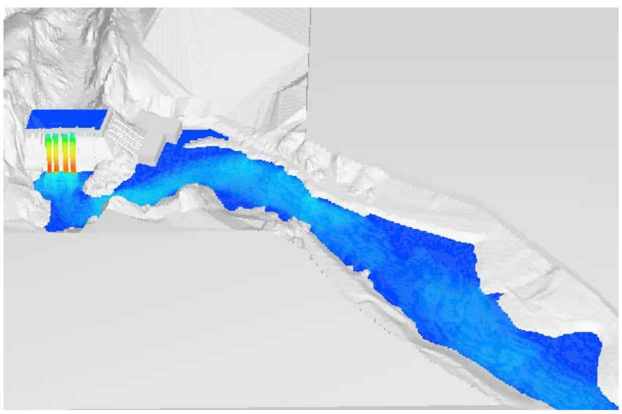
LONG LAKE HYDROELECTRIC
DEVELOPMENT (HED)
TOTAL DISOLVED GAS ABATEMENT
PHASE II FEASIBILITY STUDY

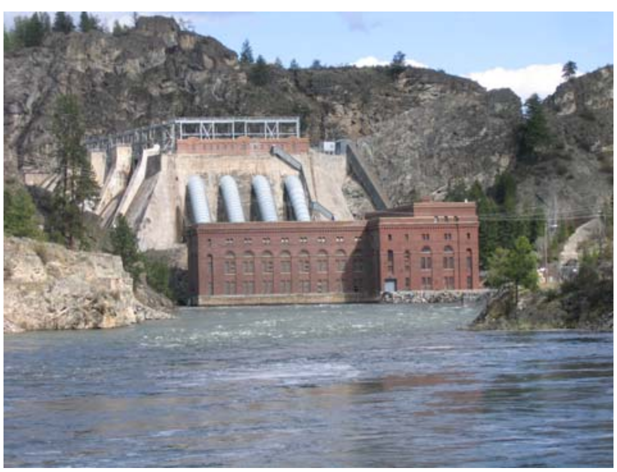
FINAL REPORT

August 18, 2010



Prepared for:

AVISTA Utilities



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EXECUTIVE SUMMARY

Northwest Hydraulic Consultants, Inc. (NHC) was contracted by Avista Utilities to conduct the Long Lake Hydroelectric Development (HED) Total Dissolved Gas Abatement Phase II Feasibility Study. McMillen, LLC (McMillen) was a subconsultant to NHC and provided civil and structural engineering, constructability, and cost estimating services. In 2006, Avista Utilities managed the Phase I TDG feasibility evaluation for Long Lake Dam. This Phase II evaluation was initiated to carry the most promising alternatives developed in Phase I forward to the next phase of design.

As a part of the relicensing effort, water quality studies were conducted for the Spokane River Project; and, the new license requires water quality monitoring and attainment plans. One component of the water quality studies completed during the relicensing process included a Total Dissolved Gas (TDG) evaluation at Long Lake Dam. Due to the physical characteristics of Long Lake Dam and the potential for high TDG levels, a TDG Water Quality Attainment Plan (WQAP) is required to fulfill the terms of the license.

Long Lake HED is a high head facility, and TDG levels downstream of the project can range from 120% to 140% during spill operation. Current state standards mandate that the TDG downstream of projects must not exceed 110% for flows less than the 7Q10 discharge at the project (where "7Q10" is the highest average seven consecutive day discharge with an average recurrence probability of 10 percent in any given year, commonly referred to as a 10 year frequency). The 7Q10 discharge for the Long Lake project corresponds to a total river discharge of 32,000 cfs, according to the Washington State Department of Ecology estimate. With the powerhouse operating at 6,800 cfs, the resulting 7Q10 for the spillway is 25,200 cfs.

The Phase II evaluation included the development of conceptual level designs including additional hydraulic analysis, civil design, cost estimates, constructability reviews, and drawings for five TDG alternatives from the Phase I study (EES, 2006). The alternatives under consideration include the following:

- Alternative 1 – Spill Bay 7-8 Deflectors
- Alternative 2 – Spill Bay 7-8 Super-elevated Spillway Extension
- Alternative 3 – Spill Bay 1-2 Toe Modifications and Downstream Deflector
- Alternative 4 – Cut-off Dam Chute Spillway with Deflector
- Alternative 5 – New Second Powerhouse

This hydraulic analysis for the Phase II study included CFD modeling of Alternatives 1 through 3. The CFD results were used to evaluate the hydraulic performance of the alternatives. At the end of the evaluation, a qualitative assessment of the advantages and disadvantages of each alternative were developed.

Alternatives 1 through 3 all include modifications to the existing spillway in an attempt to prevent plunging flow downstream of the spillway. Plunging flow draws air deep into the water column which leads to increased levels of TDG. These alternatives assume that all flow up to the 7Q10 would be diverted through only two of the eight total spill bays. Alternative 1 uses the deflector design concept that has been used on several Lower Columbia, Mid-Columbia, and Snake River Dams. For this alternative, a flow deflector would be added to Spill Bays 7 and 8, and the existing plunge pool below the bays would be filled with concrete. Alternative 2 is similar to a flip bucket concept with a curvilinear, super-elevated spillway chute extension to

deflect the flow towards the rock outcrop below Spill Bays 7 and 8 and adjacent to the left side of the spillway looking downstream. Alternative 3 includes the addition of a toe modification downstream of Spill Bays 1 and 2. The existing rock shelf below these bays would be modified to help minimize plunging flow by the addition of a flow deflector to the downstream end of the modified shelf.

Alternatives 4 and 5 include a new spillway and a new powerhouse alternative, respectively. Alternative 4 is comprised of an auxiliary chute spillway constructed downstream of the existing cut-off dam with a high velocity chute extending downstream to the tailrace below the powerhouse. The auxiliary spillway would be sized to pass the 7Q10 flow, with higher flows passed through the existing gated overflow spillway. Alternative 5 would include the addition of a new powerhouse located adjacent to the existing powerhouse. The new powerhouse would have a hydraulic capacity of 9,600 cfs and a generating capacity of 120 MW. Combined with the existing powerhouse, the total hydraulic capacity of the two powerhouses would be 16,400 cfs, equivalent to approximately 51 percent of the 7Q10 TDG design discharge.

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CREDITS AND ACKNOWLEDGEMENTS

The following individuals have contributed to the Long Lake Hydroelectric Development TDG Abatement Phase II Feasibility Study.

Mr. Hank Nelson is Avista's project manger and Mr. Ryan Bean is Avista's project engineer. Mr. Speed Fitzugh is the project manager for the implementation of Avista's Spokane River Project license articles, and directed the project in relation to the overall license. Mr. Guy Paul, Mr. Steve Fry, and Mr. John Hamill provided engineering input and project information on behalf of Avista.

Ms. Lisa Larson was Northwest Hydraulic Consultants' (NHC) project manager and principal in charge. Mr. Brian Hughes managed NHC's CFD modeling and was supported by Dr. Pepe Vasquez and Ms. Kara Hurtig. Mr. Jim Lencioni provided internal technical review for NHC; Mr. Andre Ball provided NHC's hydraulic engineering support; and, Mr. Ed Zapel provided technical guidance. Dr. Steve Wilhelms served as NHC's TDG expert and provided review throughout the project.

Mr. Mort McMillen managed the civil design aspects of the project and was supported by several individuals from McMillen, LLC, including Dan Axness, Jason Starner, Chris Boyd, and Joe Keller. Mr. Kim de Rubertis was McMillen's lead geotechnical engineer for the project.

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1 INTRODUCTION

In June 2009, Avista Utilities (Avista) received a new 50-year FERC license (#2545) for the Spokane River Project. This project consists of five hydroelectric developments (HED) including Post Falls HED in Idaho and Upper Falls, Monroe Street, Nine Mile, and Long Lake HEDs, located in Washington. As a part of the relicensing effort, water quality studies were conducted for the Spokane River Project, and the new license requires water quality monitoring and attainment plans. One component of the water quality studies completed during the relicensing process included a Total Dissolved Gas (TDG) evaluation at Long Lake Dam. Due to the physical characteristics of Long Lake Dam and the potential for high TDG levels, a TDG Water Quality Attainment Plan (WQAP) is required to fulfill the terms of the license.

1.1 PROJECT DESCRIPTION

The Long Lake HED is located at river mile 34, approximately 5 miles upstream of Little Falls Hydrolectric Project, and is the most downstream project of Avista's FERC licensed Spokane River Project. Figure 1-1 shows an aerial photograph of the Long Lake HED facility.

Figure 1-1: Aerial view of Long Lake HED

The facility was designed and constructed between 1910 and 1915 and includes a concrete gravity dam with a gated spillway section, a horizontal curving non-overflow gravity arch dam, referred to as the 'cut-off' dam, and a non-overflow powerhouse intake section. The ogee shaped spillway crest is 353 ft long and set at El. 1508 ft. There are eight spillway bays, numbered sequentially from Bay 1 at the east (right) end of the spillway to Bay 8 at the west (left) end of the spillway (adjacent to the powerhouse intakes). Each spillway bay is controlled by a 29 ft high by 25 ft wide vertical lift gate. Spill Bays 3 through 6 discharge into a deep plunge pool, while Spill Bays 1, 2, 7 and 8 discharge onto rock outcrops adjacent to the plunge pool area. The hydraulic capacity of each spillway bay is approximately 14,000 cfs at the normal pool elevation of 1536.0 ft. The tailwater level below the dam varies with flow, ranging between El. 1361.0 ft and El. 1378.0 ft . Approximately 300 ft downstream from the spillway plunge pool, flow from the spillway passes through two sharp bends before meeting with powerhouse flows in the downstream river channel.

The cut-off dam is located in a saddle along the left bank of the reservoir, approximately 600 ft upstream of the intake dam. The crest of the cut-off dam is at El. 1537.0 ft, and the total crest length is approximately 247 ft with a constant radius of 170 ft.

The non-overflow powerhouse intake section has a length of 148 ft as measured along the face of the headgate section. There is an additional 100 ft of length that connects the intake dam to the spillway and to the west abutment. The intake to the powerhouse is comprised of four steel penstocks that penetrate through the intake dam at El. 1499.0 ft. Vertical slide gates, 18 ft square, are located on the upstream face of the dam at the inlet to the penstocks. The penstocks extend along the downstream face of the dam and an exposed rock surface to the powerhouse, which contains four 17.9 MW double-runner horizontal Francis turbines and has a hydraulic capacity of 6,800 cfs.

1.2 PROJECT BACKGROUND

Long Lake HED is a high head facility, and TDG levels downstream of the project can range from 120% and 140% during spill operation. Current Washington State water quality standards specify that 'total dissolved gas measurements shall not exceed 110%' up to the 7Q10 discharge. In 2006, as part of the Federal Energy Regulation Commission (FERC) relicensing studies, Avista conducted an initial TDG Feasibility Study for the Long Lake HED (EES Consulting, 2006) which resulted in a recommendation of five TDG abatement alternatives for the project.

As part of the license implementation for this project, Avista is undertaking the Phase II TDG Feasibilty study as described in this report. The Phase II study includes more detailed evaluation and engineering for the five TDG abatement alternatives that were recommended for further evaluation in the 2006 study, as listed below:

- Alternative 1 – Spill Bay 7-8 Deflectors
- Alternative 2 – Spill Bay 7-8 Super-elevated Spillway Extension (previously referred to as Spill Bay 7-8 Deflectors with Training Walls)
- Alternative 3 – Spill Bay 1-2 Toe Modifications and Downstream Deflector (previously referred to as Spill Bay 1-2 Deflectors)
- Alternative 4 – Cut-off Dam Chute Spillway with Deflector (previously referred to as Cut-off Dam Spillway)

- Alternative 5 – New Second Powerhouse

The names of three of the alternatives have been revised slightly to better describe the type of hydraulic structure being evaluated. The concepts are similar to those presented in 2006; however, with more detailed engineering, the preliminary design of the structures has been refined.

1.3 SCOPE OF PHASE II STUDY

The purpose of this Phase II study is to further develop the five TDG abatement alternatives for the Long Lake HED as recommended in the 2006 Phase I feasibility study. To develop these alternatives in more detail, the following tasks were completed:

- Review and comment on the five alternatives selected in the Phase I study.
- Conduct additional hydraulic design analyses to further develop the alternatives.
- Conduct Computational Fluid Dynamics (CFD) analyses of Alternatives 1 through 3 to gain a better understanding of the hydraulics.
- Develop the preliminary civil design, geotechnical, and cost estimating components of each project.
- Compare the alternatives by utilizing a matrix analysis approach.

This report provides a detailed description of the development of these alternatives including the hydraulic design, civil engineering, geotechnical engineering, and cost/schedule estimating. CFD modeling was conducted as a part of the hydraulic design and analysis.

2 TDG ABATEMENT BACKGROUND

2.1 TDG DISCUSSION

During spill operations, elevated levels of TDG can be generated by the exposure of heavily aerated flow to high hydrostatic pressure within deep stilling basins or plunge pools. The hydrostatic pressure causes the absorption of atmospheric gases to concentrations above equilibrium at the local atmospheric pressure. Water will absorb (or desorb) gases until the pressure of the dissolved gas equilibrates with atmospheric pressure, at which point it is considered "saturated." In reality, water is rarely at equilibrium – rather it is either over-saturated (supersaturated) or under-saturated. When spilling, the flow can become supersaturated, often exceeding State and Federal water quality standards.

Current state standards mandate that the TDG downstream of projects must not exceed 110% for flows less than the 7Q10 discharge at the project (where "7Q10" is the highest average seven consecutive day discharge with an average recurrence probability of 10 percent in any given year). The 7Q10 discharge for the Long Lake project corresponds to a total river discharge of 32,000 cfs, according to the Washington State Department of Ecology estimate. With the powerhouse operating at its maximum capacity of 6,800 cfs, the resulting 7Q10 for the spillway is 25,200 cfs. The tailwater level at the spillway for the 7Q10 total discharge of 32,000 cfs is approximately El. 1373.0 ft as shown on Figure 2-1.

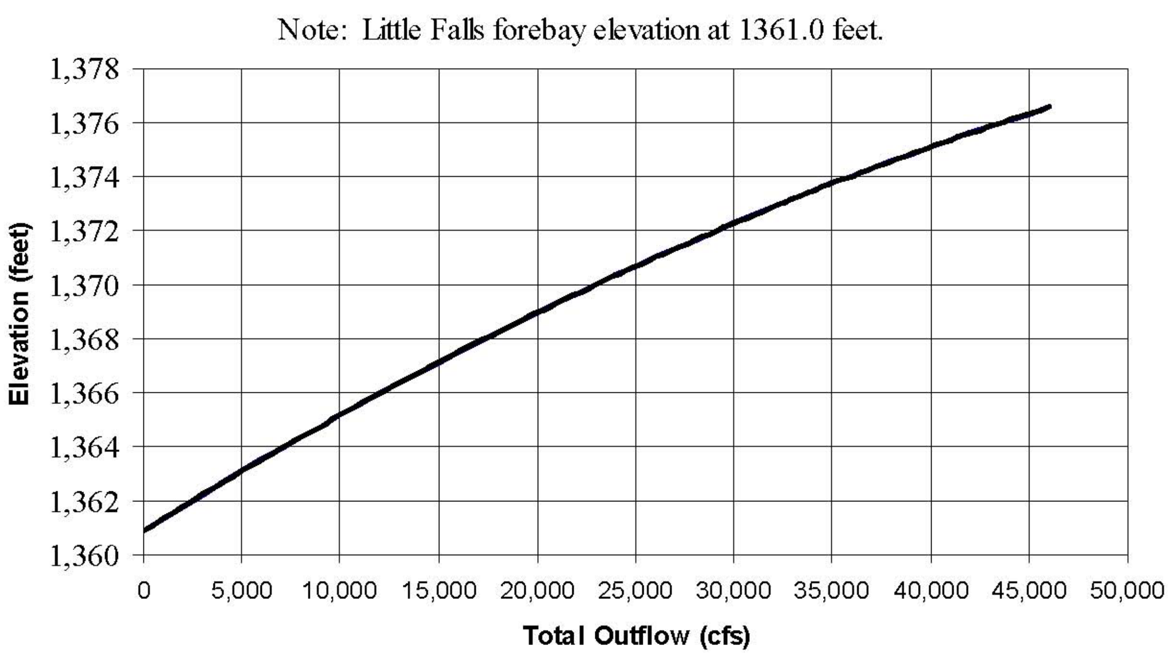

Figure 2-1: Graph of Tailwater Surface Elevation below Long Lake Dam

2.2 TDG Abatement Description

TDG abatement refers to the reduction of TDG levels downstream from hydroelectric projects. The main goal of TDG abatement design at spillways is usually to reduce the depth of plunging that occurs during spillway operation. If the depth of plunging and the amount of time that entrained air bubbles reside at depth can be reduced, the total amount of gas absorbed into the water will also be reduced. Therefore, preventing plunging flow is a key design element when considering TDG abatement alternatives. Operational changes can include changing the sequence of spillway bay operation to avoid discharging flow into deep areas downstream of the spillway, or spreading the flow across a larger portion of the spillway to reduce the unit discharges, which typically reduces the magnitude of plunging flow. However, operational solutions are often limited to large spillways with a high design capacity where there is the opportunity to reduce the unit discharge by operating additional bays up to the 7Q10 design flow.

The U.S. Army Corps of Engineers (USACE) has evaluated TDG abatement alternatives since the 1970's; and, in the 1990's, the USACE investigated numerous structural gas abatement alternatives (NHC, 1998). Those evaluations made use of physical models to develop spillway deflector design criteria that resulted in deflector flow patterns that significantly reduced plunging flow downstream of the spillway. Subsequent TDG testing at prototype projects where spillway deflectors have been installed confirmed that the deflectors are effective at reducing downstream TDG concentrations. To date, the USACE has implemented flow deflectors at numerous dams to deflect flow in a "skimming" or "undular" flow pattern to reduce plunging flow. As projects operated by public and private utilities have undergone the re-licensing process in the Northwest, additional TDG abatement evaluations have been undertaken. These evaluations have resulted in recommendations for both structural abatement alternatives (eg. flow deflectors) and operational changes.

In evaluating alternative deflector designs (typically with the aid of physical model studies), the USACE Engineering Research and Development Center (ERDC) has developed TDG flow classifications as shown in Figure 2-2, which are associated with increased or reduced chances of elevated TDG levels.

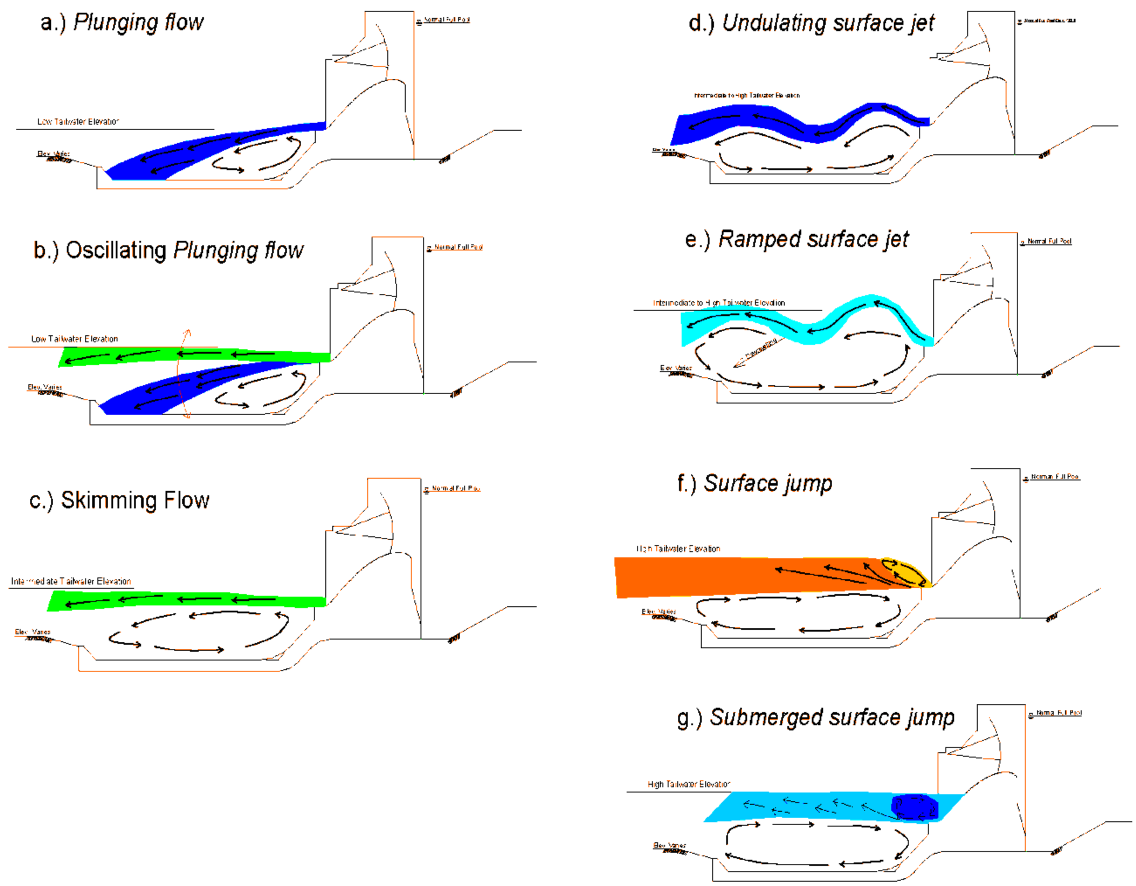

Figure 2-2: ERDC Flow Classifications

The “skimming” flow regime is the optimal flow regime for TDG reduction, and deflectors are typically designed to provide skimming flow at the 7Q10 flow. However, since the performance of the deflectors is sensitive to tailwater elevation and unit discharge, it is difficult, if not impossible, to design a stationary deflector that will produce a skimming flow regime over the full range of discharges up to the 7Q10 discharge. At lower discharges, other flow regimes, such as an undulating surface jet, are often considered acceptable.

Although deflectors have been successful in lowering TDG levels at numerous facilities, it should be noted that at several projects on the Lower Snake and Columbia Rivers, deflectors were unable to reduce TDG to 110 percent as mandated by State and Federal agencies. Furthermore, there are unique spillway configurations, such as Oxbow Dam on the Snake River, where the standard deflector design was not feasible. Long Lake HED is another example of a unique spillway and plunge pool configuration that requires a site specific design for an effective TDG abatement alternative.

2.3 LONG LAKE TDG GOALS

The goal of the Long Lake Phase II TDG Feasibility Study is to further develop five alternatives that will reduce TDG levels downstream of the project for flows up to the 7Q10 discharge. Three of the alternatives are based on incorporating modifications to the existing spillway to reduce plunging flow downstream of the spillway. The remaining two alternatives provide additional flow capacity, either through an auxiliary spillway or a new powerhouse, which reduces the discharge, and ultimately the TDG production at the existing spillway. The intent of the preliminary designs is primarily to develop an effective and practical design for each of the alternatives.

3 CFD ANALYSIS

Computational fluid dynamics (CFD) analyses were used to predict the hydrodynamic flow patterns downstream of the existing spillway for each of the selected alternatives. This information was used to assist in evaluating the potential TDG performance for the three alternatives that include modifications to the existing spillway.

3.1 CFD MODEL DESCRIPTION

A base model of the existing Long Lake spillway and downstream river channel was developed in the commercially available Flow-3D (Version 9.4) CFD model. The base model encompassed a portion of the forebay, the spillway, the plunge pool area, the confluence with the powerhouse tailrace, and approximately 2400 ft of the river channel downstream of the powerhouse. The downstream model limit was selected to include the tailrace TDG monitoring station (LLTR) used during Golder Associates' 2008 TDG field monitoring study (Golder, 2009), which included continuous TDG monitoring in the project forebay and tailrace from May 23 to July 10, 2008. These data included river discharges ranging from 6,500 cfs (July 10, 2008) to 40,000 cfs (May 23, 2008) and were used in selecting the discharges evaluated in the CFD model.

Figure 3-1 illustrates the approximate limits of the CFD model and the approximate location of the LLTR monitoring station.

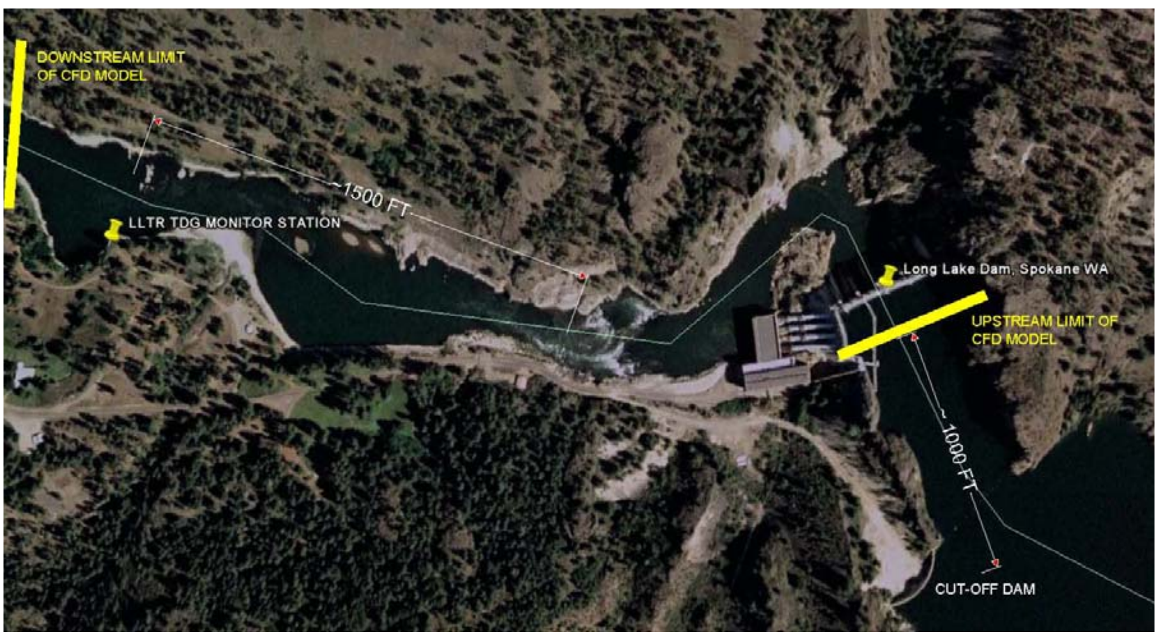

Figure 3-1: CFD Model Extents for Baseline Testing

The topography and bathymetry used in the model were developed using LiDAR and bathymetric data (July 2009) supplied by Northwest Hydro Inc. Dimensions for the structures, including the spillway, cut-off dam, and powerhouse, were based on information provided by Avista.

Due to the large area encompassed by the model, a coarse grid of approximately 10 ft x 10 ft in plan and 4 ft in elevation was used to model the dam, the plunge pool, the powerhouse tailrace and the downstream river channel. A finer nested grid (2 ft x 2 ft x 1 ft) was used to model flow through the spillway gates to the base of the spillway to increase the resolution around the gate and capture the flow patterns in better detail.

The types of data generated from the CFD model included water surface elevations, three-dimensional velocities, and simulated "air" particle tracking, which was used as a method of representing the behavior of entrained air in the plunge pool and downstream river channel. The diameter of every particle seeded into the model was set at 0.001 ft (0.012 inches) with a density of 0.00233 slugs/ft^3. These particle characteristics provided a rise velocity of 0.8 ft/s (0.25 m/s) in the CFD model, which has been empirically determined as the terminal velocity for air bubble radii between 0.001 and 0.03 ft (USDI, 1980). No adjustments were made to the density and viscosity parameters of the model fluid (water at 20 degrees Celsius) to simulate the exact temperature measured during the TDG field monitoring study, since the particle size in the CFD model was determined by the rise velocity in water at 20 degrees Celsius. The particles were released in multiple lines across the main flow stream upstream from the gates and at the base of the spillway where air is likely to be entrained into the flow. Particles were deleted by the model when they reached a void region such as the water surface or the model boundary limits.

At the completion of each test flow patterns, water levels and spillway flow trajectory characteristics were extracted from the model and used to compare the hydraulic characteristics of the modified spillway bays. The time history of the position of the "air" particles (x, y, z position), the particle velocity (three velocity components and resulting velocity magnitude) and water levels throughout the length of the model were also extracted from the model results

3.1.1 CFD Model Baseline and Validation Simulations

The base model was used to evaluate the spillway and tailrace hydraulics for the existing (unmodified) configuration, and as a basis of comparison to the hydraulic performance of the TDG abatement alternatives being evaluated.

The first baseline model test (Test 1) was conducted for the project's 7Q10 discharge, which occurred on June 2, 2008 during the TDG monitoring program. The hydrograph and TDG measurement data from Golder Associates' 2008 TDG field monitoring study (Figures 3.1-1 and 3.1-2; Golder, 2009) are included as Figures 3-2 and 3-3, below. The average TDG levels on this date were 123% in the forebay and 138% in the tailrace. The spillway operating conditions recorded for this date were used as the testing conditions for Test 1 (as outlined in Table 3-1).

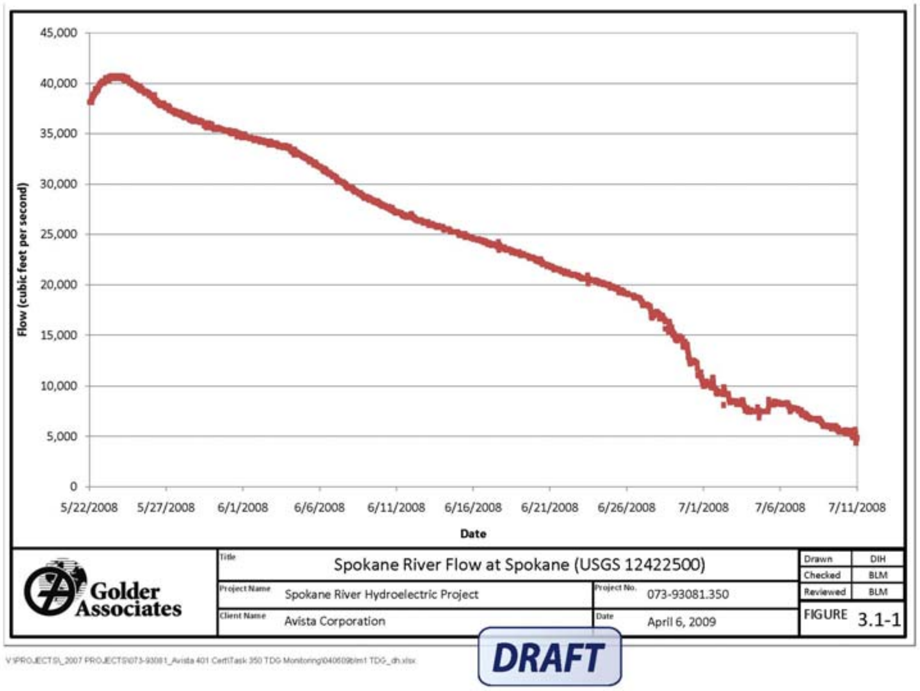

Figure 3-2: Spokane River Flow at Spokane USGS 12422500 (Golder, 2009)

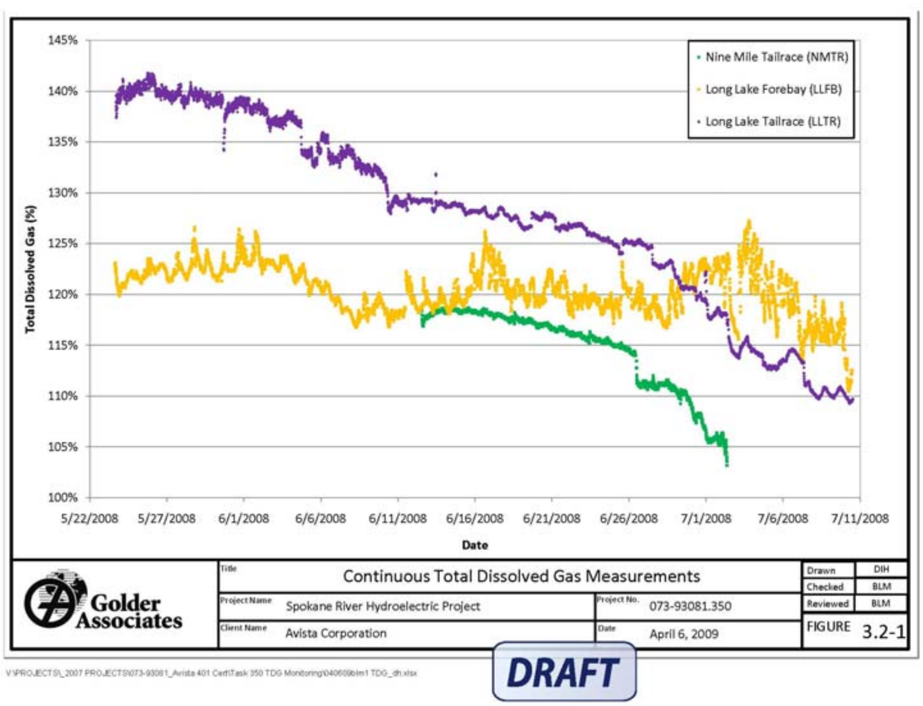

Figure 3-3: TDG field measurements (Golder, 2009)

Since the spillway will generally operate at flows below the 7Q10 discharge, the second model test considered a more common operating condition. As shown in Figure 3-3, the recorded TDG levels during the monitoring period were greater in the tailrace than in the forebay until midday on June 29, 2008 when the river discharge reached approximately 14,500 cfs. Based on this information, the second model test was conducted at the lowest discharge that still provided higher TDG levels in the tailrace than in the forebay. This corresponds to the conditions at noon on June 29, 2008 when the river discharge was approximately 15,300 cfs and the average TDG levels were 119% in the forebay and 121% in the tailrace. Details for the two test conditions are summarized in Table 3-1.

Table 3-1 CFD Base Model Testing Program

Test No.	Date	River Discharge (cfs)	Powerhouse Discharge (cfs)	Spillway Discharge (cfs)	Spillway Gate Openings (ft)	Forebay Water Level (ft)	Tailrace Water Level (ft)
1 (7Q10)	Jun 2, 2008	32,333	6,382	25,951	Gate 3: 10.5 ft Gate 4: 10.0 ft Gate 5: 10.0 ft Gate 6: 11.0 ft (gates set at 12:00)	El. 1533.1 (at 16:00)	El. 1373.2 (at 16:00)
2	Jun 29, 2008 (until noon)	15,312	6,904	8,408	Gate 5: 6.0 ft Gate 6: 6.0 ft (gates set at 5:00)	El. 1535.3 (at 8:00)	El. 1368.1 (at 8:00)

3.1.2 CFD Model Design Alternatives Simulations

The performance of each alternative was evaluated at the same two operating scenarios tested in the base model. The results of these two tests were used to assess TDG abatement performance of each alternative when compared to the existing conditions (base model).

An additional test was conducted for each alternative to evaluate hydraulic performance of the design at a higher spillway discharge. The maximum average discharge recorded during the 2008 monitoring period was 40,050 cfs, which occurred on May 23, 2008. Details of the third test condition that was used for evaluating the performance of the alternatives is summarized in Table 3-2, below.

Table 3-2 Additional Design Alternatives CFD Model Test

Test No.	Date	River Discharge (cfs)	Powerhouse Discharge[1] (cfs)	Spillway Discharge (cfs)	Spillway Gate Openings (ft)	Forebay Water Level (ft)	Tailrace Water Level (ft)
3	May 23, 2008	40,050	6,286	33,764	Dependent on design alternative	1533.25 (at 16:00)	1375.52 (at 16:00)

3.2 CFD MODEL BASELINE AND VALIDATION RESULTS

Figure 3-4 illustrates the existing spillway configuration as utilized in the CFD model. Figures displaying the CFD model testing results for this configuration are included as Appendix C of this report. The following section provides a summary of these results.

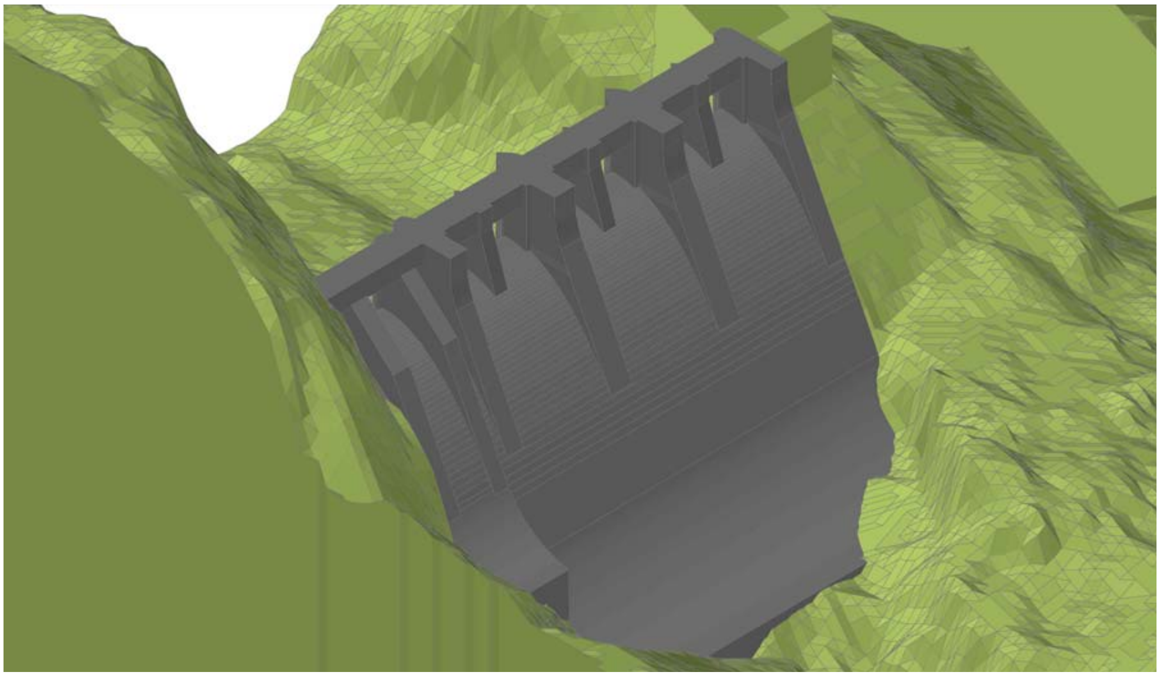

Figure 3-4: Schematic of existing spillway (baseline)

3.2.1 CFD Model Results

The CFD modeling results for the existing spillway configuration exhibited plunging flow at the base of the spillway for both operating conditions tested. Figures C-1 and C-2 (in Appendix C) illustrate the jet trajectory at the base of the spillway at a section through each operating gate. Baseline Test 1 was conducted with four spillway gates operating, while Baseline Test 2 had only two gates operating (in accordance with operating conditions followed for the project).

The spillway flow plunges to depths of more than 50 ft and the maximum flow velocities at the base of the spillway were approximately 85 ft/s for both operating conditions. The resulting unit discharge at each spill bay gate was approximately 260 cfs/ft for Baseline Test 1 and 180 cfs/ft for Baseline Test 2.

For Baseline Test 1, the four center gates (Gates 3 to 6) were operated with approximately equal gate openings (10 – 11 ft), which resulted in reasonably uniform flow across the width of the tailrace. For Baseline Test 2, the flow was concentrated through two gates (Gates 5 and 6), which resulted in areas of flow recirculation along both sides of the spillway plunge pool. These flow patterns are shown in Figures C-3 and C-4 in Appendix C.

3.3 TDG PREDICTIONS

3.3.1 TDG Predictive Model

The CFD model results were provided to Dr. Gulliver of the University of Minnesota for use in his Bubble Swarm Gas Transfer (BSGT) model (Gulliver et al. 2009), which was developed to predict TDG levels downstream of hydroelectric projects. The ability of the two models (CFD and BSGT) to predict the TDG level in the downstream river channel was assessed for spillway operation at two discharges (Baseline Tests 1 and 2). If the model TDG levels compared well to the TDG values measured in the field, the models would be considered as verified and could be used as one of the tools to estimate the TDG levels for the design alternatives. However, the combination of flow and bubble path prediction by the CFD software and the water-bubble mass transfer calculations of the BSGT model did not compare well to the field measurements for the Long Lake project. It is noted that these types of predictions are relatively innovative, and many factors could have contributed to the inconsistencies with the field data including model effects, the complexities of the Long Lake site layout, and even the location of the field measurements. The memorandum (Gulliver, July 2010) summarizing this work provides background on the TDG predictive model,

Efforts were made to investigate potential model effects on the predicted TDG levels such as the number of particles used in the simulations, the location where the particles were seeded, and the impact of activating the air modules within the CFD model. Sensitivity checks on the number of tracked particles between 48 and 100 particles showed no significant differences in the predicted TDG levels. Several tests were conducted with changes made to the particle initiation locations. Various combinations were investigated including: the headpond upstream from the spillway gates, an array of particles at base of the spillway, and particles placed at the junction where the spillway flow impacts the spillway plunge pool; however, the results were inconclusive. The CFD model was run with and without the air modules activated for simulation of air bulking and air entrainment, but again the results were inconclusive. As a result of the verification issues, the BSGT model was not pursued further for this site.

3.3.2 Particle Tracking Results

History particles were seeded in the CFD model upstream of the dam at locations 10, 20 and 30 ft upstream from the operating gates at El. 1510 ft and El. 1512 ft (spillway crest at El. 1508 ft) for all geometries tested. There were a total of 24 particles initiated upstream from the spillway gates. An additional 63 particles were seeded into the CFD model downstream from the spillway. For baseline testing, particles were initiated at the base of the spillway in the pool below the operating gates at El. 1340, 1350 and 1360 ft. Images showing the particle placement locations for these tests are included in Figures C-5 and C-6 in Appendix C. Particle locations after 120 and 240 seconds are shown in Figures C-7 and C-8 (Appendix C), respectively.

The particles were driven to depths of 60 ft or more for both tests with a significant quantity of the particles held at depths of greater than 17 ft for extended periods of time. Baseline Test 1 resulted in particles being driven approximately 10 ft deeper (70 ft depth) than Baseline Test 2, while Baseline Test 2 resulted in particles that remained at depth for a longer period of time (refer to Figure C-9 in Appendix C). The particle tracking information provides information on the depth of the particles and the length of time the particles remained at depth. Since high

TDG values are associated with particles being driven to depth for extended periods of time, the baseline particle tracks may be compared to the particle tracks associated with TDG abatement alternatives to provide additional information on how the alternatives modify the flow patterns and TDG levels.

3.3.3 TDG Qualitative Estimates

Qualitative estimates of TDG are based on the hydraulic conditions estimated for the structural abatement alternative and field testing results from existing TDG abatement structures. For each alternative, a qualitative TDG range is provided in Section 4.0. The CFD model discussed in Sections 3.0 and 4.0 provided the hydraulic characteristics of the alternatives and those results were used in the qualitative analysis.

4 DEVELOPMENT OF ALTERNATIVES

Five TDG abatement alternatives were evaluated in this feasibility study. The alternatives evaluated were limited to those that had been developed in Avista's 2006 Phase I TDG feasibility study (EES Consulting, 2006). The first three involve modifications to the existing spillway, the fourth alternative includes the construction of a new auxiliary spillway and the fifth alternative includes the construction of a new powerhouse. Appendix A includes the conceptual level drawings of the alternatives, and Drawing 10 provides a key plan showing the location of the alternatives.

4.1 ALTERNATIVE 1 – SPILL BAY 7-8 DEFLECTORS

Alternative 1 includes the addition of a continuous deflector on the downstream face of the spillway ogee below Spill Bays 7-8, as illustrated in Figure 4-1.

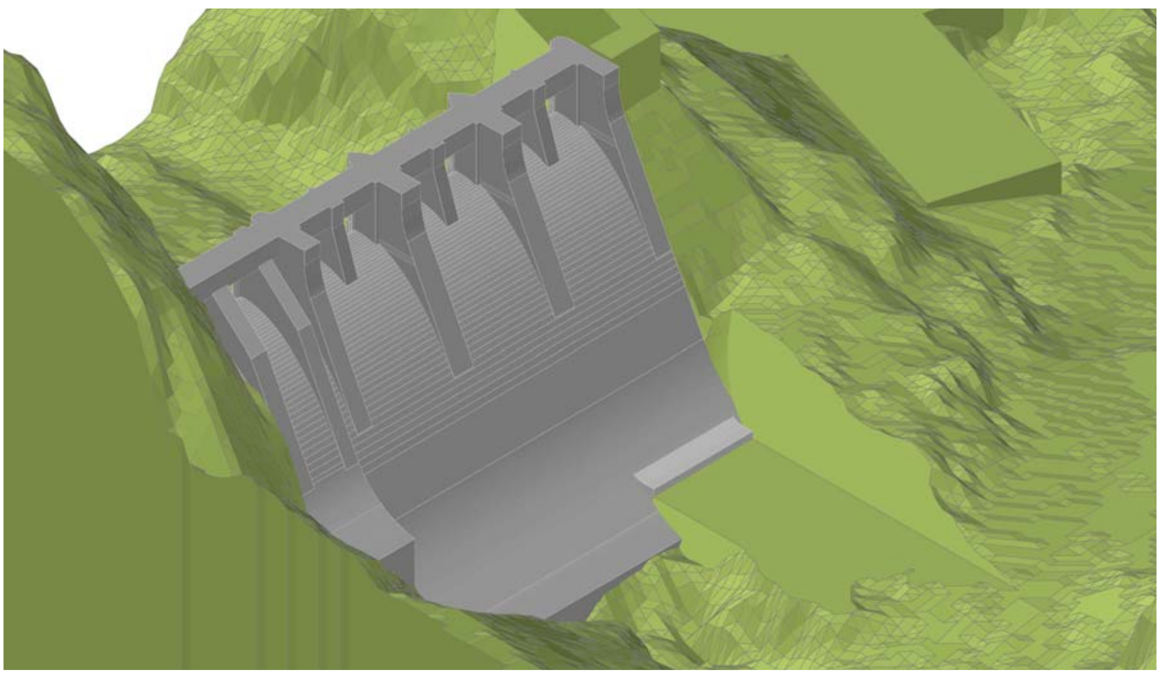

Figure 4-1: Schematic of Alternative 1

Drawings 100 through 104 (Appendix A) provide preliminary plan and sections of Alternative 1. As shown on Drawing 101, the deflector lip would be set at El. 1358 ft and a 5 ft radius curve would form the transition from the spillway face to the deflector. The deflector is 12.5 ft long from the point of intersection (PI) of the horizontal deflector elevation and the face of the spillway chute. The existing rock outcropping downstream of Bays 7 and 8 would be excavated down to El. 1353 ft (5 ft below the deflector lip elevation), and the area between the deflector and the excavated rock shelf would be filled and capped with concrete (described further in Section 4.1.4).

4.1.1 Hydraulic Analysis

Based on research conducted for the design of existing deflectors, there are certain design criteria and guidance that should be used for deflector designs. Deflector designs are sensitive to the unit discharge, tailwater elevation, and the deflector design characteristics including elevation, length, and transition to the existing spillway chute. Past experience with deflectors on relatively high head spillways (high energy flow exiting from the deflector) has shown that the desirable stable flow regime associated with acceptable TDG abatement can not typically be attained with unit discharges in excess of about 200 - 250 cfs/ft unless the submergence of the deflector (tailwater depth above the deflector) exceeds about 20 ft. However, tailwater depths on the deflector should generally be limited to 15 ft or less to minimize the hydrostatic pressures in the flow and prevent forcing entrained air into solution.

The 7Q10 discharge for the Long Lake spillway is estimated at 25,200 cfs and the 7Q10 tailwater level is estimated at El. 1373. If this flow is passed through only two spillway bays, the resulting unit discharge will be between 300 and 500 cfs/ft, depending on how the jet expands downstream of the piers and onto the deflector. The deflector submergence required to produce acceptable TDG abatement for unit discharges in this range exceeds the recommended value of 15 ft. Therefore the feasibility of developing a fully effective deflector at Bays 7 and 8 for the 7Q10 design flow is questionable. Given these constraints, a preliminary submergence value of 15 ft was used to set the elevation at El. 1358 ft for the initial deflector design for this alternative. Installation of deflectors on two additional bays would reduce the TDG design unit discharge to 150-250 cfs/ft, which is in accordance with previous deflector designs that have proven to provide acceptable TDG reduction conditions. This should be considered as a possible refinement to the design, but physical modeling will be required to verify the maximum discharge for acceptable deflector performance.

In addition to the deflector elevation, there have been many studies conducted to evaluate the optimum length of deflectors. These studies have demonstrated that 12.5 ft long deflectors perform well; however, extended deflectors have also been shown to perform well in model studies. The radial transition connecting the deflector to the face of the spillway chute can influence the performance of the deflector and is usually verified in a model study. However, a transition radius of 15 ft is a standard used for spillways and was used as a starting point for the Alternative 1 design. In addition to the references listed in Section 10, flow performance curves for the Wanapum Dam and Chief Joseph Dam were used for reference.

The existing rock outcropping downstream of Bays 7 and 8 would be excavated down to El. 1353 ft and the area between the deflector and the rock bench would be filled and capped with concrete to provide a relatively shallow tailrace downstream of the deflector. With a shallow basin and tailrace, aerated flow cannot plunge to depth, thereby limiting TDG absorption (Schneider and Wilhelms 1996). However, by filling in the plunge pool area downstream of Bays 7 and 8, the hydraulic conditions in the tailrace channel will be significantly impacted which could affect the spillway's ability to safely pass the PMF discharge. In the existing configuration, a significant amount of energy will be dissipated in the plunge pool area between the toe of the spillway and the rock outcropping. By comparison, with the proposed design the spillway flow will be re-directed across the new filled area and the excavated rock shelf, resulting in much less energy dissipation and hence higher energy flow entering the downstream channel. The ability to develop a tailrace design that can withstand the hydraulic conditions associated with these flow conditions is not a trivial matter. It is anticipated that a physical model will be required to evaluate the potential impacts and develop an acceptable design.

4.1.2 CFD Modeling

Alternative 1 was evaluated at the same two operating conditions that were examined for the existing (baseline) geometry; however, since the proposed modifications are restricted to two bays, Tests 1 and 2 were conducted with only spillway gates 7 and 8 operating. Test 3, which is above the 7Q10 discharge, had two additional gates (4 and 5) operating. The resulting unit discharge at each spill bay was approximately:

- 480 cfs/ft for Test 1;
- 180 cfs/ft for Test 2; and,
- 480 cfs/ft through gates 7 and 8 plus 220 cfs/ft through gates 4 and 5 for Test 3.

The CFD modeling results showed that the deflectors designed for spill bays 7 and 8 reduced the depth of the plunging flow by approximately 15 to 20 ft from that observed for the baseline conditions. Figures C-10 to C-12 (Appendix C) illustrate the flow patterns at the base of the spillway through gates 7 and 8 for the three tests conducted for Alternative 1. As shown in these images, the pool below spill bays 7 and 8 is relatively shallow – in the order of 15 to 25 ft. Maximum flow velocities at the base of the spillway were approximately 85 ft/s for Tests 1 and 2, and reached 88 ft/s for Test 3.

For Tests 1 and 2, the flow was concentrated through gates 7 and 8, resulting in a large area of flow recirculation on the right side of the pool, as shown in Figures C-13 and C-14. This area of flow recirculation remained for Test 3, but was reduced in size and strength as a result of operating gates 4 and 5. This is evident in Figure C-15, which illustrates the depth averaged velocities and flow streamlines for Alternative 1 Test 3.

Similar to baseline testing, history particles were initiated upstream from the operating gates and at the base of the spillway below the operating gates. These particles simulate bubble travel paths and give an indication of the potential resulting TDG levels for the design Images showing the particle paths for these tests are included in Figures C-16 and C-17 in Appendix C.

4.1.3 TDG Performance Estimates

Alternative 1 is similar in concept to the spillway deflectors located on the Lower Snake, Mid-Columbia River, and Lower Columbia River projects. However, with the two-bay design concept, the design unit discharge for Alternative 1 at Long Lake is significantly greater than that at which the Lower Snake and Columbia River projects have proven to be successful; and, the receiving pool at Long Lake is significantly shallower. The actual TDG reduction at any project depends on site specific factors. The TDG reduction benefits with deflectors are highly influenced by the unit discharge, stilling basin or receiving pool depth, and hydraulic conditions and flow depth between the spillway and the measurement location downstream. The deflectors installed at the Lower Snake and Columbia River projects have been shown through field testing to reduce TDG levels to the 120% to 125% range just downstream of their stilling basins. The higher unit discharge at the TDG design discharge condition with the 2-bay concept at Long Lake will likely produce less reduction than the Snake / Columbia River projects; however, the shallower receiving pool depth will likely tend to increase the reduction potential as compared to the Snake and Columbia River projects. Thus, it is expected that Alternative 1 has a very reasonable potential of reducing TDG levels at Long Lake to the 120% to 125% range, similar to that at the Snake and Columbia River projects. Furthermore, it may be possible to generate an even higher reduction if deflectors were installed on more than 2 bays in order to reduce the unit discharge.

4.1.4 Civil and Structural Design

For Alternatives 1, 2 and 3, new concrete structures will be constructed within and downstream from the existing spillway. The extent and complexity of the new structures ranges from relatively simple for the Alternative 1 deflector to more extensive and complex structural concrete sections for Alternatives 2 and 3. For all three alternatives, a portion of the existing spillway concrete will have to be removed down to bedrock to provide a suitable foundation for the new concrete structures. Careful attention to the joint detail between the existing and new concrete will be required to ensure long term durability during high energy flow conditions.

Alternative 1 has the least impact to the existing concrete spillway due to the location of the spillway deflector at the toe of the existing spillway. The lower 40 feet of the spillway toe will need to be removed down to bedrock. This section will provide a suitable vertical joint between the new concrete flow deflector and the existing concrete spillway. Anchors will be drilled and placed between the new and old concrete as well as into the rock foundation and abutments. The anchors will tie the structures together and serve to resist shear and moment forces. The rock outcropping downstream from flow deflector will be excavated down to approximately El. 1353.0 ft. The excess rock will be placed along the north bank of the tailrace. A structural concrete fill section will be required in the tailrace area to fill in the low point between the flow deflector and the excavated rock outcropping to prevent erosion from occurring at the toe of the new flow deflector.

Structural Evaluation

The assumed hydraulic parameters for design purposes of Alternatives 1 to 3 are given in Table 4-1 below.

Table 4-1 Hydraulic Parameters for Alternatives 1 to 3

Hydraulic Parameter	Value
Flow Volume, Q	400 cfs/ft
Flow Velocity, V	80 ft/s
Drag Coefficient, Cd	0.005

A constant and uniform flow is assumed to exist across the operating spillway bays and the resulting loading is assumed to be uniform across the deflector. These assumptions are considered to be reasonable for a conceptual level analysis and provide a reasonable basis for the comparison of the alternatives.

The thrust and drag forces were computed to evaluate the structural stability of the proposed structures for Alternatives 1 through 3. The thrust forces were predicted based on the design approach provided in the Fundamentals of Fluid Mechanics by Gerhart and Gross. The equation used for estimating thrust is given below:

$$F = \rho * Q * (V2 - V1)$$

Where,

ρ = Fluid Density

Q = Flow volume, cfs

V = Flow Velocity, ft/s

The associated drag force was estimated based on the equation below:

$$F = (Cd * \rho * V^2)/2$$

Where,

Cd = Drag Coefficient

ρ = Fluid Density

V = Flow Velocity, ft/s

The concrete chute of the existing spillway is showing signs of significant scour and abrasion. Concrete technology has improved significantly since the original dam and spillway were constructed, including the development of new High Performance Concrete (HPC) mix designs which provide increased strengths, increased density, decreased permeability, and superior wearing characteristics. A 12-inch HPC topping slab is included in each of the spillway modification alternatives to provide added durability and longevity for the new structure.

Alternative 1 Structural Design

Spill Bays 7 and 8 currently spill onto a rock outcropping at the base of the spillway. This alternative includes the construction of a spill deflector beneath Spill Bays 7 and 8 and excavation of a portion of the rock outcropping. The spill deflectors would be constructed by removing a large section of the rock bluff and a portion of concrete at the base of the spillway, followed by construction of the new deflectors and the deflector slab using mass concrete.

The impact of the falling water on the base of the spillway would impart a thrust force down into the bedrock foundation and back against the mass of the dam, while the hydraulic drag will act to pull the block away from the dam. The thrust and drag forces computed for Alternative 1 are shown in Table 4-2 below.

Table 4-2 Alternative 1 Load Summary

Parameter	Force
Vertical Thrust	3325 kips
Longitudinal Thrust (into the dam)	1310 kips
Hydraulic Drag (away from the dam)	-55 kips
Max Vertical Bearing Pressure	3300 psf
Friction from Dead Load	1280 kips

The horizontal thrust force is in the opposite direction from the drag force; therefore, the resultant horizontal force will be acting into the face of the dam, stabilizing the new section of concrete placed for the flow deflector.

Based on a review of previous geotechnical evaluations, the estimated bedrock bearing capacity is approximately 30,000 psf. Assuming a factor of safety of 3, the allowable bearing capacity is estimated to be approximately 10,000 psf, which is significantly higher than the estimated bearing pressure given above.

This alternative provides an overall increase of the concrete mass placed at the toe of the spillway. The increased mass will add to the dam's ability to resist sliding and overturning.

Based on the conceptual analysis, the weight and friction from the added concrete is sufficient to resist the resulting hydraulic loads. In order to account for secondary force effects, this preliminary design assumes that the new concrete will be anchored to the existing spillway using grouted reinforcing steel dowels on the vertical and horizontal faces.

4.2 Alternative 2 - Spill Bay 7-8 Super-elevated Spillway Extension

Alternative 2 is comprised of a super-elevated extension to the spillway chute below Bays 7 and 8, as illustrated in Figure 4-2. The extension is designed to redirect and spread the spillway flow onto the rock outcropping located downstream of these bays. The rock outcropping would help dissipate energy and prevent flow from plunging into the deep plunge pool. Drawings 200 through 204 (Appendix A) show a preliminary plan view and sections of the proposed extension. At the terminus of the super-elevated extension, a flip bucket will be incorporated into the design to help redirect and expand the jet in an upward direction to spread the jet onto the rock outcropping (the design details of the flip bucket are not shown on the Appendix A drawings and would be developed at the next level of design).

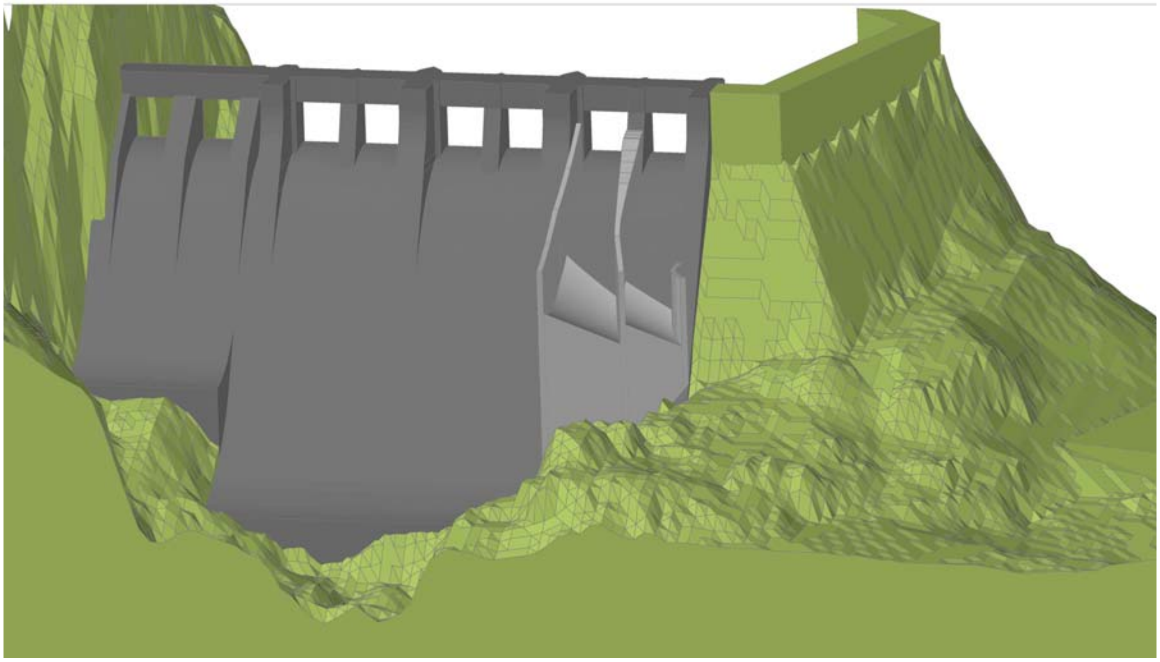

Figure 4-2: Schematic of Alternative 2

This concept is very similar to the designs presented in the 2006 report referred to as 'Spill Bay 7-8 Deflectors with Training Wall'. However, since 'flow deflectors' generally refer to submerged deflectors that conform to the USACE guidelines (as discussed in Section 4.1), this alternative has been renamed to eliminate any confusion between the super-elevated spillway extension that terminates well above the tailwater elevation and a standard 'spillway deflector'.

4.2.1 Hydraulic Analysis

The super-elevated spillway extension would redirect the flow toward the rock outcropping located downstream of Bays 7 and 8. Incorporating a flip bucket terminal structure would direct the high velocity jet into the air where the flow would spread and dissipate. This structure is unique since it includes a complex turning structure, a super-elevated chute and a flip bucket at its downstream end. While there are design guidelines for curved chutes, super-elevated chutes and flip bucket structures, combining all three features into a single design results in a very complex analysis. It is anticipated that a physical model would be required to refine and confirm the performance of this design if it is carried forward to the next level of design.

The chute extension would commence tangent to the spillway downstream of spill gates 7 and 8, then transition into a westward curve that will guide the flow around the base of the existing spillway piers. The flow from Bay 8 would travel through a bend of approximately 35 degrees to the west, and the flow from Bay 7 would travel along a larger radius bend of approximately 22 degrees to the west. Guide walls on the sides of each of the curving chutes would be required to constrain and guide the flow. The spillway extension would begin between El. 1495.8 ft and 1445.7 ft in Bay 8 and between El. 1490.1 and 1460.3 ft in Bay 7. Due to the complex shape of the super-elevated chute, the vertical radius will vary across the chutes from approximately

25 ft at the inside (west) edge of Bay 8 to approximately 71 ft at the outside (east) edge of Bay 7.

Preliminary estimates indicate that the flow trajectory will impact the rock outcropping approximately 100 ft downstream of the flip bucket in the vicinity of the El. 1400 ft contour. From this point, the flow will cascade down the sloping face of the rock outcropping and re-enter the downstream river channel. Given the uncertainty in these estimates, the location of the impact zone must be confirmed in a physical model study to ensure that the flow is impacting on the rock promontory versus the river channel or far river bank. The proposed design does not include any re-grading or excavation of the rock outcropping; however, options to re-grade the surface could be considered pending physical model study results.

Other concerns that should be evaluated in a physical model include the potential for downstream erosion of the rock outcropping and downstream river channel. Modifications to the design can be made to ensure that the jet is directed in an acceptable location. The Richard Russell spillway, which is located on the Savannah River and is owned and operated by the USACE, provides an example of a flip bucket design that caused severe erosion downstream. The flip bucket worked well for energy dissipation; however, the impact of the flow trajectory on a mountain slope that was located downstream of the east abutment was not considered during the design, and significant erosion was generated.

4.2.2 CFD Modeling

Alternative 2 was evaluated for the same operating conditions as Alternative 1. Tests 1 and 2 were conducted with spillway gates 7 and 8 operating, while Test 3 had two additional gates (4 and 5) operating. The resulting unit discharge at each spill bay was approximately:

- 480 cfs/ft for Test 1;
- 180 cfs/ft for Test 2; and,
- 480 cfs/ft through gates 7 and 8 plus 290 cfs/ft through gates 4 and 5 for Test 3.

The CFD modeling results showed that the spillway flow trajectory from the super-elevated spillway extension reached the rock outcrop for all three operating conditions. Figures C-19 to C-21 in Appendix C show an isometric view of the jet trajectory for each test. These images show that most of the flow travelled downstream over the rock outcrop and entered the river on the downstream side of the bend, away from the plunge pool at the base of the spillway. Figures C-22 to C-24 show the depth-averaged flow velocities and streamlines for the three operating conditions evaluated in the model, and Figure C-25 illustrates the velocity magnitude at a section through the rock outcrop. These images show that the rock outcrop dissipates the high velocity jet prior to the flow re-entering the river channel and reduce the depth of plunge to approximately 30 ft.

For Alternative 2, the particles were initiated upstream from the operating gates and at the base of the rock outcrop, both upstream in the spillway pool and downstream in the river channel. Images showing the particle paths for these tests are included in Figures C-26 and C-27 in Appendix C.

4.2.3 TDG Performance Estimates

Alternative 2 is similar to a flip bucket where the flow trajectory is spread into the air and the unit discharge at impact becomes quite low as a result of the flow spreading. These types of directional flip buckets have a high potential to significantly reduce TDG levels. With the Alternative 2 design at Long Lake the intent of the design is to create a condition where the jet trajectory impacts on the existing rock outcrop, where the receiving depth would be very shallow. By the time the flow re-enters the river channel downstream of the rock outcrop, the energy is largely dissipated and plunging will be largely eliminated, thereby eliminating the production of high levels of TDG. There are some design details that would require further investigation to confirm that this alternative performs as desired from a hydraulic perspective. Specifically, it will be essential to ensure that the jet trajectory spreads and impacts the rock outcrop before it enters the downstream river at all discharges up to the 7Q10 discharge. This will present a very challenging design process. Although the CFD results indicate that this will occur for spillway flows ranging from 8,400 to 33,800 cfs, a physical model is the only reliable method to attempt to develop a design that produces the desired hydraulic characteristics. If the hydraulic design aspect of this alternative proves to be feasible, there is a reasonable potential that it could reduce TDG levels to perhaps as low as 115%.

4.2.4 Civil and Structural Design

Alternative 2 will require more extensive removal of the existing concrete spillway, and the new spillway structure will require extensive concrete placement to form the super-elevated spillway sections. A reinforced concrete foundation tied directly to the exposed bedrock will be required and careful consideration of the impact to the spillway stability during construction will have to be considered as part of the design development. Limited rock excavation in the project tailrace will be required to support this alternative, but forming and placement of the new reinforced concrete sections will be the major constructability challenge associated with this alternative.

<u>Structural Evaluation</u>

This alternative would be constructed by removing a section of the lower spillway face and construction of two new curved, elevated ramps and training walls. The construction of an elevated bridge section was investigated instead of a mass concrete section, but the relatively large lateral force and tall support columns which would be required made the use of an elevated section infeasible.

The impact of the falling water on the base of the spillway would impart a thrust force down into the bedrock foundation and back against the mass of the dam. The horizontal curvature would impart a longitudinal thrust force away from the face of the spillway and a lateral thrust force perpendicular to the face of the existing spillway. The longitudinal force components were compared, and it was found that the net longitudinal force will act towards the face of the spillway, acting to stabilize the new spillway section. The hydraulic drag force will act to pull the block away from the dam.

The computed thrust and drag forces for Alternative 2 are shown in Table 4-3 below.

Table 4-3 Alternative 2 Load Summary

Parameter	Force
Vertical Thrust	870 kips
Longitudinal Thrust (into the dam)	300 kips
Lateral Thrust (across the dam)	1525 kips
Hydraulic Drag (away from the dam)	-95 kips
Max Vertical Bearing Pressure	1400 psf
Friction from Dead Load	6360 kips

As noted for Alternative 1, the allowable bearing capacity is estimated to be approximately 10,000 psf which is significantly higher than the estimated bearing pressure given above. Similar to Alternative 1, this alternative also provides an overall increase of the concrete mass placed at the toe of the spillway; and, the increased mass will add to the dam's ability to resist sliding and overturning.

Based on the conceptual analysis, the weight and friction from the added concrete is sufficient to resist the resulting hydraulic loads. In order to account for secondary force effects, the preliminary design assumes that the new concrete will be anchored to the existing spillway using grouted reinforcing steel dowels on the vertical and horizontal faces.

4.3 Alternative 3 - Spill Bay 1-2 Toe Modification and Downstream Deflector

Alternative 3 is comprised of extending the toe of the spillway chute and incorporating a flow deflector downstream of Spill Bays 1 and 2, as shown in Figure 4-3 and Drawings 300 through 304 (Appendix A). The rock outcropping located downstream of these bays would be reshaped to form a shelf at El. 1370 ft, followed by a 35 ft long deflector set at El. 1363 ft. Due to the configuration of the existing spillway chute and rock surface downstream of the spillway, the construction of a standard deflector was not feasible.

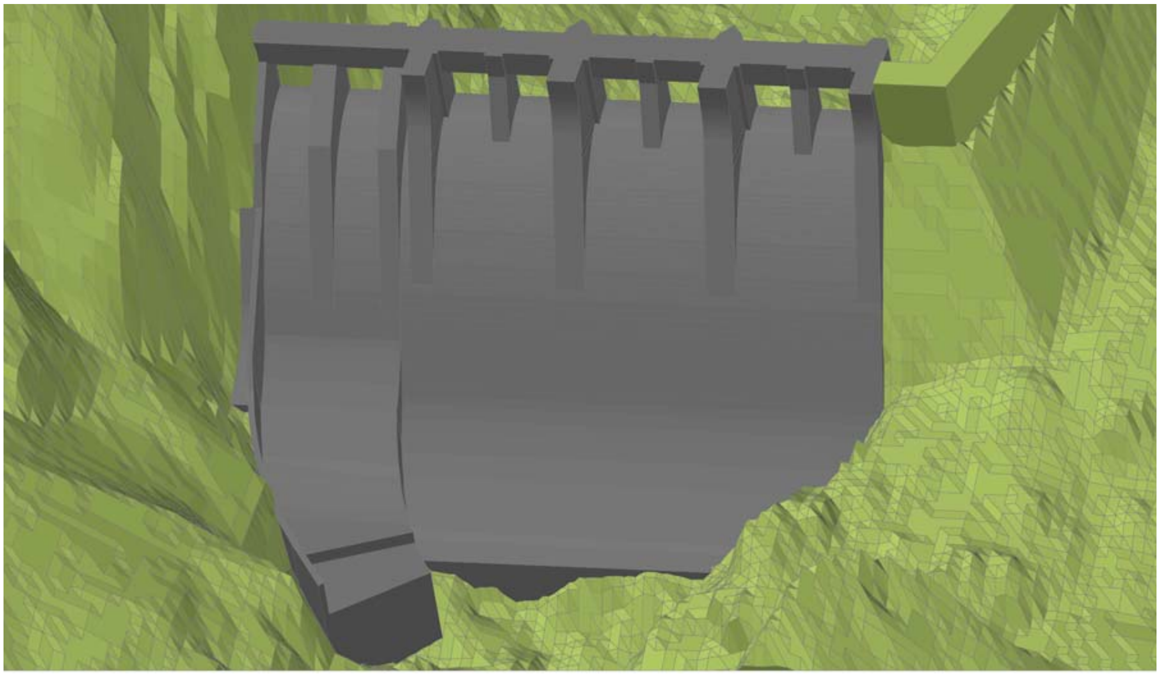

Figure 4-3: Schematic of Alternative 3

4.3.1 Hydraulic Analysis

For Alternative 3, the downstream rock outcropping will be reshaped to form a modified toe apron that will lead into a deflector. The configuration of the proposed toe modification is significantly impacted by the existing topography in this area as well as the design of the existing spillway chute. The proposed rock apron at El. 1370 ft is too high to function as a standard flow deflector (estimated tailwater level is at El. 1373 ft), so a new deflector will be required at the downstream end of the shelf to prevent plunging flow.

The proposed deflector is similar to the recent design of a deflector at Oxbow Dam (NHC, 2007), where an L-shaped deflector was designed at the terminus of a high velocity chute to provide an acceptable flow regime for TDG up to the 7Q10 discharge. The unit discharge at Oxbow varied along deflector, with unit discharges approaching 375 cfs/ft at the downstream end deflector for the 7Q10 discharge. Through model testing, a deflector submergence of 10.5 ft and a deflector length of 40 ft were shown to provide an acceptable flow regime at the downstream deflector, while the side deflector required a submergence of 8.5 ft and a length of approximately 16 ft. This information was used to provide a preliminary deflector design for the Long Lake spillway. The proposed deflector elevation at 1363 ft provides 10 ft of submergence and the proposed length of 35 ft was selected on the basis of the estimated unit discharge for the project.

Again, physical modeling will be required to refine the Alternative 3 spillway toe modification and deflector design to ensure an acceptable flow regime can be achieved. The model should also be used to verify the performance of this design under a PMF condition and evaluate the potential impacts to downstream erosion.

4.3.2 CFD Modeling

Alternative 3 was evaluated at the same discharges as the previous alternatives; however, flow was passed through spillway bays 1 and 2 rather than 7 and 8. Due to instabilities with the CFD model, it was necessary to reconfigure the apron and toe modification slightly from that described in the previous section. As shown Figure C-29 (Appendix C), void regions formed immediately downstream of the 7 ft vertical drop from the apron set at El. 1370 ft to the deflector set at El.1363 ft. These void regions caused the CFD simulations to run extremely slowly and eventually become unstable and fail. As a result, in order to evaluate the design in the CFD model, the apron was removed from the design and the spillway chute transitioned directly to the deflector at El. 1363 ft, as shown in Figure C-30. The model simulations were run successfully once the 7 ft vertical drop at the base of the spillway was eliminated.

Alternative 3 Tests 1 and 2 were conducted with spillway gates 1 and 2 operating, while Test 3 had two additional gates (4 and 5) operating. The resulting unit discharge at each spill bay gate was approximately:

- 405 cfs/ft for Test 1;
- 190 cfs/ft for Test 2; and,
- 450 cfs/ft through gates 1 and 2 plus 305 cfs/ft through gates 4 and 5 for Test 3.

The CFD modeling results showed that the toe modification and downstream deflector reduced the depth of the plunging flow by approximately 10 ft from that observed for the baseline (existing) conditions. Figure C-31 (Appendix C) illustrates the spillway jet trajectory at the base of the spillway toe modification and deflector.

For Tests 1 and 2, the flow was concentrated through gates 1 and 2, resulting in a large area of flow recirculation on the left (west) side of the pool, as shown in Figures C-32 and C-33. This area of flow recirculation remained for Test 3, but was reduced in size and strength as a result of operating gates 4 and 5. This is evident in Figure C-34, which illustrates the depth averaged velocities and streamlines for Alternative 3 Test 3.

For Alternative 3, history particles were initiated upstream from the operating gates and at the base of the spillway at the downstream end of the spillway toe modification at depths of El. 1365, 1366 and 1367 ft.

4.3.3 TDG Performance Estimates

Although Alternative 3 includes a deflector design, the topography in area of Bays 1 and 2 requires the flow to be discharged in the existing deep plunge pool which will likely limit the TDG reduction potential. In addition, the deflector is located downstream of a rock apron so it is in a slightly different location in relation to the spillway face compared to the Alternative 1 design. As a result of these site constraints, the reduction potential is considered to be less with this alternative when compared to Alternative 1. The TDG will likely be reduced to the order of 125% to 130% at the 7Q10 discharge.

4.3.4 Civil and Structural Design

Alternative 3 will require extensive rock excavation on the east abutment downstream from spillway bays 1 and 2. The proposed new concrete channel will require a training wall founded on bedrock. The height and width of the wall will vary along the concrete channel and deflector length. Based on the preliminary design layout, it appears that the downstream portion of the flow deflector will be partially elevated above the existing rock outcropping, which may require a reinforced concrete channel supported by cast-in-place piers, or a deep concrete foundation section. A more in-depth analysis of the structural support design will be completed as the design advances. Similar to Alternatives 1 and 2, a portion of the existing spillway toe will have to be removed to construct the new spillway section.

Structural Evaluation

This alternative includes the construction of a shelf with training walls to deflect the flow away from the rock bluff immediately downstream of spillway bays 1 and 2. The spill deflector would be constructed by removing a section of the lower spillway face and constructing a new curved apron and elevated deflector with training walls on the eastern edge to protect the existing abutment and rock slope. The curved section of the new spillways will be constructed from mass concrete and the lower, tangent deflector section will be an elevated bridge structure since there are minimal thrust forces acting on this section.

The impact of the falling water on the base of the spillway would impart a thrust force down into the bedrock foundation and back against the mass of the dam. The horizontal curvature would impart a longitudinal thrust force away from the face of the spillway and a lateral thrust force perpendicular to the face of the existing spillway. The longitudinal force components were compared and it was found that the net longitudinal force will act towards the face of the spillway, acting to stabilize the new spillway deflector section. The hydraulic drag force will act to pull the block away from the dam.

The thrust and drag forces for Alternative 3 are presented in Table 4-4 below.

Table 4-4 Alternative 3 Load Summary

Parameter	Force
Vertical Thrust	3910 kips
Longitudinal Thrust (into the dam)	585 kips
Lateral Thrust (across the dam)	3520 kips
Hydraulic Drag (away from the dam)	-350 kips
Max Vertical Bearing Pressure	500 psf
Friction from Dead Load	370 kips

As noted for Alternatives 1 and 2, the allowable bearing capacity is estimated to be approximately 10,000 psf, which is significantly higher than the estimated bearing pressure given above. Similar to those alternatives, Alternative 3 also provides an overall increase of the concrete mass placed at the toe of the spillway, and the increased mass will add to the dam's ability to resist sliding and overturning.

Based on the conceptual analysis, the longitudinal component of thrust will act into the face of the existing spillway and the lateral component will act against the existing hillside to the east of the spillway bays. For these reasons, the weight, friction and passive resistance is sufficient to resist the resulting hydraulic loads. In order to account for secondary force effects, the preliminary design assumes the new concrete will be anchored to the existing spillway using grouted reinforcing steel dowels on the vertical and horizontal faces.

4.4 Alternative 4 - Cut-off Dam Chute Spillway with Deflector

Alternative 4 is comprised of a new high-velocity chute spillway constructed downstream of the existing cut-off dam. Drawings 400 through 407 (Appendix A) show preliminary plan and section views of this concept. There are two options under consideration for the location of the spillway crest. For Option 1, the spillway crest is located at the spillway cut-off dam location; and, for Option 2, the spillway crest is located approximately 300 ft downstream of the cut-off dam. Option 1 was used for the hydraulic and civil design calculations. For either alternative the crest elevation is set at El. 1513.5 ft and flow to the spillway chute will be controlled using two 24 ft high tainter gates. The spillway chute is 60 ft wide for the first 725 ft before expanding to 100 ft wide at the chute terminus where there is an 80 ft long deflector set at El. 1365 ft.

4.4.1 Hydraulic Analysis

This alternative includes the design of a spillway crest, chute, and terminus structure. The chute is designed as a high velocity chute to minimize the depth of excavation required and the chute wall heights. A variety of spillway types were considered, including baffled and stepped chute concepts that can be designed to dissipate energy and de-gas spillway flows. However, given the relatively confined area available for the spillway chute, the resulting unit discharge (420 cfs/ft) was considered to be too high for either a baffled or stepped chute (typically used for unit discharges less than about 100 cfs/ft). As a result, a standard ogee crest spillway and high velocity chute were designed with a deflector incorporated at the downstream end of the chute to minimize plunging flow into the tailrace.

The hydraulic design of this spillway was based on USACE design guidelines (USACE, 1990). The practical width available for a spillway chute downstream of the cut-off dam is estimated at 60 ft, based on minimizing excavation requirements while maintaining reasonable flow depths and velocities within the spillway chute. The ogee crest was designed in accordance with USACE guidelines using the following equation.

$$X^n = K * H_d^{\,n-1} * Y$$

where,

X = horizontal coordinate positive to the right, feet
n = 1.85 (from USACE, EM 1110-2-1603)
K = variable, dependent on P/H_d
Y = vertical coordinate positive downward, ft

For this design the 'K' variable is 2.2 in conformance with a low crest height (P) to crest shape design head (H_d) ratio.

As shown on Drawings 400 and 401, the chute design includes horizontal and vertical transitions along its length. The horizontal transitions include a large radius arc designed to follow the alignment of the existing ravine and an expansion in the chute width from 60 ft to 100 ft. Vertical transitions have been designed to minimize the excavation required along the chute length and provide stable flow at the downstream end of the spillway. The overall chute is approximately 945 ft long, and the chute is 60 ft wide from the crest to the second parabolic transition where the chute width expands to 100 ft.

The high velocity chute hydraulic parameters were modeled using the HEC-RAS 1-dimensional computer program to estimate the water surface elevations and velocities. The velocities along the 60 ft wide portion of the chute are estimated to range from 24 ft/s at the spillway crest to over 80 ft/s before the chute expands from 60 ft to 100 ft wide. At the spillway crest the depth of flow will be approximately 18 ft deep and within the high velocity chute the flow depth ranges from approximately 9 ft at the upstream transition to 5 ft at the downstream transition. Preliminary estimates of the chute wall heights range from approximately 30 ft at the spillway crest to 10 ft at the downstream end of the chute.

At the downstream end of the chute, the walls diverge to increase the chute width to 100 ft and decrease the unit discharge from 420 cfs/ft to approximately 250 cfs/ft. The expansion rate of 1:10 was selected to minimize standing waves and provide uniform flow at the downstream end of the chute where a deflector will be installed to minimize plunging flow at the chute terminus. The Alternative 4 deflector was designed by estimating the trajectory at the end of the chute and by applying the Oxbow deflector design information (NHC, 2007). The proposed deflector is set at El. 1365 ft, providing approximately 8 ft of submergence, and the deflector length is set at 80 ft to ensure that the flow trajectory leaving the chute impacts the deflector. The final design for the spillway crest, chute and deflector must be refined and verified in a physical model.

4.4.2 TDG Performance Estimates

The spillway and chute width for Alternative 4 is limited to approximately 100 ft at its downstream end by the site topography. This results in a unit discharge considerably higher than those encountered at other spillway projects where deflectors have been shown to be effective at reducing TDG. While the general concept of this alternative is considered to have a high potential for TDG reduction, the restricted spillway chute width and resulting high unit discharge at Long Lake may limit the potential effectiveness of this alternative. When compared to the alternatives at the existing spillway, the TDG reduction potential with this alternative would likely be greater than that of Alternative 3, but not any greater than the 120 – 125% range associated with Alternative 1.

4.4.3 Civil and Structural Design

For Alternative 4, the civil design aspects of the proposed facilities will be challenging due to the existing ravine geology and limited space downstream of the cut-off dam. Construction of the facility will require temporary shoring to provide sufficient access along the spillway chute alignment. Provisions for a new access road along the completed facilities while maintaining access to the existing powerhouse facility will also prove to be a challenging aspect of the project. Construction of the flow deflector will require a downstream cofferdam and dewatering within the tailrace area. From a structural design standpoint, the new structures will be tied to exposed and prepared bedrock to ensure structural stability. The new spillway will require extensive preparation of the bedrock foundation and construction of a grout curtain. The structural design will depend on the final geotechnical investigations and recommended bedrock design parameters. The chute spillway will require anchoring to exposed bedrock and/or construction of thrust blocks.

Structural Evaluation

This alternative includes the construction of a new spillway to the west of the existing powerhouse at the location of the existing arch cutoff dam. The proposed structure would include a new spillway dam gate structure, two new spillway gates, and a spillway chute approximately 1000 ft long. The spillway chute would be approximately 60 ft wide by 25 ft deep and partially buried over the majority of its length. Because it consists of a buried structure, the thrust and drag forces would be transferred directly into the foundation soils. The lower section of the chute may be constructed on fill, but the flow in this section will be relatively straight and uniform, so significant thrust loads are not anticipated.

The concrete member size, including the crest approach walls, chute side walls and slabs, and top struts were estimated based on experience with similar sized hydraulic structures. The walls would be designed for two conditions: the first would assume full backfill behind the wall with no water in the chute; and the second would be for the chute full of water without any backfill on the outside. In addition, the chute will be designed for buoyancy over the length of the structure.

4.5 ALTERNATIVE 5 – NEW SECOND POWERHOUSE

4.5.1 Existing Long Lake Powerhouse Description

The existing powerhouse is an indoor structure housing four 17.9 MW double-runner horizontal Francis turbines. At full gate, the plant is capable of producing 72.5 MW. The turbines have two runners each; both mounted on a common shaft. The machines rotate at 200 RPM with a maximum hydraulic capacity of 1,750 cfs each. The total hydraulic capacity of the powerhouse is 6,800 cfs. The normal operating range of the reservoir is between El. 1535 and 1536 ft. At low river flows, the plant is typically operated on a six hour per day peaking schedule with the generating units shut down at night. During periods of high flow, the power plant operates continuously and the spillway is operated to maintain the reservoir level at the full pool level of El. 1536 ft.

4.5.2 Review of Previous Studies

Three primary studies have previously evaluated the addition of generation facilities at the Long Lake Project:

1. Long Lake Expansion Plan prepared by Bechtel Corporation for Washington Water Power, December 3, 1990.
2. Long Lake Hydroelectric Development Total Dissolved Gas Abatement Initial Feasibility Study Report, prepared by EES Consulting Inc. for Avista Utilities, September 2006

3. Hydro Additions and Construction Cost Estimates for Cabinet Gorge, Long Lake, and Monroe Street Projects, prepared by Hatch Acres for Avista, June 2009.

The Bechtel report was a relatively comprehensive study of potential generation alternatives ranging from upgrading the existing powerhouse, various combinations of generation capacity and configurations for a new second powerhouse, and a combination of a new second powerhouse and utilization of the existing powerhouse. The EES report utilized the data presented in the Bechtel report with estimated costs escalated to reflect 2006 cost basis (no additional evaluation of the proposed generation alternatives was completed as part of the EES work effort). The Hatch Acres study effort was focused on updating the cost estimates from the previous generation study efforts to 2009 market pricing.

For the purposes of the present study, the Bechtel work effort has been used as the foundation for review, evaluation, and modification of the technical aspects of the second powerhouse alternative. The Hatch Acres report will be used as the basis for the construction cost estimates. In general, the proposed second powerhouse features will be similar in scope, layout, and operation as propose in the Bechtel report. An updated energy evaluation model was prepared as part of this study effort as well as updated cost estimates. A brief summary of the proposed second powerhouse generation facilities features is presented in the following paragraphs.

4.5.3 Second Powerhouse Alternative

In 1990 under contract to the Washington Water Power Company (now Avista), the Bechtel Corporation completed the Long Lake Expansion Project Final Plan (Bechtel, 1990). The Final Plan Report examined numerous alternatives for renovation and expansion of the Long Lake Hydroelectric Development on the Spokane River Project. The report examined three possible options:

Case 1: Rehabilitate the existing powerhouse to such an extent to provide minimum 35 years of reliable service (this option was dropped due to the overall cost, site constraints, and lost revenue generation while the system was down);

Case 2: Retire the existing powerhouse, remove it and rebuild a new powerhouse on the current site, including construction of a new intake structure and penstocks (this option was dropped due to similar cost and site constraints identified in Option 1); and

Case 3: Construct a new powerhouse with a similar configuration to the existing powerhouse. Locate the new powerhouse to the west of the existing facility; add new penstock(s) and an intake structure at the location of the cut-off dam. Continue to maintain generation operations at the existing powerhouse as back-up to the new facility.

Case 3, which consists of building a new independent powerhouse in addition to the existing powerhouse was considered to be the best option. Under current site conditions, it is possible to construct an entire new system without significant shutdown of the existing plant or loss of revenue from energy production.

In addition, under Case 3, instead of retiring the existing powerhouse it would be retained, after completion of the new powerhouse, for infrequent use when river flows are sufficiently large to load new units to their full capability and still provide enough surpluses to operate the older generation units. Alternatively when river flows are so low that the outflow from Long Lake would be less than 3,200 cfs, the old units due their smaller size, are more efficient than the new turbines at low discharges.

Under Case 3, three turbine design concepts were prepared by Bechtel using a vertical shaft Francis Turbine generator model. The Francis Turbine generator was chosen over a Kaplan turbine based on high head and high river flow conditions anticipated at Long Lake. River flows are estimated at an annual average of 7,700 cfs (Table 6.1 1990 Bechtel Report); and, approximately 165 feet of head. Kaplan turbines are more efficient at high flows but at much lower heads than noted at Long Lake. A Francis Turbine generator package comes as both a vertical and horizontal shaft configuration. A horizontal Francis Turbine is more efficient at higher heads and lower flows, while a vertical shaft Francis Turbine is more efficient at higher heads and higher flows. The efficiency cut-off between vertical and horizontal shaft Francis turbines is at the 20 MW name plate generation. Concepts considered by Bechtel included:

> Concept 1: A single vertical Francis turbine-generator in a new powerhouse, served by a new intake and penstock. The unit will be sized at 40, 50, or 60 MW rated capacities, each with an allowable 15 percent increase in load at full gate conditions.
>
> Concept 2: Two vertical Francis turbine-generators in a new powerhouse, each unit served by its own gated intake bay and separate penstock, to eliminate the need for turbine shutoff valves. Each unit will be sized at 40, 50, or 60 MW rated capacity, with an allowable 15 percent overload.
>
> Concept 3: Three vertical Francis turbine-generators in a new powerhouse, each unit served by its own gated intake bay and separate penstock, to eliminate the need for turbine shutoff valves. Again, each unit will be sized at 40, 50, or 60 MW rated capacity with 15 percent overload capability. Space limitations in the intake area (at the cut-off dam) suggest that this alternative will involve substantial increases in quantities-particularly rock excavation-over the previous two alternatives. The third intake bay will be required on the left side of the approach channel, requiring the third penstock to be longer and have another bend as compared with the first two penstocks.

As summarized in the 1990 Bechtel report, based on a river flow of 7,700 cfs and a head of 165 feet, Concept 2 (Two vertical Francis turbine generators) was selected for layout and cost development. Concept 2 included a new intake structure, two new steel penstocks (17 feet in diameter, 658 and 682 feet in length), and a new powerhouse containing two vertical Francis turbine generator units, each rated at 50 to 60 MW. Drawings 500 and 501 show the overall layout of the new powerhouse as presented in the Bechtel report.

The existing powerhouse will be maintained to operate during very high and very low flows on the Spokane River. During very dry periods, such as the months of August and September, inflows to Long Lake may fall so low that regulated outflows during the 6-hour daily peak generating period will fall below 2,500 cfs. This would happen when daily inflows fall below about 625 cfs, or when the generating schedule does not permit an 18-hour shutdown each day. In those circumstances, one of the new Francis turbines would operate below about 60 percent of rated load. This would result in turbine efficiency below that of the old units operating at or near best gate. When Long Lake inflow exceeded the 9,600 cfs hydraulic capacity of the new plant, the old plant would be operated up to its hydraulic capacity (6,800 cfs) to utilize additional flow.

The recommended improvements included modernization of the generating units and rehabilitation or replacement of the spillway gates and power intake gates. The generating units were modernized during the 1990s and all the dam gates and gate hoists were replaced with new equipment in the same time frame.

As discussed above, the alternatives considered in the 1990 modernization study included new powerhouse arrangements with one, two, or three generating units and total installed capacities of 40, 50, 60, 80, 100, and 120 MW. These power ratings correspond to hydraulic capacities ranging from 3,200 cfs for the 40 MW alternative to approximately 9,600 cfs for the 120 MW alternative.

4.5.4 New Second Powerhouse Design Characteristics

The Concept 2 (two units) design characteristics, as proposed by Bechtel, include a new intake structure, new penstock, and major mechanical equipment. A brief description of these design characteristics is described below:

Intake Structure

The new intake structure is proposed to be located at the cut-off dam. The new intake structure will be two identical bell-mouthed intakes placed side-by-side. The structure will be an above-ground buttressed reinforced concrete water retaining element. In function, it will replace the existing cutoff dam as well as control the flow entry into the two new penstocks. It is proportioned to be completely stable with its gates either open or closed, thus forming a new cutoff dam. The existing cutoff arch dam will be breached after the new intake structure is completed and tested. The new structure will house trashracks, raking equipment, two fixed wheel intake gates, and hoists, plus stoplog slots to permit the dewatering of the gates and appurtenances. The gates will serve for maintenance dewatering of penstocks and turbines, but will also be capable of emergency closure in the event of a turbine runaway or other penstock or powerhouse emergency. The gates will each be 11 feet wide by 18 feet high. Access to the penstocks will be provided via a dry shaft for each gate. Refer to Section 4.4 for additional civil engineering information on the location of the Alternative 5 powerhouse intake.

Penstock

Water will be conveyed from each intake bay to its corresponding turbine in the powerhouse by one of two new steel penstocks. Each penstock will be 16 feet in inside diameter, which ensures a water velocity of under 20 fps at rated power. At full gate, the velocity will reach nearly 25 fps. Each penstock will be approximately 630 feet long from the intake to the turbine spiral case inlet. The penstocks will be of welded steel construction, supported by ring girders on concrete cradles. Mechanical couplings will be provided in each penstock to allow for thermal expansion and contraction without inducing large stresses into the penstocks. Use of a dedicated penstock for each turbine eliminates the need for turbine shutoff valves. The penstocks will be exposed except for the downstream ends, which will be backfilled in the vicinity of the powerhouse. This will permit the existing powerhouse access road to pass over the penstocks, and will prevent drainage problems where the elevation of the penstocks drops below tail water.

Powerhouse

The new powerhouse is proposed to be located just downstream from and to the left of the existing powerhouse. It will be a two unit reinforced concrete, semi-outdoor structure, 90 ft by 157 ft in plan, and 85 ft from the bottom of the dewatering sump to the top deck. The top deck will be set at elevation 1392 ft, fully 20 ft higher than the main machine floor in the existing powerhouse. This will provide for total protection from a flood of the magnitude of the full capacity of the spillway. The powerhouse will have three operating floors in addition to the top deck. The lowest operating floor, the pump floor, will be set at elevation 1343 ft, 20 ft below minimum tailwater level. The next floor above, the turbine floor, will be at elevation 1359 ft. The next floor above, the generator floor, will be at elevation 1374 ft. The powerhouse will have two sumps: a drainage sump, which will receive powerhouse drainage water; and a dewatering sump, into which the draft tubes may be emptied.

Major Mechanical Equipment

The powerhouse will contain two vertical-shaft, Francis-type hydraulic turbines, each rated to produce 68,000 hp when operating under a net head of 165 ft at a speed of 180 rpm. The turbines will be capable of producing a maximum output of approximately 78,000 hp and will be designed to operate under a maximum net head of 160.8 ft. Wicket gates will regulate the flow through the turbines. Each turbine will have two servomotors with sufficient capacity to open and close the gates with minimum operating oil pressure. PID type electro-hydraulic governors will control turbine speed and power output. The powerhouse will be equipped with a gantry crane for generating-unit erection and for subsequent maintenance operations. The crane will have a main hook capacity of 275 tons and an auxiliary hook with a capacity of 50 tons. The crane will be located on the top deck at El. 1392 ft and will travel the full length of the powerhouse to service both generating units as well as the assembly area. The powerhouse will be furnished with two bulkhead type draft tube gates approximately 19 ft wide and 15 ft high. These gates will be placed in the outlet of either turbine draft tube to enable turbine dewatering

4.5.5 Summary of Energy Evaluation Model

Purpose

As noted previously, in the 1990 Bechtel evaluation several hydropower turbine package cases were examined against efficiency, energy generation and infrastructure costs. The Alternative 5 Energy Evaluation will review the Bechtel report recommendations and examine updating generation capacities including turbine unit sizes to increase efficiency and hydraulic capacity operating range. This Energy Evaluation was completed using the TURBNPRO™ software. The previous work completed by Bechtel served as the basis for the analysis presented within this report for Alternative 5.

The TURBNPRO™ (www.turbnpro.com) software is an industry recognized program for evaluating and modeling hydropower turbine sizing to specific turbine packages (i.e. Pelton, Axial and Francis), based on actual site data. The TURBNPRO™ software was used in the Avista - Nine Mile Evaluation. This software program provides the user with typical or anticipated performance; dimensional data of the turbine runner size, efficiency cycles, runaway speeds and conditions that will start potential cavitation in the system. The TURBNPRO™ software also provides the user with the estimated energy generation based on the turbine design created for the site using net head and flow data. The Version 3 of the software program was used in the Long Lake Alternative 5 energy evaluation. Input parameters for the software are identified under the Key Numbers section of this Alternative.

The focus of this evaluation was to determine the potential energy generation revenue which could be realized from the new powerhouse alternative. This estimated revenue would then be considered as part of the overall evaluation of the effectiveness of the new powerhouse alternative in reducing overall TDG and the cost associated with implementing the alternative. A discussion of the evaluation approach and results is presented in the following paragraphs.

Background

The basic turbine package would include two vertical Francis turbine-generators, each unit served by its own gated intake bay and separate penstock in a new powerhouse. Each unit was sized at 40, 50, or 60 MW rated capacities, with an allowable 15 percent overload to be rated for 68,000 hp when operating under a net head of 165 ft and a speed of 180 rpm. The turbines will also be capable of producing a maximum output of approximately 78,000 hp and will be designed to operate under a maximum net head of 160.8 ft. Wicket gates will regulate the flow through the turbines under lower operating conditions. Drawings 500, 501, and 502 illustrate the conceptual layout for Alternative 5.

The existing powerhouse will be maintained to operate during very high and very low flows on the Spokane River. During very dry periods (August and September), inflows to Long Lake fall so low that regulated outflows during the 6-hour daily peak generating period will fall below 2,500 cfs (the minimum operating range for a vertical Francis turbine is 3,200 cfs). This happens when inflows to Long Lake fall below 625 cfs, or when the generating schedule does not permit an 18-hour shutdown each day. Under those circumstances, Bechtel assumed that only one of the new Francis turbines would operate below 60% of rated discharge (cavitation was assumed to start to occur at 65% of rated discharge for a vertical Francis turbine). When Long Lake has low inflow conditions, the new plant will be shut down and the water will be cycled through the old powerhouse to the smaller but higher efficiency turbines. Alternatively when Long Lake inflow exceeds the 9,600 cfs hydraulic capacity of the new plant, the old plant would be operated up to its rated hydraulic capacity of 6,800 cfs to generate electricity from the additional flows.

For Alternative 5 Energy Evaluation, two vertical shaft Francis turbines were modeled in the TURBNPRO™ software package using similar flow and net head conditions as proposed in the new powerhouse configuration.

Long Lake Flow Conditions

For the 1990 Bechtel evaluation, mean daily flows of the Spokane River for the August 1928 through July 1988 time period were provided by Avista (WWP at the time of the study). The average monthly inflow records for Long Lake is based on the USGS recording station 12433000 located below the Long Lake Dam (see Appendix D, - Site Map for Washington USGS Station 12433000). As part of the energy calculation for the TURBNPRO™ software, the average monthly inflow record for Long Lake will be based on the regulated mean daily flows of the Spokane River for the April 1939 through September 2009 time period (see Appendix D - Recorded Discharge from Long Lake 1939-2009 Table).

A second USGS station is located at the Long Lake site. USGS Gauge Station 12432500 is located at the top of the Long Lake Dam and records the reservoir water surface elevation/lake level above NGVD 1929 feet. This station has recorded data from 1979 through 2009 (see Appendix D - USGS Station 12433000 – Daily Lake Reservoir Water Surface). Fluctuations in the Long Lake at the intake are utilized by the TURBNPRO™ software to determine gross and net head on the vertical Francis Turbine.

The hydraulic capacity of the proposed new hydropower plant is 9,600 cfs. The hydraulic capacity of the old plant is 6,800 cfs. The combined hydraulic capacity of both plants is 16,400 cfs. Any flows over the 16,400 cfs will released over the spillway. On an average year, the month of May has the highest flows and will have water passed over the spillway. In a very wet year, similar to recorded levels in 1997, several months in a row may have water sent through the spillway (Table 4-5).

Table 4-5 Average Monthly Flow, Long Lake for 1939 to 2009 versus Hydraulic Capacity

Month	Flow (cfs) Average Year	Combined Hydraulic Capacity (16,400 cfs)	Flow (cfs) 1997 High Flow Year	Combined Hydraulic Capacity (16,400 cfs)
January	7,199	No Spill	14,870	No Spill
February	8,693	No Spill	11,630	No Spill
March	10,481	No Spill	17,380	980 cfs
April	15,253	No Spill	22,160	5,760 cfs
May	18,174	1,774 cfs	36,910	20,510 cfs
June	11,233	No Spill	21,950	5,550 cfs
July	3,416	No Spill	6,483	No Spill
August	1,983	No Spill	2,855	No Spill
September	2,243	No Spill	2,984	No Spill
October	2,900	No Spill	4,186	No Spill
November	4,034	No Spill	4,701	No Spill
December	6,292	No Spill	6,131	No Spill
(Average)	7,658	-	12,687	-

The temperature of the water entering the penstock also affects the efficiency of the turbines and is required value in the TURBNPRO™ software program. USGS Gauge Station 12432500 has recorded water temperature data for the time period of 1962 through 2003. A summary of the water surface temperature is provided in Table 4-6 below.

Table 4-6 Average Temperature for 1962 to 2003 (All temperatures recorded in Celsius)

Mean Temperature	Mode Temperature	Low Temperature	High Temperature
12.2	18	0.5	23
Average	Most entries	Date: 2/28/1965	Date: 8/11/1968

The USGS Gauging Station 12433000 at Long Lake has recorded peak storm events from 1939 through 2008. These peak events are provided in Appendix D. These peak storm events have not been utilized in the TURBNPRO™ software energy evaluation. As the combined hydraulic capacity of both power plants is rated at 16,400 cfs, most of the identified storm events would be passed through the spillways of the Long Lake Dam.

The tailwater rating curve for the Long Lake Dam powerhouse was presented in Figure 2-1. At flows below 16,000 cfs, the tailwater level is influenced by the downstream Little Falls Project reservoir level, which can vary depending on whether the Little Falls dam flashboards are in place. However, at the higher flood flows, the tailwater at Long Lake is essentially independent of variations in the Little Falls pond level because of hydraulic restrictions in the river channel between the Long Lake and Little Falls projects. The centerline of the vertical Francis turbine in relation to the rise and fall of the tailwater is also incorporated into the TURBNPRO™ software program to determine turbine efficiency and cavitation conditions.

Long Lake Hydropower Key Numbers

The following tables summarize key numbers referenced from various reports, documents, and web sites that have been utilized in preparing this energy evaluation. The key numbers for both existing and proposed hydropower development conditions have been outlined in Table 4-7 and 4-8.

Table 4-7 Key Numbers for Energy Evaluation of Existing Conditions

Element	Description	Comment
Project No: 2545:	Federal Energy Regulatory Commission (FERC).	New Application will be required for the new powerhouse. May take upwards of two years to issue following application submittal.
6,800 cfs	Rated Hydraulic Capacity through existing powerhouse.	Maximum flow through the old powerhouse can be as high as 7,000 cfs.
72 MW/ 88 MW	Actual power/ Name Plate generation.	Powerhouse updated in the 1990s/ Cannot go to 88 MW due to generator rating.
Intake	(4) intake structures - 3 @ 16 ft and 1 @ 14 ft diameter.	Non-overflow gravity section/four gated fixed wheel structures 18' Square
Penstock	Four Penstocks at 236 feet in length.	Penstocks made of steel.
(4) 17.9 MW	Double Runner Horizontal Francis Turbines.	Each turbine has two runners on a common shaft.
200 RPM	Turbine Rotation Speed.	Reference: Bechtel
1,750 cfs	Hydraulic Capacity of each turbine.	Reference: Bechtel
EL 1535 – 1536'	Operating Range of Long Lake Forebay Elevation based on inflows, seasonal draws, spring runoff, and daily load.	Maintain constant pool at elevation 1536 feet, spill during high flows. Level in the Long Lake drops during the dry season.
6 Hours	When Long Lake water level is low. Existing power plant is run during peaking schedule only.	Plant shuts down at night. No Capacity Factor has been identified in any of the available references.
Case 3 of Bechtel Report	Keep old powerhouse, operate during low flows <3,200 cfs due to smaller size of Francis Turbines.	New Turbines should be adjusted for greater efficiency at high flows.
625 cfs	Cut-off minimum inflows into Long Lake, consider low flow at the hydropower plant.	Reference: USGS Gauge Station
104,292 Acre-ft.	Average or Preferred Storage at Long Lake Reservoir	Maximum storm event inflow into Long Lake May 24, 1948 – 49,400 cfs. Reference: USGS

Table 4-8 Key Numbers for Energy Evaluation of Proposed Conditions

Element	Purpose	Comment
cut-off Dam location for new intake structure	108' Height by 247' Crest Length	Constant Radius – 170'. Cut-Off dam is already built/in-place. Reference: Bechtel
1939 – 2009 calendar years	Record Inflow records	Recorded below the Long Lake Dam. Reference: USGS
165 feet of Head	Estimate Head from cut–off dam to proposed powerhouse.	Horizontal distance from cut-off Dam to proposed powerhouse is ~ 350 feet. Slope is 0.47 (165/350)
7,658 cfs - mean 18,174 cfs - high 1, 983 cfs - low	Average Monthly Flow through Long Lake. Average Monthly flow for May Average Monthly flow for August	See Table 4-5 – Recorded Discharge from Long Lake 1939-2009. Reference: USGS.
(2) 40/50/60 MW turbines 80 to 120 MW	Case 3 of Bechtel Report suggests the best rated concept of two (2) vertical Shaft Francis Turbines. Proposed name-plate capacity	This turbine package will allow for 15% overload. Reference: Bechtel
180 RPM	Each Francis Turbine rated to produce 68,000 HP at net head of 165 feet. Maximum output of 78,000 HP at maximum net head of 168 feet.	Reference: Bechtel Report Case 3, Concept 2
9,600 cfs 4,800 cfs	Total rated hydraulic capacity for new power plant using two (2) vertical Shaft Francis Turbines (based on efficiency at higher head and higher flows). Rated hydraulic capacity of each turbine.	Reference: Bechtel Report Case 3, Concept 2
(2) Penstocks at 458 and 482 feet in length.	Two new penstocks from cut-off dam with 16' inside diameter.	Penstock would be made from 9/16" steel. Reference: Bechtel
20 Feet/Second 25 Feet/second	Velocity of water in new penstocks. Velocity at full gate in new Penstocks	Estimated head loss from intake to turbine is approximately 11 Feet.
Intake (new)	A new structure will house trash racks, raking equipment, two fixed wheel intake gates, and hoists, plus stop log slots to permit dewatering.	The gates will be 11 ft wide by 18 ft high. Access to the penstocks will be provided via a dry shaft for each gate.

Element	Purpose	Comment
Operation Cycle	Operate new powerhouse for flows between 3,200 and 9,600 cfs	During low flows, only one turbine would operate at 60% of rated load. Case 3 proposes shutting down the new powerhouse and switching water to old power house. Reference: Bechtel
>9,600 cfs	Cut-off maximum flow to new power plant. Existing powerhouse would be utilized for flows exceeding this maximum up to 6,800 cfs.	The months of March, April, May and June have average flows exceeding 9,600 cfs. The average flow in the month of May exceeds hydraulic capacity of both hydropower plants.
EL 1343' EL 1359' EL 1363' EL 1374' EL 1392' EL 1536' EL 1522'	Lowest operating (pump) floor Turbine Floor Elevation Minimum Tailwater Level Elevation Generator Floor Elevation Top deck/over head gantry crane El. Maximum water surface in Lake Minimum water elevation in Lake	Proposed Elevations at the new power house. Reference: Bechtel.
19 feet diameter	Elbow draft tube.	Centerline of turbine is set at 1346' Elevation.

Energy Evaluation

In preparing the energy evaluation for Alternative 5, the following assumptions were made:

1. Turbine Type: Similar to the assumptions under Case 3 in Bechtel's 1990 Report, the new powerhouse will contain vertical (shaft) Francis Turbines (versus a horizontal shaft configuration). No other hydropower turbine package design (Axial, Kaplan, Turgo, etc.) will be reviewed under this energy evaluation.
2. Turbine Size: The Bechtel Corporation identified three different vertical Francis turbine sizes ranging from 40, 50 and 60 MW name plate capacity. The TURBNPRO™ software, once loaded with the specific site constraints of gross head, net head, flow rate as delivered by the penstock(s), water temperature, and site elevation (in reference to sea level) will determine a tighter range of name plate generators based on turbine efficiency.
3. Turbine Quantity: Based on this land footprint, the available space allows for two side-by-side vertical (shaft) Francis Turbines. Although additional turbines can be added to the TURBNPRO™ software to determine energy generation, only two turbines have been outlined in this evaluation.
4. Site Conditions: The 1990 Bechtel Report outlined very specific site design condition and constraints for various features of a new powerhouse. These same site conditions and constraints were loaded into the TURBNPRO™ software. A list of the design conditions and constraints are listed in the Criteria Design Table.
5. Items not included in the TURBNPRO™ software model: Items that have not been included in the energy evaluation model include A) capacity factor for the new powerhouse; and, B) electrical generation of the old powerhouse in conjunction with the new powerhouse when seasonal water flow exceeds/ or is below the hydraulic capacity of the new powerhouse (see electrical generation table at 4,800 cfs for cycling schedule). The capacity factor is a function of operation schedule, grid/PUC policy, consumer energy demand, and seasonal flows (high or low) in Long Lake. Per Avista's website on generation and loads (2010 Sustainability Report), the average hydropower plant availability factor is 92.39%. Based on the percent of water flow over a 12 month calendar cycle, the TURBNPRO™ software assumes a 100% operation cycle to generate electricity. A 100% production cycle is not realistic as cavitation (damage to the turbine runners) starts occurring in the vertical Francis Turbine, when flows drop below 3,200 cfs. Per the Electrical Generation Table – a subjective estimate of MW hrs produced based on the utilization rate of the old plant is submitted as 113,652 MW hrs.

Criteria Design

The TURBNPRO™ software allows the user to submit specific site design information in developing the turbine specifications. The TURBNPRO™ software only recognizes metric, all English values at Long Lake converted to metric. As existing site design information was prepared by Bechtel, this information was utilized in this updated energy model. The criteria design for this energy evaluation is submitted as follows:

Table 4-9 Criteria Design Information for Sizing Turbines

Element (English)	Description
4800 cfs 9,600 cfs	Hydraulic capacity of one turbine /penstock Total hydraulic capacity of new powerhouse based on delivered flow via two 16' diameter penstocks.
Two (2)	Number of Vertical Francis Turbines
172.9 Feet	Gross Head measured from maximum water surface elevation in Long Lake (1536 feet) to operating level of tailwater elevation (1363 feet).
160.8 Feet	Net Head measured from operating level in Long Lake (1535 feet) to operating level of tailwater elevation (1363 feet) – 11.7 feet for head losses in system.
147.9 Feet	Minimum Net Head under low flow conditions – Long Lake water elevation (1522 feet) to minimum operating level of tailwater elevation (1363 feet) – 11.7 feet for head losses in system.
1346 Feet	Site Elevation (m.s.l.) of the centerline of the turbine as defined in the Bechtel report.
64,0 F	Water Temperature – 18.0 C mode value (most common recorded value). Average temperature of water is 12.2 C when the lowest and highest temperatures are included.
60 Hz	Utility Frequency in the United States.
-18 Feet	Turbine Setting to tailwater (approximate) 1364 feet – 1346 feet = 18 feet. Tailwater ranges from 17 to 19 feet.
Average Flow Year – (Similar to 1962)	For estimating the amount of energy that a Vertical Francis Turbine can produce over a 12 month period using a running monthly average taken over 70 years of recorded data. See Table 4-5, Recorded Discharge from Long Lake 1939 to 2009.
1,983 cfs 18,174 cfs	Minimum flow over the Long Lake Dam in the month of August (of any given year). Maximum flow over the Long Lake Dam in the month of May (of any given year).
1536 Feet	Average (mean) operating level or elevation in the Long Lake Reservoir.
1358.5 to 1369.0 Feet	Range of elevation changes in the tailwater below the Long Lake based on average flow coming through the penstocks and over the dam spillways.

TURBNPRO™ Software Results

The results of the TURBNPRO™ software analysis are submitted under Appendix D of this report. As outlined in the Electrical Generation Table at 4,800 cfs, two vertical Francis Turbines can operate from the maximum flow of 9,600 cfs down to 5589.06 (~5,600) of available flow (each penstock transmits 4,800 cfs). Once flow drops below 4,800 cfs, one penstock will be closed and all water flow will be directed or cycled to one 60 MW vertical Francis turbine. When the flow in the single penstock drops below 3,200 cfs, water flow at the Long Lake Dam should be redirected to the old powerhouse as the smaller turbines are more efficient at lower flows. Alternatively in the spring when flow into Long Lake exceeds the 9,600 cfs hydraulic capacity of the new powerhouse, excess water can be redirected to the existing powerhouse. The break lines for cycling the water flow from the new powerhouse to the existing power are listed in the Electrical Generation Table, see Appendix D.

Summary of Energy Evaluation

Table 4-10 provides a summary of the energy evaluation.

Table 4-10 Energy Evaluation Summary

Element (English/Metric)	Description
93.8% at 4,034 cfs	Highest efficiency that vertical Francis Turbine system can provide under optimal conditions.
90.1% at 4841 cfs	Efficiency at 100% of the rated discharge of 4,800 cfs through the penstock feeding one (1) turbine.
59,3506 KW (59 MW)	Name plate generation of the vertical (shaft) Francis Turbine with spiral case inlet and elbow draft tube. At 59 MW, rated discharge is 100% with a turbine efficiency of 90.1%. Structural design dimensions of intake spiral case, elbow draft tube, shaft arrangement (vertical with runner on Turbine shaft), and wicket gates height and diameter are provided in the results.
150.2 inches/ 3816 mm 171.4 RPM	Runner diameter Unit Speed
17% Overload	From peak efficiency
20 – 25 feet per second 5.7 feet per second	Speed of water in penstock before turbine. Speed of water exiting the draft tube .
31.6 feet	Atmospheric pressure minus vapor pressure for turbine package at site.
Cavitation (damage to turbines)	Cavitation or peak draft tube surging conditions occur at 65% of rated load when intake flow falls below 3,153 cfs (~3,200 cfs). Turbine efficiency drops below 87.4%.
3,200 cfs	Turbine cutoff point, where efficiency is too low, damage starts occurring at 3,200 cfs, not advisable to run vertical Francis Turbines at this flow.
Final Results: Production	Comparison of calendar year 1990 to 2010
561,582 MWH- annual 110,686 MWH-annual 672,268 MWH-annual	**Year 1990** - Theoretical accumulated amount of MWHs – produced on 2 turbines rated at 60 MWs (Table 6.17, Page 6-28, 1990 Bechtel Report). **Year 1990** - Amount of electricity produced from the 2 turbines at the old powerhouse in conjunction with the two 60 MW turbines in the new powerhouse (Table 6.17, Page 6-28, 1990 Bechtel Report). **Year 1990** – Total amount of MWHs accumulated from the old powerhouse and the new powerhouse running in conjunction (561,582 + 110,686).
757,600.75 MWH-annual 709,564 MWH-annual	**Year 2010** - Theoretical accumulated amount of MWHs – produced if turbines are ran below 3,200 cfs flow (not practical due to cavitation of the vertical Francis Generator as identified in the TURBNPRO™ software). **Year 2010** - Realistic accumulated MWHs – produced if turbines are shut down when flow through the penstock falls below 3,200 cfs.

Element (English/Metric)	Description
655,566 MWH- annual 113,652 MWH per 6 month cycle 769,219 MWH- annual	**Year 2010** – At 92.39% average hydro plant availability factor - 2010 Avista Sustainability Report under generation and loads. **Year 2010** - Amount of electricity produced from two (2) turbines at the old powerhouse in conjunction with the two (2) new 60 MW turbines in the new power house. Electrical generation at the old power house is estimated based on utilization rates and reduced water flow redirected from the new plant. **Year 2010** - Theoretical total power of new power house (655,556 MWHs) with the addition of the old powerhouse (113,652 MWHs) from the utilization rate factors.
(+) 96,951 MWH-annual	Difference between Calendar Year 1990 and Calendar Year 2010 Generation Report

Conclusion

The 1990 Bechtel report identified Case 3 - building a new powerhouse and maintaining the existing powerhouse as the best option for new or expanded electrical generation at Long Lake Dam. The water flow required to operate the new powerhouse will reduce the amount of water flow that is currently going over the spillway. As originally outlined in the Bechtel Report and supported by the TURBNPRO™ software, the new powerhouse (with associated site infrastructure constraints) should optimally contain two (2) vertical (shaft) Francis turbines with 60 MW name plate capacity. The best efficiency that can be achieved by the vertical Francis turbine is 93.8% at approximately 4,000 cfs flow/penstock or 90.1% at 4,800 cfs flow/full flowing penstock.

In the 1990 Bechtel Report, the electrical generation for two 60 MW vertical Francis generators in conjunction with two 17.9 MW double runner horizontal Francis Turbines is approximately 672,268 MWH-annual. Utilizing the TURBNPRO™ software, the same turbine package (2-vertical and 2 horizontal Francis turbines) generates approximately 769,219 MWH-annual. The difference between 1990 and 2010 model is approximately 96,951 MWH-annual of electrical generation.

4.5.6 TDG Performance Estimates

The new second powerhouse would provide an additional 9,600 cfs of powerhouse capacity. As a result, the spillway flow at the 7Q10 would be reduced from 25,200 cfs to 15,600 cfs since both the existing and a new powerhouse would operate under those flow conditions. The discharge from turbines retains the TDG levels from the forebay. The additional turbine capacity would result in some TDG reduction in the tailrace; however, field data (Baseline 2) show that even at a spillway flow of 8,400 cfs, the TDG levels are still above the 110% standard as a result of the plunging flow at the spillway.

5 GEOTECHNICAL REVIEW

The Long Lake HED site is underlain by slightly to moderately altered, coarse-grained granitic which crop out as near-vertical, narrow ridges in parallel sets. Jointing and shearing of the granitic have accelerated the weathering process which is evidenced by the talus deposits located on both sides of the ridges (Bechtel, 1990). The granitic encountered at the Long Lake site have random, irregular joint patterns. Several joint patterns can be identified in surface exposures with non-specific exposure dominating. Overburden materials consist of river deposits, talus, and fill material.

Existing as-constructed drawings, construction photos, and field observations were used to determine the geotechnical considerations for design and construction of the proposed TDG alternatives. As part of a site visit conducted on March 22, 2010, our team geotechnical engineer, Kim de Reburtis, pointed out the geologic considerations of the Long Lake Dam site. A fault can be observed in the existing rock slope immediately downstream from the existing dam. The fault is visible on the north bank of the tailrace and appears to trend upstream under the spillway dam, cross under the forebay, and continue through the west rock abutment immediately upstream from the powerhouse intake dam. A low saddle found on the west abutment of the reservoir is further evidence of the fault.

It is important to consider the fault location when considering TDG alternatives. The hydraulic design of the TDG alternatives should avoid directing high velocity flow directly at the exposed fault line in the project tailrace. High energy flow will result in continued degradation and erosion of the exposed rock surface leading to continued erosion and potentially unstable slopes. The hydraulic design should provide a physical geometry that dissipates and redirects energy away from the north bank of the tailrace. The tailrace area immediately downstream from Spillway gates 3 through 6 has experienced continued erosion over time due to spillway discharges of high energy flows. When the tailrace was dewatered in 1990, evidence of erosion of the spillway toe was found as well as rock erosion on the east and west abutments of the spillway dam. The hydraulic design of Alternatives 1, 2 and 3 which are located in Spillway Bays 7-8 and 1-2, should consider the potential impact of high velocity flow on the existing rock surfaces.

In general, the existing rock found on the spillway abutments is hard and durable. Construction of Alternatives 1, 2, and 3 will require rock removal to support the deflector construction as well as modify the downstream channel to improve hydraulic flow characteristics. Removal of excess rock in both abutments is not considered to be a stability problem, though the removal processes will have to be carefully considered to ensure the dam stability and safety during construction. Additional design analysis and rock removal techniques will be considered during the alternative development. Construction of these concrete structures proposed with these three alternatives will require exposure and preparation of the rock foundation and abutment material. All loose rock on the exposed foundation and walls abutments will have to be removed. Rock anchors will be required to tie the new concrete structures to the abutment floor and walls. For Alternative 3, the training wall located on the west side of the spillway deflector channel will have to be tied to the exposed rock abutment. Provisions for collecting rock debris, which will naturally displace from the exposed rock formations above the channel, will have to be included in the channel design.

For Alternatives 4 and 5, new concrete structures and pipelines are proposed for the natural ravine which extends from the existing cut-off dam to the tailrace area downstream from the existing powerhouse. Construction through this area will be very difficult due to the loose rock abutments found through the narrow ravine section and limited access. In past studies (Bechtel 1990; EES 2006), a new dam was proposed approximately 300 ft north of the existing cutoff dam. A similar approach was proposed in the preliminary layouts for the current study Alternatives 4 and 5. For Alternative 4, the new dam would consist of a spillway gate structure containing two gates used to control the flow into a new chute spillway structure. The chute spillway would then continue downstream through the existing ravine discharging through a flow deflector structure into the project tailrace. The new powerhouse proposed under Alternative 5 requires a new intake dam located at the same location.

In 1990, a preliminary geological investigation to determine the depth to bedrock, the bedrock profile, and rock quality to support preliminary design of the proposed facilities was conducted. A geologic investigation report was developed as part of the Long Lake Expansion Final Plan (Bechtel, 1990). This preliminary geologic investigation acquired data from geologic site survey, a seismic refraction survey, and core drilling along the proposed project alignment. The preliminary conclusions were:

- All proposed structures will be constructed on granitic material which is located within a seismically and geologically stable area.
- For the intake structure, bedrock was encountered at approximate elevations of 1489 feet near the right abutment and 1471 feet near the left abutment. The bedrock was determined to be good foundation quality, though the fractured nature of the rock will require temporary supports for excavations and foundation treatment will be required.
- For the penstocks and chute spillway, seismic refraction data indicated that good quality rock exists along the penstock alignment at depths averaging 15 feet.
- At the proposed powerhouse location, bedrock was encountered at elevations ranging from 1384 to 1310 feet. Overburden consists of fill materials of sand, boulders and blocks, and river deposits of sand, gravels, and boulders.

Overall the preliminary geologic investigation supported the location and proposed design approach of the Alternative 4 and 5 facilities. Additional field investigations consisting of a more robust geologic mapping effort, additional subsurface investigations at the proposed structures location, and more refined selection of the powerhouse location was recommended.

During the March 22nd site visit, concern was raised by the design team members about the feasibility of Alternative 4, and potentially Alternative 5 due to potential geologic, foundation, and constructability issues. Observation of the exposed rock abutments led to the possible conclusion that utilizing the existing cutoff-dam for the spillway gate structure and penstock intake structure may be a more feasible design approach than constructing a new structure at the proposed location 300 feet north of the existing dam. Based on the review of the 1990 study, it appears that either the existing cutoff dam or the proposed new location is feasible. Both locations could provide benefits in terms of constructability and construction sequencing. The viability of each alternative will be investigated and evaluated as the design advances. The preliminary geologic investigation indicate that suitable bedrock is available relatively close to the existing ground surface upon which to construct either the new chute spillway or the penstocks.

6 CONSTRUCTION COST ESTIMATES

This section presents the estimated construction costs for the five TDG alternatives presented in the previous sections. The cost estimates were developed based on the conceptual design drawings and details developed as part of this study, as well as data developed in previous projects and updated to include recommended design modifications, updated unit costs, and changes in the site conditions which resulted in variations in the cost estimates. Appendix E presents the itemized cost estimate summary spreadsheets, calculations, and assumptions.

6.1 COST ESTIMATING METHODOLOGY (INCLUDING BASIS OF UNIT COSTS)

In considering cost estimate preparation, it is important to realize that changes during final design as well as in the cost of materials, labor, and equipment will cause considerable changes in the cost estimates presented within this report. A good indicator of changes in the construction cost is the Engineering News-Record (ENR) Construction Cost Index. Cost data presented within this report are based on an ENR Construction Cost Index, 20 City Average, of 8761.47 for 2010. Cost data can be adjusted to anytime in the past or future by applying a ration of the prevailing ENR Index to the current index of 8761.47.

The construction cost data presented within this report is not intended to be the lowest cost for completing the work. Instead, the costs represent the median costs that would result from responsible bids received from qualified contractors. The unit costs presented within this report were derived from a number of sources. Material costs for much of the work were obtained though project experience on Pacific Northwest projects of similar size and complexity.

6.2 ESTIMATES FOR EACH ALTERNATIVE

Table 6-1 presents a summary of the total project cost associated with the five alternatives considered for TDG reduction at Long Lake Dam. The total project cost includes materials and construction, planning, engineering, environmental and permitting support, and supervision and administration during construction. A brief summary of the major project cost elements are presented in the following paragraphs:

Materials and construction consisting of the labor, equipment, and materials required to construct the alternative features.

Planning required to develop the project from conceptual to a point where detailed engineering construction plans and specifications can be prepared. Typically, planning costs would include the internal Avista management and administration costs, conceptual design development, and related work activities.

Engineering consisting of the calculations, drawings and technical specifications required to support bidding and construction.

Environmental and permitting costs are associated with all projects. Environmental requirements may include cultural surveys, wetland surveys, preparation of environmental documents, and review of environmental impacts to support the project permitting.

Coordination with the resource agencies including WDFW, Washington Department of Ecology, USFWS, and participating stakeholders will be required.

Construction supervision and administration (S&A) consists of the staff and resources required to administer the project construction.

Real estate acquisition costs were not included in the cost estimate since all of the project alternative features fall within Avista property boundaries.

Table 6-1 Summary of Construction Cost Estimates

Item	Cost ($)				
	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5
	Spill Bay 7/8 Deflector	Spill Bay 7/8 Deflectors with Training Walls	Spill Bay 1/2 Deflectors	Cut-off Dam Spillway	New Powerhouse
Construction	$4,126,076	$12,912,540	$12,146,867	$30,754,852	$130,762,100
Planning (5%)	$206,304	$645,627	$607,343	$1,537,743	$6,538,105
Engineering (10%)	$412,608	$1,291,254	$1,214,687	$3,075,485	$13,076,210
Environmental and Permitting (5%)	$206,304	$645,627	$607,343	$1,537,743	$6,538,105
Construction S&A (4%)	$165,043	$516,502	$485,875	$1,230,194	$5,230,484
Subtotal	$5,116,334	$16,011,550	$15,062,115	$38,136,016	$162,145,004
Contingency	30%	30%	30%	30%	30%
Total Project Cost	$6,651,235	$20,815,014	$19,580,750	$49,576,821	$210,788,505

6.3 Level of Accuracy and Contingency Discussion

A planning level cost estimate, as defined by the American Association of Cost Engineers, is appropriate for a conceptual design study. A planning level cost estimate is prepared based on the level of detail presented within a conceptual design study and drawings and generally includes preliminary estimates of quantities and equipment requirements. A planning level estimate has en expected accuracy of ±30 percent. All estimates presented within this report are planning level estimates.

Planning level cost estimates were prepared for the conceptual design alternatives presented in this report, and they are summarized above in Table 6-1 of this section. The design alternative cost estimates were based on the drawings developed for each alternative. Changes in the alternative function, layout, materials, and pricing will affect the construction costs. The labor burden, insurance, overhead, profit, bond, and sales tax are included as separate line items within these estimates. The cost of engineering, construction management, permitting, and environmental compliance was not included in the construction capital cost. These cost items are included as separate lines items in Table 6-1.

At a conceptual design level, many of the design elements are not detailed to a sufficient level to allow advanced quantity takeoffs. Examples of these design elements include, but are not limited to: water control gates, electrical/instrumentation, and piping. Cost estimates for these features were obtained from similar projects where detailed cost estimates were developed or as-constructed cost estimates were available. It is expected that where possible conventional construction methods will be utilized in the construction of these facilities.

6.4 Construction Sequencing and Assumptions

Alternatives 1 through 4 consist of modifications to the existing Long Lake Dam facilities to incorporate TDG reduction measures. Alternatives 1 through 3 focus on adding flow deflectors to the existing spillway to improve flow conditions entering the tailrace. Alternative 4 consists of a new intake structure and chute spillway extending from the existing cutoff dam to the project tailrace. For these alternatives, preliminary design drawings were developed outlining the major features, alignment, and elevations. Quantity takeoffs were obtained from these drawings to support development of the construction cost estimates. A construction sequencing plan, assumptions, and schedule were developed for each of these alternatives to support the cost estimating process. Section 7 presents the construction schedule, sequencing, and assumptions for Alternative 1 through 4.

Alternative 5 consists of a new powerhouse constructed adjacent to the existing powerhouse. With this alternative, a new intake structure would be constructed approximately 300 ft downstream from the existing cutoff dam. Two new steel penstocks would extend from the intake to the new powerhouse. The powerhouse alternative was based on the preliminary design layout developed by Bechtel in their 1990 Feasibility Report (Bechtel, 1990). Within this study, Bechtel completed a comprehensive review of a wide range of alternatives for upgrading the existing powerhouse, a new stand alone powerhouse, and a combination of both. In 2009, Hatch Acres updated the Bechtel cost estimates. The Hatch Acres estimate was used as the basis for the total project costs. The drawings presented within this report were obtained from the Bechtel feasibility study. No additional design details were developed as part of the current study effort.

7 CONSTRUCTION OF ALTERNATIVES

This section presents a discussion of the potential construction issues, construction sequencing, and preliminary construction schedules anticipated for each of the five alternatives presented within this report. The required timeline and effort associated with each alternative for planning, engineering, environmental and permitting will vary. Alternatives 1 through 3 would be expected to have the shortest durations for these project elements. Alternatives 4 and 5 would have the longest periods.

7.1 CRITICAL CONSTRUCTION ISSUES

The Long Lake Dam site provides several challenges in executing construction of the TDG alternatives presented within this report. These include, but are not limited to, the following:

- Site Access/Topography
- Proximity to the Existing Dam
- Site Geological Composition
- Removal/Relocation of Excavated Material
- Large Quantities of Construction Materials
- In-Stream Work Window
- Spillway Capacity

These issues can be presented in three key categories: (1) construction site constraints, (2) material supply constraints, and (3) in-stream work window constraints. Each of the categories will be discussed in more detail in the following sections.

7.2 CONSTRUCTION SITE CONSTRAINTS

The Long Lake Dam topography and location presents several issues that directly affect the manner in which construction of the design alternatives can be executed. The issues include site access and topography, proximity to the dam, site geological composition, and removal and relocation of excavated material.

Access to the construction sites for Alternatives 1 through 3 is challenging due the steep site topography and location of the existing powerhouse. Steep canyon walls are present on the north, east, and west sides of the spillway tailrace, with the spillway structure to the south. The location of the powerhouse, on the west side of the spillway, also blocks direct access to the construction site for Alternatives 1 through 3. Several access options were developed to transport workers, materials, and equipment to the construction sites. Option 1 consisted of launching a barge from downstream of the existing powerhouse and moving it upstream to the spillway. A land based winch system would be used to pull the barge from the powerhouse tailrace to the construction site. Option 2 consisted of launching a barge into Long Lake (Lake Spokane) with a crane designed to lift materials over the existing spillway structure including the existing spillway gate lift frames. An in depth analysis would be required to determine if a crane could provide both the desired reach as well as height to clear the spillway gate structures. It should also be noted that the closest boat ramp is approximately 3 miles upstream from Long Lake Dam which could create long transport times. The structural capacity of this ramp to handle the size of equipment required for construction is unknown at this time. The third access option explored was the installation of a temporary tram from the existing visitor's center overlooking the tailrace from the east. Although feasible, Option 3 is perceived to be high cost and less effective than the other two options. The potential environmental impacts and resulting permitting requirements could also make this approach difficult to implement. After careful consideration, Option 1 was deemed the most efficient and cost effective approach for accessing Alternatives 1 through 3.

The site topography adds complexity, other than site access, to both the design and construction for each of the alternatives. For Alternative 4 this becomes evident when considering construction of an access road over the new chute spillway. Construction of the new chute spillway will effectively isolate the existing powerhouse from the main tailrace access road. The topography of the site is such that a road with a very severe grade is the only means in which to ramp up to a bridge across the new chute spillway. In addition, the option of an underpass was explored. This option soon proved to be infeasible because of the depth of the road surface required for adequate clearance would be well into the ground water, which would flood the underpass.

Site access for Alternatives 4 and 5 is relatively straight forward. Alternative 4 construction would begin at the new intake structure located approximately 300 ft downstream from the existing cut-off dam and work back towards the existing powerhouse. This would allow the access road to the new intake to be constructed simultaneously with the new spillway channel. Access for Alternative 5 would be provided via the existing powerhouse access with some modifications potentially required to the existing access road alignment. For both of Alternatives 4 and 5, the new access road construction will be difficult and expensive to construct.

Alternatives 1 through 3 would require demolition of a portion of the existing spillway structure as well as varying levels rock excavation near the spillway. This may require alternative demolition techniques to be utilized to prevent excess disturbance or vibrations of the existing structures. For Alternative 1, a large portion of the existing rock knob on the west side of the spillway must be removed. Traditionally this would be done using drill and blasting techniques. Due to the close proximity of the spillway, extreme caution must be used in order to determine the appropriate charge size and location. For Alternative 3, the rock slope on the east side of the spillway must be stabilized during construction. This would require removing all unsecure rock from the slope. At a minimum, manual high scaling would be required to ensure all loose rock which could impact worker safety would have to be removed. A permanent rock fence may also be required to protect the completed structure. Excavation is also required to create an adequate area for installation of the flow deflector. Similar issues with blasting are present as mentioned previously for Alternative 1.

Alternatives 4 and 5 present a unique challenge with the removal of the existing cut-off dam. Review of the FERC Part 12 reports indicate this structure has experienced varying levels of concrete deterioration and potential stability issues. Once the new intake structure construction is completed, the existing cutoff dam will be removed. One approach to accomplishing this work effort is to water up the area between the existing cutoff dam and the new intake structure. This could be accomplished by removing a small top section of the existing cutoff dam to allow a controlled water flow. Once the water level has been stabilized on both sides of the cutoff dam, the structure could be removed in sections.

For each of the design alternatives it is important to understand the localized geological composition of both the material being removed and the material upon which the structures will be supported. All the structures must have a secure foundation to prevent movement or instability during peak flow and extreme operating conditions.

For Alternatives 1 through 3, the new structures will be founded on the existing spillway and rock formations within the tailrace, which will provide a stable foundation. Localized pockets of loose or substandard rock may have to be removed to ensure firm foundation conditions. For Alternatives 4 and 5, previous rock exploration and evaluation (Bechtel, 1990) indicate that competent bedrock is located under the cobble and gravel overburden. For the intake structure, the overburden material will have to be removed down to competent bedrock to provide a suitable foundation for the intake structure. Pressure grouting of the intake foundation will also be required to fill any fissures and voids preventing seepage under the dam or through the abutments. The overburden material located between the new intake structure and project tailrace will also have to be excavated to expose the bedrock for anchoring the penstock concrete anchors, and placement of the powerhouse. For the chute spillway under Alternative 4, careful consideration of the anchoring design method will be required to ensure the structure does not experience excessive lateral or vertical movement. Extensive subsurface explorations will be required to confirm the geotechnical conditions and design criteria for the new structures.

As previously discussed, disposal of excavated material will be required for all the alternatives. In Alternatives 1 through 3, it was assumed that the material can be placed in the northern most area of the tailrace creating a more sweeping channel that directs flow downstream. Any excess material that cannot be relocated on site will have to be transported via the barge system and disposed of at an offsite location. For Alternatives 4 and 5, suitable excavated material can be used as backfill, construction of both the access road for the new intake structure, and the new bridge ramps.

7.3 MATERIAL SUPPLIES CONSTRAINTS

Due to the limited open area within the Long Lake Dam project, the storage and handling of the large quantities of materials required for the construction of the proposed design alternatives will make materials delivery and storage a critical scheduling issue. In order to control the delivery time and quality of concrete, the most viable option may be to construct an onsite batch plant. For Alternative 1 through 3, the batch plant would be located adjacent to the powerhouse. A concrete pump(s) and pipeline would be used to delivery concrete from the batch plant to the concrete forms at the construction site. This approach could lower the concrete cost as well as eliminate the potential for rejection of concrete loads due to long truck runs from the offsite batch plant. Due to the small on site space available, rebar and other materials would have to be ordered and delivered to the site on a weekly basis corresponding with the construction schedule. This timing and schedule must be highly detailed such that construction can be executed with minimal delays.

7.4 In-Stream Work Window Constraints

In-stream work windows are implemented and enforced to minimize impacts on aquatic species and their habitats. The Washington Department of Fish and Wildlife (WDFW) define the fresh water work windows within the state. The in stream work window for the Spokane River, as of March 2009, is June 16th through August 31st. Therefore, the construction site would have to be completely isolated from river flows using a cofferdam within this window. We have assumed that construction could then occur outside of the in-stream window as long as the work activities were confined behind the cofferdam walls. Once construction is completed, the cofferdam could then be removed. A temporary, limited duration extension may be issued by WDFW to complete the cofferdam removal.

7.5 Spillway Capacity Constraints

The construction process and sequencing will also have to be developed to accommodate flows within the Spokane River above Long Lake Dam. Analysis of river gauge number 12433000 shows that flows above the powerhouse capacity of 6,800 cfs can be present into July, which is well into the WDFW in-stream work window. The river flows increase in November through early July period resulting in spill conditions which could impact a cofferdam located in the project tailrace areas. As a result, careful consideration of the river flows will have to be incorporated into the construction sequence and schedule to ensure the work site is protected during spill conditions, or the work is completed during the non-spill periods.

7.6 Construction Schedule for Each Alternative

Preliminary construction sequencing plans and schedules were developed for each of the five alternatives presented within this report. The construction sequencing plans are illustrated on Drawings 105, 205, 305, 408, and 502 (Appendix D) for Alternative 1 through 5, respectively. The construction schedules are presented in Appendix F. It should be noted that the construction sequencing and schedules for Alternatives 1 through 3 were developed to ensure the construction work and concrete placement was advanced to a point to allow the main tailrace cofferdam to be removed. This will ensure the new structures will not be damaged during a spill event. For Alternative 4 and 5, the proposed structure's can be constructed with minimal impact to the existing powerhouse structure and no impact to the existing spillway. The construction schedule for these two alternatives can be more relaxed with the only restrictions related to the in-water work windows. A brief discussion of the anticipated construction sequencing and schedule of each alternative is presented in the following paragraphs.

7.6.1 Alternative 1 – Spill Bay 7-8 Deflectors

Construction for Alternative 1 would begin with mobilization of the necessary equipment and materials to the job site. This would require the necessary equipment and crews to relocate to the job site and set up all temporary construction buildings, material yards, and staging areas. It is estimated that this process would take about two weeks. Erosion and sediment control measure would be installed along with any project safety and construction fencing designed to maintain the project security. The proposed barge and related winches and equipment would be transported to the site.

As a first step, a survey crew will locate and stake key locations for construction. This would include, but is not limited to, required blasting areas, concrete deflectors boundaries, and cofferdam/bladder bag dam locations. This ensures that all features, both temporary and new, will be located correctly to allow for the most efficient and precise completion of the construction project.

The barge transport system would be installed and tested to provide access to the construction site. The barge would be launched from the bank into the project tailrace. The barge launching would have to be coordinated with the powerhouse operation to ensure safe launching conditions. The barge would be designed to provide the largest load capacity possible with the shallowest draft. This would ensure the barge can transport the required materials and equipment to the construction site without impacting the channel bottom. A temporary access dock may be required at the construction site to allow safe and efficient unloading of materials and equipment from the barge. An access road and platform must be constructed to allow the drill rig to access the work area. This access road would have to be done by hand since no equipment would be able to access the blast site.

Before any blasting or demolition of the existing spillway and rock knob can occur, the construction area must be dewatered. This would require the placement of either a cofferdam or bladder bag dam within the tailrace. The tailrace/construction site can then be dewatered using a pump system. The water would be lifted from the tailrace, over the temporary cofferdam, and discharged into the river downstream of the construction site. The construction of the selected dam and dewatering of the tailrace must be completed during the in-stream work window as described in Section 7.4 (mid-June through end of August). For the initial dewatering period, it was assumed that the water quality would be adequate to allow direct discharge into the river downstream from the temporary cofferdam. When the tailrace has been completely dewatered to expose the spillway toe and rock foundation, the leakage through the dam structure will be collected and pumped through a pre-packaged sediment structure such as manufactured by Rain-for Rent. During ground disturbing activities such as rock excavation, all water collected within the active construction zone will be treated through the sedimentation basin prior to discharge.

Once the dewatering system is completed, the drill rig and excavating equipment can be launched to the blasting site. The rock knob will then be drilled and shot in 10-ft lifts. Particular attention will have to be paid to the depth and location of the blasting loads to prevent excess seismic loads on the existing spillway and powerhouse structures.

Excavated material from the existing rock knob will be relocated to the northern most area of the tailrace. The material will be transported to the fill area by a haul truck or conveyor system. A temporary road would be constructed which travels across the dewatered tailrace allowing the haul truck to travel from the rock knob to the fill area. Once material is transported to the tailrace deposit area, it will then be assembled in such a manner to provide structural stability and prevent erosion or transport of material downstream during spill events.

When the rock knob has been excavated to the desired elevation and shape, crews will then be mobilized, via the temporary barge system, to the downstream face of the existing spillway. These crews will remove the existing face of the spillway as required to support construction of the new flow deflector. This excavated material, similarly to the rock knob, will be transported and deposited to the northern most corner of the tailrace.

The freshly exposed portion of the spillway must then be prepped to receive the new concrete deflector. This will be done by removing any loose or unsecure material still remaining on the excavated area. In addition holes will be drilled and dowels will be installed into the excavated face and secured with epoxy. These dowels will provide a structural anchor for the new deflector concrete and will prevent the new concrete from detaching during high flow events.

Once the existing spillway is prepped to receive new concrete, the new spillway deflector will be constructed. This will be done by placing concrete forms in segmented lifts to obtain the desired design profile for the deflector. Concrete pumps will be mobilized to transport the concrete from the area adjacent to the powerhouse to the flow deflector construction area. The pump line will be routed around the backside of the powerhouse, under the penstocks, and into the construction area. The concrete will then be pumped into the appropriate lifts to construct the new spillway deflector. The upper two feet of concrete, exposed to the spillway flows, will be constructed using high performance concrete to improve wear properties and increase the life of the deflector structure. Once all the concrete has been placed, the forms will be removed and any surface finishing work that is required will be completed. It should be noted that the construction schedule will have to be developed to ensure the entire concrete deflector construction is completed by mid-November to ensure spill conditions will not submerge the new structure, or a temporary cofferdam will have to be constructed around the deflector work area. The main tailrace cofferdam will have to be removed prior to high fall flows beginning in November.

Simultaneously to the concrete work on the new deflector, the earthwork and drilling equipment used for removal and excavation of the existing rock knob and spillway can be demobilized from the job site. The equipment will be moved from the dewatered tailrace using the temporary barge system.

After completion of the new concrete deflector, all remaining equipment in the tailrace will be removed via the temporary barge system. This will allow for the removal of the temporary cofferdam/bladder bag dam, flooding the tailrace. This removal process will require either an in-stream work window violation permit, or it will be done during the previously mentioned work window.

Once the tailrace has been filled and the deflector has been tested, demobilization from the job site will occur. This will require removal of the temporary barge system and any other remaining equipment from the job site, including temporary construction buildings and staging areas. Overall, it is estimated that construction of Alternative 1 would require less than 1 year from contract award to completion with the in-water work completed in one season.

7.6.2 Alternative 2 – Spill Bay 7-8 Super-elevated Spillway Extension

Similar to Alternative 1, construction would initiate with mobilization of the necessary equipment and materials to the job site. This would require the equipment and crews to relocate to the job site and set up all temporary construction buildings, material yards, and staging areas. It is estimated that this process would take about three weeks. Erosion and sediment control measure would be installed along with any project safety and construction fencing designed to maintain the project security. The proposed barge and related winches and equipment would be transported to the site.

The construction sequence for erection of the barge system and cofferdam would be identical to that outlined for Alternative 1. In addition to the barge system assembly, an onsite batch plant can be constructed. An onsite batch plant was selected to ensure high quality concrete will be delivered to the concrete forms as required. Long transport times from the closest concrete batch plant could result in potential delivery and quality issues. A concrete pump and pipeline system would be installed to pump the concrete from the batch plant to the construction site.

Once the tailrace and construction site are dewatered, the drill machine, scaffolding, excavator, haul truck, and rail platform can be barged into the tailrace. The equipment would be transported via the temporary barge system.

The drilling rig can then begin to drill holes for blasting the required excavation area of the existing spillway. Extra care must be taken while blasting to ensure no excessive loads are distributed into the existing spillway potentially making it unstable. As material is removed from the existing spillway, it will be transported across the dewatered tailrace to the north most corner of the tailrace. Once deposited in the fill area, the excavated material will be positioned to ensure stability and prevent erosion during future tailrace operation. Once all of the excavation and transport of material is completed the earthwork equipment can then be demobilized from the job site.

Once the required excavation of the existing spillway is completed, the scaffolding and rail platform can be erected. The scaffolding and rail platform will allow workers to perform the work required to construct the new concrete flow deflectors for spillway bays 7 and 8.

Prior to construction of forms, the existing spillway must be prepped to receive new concrete. This will involve removing any loose or unstable material remaining on the existing spillway. As well, steel dowels will be installed into the existing spillway face. The dowels will be placed into a drilled hole and held in place using epoxy.

Once the existing spillway is prepped, the new spillway deflector will be constructed. This will be done by placing concrete forms in segmented lifts to obtain the desired design profile for the deflector. Pump trucks will be simultaneously mobilized to the existing tailrace access area during form placement to ensure that concrete can be placed once the forms are ready. The pump lines will be floated across the existing tailrace and over the rock knob to the deflector site. The concrete will then be poured in the above mentioned lifts to construct the new spillway deflector. The lifts will be designed such to minimize down time while waiting for concrete to cure. This can be done by pouring nonadjacent sections simultaneously. The upper two feet of concrete exposed to the spillway flows will be constructed using high performance concrete to improve wear properties and increase the life of the deflector structure. Once all the concrete has been placed, the forms will be removed and any surface finishing work that is required will be completed. Next, the scaffolding and rail platform will be removed from the deflector.

After completion of the new concrete deflector, all remaining equipment in the tailrace will be removed via the temporary barge system. This will allow for the removal of the temporary cofferdam/balder bag dam, flooding the tailrace. This removal process will either require an in-stream work window violation permit, or be done during the previously mentioned work window. It should be noted that the elevated spillway design allows the temporary cofferdam to be removed as soon as the elevation of the concrete pour exceeds the selected design tailwater elevation. This allows the construction window to be extended into the winter months, if required. Operation of the barge system may be restricted during large flood events, but these delays would be expected to be short in duration.

Once the concrete deflector has been constructed, demobilization from the job site will occur. This will require removal of the temporary barge system and any other remaining equipment for the job site, including temporary construction buildings and staging areas. Overall, construction of Alternative 2 is estimated to require less than 1 year from contract award to completion.

7.6.3 Alternative 3 – Spill Bays 1 and 2 Toe Modification and Downstream Deflector

The initial project mobilization and setup will be identical for Alternative 3 as outlined for Alternatives 1 and 2. The biggest challenge with this alternative is that the work effort is on the opposite side of the tailrace channel. Access to the work area from the rock knob will be required to support excavation and concrete placement. Installation of the temporary cofferdam and dewatering system is identical to the previous alternative descriptions.

Once the tailrace and construction site are dewatered, the drill machine, excavator, and haul truck can be barged into the tailrace. The equipment would be transported via the temporary barge system. With the construction site is dewatered and all key locations have been surveyed and staked, the drilling and shooting can begin for the east abutment and existing spillway. This process would remove all material required to accommodate the new concrete deflector as well as eliminating any loose material on the abutment. As rock is blasted, crews would be able to simultaneously relocate the material to the north corner of the tailrace. An excavator and haul truck or conveyor would be used to complete the transportation of the excavated material.

Once all excavation is completed, all formwork, falsework, and scaffolding can be transported to the job site via the temporary barge system. Crews would then erect the scaffolding and begin prep work for the placing forms. This would include any finish work required to the exposed rock face and spillway as well as the installation of dowels. The dowels will be placed into a drilled hole and held in place using epoxy. Simultaneously the earthwork equipment used for excavation can be demobilized from the job site using the temporary barge system.

Once the exposed rock and spillway is prepped, the new spillway deflector will be constructed. This will be done by placing concrete forms in segmented lifts to obtain the desired design profile for the deflector. Pump trucks will be simultaneously mobilized to the existing tailrace access area during form placement to ensure once forms are ready concrete can be placed. The pump lines will be routed along the backside of the existing powerhouse over to the construction site. The concrete will then be pumped into the above mentioned lifts to construct the new spillway deflector. The lifts will be designed such to minimize down time while waiting for concrete to cure. This can be done by pouring nonadjacent sections simultaneously. The upper two feet of concrete exposed to the spillway flows will be constructed using high performance concrete to improve wear properties and increase the life of the deflector structure. Once all the concrete has been placed, the forms will be removed and any surface finishing work that is required will be completed. Once the finish work is completed the scaffolding can also be removed from the construction site.

After completion of the new concrete deflector, all remaining equipment in the tailrace will be removed via the temporary barge system. This will allow for the removal of the temporary cofferdam/balder bag dam, flooding the tailrace. This removal process will either require an in-stream work window violation permit, or be done during the previously mentioned work window.

Once the deflector has been completed, demobilization from the job site will occur. This will require removal of the temporary barge system and any other remaining equipment for the job site, including temporary construction buildings and staging areas. Overall, construction of Alternative 3 is estimated to require less than 1 year from contract award to completion.

7.6.4 Alternative 4 – Cut-off Dam Chute Spillway with Deflector

Construction for Alternative 4 would begin with mobilization to the job site. This would require the necessary equipment and crews to relocate to the job site and set up all temporary construction buildings, material yards, and staging areas. It is estimated that this process would take about three and a half weeks.

Overall, the construction would be executed from upstream to downstream starting first with the new intake gate structure, then proceeding downstream with the chute spillway. This approach will maintain access to the gate structure for the heavy equipment required to excavate the structure foundation, erect rebar and place concrete, and installation of the intake gates. During the in-water work window, a cofferdam will be constructed in the tailrace for the outlet channel and deflector work. A portion of the chute spillway will be left open to provide unimpeded access through the entire construction period. When the new bridge and access road is complete, the chute spillway can be completed.

Similar to the previous alternatives, an on-site batch plant will be erected. Utilizing an on-site facility will ensure an efficient delivery of concrete to the forms as well as maintain a high quality product. Aggregate will be imported to support the concrete batching process.

Simultaneously to the batch plant construction, a survey crew will locate and stake key locations for construction. This would include, but is not limited to, required excavation areas, concrete spillway channels, and the intake structure. This ensures that all features, both temporary and new, will be located correctly to allow for the most efficient and precise completion of the construction project.

Excavation of the new spillway channel and intake structure would be one of the first work elements. As discussed previously, the construction sequence would start at the intake structure then move downstream towards the river outlet. Structural fill would be used as required during the excavation process. As excavation and fill continues, crews will begin forming and erecting the steel reinforcement for the intake structure. The pour sequence will be developed to provide a minimum 7 day pour back time while maximizing daily concrete placement volumes as much as possible.

In order for crews to construct the final portion of the new spillway channel, a cofferdam must be installed within the tailrace. This would allow workers to finish any required final earthwork to accommodate the channel. The cofferdam would have to be installed, and the construction zone dewatered, during the in stream work window as described above in section 7.4.

Once the concrete for the intake structure is finished, the forms will be stripped from the intake structure and the concrete will then begin to be poured for the new spillway channel. The crews will work from the new intake structure towards the tailrace in a similar fashion to how the channel excavation and fill was performed. Access will be maintained from the downstream side of the work area. The permanent access road will be constructed when the chute spillway is completed.

The intake gates and steel stoplogs are the only long lead items associated with Alternative 4. Procurement packages for these items will be issued prior to mobilization to allow delivery and installation as soon as the concrete placement work efforts are complete. This work effort will require a large crane for erection. With early delivery of the gate system, the construction crane will be used to install the gates; and, then it will be moved out of the work area as the chute spillway construction advances.

As concrete cures in the new spillway channel, crews will follow behind and strip forms from the channel, install French drains along the sides of the channel, backfill behind the spillway walls, perform the necessary earth work for both the intake structure access road and the new vehicle bridge over the spillway channel, install chain link fence on the south side of the channel, and install parapet structures between the new access road and diversion channel.

Once the work on the new spillway channel is completed at the bridge section, construction of the new vehicle bridge will begin. Backfill on the south side of the channel will be required to create the necessary severe ramp up to the bridge. Additional earthwork will be required to ensure the landing is sloped at a grade that catches with the existing powerhouse access area in the allotted spaces. Once the bridge girders and piles have been installed, the concrete roadway will then be placed. Gabion walls will be used where necessary to accommodate the earthwork required for driving the bridge piles as well as the access road.

Construction of the outlet structure can be moved forward concurrently with the intake gate structure. Maintaining access to the construction area as well as the existing powerhouse will be the biggest challenges during construction. Once the outlet channel is completed, the cofferdam can be removed; and, the final closure section of the chute spillway will be constructed along with the access bridge as discussed previously.

The final tasks of the construction process will include performing any required dry lands restoration to ensure stability of the ground surrounding the newly constructed spillway channel and intake as well as any required signage for the new vehicle bridge and intake structure access road.

Once all of the required construction and restoration tasks have been completed the construction crews will demobilize from the job site. This will require removal of the temporary barge system and any other remaining equipment for the job site, including temporary construction buildings and staging areas. Overall, construction of Alternative 4 is estimated to require less than 2 years from contract award to completion and represents no impact on spillway capacity during the construction period.

7.6.5 Alternative 5 – New Powerhouse

Construction of the new intake structure, penstocks, and powerhouse facilities associated with Alternative 5 will require a very similar construction sequence to that described for Alternative 4. The biggest difference between the two alternatives is that Alternative 5 has a number of long lead items which will require extensive pre-planning and advanced procurement. The intake gates, penstocks, turbine, generators, and associated equipment will require anywhere from 12 to 24 months to fabricate and deliver to the site. Consequently, the construction schedule will have to be carefully developed to ensure efficient construction of the on-site concrete structures to support installation of the major equipment packages when delivered to the site.

Similar to Alternative 4, the work effort will start at the intake gate structure. The foundation will be excavated and prepared to support construction of the concrete intake structure. This work effort will include removal of overburden material, removal of unsuitable bedrock material, and foundation grouting. With the foundation work complete, the concrete gate structure will be constructed. The schedule for this work activity will be driven by the pour sequence and required pour back time of approximately 7 days. Due to the limited access, the fabricated gates and stoplogs will have to be delivered and installed prior to installation of the steel penstocks. It is anticipated that the erection crane will install the gates and associated equipment at the intake structure; and, it will be used to begin installing the penstock section on the concrete anchor blocks moving from upstream to downstream.

Construction of the powerhouse structure will be the critical path schedule item. Consequently, it will be important to initiate this work effort as soon as possible in the overall work sequence. It is anticipated that the cofferdam will be installed during the in-water work window. The powerhouse construction will then continue concurrently with the intake gate structure. The penstocks will be buried immediately upstream from the powerhouse providing access to the new and existing powerhouses. With the entire structure complete, the existing cutoff dam can be removed providing water to the completed facilities; and, the cofferdam will also be removed. Startup and testing work can then be initiated. Overall, construction of Alternative 5 is estimated to require less than 3 years from contract award to completion.

8 COMPARISON OF ALTERNATIVES

The comparison of the five alternatives included a qualitative analysis. This process included identifying the advantages and disadvantages of each alternative. Other tools are available to compare alternatives such as a matrix analysis, and the application of a matrix evaluation is discussed and presented for future consideration.

8.1 ADVANTAGES AND DISADVANTAGES OF EACH ALTERNATIVE

The following sections describe the advantages and disadvantages for each of the TDG alternatives.

8.1.1 Alternative 1 – Spill Bay 7-8 Deflectors

Advantages:

- The Alternative 1 deflector concept is similar to the types of deflectors that have been constructed and extensively evaluated on the Columbia and Snake Rivers. This provides a significant amount of precedent for this type of alternative. While there are some unique conditions at Long Lake, including the high unit discharge and the configuration of the plunge pool, a deflector type of concept is the only structural TDG abatement alternative that has been constructed to date that has been shown to be effective at reducing TDG to acceptable levels.
- The construction cost estimate for this alternative is the lowest, and the construction schedule is estimated to be less than one year.

Disadvantages:

- The unit discharge for a deflector downstream of Bays 7 and 8 has a higher unit discharge than what has been shown to be effective for TDG abatement at other projects. The shallow tailrace depth downstream may help compensate for this since the shallow depths help to de-gas the flow.
- The area between the proposed flow deflector and the excavated rock shelf will need to be filled with mass concrete. The high velocities and decreased energy dissipation associated with the operation of the spillway could impact the long term stability of this alternative. In addition, some blasting will be required during construction which adds some risk to the stability of the existing dam during construction.
- Spillway capacity may be reduced during construction of the flow deflector.

8.1.2 Alternative 2 – Spill Bay 7-8 Super-elevated Spillway Extension

Advantages:

- The potential of this alternative to spread the spillway flow over a wide area where the receiving depth is very shallow makes the TDG reduction potential promising.

- Since the jet trajectory will be deflected onto the existing rock outcropping, filling in the plunge pool is not required. In fact, the spillway flow would be directed away from the toe of the spillway, which will reduce the potential for scour at the base of the spillway.
- The construction cost estimate for this alternative are similar to Alternative 3 and are considered intermediate in the range of alternatives evaluated.
- Similar to Alternative 1, the construction schedule is estimated to be less than one year.

Disadvantages:

- The hydraulic design of a flip bucket geometry to optimize the spreading and impact zone of the jet on the existing rock outcropping over the full range of spillway discharges up to the TDG design discharge will be challenging.
- The existing rock outcrop will need to remain stable over the life of the project, and may require some preparatory work.
- Spillway capacity may be reduced during construction.
- This option could result in injury to fish passing downstream over the spillway.

8.1.3 Alternative 3 – Spill Bay 1-2 Toe Modification and Downstream Deflector

Advantages:

- This alternative should be able to provide skimming or undular flow patterns downstream of the spillway which should provide a moderate reduction in TDG
- This alternative would deflect flow away from the base of the spillway, reducing the potential for further erosion in this area.
- The estimated construction costs are considered intermediate and are similar to Alternative 2.
- Similar to Alternatives 1 and 2, the construction schedule is estimated at less than one year.

Disadvantages:

- The geometry of this alternative is limited by the existing topography and bathymetry, making its design more complex and its performance questionable.
- The flow may be directed both parallel and perpendicular to the end of the shelf. Flow perpendicular to the end of the shelf will discharge into the plunge pool.
- Similar to Alternative 1, the high unit discharge resulting with utilization of only two spillway bays makes the TDG performance for this alternative questionable.
- Spillway capacity may be reduced during construction.

8.1.4 Alternative 4 – Cut-off Dam Chute Spillway with Deflector

Advantages:

- This alternative increases the total spillway capacity, which has a positive impact on the overall dam safety.
- This alternative presents no impact on the existing spillway capacity during construction.
- The new spillway discharge for this alternative would be well downstream of the existing plunge pool where the water depths are shallower and there is less potential for flow recirculation. These conditions provide significant TDG benefits.
- Flows up to the 7Q10 discharge will not be passed through the existing spillway, reducing the potential for erosion at the toe of the existing spillway.
- A similar deflector was designed for the high velocity spillway chute at Oxbow Dam on the Snake River; and, the performance in a physical model indicated that it has a relatively high potential to be effective in reducing TDG.

Disadvantages:

- This alternative requires a new spillway structure as well as access to the existing facility, which makes this a large addition to the existing facility
- Although a similar deflector concept has been evaluated in a physical model, there is no known precedent in the field for this type of abatement structure.
- High velocity flow from the spillway may produce erosion in the downstream river channel and along the channel banks.
- The estimated construction costs for this project are approximately twice those estimated for Alternatives 2 and 3.
- The construction schedule is on the order of two years (as compared to less than one year for Alternatives 1, 2 and 3).

8.1.5 Alternative 5 – New Second Powerhouse

Advantages:

- This alternative provides the benefit of producing additional power generation while passing approximately 38% (9,600 cfs) of the 7Q10 discharge.
- There are new hydropower incentives that may be applied for to assist with the financial aspect of the project.

Disadvantages:

- This alternative would only pass a portion of the 7Q10 discharge; therefore, modifications at the existing spillway, or construction of an auxiliary spillway, would still be required to provide TDG reduction for the remaining 15,600 cfs.
- There is a high capital cost associated with building a new powerhouse structure.

- Due to the lead times for the mechanical equipment, new intake and new powerhouse, the construction schedule is the longest for this alternative.

8.2 MATRIX EVALUATION TOOL

Another tool that could be used to compare the alternatives is to utilize a matrix evaluation. Table 8.1 provides an example matrix table with six categories. This type of exercise would be competed after the evaluators have reviewed the alternatives presented in this report.

Table 8-1 Example of Alternative Evaluation/Ranking Matrix

Criteria	Weighting Factor	Alternative Score	
		Raw Score	Weighted Score
TDG Reduction Potential/Exceedance Duration	20%		
Dam Safety	30%		
Feasibility (constructability/risk/confidence)	15%		
O & M Costs and/or Hydropower Benefit	15%		
Construction Cost and Schedule	10%		
Resource Issues (fisheries, river morphology)	10%		
Evaluation Total	**100%**		

9 SUMMARY AND RECOMMENDATIONS

The Phase II TDG Abatement study for Long Lake Dam evaluated five potential TDG abatement alternatives for the project. These alternatives included modifications to the existing spillway, the construction of a new auxiliary spillway or the construction of a new powerhouse. The following alternatives were developed to a conceptual design level.

1. Alternative 1 – Spill Bay 7-8 Deflectors
2. Alternative 2 – Spill Bay 7-8 Super-elevated Spillway Extension
3. Alternative 3 – Spill Bay 1-2 Toe Modifications and Downstream Deflector
4. Alternative 4 – Cut-off Dam Chute Spillway with Deflector
5. Alternative 5 – New Second Powerhouse

Hydraulic, civil, and structural conceptual designs were developed for Alternatives 1 through 4. For Alternative 5, past studies were used with a new energy evaluation model to update the energy potential of this alternative. For all alternatives, costs, construction techniques and construction schedules were developed. To assist with the complex hydraulics associated with the spillway modification alternatives (Alternatives 1 to 3), a CFD model was developed and tested for a range of flows. Qualitative assessments of the TDG reduction potential associated with Alternatives 1 through 4 were developed on the basis of past experience, existing TDG research and the results of the CFD analyses.

Alternatives 1 through 3 are designed to reduce the potential for plunging flow downstream of the existing spillway which currently leads to TDG levels in excess of 140% downstream of the project. Alternative 1 is comprised of constructing a flow deflector on the spillway chute downstream of bays 7 and 8, excavation of a shelf within the rock outcropping located at the base of the spillway and infilling the area between the flow deflector and the excavated rock shelf. The proposed design is similar to the spillway deflectors that have been installed on several of the Lower Columbia, Mid-Columbia and Snake River Dams. This is the only type of structural gas abatement alternative that has been built and tested in the field. It is estimated that Alternative 1 could reduce TDG levels to between 120% and 125%. The construction costs for Alternative 1 are estimated at $6,650,000; and, it is estimated that construction will require less than one year to complete.

Alternative 2 is a super-elevated chute extension below spillway bays 7 and 8 with a flip bucket constructed at its downstream end to redirect and spread the spillway flow onto the existing rock outcrop below the spillway. Although flip buckets have been used for energy dissipation, this type of structure has not been built for TDG abatement in the field. While the proposed concept has a high potential for reducing TDG, physical modeling would be essential to develop this design as the hydraulics are extremely complex. It is estimated that Alternative 2 could reduce TDG levels to 115% if additional design work shows that the hydraulics of this alternative will achieve the desired goal of spreading flow onto the rock promontory. The construction costs for Alternative 2 are estimated at $20,800,000; and, it is estimated that construction will require less than one year to complete.

Alternative 3 is similar to the spillway flow deflector concept described for Alternative 1, but the proposed deflector would be constructed below Bays 1 and 2, rather than 7 and 8. However, due to the existing topography below Bays 1 and 2, the standard deflector design was substantially modified to fit the site and it is expected that this could adversely impact its TDG reduction potential. It is estimated that Alternative 3 would reduce TDG levels to between 125% and 130%. The construction costs for Alternative 1 are estimated at $19,600,000; and, it is estimated that construction will require less than one year to complete.

Alternative 4 is comprised of constructing a new chute spillway downstream of the cut-off dam location designed to pass flows up to the 7Q10 spillway discharge. The existing site limits the available width for the proposed spillway, which results in a relatively high unit discharge for the flow deflector at the chute terminus. It is estimated that Alternative 4 would reduce TDG levels to values similar to those of Alternative 1, between 120% and 125%. The construction costs for Alternative 4 are estimated at nearly $50,000,000; and, it is estimated that construction will require around two years to complete.

Alternative 5 includes the construction of a new powerhouse. This alternative would increase the powerhouse capacity since the existing powerhouse would still be operable. The new powerhouse would typically be the primary powerhouse to operate; however, during higher events, the existing powerhouse and the new powerhouse would both operate. Therefore, the required spillway discharge would be about 49% of the 7Q10 discharge. A financial review of Alternative 5 is recommended to evaluate the generation benefits and potential hydropower incentives through the federal government. Due to the equipment and structure required for this alternative, the costs for Alternative 5 are around $210,000,000; and, the construction would take approximately three years.

9.1 RECOMMENDATIONS

Based on the conceptual designs of the alternatives and the estimated TDG reduction potential, further investigation is recommended for the alternatives that modify the existing spillway (Alternatives 1, 2 and 3). Alternative 4 was found to be a high cost alternative, with limited potential for reducing TDG in the downstream river channel, and as a result, it is not recommended for further evaluation at this time. Alternative 5 would reduce the 7Q10 spillway discharge by approximately 38%, and it is the only alternative that would increase power production at the project.

A physical model study is recommended for evaluating Alternatives 1, 2 and 3. The physical model would be used to confirm their hydraulic performance, refine their designs and select an alternative to carry forward to detailed design.

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