage Reflecting River Regulation at Lost Creek Dam

2.0 GOLD RAY DAM REHABILITATION

2.1 SCOPE

Jackson County and NOAA are examining the feasibility and costs associated with a rehabilitation alternative. Rehabilitation would involve retrofitting and/or reconstructing all or portions of the dam to meet current design standards (i.e., dam safety, fish passage, fish screening) and potentially generate power. This technical memorandum addresses developing a reconnaissance level plan and estimate of probable construction cost for bringing Gold Ray dam up to current dam safety standards. Two alternatives will be examined; rehabilitating the dam in conjunction with reestablishing hydroelectric production, and rehabilitation only to meet current dam safety standards. This evaluation is based upon information obtained in a data search and during a site visit. The scope of the site visit was limited to visible elements only and excluded covered, buried, or hidden conditions. The visit did not include any special investigations, equipment testing, field or laboratory testing, geotechnical investigations, or materials testing.

2.2 DATA SEARCH

HDR reviewed historical records obtained from Jackson County and contacted George Kramer of Kramer and Company to discuss his review of historical Pacific Corps files and photographs. Very little technical data or plans are available for the project. Project plans dating from 1941 include only a site plan and five plan sheets for the main dam. A limited number of 1906 original construction as-built drawings were available for the headworks, fish ladders and powerhouse. No dam safety reports of any age were available for review. HDR also reviewed the 1982 Tudor Engineering Company report *Feasibility Report on the Potential Hydroelectric Development Gold Ray Power Project* as well as historical correspondence dating from 1941 regarding the original slab and buttress dam construction. A significant number of historical photographs chronicling the construction of the 1941 structure were also reviewed.

2.3 SITE VISIT

The dam site was visited by Samuel M. Planck, PE, and Steve Mason, PE, of HDR Engineering, Inc. on December 8th, 2009. Conditions were clear with temperatures ranging from 20 to 30 degrees F. Visual observations of the dam and appurtenant structures were made from the headworks bridge, training and forebay walls, tailrace walls, and powerhouse walkways. No observations were possible of the main dam slabs or buttresses due to the continuous overflow nature of the dam and a lack of inspection galleries, hatches, or access under the slabs. Correspondingly, there is no historical record of inspection or observation of the interior portions of the dam.

Figure 5: Gold Ray Dam from South Abutment, December 8, 2009

2.3.1 Observations

The Gold Ray Dam is a 1941 slab and buttress design (often referred to as an "Ambursen" dam) consisting of an inclined upstream slab spanning simply supported between downstream buttresses. The dam is 480 feet long with an overflow section of 370 feet. The elevation of the crest of the dam is 1146.0 feet. The maximum height of the dam is 40 feet. At Gold Ray Dam the upstream slab varies in thickness from 14 inches at the crest to 25 inches at the bottom of the tallest sections. The bottom of the slab is noted on the drawings to be keyed into "firm rock" a minimum of 2 feet. The buttresses, which are spaced at 15 feet on center, vary in thickness from 21 at the deepest sections to 15 inches near the top. The left and right abutments consist of a thickened buttresses acting as a seat-type (bridge) abutment for the upstream slab.

In general, the dam and appurtenant structures appear to be in remarkably good condition given their age, environment, and apparent minimal maintenance and upkeep. That said, the majority of dam and water control structures on the site are in a severe state of deterioration. With the exception of the fish ladder, which appears to still be functioning to some extent, the entire facility is functionally obsolete and appears unsafe from both a dam safety and public safety perspective. There is evidence of significant concrete deterioration, cracking and displacement throughout the site. Of primary concern, however, is the fact that no routine inspections or observations can be made of the interior portions of the slabs and buttresses.

2.4 EVALUATION OF EXISTING DAM

The long-term performance and safety concerns associated with slab and buttress dams are well documented in available literature (ref 8, 9, 10, 11). The design, while considered very efficient by 1930's and 1940's standards when material was expensive and labor was cheap, would not be permitted within the United States today. The design is particularly susceptible to deterioration of the upstream slabs due its relative thinness and minimal reinforcing. Furthermore, according to FEMA, "Buttress dams also are particularly vulnerable to cross-valley shear motions that can result in tipping of the buttresses and loss of support for the reinforced concrete slab." As such, the primary deficiencies with the existing structure are associated with hydraulic and seismic sliding and overturning stability and with seismic structural capacity and ductility. Preliminary calculations indicate that the structure does not meet the required minimum sliding or overturning factors of safety for even the historical flood of record let alone the current Probable Maximum Flood (PMF). Additionally, the simply-supported (unreinforced) seat detail between the existing slabs and buttress corbels offers no resistance to seismically induced displacements. The corbel itself, which is typically the "weak link" in this type of design, is unreinforced except for the upstream tension face.

2.5 REHABILITATION

2.5.1 Required Investigations for Rehabilitation Evaluation

Prior to a preliminary design of the rehabilitation several investigations will be required to obtain the necessary analysis and design information. These include a comprehensive structural inspection, a geotechnical and geologic investigation, a comprehensive survey, and selected material testing.

2.5.1.1 Structural Inspection

Prior to the design of the remediation measures for the existing dam and appurtenant structures a comprehensive structural inspection will be required. The inspection will focus primarily on the existing slab and buttress dam, and specifically on the condition of the interior portions of the dam. Temporary access will need to be obtained into the interior of the dam by diverting the flow over the dam at several locations and providing safe boat access to the then exposed interior. The condition of the bottom of the slab, the buttresses, and the struts can then be ascertained.

2.5.1.2 Geotechnical

Prior to the design of the remediation measures for the existing dam and appurtenant structures a comprehensive geotechnical investigation will be required. A geotechnical field investigation will be performed to characterize subsurface conditions at the site. Samples of subsurface soil and rock will be collected in test pits and borings and laboratory tests will be performed to determine physical engineering properties. The field investigation will consist of approximately 10 borings; five along the centerline of the dam, one each at the left and right abutments, and three conducted along the training walls and forebay walls. Boring depths will vary from 75 to 150 feet. Test pits could also be required at the left and right abutments. A geotechnical report would then be developed including the investigation results, analysis and design parameters. The report will also include an updated seismicity.

2.5.1.3 Survey

Prior to the design of the remediation measures for the existing dam and appurtenant structures a comprehensive survey will be required. Typically aerial photogrammetry would be used to develop the background mapping for the topographic survey and a primary horizontal and vertical survey control network which covers the entire project area would be established. Project survey control would be established utilizing either GPS or conventional survey methods. Site aerial control panels will be set at locations and frequency adequate to meet National Mapping Accuracy Standards for 1-inch = 200 feet scale mapping with 2-foot contour intervals over the entire proposed project area. Aerial based topography mapping would be prepared at the stated mapping scale and contour interval showing all visible surface features, contours and spot elevations within the mapping limits.

Bathymetric ground surveys would be utilized to supplement topography mapping where obstructed by water at the reservoir, forebay, or downstream river. A survey grade GPS receiver and a survey grade depth sounder would be utilized. Bathymetric surveys would be limited to areas suitable for GPS and safely accessible with a shallow draft boat.

2.5.1.4 Material Testing

Prior to the design of the remediation measures for the existing dam and appurtenant structures, material testing of numerous existing structural elements will be required. Concrete core and reinforcing samples would be collected at the following locations: slabs and buttresses, forebay walls, powerhouse walls, fishway walls, and existing bridges.

2.5.2 Rehabilitation Approaches

Rehabilitation of the dam to meet current dam safety standards will follow a similar approach for the two cases of with or without hydroelectric generation being restored to the project. In both cases the primary approach would be to turn the slab and buttress design into a gravity dam. This would be accomplished by essentially filling the voids beneath the slabs with concrete and tying the existing structure into the new interior concrete with dowels and reinforcing. Even with the new added mass however the new cross section will not meet the required minimum overturning or sliding factors of safety. This is partially due to the orientation of the cross section with the inclined face on the upstream side and the vertical face on the downstream (exactly the opposite of what is desired on a typical gravity dam). Additionally, the existing slabs are only keyed into the foundation rock a reported minimum of 2 feet and the buttresses only 12 inches. To increase the sliding and overturning resistance the new solid cross section would be tied-down into the bedrock with high-strength ground anchors. Preliminary calculations using appropriate but assumed geotechnical values indicate that tie-downs at the tallest dam section will be required at a spacing of between 3 and 5 feet on center. For the shorter northern end of the dam, tie-downs will be required at a spacing of between 5 and 10 feet on center depending on the specific section height. This results in a total of 55 tie-down anchors for the dam. As no data is available for rock strengths, a typical length of 100 feet has been assumed for the tie-downs. Figure 6 illustrates the new rehabilitated cross section.

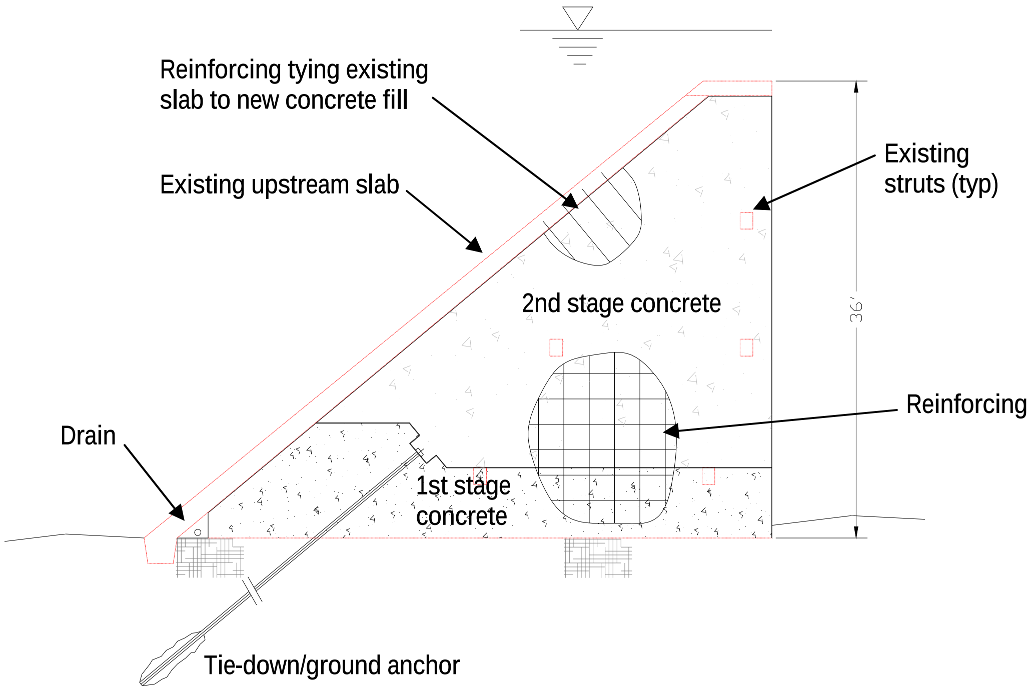


Figure 6: Cross section of rehabilitated dam showing concrete fill and tie-down

To begin the construction of the tie-down system a continuous drain would be installed at the furthest bottom upstream corner of the section to relieve any leakage and pressure between the existing slab and the new concrete. These would be free draining to downstream of the dam.

Next the bottom (first stage) concrete would be poured in place. This concrete would serve as a working platform for the tie-down installation and include an integral grade beam. A grade beam is essentially an embedded reinforced concrete beam the function of which is to transfer the tie-down forces (point loads) to the wider section. The first stage concrete would be in the range of 5 to 8 feet thick and heavily reinforced to withstand the tie-down forces and in order to be anchored to the second stage concrete fill above. The tie-down holes would then be drilled into the bedrock below the dam using pre-formed holes in the first stage concrete as guides. Corrosion protected tie-downs would then be installed and the anchors grouted into the ground. After the grout achieves the required strength the anchors would be tensioned. Last the holes would be grouted full for additional corrosion protection. The next step would involve installing steel reinforcing which will tie the first stage concrete to the second stage concrete and tie the second stage concrete to the existing structure. After the reinforcing is constructed the second stage concrete would be poured in place.

For the case with restored hydroelectric generation, this approach would be applied to nearly the entire existing dam, with the possible exception of a portion on the north (right) abutment where the existing structure would tie into the new forebay walls.

For the case without hydroelectric generation being restored, a new dam section would be constructed from the north end of the existing dam into the right bank of the river, effectively eliminating all of the downstream forebay and powerhouse walls from having to restrain water. Figure 7 and Figure 8 illustrate the new dam extension between the existing dam and the right bank of the river.

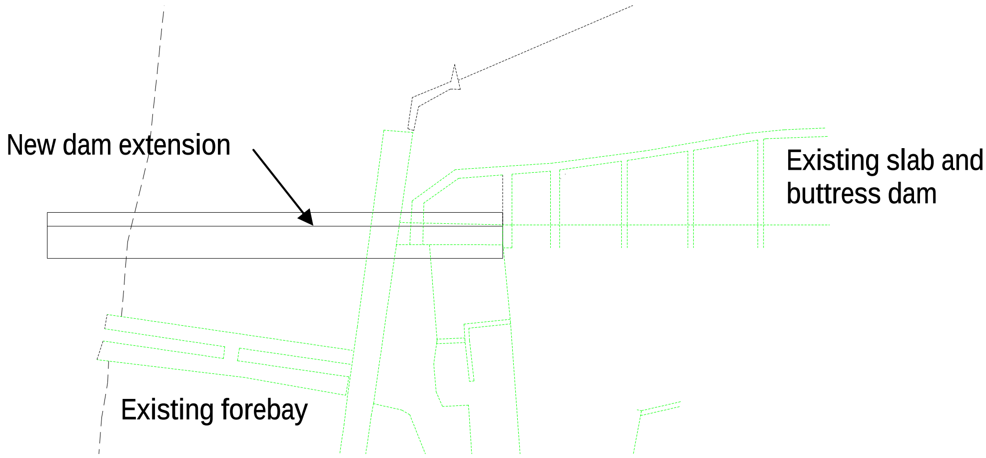


Figure 7: New extension from existing dam to right bank (precluding hydroelectric generation)

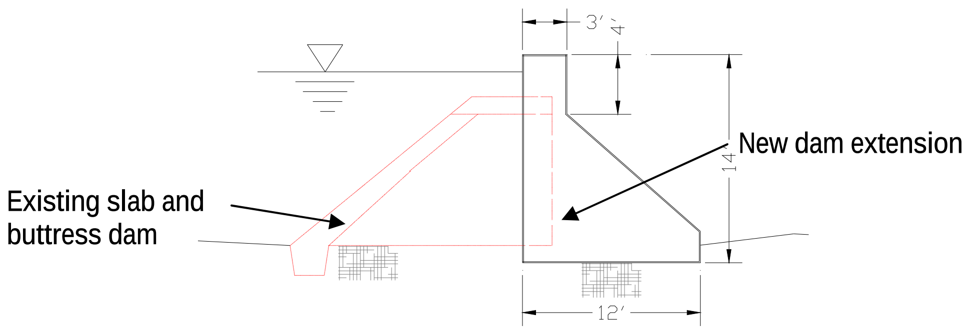


Figure 8: New extension of dam to right bank (precluding hydroelectric generation)

The new extension from the right abutment to the north river bank would consist of a solid gravity dam section keyed into the foundation rock and extending 10 to 20 feet into the shoreline. With such a cut-off installed, all of the water retaining structures downstream of the dam; the head gates, forebay walls, powerhouse walls and bulkheads would no longer be required to retain water and would not require retrofit from a dam safety perspective. Many of these structures would likely still require rehabilitation for public safety issues, particularly related to seismic loads, but strengthening for dam safety reasons would not be necessary.

2.6 ESTIMATE OF PROBABLE CONSTRUCTION COSTS

The following estimate of probable construction costs is based on very limited information. As such, contingencies for items vary from 15% to 30%. The estimate does not include any allowances for environmental studies, reports, permitting, mitigation, new fish passage, or restored hydropower generation. All estimates are in 2010 dollars.

Table 4: Estimate of Probable Construction Costs

Account Number	Item	Quantity	Unit	Unit Price $	Amount $	Contingency %*	Contingency $	Subtotal With Contingency
1	**Site Investigations**							
	Geotechnical	1.0	LS	$75,000	$75,000	20%	$15,000	$90,000
	Surveying	1.0	LS	$50,000	$50,000	20%	$10,000	$60,000
	Material Testing	1.0	LS	$25,000	$25,000	20%	$5,000	$30,000
	Structural Inspection	1.0	LS	$25,000	$25,000	20%	$5,000	$30,000
2	**Engineering Design**							
	Engineering	1	LS	$400,000	$400,000	20%	$80,000	$480,000
3	**Site Preparation**							
	Mobilization/Demobilization	1	LS	$300,000	$300,000	30%	$90,000	$390,000
	Dewatering	1	LS	$500,000	$500,000	30%	$150,000	$650,000
	Concrete Demolition and Disposal	1	LS	$60,000	$60,000	20%	$12,000	$72,000
	Clearing and Grubbing	1	LS	$10,000	$10,000	20%	$2,000	$12,000
	Misc. Site Improv. (Elec. Erosion, Fence, etc)	1	LS	$50,000	$50,000	20%	$10,000	$60,000
4	**Existing Dam Retrofit**							
	Mass Concrete Fill	5,800	CY	$600	$3,480,000	20%	$696,000	$4,176,000
	Reinforcing	754,000	LBS	$3	$2,262,000	20%	$452,400	$2,714,400
	Grouting	1	LS	$65,000	$65,000	120%	$78,000	$143,000
	Tie-downs	55	EA	$35,000	$1,925,000	20%	$385,000	$2,310,000
	Drainage	1	LS	$40,000	$40,000	20%	$8,000	$48,000
5	**New Dam Extension**							
	Concrete for Extension	240	CY	$400	$96,000	20%	$19,200	$115,200
	Foundation Excavation	200	CY	$300	$60,000	20%	$12,000	$72,000
	Rebar for Extension	31,200	LBS	$3	$93,600	20%	$18,720	$112,320
6	**Construction Management**							
	5% of Construction Costs	1	LS	$500,000	$500,000	15%	$75,000	$575,000

Estimated Project Total (w/o Cont.)	**$10,016,600**
Estimated Project Total with Contigency	**$12,139,920**

The above estimate includes the new dam extension associated with non-hydroelectric generation and illustrated in Figure 7. The estimate of probably construction cost without this element (hydroelectric generation restored) with contingencies is $11,840,000.

3.0 GOLD RAY FISH LADDER (PRELIMINARY DESIGN)

3.1 INTRODUCTION

3.1.1 Project Scope

This chapter presents a preliminary design for a fishway that would meet the current ODFW and National Marine Fisheries Service (NMFS) design criteria. The fishway consists of a pool and weir ladder constructed from reinforced concrete. The fishway has 38 pools with 0.5 ft of drop between each pool. Entrance pools are larger than fishway pools as a result of a required auxiliary water system to meet minimum attraction flow requirements. A controllable weir at the upstream end of the fish ladder provides flexibility for future water management purposes. The fish ladder alignment occupies a similar footprint as the existing fish ladders for ease of construction and alignment. The preliminary fish ladder design meets state and federal fish passage criteria.

The following tasks are addressed in this chapter:

- Describe existing fish passage facilities.
- Identify fish passage flow criteria based on existing hydrology and flow characteristics at the site.
- Develop a fish ladder design concept that meets current state and federal fish passage criteria.

A project vicinity map of the Gold Ray Dam site is included on Figure 1.

3.2 METHODS

The following section outlines RDG's methods for evaluating the existing conditions and preparing the conceptual fish ladder design. Field surveys and remote sensing were used to evaluate existing conditions at the project site.

3.2.1 Site Investigation and Survey

Surveys were completed using various methods. First, Watershed Sciences, Inc. collected Light Detection and Ranging (LiDAR) data for the project area in May 2009. Next, RDG completed detailed field data collection in September 2009 to characterize the existing site conditions at the dam, as well as conditions in the upstream reservoir area and in the river downstream from the dam. Data collection included topographic survey of the existing fish ladder, concrete dam, and surrounding structures. Water surface elevations were collected along with velocity profiles at the fish ladder for calibration of the hydraulic model. RDG data collection efforts utilized a total station (Topcon 211d) with data collector and a survey-grade GPS (Trimble R8) system to georeference the site. RDG also established horizontal and vertical control benchmarks for use throughout the project. Conditions in the upstream portion of the existing fish ladder are show on Figure 9.

Figure 9: Surveying the Existing Fish Ladder with a Survey-grade GPS and Total Station

In cooperation with Watershed Sciences, Max Depth Aquatics, Inc. performed a hydroacoustic bathymetric survey of the Rogue River in and around the project site. Finally, Watershed Sciences integrated both LiDAR and bathymetric surveys into seamless models of terrestrial bare earth and submerged bathymetry. The vertical accuracies for the LiDAR data and bathymetric data are 3 cm and 5 cm, respectively (Watershed Sciences, 2009). The resulting elevation model of the project site allows hydraulic modeling of existing conditions and likely hydrological outcomes of the alternative scenarios.

Hydraulic modeling data were evaluated in HEC-RAS 4.0 (HEC, 2008) and displayed using AutoCAD Civil 3D. ArcGIS programs were used to develop field base maps and visualization figures. Programs included ArcGIS Version 9.2 (ESRI, 2005a) and ArcGIS extensions, Spatial Analyst (ESRI, 2005b) and 3D Analyst (ESRI, 2005c).

3.2.2 Fish Passage Flows

In addition to general hydrologic conditions and peak flow events, fish passage flows of 5% and 95% exceedance are of specific interest. An analysis of average daily flows was developed from OWRD data for the last 30 years**.**

Low fish passage flow for anadromous fish is specified in the NMFS Fish Passage Criteria (2008) and is defined as "the mean daily average streamflow that is exceeded 95% of the time when migrating fish are normally present at the site", taken from the previous 25 years of record. Because adult fish are present in the Rogue River throughout the year, fish passage flows were determined using the entire calendar-year record of streamflow. The regulated flow regime on the Rogue River has been consistent since the 1977 completion of Lost Creek Dam, and low fish passage flow was taken from the previous 30 years of record at the Raygold Gage. NMFS criteria specify that high fish passage flow is "the mean daily average streamflow exceeded 5% of the time during periods when migrating fish are normally present at the site." Table 5 summarizes the fish passage flows based on these requirements.

Table 5: Average Daily Discharge Ranges for the Rogue River at Gold Ray Dam Based on Most Recent 30 Years of OWRD data

June 15 – Oct 15	Average (cfs)	Min (cfs)	Max (cfs)
High Flow	2,710	1,410	4,700
Low Flow	1,350	900	2,100
Average Flow	2,030	1,250	3,150
5% Exceedance	4,368	High Fish Passage Flow	
95% Exceedance	1,380	Low Fish Passage Flow	

3.2.3 Fisheries and Habitat Flow Analyses

A fish presence and life stage chart was developed by ODFW to determine fish usage of the Rogue River throughout the year. Salmonid presence is referenced to the time of year (Table 6) to help identify species and usage periods.

Table 6: Fish Periodicity Chart Based on ODFW Historical Information and Field Observations

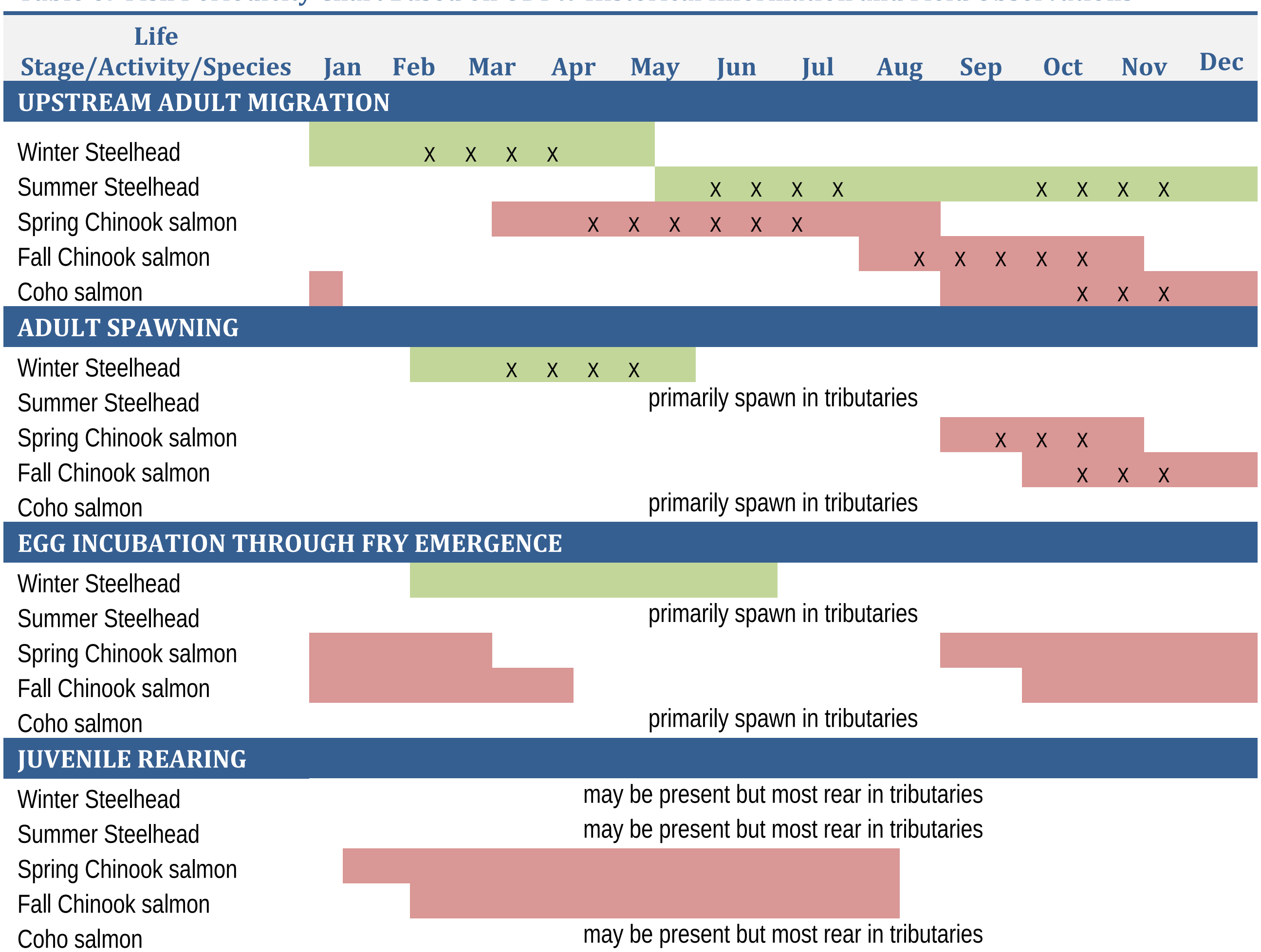


Table 6: Fish Periodicity Chart Based on ODFW Historical Information and Field Observations

Life Stage/Activity/Species	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
DOWNSTREAM JUVENILE MIGRATION												
Winter Steelhead				X X	X X	X						
Summer Steelhead				X X	X X	X						
Spring Chinook salmon						X X	X X	X X				
Fall Chinook salmon						X X	X X	X X				
Coho salmon				X X	X X	X X						

X - Denotes peak timing

3.3 EXISTING CONDITIONS

3.3.1 Hydraulics

The existing dam is a run-of-the-river type structure that does not have mechanisms to control flows. The existing concrete dam acts as a weir with flow going over the dam similar to a broad crested weir. In addition, water flows down the fish ladder and the powerhouse raceway. Figure 10 shows the existing upstream entrance to the fish ladder in relationship to the dam structure.

Figure 10: Looking Upstream at Fish Ladder Flow Control Structure with 3,800 cfs Total Flow in the River

The hydraulic characteristics of the dam and fish ladder were developed to understand existing conditions and calibrate the existing weir coefficients. Table 7 summarizes the stage-discharge relationship for the dam and fish ladder. A broad crested weir coefficient of 3.05 was determined for the concrete dam for a discharge of 1,310 cfs based on a known water surface and discharge. In addition, water surface elevations and weir conditions were measured for the fish ladder to determine an estimated flow rate and to calibrate the integrated system as shown in Table 7.

Table 7: Existing Stage Discharge for Gold Ray Dam based on calibrated model.

Water Surface Elev. (ft)	Head on Dam (ft)	Dam Discharge (cfs)	Avg Flow Velocity Dam (ft/sec)	Head on Fish Ladder (ft)	Fish Ladder Weir Flow (cfs)	Total Discharge (cfs)
1150.80	0.90	936	2.89	1.20	37	973
1151.00	1.10	1,265	3.20	1.40	46	1,311
1151.20	1.30	1,625	3.48	1.60	56	1,682
1151.40	1.50	2,014	3.74	1.80	67	2,082
1151.60	1.70	2,430	3.98	2.00	79*	2,509
1151.80	1.90	2,872	4.20	2.00	79*	2,951
1152.00	2.10	3,337	4.42	2.00	79*	3,416
1152.20	2.30	3,825	4.63	2.00	79*	3,904
1152.40	2.50	4,334	4.82	2.00	79*	4,413
1152.60	2.70	4,865	5.01	2.00	79*	4,943
1152.80	2.90	5,415	5.19	2.00	79*	5,494
1153.00	3.10	5,985	5.37	2.00	79*	6,064
1153.20	3.30	6,573	5.54	2.00	79*	6,652
1153.40	3.50	7,180	5.71	2.00	79*	7,259

** Water overflows sides of fish ladder walls above 2 feet, however, ODFW adjusts the upstream flashboards to control the amount of flow down the fish ladder at high flows so an estimate of 79 cfs is used for high flow conditions.*

ODFW uses an energy dissipation factor (EDF) as a surrogate for velocity criteria in fishways in accordance with Oregon Administrative Rules (OAR) 635-412-0035. Based on the existing conditions it is evident that the current fish ladder does not meet these criteria for all flow levels. In addition, step heights between weirs exceed 1.25 ft which is outside current criterion for fish passage.

3.3.2 Fisheries

The following fisheries information was developed based on general salmonid characteristics and also from information provided by ODFW fisheries biologist Jay Doino. The following sections present the migration, spawning and rearing characteristics of the three target salmonid species; coho salmon, Chinook salmon (spring and fall), and steelhead (summer and winter).

3.3.2.1 Coho Salmon

Migration and Spawning

Coho salmon typically migrate through this section of the Rogue River from September through January. Adults migrating upstream may rest in pools or other areas with slow currents and cover features. Adult spawning occurs primarily in tributaries from October through January, with the peak occurring in December. Time required for egg incubation varies with temperature, and eggs or coho salmon fry could be within gravels anytime between October and May.

Rearing

Fry attempt to establish territories and remain in tributary streams as juveniles before smolting and migrating down the Rogue river to salt water during spring or possibly fall of the year following emergence. Peak downstream migration occurs April – July. Like other salmonids, juvenile coho salmon require cold water (less than 64 °F or 17.8 °C), high dissolved oxygen levels, and deep pools for feeding and cover from predators. Access to tributary streams to find refuge from high flows in the winter is also important. Winter parr are especially dependant on slow water habitat for survival.

3.3.2.2 Spring Chinook Salmon

Migration and Spawning

Adult spring Chinook salmon enter this reach of the Rogue River from March until August with the bulk of the run arriving from mid-April to mid- July. Before spawning, adult Chinook salmon hold in pools, preferring deep pools with cool water, abundant large wood, and undercut banks for cover. Peak spawning occurs during September and October. Chinook salmon die after spawning, providing an important marine-derived nutrient source to Rogue River. Spring Chinook fry typically emerge from gravels beginning in January through March.

Rearing

Unlike steelhead and coho salmon, juvenile Chinook salmon only spend a few weeks to months near their spawning grounds before migrating to salt water and are usually out of the freshwater system by late summer. Like other salmonids, juvenile Chinook salmon require cold water and deep pools for feeding and cover from predators. Access to tributary streams to find refuge from high flows in spring is also important. Juvenile Chinook salmon require cool water temperatures (less than 64°F), and high dissolved oxygen levels.

3.3.2.3 Fall Chinook Salmon

Migration and Spawning

Adult fall Chinook salmon enter this reach of the Rogue River from mid-July to mid-November. Spawning occurs in the fall with the peak occurring during October and November. Chinook salmon die after spawning, providing an important marine-derived nutrient source to Rogue River. Fall Chinook fry typically emerge from gravels beginning in February through April.

Rearing

Unlike steelhead and coho salmon, juvenile Chinook salmon only spend a few weeks to months near their spawning grounds before migrating to salt water and are usually out of the freshwater system by late summer. Like other salmonids, juvenile Chinook salmon require cold water and deep pools for feeding and cover from predators. Access to tributary streams to find refuge from high flows in spring is also important. Juvenile Chinook salmon require cool water temperatures (less than 64°F), and high dissolved oxygen levels.

3.3.2.4 Winter Steelhead

Migration and Spawning

Adult winter steelhead migrate into this reach of the Rogue River to spawn from February through May. Peak spawning occurs in March and April in the mainstem of the Rogue River as well as in low/moderate gradient streams (up to 8%). Eggs or fry can be present in the gravel from February to June.

Rearing

Juvenile steelhead can remain in the Rogue River for one to two years before migrating as smolts to salt water. Juvenile steelhead are likely to use both the mainstem and cool water tributaries for rearing. They can be found in riffles and pools with cover, large wood (Figure 10), and cool water temperatures (less than 64°F or 17.8°C), and high dissolved oxygen levels. Winter steelhead may make seasonal migrations into and out of tributaries and the mainstem Rogue River throughout their freshwater residence time.

3.3.2.5 Summer Steelhead

Migration and Spawning

Adult summer steelhead migrate into this reach of the Rogue River from May through December. Two peaks in migration occur, the first being in June and July, and the second occurring in October and November after fall rains have raised the river level and cooled water temperatures to a more optimal range. Spawning occurs primarily in tributaries to the Rogue from December through March, with the peak of the spawn occurring in January and February. Summer steelhead typically prefer smaller, sometimes ephemeral tributaries, than winter steelhead. Eggs or salmon fry can be present in the gravel from December through June.

Rearing

Juvenile steelhead can remain in the Rogue River for one to two years before migrating as smolts to salt water. Juvenile steelhead are likely to use both the mainstem and cool water tributaries for rearing. They can be found in riffles and pools with cover, large wood (Figure 10), and cool water temperatures (less than 64°F or 17.8°C), and high dissolved oxygen levels. Summer steelhead often make seasonal migrations into and out of tributaries and the mainstem Rogue River throughout their freshwater residence time.

3.3.3 Summary

The existing dam is essentially an in-line weir with no flow control capabilities. The existing fish ladder does not meet current criteria for fish passage at all water levels due to excessive jump heights and energy dissipation criteria.

3.4 FISH PASSAGE DESIGN

This section provides a conceptual design for a fishway that could be built in the current location of the current ladder and meet state and federal fish passage criteria.

3.4.1 Historical Fish Passage

Fish passage has been provided at the Gold Ray Dam since the original log crib dam was installed and further upgraded in 1941 when the concrete fishway was built (Figure 11). This current fishway does not meet existing fish passage criteria and cannot be retrofit to provide passage. Therefore, to provide passage meeting current criteria, a new ladder would be required.

Figure 11: A View of Dam and Fish Ladder Construction in 1940
(photo courtesy of PacifiCorps historical records)

3.4.2 Fish Ladder Design

The Gold Ray Dam fish passage solution must meet the juvenile and adult fish passage criteria presented by Oregon Department of Fish and Wildlife (ODFW 2004) and NMFS (2008). Table 8 and Table 9 summarize the ODFW and NMFS criteria, respectively. This effort utilizes both the ODFW and NMFS design criteria in addition to guidance from other references.

Table 8: ODFW Fish Passage Criteria (2004) for Target Species in the Rogue River

Parameter	Steelhead	Cutthroat	Chinook	Lamprey	Juveniles	Limiting Value
Velocity (ft/sec)	6	4	6	2	2	2 for juveniles 4 for adults
Depth (inches)	12	8	12	—	8	12 during migration 8 during nonmigration
Jump Height (inches)	12	6	12	—	8	6
Jump Pool Depth (inches)	24	24	24		24	24

Table 9: National Marine Fisheries Service Anadromous Salmonid Passage Criteria (2008) for Fish Ladders and Fishway Transitions

Parameter	Fish Ladder	Fishway Transitions
Velocity (ft/sec)	--	1.5 - 4
Hydraulic Drop (inches)	12	4 – 12
Min Pool Dimensions (ft)	Length – 8 Width – 6 Depth – 5	—

A conceptual design for a pool and weir fish ladder was developed based on current fish passage criteria (see drawings in Appendix A). The fish ladder consists of individual pools that are 12 ft long, 10 ft wide, and 10 ft deep with incremental steps of 6 inches. Five larger entrance pools exist at the downstream end of the fishway and are plumbed with an auxiliary water system (AWS) to meet fishway attraction flow requirements.

3.4.3 NMFS Criteria and Response

The following section contains criteria from NMFS 2008 design guide are summarized and a response is provided that outlines how that criterion was met or addressed in the proposed design.

3.4.3.1 Criterion 4.5.3.1 Hydraulic Drop (Drop ≤ 1 ft)

The ladder was designed with 0.5 ft drops between pools for adult and juvenile passage. This meets required hydraulic drop requirements as put forth by NMFS.

3.4.3.2 Criterion 4.5.3.2 Flow Depth (Flow Depth ≥ 1ft)

Flow depths in the ladder vary from 1.0 ft at low fish passage flow to 2.6 ft at high fish passage flow. These depths exceed the minimum flow depth requirement of 1.0 ft.

3.4.3.3 Criterion 4.5.3.3 Pool Dimensions (Min 8 ft Long, 6 ft Wide, 5 ft Deep)

The pool dimensions for the proposed fishway exceed the minimum pool dimension requirements. Fishway pools are sized to keep the EDF in the range of 2 (ft-lbs/s)/ft^3 for the operational flow range. As a result, the pools are 12 ft long, 10 ft wide and 10 ft deep.

3.4.3.4 Criterion 4.5.3.4 Turning Pools (Min 2 Times Length of Standard Pool)

The proposed ladder contains two turning pools at Pool 5 and Pool 19. Standard pool length is 12 ft, and turning pool centerline length is set to 24 ft to meet the turning pool requirement.

3.4.3.5 Criterion 4.5.3.5 Pool Volume (EDF < 4 (ft-lbs/s)/ft^3)

Fishway pool volumes were sized so that the EDF is less than 4.0 (ft-lbs/s)/ft^3 for all fish passage flows. At Q_{HFP}, the EDF is 2.4 (ft-lbs/s)/ft^3 or less.

Additionally, entrance pools were also sized to meet the EDF requirements. At Q_{HFP}, the EDF in the entrance pools is 3.4 (ft-lbs/s)/ft^3.

3.4.3.6 Criterion 4.5.3.6 Freeboard (Min 3 ft Above Q_{HFP})

The fish ladder walls are extended at least 3 ft above the high fish passage flow to ensure fish are contained within the fish ladder. Total wall height above the fishway weirs is 5.5 ft.

3.4.3.7 Criterion 4.5.3.7 Orifice Dimensions (Min 15 Inches High by 12 Inches Wide, Chamfered)

Orifices exist on the weir walls of the entrance pools, Pools 1 to 5. The orifice is 12 inches wide and 2.3 ft high. The orifice section is chamfered, with a ¾ inch chamfer on all edges on the upstream side and a 1.5 inch chamfer on all edges on the downstream side.

3.4.3.8 Criterion 4.5.3.8 Lighting (Ambient Lighting Preferred)

The proposed fishway is open to ambient lighting from daylight throughout the fishway.

3.4.3.9 Criterion 4.5.3.9 Change in Flow Direction (Miters or Radius Required Beyond 60° Directional Change)

Corner chamfers are provided at outside pool corners for fish ladder directional changes exceeding 60°. There are two locations where directional changes exceed 60°, located at Pool 5 and Pool 19. At these locations, a 2 ft radius corner fillet is proposed for the outside pool corners.

3.4.4 Fishway Entrance

3.4.4.1 Criterion 4.2.2.1 Configuration and Operation

The fishway downstream entrance consists of a series of five pools, with concrete weirs and concrete walls. They are rectangular in cross-section, 12 ft wide, 24 ft long, and 10 ft deep. The entrance operates in a fully open position throughout the range of operational flows. At higher flows, the downstream weirs of the entrance become submerged.

3.4.4.2 Criterion 4.2.2.2 Location

The fishway entrance is placed in the location of the existing fishway entrance. This area consists of a tranquil zone adjacent to overflow from the dam. The entrance is rectangular in section and is 12 ft wide.

The approach channel to the fishway entrance is the main stem of the Rogue River. Because the entrance is located within the main channel, it is designed to be flexible in operation because lower bays become inundated at higher river stages. The base flow water surface elevation on the Rogue River at the fishway is approximately 1131.2 ft. At low flow, the most downstream weir is submerged, and higher elevation weirs progressively submerge with increased river stage.

3.4.4.3 Criterion 4.2.2.3 Attraction Flow (5% to 10% of High Fish Passage Design Flow)

The fishway uses an auxiliary water system to meet attraction flow requirements. At low fish passage flow, attraction flow is 5% of high fish passage flow, and at high fish passage flow, attraction flow is 6.4% of high fish passage flow.

3.4.4.4 Criterion 4.2.2.4 Hydraulic Drop (1.0 to 1.5 ft or 0.5 ft to 2.0 ft)

Because juvenile fish are present in the Rogue River throughout the year, the entrance is designed for juvenile passage and to operate at 0.5 ft drop.

3.4.4.5 Criterion 4.2.2.5 Dimensions (Min 4 ft wide, 6 ft deep)

The fishway entrance is 18 ft wide and 10 ft deep, which exceeds the minimum required entrance dimensions.

3.4.4.6 Criterion 4.2.2.6 Additional Entrances (provide entrance for each zone of fish accumulation)

The proposed fishway has one entrance location. Weirs are provided at a range of elevations that extend below the tailwater elevation corresponding to summer baseflow. Therefore, the weirs become submerged by increasing tailwater elevations and continue to function over a range of tailwater elevations.

3.4.4.7 Criterion 4.2.2.7 Types of Entrances (misc; submerged weir preferred)

The proposed fishway entrance consists of concrete weirs and will be submerged at various discharge levels. This allows for a flexible range of operation with varying tailwater stage.

3.4.4.8 Criterion 4.2.2.8 Flow Conditions

The tailwater elevation exceeds the entrance weir elevation at all flows, creating streaming flow at the entrance.

3.4.4.9 Criterion 4.2.2.9 Orientation

The entrance is located along a seam between high velocities leaving the dam and adjacent tranquil water and is oriented approximately perpendicular to river flow.

3.4.4.10 Criterion 4.2.2.10 Staff Gages

Staff gages are proposed for the entrance section, exit section, and midway along the fishway, at locations clearly visible from the bank.

3.4.4.11 Criterion 4.2.2.11 Entrance Pools

There are five entrance pools at the downstream end of the fishway that combine fishway flow with auxiliary water to attract fish. The entrance pools are larger than the fishway pools and are designed to operate within a specific velocity range to attract fish.

3.4.4.12 Criterion 4.2.2.12 Transport Velocity (velocity between 1.5 and 4.0 ft/s)

Transport velocity in the entrance pools is reduced by incorporating an orifice at the bottom of the entrance pool weir walls to meet velocity criteria. At low fish passage flow, the transport velocity in the entrance pools is 3.2 ft/s and at high fish passage flows, the transport velocity in the entrance pools is 4.0 ft/s.

3.4.4.13 Criterion 4.2.2.13 Entrance Pool Geometry

The entrance pool geometry is sized to convey the fish ladder flow combined with auxiliary water to attract fish to the fishway entrance. Specific design of the AWS system and incorporation of AWS diffusers and plumbing within the entrance pools is beyond the level of detail of this document.

3.4.5 Auxiliary Water System

NMFS criteria specify attraction flow requirements in Section 2.4.2 as 5% to 10% of the Design High Passage Flow. Because flow in the fish ladder is less than the required attraction flow, this design concept requires an AWS to meet attraction flow requirements. Anticipated AWS flow rates are listed in Table 10.

Table 10: Summary of Fish Ladder Flows, AWS, and Attraction Flows

	Fishway Discharge (cfs)	AWS Discharge (cfs)	Attraction Flow (cfs)	% of Q_{HFP}
Q_{LFP}	32	185	217	5.0
Q_{HFP}	93	185	278	6.4

Screened water can be taken from the powerhouse forebay through a submerged intake to minimize air entrainment. The powerhouse intake is already screened to prevent entraining.

3.4.6 Energy Dissipation

Required energy dissipation for AWS is specified in NMFS Criteria 4.3.6. Energy dissipation in the AWS would be by means of a turbine or series of turbines, selected to operate at 185 cfs and 18 to 20 ft of head. Flow leaving the turbines would enter a stilling basin, and flow leaving the stilling basin would pass through vertical plate diffusers before entering the fishway. Flow would enter the fishway at Pool 35 through the wall of the ladder. Vertically-oriented diffusers installed within the fishway wall are preferred for ease of maintenance (WDFW, 2000).

The stilling basin is sized to provide an EDF of 16 (ft-lbs/s)/ft^3 at heads up to 1.5 ft, in case the turbines do not extract all potential energy from the auxiliary water. The stilling basin collects discharge from the turbines prior to flow passing through the diffusers.

Before AWS flow enters the ladder, it first passes through a series of horizontal diffusers, intended to minimize turbulence and direct flow into the fishway. The diffusers consist of rounded bar stock installed vertically in the wall of the fishway for ease of operation and maintenance.

Specific design of the AWS system is beyond the scope of this document but provisions have been incorporated into the conceptual design and price structure.

3.4.7 Upstream Juvenile Passage

3.4.7.1 Criterion 10.2 Entrance Hydraulic Drop (0.5 ft)

Because juvenile fish are present in the Rogue River throughout the year, the entrance is designed for juvenile passage and to operate at a 0.5 ft drop.

3.4.7.2 Criterion 10.2.1 General Criteria Head (Head over weir ≥ 2X weir breadth)

Weir breadth is 12 inches at weir wall base and tapers to 6 inches at the weir crest. Minimum flow depth over the fishway weirs is 12 inches, so the fishway weirs will exceed twice the weir breadth at all flows.

3.4.7.3 Velocity Guidelines (Velocity for Swimming Distances less than 1 ft 3 to 4.5 ft/s)

Velocity guidelines for short swimming distances are applicable to areas of transitional flow and flow over weir crests. For the proposed installation, the weir crests would be the sole locations where this criterion applies, and the limiting value would be taken at the 10% exceedance flow ($Q_{10\%}$). The 10% exceedance has been shown as the upper limit for juvenile migration (NMFS, 2008). Velocity over fishway weirs at the $Q_{10\%}$ is 4.4 ft/s.

3.4.7.4 Recommended Guideline to Reduce EDF to 2.0 (ft-lbs/s)/ft^3 for Juvenile Passage

At 10% exceedance flow ($Q_{10\%}$), EDF = 2.2 (ft-lbs/s)/ft^3. Pool sizing was increased to keep EDF's near 2.0 for the range of operational flows to aid juvenile passage.

3.5 ESTIMATED COSTS

Contingencies for items vary from 15% to 20%. The estimate does not include any allowances for environmental studies, reports, permitting, mitigation, new fish passage, or restored hydropower generation. All estimates are in 2010 dollars.

Table 11: Construction Costs for Fish Passage

Item	Qty	Units	Unit Price	Amount	Contingency %	Contingency $	Subtotal
Site Investigations							
Geotech	1	LS	25,000.00	25,000.00	20	5,000.00	30,000.00
Surveying	1	LS	15,000.00	15,000.00	20	3,000.00	18,000.00
Materials Testing	1	LS	8,000.00	8,000.00	20	1,600.00	9,600.00
Structural Inspection	1	LS	5,000.00	5,000.00	20	1,000.00	6,000.00
Engineering Design							
Engineering	1	LS	125,000.00	125,000.00	20	25,000.00	150,000.00
Site Preparation							
Mob/Demobilization	1	LS	50,000.00	50,000.00	20	10,000.00	60,000.00
Erosion-Turbidity Controls	1	LS	100,000.00	100,000.00	20	20,000.00	120,000.00
Dewatering	1	LS	20,000.00	20,000.00	20	4,000.00	24,000.00
Concrete Demolition and Disposal	1	LS	275,000.00	275,000.00	20	55,000.00	330,000.00
Rock Excavation	500	CY	150.00	75,000.00	20	15,000.00	90,000.00
Concrete Fish Ladder Construction							
Concrete work	1,500	CY	600.00	900,000.00	20	180,000.00	1,080,000.00
Reinforcing	300,000	LBS	2.50	750,000.00	20	150,000.00	900,000.00
Inlet Control Structure	1	LS	150,000.00	150,000.00	20	30,000.00	180,000.00
Auxiliary water supply system	1	LS	500,000.00	500,000.00	20	100,000.00	600,000.00
Electrical and Controls	1	LS	200,000.00	200,000.00	20	40,000.00	240,000.00
Construction Management							
5% of Construction Cost	1	LS	149,900.00	149,900.00	15	22,485.00	172,385.00
Estimated Project Value (without contingency)				3,347,900.00			
Estimated Project Value with contingency				4,009,985.00			

4.0 HYDRO FEASIBILITY PRELIMINARY ASSESSMENT

4.1 INTRODUCTION

HDR|DTA reviewed the 1982 Tudor Engineering feasibility study for a new hydro plant at the Gold Ray Dam. The Tudor concept included revising and enlarging the intake works, removing some material from the streambed downstream of the plant to lower the tailwater, and constructing a completely new powerhouse with a 4,500 kW S-type turbine generator set and modern switching and control equipment.

To update the study, HDR|DTA accepted the conceptual design prepared by Tudor, and the resulting quantity estimates that they produced based on that design. While there have been some advances in hydro turbine technology since the time of the original study, particularly in the development of axial flow turbines, we have not attempted to develop alternatives to the earlier design. We do expect that application of newer technology such as S turbines could decrease the excavation depth for the tailrace, but that is only a small fraction of the construction cost.

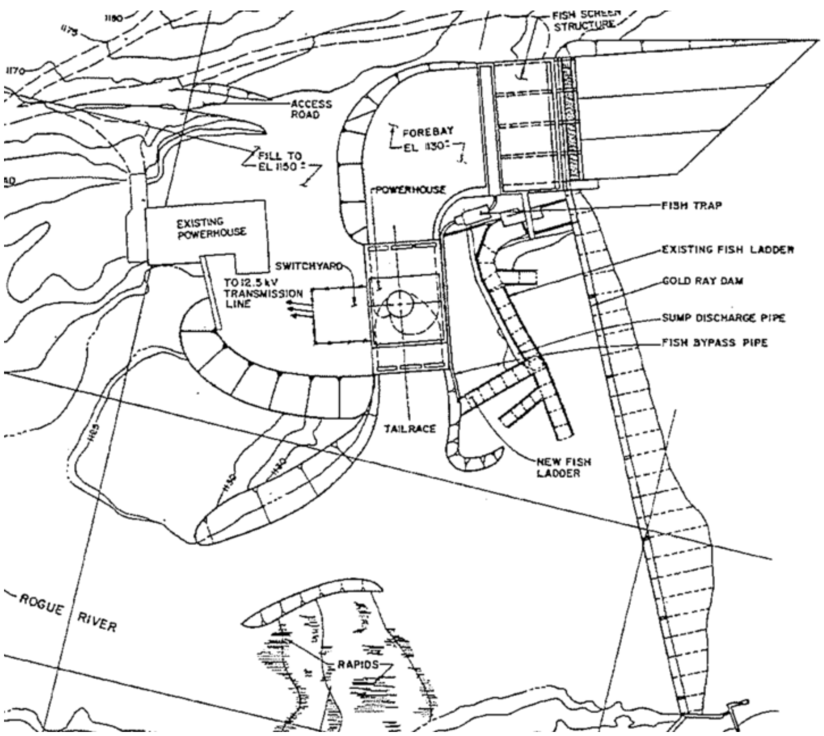


Figure 12: New Hydroelectric Concept

4.2 METHODOLOGY

4.2.1 Costs

the Tudor cost estimates for the major project components and construction categories were updated to current costs. Turbine-generator pricing was obtained in January 2010 for a nearly identical turbine-generator complete package from Andritz Hydro. Where possible, recent pricing developed for other projects was used to recalculate the costs. Where such data was not available, or was not applicable to this site, the costs were updated using escalation indices for materials and equipment as developed by the U.S. Army Corps of Engineers. Costs were updated for 2010 conditions.

4.2.2 Benefits

HDR accepted the energy production calculated by Tudor. There is little reason to expect that there would be significant change in the conditions for generation. While there have been incremental improvements in turbine efficiency since 1982, but those are marginal, in the order of 1% to 2%. Those differences are not significant in comparison with the uncertainties of how the downstream channel might be modified, which could affect the energy production by 10% or more.

4.3 EVALUATION OF ECONOMIC FEASIBILITY

4.3.1 Capital Costs

4.3.1.1 Construction

The summary of costs for the construction and equipping of the new power house are tabulated below in table 1. The costs for the intake work and fish screen developed by the Gig Harbor office must be added to for the sum of the construction costs. The Costs for Electro-mechanical equipment have been escalated by a factor of 2.15, civil construction costs by 2.5 to 3.0.

4.3.1.2 Engineering

The engineering costs for this project are estimated to be approximately 12% of the construction costs, in line with similar projects.

4.3.1.3 Licensing

This project would require a FERC license, and for a similar project on a typical river, the costs of the studies and filings necessary for the license would be in the order of $2,000,000 to 3,000,000. However, the Rogue River is a very significant river in the view of local residents and fisheries interests, and at the least, there would be significant opposition, increasing the number and costs of environmental studies. There is one threatened species of salmon in the Rogue River, but 5 total runs present. Oregon Coast Coho are threatened. Even with only one protected species, the five present runs of salmonids and the wild and scenic status downstream of the project, make this a difficult place to license hydroelectric generation.

An allowance of an additional $2,000,000 for extra public outreach and permitting activities was included, but this amount could easily be exceeded. As noted above, the final license could include conditions that would reduce the estimated potential generation.

4.3.1.4 Contingency

Based on the very brief assessment to update the feasibility study, a contingency for uncertainties and unknowns of 25% -30% should be applied to the total of the above capital costs.

4.3.1.5 Cost Summary

Based on the above, HDR|DTA adjusted the Tudor cost estimate to 2010 costs, and the estimate of Capital Cost is summarized as follows:

Table 12: Adjusted Capital Cost Summary

Environmental and permitting	$6,000,000
Land and Land Rights	0
General Construction & mobilization	435,000
Powerhouse, forebay, & tailrace construction	5,455,000
Turbine and Generator	6,060,000
Accessory Electrical Equipment	1,040,000
Miscellaneous Mechanical Equipment	110,000
Transmission Line	282,000
Subtotal	$19,382,000
Contingency 30%	5,815,000
Engineering and construction management 12% (Contingency Included)	3,024,000
Owners misc costs 5% (Contingency Included)	756,000
Powerhouse Total Capital Cost	$28,977,000
Intake/Fish Screen/ Fish Trap and Fish Ladder (see 5.2 below)	24,770,000
TOTAL	$53,747,000

4.3.2 Economic Benefits

HDR has re-estimated the pricing of the energy production from the project based on our understanding of the current market for renewable energy. The following components of energy production value were considered:

4.3.2.1 Basic energy

The spot market for electric energy at the California Oregon border ranges from a low of nearly zero during spring runoff to a high of $200/MWH during some hours when air-conditioning loads in California are high, with occasional higher brief peaks. We have estimated that the near future average price for energy will be approximately $60/MWH.

4.3.2.2 Renewable Energy credits

We expect very slow escalation of this price, in the order of 1% per year.

"Green Tags" or other marketable benefits for renewable energy: The value of renewable energy attributes has been steadily rising as more states establish renewable portfolio standards. We would currently estimate the price of green tags in this region to be $40/MWH, and expect inflation of 5% to 10% per year for the next few years. However, given the location of the project on the Rogue River where the dam is something of a barrier to fish passage, we consider that there will be little possibility of project qualifying for renewable energy benefits, and we have not included any such benefits for the project.

4.3.2.3 Capacity

Firm capacity has value in the market, as assurance that utilities can provide for all the conceivable demand that their customer will have at any given time. However, the energy production from run-of-the-river projects such as the Gold Ray plant are entirely subject to the vagaries of weather, and as in this case, minimum flow requirements for habitat. Since there is no storage at the dam, this plant will not always be able to generate a consistent amount whenever called on, and its capacity will not qualify as "Firm." No revenue was attributed to firm capacity, or to the related service of "spinning reserve."

Other ancillary services such as reactive power contribution or voltage stabilization from this small plant would not be significant to the local utility, and were given no value.

4.4 SCHEDULE

The planning for the project should include not less than 8 years for permitting and construction, before any revenue is produced by the plant. In a more detailed financial analysis, the cash flow for licensing and engineering should be distributed over this entire period, with 50% of these costs occurring in the first three years.

4.5 REVENUE

The 1982 report forecast average annual energy production of 31 million kW/hr (31 GWH). However, it appears that the energy calculations were made using monthly average flows, which always overstate the generation, because in many months there are days when the flow exceeds the plant capacity, increasing the average for that month. The Tudor study used only 19 years of records, and best practice would use 50 years of daily flow data. We also caution that for this site, the generation could be reduced by operational constraints of license conditions. For example, the study made allowance for 100 cfs continuous bypass for fish passage, and that could be changed by license conditions to a larger quantity.

In addition, the earlier study noted that the calculated annual energy production ranged from 23.3 GWH to 35.2 GWH. The project needs to be able to make debt service even if the first years of operation are near the low end of this range. HDR estimates that generation of 31 GWH/year would produce revenue of $1,800,000 per year. If the energy could be classified "renewable" as defined by Oregon or California renewable portfolio standards, then the green tags could potentially increase the revenue by 30% to 50% in the current market.

Table 13: Net Annual Revenue Generation

Generation	Revenue	O&M Costs	Net Annual Revenue
23.3 GWH low year	$1,400,000	$300,000	$1,100,000
31 GWH average year	$1,800,000	$300,000	$1,500,000
35.2 high year	$2,100,000	$300,000	$1,900,000

Operation and Maintenance costs were estimated to be $300,000/year, including plant operation, sinking fund for future repairs, compliance monitoring, and insurance.

Assuming a discount rate of 4.5% and a 40 year life, the present value of the revenue stream of $1,800,000 per year, less O&M costs is approximately $25,000,000, calculated from the beginning of operation.

4.6 CONCLUSIONS

4.6.1 Preliminary Economic Feasibility

On the basis of the above estimates for costs and benefits, HDR predicts that the Gold Ray site on the Rogue River would likely not be a profitable investment, even if the environmental barriers could be overcome. The benefit / cost ratio for the project is estimated to be 0.40.

4.6.2 Future Steps

If the project were to be advanced, a more detailed feasibility study must be made, which would include obtaining updated manufacturers quotations for equipment, thorough review of the conceptual design to evaluate alternative equipment configurations, which could impact the construction costs of the powerhouse also. Actual construction would not start for several years, so future escalation of costs must be considered in a detailed feasibility study. However, given the economics of the project, such future work cannot be recommended.

The energy calculations should be redone based on daily average flows and a longer period of record, and the effect of higher fish passage flows and impact of other foreseeable license constraints should be modeled. Operation of the plant would not begin for several years, and the prediction of the energy market that far in the future is a subject beyond the scope of this report, but should be considered in a detailed study.

The capacity for the plant should be also be modeled to optimize it for assumed license constraints and energy values. A financial model that includes input from prospective buyers of the output for valuing the energy, and a cash flow model for the project development, construction, and operation and maintenance costs of the project with owner specified rates for interest, insurance, operating labor included. The financial analysis should include a stochastic model for the variability of the water resource to evaluate the risk posed by low water years.

5.0 FISH SCREENING FOR HYDROELECTRIC FACILITY

Gold Ray Dam is currently under evaluation for the possible incorporation of power generating hydro facilities. As part of the new facilities, the flow that will enter the turbine directly from the forebay will require physical fish protection to prevent fish entrainment. This is proposed to be accomplished by incorporating a physical fish screen facility that meets current NOAA fisheries design guidelines. This cursory review is being performed on an assumed configuration of screen solution the capitol cost, operation and maintenance cost, and life cycle cost and salvage value based on 50 years of operation.

Two documents were utilized as the basis for this cursory review. A feasibility report of the potential hydroelectric development prepared by Tudor Engineering, November 8th,1982 and the NOAA 2008 Anadromous Salmonid Passage Facility Design Criteria for Northwest Region, Portland, Oregon. Utilizing the provided diversion rate, approximate water surface operation elevations, bathymetry elevations a V-screen configuration was selected as the assumed fish protection screen configuration for this level of review.

5.1 ASSUMPTIONS

The following assumptions have been made for the purpose of providing this conceptual assessment of a Dual V-screen intake at Gold Ray Dam.

- Maximum diversion flow required at Gold Ray dam will not exceed 3,115.5 cfs at the lowest forebay pool elevation, as stated in the hydroelectric Operation Study (November 8, 1982) by Tudor Engineering Company
- The minimum pool (forebay) elevation at the diversion structure is 1146.0 feet.
- Top of slab elevation for diversion structure is 1130.0 feet.
- Current intake structure can be extended downstream from its current position
- Salvage value of existing screens and trash racks not included in costs
- Permitting costs have not been included into the capital cost
- Inflation on labor, material, and power costs have been set at an average annual increase of 4%
- Engineering design cost is set at 6% of construction costs: $1,402,000

5.2 APPRAISAL LEVEL SCREEN REVIEW

This cursory review is focused on looking at an assumed V-screen configuration based on information supplied by client, field pictures, and acceptable diversions designed for this magnitude of flow.

5.2.1 V-Screen Description

V-screen intakes are comprised of two vertical flat plat screens composed of wedge wire or profile bar. The mouth of the V-screen is set up stream and the throat at the down stream. In general each side is composed of vertical flat plat screen that converge to a fish bypass system in the throat. Prior to entering the V-screen a coarse trash rack is placed to keep larger debris from entering. Access to the trash rack is provided for debris removal via a bridge system. After the coarse trash rack, the entrance to the V-screen straightens out for some distance to help assure that a laminar flow prior to the mouth of the flow convergence of the V-screen is provided. The two vertical screens then converge to a fish bypass channel that directs fish, into the fish bypass at the throat. Down stream of the V-screen, gates are used to moderate the diverted flow through the screen so they do not exceed the approach criteria for fish screening, maintain screen submergence depths and allow for the best hydraulic conditions at the entrance to the bypass. The fish bypass conduit will follow NOAA and ODFW fisheries design guidelines relative to proper conveyance in a non-pressurized conduit with approved point of downstream release. The V-screen is cleaned by brushes which can automatically or be manually timed to travel back and forth along the length of the screen. Large debris will be caught by the coarse trash rack and smaller debris will be removed by the brush cleaning system. The small debris will be moved by the sweeping velocity to the through of the V-screen and enter the fish by pass channel and ultimately deposited on the down stream side of the dam. Figure 13 describes a potential plan view of the proposed V-screen capable of taking 3115.5 cfs.

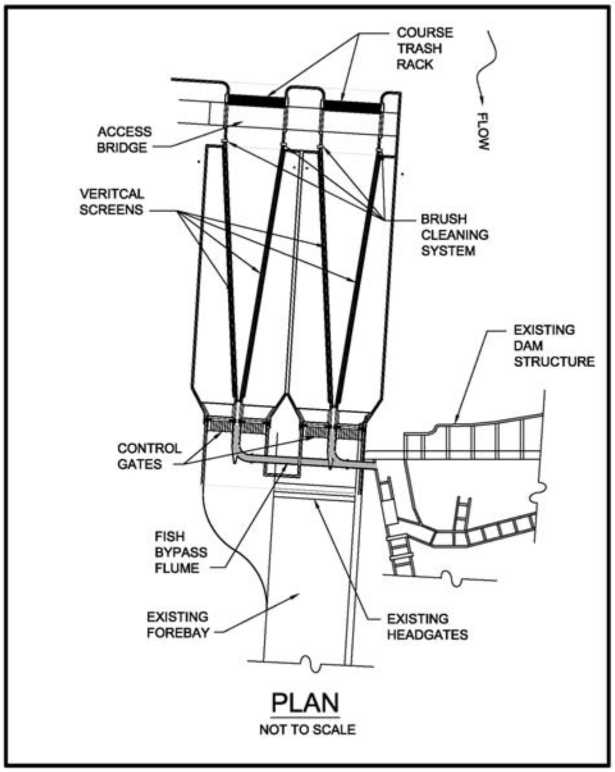


Figure 13: V-Screen Plan View

The following paragraphs generally describe the major features of a V-screen configuration.

5.2.1.1 Trash Boom and Trash Racks

Large floating objects, such as trees, can damage a fish screen and its cleaning system. Therefore, trash booms and racks must be considered. Where applicable, a floating trash boom or piles can be placed in the water in front of the facility to deflect any large floating objects away. A trash rack should be placed across the intake far enough from the screen entrance to stop any object that avoids the rash boom or piles (the first line of defense for a fish screen) and cause minimal effect on laminar flow to the screened area.

5.2.1.2 Screen Cleaning

For this installation it is assumed that screen cleaning will be accomplished by a brush system. Regulatory criteria state that screens must be cleaned as frequently as necessary to prevent head loss greater than 0.10 ft occurring from the upstream to downstream points of the screen surface. Automated pressure differential settings are required to accommodate the maintenance of this minimum differential. In addition a manual cleaning cycle will be suggested once in operation and will include a set minimum interval. When a fish screen collects debris, the total available screen area is reduced, increasing the approach velocity above the accepted criteria at other locations (commonly referred to as "hot spots") and possibly impinging/injuring fish.

5.2.1.3 Fish Screen

Fish screen material is generally composed of perforated plates, wedge wire, or profile bar. In all cases the required screen slot width shall be equal to or less than 1.75 mm. If perforated plate is used round openings should be a maximum diameter of 3/32nd of an inch. If the preferred screening material is wedge wire or profile bar, the arrangement of the bars shall be perpendicular to the flow. This will assist in moving small debris by the screens. The screening material in any case is to be stainless steel.

5.2.1.4 Corrosion and Fouling

It is recommended that stainless steel be used for screen to reduce clogging due to corrosion and to provide the best useful life. Active and passive corrosion protection system and/or contact protection can also be considered if there is dissimilar metal contact with the framing. Consideration may be given to the use of antifouling protection on the screen surface to reduce biological growth on the screens.

5.2.1.5 Baffles

Adjustable flow control baffles can be placed behind the screen to control and better distribute the flow, thereby helping to help assure consistent approach velocities across the screen face. In some cases provisions for baffles are made for future application if needed. Screen design safety factors and proper design may exclude the need for baffles and provisions are usually made even if not initially provided. If baffles are necessary and not used, high velocity "hot" spots may occur, possibly impinging fish on the screen face. Baffles should be placed close as possible behind the screen to be best effective. Also, baffle controls should be readily accessible above the water surface and individually controllable.

5.2.1.6 Flow Control

As opposed to velocity control with baffles, the regulatory agencies usually require assurance that actual diversion flow will be at or less then the screen design flow at the minimum submergence depth and that flow diversion rates are measured and controlled. Consequently, every diversion facility should incorporate flow control and some level of diversion rate monitoring, whether it is gates, weirs, valves, or pumps.

5.2.1.7 Structural Integrity and Flood Protection

The facility structure should be built to withstand high flows and debris loads. As mentioned earlier, if screen cleaning fails and debris collects on the fish screen, a water surface elevation differential between the front and back of the screen will develop. An alarm system should be in place to warn of such and emergency, but if the alarm fails also, the structure needs to be strong enough to handle this increase in load. Blowout panels above the screen or in a non-screed bay can be installed to open, or a switch can turn off the turbine, if the load gets too great.

5.2.1.8 Maintenance

Maintenance is the single most critical aspect of an installed fish facility. Screen cleaning systems, pumps, valves, and gates are all mechanical system that need care to function properly. Screen removal and cleaning, trash boom and rack cleaning, and sediment removal must be performed on a regular basis. Brush screen cleaners need to be check for wear and proper orientation and contact pressure to the screen. Trash booms and racks need to be inspected for debris loads. Most importantly, each facility is usually required to have and follow an operations and maintenance plan.

5.2.1.9 River Flows and Stages

An adequate area of screen must be submerged to meet regulatory screening criteria. Historical flows and stages must be studied so that the fish screen is placed at the proper elevation. Improper placement of the fish screen could result in not enough submerged screen available and approach velocity criteria exceedance.

5.2.2 Current Intake Design at Gold Ray Dam

♦ Pre – Screening	Inclined Trash Rack
♦ Fish Screen	Inclined Fish Screen (17°)
♦ Fish Screen Slot Width	2 mm
♦ Fish Screen Approach Velocity	1.3 ft/s (approximate)
♦ Fish Screen Cleaning Method	Reverse Flow through Screen

5.2.3 Proposed Screen Design

Current NMFS criteria states that the maximum allowable screen slot width is 1.75mm or 3/32 of an inch for perforated plate–approximately 6% smaller then the current configuration. The highest approach velocity that can be seen by the fish screen is 0.4 ft/s. The current design has an approach velocity over 3 times than that allowed by NMFS, which will cause impingement. Finally, the current design allows fish to bypass the fish screen during a cleaning cycle, allowing entrainment to the turbines.

In reviewing the existing site, it appears that one possible screen option would be a dual V-screen arrangement in parallel. The dual configuration would lessen the overall length of the intake structure required to maintain approach velocities at or below 0.4 ft/s The V-screens will be cleaned by traveling brushes capable of continuous operation in high sedimentation periods. This arrangement of a dual V-screen and traveling brushes meets all criteria for anadromous salmonid fish passage.

5.2.4 Dual V-Screen Design

♦ Pre- Screening Protection	Inclined Trash Racks and Boom
♦ Fish Screens	Vertical Wedge Wire Profile Bar Wire
♦ Fish Screen Slot Width	1.75mm
♦ Fish Screen Approach Velocity	0.4 ft/s (maximum)
♦ Total Screen Area Required	9,800 SF (approximate)
♦ Fish Screen Cleaning Method	Traveling Brush

5.2.5 Picket Barrier Design

A picket barrier may be necessary in the tailrace to keep migrating fish from moving up into the turbines. The intent of the picket barrier is to diffuse nearly the entire flow through pickets extending the entire width of the impassable route, sufficiently spaced to provide a physical barrier to upstream migrant fish. This type of barrier is known as an exclusion barrier. One potential system that may be used for this site is assumed to be a picket barrier system. This system would have a concrete foundation and walls with a heavy steel frame supporting pickets that are approximately 1.5 inch in diameter with a clear space of 1 inch. An option for this site could be a hydro thane HDPE trash rack that is made of flat HDPE bars with a 1 inch clear space.

The average design velocity though the pickets should be less than 1.0 ft/sec for all design flows with a maximum velocity less than 1.25 ft/s, or half the velocity of adjacent passage route flows whichever is lower. The average design velocity is calculated by dividing the flow by the total submerged picket over the design range of stream flows. There is an additional criteria relating to head differential control. This is the maximum head differential across the pickets must never exceed 0.3 feet over the clean picket condition. If this differential is exceeded, the pickets must be cleaned as soon as possible. The orientation of picket barrier must be designed to lead fish to a safe passage route. Free board of the barrier is set for 2 feet with a minimum submergence depth at the picket barrier at low design discharge must be two feet for at least 10% of the river cross section at the barrier. One final picket criteria is the porosity of the picket array must have a minimum of 40% open area.

Debris management is a concern along with sediment. While this is screened flow from the V-Screen in the forebay, there could be debris that may lay on the screen from the river during high water events. A debris management plan should be determined for potential condition that may deposit debris on the down steam side of the barrier as well as sediment accumulation in the tailrace. Assuming the normal operating depth is 15 ft in depth in the tailrace, a barrier inclined at an angle of 60 degrees would need to be approximately 150 ft in length. This is assuming that the submerged length of the barrier is approximately 150 ft in length. This assumes 3115.5 cfs and a maximum velocity less than 1.25 ft/s.

5.2.6 Horizontal Draft Tube Diffusers Design

A horizontal draft tube diffuser may be applicable below the power house at the turbine draft tube outlet to prevent fish from accessing the turbine runners if the velocity is lower than 16 ft/s. The criteria for this will be to manage the flow to an average velocity exiting the horizontal diffuser grating to be less than 1.25 ft/s, and distributed as uniformly as possible. The maximum velocity should not exceed 2.0 ft/s. The clear spacing between diffuser bars and any other pathway from the tailrace to the turbine runner bust be less than 1 inch clear. The placement of the diffuser must be submerged a minimum of 2 feet for all tailwater elevations.

5.3 CURSORY REVIEW OF COSTS

Table 14: Life Cycle Cost of Facility Based on 50-Year Operation

Item	Year 1	O&M Cost	Present Value	Total Present Value Coat
Capital Cost of Fish Screen	$23,368,366		$23,368,366	$23,368,366
Salvage in 50 years (5% initial) = $1,168,418				
Yearly Labor		$23,750	$487,365	$487,365
Facility Engineer/Management Staff	2,500			
Facility Superintendent	4,250			
Facility Technician	7,000			
Facility Technician	10,000			
Yearly General Maintenance/Repair and Replacement	$3,000	$176,599	$3,623,930	$3,623,930
Yearly Power Cost	$7,796	$7,796	$159,987	$159,987
Discount Rate and Escalation Rate = 4%				
Number of Years of Operation = 49 (after first year of operation)				
TOTAL ESTIMATED PRESENT VALUE				$27,468,661

Notes:

1. *Salvage value based on mechanical equipment, no salvage on concrete structure*
2. *General Maintenance / Repair & replacement based on Power costs, general O&M costs and mechanical equipment cost.*
3. *Power cost assuming total power demand of 40 HP running once every 2 hours/day for 20 minutes.*

5.4 CONCLUSION

For this cursory review, focus is on reviewing a V-screen system that would meet ODFW and NMFS criteria for screening downstream migrating anadromous fish. The screening system chosen for this level of review is one that HDR has experience in designing, constructing, and monitoring in the field. In summary, the double V-screen has 4 screens that are approximately 155 feet in length and 16 feet in height. Each screen has brush cleaning system and a baffle system behind. Coarse trash racks are placed at the mouth of the screen with a fish by pass system in the throat. Control gates are located in the forebay behind the screen to control the amount of diverted flow and maintain proper submergence. The screens are supported by a structural steel framing system that mounts to a reinforced concrete sub substructure. It is projected that the cost of this system will be in the range of $24,000.000 to $30,000,000 present worth value based on a 50 yr life cycle cost analysis.

6.0 ABBREVIATIONS AND ACRONYMS

ARRA	American Recovery and Reinvestment Act
AWS	Auxiliary Water System
cfs	
EA	Environmental Assessment
EDF	energy dissipation factor
FERC	Federal Energy Regulatory Commission
ft/sec	feet/second
LiDAR	Light Detection and Ranging
NOAA	National Marine Fisheries Service
NOAA	National Oceanic and Atmospheric Administration
OAR	
ODFW	Oregon Department of Fish and Wildlife
ORS	Oregon Revised Statute
PMF	Probable Maximum Flood
RDG	River Design Group
SCG	Slayden Construction Group
USGS	United States Geological Survey

7.0 REFERENCES

Chapter 1:

Bengtson, Glen S.
1989 Analysis *and Rehabilitation of Ambursen Dams*, pp. 466-476, Conference Proceeding Paper, Waterpower Conference '89

The California Oregon Power Company
1941 Correspondence January 30th , 1941 to May 2nd , 1941
1941 Gold Ray Dam reconstruction Design Data and Buttress Stresses, January 15th, 1941
1941 Plans Gold Ray Dam, January 1941: G-6188, F-6138, E-6139.

Lund, Guy S.
2008 *Duckett Dam — An Ambursen Dam Evaluation*, Guy S. Lund, URS Corporation, U.S. Society on Dam, 2008 Annual Conference.

Oregon Game Commission
1972 Portland, Oregon: Correspondence January 18th, 1972

Parks & Recreation Commission
1971 Medford, Oregon: Correspondence October 14, 1971 to November 15, 1971

Quinn, Michael S. and Gary Dale
Zen and the Art of Passive Rock Mass Reinforcement to Stabilize an Ambursen Dam, Michael S. Quinn, Senior Geotechnical Engineer, and Gary Dale, Geotechnical Engineer, Clough, Harbour & Associates, ASDSO Northeast Regional Conference

Tudor Engineering Company
1982 *Feasibility Report of the Potential Hydroelectric Development, Gold Ray Dam*; Jackson County, Oregon; December, 1982

Waterwatch
Gold Ray Dam Removal Fact Sheet.

Wheelock, David C. and Neal A. Wilkins
1991 *Analysis and Stabilization of Ambursen Dams—Two Case Studies*. pp. 1467-1476, Conference Proceeding Paper, Waterpower '91 Conference: A New View of Hydro Resources.

Chapter 2:

Allen, M. A., and T. J. Hassler
1986 Species profiles: life histories and environmental requirements of coastal fishes and invertebrates (Pacific Southwest) – Chinook salmon. U.S. Fish and Wildlife Service, Biological Report 82 (11.49). U.S. Army Corps of Engineers, TR EL-82-4.

Beauchamp, D. A., M. F. Shepard, and G. B. Pauley
1983 Species profiles: life histories and environmental requirements of coastal fishes and invertebrates (Pacific Northwest) – Chinook salmon. U.S. Fish and Wildlife Service, Division of Biological Services, FWS/OBS-82/11.6. U.S. Army Corps of Engineers, TR EL-82-4.

BOSS International
2008 RiverCAD Software. BOSS International, Madison, Wisconsin.

Bustard, D.R., and D.W. Narver
1975 Aspects of the winter ecology of juvenile coho salmon (*Oncorhynchus kisutch*) and steelhead trout (*Salmo gairdneri*). Journal of the Fisheries Research Board of Canada 32(3):667-680.

Clay, C.H.
1995 Design of Fishways and Other Fish Facilities. Lewis Publishers, Boca Raton, Florida.

ESRI
2005a ArcGIS Version 9.1 Software. Environmental Systems Research Institute, Redlands, California.
2005b Spatial Analyst Extension. Environmental Systems Research Institute, Redlands, California.
2005c 3D Analyst Extension. Environmental Systems Research Institute, Redlands, California.

Garland, John D., P.E.; Waters, Ronald H., P.E.; Focht, Jr., John A. P.E.; Rutledge, John L., P.E.
1995 "Dam Safety Morris Sheppard Dam Rehabilitation," *Waterpower '95* 1995, pp. 2095-2105.

Groot, C. and L. Margolis
1991 Pacific Salmon Life Histories. University of British Columbia Press, Vancouver, British Columbia.

Hickman, T., and R. F. Raleigh
1982 Habitat suitability index models: Cutthroat trout. U.S. Fish and Wildlife Service, FWS/OBS-82/10.5.

Jones, A., and Rafford, C.
1997 "Buttress Dam Rehabilitation" *Maine Section ASCE Technical Seminar* Presented March 19, 1997, Lewiston, Maine

Katapodis, C
1992 Introduction to Fishway Design. Freshwater Institute, Winnipeg, Manitoba.

LaMar, Bruce H .; Ivarson, W. Robert; Tenke-White, Karen
1991 "Rehabilitation and Replacement of Slab and Buttress Dams at the Wissota Hydroelectric Project" *Waterpower '91* 1991, pp. 998

Laufle, J. C., G. B. Pauley, and M. F. Shepard
1986 Species profiles: life histories and environmental requirements of coastal fishes and invertebrates (Pacific Northwest) – coho salmon. U.S. Fish and Wildlife Service, Biological Report 82 (11.48). U.S. Army Corps of Engineers, TR EL-82-4.

National Marine Fisheries Service (NMFS)
2008 Northwest Region. Anadromous Salmonid passage facility design.

Nickelson, T.E., J.D. Rodgers, S.L. Johnson, M.F., and Solazzi.
1992 Seasonal changes in habitat use by juvenile coho salmon (*Oncorhynchus kisutch*) in Oregon coastal streams. Canadian Journal of Fisheries and Aquatic Sciences 49:783–789.

Niziol, Jacob S., P.E.; Paolini, Edward M.
1993 "Case History of Sherman island Hydro Buttress Dam," *Waterpower '93* 1993, pp. 698-709.

Pauley, G.B., B.M. Bortz, and M.F. Shepard
1986 Species profiles: life histories and environmental requirements of coastal fishes and invertebrates (Pacific Northwest) – steelhead trout. U.S. Fish and Wildlife Service, Biological Report 82 (11.62). U.S. Army Corps of Engineers, TR EL-82-4.

Pauley, G.B., K. Oshima, K.L. Bowers, and G.L. Thomas
1989 Species profiles: life histories and environmental requirements of coastal fishes and invertebrates (Pacific Northwest) – sea-run cutthroat trout. U.SFW, Biological Report 82 (11.86). U.S. Army Corps of Engineers, TR EL-82-4.

Reynolds, Robert D., M. ASCE; Joyet, Robert A., M. ASCE; and Curtis, Max O.
1993 "Victoria Dam Rehabilitation," *Waterpower '93* 1993, pp. 688-697.

Sandercock, F.K
1991 Life History of Coho Salmon. *In* Groot, C. and L. Margolis (Eds.). Pacific Salmon Life Histories. UBC Press, Vancouver. 564 p.

Sullivan, K., T. E. Lisle, C. A. Dolloff, G. E. Grant, and L. M. Reid
1987 Stream channels: the link between forests and fishes. Pages 39-97 *In*: Salo, E. O., and T. W. Cundy (eds.), Streamside Management: Forestry and Fishery Interactions, Contribution No. 57, Institute of Forest Resources, University of Washington, Seattle.

U.S. Committee of Large Dams (USCOLD)
1996 Anthology of Dam Modification Case Histories, April 1996. ;Bartlett Dam, p. 3.; Bear Valley Dam, p. 3; Buchanan Dam, p. 6; Cedar Falls Hydro Dam, p. 8; Ellsworth Dam, p. 15; Mathis Dam, p. 31; Morris Sheppard Dam, p. 38; Saxon Falls Hydro Dam, p. 50; Webber Creek Dam, p. 58; Wissota Dam (North Section), p. 58; Wissota Dam (South Section), p. 59

Wolman, M.G
1954 A method of sampling coarse riverbed material. Transactions of the American Geophysical Union 35(6): 951-956.

8.0 APPENDICES

Appendix A: Preliminary Fish Ladder Drawings

Appendix B: Data and Calculations

Appendix A:
Preliminary Fish Ladder Drawings

GOLD RAY DAM

ENVIRONMENTAL ASSESSMENT ALTERNATIVE 2
FISH LADDER CONCEPTUAL DESIGN

PROJECT PARTNERS

JACKSON COUNTY Roads

RESTORATION CENTER

OWEB

AMERICAN RECOVERY AND REINVESTMENT ACT

PROJECT DESCRIPTION

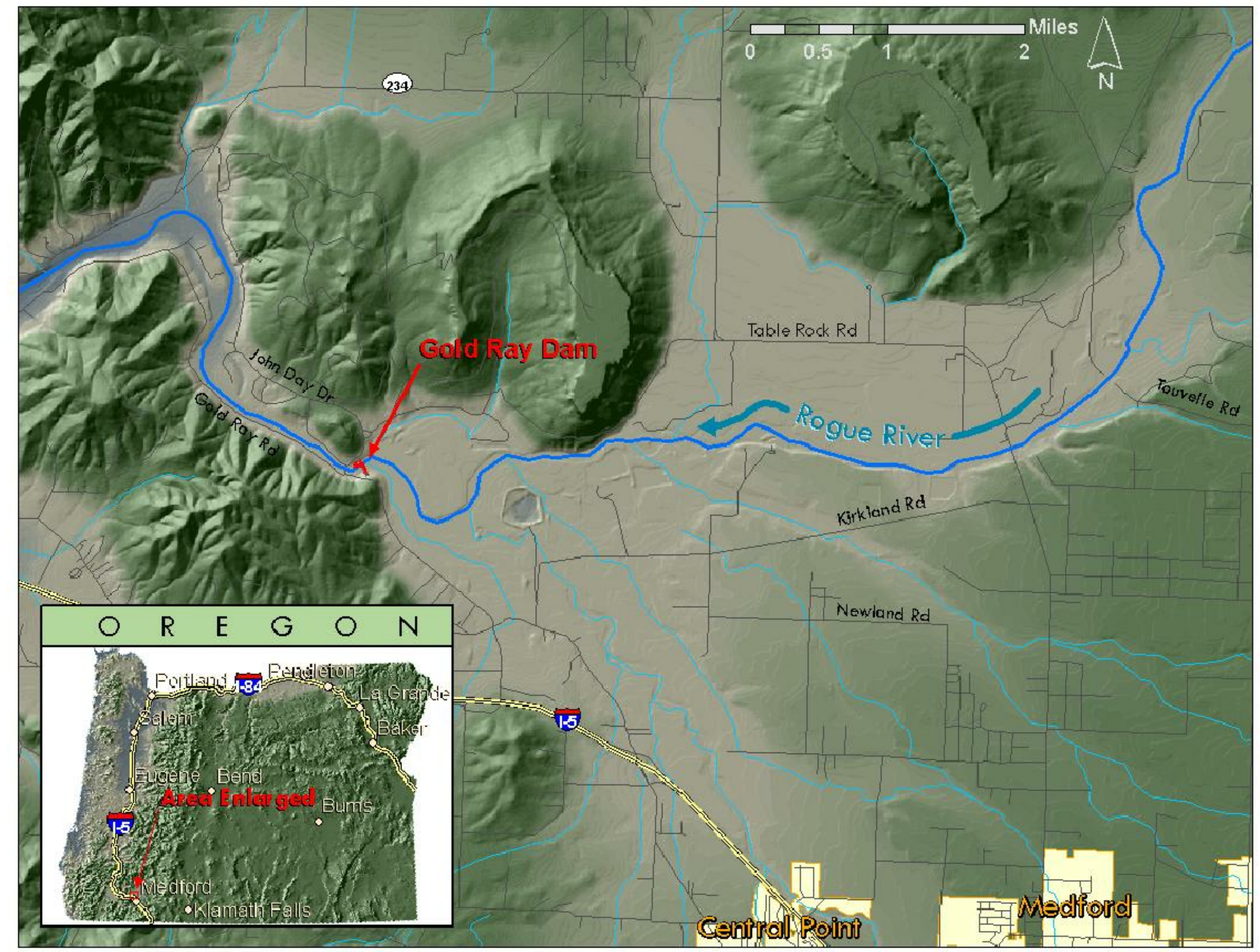
AS PART OF THE NEPA ALTERNATIVES DEVELOPMENT, THREE OPTIONS ARE BEING LOOKED AT FOR THE GOLD RAY DAM. OPTION 1 IS TO REMOVE THE EXISTING DAM AND POWERHOUSE, OPTION 2 IS TO RETROFIT AND REFURBISH THE EXISTING DAM, FISHWAY, AND POWERHOUSE TO PROVIDE HYDROPOWER, AND OPTION 3 IS THE "DO NOTHING" ALTERNATIVE. THIS SET OF DRAWINGS PROVIDES A PRELIMINARY DESIGN FOR A NEW FISH LADDER ON THE NORTH SIDE OF THE DAM THAT MEETS CURRENT STATE AND FEDERAL FISH PASSAGE DESIGN CRITERIA. THE FISH LADDER CONSISTS OF A POOL AND WEIR TYPE CONCRETE STRUCTURE WITH AN AUXILIARY WATER SYSTEM (AWS) THAT PROVIDES REQUIRED ATTRACTION FLOWS AT THE DOWNSTREAM ENTRANCE TO THE FISH LADDER.

BENCHMARK

SURVEY CONTROL USED FOR THE PROJECT IS PROVIDED ON DRAWING 2.0. THE HORIZONTAL DATUM IS NAD 83, STATE PLANE COORDINATES, OREGON ZONE SOUTH, AND THE VERTICAL DATUM IS NAVD 88. THE BENCHMARK COORDINATES CORRESPOND TO THE TOP CENTER OF CONTROL MARKERS LISTED ON DRAWING.

DRAWING INDEX

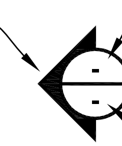
GOLD RAY DAM VICINITY MAP



T36S-R2W SECTION 18
USGS QUADRANGLE: SAMS VALLEY, OREGON
GOLD RAY DAM AT RIVER MILE 126

DIRECTION OF SECTION

SECTION OR DETAIL



SHEET WHERE SECTION OR DETAIL IS SHOWN

CROSS-SECTION SHEET REFERENCE

RIVER DESIGN GROUP, INC.
5098 Hwy 93 South
Whitefish, MT 59937
406.862.4927
311 SW Jefferson Avenue
Corvallis, OR 97333
541.738.2920

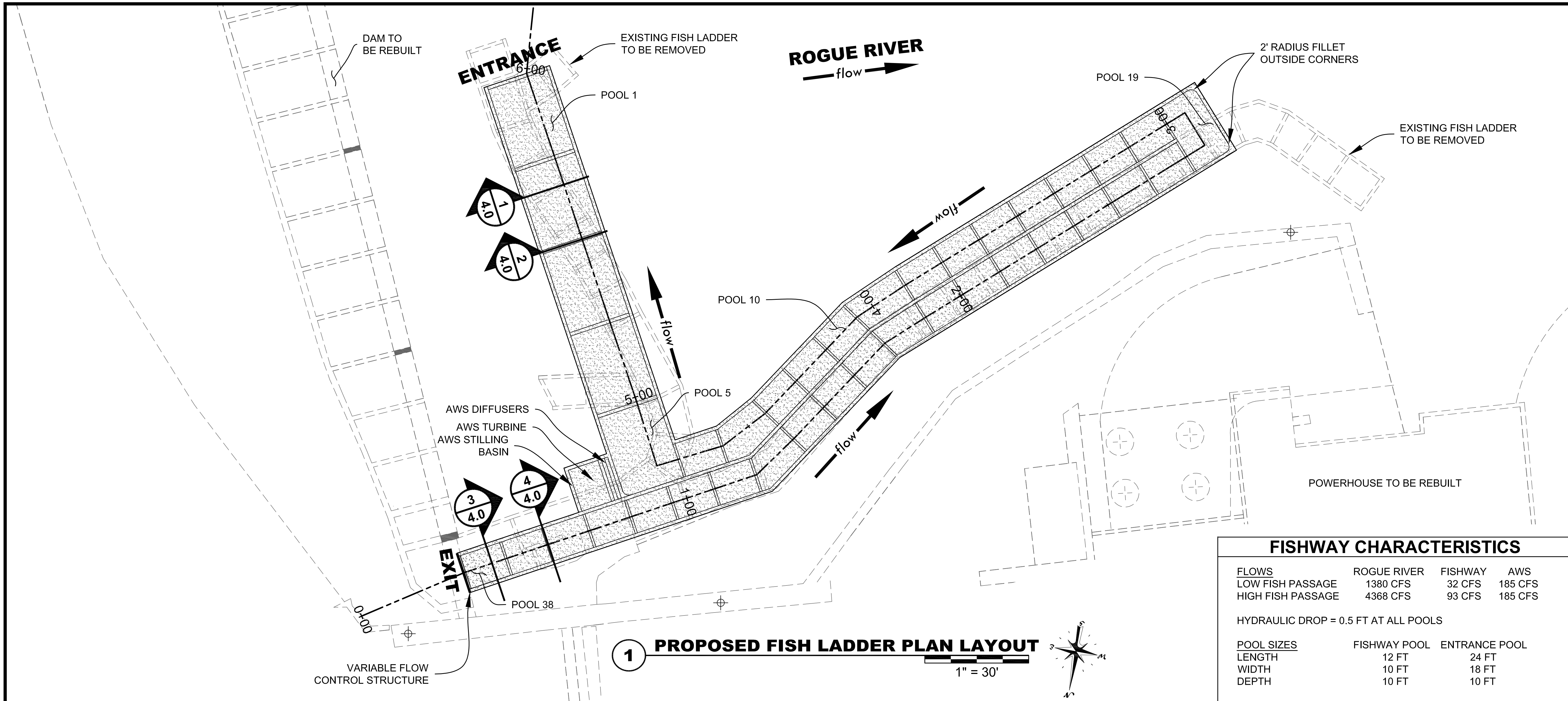
COVER PAGE AND NOTES
GOLD RAY DAM FISH LADDER
JACKSON COUNTY

NO.	DATE	BY	DESCRIPTION	CHK
0	02/11/10	CS	ENVR. ASSESSMENT	SW

PROJECT NUMBER
RDG-09-048

DRAWING NUMBER
1.0

Drawing 1 of 6



1 **PROPOSED FISH LADDER PLAN LAYOUT**

1" = 30'

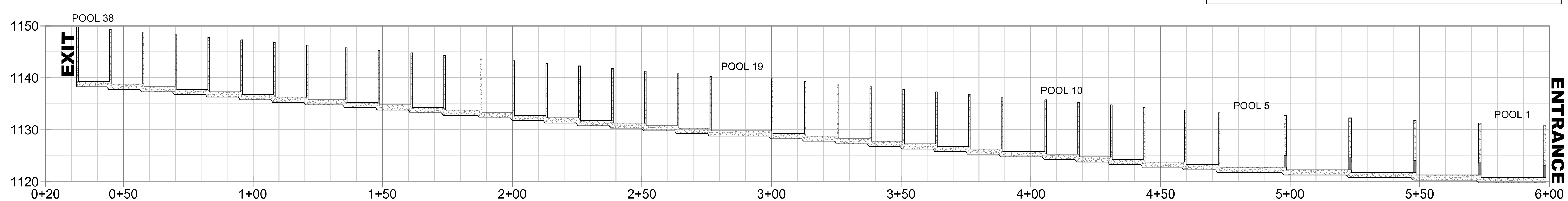
FISHWAY CHARACTERISTICS

FLOWS	ROGUE RIVER	FISHWAY	AWS
LOW FISH PASSAGE	1380 CFS	32 CFS	185 CFS
HIGH FISH PASSAGE	4368 CFS	93 CFS	185 CFS

HYDRAULIC DROP = 0.5 FT AT ALL POOLS

POOL SIZES	FISHWAY POOL	ENTRANCE POOL
LENGTH	12 FT	24 FT
WIDTH	10 FT	18 FT
DEPTH	10 FT	10 FT

CRITERIA	MAX EDF	MAX VELOCITY	MIN DEPTH
FISHWAY POOL	2.4	4.5 FT/S	1.0 FT
ENTRANCE POOL	2.0	4.0 FT/S	1.0 FT



2 **FISH LADDER LONGITUDINAL PROFILE**

HORIZ 1" = 40'
VERT 1" = 20'

RIVER DESIGN GROUP, INC.
5098 Hwy 93 South
Whitefish, MT 59937
406.862.4927
311 SW Jefferson Avenue
Corvallis, OR 97333
541.738.2920

PROJECT LAYOUT

GOLD RAY DAM FISH LADDER

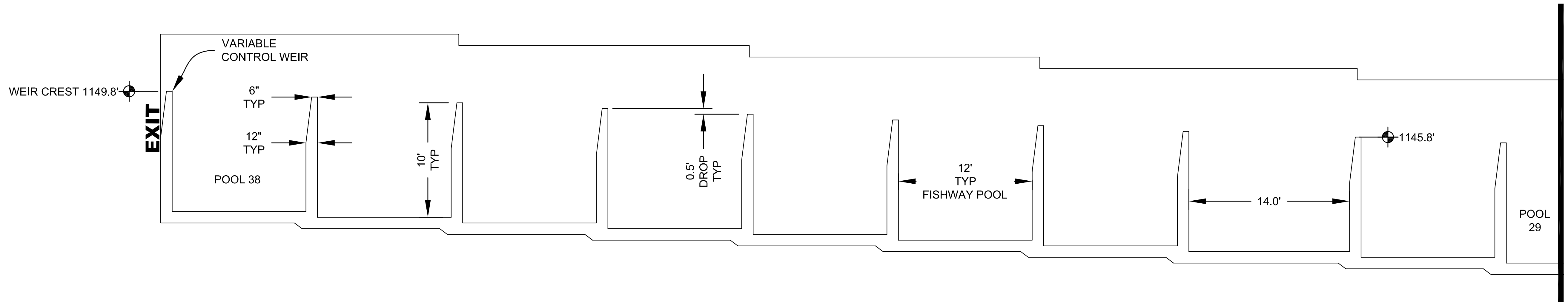
JACKSON COUNTY

NO.	DATE	BY	DESCRIPTION	CHK
0	02/11/10	CS	ENVR. ASSESSMENT	SW

PROJECT NUMBER RDG-09-048

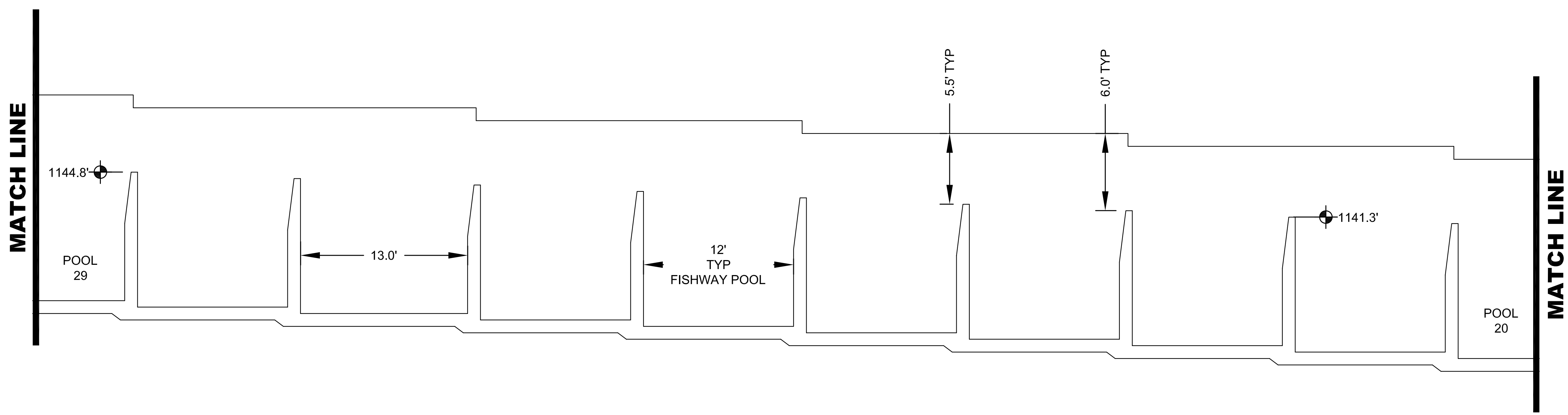
DRAWING NUMBER 3.0

Drawing 3 of 6



1 **FISH LADDER LONGITUDINAL SECTION**

1" = 10'



2 **FISH LADDER LONGITUDINAL SECTION**

1" = 10'

RIVER DESIGN GROUP, INC.
5098 Hwy 93 South
Whitefish, MT 59937
406.862.4927
311 SW Jefferson Avenue
Corvallis, OR 97333
541.738.2920

FISHWAY LONG. PROFILE - 1

GOLD RAY DAM FISH LADDER

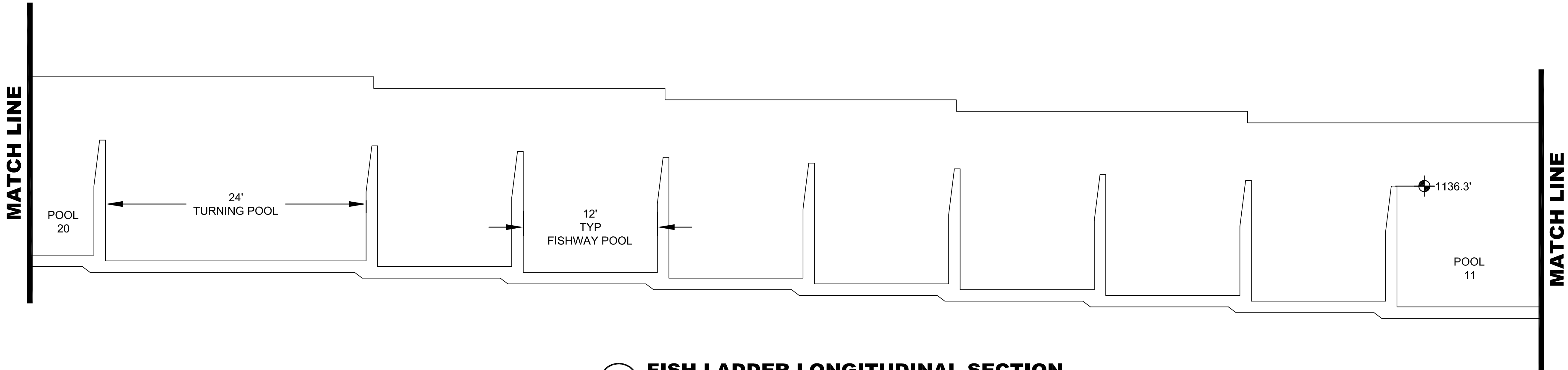
JACKSON COUNTY

NO.	DATE	BY	DESCRIPTION	CHK
0	01/06/10	CS	ENVR. ASSESSMENT	SW

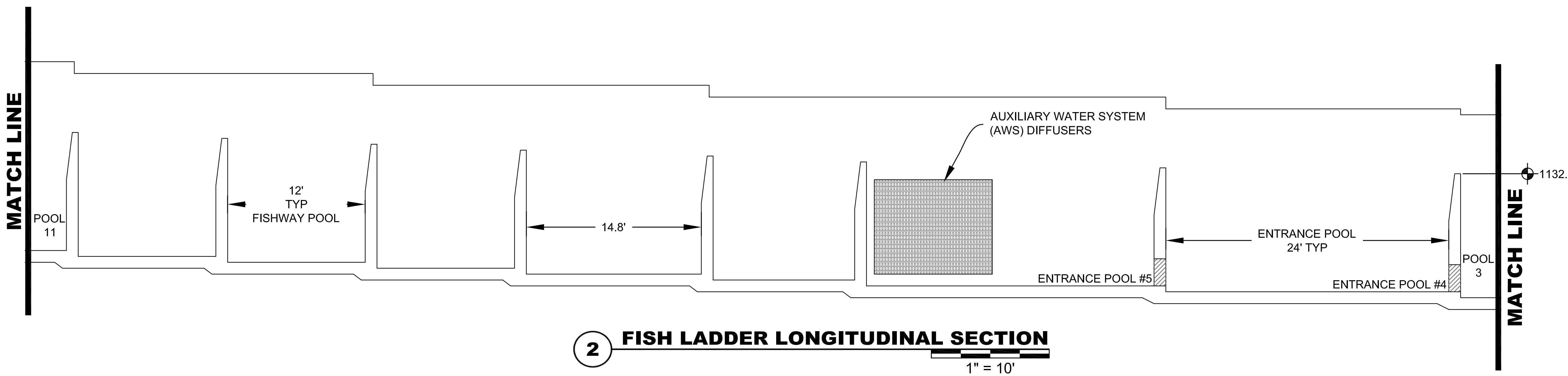
PROJECT NUMBER
RDG-09-048

DRAWING NUMBER
3.1

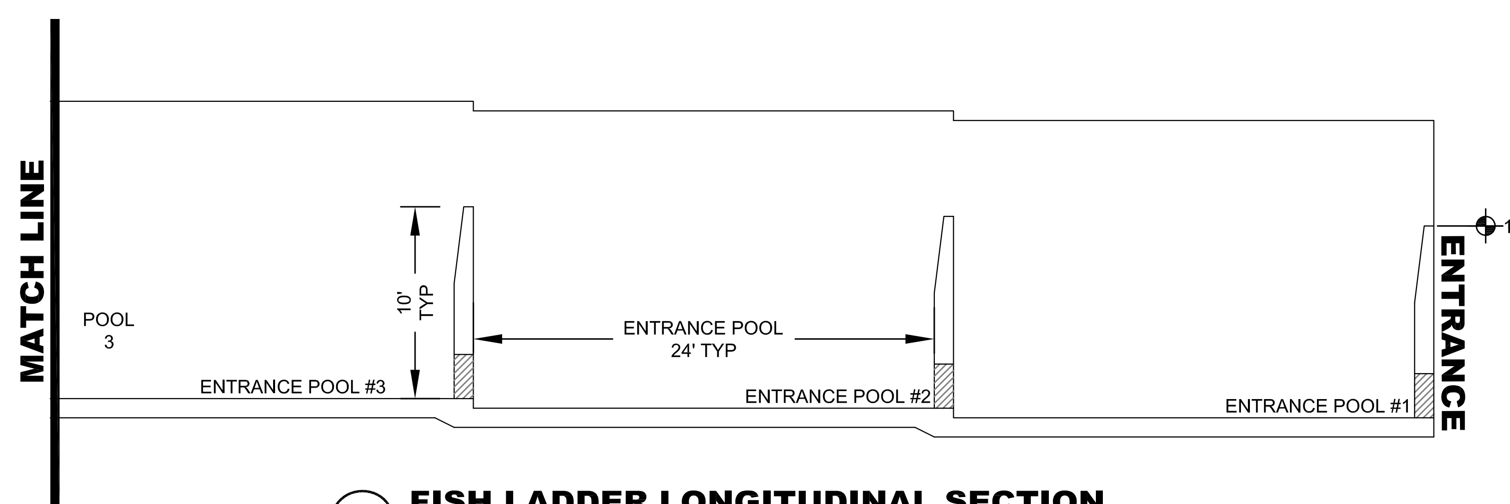
Drawing 4 of 6



1 FISH LADDER LONGITUDINAL SECTION
1" = 10'



2 FISH LADDER LONGITUDINAL SECTION
1" = 10'



3 FISH LADDER LONGITUDINAL SECTION
1" = 10'

RIVER DESIGN GROUP, INC.
5098 Hwy 93 South
Whitefish, MT 59937
406.862.4927
311 SW Jefferson Avenue
Corvallis, OR 97333
541.738.2920

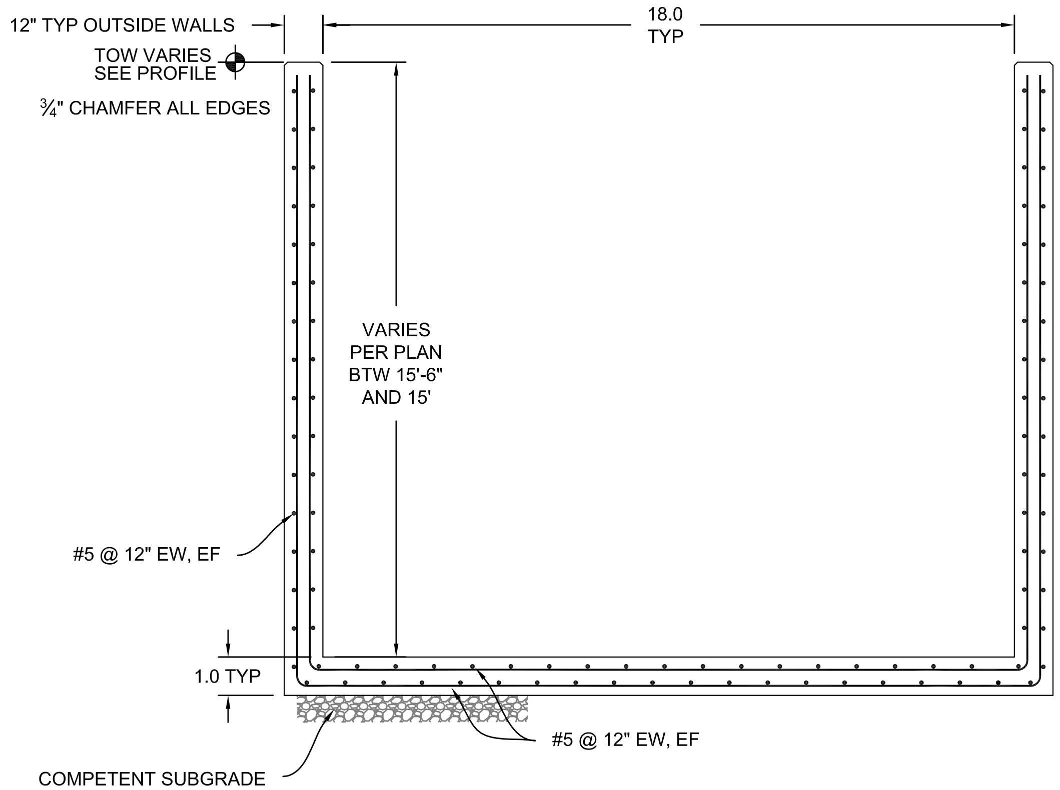
FISHWAY LONG. PROFILE - 2

GOLD RAY DAM FISH LADDER
JACKSON COUNTY

NO.	DATE	BY	DESCRIPTION	CHK
0	02/11/10	CS	ENVR. ASSESSMENT	SW

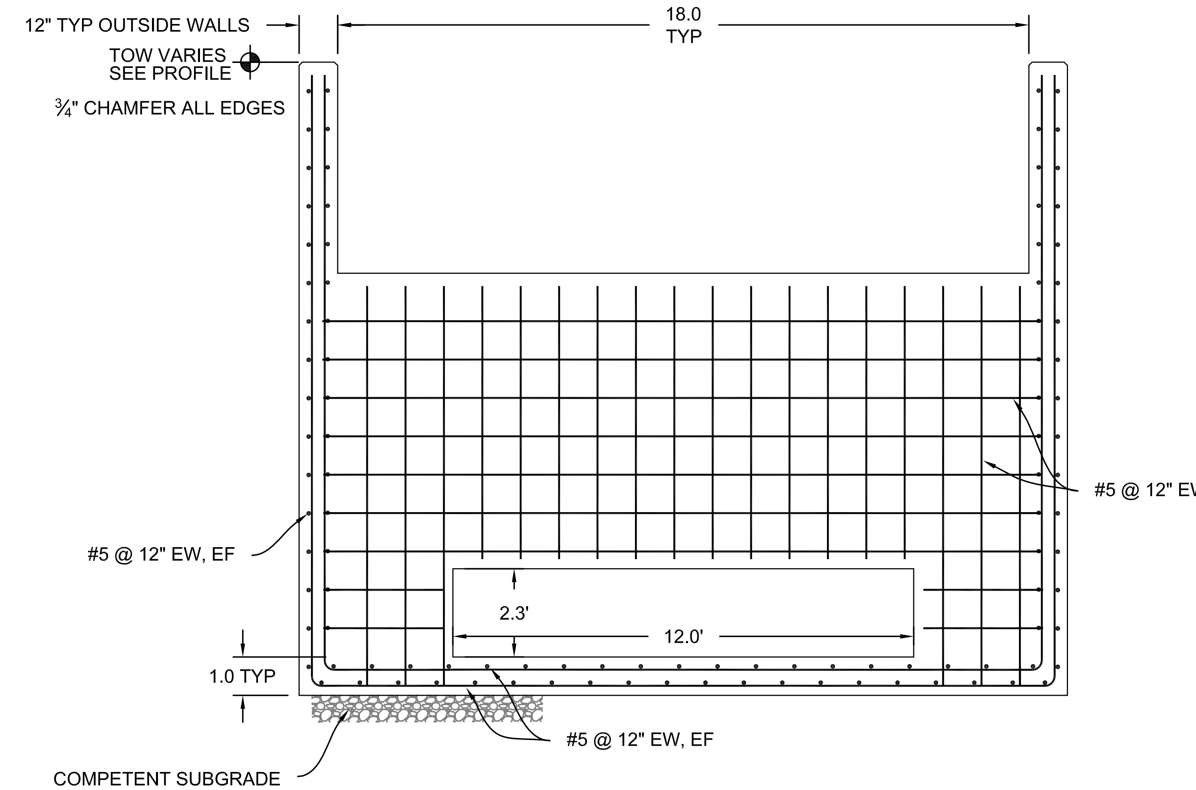
PROJECT NUMBER
RDG-09-048

DRAWING NUMBER
3.2



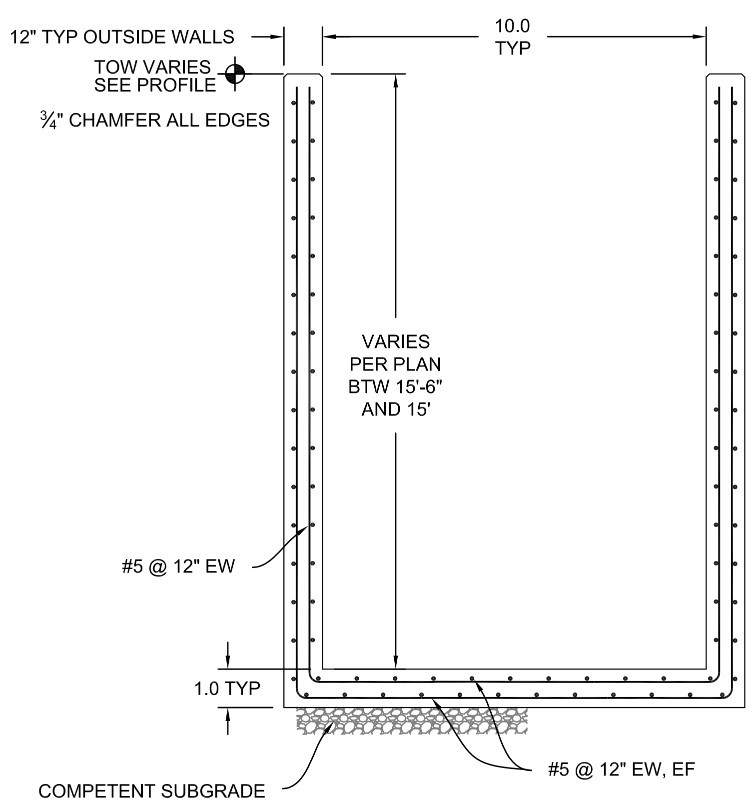
1 ENTRANCE POOL SECTION

1" = 5'



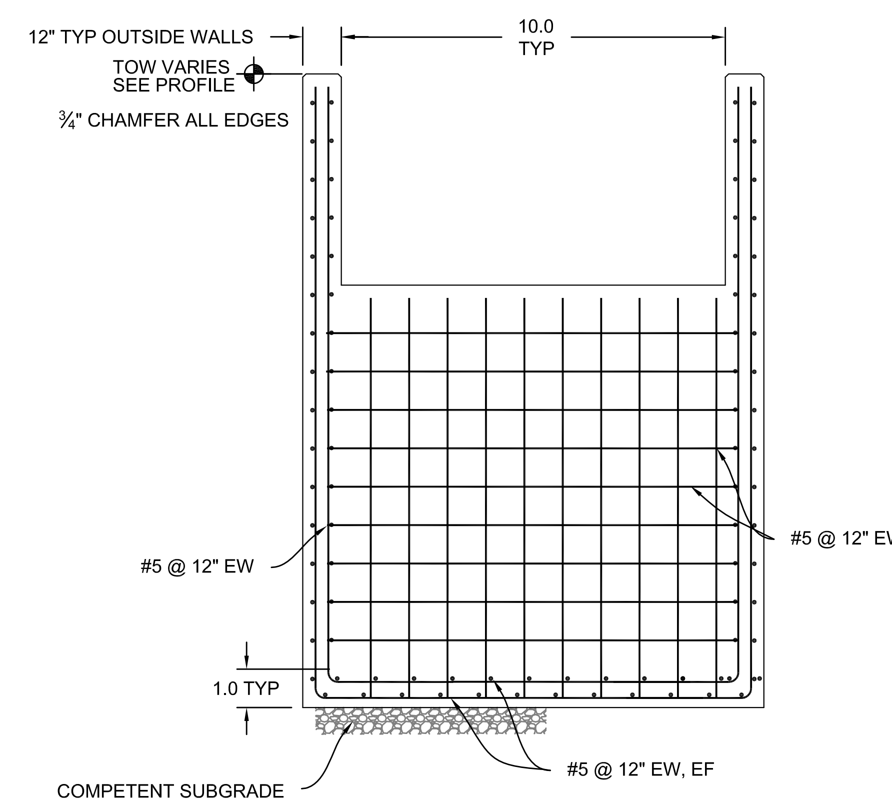
2 ENTRANCE WEIR SECTION

1" = 5'



3 FISH LADDER POOL SECTION

1" = 5'



4 FISH LADDER WEIR SECTION

1" = 5'

RIVER DESIGN GROUP, INC.
5098 Hwy 93 South
Whitefish, MT 59937
406.862.4927
311 SW Jefferson Avenue
Corvallis, OR 97333
541.738.2920

STRUCTURAL DETAILS

GOLD RAY DAM FISH LADDER

JACKSON COUNTY

NO.	DATE	BY	DESCRIPTION	CHK
0	02/11/10	CS	ENVR. ASSESSMENT	SW

PROJECT NUMBER RDG-09-048

DRAWING NUMBER 4.0

Drawing 6 of 6

Appendix B:
Data and Calculations

Project: Gold Ray Dam
Description: Fishway Option
Date: December 21, 2009

DRAFT

Broadcrested Coeff (Cw)	3.05	dimensionless
Weir Length	360	feet *
Elevation of Dam Weir	1149.90	feet
Fishway Entrance Weir Coeff	3.20	dimensionless
Fishway Entrance Weir Length	7.00	feet

** Dam weir is 367' minus 7.5' for aeration piers*

Fishway Weir Coeff	3.20	dimensionless
Fishway Weir Length	10.00	feet
Elevation of Weir	1149.80	feet
Fishway Pool Depth	10.00	feet
Fishway Pool Length	12.00	feet
Fishway Step Drop Height	0.50	feet

$$EDF = \frac{\gamma \cdot Q \cdot \Delta H}{Vol}$$

$$Q = C_w \cdot L \cdot H^{1.5}$$

Water Surface Elev. (ft)	Head on Dam (ft)	Dam Overflow Discharge (cfs)	Average Flow Velocity (ft/sec)	Head on Entrance (ft)	Fish Ladder Weir Flow (cfs)	Head on Ladder (ft)	Avg Velocity Over Fishway (ft/sec)	Avg Energy Dissipation Factor	Total Discharge (cfs)	Flow Significance (-)	AWS Flow (cfs)	Attraction Flow (cfs)	Attraction Flow (% of Q_{HFP})
1151	1.10	1,265	3.20	1.20	29	0.95	3.11	0.8	1294.4		185	214	4.9%
1151.02	1.12	1,300	3.23	1.22	30	0.96	3.14	0.8	1329.8		185	215	4.9%
1151.04	1.14	1,335	3.26	1.24	31	0.98	3.16	0.8	1365.5		185	216	4.9%
1151.06	1.16	1,370	3.28	1.26	32	0.99	3.19	0.8	1401.6	$Q_{95\%} = Q_{LFP}$	185	217	5.0%
1151.08	1.18	1,405	3.31	1.28	32	1.01	3.21	0.8	1437.9		185	217	5.0%
1151.1	1.20	1,441	3.34	1.30	33	1.02	3.24	0.9	1474.6		185	218	5.0%
1151.12	1.22	1,478	3.37	1.32	34	1.04	3.26	0.9	1511.5		185	219	5.0%
1151.14	1.24	1,514	3.40	1.34	35	1.06	3.29	0.9	1548.8		185	220	5.0%
1151.16	1.26	1,551	3.42	1.36	36	1.07	3.31	0.9	1586.3		185	221	5.0%
1151.18	1.28	1,588	3.45	1.38	36	1.09	3.34	0.9	1624.2		185	221	5.1%
1151.2	1.30	1,625	3.48	1.40	37	1.10	3.36	1.0	1662.3		185	222	5.1%
1151.22	1.32	1,663	3.50	1.42	38	1.12	3.39	1.0	1700.8		185	223	5.1%
1151.24	1.34	1,701	3.53	1.44	39	1.14	3.41	1.0	1739.5		185	224	5.1%
1151.26	1.36	1,739	3.56	1.46	40	1.15	3.43	1.0	1778.5		185	225	5.1%
1151.28	1.38	1,778	3.58	1.48	40	1.17	3.46	1.0	1817.9		185	225	5.2%
1151.3	1.40	1,816	3.61	1.50	41	1.18	3.48	1.1	1857.5		185	226	5.2%
1151.32	1.42	1,855	3.63	1.52	42	1.20	3.50	1.1	1897.4		185	227	5.2%
1151.34	1.44	1,895	3.66	1.54	43	1.21	3.53	1.1	1937.5		185	228	5.2%
1151.36	1.46	1,934	3.69	1.56	44	1.23	3.55	1.1	1978.0		185	229	5.2%
1151.38	1.48	1,974	3.71	1.58	44	1.25	3.57	1.2	2018.7		185	229	5.3%
1151.4	1.50	2,014	3.74	1.60	45	1.26	3.59	1.2	2059.7		185	230	5.3%
1151.42	1.52	2,055	3.76	1.62	46	1.28	3.62	1.2	2101.0		185	231	5.3%
1151.44	1.54	2,095	3.78	1.64	47	1.29	3.64	1.2	2142.5		185	232	5.3%
1151.46	1.56	2,136	3.81	1.66	48	1.31	3.66	1.2	2184.3		185	233	5.3%
1151.48	1.58	2,178	3.83	1.68	49	1.32	3.68	1.3	2226.4		185	234	5.4%
1151.5	1.60	2,219	3.86	1.70	50	1.34	3.70	1.3	2268.8		185	235	5.4%
1151.52	1.62	2,261	3.88	1.72	51	1.36	3.73	1.3	2311.4		185	236	5.4%
1151.54	1.64	2,303	3.91	1.74	51	1.37	3.75	1.3	2354.3		185	236	5.4%
1151.56	1.66	2,345	3.93	1.76	52	1.39	3.77	1.4	2397.4		185	237	5.4%
1151.58	1.68	2,388	3.95	1.78	53	1.40	3.79	1.4	2440.8		185	238	5.5%
1151.6	1.70	2,430	3.98	1.80	54	1.42	3.81	1.4	2484.5		185	239	5.5%
1151.62	1.72	2,473	4.00	1.82	55	1.43	3.83	1.4	2528.4		185	240	5.5%
1151.64	1.74	2,517	4.02	1.84	56	1.45	3.85	1.5	2572.6		185	241	5.5%
1151.66	1.76	2,560	4.05	1.86	57	1.47	3.87	1.5	2617.0		185	242	5.5%
1151.68	1.78	2,604	4.07	1.88	58	1.48	3.90	1.5	2661.7		185	243	5.6%
1151.7	1.80	2,648	4.09	1.90	59	1.50	3.92	1.5	2706.6		185	244	5.6%
1151.72	1.82	2,692	4.11	1.92	60	1.51	3.94	1.5	2751.8		185	245	5.6%
1151.74	1.84	2,737	4.14	1.94	61	1.53	3.96	1.6	2797.2		185	246	5.6%
1151.76	1.86	2,781	4.16	1.96	61	1.55	3.98	1.6	2842.9		185	246	5.6%
1151.78	1.88	2,826	4.18	1.98	62	1.56	4.00	1.6	2888.8		185	247	5.7%
1151.8	1.90	2,872	4.20	2.00	63	1.58	4.02	1.6	2935.0		185	248	5.7%
1151.82	1.92	2,917	4.23	2.02	64	1.59	4.04	1.7	2981.4		185	249	5.7%
1151.84	1.94	2,963	4.25	2.04	65	1.61	4.06	1.7	3028.1		185	250	5.7%
1151.86	1.96	3,009	4.27	2.06	66	1.62	4.08	1.7	3075.0		185	251	5.8%
1151.88	1.98	3,055	4.29	2.08	67	1.64	4.10	1.7	3122.1		185	252	5.8%
1151.9	2.00	3,101	4.31	2.10	68	1.66	4.12	1.8	3169.5		185	253	5.8%
1151.92	2.02	3,148	4.33	2.12	69	1.67	4.14	1.8	3217.1		185	254	5.8%
1151.94	2.04	3,195	4.36	2.14	70	1.69	4.16	1.8	3264.9		185	255	5.8%
1151.96	2.06	3,242	4.38	2.16	71	1.70	4.18	1.8	3313.0		185	256	5.9%
1151.98	2.08	3,289	4.40	2.18	72	1.72	4.20	1.9	3361.3		185	257	5.9%
1152	2.10	3,337	4.42	2.20	73	1.73	4.21	1.9	3409.9		185	258	5.9%
1152.02	2.12	3,385	4.44	2.22	74	1.75	4.23	1.9	3458.7		185	259	5.9%
1152.04	2.14	3,433	4.46	2.24	75	1.77	4.25	2.0	3507.7		185	260	6.0%
1152.06	2.16	3,481	4.48	2.26	76	1.78	4.27	2.0	3556.9		185	261	6.0%
1152.08	2.18	3,529	4.50	2.28	77	1.80	4.29	2.0	3606.4		185	262	6.0%
1152.1	2.20	3,578	4.52	2.30	78	1.81	4.31	2.0	3656.1		185	263	6.0%
1152.12	2.22	3,627	4.54	2.32	79	1.83	4.33	2.1	3706.0		185	264	6.0%
1152.14	2.24	3,676	4.56	2.34	80	1.84	4.35	2.1	3756.1		185	265	6.1%
1152.16	2.26	3,725	4.59	2.36	81	1.86	4.36	2.1	3806.5		185	266	6.1%
1152.18	2.28	3,775	4.61	2.38	82	1.88	4.38	2.1	3857.1		185	267	6.1%
1152.2	2.30	3,825	4.63	2.40	83	1.89	4.40	2.2	3907.9		185	268	6.1%
1152.22	2.32	3,875	4.65	2.42	84	1.91	4.42	2.2	3959.0	$Q_{10\%}=Q_{Juvenile}$	185	269	6.2%
1152.24	2.34	3,925	4.67	2.44	85	1.92	4.44	2.2	4010.2		185	270	6.2%
1152.26	2.36	3,975	4.69	2.46	86	1.94	4.46	2.2	4061.7		185	271	6.2%
1152.28	2.38	4,026	4.71	2.48	87	1.96	4.47	2.3	4113.4		185	272	6.2%
1152.3	2.40	4,077	4.73	2.50	89	1.97	4.49	2.3	4165.3		185	274	6.3%
1152.32	2.42	4,128	4.74	2.52	90	1.99	4.51	2.3	4217.4		185	275	6.3%
1152.34	2.44	4,179	4.76	2.54	91	2.00	4.53	2.4	4269.8		185	276	6.3%
1152.36	2.46	4,231	4.78	2.56	92	2.02	4.55	2.4	4322.3		185	277	6.3%
1152.38	2.48	4,282	4.80	2.58	93	2.03	4.56	2.4	4375.1	$Q_{5\%}=Q_{HFP}$	185	278	6.4%
1152.4	2.50	4,334	4.82	2.60	94	2.05	4.58	2.4	4428.1		185	279	6.4%
1152.42	2.52	4,386	4.84	2.62	95	2.07	4.60	2.5	4481.3		185	280	6.4%
1152.44	2.54	4,439	4.86	2.64	96	2.08	4.62	2.5	4534.7		185	281	6.4%
1152.46	2.56	4,491	4.88	2.66	97	2.10	4.63	2.5	4588.3		185	282	6.5%

Project: Gold Ray Dam
Description: Fishway Option - Entrance Pool Sizing
Date: January 6, 2010

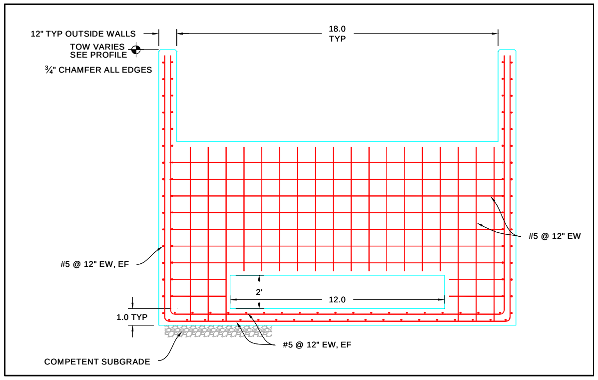
DRAFT

--> Follows 2008 NMFS Fish Passage Criteria

Max Flow Analysis

Inputs			Criteria			OK?
Q	278.00	cfs	Q	evaluated elsewhere		
△H	0.50	ft	△H	0.5	ft	OK
Pool Dimensions			Pool Dimensions			
Width	18.00	ft	evaluated elsewhere			N/A
Depth	10.00	ft	evaluated elsewhere			N/A
Length	24.00	ft	evaluated elsewhere			N/A
Orifice Dimensions			Orifice Dimensions			
Height	2.30	ft	Height	15	inches	OK
Width	12.00	ft	Width	12	inches	OK

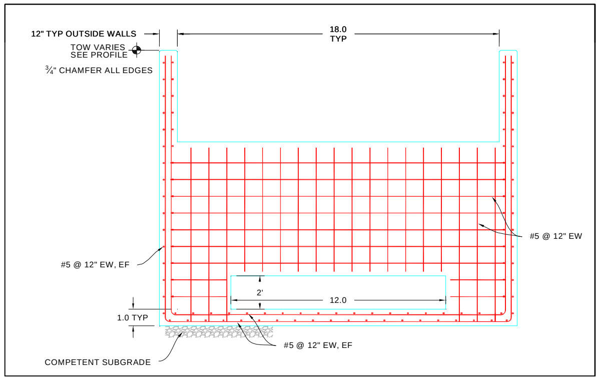
Results			Criteria			OK?
EDF	2.0	(ft-lbs/s) / ft3	max EDF	4	(ft-lbs/s) / ft3	OK
Orifice Flow			Orifice Flow			
C	0.6		N/A			N/A
Qorifice	156.6	cfs	N/A			N/A
Depth Over Weir			Min Depth Over Weir			
C	3.10		N/A			
H	1.7	ft	H	1	ft	look @ min flow
Velocity over Weir			Velocity over Weir			
V	4.0	ft/s		1.5 to 4	ft/s	NO



Min Flow Analysis

Inputs			Criteria			OK?
Q	217.00	cfs	Q	evaluated elsewhere		
△H	0.50	ft	△H	0.5	ft	OK
Pool Dimensions			Pool Dimensions			
Width	18	ft	evaluated elsewhere			N/A
Depth	10	ft	evaluated elsewhere			N/A
Length	24	ft	evaluated elsewhere			N/A
Orifice Dimensions			Orifice Dimensions			
Height	2.3	ft	Height	15	inches	OK
Width	12	ft	Width	12	inches	OK

Results			Criteria			OK?
EDF	1.6	(ft-lbs/s) / ft3	max EDF	4	(ft-lbs/s) / ft3	OK
Orifice Flow			Orifice Flow			
C	0.6		N/A			N/A
Qorifice	156.6	cfs	N/A			N/A
Depth Over Weir			Min Depth Over Weir			
C	3.1		N/A			
H	1.1	ft	H	1	ft	OK
Velocity over Weir			Velocity over Weir			
V	3.2	ft/s		1.5 to 4	ft/s	look @ max flow



Project: Gold Ray Dam
Description: Fishway Option - Quantity Estimate
Date: January 7, 2010

DRAFT

RIVER DESIGN GROUP, INC.

Fishway

Fishway Dimensions		
Weir Width	10	feet
Fishway Pool Depth	10	feet
Fishway Pool Length	12	feet
Fishway Step Drop Height	0.5	feet
Total Vertical Drop	**16.5**	feet
# pools	33	
Floor Thickness	**1**	feet
Wall Thickness	**12**	inches
Wall Height above Weir	**5.5**	feet

Concrete Volume Per Pool		
Floor Volume	143.0	ft^3
Weir Wall Volume	100.0	ft^3
Side Wall Volumes - 2 walls	403.0	ft^3
Concrete Volume per pool	646.0	ft^3
Concrete Volume per pool	**23.9**	**CY**

Total Concrete Volume		
Additional End Wall	3.7	CY
Volume for 33 pools	789.6	CY
Total Concrete Volume	**793.3**	**CY**

Entrance Pools

Fishway Dimensions		
Weir Width	18	feet
Fishway Pool Depth	10	feet
Fishway Pool Length	24	feet
Fishway Step Drop Height	0.5	feet
Total Vertical Drop	**2.5**	feet
# pools	5	
Floor Thickness	**1**	feet
Wall Thickness	**12**	inches
Wall Height above Weir	**5.5**	feet

Concrete Volume Per Pool		
Floor Volume	475.0	ft^3
Weir Wall Volume	180.0	ft^3
Side Wall Volumes - 2 walls	775.0	ft^3
Concrete Volume per pool	1430.0	ft^3
Concrete Volume per pool	**53.0**	**CY**

Total Concrete Volume		
Additional End Wall	6.7	CY
Volume for 5 pools	264.8	CY
Total Concrete Volume	**271.5**	**CY**

Combined Volume Estimate - Entire Structure		
Total Concrete Volume	**1065**	**CY**