Project Memo

To: Scott Spahr, Snohomish County PUD  
From: Carl Mannheim  
cc: Kim Moore; Project File

Snohomish County PUD  
Sunset Fish Passage and Energy Project

TM 2A-4.2 Preliminary Design Basis Memorandum

1. Introduction

Snohomish County Public Utility District No 1 (PUD) has retained engineering services for evaluating the feasibility and potential features of a joint fish passage improvement and hydroelectric facility located on the South Fork of the Skykomish River near Index, WA. For the purposes of this evaluation, a design concept has been developed as described herein.

1.1 Purpose and Scope

This Preliminary Design Basis Memorandum (Preliminary DBM) establishes criteria for the preliminary design of the Sunset Fish Passage and Energy Project (Project). This document summarizes basic project site data, specified design criteria, relevant design guidelines, codes, and other key project information. The basic project features outlined in this Memorandum are intended to provide a starting point for the development of preliminary design and, as later modified, will be the basis for subsequent phases of design, should the District determine to proceed with the project.

Project features addressed by the Preliminary DBM include the diversion, intake, power tunnel, and powerhouse as well as the power generation equipment comprising the turbine inlet valves, turbines, generators, flow continuation, controls, switchgear, transformer, and associated electrical and mechanical works. Project features not addressed in this document to be completed by others include:

- Power lines to connect the plant switchyard to the existing transmission line along Highway 2.
- Trap and Haul improvements to the existing Washington Fish and Wildlife facility at the bottom of Sunset Falls and adjacent to the proposed powerhouse site.

Hatch’s 2012 activities have included coordination of geotechnical investigations, review of flood hydraulics, development of the intake design starting with the Preliminary Project Definition Report (PDR) arrangement, development of 3D images of the intake/diversion, preparation of a Preliminary Construction Plan, and completion of an initial acoustic assessment of the site. The scope was expanded to include conceptual design of an...
underground cavern intake in order to reduce the visible foot print of the intake/diversion. Additional subsurface investigations for the powerhouse, tunnel and cavern intake are recommended prior to Preliminary Design.

This DBM is a living document that will be updated as the design progresses. In its current form, this memorandum summarizes the project conceptual design at the end of Preliminary Design Support Activities in 2012. The information contained herein should be considered preliminary and subject to further confirmation and refinement. Items and subjects that are yet to be defined are noted “TBD” (to be determined). We anticipate that the Preliminary DBM would be updated as new information (such as additional subsurface investigations) become available.

1.2 Project Overview
As identified in studies to date, the Project is a run-of-river hydroelectric facility located upstream of the mouth of the South Fork Skykomish River near Index, Washington. The drainage area extends into the Cascades, with snowmelt and glacial melt contributing significantly to spring flows. Annual peak flows typically occur in November.

The proposed project includes a low head 132 feet long gated diversion structure upstream of Canyon Falls, intake deck with a course trash rack just upstream of the left abutment of the diversion, three intake tunnels, an underground “cavern” type intake with fish exclusion screens in a V-shaped configuration, trash racks and trash rake facilities, a fish bypass conveyance, a 19.5 ft diameter horseshoe-shaped primarily unlined power tunnel in rock, 2,250 ft long, and an underground shaft type powerhouse and tailrace located on the right bank below Sunset Falls.

The proposed maximum flow diversion is approximately 2,500 cfs. Pneumatically operated bottom-hinged flap gates across the diversion would control the water level at the intake during lower flows (<3,500 cfs), but will be lowered essentially flush with the river bottom during higher flows (>3,500 cfs). The general intake arrangement with an underground fish screen facility and very low head diversion structure was the preferred alternative to minimize the visual impact of the intake works. The design will include provisions to control sediment accumulation in front of the intake.

The power tunnel routes the flow under South Fork of the Skykomish River to the underground shaft type powerhouse on the right bank of the river just downstream of Sunset Falls. The powerhouse will contain two 15 MW vertical Francis turbines, and a synchronous bypass bay with a 72 inch diameter fixed-cone valve to control ramping rates and ensure flow continuation downstream of the diversion and powerhouse structures.

See Table 1-1 for a brief summary of project data and the attached Figures C-1 through C-6.

1.3 Intake Cavern with Fish Screens
- Underground cavern size, approximately: 120 ft wide; 200 ft long; and 75 ft high (allows operating deck to be clear of the 100-yr flood level)
- Three V-shaped fish screen structures (v-screens), each with a length of approximately 100 feet and a fish bypass flow of approximately 43 cfs
Table 1-1  Preliminary Project Data Summary

<table>
<thead>
<tr>
<th>Type of operation</th>
<th>Run of the river.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Estimated average annual energy (based on 30 MW plant)</td>
<td>123,900 MWh</td>
</tr>
<tr>
<td>(District projection assuming minimum flows identified in 1983 Crippen report (Crippen Consultants, Inc., 1983))</td>
<td></td>
</tr>
<tr>
<td>Proposed installed capacity (to be confirmed)</td>
<td>30 MW</td>
</tr>
<tr>
<td>Normal gross head</td>
<td>156 ft</td>
</tr>
<tr>
<td>Maximum powerhouse flow</td>
<td>2,500 cfs</td>
</tr>
<tr>
<td>Normal Maximum Power Pool Elevation</td>
<td>650.0 ft</td>
</tr>
<tr>
<td>Diversion height with gates</td>
<td>7 ft</td>
</tr>
<tr>
<td>Diversion crest gate length</td>
<td>132 ft</td>
</tr>
<tr>
<td>Power tunnel size</td>
<td>19.5' horseshoe in rock</td>
</tr>
<tr>
<td>Power tunnel length</td>
<td>Approximately 2,235 ft</td>
</tr>
<tr>
<td>Powerhouse type</td>
<td>Shaft in rock</td>
</tr>
<tr>
<td>Tailrace</td>
<td>Open discharge into pool below Sunset Falls</td>
</tr>
</tbody>
</table>

- Concrete lined invert and walls
- Steel support columns
- Personnel and equipment access through a vertical shaft with elevator
- Working deck El. 676 ft based on a 100-year flood event (to be evaluated – if working deck is kept at this level, flood gates would be eliminated).
- Slide gates at end of intake tunnels to protect against flooding the working deck of the v-screens
- Fish bypass pipe routed back to intake with discharge location downstream of the diversion

1.4 Power Tunnel

- Approximately 2,235 ft long, 19.5 feet diameter horseshoe shaped primarily unlined tunnel in rock
- Tunnel entrance lined with concrete
- Transition to steel lined near powerhouse
• Minimum rock cover of 60 feet

1.5 Powerhouse and Tailrace
• Partially underground powerhouse with ground level access
• Two vertical Francis units, 15 MW each, for a total generating capacity of 30 MW
• Unit centerlines at approximate El. 476.6 ft
• One bypass bay with a synchronous fixed-cone valve to ensure compliance with ramping rate limitations and flow continuation

2. Site and Environmental Conditions

2.1 Geotechnical Conditions

2.1.1 General
With the likely exception of a portion of the diversion structure (right abutment), all important project components are expected to be constructed on, or within, the granodiorite located in the southern portion of the Index Batholith. Further details on the site geotechnical conditions are provided in the Geotechnical Design Memorandum (Hatch and GeoEngineers, 2012). As discussed in this memorandum additional investigations have been recommended in order to provide the material parameters required to advance this project through to final design.

2.1.2 Diversion
The left abutment of the diversion structure is expected to be constructed on competent rock. The bedrock at the right abutment is expected to be over 100 ft deep under a 15 to 30 ft deep surface layer of alluvial sands and gravels and a 70 to 85 ft deep layer of glaciolacustrine silts and clays.

2.1.3 Intake
The intake, intake tunnels, and cavern intake with underground fish screens are expected to be located entirely within rock. The underground cavern is located within a ridge characterized by two rock knobs separated by a saddle which is approximately 10 ft below the elevation of the two knobs. The results of seismic refraction surveys in this area indicate that there is approximately a 40-ft thick layer of glacial sediments overlying the granodiorite. However the seismic refraction surveys also indicate that there is a low-velocity zone within the rock that could correspond to a shear zone, a zone of softer rock, or a relatively narrow rock-walled gully with deeper sediment infilling. As described in the geotechnical design memorandum, additional investigations are recommended in this area to obtain the material parameters required to develop the intake design, confirm the feasibility of the cavern intake concept and eventually carry out preliminary design.

2.1.4 Power Tunnel
The power tunnel is located entirely within the granodiorite rock. As described in the Geotechnical Design Memorandum, TM-3.1 (Initial Geotechnical Investigations) and TM-2A-3.2 (Final Draft Geotechnical Investigations Memorandum), the rock is generally of suitable quality for tunneling, however, there may be zones with poorer quality rock where additional support and reinforcement may be required.
2.1.5 **Powerhouse and Tailrace**

The powerhouse and tailrace are expected to be constructed entirely within the granodiorite as described in the Geotechnical Design Memorandum. The rock at the powerhouse site appears to be of good quality; massive, moderately to widely jointed; and fresh or slightly weathered (Hatch and GeoEngineers, 2011).

2.2 **Seismicity**

As described in the Geotechnical Design Memorandum, the peak ground accelerations for the 2% and 10% exceedances in 50 years are 0.23g and 0.43g, respectively. The 2006 International Building Code parameters are provided in the Geotechnical Design Memorandum.

2.3 **Climate**

The closest town to the project for which The Western Regional Climate Center (WRCC) provides climatological data is Startup, WA.

2.3.1 **Temperature**

See Table 2-1 for a summary of monthly temperature data.

- Minimum design temperature = -10° F
- Maximum design temperature = 110° F

### Table 2-1 General Temperature Summary 1924 – 2012

<table>
<thead>
<tr>
<th>Month</th>
<th>Max</th>
<th>Min</th>
<th>Avg</th>
<th>High</th>
<th>Low</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>45.4</td>
<td>32.9</td>
<td>39.1</td>
<td>68</td>
<td>-8</td>
</tr>
<tr>
<td>February</td>
<td>50.2</td>
<td>34.6</td>
<td>42.4</td>
<td>77</td>
<td>-4</td>
</tr>
<tr>
<td>March</td>
<td>54.1</td>
<td>36.2</td>
<td>45.1</td>
<td>83</td>
<td>6</td>
</tr>
<tr>
<td>April</td>
<td>60.2</td>
<td>39.4</td>
<td>49.8</td>
<td>90</td>
<td>17</td>
</tr>
<tr>
<td>May</td>
<td>66.6</td>
<td>44.1</td>
<td>55.3</td>
<td>102</td>
<td>27</td>
</tr>
<tr>
<td>June</td>
<td>71.1</td>
<td>48.4</td>
<td>59.7</td>
<td>106</td>
<td>31</td>
</tr>
<tr>
<td>July</td>
<td>77.0</td>
<td>50.6</td>
<td>63.8</td>
<td>104</td>
<td>35</td>
</tr>
<tr>
<td>August</td>
<td>77.2</td>
<td>50.6</td>
<td>63.8</td>
<td>100</td>
<td>36</td>
</tr>
<tr>
<td>September</td>
<td>71.5</td>
<td>47.1</td>
<td>59.3</td>
<td>98</td>
<td>30</td>
</tr>
<tr>
<td>October</td>
<td>61.8</td>
<td>42.0</td>
<td>51.9</td>
<td>92</td>
<td>19</td>
</tr>
<tr>
<td>November</td>
<td>51.7</td>
<td>37.5</td>
<td>44.6</td>
<td>78</td>
<td>4</td>
</tr>
<tr>
<td>December</td>
<td>46.1</td>
<td>34.1</td>
<td>40.1</td>
<td>67</td>
<td>3</td>
</tr>
<tr>
<td>Annual</td>
<td>61.1</td>
<td>41.5</td>
<td>51.3</td>
<td>106</td>
<td>-8</td>
</tr>
<tr>
<td>Winter (Dec – Feb)</td>
<td>47.2</td>
<td>33.8</td>
<td>40.5</td>
<td>77</td>
<td>-8</td>
</tr>
<tr>
<td>Spring (Mar – May)</td>
<td>60.3</td>
<td>39.9</td>
<td>50.1</td>
<td>102</td>
<td>6</td>
</tr>
<tr>
<td>Summer (Jun – Aug)</td>
<td>75.1</td>
<td>49.9</td>
<td>62.5</td>
<td>106</td>
<td>31</td>
</tr>
<tr>
<td>Fall (Sep – Nov)</td>
<td>61.7</td>
<td>42.2</td>
<td>51.9</td>
<td>98</td>
<td>4</td>
</tr>
</tbody>
</table>

2.3.2 **Snow and Rain**

See Table 2-2.
Table 2-2 General Precipitation Summary 1924 - 2012

<table>
<thead>
<tr>
<th>Month/Season</th>
<th>Precipitation</th>
<th>Total Snowfall</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>High</td>
</tr>
<tr>
<td>January</td>
<td>8.22</td>
<td>18.22</td>
</tr>
<tr>
<td>February</td>
<td>5.84</td>
<td>11.27</td>
</tr>
<tr>
<td>March</td>
<td>6.24</td>
<td>13.23</td>
</tr>
<tr>
<td>April</td>
<td>5.21</td>
<td>11.7</td>
</tr>
<tr>
<td>May</td>
<td>4.58</td>
<td>8.98</td>
</tr>
<tr>
<td>June</td>
<td>3.88</td>
<td>7.76</td>
</tr>
<tr>
<td>July</td>
<td>1.77</td>
<td>5.67</td>
</tr>
<tr>
<td>August</td>
<td>2.07</td>
<td>6.16</td>
</tr>
<tr>
<td>September</td>
<td>3.57</td>
<td>8.36</td>
</tr>
<tr>
<td>October</td>
<td>6.35</td>
<td>13.52</td>
</tr>
<tr>
<td>November</td>
<td>8.45</td>
<td>17.15</td>
</tr>
<tr>
<td>December</td>
<td>8.48</td>
<td>18.53</td>
</tr>
<tr>
<td>Annual</td>
<td>64.65</td>
<td>85.34</td>
</tr>
<tr>
<td>Winter (Dec – Feb)</td>
<td>22.54</td>
<td>36.39</td>
</tr>
<tr>
<td>Spring (Mar – May)</td>
<td>16.04</td>
<td>31.63</td>
</tr>
<tr>
<td>Summer (Jun – Aug)</td>
<td>7.71</td>
<td>15.93</td>
</tr>
<tr>
<td>Fall (Sep – Nov)</td>
<td>18.36</td>
<td>29.48</td>
</tr>
</tbody>
</table>

2.4 Hydrology and Flows

2.4.1 General
Estimated stream flow statistics are based on data from two USGS gages, USGS #12133000 at Sunset Falls and USGS #12134500 near Gold Bar. See Table 2-3 for gage descriptions and respective flow records.

The hydrologic record for USGS #12133000 on the South Fork Skykomish was extended by correlating overlapping data with USGS #12134500 on the main stem Skykomish River. The daily average flow correlation for each month are indicated in Table 2-4, where $Q_{SF}$ is the flow at Sunset Falls and $Q_{GB}$ is the flow at the USGS gage at Goldbar, respectively.

2.4.2 Flood of Record
- 70,000 cfs (1897)
- A possibly greater flood occurred in November 2006. There is no direct observation of the peak flow on the South Fork Skykomish River for this event, because the USGS gage on the South Fork Skykomish River was discontinued.
Table 2-3  Stream Flow Gages

<table>
<thead>
<tr>
<th>Gage and Location</th>
<th>Period of Record</th>
</tr>
</thead>
<tbody>
<tr>
<td>USGS #12134500 Near Goldbar on Skykomish River</td>
<td>Oct 1928 – Today</td>
</tr>
</tbody>
</table>

Table 2-4  Monthly Flow Correlations

<table>
<thead>
<tr>
<th>Month</th>
<th>Correlation Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>$Q_{SF} = 0.604(Q_{GB}) + 42.5$</td>
</tr>
<tr>
<td>February</td>
<td>$Q_{SF} = 0.630(Q_{GB}) - 43.9$</td>
</tr>
<tr>
<td>March</td>
<td>$Q_{SF} = 0.609(Q_{GB}) + 43.1$</td>
</tr>
<tr>
<td>April</td>
<td>$Q_{SF} = 0.613(Q_{GB}) + 90.9$</td>
</tr>
<tr>
<td>May</td>
<td>$Q_{SF} = 0.636(Q_{GB}) - 19.9$</td>
</tr>
<tr>
<td>June</td>
<td>$Q_{SF} = 0.629(Q_{GB}) - 6.22$</td>
</tr>
<tr>
<td>July</td>
<td>$Q_{SF} = 0.613(Q_{GB}) + 5.47$</td>
</tr>
<tr>
<td>August</td>
<td>$Q_{SF} = 0.583(Q_{GB}) + 23.1$</td>
</tr>
<tr>
<td>September</td>
<td>$Q_{SF} = 0.578(Q_{GB}) + 14.8$</td>
</tr>
<tr>
<td>October</td>
<td>$Q_{SF} = 0.599(Q_{GB}) - 15.9$</td>
</tr>
<tr>
<td>November</td>
<td>$Q_{SF} = 0.626(Q_{GB}) - 39.5$</td>
</tr>
<tr>
<td>December</td>
<td>$Q_{SF} = 0.615(Q_{GB}) + 11.3$</td>
</tr>
<tr>
<td>Annual</td>
<td>$Q_{SF} = 0.6212(Q_{GB}) - 6.40$</td>
</tr>
</tbody>
</table>

2.4.3  Flood Frequencies
See Table 2-5.
2.4.4 **Design Floods**

The project will meet a no net rise criterion of the 100-yr water level. A HEC-RAS model of the South Fork Skykomish River\(^1\) will be used to determine flood levels for various flood flows. See Tables 2-6 for estimated design flood levels of the existing river conditions.

### Table 2-5 Flow Exceedence and Return Periods

<table>
<thead>
<tr>
<th>Statistic</th>
<th>Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5% exceedance probability flow (annual)</td>
<td>6,700</td>
</tr>
<tr>
<td>5% exceedance probability flow</td>
<td>4,900</td>
</tr>
<tr>
<td>(fish passage season Jul 15-Dec 15)</td>
<td></td>
</tr>
<tr>
<td>95% exceedance probability flow (annual)</td>
<td>420</td>
</tr>
<tr>
<td>95% exceedance probability flow</td>
<td>360</td>
</tr>
<tr>
<td>(fish passage season Jul 15-Dec 15)</td>
<td></td>
</tr>
<tr>
<td>Flood of Record (1897)</td>
<td>70,000</td>
</tr>
<tr>
<td>2-yr Flood</td>
<td>23,200</td>
</tr>
<tr>
<td>10-yr Flood</td>
<td>44,300</td>
</tr>
<tr>
<td>25-yr Flood</td>
<td>55,700</td>
</tr>
<tr>
<td>50-yr Flood</td>
<td>65,200</td>
</tr>
<tr>
<td>100-yr Flood</td>
<td>74,700</td>
</tr>
</tbody>
</table>

1 The HEC-RAS model was originally developed by Snohomish County Department of Public Works in 2009 and reviewed by northwest hydraulic consultants (nhc, 2009a).

### Table 2-6 Design Flood Elevations

<table>
<thead>
<tr>
<th>Location</th>
<th>2-yr</th>
<th>10-yr</th>
<th>50-yr</th>
<th>100-yr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intake/Diversion</td>
<td>659.4</td>
<td>665.3</td>
<td>672.4</td>
<td>676.0</td>
</tr>
<tr>
<td>Powerhouse</td>
<td>505.3</td>
<td>510.6</td>
<td>514.9</td>
<td>516.7</td>
</tr>
</tbody>
</table>
2.4.5 Rating Curves

The District restored the former USGS gage above Sunset Falls (#12133000) in May 2012 and at the same time added a staff gage at the location of the proposed intake. A staff gage had already been installed near the powerhouse tailrace. The resulting rating curves for the diversion tailrace and the powerhouse tailrace are presented in Figures 2-1 and 2-2, based on data provided by the District.

Figure 2-1 Diversion Tailrace Rating Curve

Best Fit (Power): \( y=648.3081+0.0204^x+0.6280 \)

Notes:
1) Flow per old USGS gage restarted by PUD in 2012
2) Water surface elevation based on installed water level gage near proposed intake location at vertical datum 650.9 ft (NAVD88)
2.4.6 Minimum Instream Flows and Ramping Rates

- To be established based on agency consultation and FERC license conditions.
- Assumed instream flow requirements for sizing of facilities included herein are per Table 2-7 (Crippen Consultants, Inc., 1983).
- Assumed compliance point above Sunset Falls
- No assumption made on the ramping rate requirement. A synchronous fixed-cone unit bypass valve is included in powerhouse to ensure ramping rate meet requirements when turbines turn off.

2.5 Surveys and Controls

2.5.1 Datum For Design

- All design coordinates are to reference NAD 83 Washington State Plane North Zone
- All design elevations are to reference NAVD 88.

2.5.2 LiDAR Surveys

- Snohomish County LiDAR survey
Table 2-7  Assumed Monthly Instream Flow Requirement for Purposes of Facility Sizing

<table>
<thead>
<tr>
<th>Month</th>
<th>Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>430</td>
</tr>
<tr>
<td>February</td>
<td>430</td>
</tr>
<tr>
<td>March</td>
<td>430</td>
</tr>
<tr>
<td>April</td>
<td>430</td>
</tr>
<tr>
<td>May 1-15</td>
<td>330</td>
</tr>
<tr>
<td>May 15 - June 30</td>
<td>330</td>
</tr>
<tr>
<td>July 1-15</td>
<td>330</td>
</tr>
<tr>
<td>July 16 - August</td>
<td>330</td>
</tr>
<tr>
<td>September</td>
<td>430</td>
</tr>
<tr>
<td>October</td>
<td>430</td>
</tr>
<tr>
<td>November</td>
<td>430</td>
</tr>
<tr>
<td>December</td>
<td>430</td>
</tr>
</tbody>
</table>

2.5.3  Ground Surveys
- Both banks of the Canyon Falls constriction
- Left bank of the proposed diversion

2.5.4  Bathymetric Surveys
- River cross-section data were collected in 2011 and 2012

2.6  Project Access

2.6.1  Roads
- Main access to project site is from Highway 2
- Local, private access roads are about 14-16 feet wide and generally in average condition. Local road improvements will be necessary to accommodate construction vehicles, and as mitigation for project impacts, anticipated to include improved all-weather surfacing for primary access routes.
- Portions of local road across from the powerhouse tailrace pond are prone to flooding and will be raised (subject to permits being received) to ensure road is not washed out during large floods. Design water level will correspond to an approximately 100-yr flood event.

2.6.2  Bridges
- A narrow limited capacity bridge on the powerhouse access road from Highway 2 may need to be improved to accommodate construction vehicles. This bridge spans a former railroad grade, on which a new 2-lane road could be constructed at grade with rock spoils.
The access road to the powerhouse site passes under a BNSF railroad bridge with approximately 18 ft clearance, which should be sufficient for construction vehicles.

A bridge over the BNSF railroad provides access to the intake and diversion site. The capacity of this bridge (H-15) is not sufficient for expected construction vehicles and would require replacement or structural upgrade.

A new temporary bridge is proposed across Canyon Falls at the location of an earlier bridge. This bridge would improve construction access to the diversion and intake site.

2.6.3 Residential Access During Construction

- Residential access is expected to be maintained during construction, with only limited interruptions.
- The cavern intake concept significantly reduces the traffic at the intake site by completing all excavation underground and hauling tunnel and cavern spoils through the tunnel towards the powerhouse site.

3. Facility Sizing

3.1 Project Economic Analysis

Economic minimum cost analyses will be prepared by the District.

3.2 Tunnel Sizing

The preliminary tunnel concept considers a 19.5 ft diameter and 2,235 ft long horseshoe shape primarily unlined tunnel constructed by drilling and blasting. The diameter may be refined as design progresses and more detailed cost estimates become available.

The preliminary tunnel was sized by a minimization cost function as outlined by ASCE for sizing of penstocks (ASCE, 1993), in which the optimum diameter results in the minimum total cost of construction and the present value of power revenue losses $PRL$ per USBR:

$$PRL = \frac{0.00219 f E Q^3 h M}{D^5} (pwf)$$

Where $f$ is the Darcy-Weisbach friction factor, $Q$ is the design flow (cfs), $E$ is the overall turbine/generator efficiency, $h$ is the hours of operation per year, and $M$ is the value of power, and $pwf$ is the present worth factor. The present worth factor is equal to:

$$pwf = \frac{(i + 1)^n - 1}{i(i + 1)^n}$$

where $i$ is the discount rate and $n$ is the repayment period in years.

Tunnel construction costs were estimated by Hatch. The construction cost per unit volume of tunnel was assumed to be essentially constant for the range of diameters of interest, which allowed estimation of construction costs for several different diameters.

The energy losses are primarily due to friction, which were estimated based on Darcy-Weisbach’s friction loss equation:
\[ h_f = f \frac{L V^2}{D 2g} \]

Where \( f \) is the friction coefficient as a function of the tunnel roughness \( k \), \( L \) is the tunnel length, \( D \) is the equivalent diameter, and \( V \) is the flow velocity in the tunnel.

A mean overbreak thickness \( k \) of approximately 10 inches, typical for a tunnel in good granitic rock (U.S. Army Corps of Engineers, 1987), was assumed based on drilling and blasting. This roughness value should be refined as better geotechnical information becomes available and the assumed blasting technique is better defined.

The friction coefficient \( f \) was estimated based on the von Karman-Prandtl equation for fully rough flow:

\[ \frac{1}{f} = 2 \log \left( \frac{D_m}{k} \right) + 1.14 \]

Where \( D_m \) is the mean equivalent tunnel diameter.

In addition, the maximum flow velocity in the tunnel is recommended to be limited to 5 fps or less to prevent damage to downstream valves and turbines from migrating fines and rock falls (U.S. Army Corps of Engineers, 1987).

3.3 Power Generation Model

A preliminary power and generation study was performed by EES Consulting in 2009, which was updated by the District assuming 30 MW capacity and flows as described in section 2.

The preliminary annual average generation is 123,900 MWh, based on the analysis by the District.

3.4 Turbine & Generator Sizing

Hatch has assumed a total generating capacity of 30 MW with two 15 MW vertical Francis units to conservatively estimate the physical size of the project. Turbine and generator sizes may be updated during next phase of design.

4. General Design Requirements

4.1 Design Approach

4.1.1 Design Life

The project components shall be designed for at least a 50-year life span.

4.1.2 Hazard Classification

The Hazard Classification will be reviewed with the District and with FERC Portland Regional Office prior to starting the Final Design Phase.

4.2 Design Assumptions and Loading

This section presents the general design assumptions including design loads applicable to the design of the principal project features. Dimensions and characteristics shown are preliminary and subject to change during the course of design. For detailed hydraulic and structural design criteria of a particular structure, refer to the corresponding sections.
4.2.1 Geotechnical Design Parameters – Overburden Materials

See Table 4-1 for key geotechnical design parameters.

### Table 4-1 Key Geotechnical Design Parameters

<table>
<thead>
<tr>
<th>Material Properties</th>
<th>Unit Weight (pcf)</th>
<th>Allowable Bearing Pressure (psi)</th>
<th>Friction Angle</th>
<th>Cohesion (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium - Loose to Medium Dense Sand</td>
<td>110</td>
<td>20</td>
<td>28-30°</td>
<td>0</td>
</tr>
<tr>
<td>Alluvium - Medium Dense Gravel/Cobbles</td>
<td>118</td>
<td>28</td>
<td>35°</td>
<td>0</td>
</tr>
<tr>
<td>Glacio-lacustrine - Very Stiff to Hard Clay</td>
<td>112</td>
<td>28</td>
<td>25°</td>
<td>5</td>
</tr>
<tr>
<td>Rockfill / Riprap</td>
<td>135</td>
<td>-</td>
<td>40°</td>
<td>0</td>
</tr>
</tbody>
</table>

4.2.2 Earth Pressure

The lateral earth pressure will be computed on basis of Rankine theory, in accordance with the following:

- **Active Pressure**: For yielding walls and level backfill, the force $P$ per foot of wall, will be calculated by the following, including effect of earthquake, water in the soil, surcharge, and sloping backfill:

  $$ P = \frac{K_a w h^2}{2} $$

  $$ K_a = \tan^2 \left( 45° - \frac{\theta}{2} \right) $$

  where $\theta$ = soil angle of internal friction

- **At-Rest Pressure**: For unyielding walls and level backfill, the force $P$ per foot of wall, will be calculated by the following, including effect of earthquake, water in the soil, surcharge, and sloping backfill:

  $$ P = \frac{K_r w h^2}{2} $$

  $$ K_r = 0.50 $$

- **Passive Pressure**: For level backfill, the force $P$ per foot of wall, will be calculated by:

  $$ P = \frac{K_p w h^2}{2} $$

  $$ K_p = \tan^2 \left( 45° - \frac{\theta}{2} \right) $$
The assumed location of the earth pressure resultant will be as shown in Table 4-2.

<table>
<thead>
<tr>
<th></th>
<th>Horizontal Backfill</th>
<th>Upward Sloping Backfill</th>
</tr>
</thead>
<tbody>
<tr>
<td>Walls on Rock</td>
<td>0.38h above base</td>
<td>0.45h above base</td>
</tr>
<tr>
<td>Walls on Earth</td>
<td>0.33h above base</td>
<td>0.38h above base</td>
</tr>
</tbody>
</table>

4.2.3 Geotechnical Design Parameters - Rock

The following is a summary of the principal material parameters assumed appropriate for the preliminary engineering stage of design. More detailed information and requirements can be found in the relevant material specifications and actual test results. These values will be updated based on subsurface investigations and rock testing to be completed in 2013.

The unit weight of the rock is estimated as follows:

- Moist or saturated: 175 pcf;
- Submerged: 113 pcf

The modulus of elasticity for the rock mass will be calculated using RocLab and is based on the GSI value of the rock. The range of the GSI values of the exposed outcrops along the tunnel alignment and the powerhouse vary from approximately 60 to 85. GSI values in the vicinity of the fish screen cavern may be lower, as indicated by the low seismic velocity zone in the saddle area of the high peninsula. The low velocity zone suggests the presence of localized zone with considerably lower bulk strength characteristics and quality than the typical granodiorite in the area.

For example, a preliminary value for the modulus of elasticity of the rock mass is estimated using the Hoek-Brown Failure Criterion using the software program, RocLab, (vers. 1.032), developed by Rocscience (2011), with the following input parameters:

- GSI of 60;
- UCS of 14,500 psi;
- Mi (Granodiorite) of 29;
- Disturbance Factor of 0.5;
- Modulus ratio (granodiorite) of 425; and
- Average tunnel depth of 100 ft
Using the above-listed input parameters, the resultant deformation modulus is 1,675,000 psi within the first 2 to 6 ft of the underground excavations. Deformation moduli will be determined for the different rock mass classifications encountered in the project area.

The rock mass rating (RMR) values for the underground excavations will be estimated using the results of the geotechnical drilling investigations supplemented by the data collected from mapping of surface rock outcrops.

The in-situ stresses are assumed to be directly proportional to the height of rock above the structures. The ratio (k) of horizontal stress to vertical stress will be estimated using the method developed by Sheory (1994) where:

\[
k = 0.25 + 7Eh \left(0.001 \frac{1}{z}\right)
\]

Where \(Eh\) is the deformation modulus and \(z\) is the depth below the surface.

The assumed Shear Strength values for preliminary underground rock support calculations are as follows:

- Rock mass: \(\Phi = 66^\circ\), \(c' = 100\) psi
- Clean rock discontinuities: \(\Phi = 40^\circ\), \(c' = 0\) psi

### 4.2.4 Hydrostatic Load

The lateral load on structures may be calculated from a triangular load distribution based on the density of water times the depth of water.

Where water levels exist in backfill materials, the lateral load from soils below the water level shall be reduced to account for the buoyant weight of soils. For concrete structures cast against rock assume water seepage between rock and concrete which impose a hydrostatic load on the concrete structure.

### 4.2.5 Hydrodynamic Load

Hydrodynamic pressures for design will be calculated based on the Westergaard (1933) method. These pressures are in addition to hydrostatic pressures.

To account for the combined effect of the free water in front of a wall and water in the pores of coarse grained backfill, a procedure, as summarized by Seed and Whitman (1970) is as follows:

1. Water forces on the “seaward” side of the wall would be reduced during the earthquake by an amount equal to that computed by the Westergaard solution.
2. Water pressure on the “landward” side of the wall would increase by an amount equal to 70% of the Westergaard values and act together with the soil.

### 4.2.6 Wind Load

Structures will be designed in accordance with the International Building Code (IBC) latest edition, assuming an 85 mph wind speed, Exposure C, Importance factor TBD. Wind load will not be combined with earthquake loads.
4.2.7 **Snow Load**
Snow load = 40 psf

4.2.8 **Frost Depth**
Frost line is 18 inches for frost susceptible materials with 12 inches minimum bury over the top of base footings where percolation drainage materials are provided.

4.2.9 **Earthquake Load**
*TBD*

4.3 **Design Criteria and Requirements**

4.3.1 **Geotechnical Design**
Based on the geotechnical information available to date including the regional geology and subsurface lithology geotechnical design requirements have been selected for preliminary design. The geotechnical design requirements will be modified as additional site investigation results become available.

4.3.1.1 **Tunnel**
The tunnel will be a drill and blast tunnel through competent rock.
- Minimum cover = 60 ft
- Target overbreak = 2 ft

4.3.1.2 **Temporary excavations**
Maximum slope for temporary unshored excavations in soil less than 20 ft deep is 3H : 4V. All areas adjacent to excavations graded to drain water away from the excavation. Store and stockpile materials more than 6 ft or half the excavation depth away from excavation edges.

4.3.1.3 **Backfill pressures**
Select native free-draining granular materials, compacted in controlled lifts, will be used for structural backfills.

4.3.1.4 **Intake and Diversion Foundations**
- Intake will be founded on competent bedrock
- Diversion will be founded on alluvial material at the right abutment and competent bedrock at the left abutment.

4.3.1.5 **Powerhouse Foundation**
- The powerhouse will be an underground structure in competent bedrock

4.3.1.6 **Seismic Classification and Parameters**
- Lateral dynamic earth pressures will be determined in accordance with the procedures outlined in “Retaining and Flood Walls”, EM 1110-2-2502 by the U.S. Army Corps of Engineers (U.S. Army Corps of Engineers, 1989).
- Earthquake loads are assumed to have no effect on the water pressures considered for uplift.
4.3.2 **Concrete Design**

See Table 4-3.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mass Concrete</th>
<th>Structural Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cylinder Compression Strength, f’c</td>
<td>3,000 psi (at 90 days)</td>
<td>4,000 psi (at 28 days)</td>
</tr>
</tbody>
</table>

### Table 4-3 Concrete Mix Properties

4.3.3 **Reinforcement Design**

The design of the reinforced concrete will be in accordance with the Alternate Design Method, Appendix A, of ACI 318.

- Reinforcing steel will be fabricated according to ACI 315.
- Reinforcing bars will be ASTM 615, Grade 60, deformed, new billet steel, uncoated.
- Development length and lap splice length per Table 4-4. Provide Class B lap splices and development lengths. When splicing bars of different sizes provides splice length of smaller bars.
- Minimum reinforcement cover per Table 4-5.

#### Table 4-4 Minimum Lap Splice and Development Lengths

<table>
<thead>
<tr>
<th>Bar Size</th>
<th>Minimum Lap Splice Length (in)</th>
<th>Minimum Development Length (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Other Bar</td>
<td>Top Bar</td>
</tr>
<tr>
<td>#3</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td>#4</td>
<td>16</td>
<td>20</td>
</tr>
<tr>
<td>#5</td>
<td>19</td>
<td>25</td>
</tr>
<tr>
<td>#6</td>
<td>22</td>
<td>29</td>
</tr>
<tr>
<td>#7</td>
<td>32</td>
<td>42</td>
</tr>
<tr>
<td>#8</td>
<td>37</td>
<td>48</td>
</tr>
<tr>
<td>#9</td>
<td>44</td>
<td>56</td>
</tr>
<tr>
<td>#10</td>
<td>50</td>
<td>65</td>
</tr>
<tr>
<td>#11</td>
<td>62</td>
<td>80</td>
</tr>
</tbody>
</table>
### Table 4-5  Minimum Reinforcement Cover

<table>
<thead>
<tr>
<th>Cast-in-Place Concrete</th>
<th>Concrete Cover</th>
</tr>
</thead>
<tbody>
<tr>
<td>A Concrete cast against and permanently in contact with ground</td>
<td>3 in</td>
</tr>
<tr>
<td>B Concrete in contact with ground or weather</td>
<td></td>
</tr>
<tr>
<td>- No. 6 bar and larger</td>
<td>2 in</td>
</tr>
<tr>
<td>- No. 5 bar, W31, or D31 wire or smaller</td>
<td>1½ in</td>
</tr>
<tr>
<td>C Concrete not exposed to weather or in contact with ground</td>
<td></td>
</tr>
<tr>
<td>- Slabs, Walls, Joists</td>
<td></td>
</tr>
<tr>
<td>o No. 14 and No. 18 bars</td>
<td>1½ in</td>
</tr>
<tr>
<td>o No. 11 bar and smaller</td>
<td>¾ in</td>
</tr>
<tr>
<td>- Beams and Columns</td>
<td></td>
</tr>
<tr>
<td>o Primary reinforcement, ties, stirrups, spirals</td>
<td>1½ in</td>
</tr>
<tr>
<td>- Shells, folded plate members</td>
<td></td>
</tr>
<tr>
<td>o No. 6 bar and larger</td>
<td>¾ in</td>
</tr>
<tr>
<td>o No. 5 bar, W31, or D31 wire or smaller</td>
<td>½ in</td>
</tr>
</tbody>
</table>

### 4.3.4 Structural Steel

See Table 4-6.

### Table 4-6  Structural Steel

<table>
<thead>
<tr>
<th>Structural Steel</th>
<th>Grade/Specifications</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structural steel, W-shapes</td>
<td>ASTM A992 Grade 50</td>
</tr>
<tr>
<td>Other structural steel shapes</td>
<td>ASTM A36 Grade 36</td>
</tr>
<tr>
<td>Structural steel plates</td>
<td>ASTM A36 Grade 36</td>
</tr>
<tr>
<td>Structural steel pins</td>
<td>ASTM A572 Grade 50</td>
</tr>
<tr>
<td>Structural tubing</td>
<td>ASTM A53</td>
</tr>
<tr>
<td>Bolts for structural steel connections</td>
<td>ASTM A325, High Strength Grade, Galvanized</td>
</tr>
<tr>
<td>Nuts</td>
<td>ASTM A563, Galvanized</td>
</tr>
<tr>
<td>Washers</td>
<td>ASTM F426, Type 1, Galvanized</td>
</tr>
<tr>
<td>Welding electrodes</td>
<td>E70XX Series, AWS A5.1 or A5.5</td>
</tr>
</tbody>
</table>
5. Project Facilities Design

5.1 Diversion

The diversion structure is an approximately 260 ft long low-profile concrete base (weir) across the river. On top of the weir is a 132 ft long bottom-hinged pneumatically operated crest gate (crest gate) that will control the flow to the intake by providing approximately 1 foot of head differential across the diversion during normal river flows (< approximately 3,500 cfs). During higher river flows (> approximately 3,500 cfs), the gate will be lowered to lay essentially flat on top of the weir to maintain the river flood capacity as hydraulic control shifts to the narrow rock valley upstream of Canyon Falls.

5.1.1 Flood Passage

The diversion will be designed to accommodate passage of discharges up to the 100-year flood event with no net rise in the 100-yr flood level as determined by hydraulic modeling.

The crest gates will be partially raised to maintain upstream water levels to elevation 650 for flows up to approximately 3,500 cfs. One gate section near the intake may also be used for sediment sluicing as required.

5.1.2 Design Water Levels

The pool level during normal operations will be regulated by the crest gates to 650 ft. The diversion tailrace water level during normal operations is controlled by the downstream river section (narrow point upstream of Canyon Falls) and is approximately 649 ft, assuming required instream flows. Table 5-1 summarizes the water levels for operations during normal flow, high flow, and various flood flows.

<table>
<thead>
<tr>
<th>Operation</th>
<th>Total River Flow (cfs)</th>
<th>Intake Flow (cfs)</th>
<th>Pool Level (ft)</th>
<th>Tailrace Level (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Operation</td>
<td>&lt; 430 cfs</td>
<td>0</td>
<td>N/A¹</td>
<td>N/A¹</td>
</tr>
<tr>
<td>Normal Flow</td>
<td>430 – 2,930</td>
<td>0 – 2,500</td>
<td>650</td>
<td>649</td>
</tr>
<tr>
<td>High Flow (&lt;100-yr)</td>
<td>2,930 &gt; Q &gt; 74,700</td>
<td>2,500</td>
<td>650 - 676</td>
<td>649 – 676</td>
</tr>
<tr>
<td>Flood Flow (&gt;100-yr)</td>
<td>&gt;74,700</td>
<td>0</td>
<td>676</td>
<td>676</td>
</tr>
</tbody>
</table>

1. During non-operation, it is anticipated that the weir would be laid flat to allow water surface elevations same as unmodified condition.

5.1.3 Flow Dynamics Across Weir

Preliminary estimates of flow patterns over the crest gates under normal operating conditions, when the weir is at full height, indicate that flow over the raised crest gate would drop approximately 1 foot vertically into the downstream channel with a velocity of approximately 2.5 fps. Most with most of that velocity would be in the downstream direction. The vertical
velocity would be very small and would not have sufficient energy to plunge the flow or an object to depth. A “downstream roller”, wherein flow moves in a circular pattern that can have a trapping effect on logs or other material, is therefore unlikely to form. When the gate is lowered, it would be essentially flat on the bottom of the weir with all flow in the downstream direction with no roller forming. A more detailed hydraulic analysis using a 2-D computational fluid dynamic model would be planned to confirm the flow patterns during various operating scenarios.

5.1.4 Geotechnical Design
The left abutment will be constructed in competent rock. Portions of the diversion and right abutment are expected to be constructed on alluvial material.

5.1.5 Structural Design
The building will be designed for the design loads and loading conditions in Section 5.2 General Design Criteria, and to the following design loads and loading conditions applicable to the diversion structure.

5.1.5.1 Hydrostatic and Hydrodynamic
See Sections 4.2.4 and 4.2.5.

5.1.5.2 Uplift Pressure
The magnitude of uplift pressure will be determined by any of the following three methods as outlined in Chapter 3 Gravity Dams by FERC: Creep theory; Flow net method; or Finite Element Method.

5.1.5.3 Earth Pressure
The earth pressure behind the diversion will be estimated assuming at-rest earth pressure.

5.1.5.4 Silt Pressure
- Vertical pressure exerted by saturated silt will be determined as if silt were a saturated soil, the magnitude of pressure varying directly with depth.
- Horizontal pressure exerted by the silt load will be calculated in the same manner as submerged earth backfill.
- Silt shall be assumed to liquefy under seismic loading. Thus, for post earthquake analysis, silt internal shear strength will be assumed to be zero unless site investigations demonstrate that liquefaction is not possible.

5.1.5.5 Ice Loading
The diversion crest gates are designed to always overtop and also to automatically open when the upstream pressure on the gate is greater than the pressure corresponding to 2 feet of overtopping. Therefore, no ice load will be applied.

5.1.5.6 Stability Analysis
The loading conditions and requirements in Table 5-2 are suitable, in general, for gravity dam of moderate height, and were adapted from FERC (Federal Energy Regulatory Commission, 2002). Loads which are not indicated, such as wave action, or any unusual loadings are
considered where applicable. Power intake sections will be analyzed with emergency bulkheads closed and all water passages empty.

Table 5-2 Diversion Load Cases

Case I: Usual Loading Combination – Normal Operating Condition
The reservoir elevation is at the normal power pool, as governed by the crest elevation of an overflow structure, or the top of the closed spillway gates whichever is greater. Normal tailwater is used. Horizontal silt pressure should also be considered, if applicable.

Case II: Unusual Loading Combination – Design Flood Discharge
For high and significant hazard potential projects, the flood condition that results in reservoir and tailwater elevations which produce the lowest factor of safety should be used. Flood events up to and including the Inflow Design Flood, if appropriate, should be considered. For dams having a low hazard potential, the project should be stable for floods up to and including the 100 year flood.

Case III: Extreme Loading Combination – Normal Operating with Earthquake
Per FERC Guidelines, Chapter 13: “In a departure from the way the FERC has previously considered seismic loading, there is no longer any acceptance criteria for stability under earthquake loading. Factors of safety under earthquake loading will no longer be evaluated.”

Stability of hydraulic structures under post earthquake static loading will be analyzed considering damage likely to result from the earthquake. The purpose of considering dynamic loading is to assess the damage that could be caused so that this damage can be accounted for in the subsequent post earthquake static analysis.

5.1.5.7 Stability Criteria
The basic requirement for stability of a gravity dam subjected to static loads is that force and moment equilibrium be maintained without exceeding the limits of concrete, foundation or concrete/foundation interface strength. This requires that the allowable unit stresses established for the concrete and foundation materials not be exceeded. The allowable stresses should be determined by dividing the ultimate strengths of the materials by the appropriate safety factors in Table 5-3, which present the recommended safety factors corresponding to a new dam per FERC 0119-2.

Factors of safety are the ratio of the resisting forces to the forces tending to cause movement. The sliding factors of safety resulting from design will meet the criteria listed below.

Allowable bearing pressures have, as a minimum, factors of safety corresponding to that required for sliding. Factor of safety against overturning will not be calculated as $FS = M_r/M_0$.

Consideration is also given to stability in a post-earthquake condition.
## Table 5-3 Stability Analysis Minimum Factors of Safety

<table>
<thead>
<tr>
<th>Load Case</th>
<th>High and Significant Hazard Dams</th>
<th>Low Hazard Dams</th>
</tr>
</thead>
<tbody>
<tr>
<td>Safety Factor with cohesion</td>
<td>3.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Usual</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Unusual</td>
<td>2.0</td>
<td>1.25</td>
</tr>
<tr>
<td>Post Earthquake</td>
<td>1.3</td>
<td>&gt;1.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Safety Factor if cohesion is not relied upon for stability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Worst Static Case</td>
<td>1.5</td>
</tr>
<tr>
<td>Flood if Flood is PMF</td>
<td>1.3</td>
</tr>
<tr>
<td>Post Earthquake</td>
<td>1.3</td>
</tr>
</tbody>
</table>

### 5.1.6 Sediment Passage
Removal of sediment deposits near the intake will be accomplished by lowering a narrow section of the crest gates near the intake. This section of the gate may be installed with a sill level slightly lower than the rest of the crest gate.

### 5.1.7 Riprap
Riprap erosion protection will be designed per HEC 11 *Design of Riprap Revetment* (Federal Highway Administration, 1989) procedures.

### 5.1.8 Fish Passage
Downstream fish passage at the diversion is assumed to be over the crest gate or through the fish bypass pipe.

### 5.1.9 Instream Flow Release
The required instream flow will be discharged over the entire width of the crest gates and through the fish bypass pipe. The assumed instream flow requirement is presented in Section 2.4.6. The head required to pass the flow over the crest gates can be estimated from a sharp-crested weir equation with correction for a sloping face per USBR (Bureau of Reclamation, 2001):

\[ Q = C_d C_e L_e h_e^{1.5} \]

Where \( Q \) is the flow in cfs, \( C_d \) is the correction factor for angle of the gate, \( C_e \) is the effective discharge coefficient for a vertical weir, and \( h_e \) is the effective head on the gate.

### 5.1.10 Lighting
Indoor lighting will be provided by ceiling-mounted luminaries controlled by manual switches. Emergency and exit lighting will be powered by 90-minute battery packs.
Lighting will be time phased to operate only when required and be discretely located to light the Project facilities. Where possible, lights would be energy efficient, shielded, recessed into the ground, or attached to the sides of structures.

5.1.11 Access
Personnel access to the diversion will be from either the left abutment or the right abutment. Vehicle access will be from the right diversion only. A small parking lot near the right abutment will be provided.

5.1.12 Crest Gates
- The crest gates will be pneumatically operated (rubber bladder) bottom-hinged gates with a total length of about 132 feet. The gates will be separated into three separate gates of different length, each individually controlled.
- The crest gates will be installed across the diversion structure for flow control. The preliminary gate sill elevation is 642 ft and the top of the gate will be approximately 649 ft.
- The crest gates will be designed for continuous submerged operation and to allow passage of flood debris and sediment for flows up to a 100-yr flood event.
- The diversion with the crest gates will be sized to ensure a no net rise in the 100-yr water level.
- The crest gates will be designed and installed with provisions to remove individual bladders or gate panels for maintenance.

Preliminary operating requirements for the crest gate(s) are shown in Table 5-4.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Property/Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spillway Gate Type</td>
<td>Pneumatically operated bottom hinged crest gate</td>
</tr>
<tr>
<td>Overtopping Height</td>
<td>The gate must withstand 2 feet overtopping when fully raised, including effects of submergence</td>
</tr>
<tr>
<td>Temperature Range</td>
<td>-10 °F to 110 °F.</td>
</tr>
<tr>
<td>Control</td>
<td>Maintain upstream pool at constant level (El 650) when river flow is greater than powerhouse flow.</td>
</tr>
<tr>
<td>Spillway Crest Level</td>
<td>642.0</td>
</tr>
<tr>
<td>Normal Design Water Levels</td>
<td>Pool El. 650.0 ft, including approximately 1 foot overtopping for instream flow, Tailwater El. 649.0 ft.</td>
</tr>
<tr>
<td>Maximum Design Water Levels</td>
<td>El. 652.0 ft, Tailwater El. 651.0 ft.</td>
</tr>
<tr>
<td>(overtopping with gate fully closed)</td>
<td></td>
</tr>
<tr>
<td>Crest Gate Closing Head</td>
<td>Crest Gate must be fully capable to raise against 9 feet of head</td>
</tr>
<tr>
<td>Ice and Debris Handling</td>
<td>The gate shall be capable of passing large logs, tree stumps, and floating ice and debris, without damaging any components</td>
</tr>
<tr>
<td>Submergence</td>
<td>The gate must be capable of operating continuously submerged</td>
</tr>
</tbody>
</table>
5.2 Intake and Intake Tunnel

5.2.1 Intake Deck and Trashracks

5.2.1.1 Design Water Levels
The intake structure maintenance deck has been set to elevation 655 ft, corresponding to an approximately 2-year flood level, allowing the intake deck to be flooded during high flows which minimizes the impact on river flood levels and reduces the visual impact of the intake.

See also Section 5.1.2.

5.2.1.2 Structural Design
The intake will be designed for the design loads and loading conditions in Section 4.2 General Design Criteria.

5.2.1.3 Trashrack and Stoplogs
- A trashrack at the intake will screen large debris and minimize headlosses.
  - Bar spacing = 4 inches
  - Design head differential = approximately 4 ft
- The headloss through the trashrack will be calculated based on the headloss coefficient per Figure 14.8 in Internal Flow Systems (Miller, 1990).
- No trashrake will be provided. Trash will be manually cleaned when needed.
- Maximum intake velocity will assume no ice in the river and consider swimmer and boater safety. A preliminary maximum intake velocity of 4 fps is assumed based on recommended values.
- Stoplog slots will be provided to allow intake tunnel to be isolated for maintenance.

5.2.1.4 Intake Deck Equipment
There will be no permanent equipment on the intake deck due to the annual risk of flooding. Permanent footings along the intake deck for temporary installation of a portable jib crane will be provided. The jib crane could be used during periods of lower flows to remove debris from the trashracks.

5.2.1.5 Access
- A set of stairs will lead from a parking pullout on the residential access road above the intake down to the intake deck.
- No vehicle access will be provided.

5.2.2 Intake Tunnel
- Geotechnical design will include stabilization design of portal face and tunnel.
- The intake tunnel design will include stop log slots and stop logs so the intake tunnels can be dewatered for inspection and maintenance.
- Structural design of the intake tunnel reinforced concrete invert, walls and crown will account for loads including external water pressure.
5.3 Intake Cavern and Fish Screens

5.3.1 Cavern Geotechnical Design
The cavern design will be based on the Norwegian Geotechnical Institute (NGI) Q System to determine rock support requirements. In addition, analyses will be performed using Phase² and Unwedge software, developed by Rocscience, Inc. to determine the rock reinforcement and support requirements and assist in developing the excavation sequence. Additional field explorations will also be completed. The rock reinforcement for the portal excavation in competent rock, where the failure mechanism is expected to be along discontinuity surfaces, will be determined using limit equilibrium software (Swedge and Rocplane) developed by Rocscience Inc. In weak rock, two-dimensional slope stability analyses using Bishop simplified method will be carried out for the overall cut slopes in soil or weak rock using SlopeW, developed by Geoslope. The Mohr-Coulomb parameters used for the analyses for weak rock will be calculated using the Hoek-Brown Failure Criterion (Hoek et al., 2002). Cavern rock stabilization design may include rock bolts, cable bolts and drainage systems.

5.3.2 Design Water Levels
- Normal water level in the intake cavern is 650 ft, less headlosses through intake and trashracks.
- The flood level of the working deck for the fish screens will be elevation 676 ft, corresponding to a 100-yr flood event. Slide gates at the entrance to the cavern (at end of intake tunnels) will close when water level reaches 100-year flood level, preventing damage to any of the fish screen equipment.

5.3.3 Civil/Structural
Reinforced concrete slabs and walls be designed to support the cavern as required based on the geotechnical analysis and to support other features internal to the cavern including fish screens, gates, and the working deck.

5.3.4 Fish Screens
The fish screens will be designed according to the guidelines outlined by NMFS (NMFS, 2008).
- All screens will have automatic screen cleaners (brushes).
- Each screen section will comprise an upstream bar screen and a downstream louver screen, which will be used to balance the flow through each screen section such that screen approach velocity criteria are complied with. Field balancing of the flow through each screen will be required.

5.3.5 Fish Screen Trashrack
A trash rack will be installed on the upstream end of each v-shaped fish screen to screen off small debris. A trashrake will be installed to automatically clean the trash rack periodically.
- Trash rack bar spacing = 1 in
- Trash rack design head differential = approximately 4 ft
The headloss through the trashrack will be calculated based on the headloss coefficient per Figure 14.8 in Internal Flow Systems (Miller, 1990).

- Removed material would be either returned to the river, or taken offsite.

5.3.6 **Fish Bypass**
- The fish bypass flow for the screens inside the cavern is 130 cfs total (43 cfs per v-screen) with the bypassed fish and flow released downstream of the diversion, assuming sufficient head is available to drive the flow back out the cavern to the river.
- The fish bypass will be designed according to the guidelines outlined by NMFS (NMFS, 2008)

5.3.7 **Working Deck**
- The working deck of the cavern will provide a platform from which the fish screens and equipment can be accessed and operated.
- The preliminary elevation of the working deck is 676, corresponding to the water surface elevation at the intake for a 100-yr flood event. This elevation should be reviewed considering the flood gates will prevent flooding of the deck. The working deck elevation dictates the cavern crown elevation that would have significant impact on cavern construction costs.
- The working deck will be a concrete floor with steel grating over equipment that require access.
- A uniformly distributed design live load of 250 psf will be used for design. Point live load design requirements are TBD.

5.3.8 **Access**
- The primary personnel and equipment access to the intake cavern and fish screens will be by an elevator in a vertical shaft at the downstream end of the cavern. The size and capacity of the elevator will be based on the size and weight of the largest and/or heaviest piece of equipment needed on deck.
- Emergency personnel access will be provided by one or more sets of stairs, the location of which have not yet been determined.

5.3.9 **Vertical Flood Gates**
Slide gates at the entrance to the cavern (at end of intake tunnels) will close when water level reaches flood level (TBD), preventing damage to any of the fish screen equipment.

5.4 **Power Tunnel**
The power tunnel will be an approximately 19.5 feet diameter horseshoe shaped tunnel, constructed by drilling and blasting. The length of the tunnel will be approximately 2,235 feet measured from the tunnel intake at the downstream end of the intake cavern to the upstream wall of the powerhouse.

5.4.1 **Geotechnical**
The rock support and reinforcement for the power tunnel will be determined using the NGI Q system as well as Phase² and Unwedge. The water inflow will be estimated using the method
developed by Heuer (1995). The potential for hydrojacking will be checked using the Norwegian Criterion.

5.4.2 Lining
The tunnel will generally be unlined, with only short portions lined with concrete if required for geotechnical reasons.

5.4.3 Headlosses
Friction losses will be calculated using the Darcy-Weisbach equation with the friction coefficient $f$ estimated based on the von Karman-Prandtl equation for fully rough flow. See Section 4.2 for estimation of friction losses based on the assumed tunnel overbreak.

Intake losses will be estimated based on a rounded tunnel entrance ($k_e = 0.2$) and average tunnel velocity, $V$:

$$h_L = 0.2 \frac{V^2}{2g}$$

5.4.4 Headgate
- A vertical slide gate will be installed at the entrance to the tunnel to allow dewatering of the entire tunnel.
- The gate will include a downstream air vent to prevent negative pressures in the tunnel during opening and closing of the gate. The size of the vent will be determined based on criteria outlined in Hydraulic Design Criteria by USACE (U.S. Army Corps of Engineers, 1987).

5.5 Powerhouse and Tailrace

5.5.1 General
A “shaft-style” powerhouse will be located near the WDFW Trap & Haul facility on the right bank, away from the shore. The powerhouse substructure would be excavated into rock with the generator floor approximately 30 feet below ground level and the unit centerline approximately 55 feet below ground level. Ground level at the Trap & Haul is approximately EL. 530. The power tunnel would cross under the South Fork of the Skykomish River and there would be a submerged low-pressure tailrace tunnel to a discharge location along the right bank just downstream from the Trap and Haul intake.

The powerhouse will have a turbine and generator floor at El 498 with two vertical Francis units, an erection and loading bay on the operating floor at El 530 (approximate elevations), mechanical equipment, and auxiliary electrical equipment.

The outside dimensions of the building will be approximately 115 ft by 60 ft. The total height of the powerhouse, from the roof of the superstructure to the floor supporting the turbine inlet valves, will be approximately 100 feet. Approximate height from existing grade is 30 ft.

An overhead bridge crane with a runway extending the full length of the turbine/generator floor will be provided.

An insulated roll-up door will be provided in the north end of the operating floor to facilitate moving equipment into the building under the bridge crane.
We expect no increased risk of local erosion of the banks around the tailrace pond due to the proposed tailrace exit. Water will pass through either the turbines and/or synchronous bypass valves, which will remove over 90 percent of the energy in the water and will result in a tailrace exit velocity less than about 5 fps. The flow velocity at the base of Sunset Falls will be significantly higher and will block or deflect much of the tailrace flow from reaching the south shoreline of the tailrace pond approximately 700-feet from the tailrace exit. The residual energy of the tailrace flow in the tailrace pond will consequently be significantly reduced both due to the high velocity Sunset Falls flow and by the normal energy dissipation occurring in the large area and volume available in the tailrace pond. Therefore, any remaining energy in the tailrace flow would be undetectable from existing conditions.

5.5.2 Geotechnical Design
The powerhouse will be an underground shaft-style structure in competent bedrock, where with the failure mechanism expected to be along major discontinuity surfaces. As a result, the stability of the powerhouse walls will be assessed and rock support will be developed using the limit equilibrium methods described in Section 5.7.1.

5.5.3 Structural Design
The building will be designed for the design loads and loading conditions in Section 5.2 General Design Criteria, and to the following design loads and loading conditions applicable to the powerhouse.

5.5.3.1 Design Stresses
Allowable stresses will depend on the materials involved, the conditions of loading, and severity of exposure.

- Structural steel and welded joints will be designed in accordance with the allowable stresses outlined in the latest version of the AISC construction manual.
- Welding details will be as outlined by the latest version of AWS D1.1 “Structural Welding Code-Steel.”
- Gates, Bulkheads, Trashracks, and associated guides will be designed using the allowable stresses outlined in EM 1110-1-2102.

Concrete structures loaded hydraulically should be designed using Load Factor Design in accordance with the procedures outlined in EM 1110-2-2104 “Strength Design for Reinforced Concrete Hydraulic Structures.” Those portions of a powerhouse that will not have water loading, such as the superstructure, may be designed in accordance with the latest version of ACI 318 “Building Code Requirements for Reinforced Concrete.”

5.5.3.2 Live Loads
See Table 5-5.
Table 5-5  Minimum Uniformly Distributed Live Loads

<table>
<thead>
<tr>
<th>Loaded Areas</th>
<th>Uniform Live Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofs</td>
<td>50</td>
</tr>
<tr>
<td>Stairways</td>
<td>100</td>
</tr>
<tr>
<td>Offices, Corridors, Reception Rooms, Toilet and Locker Rooms</td>
<td>100</td>
</tr>
<tr>
<td>Equipment and Storage Areas</td>
<td>250</td>
</tr>
<tr>
<td>Control Room</td>
<td>200</td>
</tr>
<tr>
<td>Mezzanine Floor</td>
<td>250</td>
</tr>
<tr>
<td>Shop and Turbine Floor</td>
<td>1,000</td>
</tr>
<tr>
<td>Mechanical/Electrical Floor</td>
<td>1,000</td>
</tr>
</tbody>
</table>

In general, floors will be designed for an assumed uniform load per square foot of floor area. However, the floors will be investigated for the effects of any concentrated load, minus the uniform load over the area occupied. Equipment loads will take into account installation, erection, and maintenance conditions as well as impact and vibration after installation.

Designated uniform live loads are minimum loadings for design of slabs, beams, girders, and columns in the areas indicated. These loads may be modified, if necessary, to suit more severe specific conditions. They may be reduced 20 percent for the design of a girder, truss, column, or footing supporting more than 300 sq ft of slab, except that for the erection floor, this reduction will be allowed only where the member under consideration supports more than 500 sq ft of slab.

5.5.3.3 Estimated Equipment Loads
TBD

5.5.3.4 Crane Loads
TBD

5.5.4 Substructure
TBD

5.5.5 Superstructure
The superstructure will be designed according to Washington State’s Building Code.

5.5.6 Architectural Design

5.5.6.1 Personnel Facilities
- A restroom with toilet and sink will be provided.
5.5.6.2 **Exterior Treatment**
The exterior of the powerhouse will be designed to reduce visual impact.

- Walls: *TBD*
- Roof: *TBD*
- Doors, Frames, Louvers: *TBD*

5.5.6.3 **Interior Treatment**

- Floors: *TBD*
- Walls and Partitions: *TBD*
- Doors and Frames: *TBD*
- Interior Metal Work: *TBD*

5.5.7 **Electrical Equipment**

5.5.7.1 **General**
*TBD*

5.5.7.2 **Generator Bus**
*TBD*

5.5.7.3 **Generator Switchgear**
*TBD*

5.5.7.4 **Neutral Grounding Equipment**
*TBD*

5.5.7.5 **Station Service**
*TBD*

5.5.7.6 **Grounding System**
*TBD*

5.5.7.7 **Lighting System**
*TBD*

5.5.7.8 **Conduit System**
*TBD*

5.5.7.9 **Cable and Wire**
*TBD*

5.5.7.10 **DC System**
*TBD*

5.5.7.11 **Controls and SCADA System**
*TBD*

5.5.7.12 **Protective Relaying**
*TBD*
5.5.7.13 **Fire Detection and Alarm Systems**  
*TBD*

5.5.7.14 **Intrusion Alarm**  
*TBD*

### 5.5.8 Mechanical Equipment

5.5.8.1 **Tailrace Bulkhead Gates**  
*TBD*

5.5.8.2 **Powerhouse Bridge Crane**  
- CMAA Specification No. 70 – Specifications for Top Runner Bridge and Gantry Type Multiple Girder Electric Overhead Travelling Cranes; and OSHA Standards.

### 5.5.9 Auxiliary Mechanical Equipment

5.5.9.1 **Drainage and Dewatering System**  
*TBD*

5.5.9.2 **Service Water System**  
*TBD*

5.5.9.3 **Fire Protection System**  
*TBD*

5.5.9.4 **Domestic Water System**  
*TBD*

5.5.9.5 **Sanitation and Wastewater Facilities**  
*TBD*

5.5.9.6 **Heating and Ventilation**  
*TBD*

5.5.9.7 **Standby Generator**  
*TBD*

### 5.5.10 Lighting

Outdoor lighting will be time phased to operate only when required and be discretely located to light the Project facilities. Where possible, lights would be energy efficient, shielded, recessed into the ground, or attached to the sides of structures.

### 5.6 Power Generation & Equipment Design

#### 5.6.1 Turbines and Generators

5.6.1.1 **Turbines**  
Two vertical Francis units are assumed for this phase.

The following was assumed for unit selection:

- Total powerhouse flow = 2,500 cfs
• Unit centerline elevation = 476.6 ft
• Gross head = 156 ft
• Net head = 150 ft
• Normal reservoir elevation = 650 ft
• Normal tailrace elevation = 494 ft

The estimated weights for the turbines and generator components are shown in Table 5-6:

Table 5-6 Estimated Turbine and Generator Component Weights

<table>
<thead>
<tr>
<th>Component</th>
<th>Estimated Weight (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Turbine (each)</strong></td>
<td></td>
</tr>
<tr>
<td>Runner</td>
<td>19,000</td>
</tr>
<tr>
<td>Shaft</td>
<td>10,000</td>
</tr>
<tr>
<td>Head cover</td>
<td>20,000</td>
</tr>
<tr>
<td>Bottom ring</td>
<td>15,000</td>
</tr>
<tr>
<td>Spiral case and stay ring</td>
<td>70,000</td>
</tr>
<tr>
<td>Wicket gates (total)</td>
<td>25,000</td>
</tr>
<tr>
<td>Draft tube liner</td>
<td>20,000</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>21,000</td>
</tr>
<tr>
<td><strong>Total turbine</strong></td>
<td>200,000</td>
</tr>
<tr>
<td><strong>Generator (each)</strong></td>
<td></td>
</tr>
<tr>
<td>Rotor and shaft</td>
<td>140,000</td>
</tr>
<tr>
<td>Stator</td>
<td>100,000</td>
</tr>
<tr>
<td>Upper bracket with bearing</td>
<td>35,000</td>
</tr>
<tr>
<td>Lower bracket with bearing</td>
<td>15,000</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>10,000</td>
</tr>
<tr>
<td><strong>Generator total</strong></td>
<td>300,000</td>
</tr>
</tbody>
</table>

5.6.1.2 **Governor**

The turbines will be provided with a hydraulic governing system. The system will be complete with oil pump set, oil sump tank, and piping to the wicket gates servomotors, electrical control valves, position sensors, and all accessories, controls, and instruments necessary for a complete governing system.

The system will also be provided with seals, vents, and drains to prevent any contamination of the water with oil. Any leakage will be routed to a collection system.
5.6.1.3 **Turbine Inlet Valves**

An approximately 102-inch hydraulic cylinder operated turbine inlet valve with bypass piping will be installed immediately upstream of each turbine.

- Design flow = 1,250 cfs
- Design pressure = 194 ft, assuming 100-yr flood event and dewatered turbines.

5.6.1.4 **Synchronous Bypass Valve**

An approximately 72-in fixed cone valve will be installed in a synchronous bypass bay to control the ramping rate at the diversion in case of an unexpected unit shut down.

- Design flow = 2,500 cfs

The next phase will investigate if designing the turbines for runaway speed could eliminate the need for the synchronous bypass bay and reduce the cost.

5.6.1.5 **Generator**

TBD

5.6.1.6 **Exciter**

TBD

5.6.2 **Switchyard**

The switchyard will be approximately 100 ft by 75 ft and located to the north of the powerhouse. The existing access road will be rerouted around the north side of the switchyard. The switchyard will comprise four single-phase 115 kV (including one spare) step-up transformers, a TBD kV circuit breaker, a TBD/TBD kV step-down transformer, and associated equipment and materials.

5.6.2.1 **Design Requirements**

- The design will be in accordance with or exceed the latest applicable standards of governing national guidelines and local code requirements.

- Steel and aluminum/switchyard structure will be as designed with the loading and allowable stresses contained in NEMA publication SG-6 "Power Switching Equipment," part 36.

5.6.2.2 **Equipment and Materials**

TBD

5.6.3 **Transmission Line**

Transmission line design will be completed by the District.

5.7 **Access Roads and Bridges**

The access road and bridge improvements will be designed to meet Snohomish County standards for rural roads and be based on the following references:

- Guideline for Geometric Design of Very Low Volume Local Roads (ADT<400) (AASHTO, 2001)
- Policy on Geometric Design of Highway and Streets (AASHTO, 2001)
Preliminary road and bridge design criteria:

- Existing roads and bridges that require improvement will not be widened unless there is evidence of a site specific safety problem
- Roads required to access the intake will be raised to the 100-yr water level plus one foot of freeboard
- Roads and bridges will be designed for, or upgraded to meet, expected Project construction traffic. Design vehicle will be determined in the next phase.

6. Project Public Safety

The Project design will incorporate comprehensive safety measures to protect the public that are established in coordination with the District. Any riverine structure (natural or constructed) has safety risks associated with recreating around moving water and a drop of any amount. The design concept has incorporated the following measures that may be taken at or near the diversion and intake:

- Signage upstream of the diversion to warn residents and recreational river users of the danger of the diversion and the intake, as well as the natural waterfalls downstream which are currently unmarked
- Safe landings along shore for recreational river users to land, including signage.
- Hand rails, guard rails, and/or fences will be installed where appropriate to prevent unauthorized access and where fall hazards are identified where the public has access. These do not include natural topographic features such as rock cliffs.
- Safety boom or buoys may be installed across the river upstream from the diversion. Although the preliminary analysis of the diversion indicates there is low probability of a current pattern which could be harmful to swimmers; safety booms and buoys are intended to exclude recreational boaters or swimmers from the area immediately upstream and downstream from the weir and the course trash rack.
- Safety booms designed such that the orientation facilitates debris removal and improves the opportunity for the public to self-rescue. The safety boom may be removed during period of the year when there is limited recreation in the river.
- Video surveillance of the diversion and intake.
- The flow-pattern of the River indicates that typically the Project would not be in operation in the mid-July through mid-October timeframe. This would allow full-lowering of the weir during the season where recreational use is predicted to be highest.

7. Design Codes and Standards

In addition to local codes and standards, the latest editions of the following may be used in design of the project features:
- American Concrete Institute, “Building Code Requirements for Masonry Structures”, (ACI 530-05/ASCE 5-05)
- American Concrete Institute, “Building Code Requirements for Structural Concrete”, (ACI 318-08 and 318-99, Appendix A)
- American Concrete Institute, “Specification for Masonry Structures”, (ACI 530.1-05/ASCE 6-05)
- American National Standards Institute (ANSI)
- American Society of Heating, Refrigerating and Air Conditioning Engineers, Inc. (ASHRAE);
- Insulated Cable Engineers Association (ICEA);
- International Building Code
- International Electrical and Electronic Engineers (IEEE);
- International Society for Automation (ISA):
- National Electrical Code, (NEC), ANSI/NFPA 70 latest edition;
- National Electrical Manufacturers Association (NEMA);
- National Fire Protection Association (NFPA)
- Occupational Safety and Health Administration Standards (OSHA).
- Sheet Metal and Air Conditioning Contractor’s National Association (SMACNA);
- Underwriters Laboratories Inc (UL) – for items required by NEC or local Code
- Uniform Mechanical Code
- United States of America Department of the Army, “Planning and Design of


8. References


Carl Mannheim