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Portland District (CENWP)

BONNEVILLE 2ND POWERHOUSE CORNER COLLECTOR SITE SELECTION STUDY

FINAL SUBMITTAL

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1 INTRODUCTION

1.1 Purpose

The purpose of this study is to identify, evaluate, and recommend a location and a preliminary design for a high flow outfall as part of a corner collector surface flow bypass system, formed by modifying the ice and trash (I&T) chute at the Bonneville Second Powerhouse.

1.2 Surface Flow Bypass Program

The U.S. Army Corps of Engineers, Portland District (the District), in conjunction with the Walla Walla District and other Agencies, has developed a program to study, construct, and evaluate prototype surface flow bypass systems at the Corps' Lower Snake and Columbia River hydroelectric projects. The purpose of the program is to increase survival of migrating juvenile salmonids. At the Bonneville Project, a report was prepared[1] and a corner collector prototype was selected for further development at the Second Powerhouse.

The Bonneville Project (see Plate 1), operated and maintained by the District, is located approximately 40 miles east of Portland, Oregon, at River Mile 146.1 in the Columbia River Gorge. The First Powerhouse at Bonneville began operation in 1938 and the Second Powerhouse in 1982. Both powerhouses have been modified in recent years to improve survival of downstream migrating juvenile salmonids. Plans for enhancements to the existing juvenile bypass and intake screen systems for both powerhouses are ongoing. The corner collector surface bypass is an integral part of the long-term smolt protection plan at the Second Powerhouse.

1.3 Bonneville Second Powerhouse Corner Collector Project Overview

The "Surface Bypass Alternatives Study Report" for the Bonneville Second Powerhouse (B2) recognized the cost and testing advantages associated with the development and evaluation of a corner collector using the existing I&T sluiceway. Both hydraulic model testing and biological field-testing indicated that a B2 Corner Collector (B2CC) system could provide a beneficial surface flow bypass at B2[2].

[1] Harza and ENSR, 1996. Surface Bypass Alternatives Study Report
[2] INCA et al. 1997. Bonneville Second Powerhouse Prototype Corner Collector

As currently envisioned, a B2CC production system consists of three main components: intake, conveyance channel, and outfall.

The intake component utilizes the existing I&T intake with modifications. These modifications include:

- an entrance gate modification to allow the gate to be fully removed from the I&T sluiceway, so that maximum flow can enter the sluiceway. (Currently, the gate can only be lowered to El. 61.0 whereas the bottom of the intake is at El. 52.0.) This will allow a flow through the system of approximately 5,150 cfs at a forebay elevation of 74.5 feet, and
- the addition of a shaped concrete ogee immediately downstream of the entrance gate to provide a smooth transition between the intake and the conveyance channel.

The second component of the B2CC system is the conveyance channel. The conveyance channel could be the existing I&T sluiceway with modifications or a new conveyance channel, depending upon where the B2CC system outfall is located. If the existing I&T sluiceway modifications were utilized, it would include:

- replacement of the existing auxiliary water supply bulkheads with 'fish friendly' bulkheads, and
- sloping the sluiceway floor toward the outfall. (The existing sluiceway floor is level at El. +29.0.)

An outfall is the third component of the B2CC system. The existing I&T sluiceway outfall was not designed to discharge either juvenile salmonids or high discharges of water (over 1,000 cfs). Thus a new outfall design must be developed and a location selected in the tailrace.

1.4 High Flow Outfall Design Guidelines

High flow (HiQ) outfalls (over 1,000 cfs) are comparable to spillways and sluiceways, which are accepted smolt bypass routes, even though they were not designed specifically for fish passage, as HiQ outfalls will be. Because of this potential, it was prudent to develop, evaluate, and ultimately test a set of 'guidelines' for use in the design and construction of HiQ outfalls for surface flow bypass systems.

In a report[3] prepared for the District, preliminary guidelines for HiQ outfalls were proposed. The following excerpt and list of guidelines are from pages vii-viii of the Preliminary Guidelines Report.

> "The guidelines proposed are preliminary. Additional research will be required before they can be finalized. These preliminary guidelines were divided into two categories: location and design. The location guidelines provided direction as to the physical characteristics of a site that would be appropriate for a HiQ outfall. The design guidelines represented a set of standards that could be used during the engineering process. Building from the existing outfall criteria and using premises as a basis, the following preliminary guidelines for HiQ outfall development are proposed.
>
> *Locate where:*
>
> 1. Receiving water velocities without outfall influence are greater than 4 fps, unless site-specific velocities with an operating HiQ outfall are determined to be acceptable.
>
> 2. Receiving water characteristics, especially depth in combination with magnitude and trajectory of outfall discharge, are sufficient to prevent fish injury if they contact the bottom.
>
> 3. Eddies or back-rollers in the pre-outfall receiving water are minimal.
>
> 4. Predators are not concentrated near the outfall plume.
>
> 5. Adult migration (fishway entrances, shorelines, or known adult migration paths) will not be deleteriously affected by the HiQ outfall discharge and plume.
>
> 6. Project operations do not produce changes in hydraulic conditions that result in violations of other guidelines.
>
> *Design so that:*
>
> 7. Eddies or back-rollers in the outfall pool and plume are minimized.
>
> 8. Entry velocity for HiQ outfall jets is less than 50 fps for outfall discharges of 5,000 to 10,000 cfs.

[3] Johnsen, G, A. Giorgi, C. Sweeney, M. Rashid, J. Plump. July 1999. High Flow Outfalls for Juvenile Fish Bypasses; Preliminary Guidelines and Plans for Research Implementation.

9. The HiQ outfall does not cause the cumulative total dissolved gas concentration released by the project to exceed accepted criteria.

10. Adult fish that happen to encounter the outfall discharge are not prevented from continuing to move upstream, and those that may, leap at the discharge should not strike any solid objects and are not injured."

The background, working premises, relevant literature, research and implementation plans, and conclusions and recommendations associated with these preliminary guidelines are discussed in the "High Flow Outfalls for Juvenile Fish Bypass: Preliminary Guidelines and Plans for Research and Implementation"[4] report. This report should be referenced to better understand the concepts and justifications behind these preliminary guidelines. Supporting research data are presented in "Design Guidelines for High Flow Smolt Bypass Outfalls: Field, Laboratory and Modeling Studies"[5]. The preliminary guidelines were applied for the B2 HiQ Outfall Site Selection Study.

In addition, uncertainties in the preliminary guidelines were researched in the report[5] Based on field observations at the B2 sluice chute outfall, an outfall discharge jet with entry velocities up to 14.6 m/s (48 fps) provided safe conditions for the passage of juvenile salmonids. Higher velocities could not be created at the field site, thus could not be tested *in situ*. However, in laboratory tests[5] that created higher entry velocities and more severe conditions, higher injury/mortality thresholds were demonstrated – for 100-mm Chinook salmon at 15.2 m/s (50 fps) entry velocity, injury/mortality rates were zero, and for 150-mm Chinook salmon at 15.2 m/s, minor injury rates were zero, major injury rates were only 2 percent, and mortality was zero. Thus, across all sizes of juvenile salmonids tested, entry velocities up to 15.2 m/s (50 fps) provided benign passage conditions. The authors concluded that the guideline for prescribing acceptable outfall discharge entry velocity for high flow outfalls (>28.3 m^3/s, 1000 cfs) can be established at 15.2 m/s (50 fps), or more depending on site- and species-specific conditions. Accordingly, they recommended that the following actions related to the guidelines for high flow outfalls be considered:

- Revise the entry velocity guideline to read: "*Mean entry velocity for high flow outfalls can be up to 50 fps, and may be higher depending on site-specific conditions.*"

- Retain the preliminary bottom impact guideline until new information warrants a revision -- "*Receiving water characteristics, especially depth in combination with*

[4] Johnsen, G, A. Giorgi, C. Sweeney, M. Rashid, J. Plump. July 1999. High Flow Outfalls for Juvenile Fish Bypasses; Preliminary Guidelines and Plans for Research Implementation.
[5] PNNL, July 2001. Design Guidelines for High Flow Smolt Bypass Outfalls and Field, Laboratory, and Modeling Studies.

magnitude and trajectory of outfall discharge, are sufficient to prevent mechanical fish injury if they contact the bottom."

- Adopt the preliminary guidelines proposed in Johnson et al. (1999), with the caveat noted above for bottom impact and the revision proposed for entry velocity.

1.5 Report Organization

The purpose of this study is to identify, evaluate, and recommend a location and a preliminary design for a high flow outfall as part of a B2 corner collector surface flow bypass system. This report documents the results of this two-year effort.

Section 2 of this report presents the study approach. Basically, it was a two-staged approach that investigated outfall location and outfall type separately in the initial stage, and combined the two components into a single evaluation in the second stage.

Section 3 describes the methods, analysis and results of the general B2 tailrace characterization. Section 4 outlines the process, design, evaluation and results of the initial stage of the outfall location study. Section 5 describes the initial stage investigation of the potential outfall types. Each of these studies was accomplished relatively independent of each other.

Section 6 describes the process and efforts performed for the second stage, when the locations and outfall type where integrated. Section 7 provides the latest cost estimates for the outfall alternatives.

As a result of the work accomplished in the second stage, a final recommendation for an outfall location and type was developed. This recommendation is presented in Section 8.

2 STUDY APPROACH

This study entailed two main stages: an initial stage of exploration and investigation, and a second stage of refined and focused analysis. The two flow charts at the end of this section graphically display this evolution and help delineate the progress made in the selection process. (These charts are quite specific regarding outfall location and type. Detailed descriptions of the specific locations and outfall types are contained in Sections 4 and 5. The purpose of presenting the flow charts here is to help in understanding of the study approach.)

2.1 Stage 1

The initial stage involved four main studies:

- a general tailrace investigation,
- a preliminary study of potential tailrace ranges[6] for a HiQ outfall, resulting in a recommendation of two ranges for further evaluation in the second stage,
- a precursory hydraulic analysis of possible conveyance channel designs, and
- a preliminary study of HiQ outfall types, resulting in a recommendation of two outfall types for further evaluation in the second stage.

The general tailrace investigation and tailrace ranges utilized data developed on the 1:100 General Bonneville Model located at the Engineer Research and Development Center (ERDC, formerly the Waterways Experiment Station) in Vicksburg, MS. The outfall type evaluation included data developed on the 1:30 scale outfall model built and operated by ENSR in Redmond, WA.

Although the initial studies of outfall range and outfall type proceeded in a relatively independent manner from each other, the approach used for both studies was similar. A general description of this approach follows:

- Initial Alternatives - The first step in the Stage 1 approach was the development of alternatives. This initial list of alternatives was developed using brainstorming, alternatives identified in previous studies, alternatives that were similar to successful outfalls at other projects, and alternatives that appeared favorable during earlier

[6] Outfall ranges were general regions approximately 500 ft^2 for the outfall. Outfall locations or sites were specific points within ranges. See Section 2.3 for further explanation.

hydraulic modeling. The goal was to identify and consider as many reasonable alternatives as possible so that the Site Selection Study would ultimately recommend the 'best' HiQ outfall at B2.

- Screening – The next step was to reduce the number of alternatives by eliminating those that did not meet the HiQ Outfall preliminary guidelines or were unacceptable for obvious reasons. Professional judgment was used for this initial screening.

- Evaluation – For this step, evaluation criteria were developed and a score was assigned for each of the alternatives for each criterion. This score was determined based upon physical hydraulic model results and preliminary engineering and biological analysis. The result of this evaluation step was the selection of three outfall locations and four outfall types to carry forward.

- Consolidation – For the outfall types, a consolidation step was implemented. Based upon preliminary engineering, additional modeling, and professional judgment, the outfall types were modified and one type was eliminated from additional consideration. This step reduced the number of outfall types from four to three for the final step in Stage 1.

- Selection – Utilizing refined hydraulic modeling, two outfall locations and two outfall types were selected for integrated analysis during Stage 2.

At the conclusion of each of these steps, input was sought and received from the District staff. This input was incorporated into the evaluation process before that step's reduction/selection occurred and the next step was begun. In addition, meetings, project site visits and visits to the hydraulic models with the Regional Agencies and Tribes also occurred at several times during Stage 1. Input from these organizations was also incorporated prior to proceeding with the next step.

2.2 Stage 2

The second stage involved a more refined analysis, design and evaluation of the locations and outfall types. To perform this work, these two components of the outfall had to be combined and evaluated together. As with Stage 1, this stage of analysis and evaluation relied heavily on model results from both the ERDC 1:100 model and the ENSR 1:30 model. Throughout this stage, construction cost estimates were developed for utilization in the evaluation process.

As with Stage 1, input and recommendations from both the District staff and the Regional Agencies and Tribes were incorporated into the evaluation process of Stage 2. For this reason, the final recommendation described in Section 8 represents the conclusion of the Design team, the District staff and the Regional Agencies and Tribes.

2.3 'Range' versus 'Outfall Location'

For Stage 1, the objective for the outfall location study was to find a range, or general area, within the B2 tailrace or the Bonneville spillway tailrace that met the HiQ outfall preliminary guidelines. Thus, during this stage, outfall location is usually referred to as 'outfall range' because only a general area was being evaluated. During the later portions of this stage, and during all of the study in Stage 2, a specific location and orientation for the outfall was being developed. Thus, for Stage 2, the term 'range' is replaced by the term 'outfall location' to describe the specific site for the outfall.

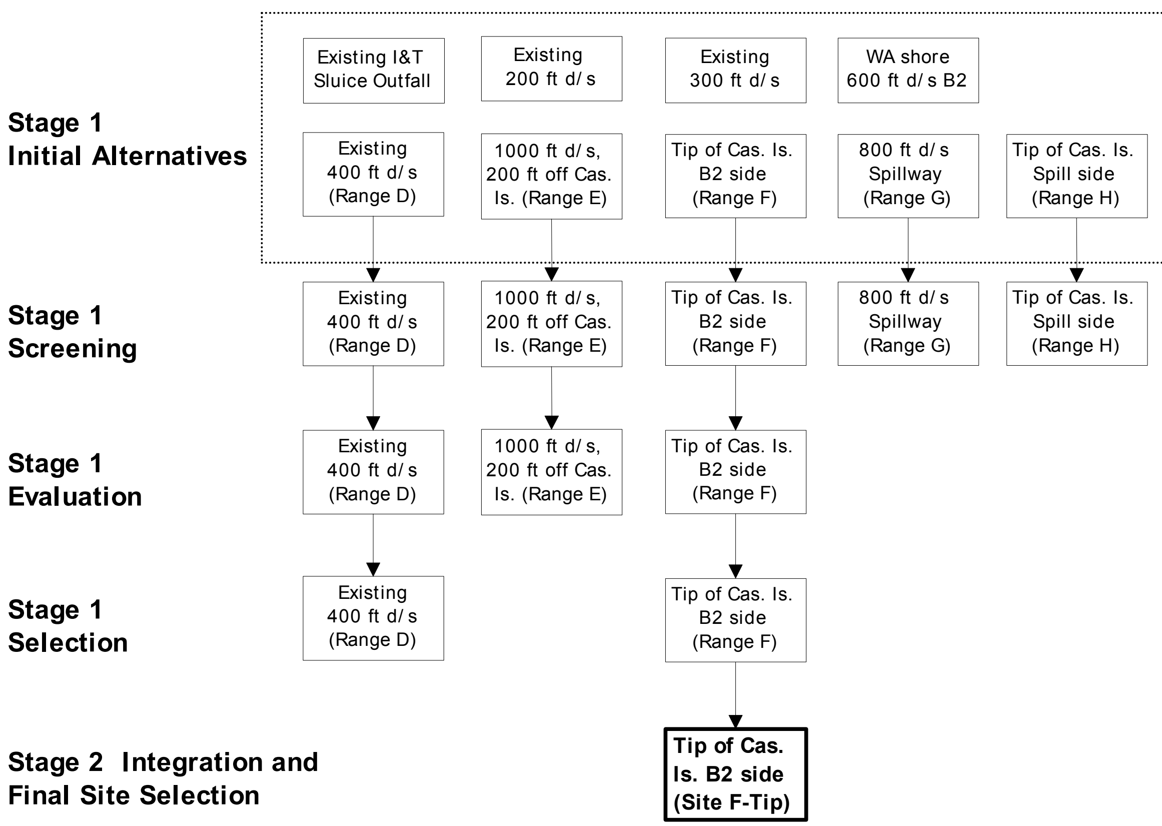

Figure 2-1 Progression for Range/Site Selection

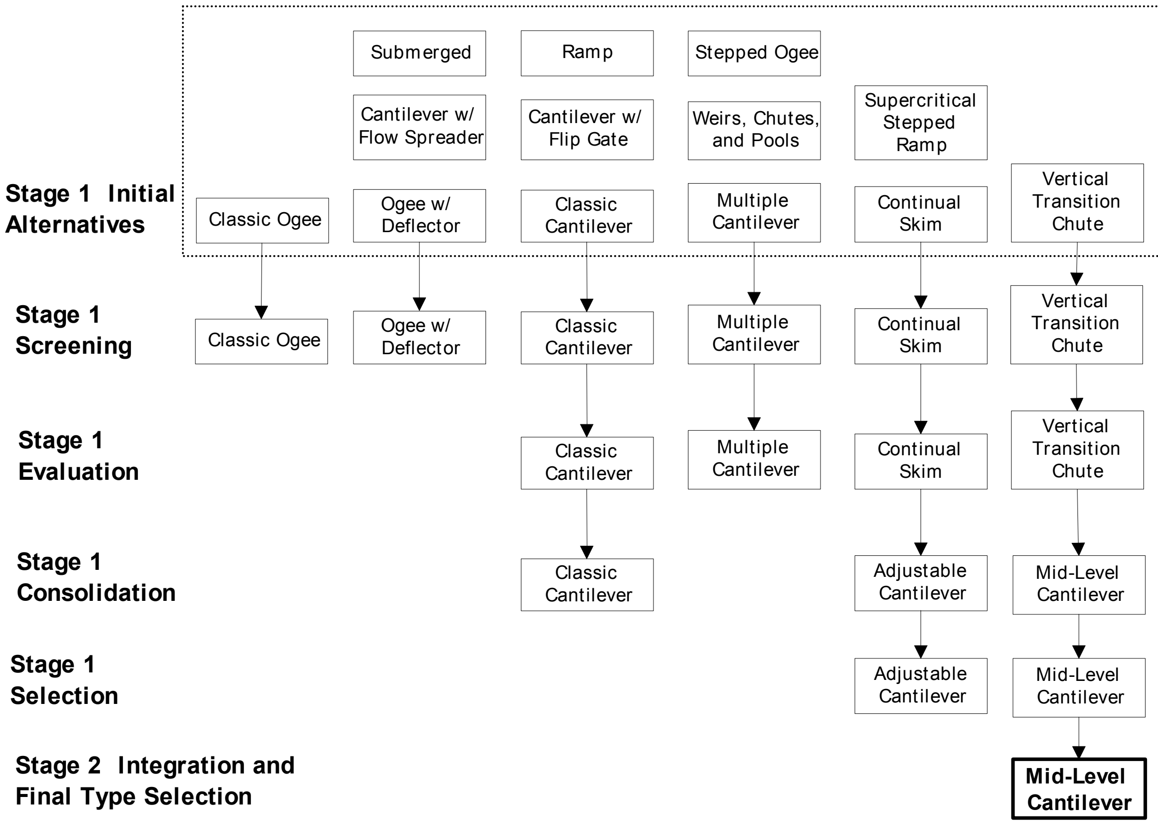

Figure 2-2 Progression for Outfall Type Selection

3 TAILRACE CHARACTERIZATION

The first step in identifying and evaluating alternative locations for high flow juvenile fish bypass outfalls in the Bonneville Second Powerhouse tailrace was to clearly describe the tailrace in terms of its physical, hydrologic, hydraulic, and biological characteristics. These characteristics provided the baseline information, by which various outfall sites could be compared to each other and to the high flow outfall design guidelines. They also provided the information used for design of outfall structures. The following sections describe B2 tailrace characteristics.

3.1 Physical

Physical characterization of the Bonneville Project tailraces includes both the bathymetry and the distribution of substrate materials. These are presented on Plates 2 and 3. The bathymetry data were acquired by Minister-Glaeser Surveying, Inc. in December 1998. The substrate information was obtained from District Drawing BD-20-104/02 dated April 28, 1997.

Bathymetry is variable in the B2PH, spillway, and B1PH tailraces. The B2 tailrace from the end of the draft tube excavation to approximately the downstream tip of Cascades Island has a fairly uniform bottom elevation in the range of –18.0 to –25.0 feet. The bank slopes on the south side of the channel are approximately 2.5 horizontal to 1 vertical and on the north bank less steep at about 4 horizontal to 1 vertical. The spillway tailrace bottom elevation is quite variable. It contains a scour hole to approximately bottom El. –40.0 feet, approximately 600 feet downstream of the center of the spillway. There is a bar of deposited material from the scour hole with approximate top El. –3.0 feet about 1,000 feet downstream of the spillway. Downstream from the bar, the bottom elevations vary between approximately –12.0 to –20.0 feet. In the main channel downstream from Cascade Island, the bottom is at approximately El. –25.0 to –30.0 feet. It gets shallower to about El. –20.0 feet at the tip of Bradford Island, El., –15.0 feet opposite the exit of the B1 tailrace, and then deepening to about El. –50.0 to 55.0 feet near the new JBS outfalls.

Bottom materials in the channels are composed of three general types:

- <u>Pre-Slide Alluvium (PSA)</u> – Primarily gravel, sand, and silt deposits, which should provide a fairly smooth bottom and little predator (or juvenile) fish cover.

- <u>Post Landslide Deposits (PLD)</u> – Undifferentiated river deposits, reworked landslide debris, and uniform Mica sand. This will likely include some larger materials, which may provide cover.

- Boulder Lag Deposits (BLD) – Remnant larger boulders that cannot be transported by river hydraulics. These materials will likely provide excellent cover for fish to hold.

The B2 tailrace downstream to the tip of Cascades Island is comprised primarily of the PSA materials. There are no bottom type data for the spillway tailrace, though it is likely that smaller materials have been transported from this area by the energy of spill flows and the remaining materials are of boulder size, or exposed bedrock.

3.2 Hydrologic

Design conditions for juvenile fish outfalls in the Bonneville project tailrace areas will be defined by project operations to meet requirements of the fish passage plan[7] in response to varying flow, tailwater levels, and power demands. The independent variables are the flow and tailwater levels. Power plant loads may be adjusted to meet the passage plan requirements. In order to support selection of the appropriate design ranges of flow and tailwater level for juvenile fish outfalls in the Bonneville Project tailraces, hydrologic analyses were performed by the District.[8]

Flow duration curves were developed based on the mean daily discharge readings for four upstream gauging stations including the Columbia River at The Dalles, OR and three tributaries to the Bonneville Pool, (Hood, White Salmon, and Klickitat rivers). The period of record of 1974-1999 was subdivided to provide information for each month, for the Majority of the Juvenile Fish Migration Period (1 April through 31 August), the Full Juvenile Fish Migration Period (1 March through 30 November) and the full calendar year (1 January through 31 December). Similar curves were also developed for river stage data at three gauges including the Bonneville Tailwater (plant records) and two USGS gauging stations below the project. The flow- and stage-duration curves for the full calendar year are presented in Figures 3-1 and 3-2.

Rating curves were also developed to provide the relationship between stage and flow. ENSR prepared a rating curve found in Figure 3-3, by pairing the stages and discharges from the tables in Figures 3-1 and 3-2 for each of the listed exceedence levels, i.e., the stage and discharge for an exceedence level of 1 percent are El. 30.1 feet and 436,300 cfs, etc. The curve is a composite of two different curves, which were constructed to provide a better fit to the data. The resulting curve provides a reasonable fit to the full calendar year data set developed by the District. It gives more weight to the higher flows and tailwater levels that

[7] USACE CENWP. 2000. Draft Fish Passage Plan for Corps of Engineers Projects.
[8] Soderlind, K. January 25, 2000. Draft Memorandum for the Record – Hydrologic Data in Support of High Flow Outfall Site Selection Letter Report. Prepared by USACE CENWP-EC-HY.

show less scatter as a result of tide and Willamette River backwater effects than would a single curve developed using the entire data set.

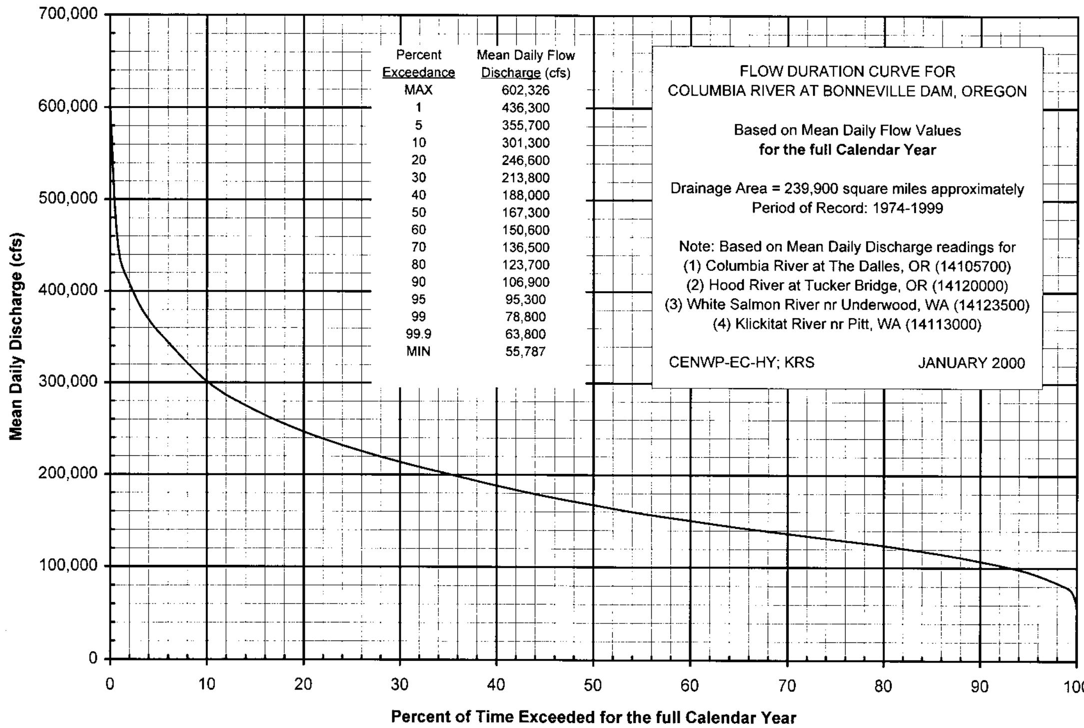

Figure 3-1 Flow Duration Curve for the Bonneville Project Site.

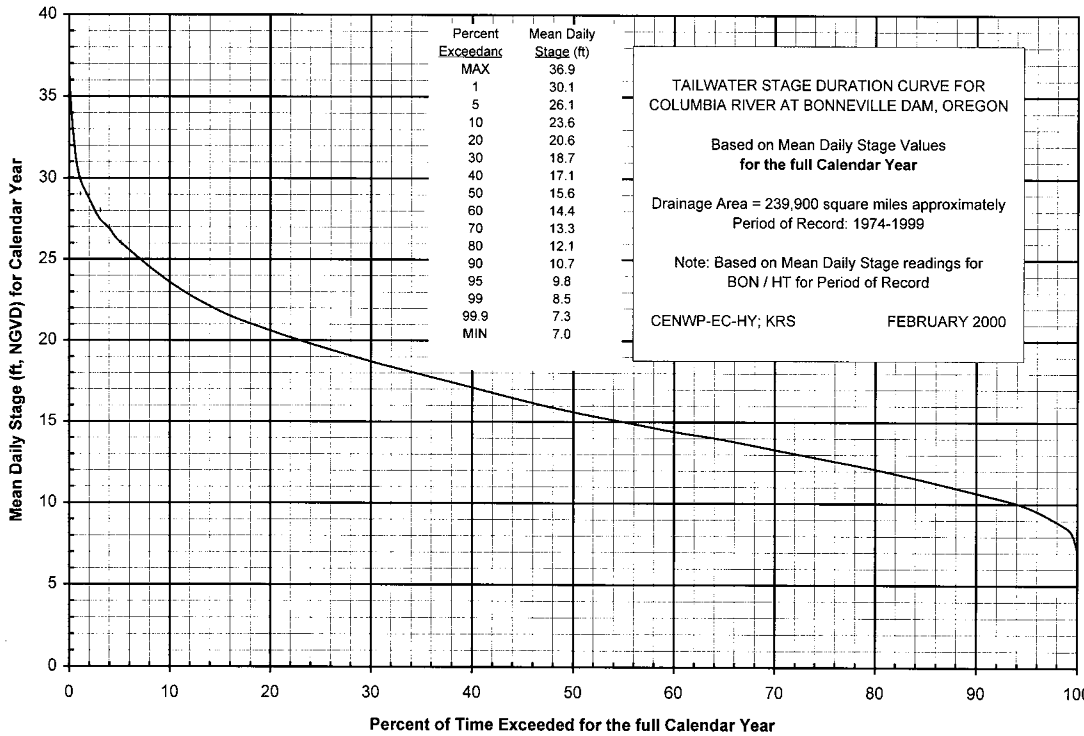

Figure 3-2 Tailwater Stage Duration Curve for the Bonneville Project Site

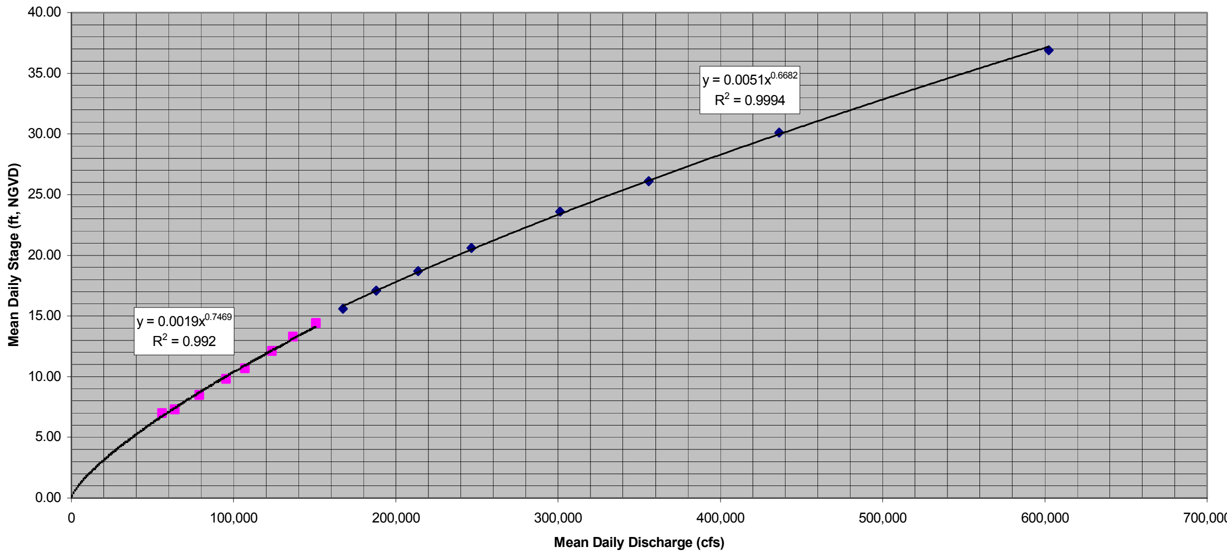

Figure 3-3 River Stage vs. Discharge Curve for the Bonneville Project Site Tailrace.

Table 3-1 summarizes the stage- and discharge-duration curve data for the three information periods and presents design ranges for these parameters.

Table 3-1 Design Conditions

Time Period	Exceeded 10-90 % of Time	Exceeded 5-95 % of Time	Exceeded 0-100 %of Time	Design Range
Mean Daily Stage, Feet				
4/1 – 8/31	25.68– 11.2	28.0 – 10.0	36.9 – 7.4	
3/1 – 11/30	26.6 – 9.5	24.0 – 10.3	36.9 – 7.0	28.0 – 8.5
1/1 – 12/31	23.6 – 10.7	26.1 – 9.8	36.9 – 7.0	
Mean Daily Discharge, kcfs				
4/1 – 8/31	400.4 – 88.7	445.2 – 73.3	584.1 – 55.8	
3/1 – 11/30	319.7 – 102.8	367.3 – 93.2	602.3 – 55.8	378.0 – 77.0
1/1 – 12/31	301.3 – 106.9	355.7 – 95.3	602.3 – 55.8	

The low end of the tailwater and flow range for each of the exceedance intervals is fairly independent of time period. The minimum-recorded tailwater level for all of the time intervals is approximately El. 7.0 feet. This corresponds to a total river flow of approximately 56,000 cfs. A tailwater level of 8.5 feet is exceeded 99 percent of the time during the migration period from April 1 through August 30. From the rating curve, Figure

3-3, this corresponds to a river discharge of approximately 77,000 cfs. Thus, El. 8.5 feet was selected as the minimum tailwater level for outfall design. If the outfall structure is designed to meet the high flow outfall guidelines presented in Section 1.4, and the tailwater level actually drops to the minimum recorded level of El. 7.0 feet, the jet entry velocity might increase from 39.3 to 40.5 fps. This 3 percent increase in velocity is expected to pose little risk to fish. The associated decrease in total river flow from 77,000 to 56,000 cfs is expected to have no measurable impact on the conformance to the other guidelines, though there may be some decrease in ambient receiving water velocities. This cannot be quantified in a general sense and requires data for the specific outfall site. The reduction in tailwater depth will pose a minor increase in risk of jet impingement on the riverbed.

The risk associated with operation of the outfall structure at tailwater levels and flows above the design range is low. Possible outcomes of these operations might include inundation or submergence of the outfall structure, which might provide local predator cover at the structure, and formation of a hydraulic jump in the structure or conveyance channel, which may lower exit velocities so adults could enter the structure. If the outfall structure would be designed to meet the guidelines for a high tailwater elevation of 28.0 feet, and the level increased to the maximum recorded El. 36.9 feet, this tailwater level would likely be associated with a total river flow of up to 602,000 cfs. The river flows would be high enough that the potential for predators to reach the outfall structure or adults to be falsely attracted to and enter it would be low. Use of the maximum-recorded tailwater levels and flows to define the design range does not seem to be warranted. It is suggested that the provisional maximum tailwater level for outfall design be set at El. 28.0 feet, which is exceeded less than 5 percent of the time during the April 1 through August 30 migration period. This corresponds to a total river flow of approximately 378,000 cfs, using the rating curve in Figure 3-3. These design ranges of flow and tailwater level were used to develop the outfall designs presented in this report.

In summary, a conservative 99 percent exceedence level tailwater of El. 8.5 feet was chosen for the low end of the design range, while a less conservative 5 percent exceedence tailwater level of El. 28.0 feet was chosen for the upper end. This approach was taken because the risk of fish injury due to jet entry and bottom impact, as well as predation due to low ambient velocity at low tailwater is somewhat greater than the risk of injury due to a low energy hydraulic jump in the channel or predation due to increases in local predator cover at high tailwater level.

Project operations for representative tailwater levels and flows within the design ranges are presented in Table 3-2. These operations were used to document velocities and flow patterns in the tailrace areas for relative evaluation of the outfall location ranges. These are the project operations, which were also used for documentation of the far-field outfall plume dynamics studied during the preliminary (Stage 1) range selection work at ERDC, described in Section 5.3 of this report.

The final outfall range and outfall structure type selected for design refinement testing were documented for flow conditions covering the 25 to 80 percent exceedance range of tailwater levels and flows during Stage 2, rather than only the representative conditions in Table 3-2.

Table 3-2 Scenarios for Preliminary HiQ Siting Investigations

Scenario	Name	Total Flow *(kcfs)*	Exceedence Level *(%)*[4]	Spill Flow *(kcfs)*[3]	B1 Flow *(kcfs)*	B1 Operations *(operating units @ flow in kcfs)*[1]	B1 Outfall *(kcfs)*	B2 Flow *(kcfs)*	B2 Operations *(operating units @ flow in kcfs)*[2]	B2 Outfall *(kcfs)*	TW Elev. *(feet)*
B2O-a	Low B2 Qout	154.0	68	74	0	none	0	75	11,12,16,17,18@14.0+5✪	5	14.4
B2O-b	Low B2 Qout, no spill	132.5	76	2.5	0	none	0	125	all @ 15.0+5✪	5	13.0
B2O-c	Med B2 Qout	253.8	37	119	0	none	0	129.8	all @ 15.6+5✪	5	20.6
B2O-d	High B2 Qout	334.2	17	74	115	all @ 11.4+1✪	0	140.2	all @ 16.9+5✪	5	24.8
B2O-e	High B2 Qout, high spill	335.2	17	119	69.4	1,2,3,5,9,10@11.4+1✪	0	140.2	all @ 16.9+5✪	5	24.8

* See ERDC Trip Reports 1 and 2 in Appendix A for spill patterns.

Notes:
[1] For Ph1 flows :+1✪ indicates approximately 0.3 kcfs for Juvenile Bypass System flow and 0.9 kcfs for Station Service Unit 0.
[2] For Ph2 flows "+5✪ indicates approximately 2.5 kcfs for each of the 2 fish units.
[3] Spillway flows include approximately 0.8 kcfs through B Branch Fishway S. end.
[4] April 1 to August 31 for period of record 1974 – 1999.

3.3 Hydraulic

Velocity data help characterize tailrace conditions for the purpose of outfall range evaluation. The data, presented in Figures 3-4 through 3-6, were generated through video imaging of floats in the 1:100 Bonneville General Model at ERDC. The data from these studies represent the average water velocity in the top 25 feet of the water column. Conditions documented were those designated as Scenarios B2O-a, b, and c, from Table 3-2. However, they did not include outfall flow. Therefore the designations in the figures have been modified to omit reference to outfall flow, e.g. scenario Bl-a, rather than B1O-a. Data were not collected for the high flow conditions, B1O-d and B2O-d and e, as these were not critical with respect to providing sufficient ambient flow field velocity to prevent predator holding.

There were two flow field features in the Bonneville Project Tailrace that were common to all of the tests. First, a counterclockwise eddy was formed in the entrance of the new navigation lock approach channel. Second, flow separated from the south bank downstream from the outlet of Tanner Creek. Downstream and offshore from this location, the flow field was fairly uniform and velocities exceeded the 4 fps guideline for ambient flow conditions for all three tests.

Velocities near the powerhouse were higher near the Cascades Island side of the channel than on the Washington shoreline. The streamline of highest velocities continued near Bradford Island and then shifted toward mid-channel at the downstream tip of Bradford Island. At a B2 flow of 74,500 cfs, a region of 4 fps and higher velocities extended from near the end of the B2 I&T chute outfall, 100-200 feet offshore along Bradford Island, almost continuously to the main channel downstream from the spillway tailrace channel. There was a 400-500 foot long region where the velocity dipped to 3.5 to 3.8 fps near the downstream tip of Cascade Island. The region of 4 fps and higher velocities was continuous when the B2 flow was increased to 129,800 cfs. As long as the spillway was operating (Figures 3-4 and 3-6), velocities near the south shore of the tip of Cascades Island always exceeded 4 fps. A large eddy formed in the exit of the B1 tailrace channel for all of these tests, where there was minimal B1 flow. In general, in the B2 tailrace, locations along the north shore of Cascades Island had higher velocities and the most direct egress of flow into the higher velocity regions in the main river channel.

In the spillway tailrace and with spill flows, regions along the south shore of Cascade Island most consistently had velocities in the 4 fps or greater range coupled with a direct egress of flow to the higher velocity regions in the main river channel.

Figures 3-7 through 3-9 present results of three-dimensional computational fluid dynamic (CFD) model simulations of typical low, mid, and high (115, 215, and 300 kcfs) river flows with B2 priority operations. Details of the flow correlations for these figures are contained in Appendix A, Trip No. 6, Table 3. These results show the same trends as the float information, but better delineate the various velocity zones.

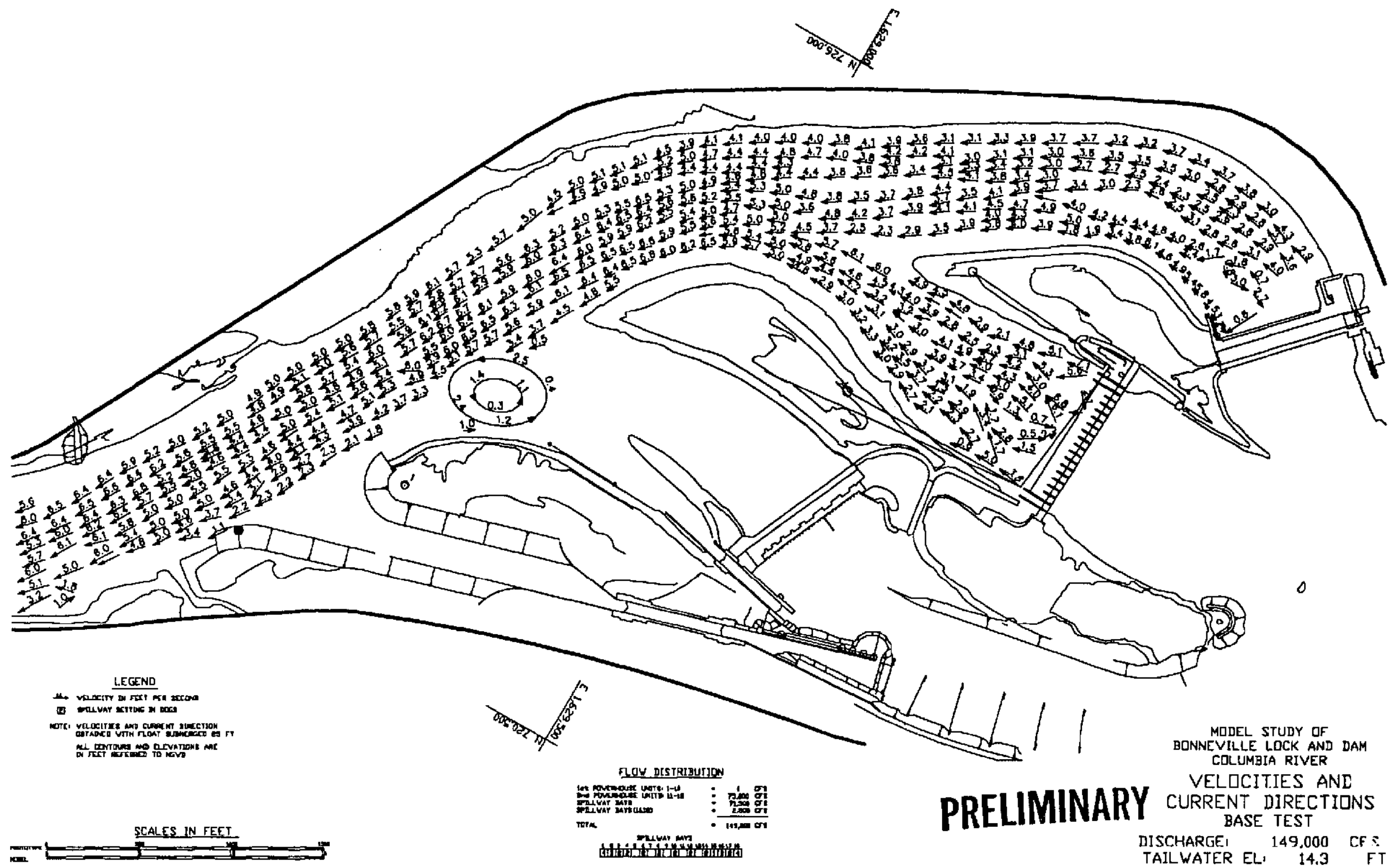

Figure 3-4 Flow Scenario B2-a

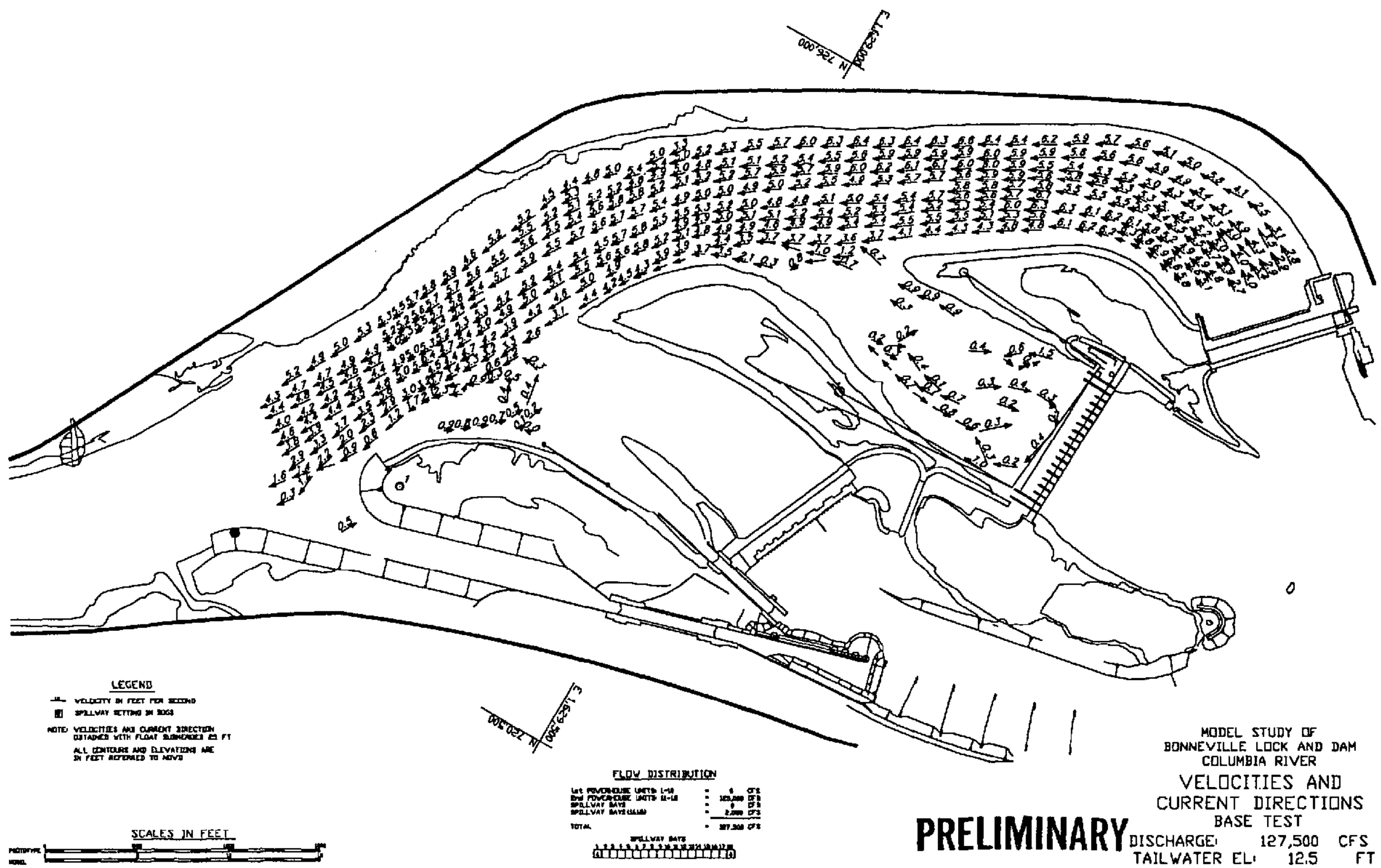

Figure 3-5 Flow Scenario B2-b

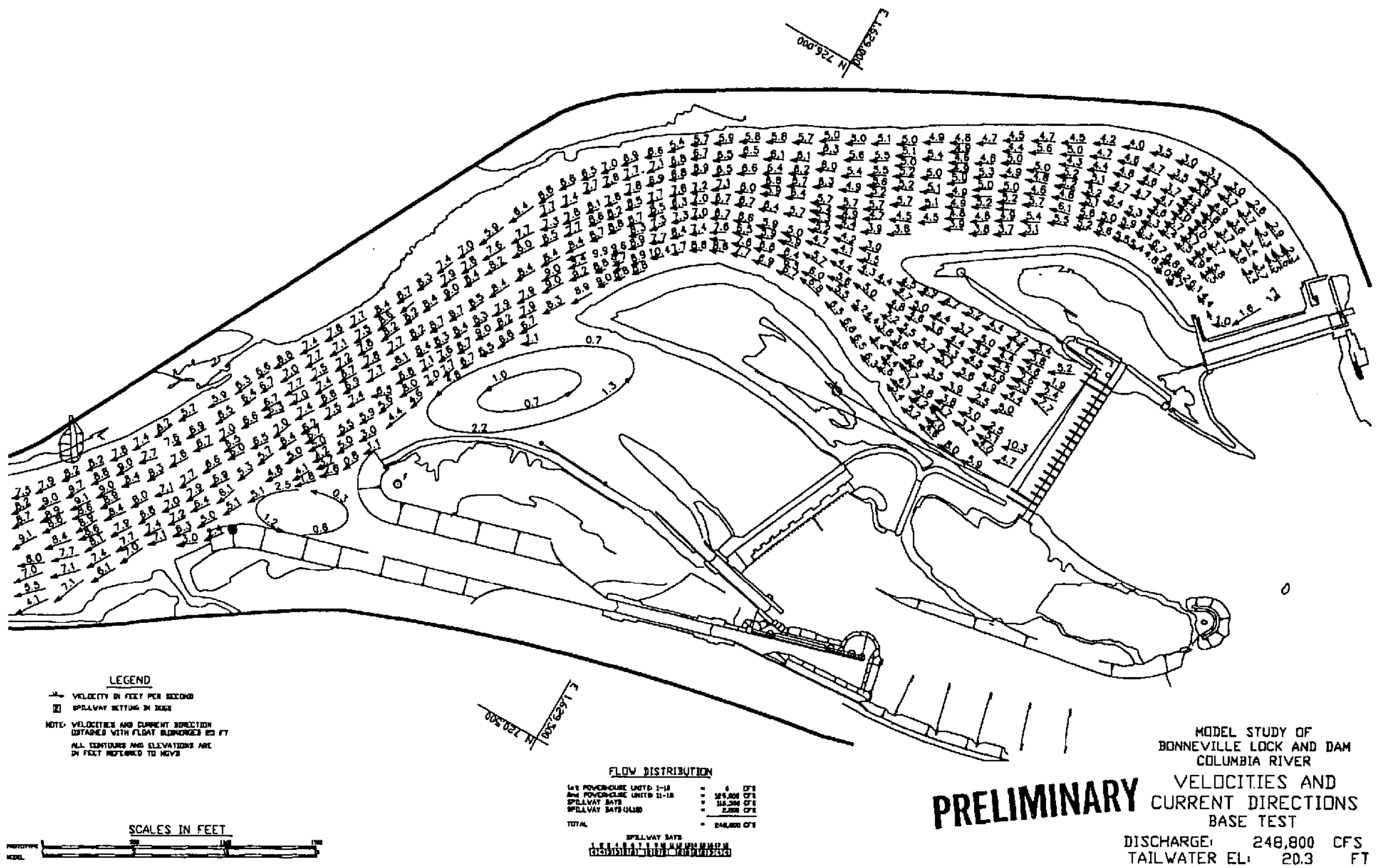

Figure 3-6 Flow Scenario B2-c

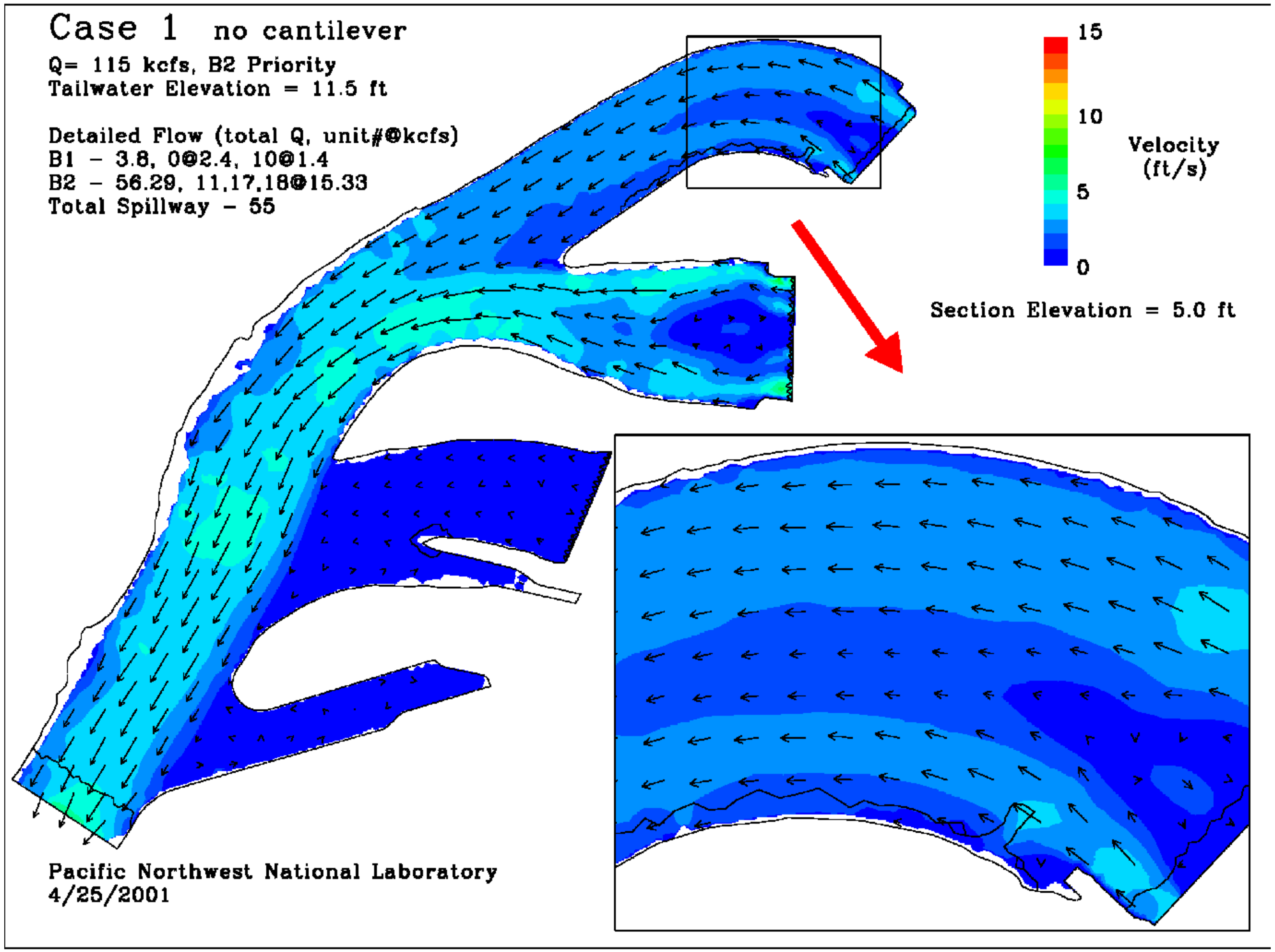

Figure 3-7 Case 1

Reference:

Pacific Northwest National Laboratory. July 18, 2001. Draft Memorandum for Record. Development and Application of a 3-D CFD Model for the Bounneville Project Tailrace for Proposed High Flow Outfall Structures.

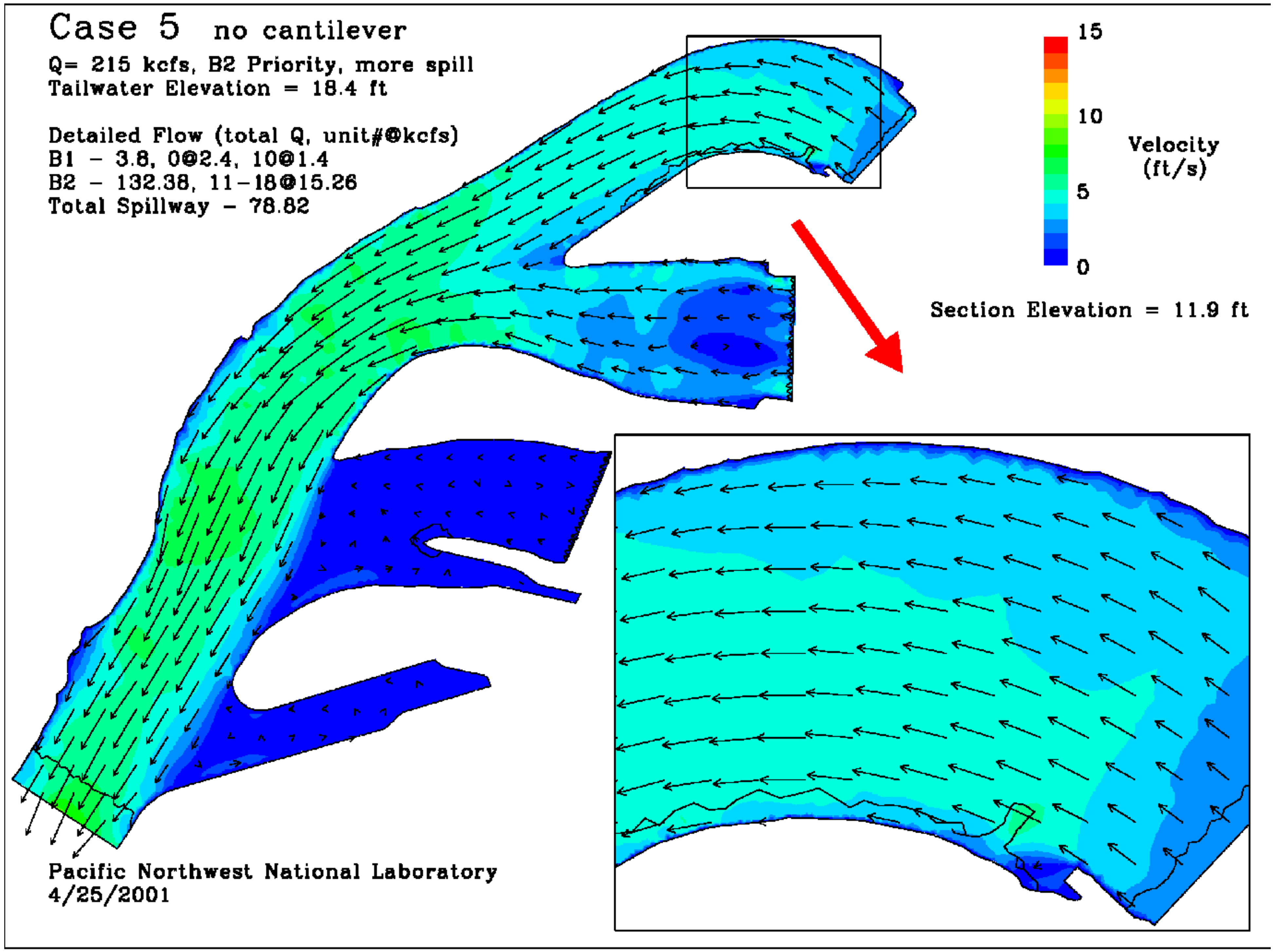

Figure 3-8 **Case 5**

Reference:

Pacific Northwest National Laboratory. July 18, 2001. Draft Memorandum for Record. Development and Application of a 3-D CFD Model for the Bonneville Project Tailrace for Proposed High Flow Outfall Structures.

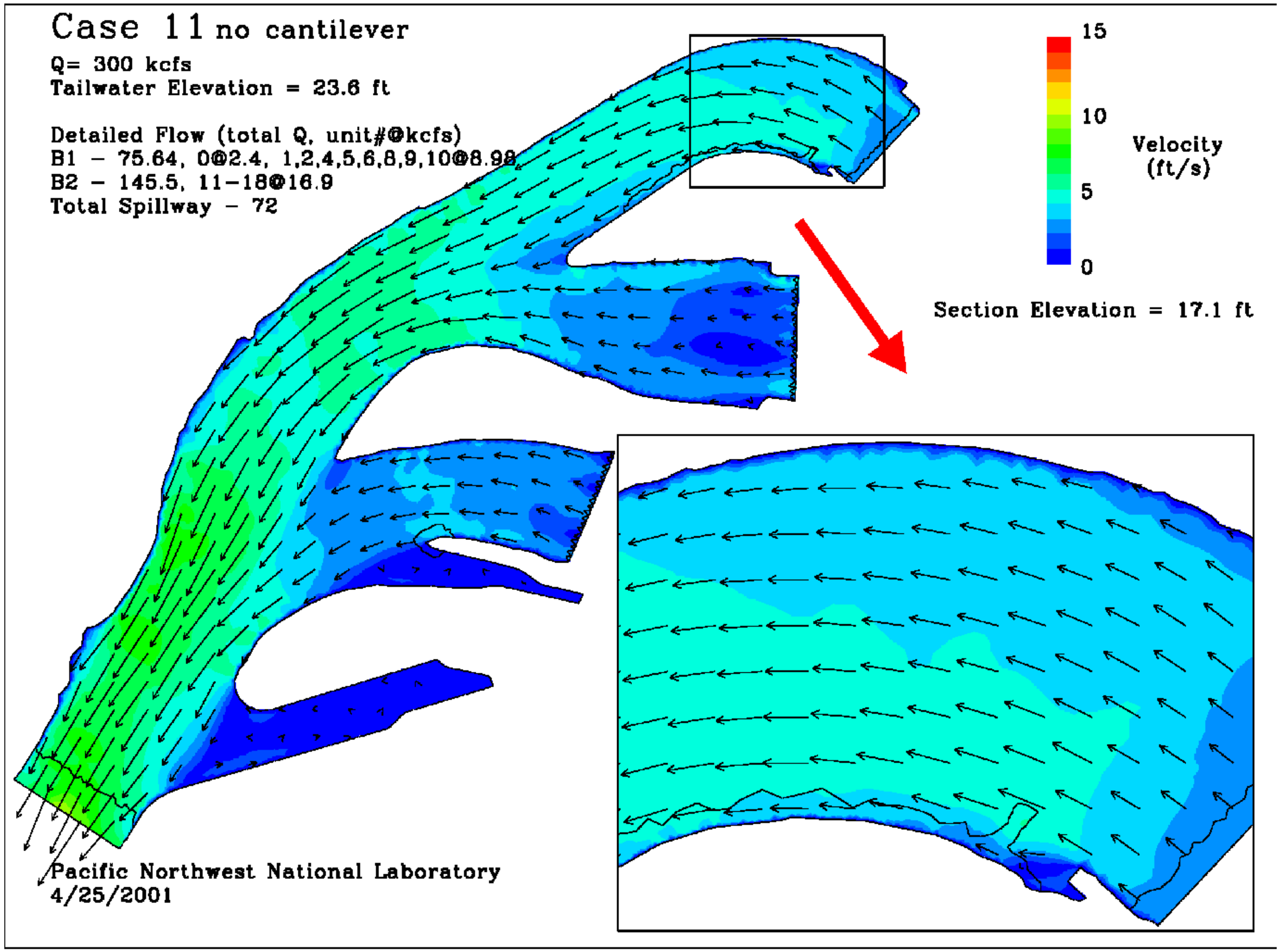

Figure 3-9 Case 11

Reference:

Pacific Northwest National Laboratory. July 18, 2001. Draft Memorandum for Record. Development and Application of a 3-D CFD Model for the Bounneville Project Tailrace for Proposed High Flow Outfall Structures.

3.4 Biological

Biological characterization of the Bonneville tailwaters includes two main topics for the purpose of outfall site selection: predator distribution and adult salmonid migration paths. Research on predator distribution and abundance was conducted by the Biological Resources Division of the US Geological Survey and the Oregon Department of Fish and Wildlife under the auspices of the Northern Pikeminnow Management Program within the Northwest Power Planning Council's Fish and Wildlife Program. Research on adult passage was conducted mainly by the University of Idaho as part of the Corps of Engineers' Anadromous Fish Evaluation Program. Predator distribution and adult pathway patterns presented here may change after new smolt outfalls and project operations are developed.

3.4.1 Predator Distribution

The primary predator in Bonneville Dam tailwaters, northern pikeminnow (*Ptychocheilus oregonensis*, previously called northern squawfish), is widely distributed (Figures 3-10 to 3-12). Catch of northern pikeminnow using electro-fishing was highest near rocky shoreline areas. In the B1 tailrace, abundance of northern pikeminnow was greatest near the low flow outfall for the submersible traveling screen bypass system (Figures 3-10 to 3-12). They were also prevalent along Robbins and Bradford islands. In the B2 tailrace, northern pikeminnow seemed to be concentrated in the tailrace at the north and south ends of the powerhouse (Plate 4). High densities were also observed in areas along the Washington shore and at the tip of Cascades Island (Plate 4).

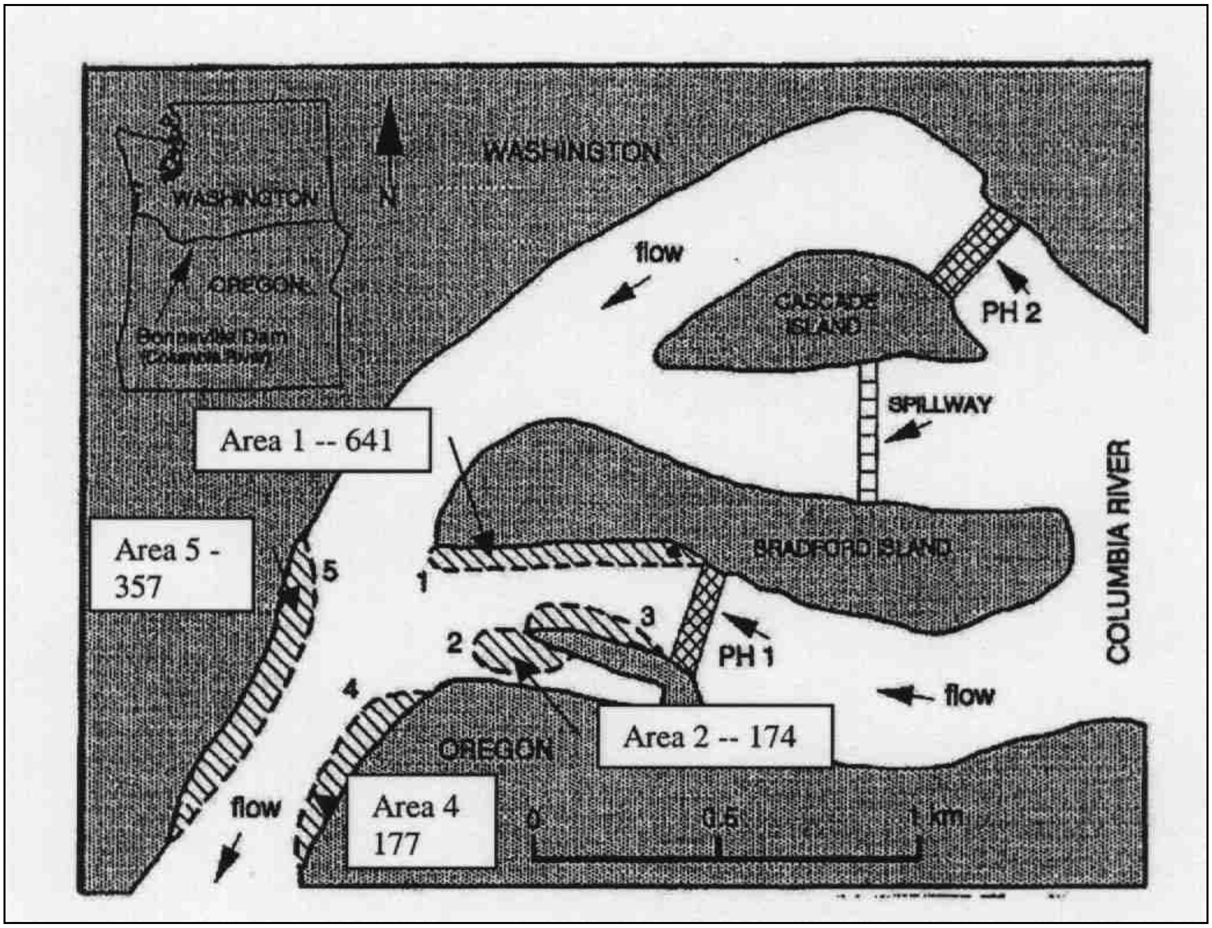

Figure 3-10 Pikeminnow Sampling (Sheet 1 of 2)

Catch of northern pikeminnow (equal effort) at four sampling areas (1, 2, 4, 5 diagonal hatching) at Bonneville Dam tailrace and location in Area 1 where tagged salmon were released. Modified from Figure 1 in Peterson et al. (1994).

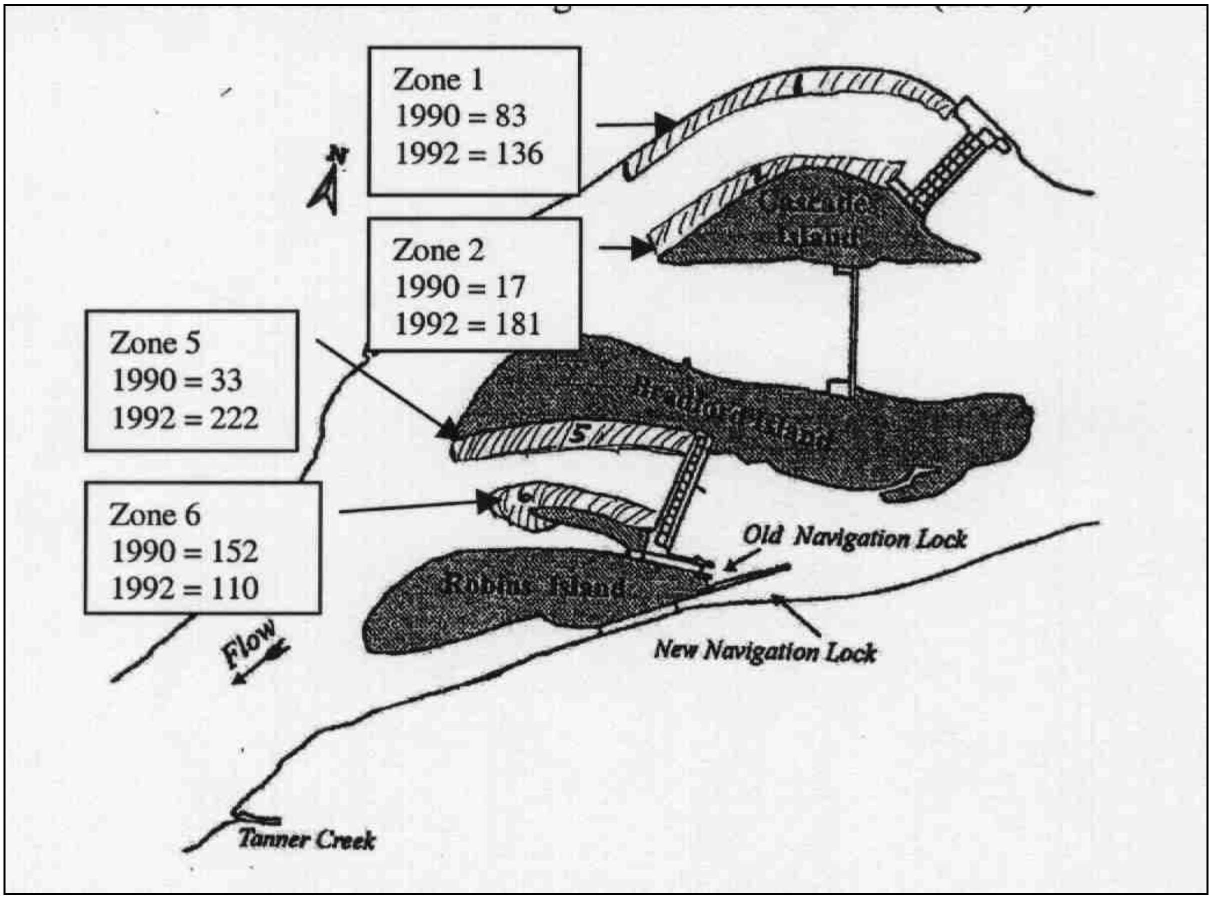

Figure 3-11 Pikeminnow Sampling (Sheet 2 of 2)

Catch of northern pikeminnow (equal effort) for Zones 1, 2, 5, and 6 during the predator indexing study in 1990 (Petersen et al. 1991) and the smolt survival study at Bonneville Dam in 1992 (Poe et al. 1994).

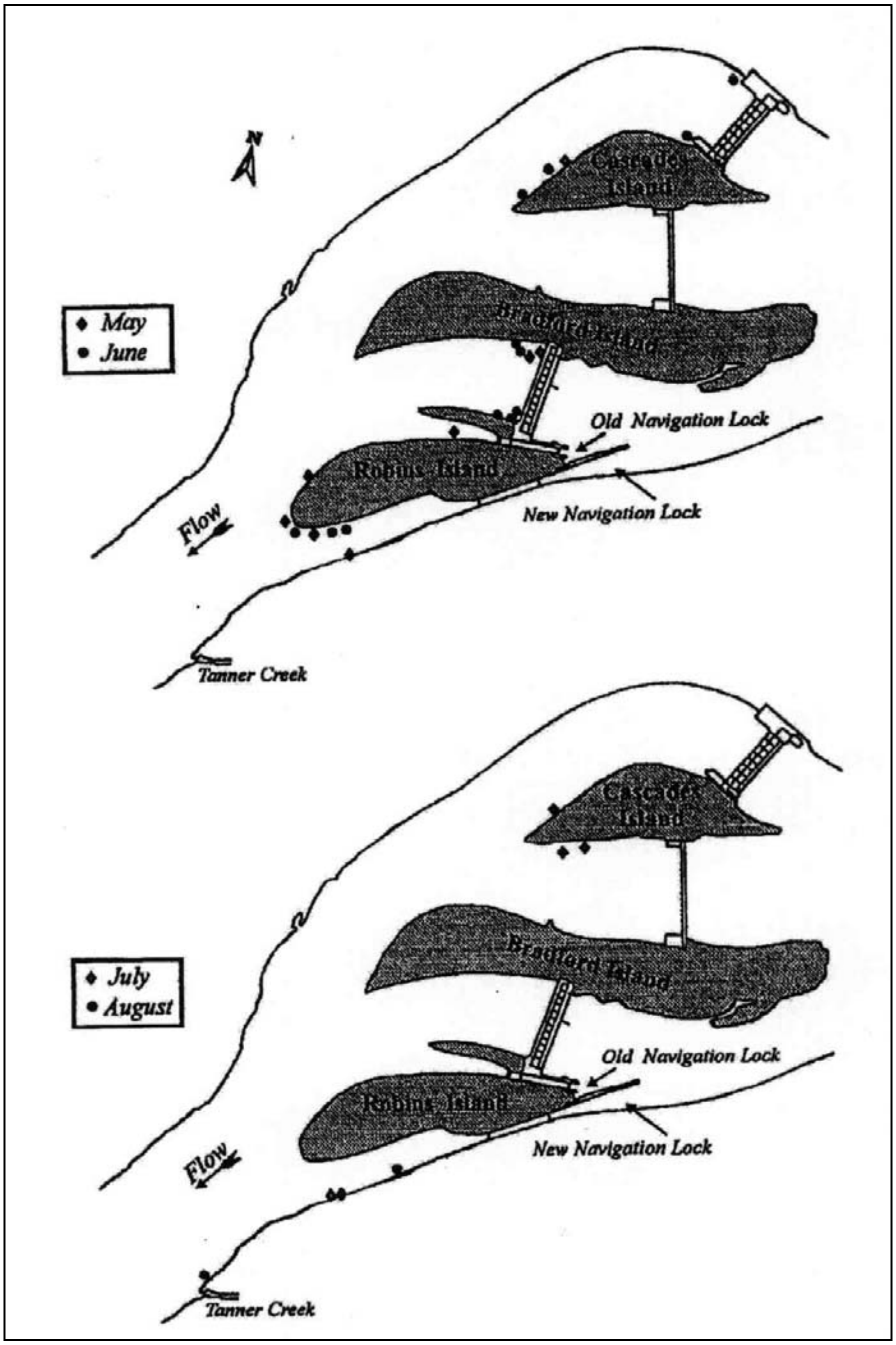

Figure 3-12 Pikeminnow Tracking

Mobile tracking contacts of radio-tagged northern pikeminnow in the tailrace area of Bonneville dam in May (n=8), June (n=12), July (n=5), and August (n=2) in 1996. Modified from Figure 10 in Knutsen et al. (1996).

Predators were not distributed completely across entire tailwater channels; they were mostly distributed near shorelines where there was cover. In general, northern pikeminnow prefer littoral, relatively shallow areas in reservoirs and dam forebays and tailraces[9]. Therefore, outfall sites will have to be away from shorelines (> ~100 feet perhaps). Also, receiving water velocities will be higher away from shore than near shore, improving outfall conditions.

Northern pikeminnow are very mobile fish[10]. They can move readily from one area to another in search of prey. For this reason, the 'known' distribution of these fish may change as the distribution of smolts changes due to new outfalls and project operations. In addition, predators have been observed in the vicinity of all potential outfall sites. Predators in the Bonneville Dam tailrace are prevalent and cannot be easily avoided. However, we can locate and design high flow outfalls for B1 and B2 such that conditions conducive to predation are minimized (e.g., see the preliminary high flow outfall guidelines).

3.4.2 Adult Salmonid Migration Pathways

Data on adult migration pathways at Bonneville Dam as determined from radio-tag studies come from the University of Idaho (M. Keefer's presentation at Anadromous Fish Evaluation program Annual Review Meeting in Walla Walla, WA October 15-18, 1999, and T. Poe personal communication with K. Tilotti on November 2, 1999 and C. Perry on November 3, 1999). The following information comes mostly from mobile radio tracking of fish released 10 miles downstream of Bonneville Dam in spring and summer 1996. Recall, 1996 was a high water year. Caution should be used with these results until the full complement of data from other study-years has been reported.

Adults tended to migrate relatively close to shorelines as they approached Bonneville Dam. Steelhead (< ~25 feet) were usually closer to shore than Chinook (~25-50 feet). Fish approaching the dam along the Oregon shore tended to migrate into Tanner Creek some, then into the new navigation lock channel, and finally into the B1 tailrace. Fish approaching B2 migrated almost exclusively along the Washington shore. Upon encountering the B2 powerhouse, they began wandering in the B2 tailrace. The outfall of the I&T chute at B2 attracted radio-tagged adults in 1996. These fish, however, did not hold there. Some fish from B1 and B2 tailraces moved along shorelines into the spillway stilling basin.

Adults could be attracted to a new high flow outfall, just as they are now to the spillway, B1 sluiceway outfall, and B2 I&T chute outfall. However, researchers do not expect them to hold there for long because of the high flow environment. It will be important to place new outfalls away from shorelines, out of adult migration paths. In conclusion, a properly located and designed high flow outfall should have minimal impact to adult migrations.

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[9] Ward et al. 1995 and Martinelli and Shively 1997
[10] Knutsen et al. 1996

3.5 Basis for Design

In summary, the following points from the site-specific B2 tailrace characterization, coupled with the preliminary high flow outfall guidelines, form the basis for outfall design.

- The design tailwater range is from El. 8.5 to 28 feet, corresponding to an exceedance interval during the majority of the juvenile fish migration season (April 1 through August 30) of 99 to 5 percent, respectively. The design river flow range associated with the design tailwater levels ranges from 77,000 to 378,000 cfs.

- The river bed elevations throughout the B2 tailrace are fairly uniform resulting in depths at the minimum design tailwater level of El. 8.5 feet from 26.5 to 33.5 feet.

- The main channel of the B2 tailrace to the tip of Cascades Island consists primarily of PSA deposits, which will not provide much hydraulic cover for predators. Rip-rap in shallow shoreline areas along Cascades Island and the Washington, however, will provide predator habitat. Predators are known to inhabit these areas.

- In general in the B2 tailrace, locations along the south shore of Cascades Island have higher velocities and a more direct path of egress of flow into the higher velocity region in the main river channel downstream than regions along the Washington shore. In this region of high water velocities, the magnitudes are consistently greater than or equal to 4 fps when the B2 flows are at least 74,500 cfs, with the exception of a region near the tip of Cascades Island. When B2 flows are 129,800 cfs or higher, the region in excess of 4 fps is continuous.

- Adult salmon and steelhead migrating upstream in the B2 tailrace tend to prefer the Washington side of the river.

- Therefore, outfall ranges away from shallow, shoreline areas in the B2 tailrace along Cascades Island from the dam to the tip of the island are likely possibilities.

4 OUTFALL TYPE SELECTION

4.1 Introduction

Finding the appropriate tailrace range for a HiQ outfall is highly dependent upon the bathymetry, substrate, and receiving water characteristics of the range. However, the overall performance of the outfall, and ultimately the bypass system, is also very dependent upon the type of outfall utilized. In order to maximize the performance of a HiQ outfall, the outfall type and tailrace range must ultimately be evaluated together as a unit. This outfall range/type combination evaluation is covered later in Section 6.

This section describes the initial identification, screening, preliminary evaluation, consolidation, and initial selection of outfall types. This initial process represents Stage 1 of the outfall type selection process. It was necessary to narrow down the number of outfall types to those that are most likely to perform satisfactorily in the B2 tailrace conditions. Because outfall types were to be evaluated on the ENSR 1:30 hydraulic model and range/type combinations were to be tested on the ERDC 1:100 model, a manageable number of outfall types were necessary. A flow chart depicting this Stage 1 process is shown on the next page (Figure 4-1).

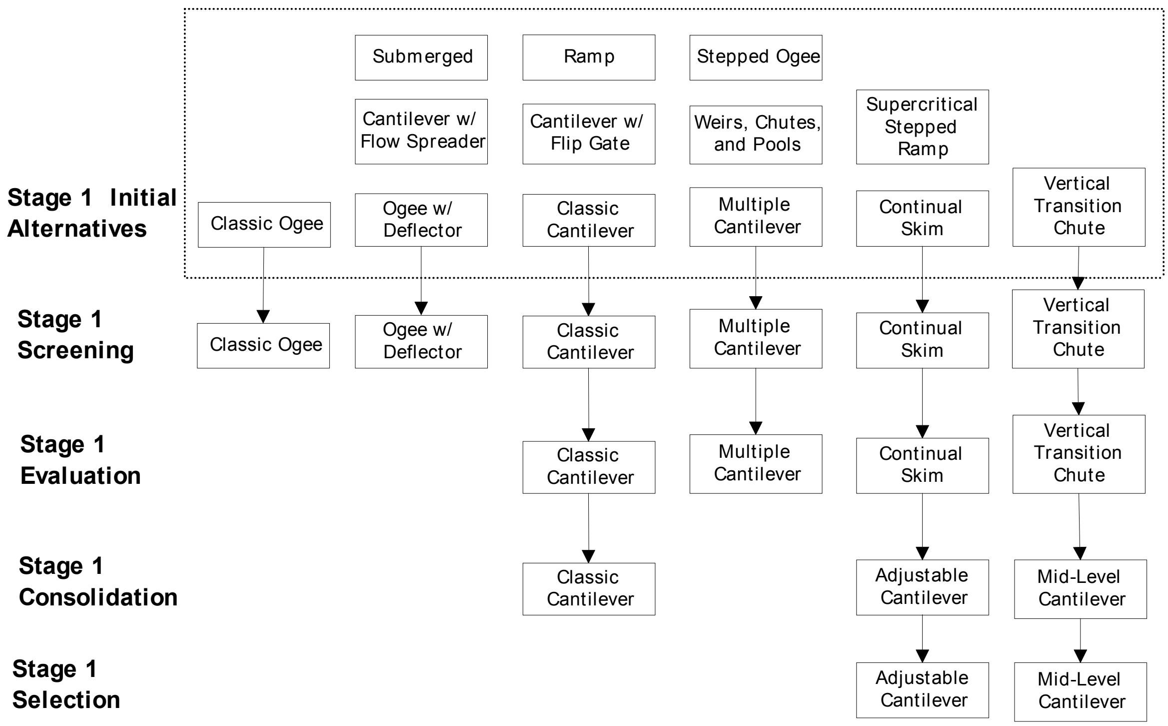

Figure 4-1 Progression for Outfall Type Selection – Stage 1

4.2 Initial Alternatives

The initial step in the outfall design process was to brainstorm the various types of outfalls that could be utilized. A discussion of the various types of outfalls that were identified, along with simplified sketches of the types, follows. (The "sketches" shown as Figures 4-2 through 4-11 are conceptual. Water flow lines are not accurate, nor meant to imply a hydraulic performance.)

4.2.1 Classic Ogee

This structure would be similar to the ogee shapes currently in use at spillways along the Columbia and Snake Rivers and is shown in Figure 4-2. The design of these structures requires low velocity subcritical flow immediately upstream of the ogee crest. Therefore, the supercritical flow exiting the I&T sluiceway would have pass through a hydraulic jump within the conveyance channel. The jump would be classified as 'weak' having a smooth

rise in water surface with small rollers. The flow depth downstream of the jump would be approximately 20 to 30 feet.
To stay within the range of available design standards, Froude numbers less than approximately 0.3 must be obtained prior to the ogee crest. This can only be accomplished by widening the conveyance channel upstream of the structure. The requisite channel width will be on the order of 30 to 40 feet.

The flow over the ogee would accelerate down the face of the structure and have a strong vertical component upon entry into the tailrace. This would cause the flow to dive deep into the water column generating a large top roller and hydraulic jump at the toe of the structure. It is very likely that the outfall flow would reach river bottom regardless of tailwater depth.

The size and configuration of the structure would act as significant obstruction to the ambient river flow. This would create a large 'hydraulic shadow' of slow moving, and possibly eddying, flow downstream of the conveyance structure. It would also eliminate any ability to improve local ambient river flow velocities through entrainment by the outfall flow.

The large roller at the toe of the structure would result in a significant amount of air entrainment and highly turbulent flow conditions. This would work in combination with the diving flow to increase dissolved gas levels in the outfall plume. The diving flow would also promote the occurrence of bottom strike.
The structure would not have any adverse impacts to adult salmonids other than the increased dissolved gas levels in the outfall plume. The entry velocity of the outfall flow will be well above 20 fps, preventing entry of adults into the conveyance channel, even at high tailwater levels.

4.2.2 Ogee with Deflector

This design would add a flow deflector to face of the standard ogee structure as shown in Figure 4-3. The deflector would turn the outfall flow horizontal prior to reaching the bottom of the structure. The deflector design would be similar to those currently in use throughout the region as dissolved gas abatement devices. The size of the top roller would be decreased and a bottom roller would be generated underneath the deflector. The deflector would significantly decrease the opportunity for bottom strike and reduce the levels of dissolved gases in the outfall plume as compared to the standard ogee structure.

4.2.3 Ramp

Rather than let the outfall flow fall uncontrolled into the tailrace a ramp could be used to guide it into the tailrace as shown in Figure 4-4. This would give the outfall flow a steady angle of entry for tailwater elevations below the conveyance channel invert and provide control over the downward component of the outfall flow and, therefore, reduce the opportunity for scour, bottom strike, and increased dissolved gas levels.

4.2.4 Stepped Ogee

This design would replace the smooth downstream face of the standard ogee structure with steps as shown in Figure 4-5. The purpose of the steps would be to dissipate some of energy of the outfall flow before it enters the tailrace. This would decrease the entry velocity of the flow and the amount of energy to be dissipated in the tailrace. The flow down the face of the structure would skim across each step trapping small cells of recirculating fluid. The energy required to recirculate these cells would be acquired from the main flow through turbulent momentum exchange. The exchange would occur via shear stresses at the boundary between the flow and the cells. This phenomenon would be the primary contributor to the energy dissipation of a stepped ogee. This type of structure would also require subcritical approach flow and a hydraulic jump in the conveyance channel upstream in order to function properly.

Design information on stepped ogees is limited. A model study of the structure will be required to properly quantify the energy dissipation across the structure and determine the tailwater entry characteristics of the outfall flow. The model must be of sufficient scale not to impede the natural processes governing the energy exchange between the main flow and recirculating cells.

The flow over a stepped face would be highly turbulent and entrain more air than a standard ogee profile. The level of air entrainment would be sufficient for air to be present in the boundary layers between the main flow and recirculating cells. The presence of air within the boundary layers would reduce the shear stresses between them. This, in turn, would reduce the amount of energy dissipated across the steps. Therefore, to properly model a stepped ogee structure, the model scale cannot artificially restrict the level of flow turbulence, air entrainment, or turbulent shear stresses at the structure.

Determination of a model scale will require a preliminary design of the structure, a review of model scaling relationships, and a review of previous stepped ogee model studies. It will also be driven by several practical considerations such as laboratory space and discharge capacity, availability of materials, cost of construction, and ease of operation and instrumentation. However, limited research on the subject suggests a minimum model scale of 1:20 would be required to correctly represent prototype behavior.

The amount of energy dissipated across the steps will likely not be sufficient enough to drastically reduce the outfall entry velocity. Therefore, the behavior at the toe of the structure will be very similar to that of the standard ogee.

4.2.5 Classic Cantilever

This structure would be similar to the many low flow outfall designs that are currently in use throughout the region. The supercritical flow exiting the I&T sluiceway would be maintained in the conveyance channel until the outfall site was reached. The flow would then be allowed to exit the conveyance channel and enter the tailrace in an uncontrolled manner, as shown in Figure 4-6.

The entry velocity of the outfall flow will be a combination of the conveyance velocity and the fall velocity gained through gravitational acceleration. Therefore, limits on the invert elevation of the terminus will be required to meet the entry velocity criterion. Limits will also be required to maintain exit velocities of 20 fps, necessary to prevent adult entry, at high tailwater levels.

At low tailwater elevations, the outfall flow will have a strong downward component upon entry into the tailrace. This will plunge the flow to the river bottom and increase the opportunity for bottom strike. The impact of flow on the river bottom will develop a large scour hole in the bed, where the flow energy will be dissipated. The plunging flow will also entrain air at impact with water surface and drive it deep in the water column increasing dissolved gas levels in the outfall plume.

At high tailwater elevations, the outfall structure may become inundated by the tailwater resulting in a hydraulic jump within the conveyance channel. The jump would likely be ‘weak’ having a smooth rise in water surface with small surface rollers. If the walls of the structure are not above the tailwater level, river water will be entrained into the outfall flow prior to its terminus.

4.2.6 Multiple Cantilever

This option involves the use of two or more cantilever outfalls at different invert elevations, as shown in Figure 4-7. Each cantilever would be used for specific range of tailwater elevations so the outfall flow could always be released near the tailwater level. This would minimize the downward component of the outfall flow upon entry into the tailrace resulting in a decreased opportunity for bed scour and bottom strike. It would also decrease the dissolved gas levels in the outfall plume.

4.2.7 Cantilever with Flip Gate

A cantilever with a flip gate at its terminus would steepen the jet trajectory, as shown in Figure 4-8. At B2 this design is not advisable because of B2's relatively shallow tailrace.

4.2.8 Cantilever with Flow Spreader (Spoon)

This would be the classic cantilever design fitted with a flow spreader at its terminus, as shown in Figure 4-9. The purpose of the flow spreader is to minimize initial plunge depth and maximize initial flow dispersion. This would decrease the opportunity for bottom strike, reduce riverbed scour potential, and reduce dissolved gas levels. The spreader works by fanning the flow out into a thin sheet prior to its entry in the tailrace. The sheet flow dissipates more quickly in the tailrace than the plug flow of a traditional cantilever.

4.2.9 Vertical Transition Chute

This structure places a horizontal flow deflector at the bottom of a ramp to minimize the downward component of the outfall flow, as shown in Figure 4-10. The deflector would significantly decrease the opportunity for bottom strike and reduce the levels of dissolved gases in the outfall plume as compared to the standard ramp structure.

4.2.10 Continual Skim

The terminal end of the continual skimming outfall would adjust to tailwater elevation, as shown in Figure 4-11, so that the outfall flow would always skim across the surface of the tailwater. This would limit the opportunity for bottom strike and minimize the potential for supersaturating dissolved gas in the outfall plume. Once the logistical difficulties of changing the exit invert elevation were overcome, this outfall would provide more consistent performance than the other options.

4.2.11 Submerged

A submerged outfall would require a hydraulic jump to be formed in the conveyance channel to transition flow to subcritical. Flow would then be introduced via a downwell into a pressurized conduit, which would discharge a submerged jet into the tailrace. This outfall type is not illustrated in this report, but would be similar in concept to, but larger in magnitude than the old JBS outfall previously employed in the B1 tailrace.

4.2.12 Supercritical Stepped Ramp

A ramp conveying supercritical flow to the tailrace could be configured as a series of steps, rather than a smooth surface. The intent of this concept would be to dissipate energy, similarly to the stepped ogee described in Section 4.2.4. However, the trajectory of supercritical flow will be very flat and step heights would necessarily be very small to provide contact with the bottom nappe of the flow and initiate the circulating cells of flow required to dissipate energy. (This alternative is not pictured in the following figures.)

4.2.13 Weirs, Chutes, and Pools

This design would transition the supercritical flow exiting the I&T sluiceway to subcritical flow through a hydraulic jump using a weir or end sill structure to form a ‘pool’. The flow would exit the pool by passing over the weir or end sill structure where it would enter a ‘chute’. The flow would become supercritical in the chute and be transitioned into subcritical at the next pool. The process would continue until the tailwater was reached. (This structure is not pictured in following figures.)

4.3 Plunge Pool Requirement

Several of these outfall types would likely require some type of plunge pool to dissipate and re-direct the residual vertical component of the outfall jet. This action is required to prevent jet impingement on the river bottom and possible bottom strike injury of fish. Since the design and costs associated with the construction of a plunge pool are expected to be significant, it was important to identify this requirement early. The plunge pool requirement has been indicated in this list of the initial outfall type alternatives.

1) Classic Ogee (with plunge pool)
2) Ogee with Deflector
3) Ramp (with plunge pool)
4) Stepped Ogee
5) Classic Cantilever (with plunge pool)
6) Multiple Cantilever (with plunge pool for lower outlet)
7) Cantilever with Flip Gate
8) Cantilever with Flow Spreader (Spoon)
9) Vertical Transition Chute
10) Continual Skim
11) Submerged
12) Supercritical Stepped Ramp
13) Weirs, Chutes, and Pools

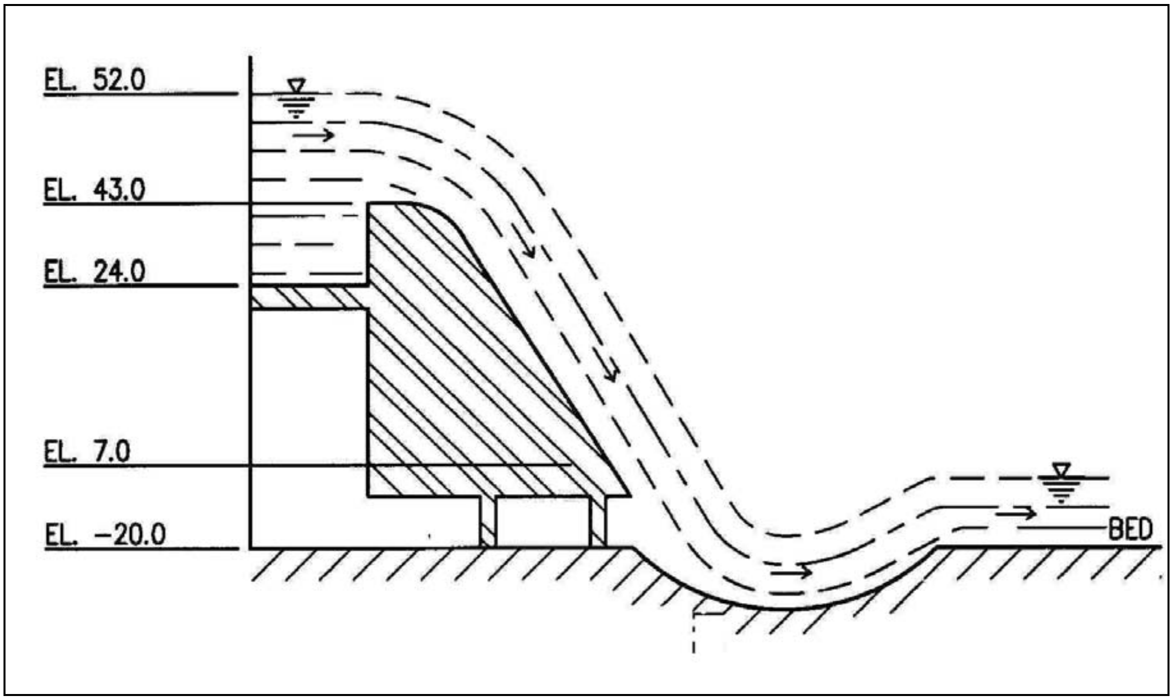

Figure 4-2 Classic Ogee

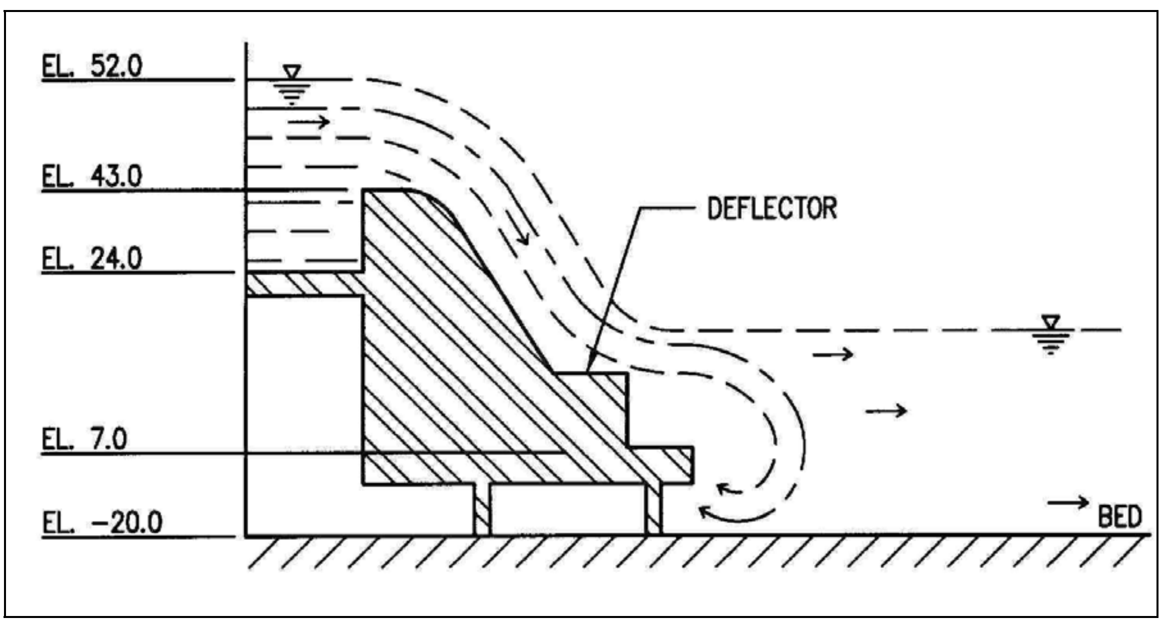

Figure 4-3 Ogee with Deflector

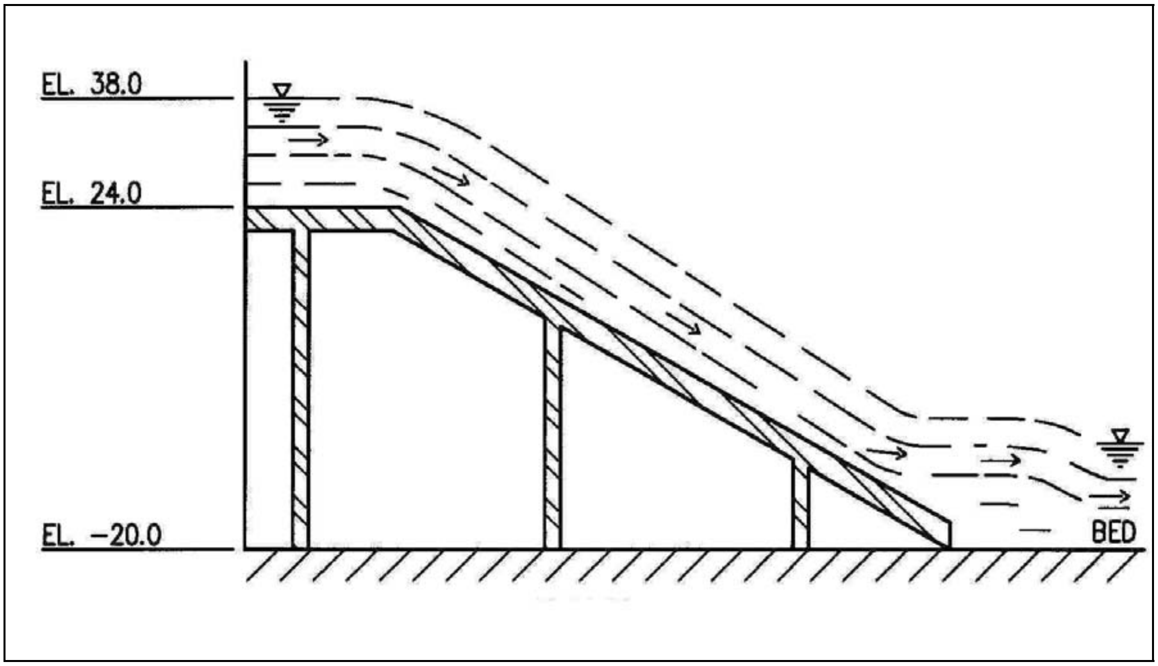

Figure 4-4 Ramp

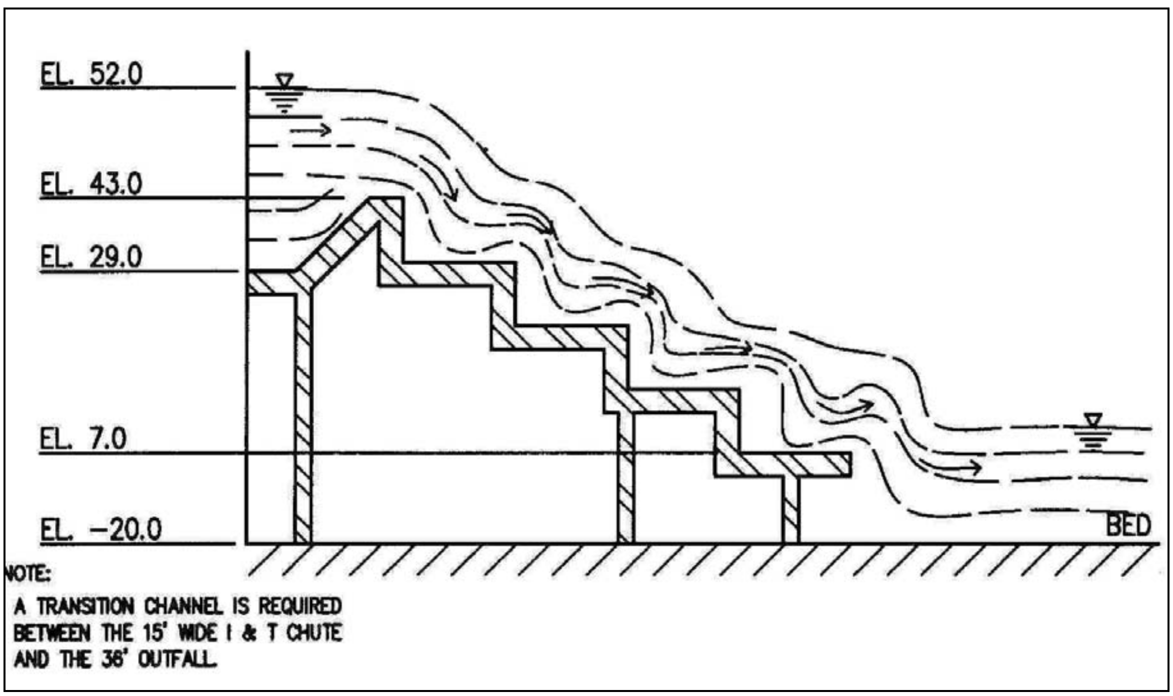

Figure 4-5 Stepped Ogee

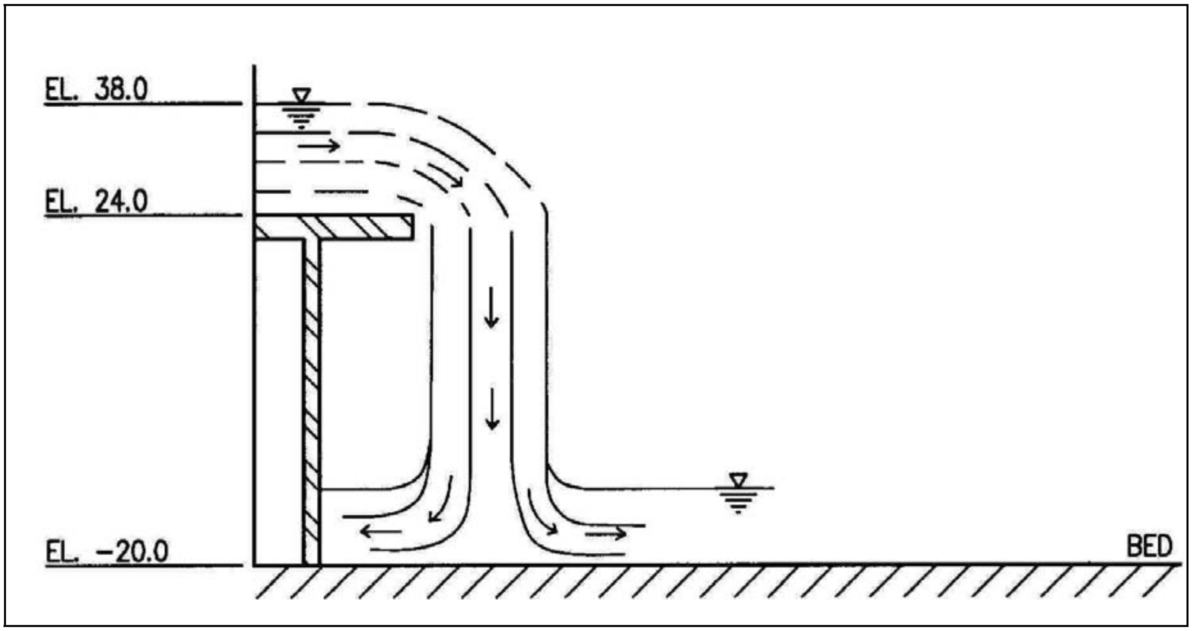

Figure 4-6   Classic Cantilever

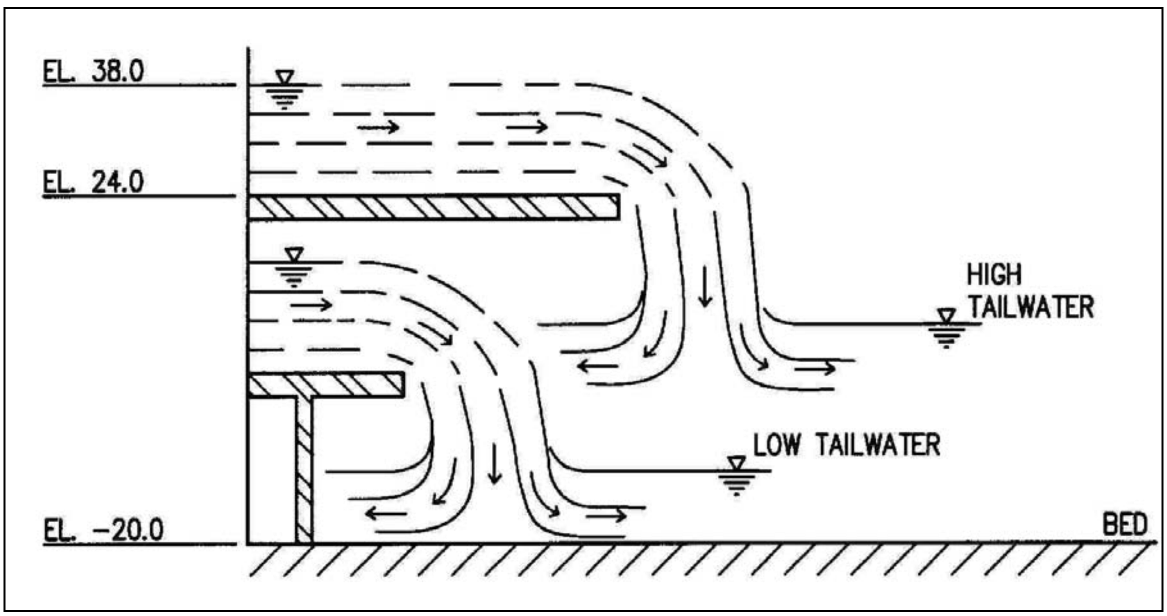

Figure 4-7   Multiple Cantilever

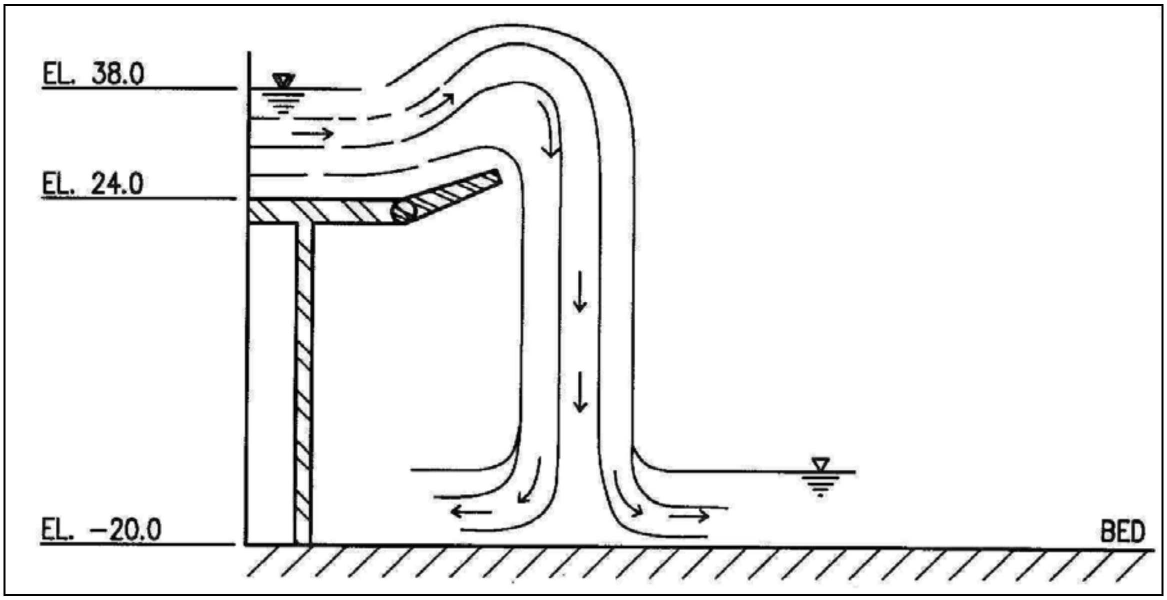

Figure 4-8 Cantilever with Flip Gate

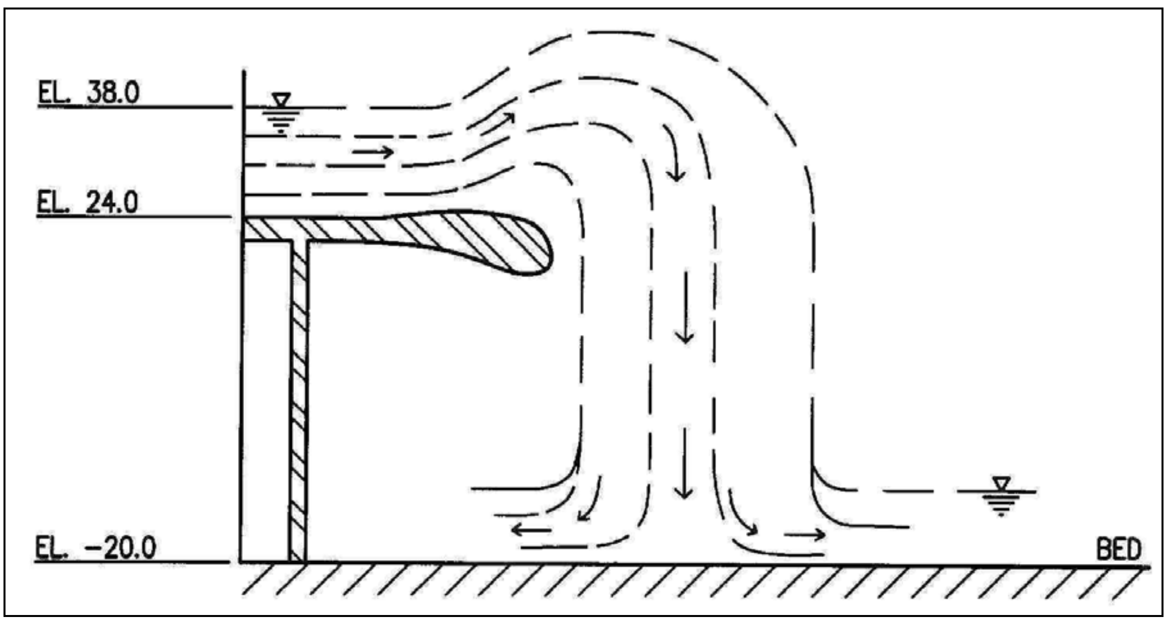

Figure 4-9 Cantilever with Spoon

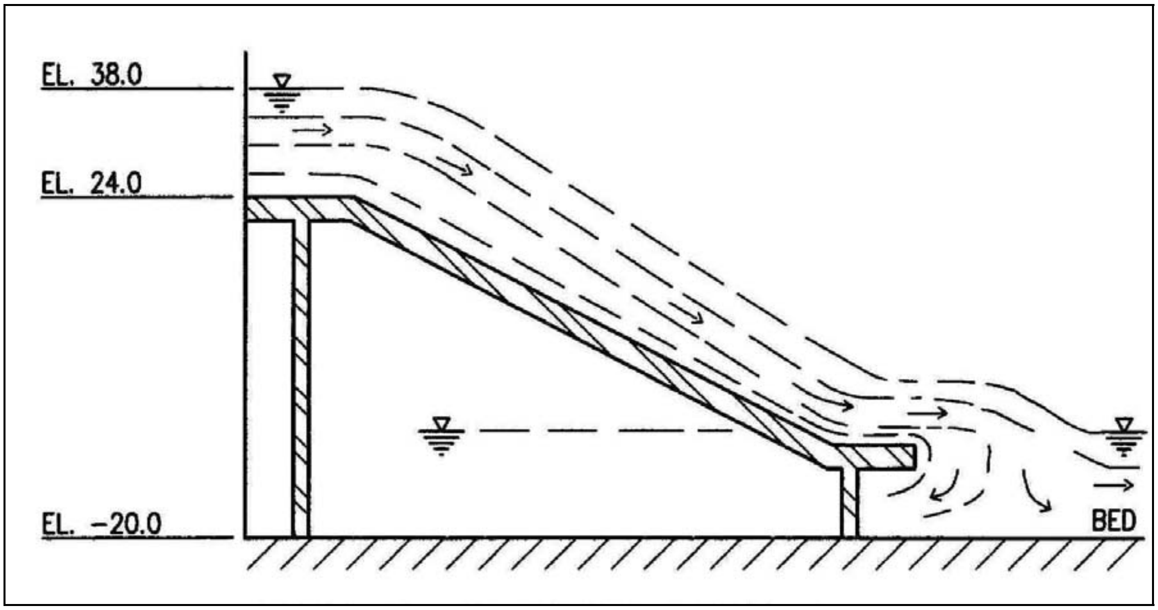

Figure 4-10 Vertical Transition Chute (VTC)

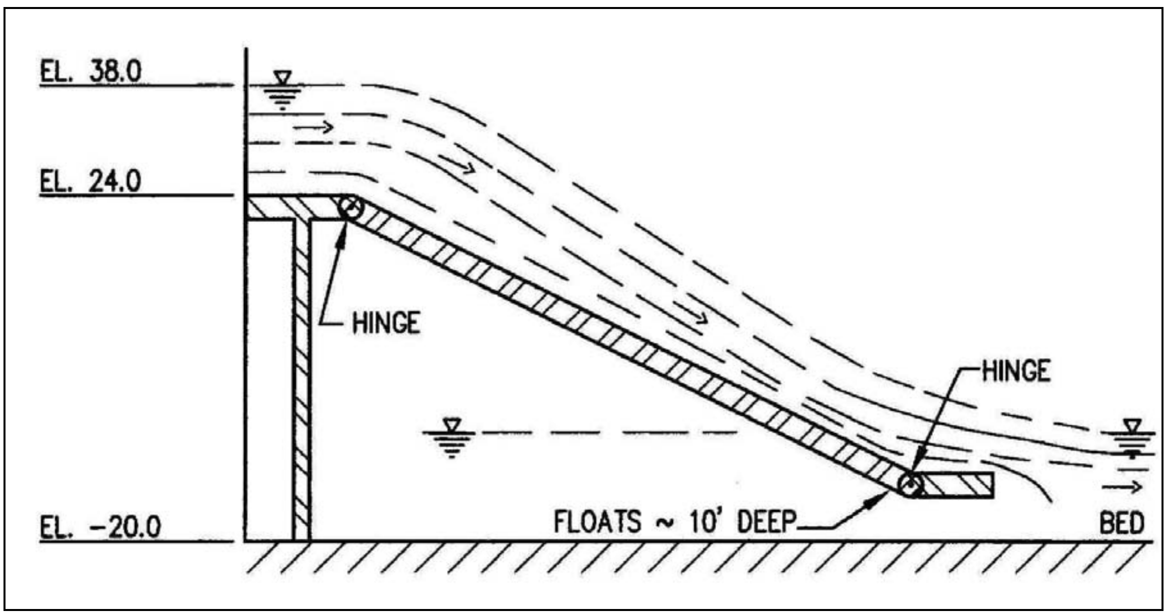

Figure 4-11 Continual Skim

4.4 Screening

Seven of the outfall types were eliminated in an initial screening for reasons described below.

4.4.1 Supercritical Stepped Ramp

With this type of outfall, the trajectory of supercritical flow will be very flat and step heights would necessarily be very small to provide contact with the bottom nappe of the flow and to initiate the circulating cells of flow required to dissipate energy. The result would be similar to a corrugated chute. Energy dissipation rates would be low enough that a considerable length of chute, approaching the length of the entire conveyance channel for the closer ranges, would be required to dissipate appreciable energy. There are no precedents for using corrugated flumes to carry fish at conveyance velocities of the magnitude (24 – 33 fps) being considered for B2. This alternative was not considered hydraulically feasible.

4.4.2 Weirs, Chutes and Pools

The structure would resemble a large fish ladder. This option was presented to regional agencies during the B1 HiQ Dewatering and Outfall Study. They considered the concept to be 'experimental' in nature and unsatisfactory in meeting their design criteria for juvenile fish conveyance.

4.4.3 Ramp with Plunge Pool

The Ramp is equivalent to the Vertical Transition Chute (VTC) for tailwater elevations above minimum tailwater level. Therefore, the advantages of this outfall type are retained in the VTC and its inclusion is redundant.

4.4.4 Stepped Ogee

This type of outfall has never been used for juvenile fish passage and has drawn negative reaction from the region when mentioned. It will require formation of a hydraulic jump in the conveyance channel upstream from the ogee crest, which violates the regional fish conveyance criteria. In addition, the tumbling turbulent energy dissipation action, which would occur on the stepped ogee, would spread the energy dissipation over a finite distance in the structure, reducing the jet energy entering the river. The large scale of the turbulent eddies, which will form on each step may not pose and injury potential to small juvenile fish but is almost certain to be injurious to adult fish, which fall back through the bypass system. With the combination of these disadvantages, this type was not considered viable.

4.4.5 Cantilever with Flip Gate

This outfall type is intended to project flow some distance from the outfall terminus into a deep pool, where bottom impingement of the jet and strike of juvenile fish is not a possibility, e.g. the new I&T chute outfall at The Dalles Dam. There are no deep receiving water locations in the Bonneville tailrace areas, so this design type is not applicable.

4.4.6 Cantilever with Spoon

This outfall type is intended to reduce the jet penetration into the tailwater and prevent bottom impingement of the jet and juvenile fish strike on the bottom by spreading the jet into a relatively thin sheet, which maintains little residual momentum after it contacts the water surface. While this has merit, the concept is totally at odds with the basic premise of the high flow outfall. Namely, that carrying the juvenile fish in a large volume of flow with a large cross sectional area will reduce their risk of exposure to the high levels of energy dissipation, deceleration, and shear, which occur at the jet boundaries as it enters the tailwater. If the spoon's goal of spreading the jet into a thin sheet is carried to its extreme case, all fish in the jet cross section will be exposed to this initial high level of boundary shear and energy dissipation which occurs at the jet entry point into the tailwater.

4.4.7 Submerged

A submerged outfall would require transition from supercritical flow in the conveyance channel to subcritical flow through a hydraulic jump, some type of headwall or downwell structure to drive flow into a closed conduit, and fish passage in pressurized closed conduit flow to the submerged outfall location. All of these actions are at odds with regional fish passage criteria and their combination led to elimination of this type from further consideration.

4.5 Primary Evaluation

The next step in the Stage 1 process was a primary evaluation by the INCA design team of the remaining six outfall types using evaluation parameters and scoring of the types against these parameters.

4.5.1 Evaluation Parameters and Scoring

The outfall design evaluation parameters are based on relevant preliminary guidelines for HiQ outfalls (see pages 32-33 in Johnson et al. 1999) and other parameters important to design of a successful HiQ outfall at B2. Johnson et al. (1999) organized the preliminary guidelines into two functional areas, outfall location and outfall design. Although this section deals with evaluating outfall designs, preliminary guidelines for locating outfalls were also included because they addressed features that could be affected by the outfall design. That is, outfall location and design are inter-related. Besides the preliminary guidelines, other parameters are included to thoroughly evaluate the site-specific B2 HiQ outfall design alternatives.

For the purposes of this evaluation of outfall design alternatives, three scoring categories were established: 1 = good; 2 = neutral; and 3 = bad. Scores are relative to the other alternatives. Scores were not assigned in an absolute context.

There is a high degree of uncertainty expected performance for most of the outfall types. This uncertainty can only be eliminated or reduced with additional hydraulic modeling and biological research. At this stage in the evaluation process, lacking definitive modeling and research, it was decided to utilize scoring ranges in the matrix to account for uncertainty about relative scores.

Evaluation Parameter No. 1 -- Improves ambient water velocity at low river flows.

<u>Explanation</u> — This modification of Preliminary Guideline No. 1 is based on hydraulic entrainment of ambient flow with outfall flow when the high flow jet enters the receiving water. Slow ambient flow may increase in velocity as it is entrained with the outfall jet, depending on jet trajectory, entry velocity, and ambient velocity.

<u>Scoring</u> — Outfall types that direct the flow horizontally as it enters the tailrace, VTC and Continual Skim, were rated as good, though this is somewhat subjective, as the VTC will maintain some vertical component above low tailwater level. The Multiple Cantilever was rated neutral because the outlet will have a skimming effect at certain tailwater levels. The Ogee with Deflector was also rated neutral, because it will only direct flow horizontally at the low end of the tailwater level range. The Classic Cantilever will have a horizontal component under some tailwater elevations and received a score of 2-3. The Classic Ogee received a score of 3 because it results in a largely vertical jet trajectory at entry to the tailwater in a majority of tailwater elevations.

Evaluation Parameter No. 2-- Minimizes bottom strike.

<u>Explanation</u> — This parameter is based on Preliminary Guideline No. 2. Bottom strike may not always be eliminated, but the design that minimizes it is preferred because such a design will minimize the probability of fish injury.

<u>Scoring</u> — Scores for this parameter reflect the degree of difficulty involved in preventing bottom strike with the outfall type. It will be more difficult to prevent bottom strike with a steep, almost vertical jet entry and relatively shallow receiving water depth. Outfalls with these characteristics will require a substantial plunge pool and will require additional modeling efforts to optimize the geometry of this plunge pool. There is also an inherent

uncertainty associated with this deep plunge pool with regards to predator habitat, constructability, and operations and maintenance. If a plunge pool is constructed, then these outfalls could have little or no bottom strike. Thus the Classic Ogee and Classic Cantilever were awarded a score of 1-3, reflecting the necessity of a large deep plunge pool to avoid or minimize bottom strike. A score of 1-2 was awarded to the Multiple Cantilever and Ogee with Deflector, because a smaller plunge pool will be required due to reduced jet fall distance to the tailwater. The VTC and Continual Skim designs were awarded 1's because their designs inherently avoid bottom strike.

Evaluation Parameter No. 3 -- Minimizes in-water structures.

Explanation - As stated in Preliminary Guideline No. 4, underwater structures may provide staging habitat for predators. Thus, these structures should be minimized.

Scoring - Both of the Ogee types will require substantial in-water structures and were awarded a score of 3. The VTC will also require in-water structure, but due to its flow egress action, will provide less shadow in the ambient flow. Both it and the Continual Skim, which will require floats to track water level, received a score of 2-3. Both of the Cantilever types will only require drilled piles for support, which will provide minimal shadow effect. However, the Multiple Cantilever type will require twice as many drilled shaft supports and the lower level outlet will be inundated, creating a shadow in the ambient flow, at higher tailwater levels, so it was also awarded a score of 1-2, while the Classic Cantilever was scored a 1.

Evaluation Parameter No. 4 -- Minimizes entry velocity.

Explanation - Preliminary Guideline No. 8 prescribes a specific, working entry velocity parameter (< 50 fps). The purpose of this outfall design evaluation is to compare designs relative to one another. Thus, in this context, the design that conceptually has the lowest entry velocity will be desirable.

Scoring - It is assumed that all of the outfall types will be designed to meet the 50 fps jet entry velocity preliminary guideline, so all rated a score of 1.

Evaluation Parameter No. 5 -- No increase in total dissolved gas (TDG)

Explanation - This parameter is based directly on Preliminary Guideline No. 9. Outfall designs that create conditions conducive to raising TDG levels are not wanted.

Scoring – All outfall types will be designed to minimize TDG. Outfall types that will result in deep plunging of the jet into the tailwater, the Classic Ogee and Classic Cantilever, will carry entrained air deep increasing the TDG potential. Since the amount of TDG produced by a HiQ outfall in relation to the TDG from the spillway will be minimal, these two types were scored a 1-2 (rather than the more severe score of 3). All other outfall types received a score of 1.

Evaluation Parameter No. 6-- Minimizes eddies and back rollers.

Explanation - Preliminary Guidelines No. 3 and No. 7 address eddies and back-rollers in the location and design phases, respectively. The idea is to minimize predator staging areas near the outfall and reduce the probability of predation.

Scoring - A plunging type jet will generate a back roller downstream from and above the jet entry location, as well as a horseshoe-shaped vortex or eddy that wraps around the upstream side of the jet below its entry location and extends to either side downstream. Therefore outfall types where the jet plunges steeply, Classic Ogee and Classic Cantilever, received a score of 3. The VTC, with significant structure in the water at higher tailwater, will create some eddies and was rated as a 2-3. The Multiple Cantilever will create less intense eddies due to its lower fall distance to tailwater. It and the Ogee with Deflector, which will create some back eddy above and lateral eddy action at the jet sides, received scores of 2. The Continual Skim type will confine its eddy generation to lateral eddies adjacent to the jet and received the best score.

Evaluation Parameter No. 7—No adverse impacts to adult salmonids.

Explanation – Impact to adults must be avoided, as stated in Preliminary Guidelines No.5 and No. 10 for outfall location and design, respectively.

Scoring – None of these outfall types are expected to be more injurious or cause more adult migration path obstacles, so all received a score of 1.

Evaluation Parameter No. 8—Minimizes energy dissipation rate and shear.

Explanation – Outfall designs can differ in energy dissipation rate and shear, even though jet entry velocities may be similar in magnitude. Since these features may be positively correlated with fish injury rates, the minimizing of energy dissipation rates and shear is advisable.

Scoring – Both the Classic Ogee and Cantilever types are expected to concentrate energy dissipation within a short distance downstream from their jet entry locations because the jets plunge almost vertically and experience shear forces around their entire periphery, although this will be minimized somewhat for the Classic Cantilever at higher tailwaters. The Classic Ogee was therefore rated as a 3 and the Classic Cantilever was rated as 2-3. The Multiple Cantilever, while having the same plunging action, reduces the fall distance, and therefore the entry trajectory is flatter, and some residual energy will be directed downstream, reducing the energy dissipation rate. It receives a score of 2, as does the Ogee with Deflector, which will also direct flow more horizontally. Both the VTC and Continual Skim types will reduce the perimeter of the flow cross section exposed to the initial shear on contact with the tailwater. The Skim type will only expose the sides and bottom of the jet at all tailwater levels. The VTC will act similarly at low tailwater. Both were given a score of 1. Preliminary review of data from the 1:30 model studies indicate that the maximum deceleration, shear and jet energy dissipation rates for the VTC and Continual Skimming types are quite similar. However, these data require further review before they may be presented and definitive conclusions drawn.

Evaluation Parameter No. 9—Accommodates a wide range of tailwater elevation.

Explanation – Tailwater elevation at Bonneville Dam can range widely depending mostly on river discharge, but also on tidal stage. Thus, it is important that the B2 outfall be functional for a wide range of tailwater elevation.

Scoring – The Classic Ogee, Classic Cantilever and Ogee with Deflector types, with their steeply plunging jet trajectories, result in increasingly severe jet entry conditions and potential for bottom impingement as tailwater level falls. An effective plunge pool may however mitigate this somewhat. Thus, they received scores of 1-3. (For the Ogee with Deflector, it is virtually impossible to design a deflector to deliver the desired skimming flow action over a large range of tailwater level.) The Multiple Cantilever splits the tailwater range into two operating ranges and is more adaptable, receiving a score of 1-2. The designs of both the VTC and Skim types accommodate similar performance over a wide range of tailwater levels and received scores of 1.

Evaluation Parameter No. 10 - Minimizes adverse impacts to fish within the structure.

Explanation - Some outfall designs may necessitate certain features within the conveyance structure, i.e., upstream of the outfall point. For example, a submerged outfall means there must be a hydraulic jump somewhere upstream of the outfall exit. Generally, hydraulic jumps or other features of the outfall structure that have potential to increase probability of fish injury should be avoided.

Scoring - Both of the Ogee types require a hydraulic jump in the conveyance channel upstream to establish the proper subcritical approach flow to the crest for them to function properly. This increases the potential for injury in the structure and they accordingly received a score of 3. The Continual Skim requires movable joints in the invert and walls of the outfall to allow its profile to adjust as tailwater level varies. The design of joints to prevent fish impingement or injury must be considered, so this design only received a rating of 2. The remaining designs appear to be benign to fish passing through them and received a score of 1.

Evaluation Parameter No. 11 — Cost.

Explanation - The cost parameter encompasses issues such as constructability, distance from the dam, underwater structures, and other factors. Low cost outfall designs are preferred.

Scoring - A number of factors affect the cost of construction of the various outfall types. Outfall types, which will probably require mass construction at elevations below the tailrace, would likely require dewatering for their construction. This is likely to be costly relative to those outfall types such as the Classic Cantilever. On this basis, the following alternatives were rated 3: Classic Ogee, Ogee with Deflector, and VTC.

The Classic Cantilever has very low construction cost, since it is just the end of the channel. However, to avoid plunging flow from striking the bottom, significant amounts of dredging will be required to create a plunge pool. Due to this factor, this alternative was rated a 2-3. The Multiple Cantilever may require a large gate structure to switch from one level outfall to another, or a significant operational commitment if the switching is not automated. Due to this additional cost it was also rated 2-3.

The Continual Skim outfall could have a relatively low initial construction cost, but was give a 3 rating since the operational and maintenance costs could be significant.

In short, all of the outfall types are expected to be expensive and/or have major construction or O&M issues associated with them.

The following Table 4-1 presents the results of this evaluation.

Table 4-1 Evaluation Matrix for High Flow Outfall Types

Parameter No.	Parameter Description	Classic Ogee (plunge pool)	Ogee with Deflector	Classic Cantilever (plunge pool)	Multiple Cantilever (single plunge pool)	Vertical Transition Chute (VTC)	Continual Skim
1	Ambient Water Velocity	3	2	2-3	2	1	1
2	Bottom Strike	1-3	1-2	1-3	1-2	1	1
3	In-water Structure (Predator Habitat)	3	3	1	2	2-3	2-3
4	Entry Velocity	1	1	1	1	1	1
5	TDG	1-2	1	1-2	1	1	1
6	Eddies	3	2	3	2	2-3	1-2
7	Impact to Adults	1	1	1	1	1	1
8	Energy Dissipation	3	2	2-3	2	1	1
9	TW range	1-3	1-3	1-3	1-2	1	1
10	Adverse in-structure impacts	3	3	1	1	1	2
11	Cost	3	3	2-3	2-3	3	3
	Total	**23-28**	**20-23**	**16-24**	**16-19**	**15-17**	**15-17**

4.5.2 Primary Evaluation Recommendations

Three observations can be made concerning the B2 tailrace area and evaluating outfall types for this area:

- The Bonneville tailrace elevation varies considerably (from El. 6.9 to El. 36.9). Some outfall types accommodate wide variations in tailwater elevation better than others.

- The Bonneville tailrace is relatively shallow at low tailwater (approximately 27 feet). Coupled with the requirement that an outfall type must also function in a high tailwater, this shallow tailrace makes some types of outfalls (with a large downward plunge component) potentially not as effective.

- There is a large degree of uncertainty associated with evaluating outfall types. This uncertainty has been recognized and hydraulic modeling and biological research is currently underway to eliminate some of this uncertainty. Until those results are available, premature elimination of outfall types carries some risk.

Based upon the Evaluation Matrix presented in Table 4-1, four types of outfalls were carried forward for conceptual design and evaluation in the next step of this B2 Site Selection Study. These four outfall types are:

1. Classic Cantilever
2. Multiple Cantilever
3. Vertical Transition Chute
4. Continual Skim

4.6 Consolidation of Outfall Types

Following the primary evaluation, the design process continued and outfall types were further assessed. As a result of this assessment, the four outfall alternatives were refined so that they:

- better accommodated site constraints of both B2 ranges still under consideration (Ranges D and F),

- maintained the desired performance characteristics of all of the outfall types, and

- eliminated the undesirable characteristics of some types.

These refined outfall types were the result of preliminary calculations of flow depths and velocities in the conveyance channel to the outfall range locations, preliminary model test data from a 1:30 scale physical outfall model study performed at ENSR, continued analysis of the types, and additional evaluation including conceptual costs. The following list presents the revised recommended outfall types that were considered for further study. The Classic Cantilever was retained, the Continual Skimming was revised to the Adjustable Cantilever, and the VTC was revised to the Mid-level Cantilever. The beneficial effects of the Multiple Cantilever may be achieved with the Adjustable Cantilever with less structure. Therefore, the Multiple Cantilever was deleted from further consideration.

4.6.1 Preliminary Conveyance Calculations

To determine the invert elevations that would be appropriate for the various outfall types at the ranges, preliminary conveyance calculations were performed. The calculations used Manning's equation to compute the normal depth of flow in the conveyance channel, assuming a constant sloped concrete rectangular cross section channel (n = 0.012) from the I&T invert elevation of 29.0 feet to the invert of the outfall. An outfall discharge of 5,300 cfs was used. The following guidelines were used to determine the acceptable ranges of invert elevations:

- Froude number in the channel cannot be less than 1.2. This was intended to prevent the water surface from becoming unstable as it approaches critical depth.

- Outfall entry velocity cannot exceed 50 fps at tailwater El. 7.0 feet. This was intended to provide a conservative level of compliance to preliminary outfall design Guideline 8, since the minimum tailwater of record, 7.0 feet, rather than the lower end of the design range, 8.5 feet, was used.

- Water level in the channel at the outfall structure must be high enough to prevent formation of a hydraulic jump in the outfall conveyance channel at high tailwater levels.

- Exit velocity, based on calculated channel velocity, must be greater than 20 fps to prevent adult fish from entering at high tailwater level.

In these analyses, the outfall exit velocity was assumed to be the average normal velocity in the channel and the jet entry velocity was calculated applying the trajectory equation to the mid-depth streamline and average velocity at the outfall exit. The potential for a hydraulic jump to form in the channel was determined by calculating the sequent depth in the channel required to force a jump, assuming minimal slope of the channel. This was converted into a tailwater elevation by adding the depth to the invert elevation. This calculation is conservative because it assumes no energy loss as the channel flow enters the tailwater.

Table 4-2 shows the elevations that satisfy the criteria for each of the ranges. At Range D, an invert elevation less than 27 feet is required to maintain adequate Froude numbers in the channel. For invert elevations less than 26 feet, the jet entry velocity will exceed the 50 fps guideline. An invert elevation of 26 feet was chosen for the Classic Cantilever analyses at Range D.

For Range F, the invert of the outfall must be lower than about El. 17.5 to maintain adequate Froude numbers in the channel. However, for inverts lower than this, the potential for a hydraulic jump to form in the channel at high tailwater levels becomes a factor. Based on the sequent depth calculation, a jump may form in the end of the outfall channel if tailwater levels near El. 35 feet for outfall invert El. 17 or so feet. At Range F adequate energy will be dissipated in the long channel that having jet entry velocities exceed the guideline is not a concern. For Range F, an outfall invert at El. 16 feet was chosen for the analyses.

The flow depths and velocities at the outfall exit supplied in this table were used in the subsequent model analyses unless otherwise noted.

Table 4-2 Preliminary Conveyance Channel Calculations

B2 Outfall Range	Conveyance Channel Length (ft)	Outfall Invert (ft)	Slope	Normal Depth (ft)	Channel Exit Velocity (fps)	Froude No.	Outfall Water Surface Elevation (ft)	Jet Entry Velocity @ Minimum Tailwater Elevation 7 ft (fps)	Tailwater Level for Hydraulic Jump in Channel (ft)
D	400	27	0.0050	14.0	25.3	1.19	41.0	48.8	44.6
D	400	26.5	0.0063	12.8	27.6	1.36	39.3	49.3	45.6
D	400	26	0.0075	11.9	29.7	1.52	37.9	49.9	46.3
D	400	25.5	0.0088	11.2	31.6	1.67	36.7	50.5	46.8
F	2300	17.5	0.0050	14.0	25.3	1.19	31.5	42.0	35.1
F	2300	17	0.0052	13.7	25.7	1.22	30.7	41.8	34.9
F	2300	16.5	0.0054	13.5	26.1	1.25	30.0	41.6	34.6
F	2300	16	0.0057	13.3	26.6	1.28	29.3	41.4	34.4
F	2300	15.5	0.0059	13.1	27.0	1.31	28.6	41.2	34.1

4.6.2 1:30 Scale Outfall Modeling

The preliminary modeling of the outfall types on the 1:30 scale model at ENSR focused on comparisons of the Classic Cantilever, Skimming, and Vertical Transition Chute outfall types. Details of these studies are provided in ENSR and INCA (June 19, 2000)[11]. The following sections summarize the methods used in the 1:30 scale modeling as well as the preliminary results.

4.6.2.1 Methods

4.6.2.1.1 Model Description

A 1:30 physical hydraulic model of the outfall structures was prepared at ENSR's Redmond, Washington laboratory. The model, shown in Figure 4-12, incorporated the following features:

- Flat river bed – The flat bed simulated El. –20 feet. It was installed in such a manner that it could be easily altered to simulate bathymetry of specific outfall sites and development of plunge pools below El. –20 feet.

- Variable River in-flow – A system of weirs and vanes allowed adjustment of the ambient river flow field approaching the outfall location to simulate specific outfall sites.

- Adjustable tailwater weir – This weir allow simulation of tailrace water levels from the minimum to maximum of record (El. 7 to 35 feet)

- Outfall Flow Supply – A metered flow supply allowing simulation of outfall flows up to approximately 10,000 cfs. It also had a gate allowing the desired depth (and velocity) of flow to be set in the conveyance channel approaching the outfall structure.

- Variable outfall structures – Alternative outfall structures could easily be installed and tested.

[11] ENSR and INCA Engineers, Inc. June 19, 2000. Bonneville Second Powerhouse Corner Collector Outfall 1:30 Scale Physical Hydraulic Model 60 % Submittal. Prepared for USACE Portland District. ENSR Document No. 3697-002-400

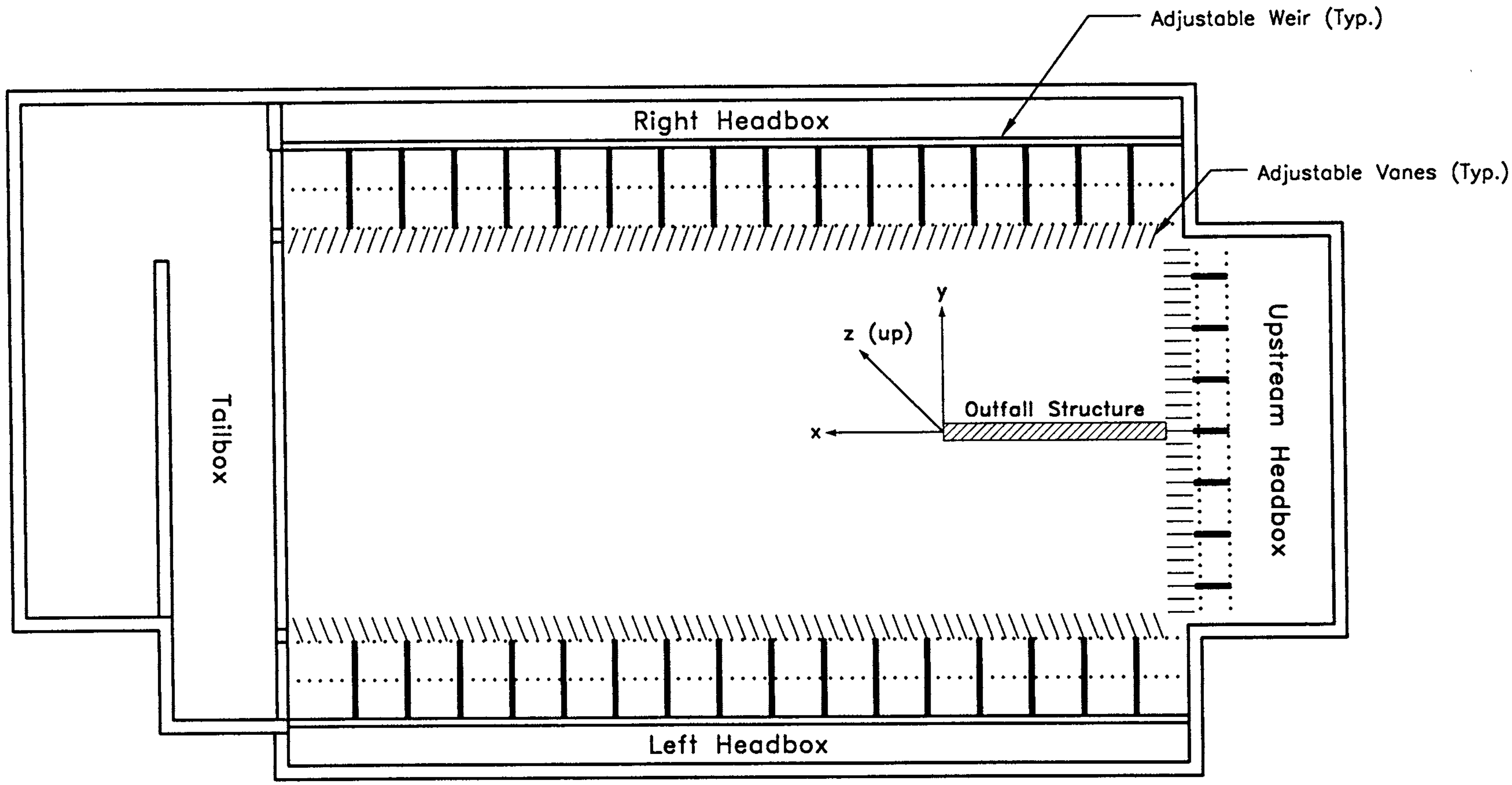

Figure 4-12 1:30 Model Plan View

4.6.2.1.2 Model Operations

Prior to outfall simulations, the approach flow to the outfall location was established by first acquiring velocity data for the particular site from the 1:100 general Bonneville Project model at ERDC and then adjusting the weirs and guide vanes on the 1:30 model to deliver the same approach flow field. For the testing described here, conditions were set to develop a generic ambient flow field of approximately 4 fps magnitude, using data acquired on the ERDC model at Range E.

4.6.2.1.3 Data Acquisition and Reduction

Velocity data were acquired on five cross-sections along the trajectory of the jet after it entered the tailwater from the outfall using a three-dimensional Sontek acoustic Doppler velocimeter (ADV). These data were used to characterize the energy dissipation and flow deceleration characteristics of the jet for comparison purposes. Qualitative observations of the performance were also made and recorded photographically. An example velocity data plot for the Classic cantilever outfall is presented in Figure 4-13.

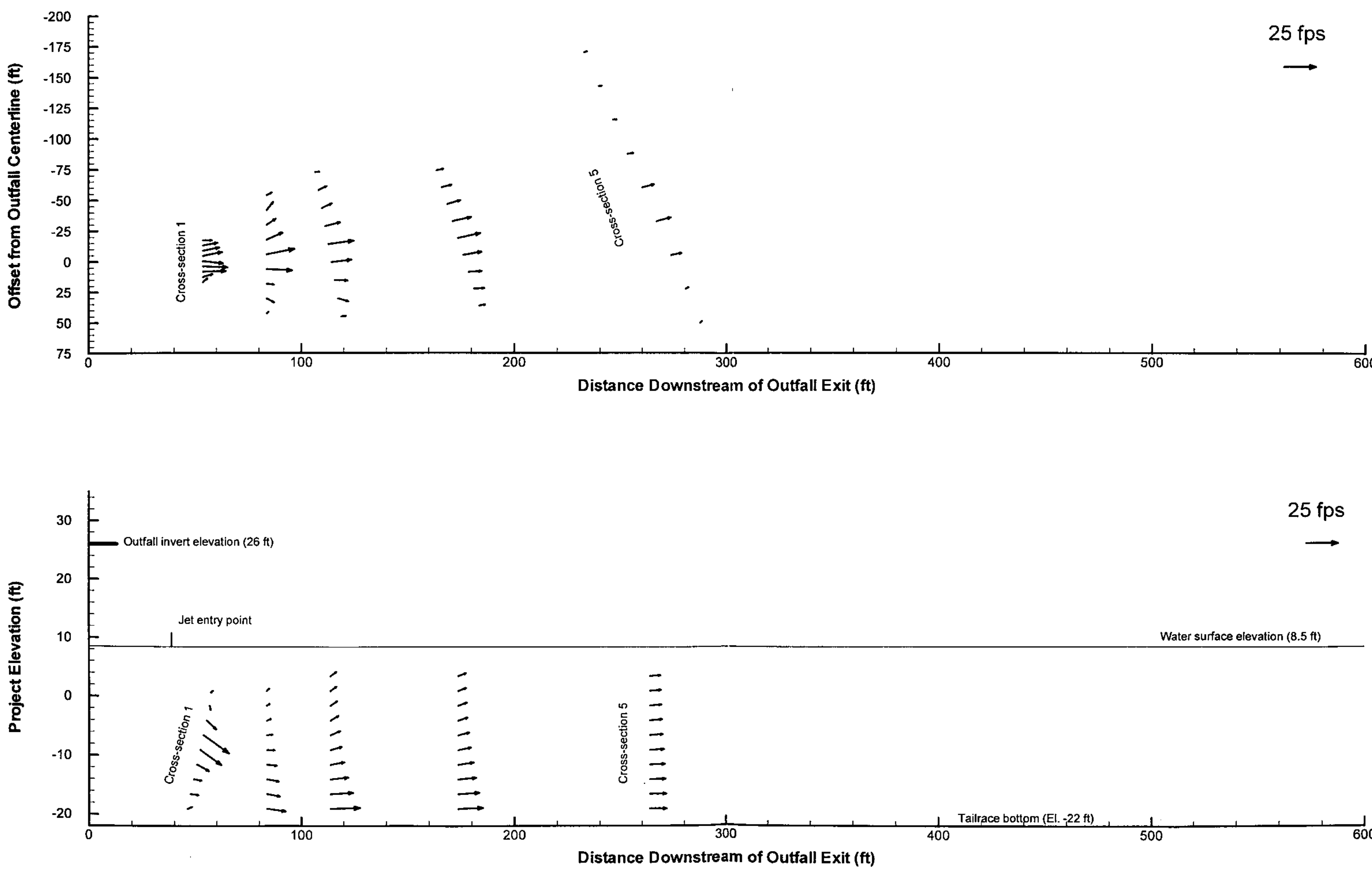

Figure 4-13 Jet Velocities for Classic without Plunge Pool

4.6.2.2 Comparison of Outfall Type Performance

Table 4-3 summarizes the results of the preliminary outfall tests in the 1:30 model. Tests were performed with the VTC approach channel at invert El. 26 feet and the outlet invert at El. 7.0 feet; the Skimming outfall approach channel at invert El. 19.25 feet and the outlet at El. 0.25 feet and the Classic cantilever outfall at invert El. 26.0 feet.

Table 4-3 Preliminary Outfall Type 1:30 Model Results

Test	Outfall Structure	Tailwater Elevation	Significant Jet Characteristics
1	VTC	8.5	Plunging jet. Third lowest energy dissipation rate based on jet profile. Large quantity of entrained air, although not as much as Tests 5 and 6.
2	VTC	30	Hydraulic jump in outfall. Fifth lowest energy dissipation rate based on discernable jet profile downstream.
3	VTC	14	Surface oriented jet. Most discernable jet downstream compared to Tests 1 and 2. Second lowest energy dissipation rate of all tests, based on downstream jet profile.
4	Skimming	8.5	Very surface-oriented jet. Most discernable jet downstream of all tests, indicating lowest energy dissipation rates.
5	Cantilever without plunge pool	8.5	Plunging jet with bottom strike. Large amount of air entrained with jet. Fourth lowest energy dissipation rate based on jet characteristics.
6	Cantilever with infinite plunge pool	8.5	Plunging jet. Large amount of entrained air that rises to the surface further upstream than for Test 5. Highest energy dissipation rate based on jet downstream characteristics.

4.6.3 Classic Cantilever

The Classic Cantilever represents the outfall type traditionally installed on fish bypass systems in the region. The invert of the outfall is placed above the tailwater level through most of the operating range. At Range D, this can be achieved with an invert elevation of 26-27 feet. At Range F, it will be necessary to place the maximum invert elevation of the outfall at about 16 feet in order to maintain supercritical flow with a Froude Number above 1.2. (This Froude Number is required for a stable water surface and to maintain high enough velocities at the end of the channel to prevent adult entry during high tailwater conditions.) At Range F, maintaining the outfall invert high enough so that it is above tailwater through most of the operating range is not possible. Therefore the Classic Cantilever is only applicable at Range D.

The classic cantilever outfall will require a plunge pool to prevent the jet from striking the river bottom.

4.6.4 Mid-level Cantilever

A Mid-level Cantilever would be placed at an invert elevation that would be below tailwater level over a large part of the operating range. The invert elevation would be selected to maintain adequate Froude numbers in the channel for stable flow and to prevent both hydraulic jump formation in the channel and to prevent adult entry at high tailwater levels. The advantage of the Mid-level Cantilever is that the jet entry angle at low tailwater levels will be flatter, the resulting potential for bottom impact less, and the need for a plunge pool to prevent bottom strike reduced. A Mid-level Cantilever of about invert El. 16-17 feet would be applicable at both Ranges D and F. However, at Range D the jet entry velocities would exceed the 50 fps guideline.

4.6.5 Adjustable Cantilever

An Adjustable Cantilever would provide the operational advantages of the Continual Skimming outfall while its invert elevation could be matched as closely as required to the tailwater level. This would allow a flatter jet entry angle at most tailwater elevations. The end of the cantilever would be suspended from an overhead frame founded on a pair of drilled shafts about 30 feet from the end of the outfall. The adjustment would be achieved using a hydraulic cylinder. The large floats required for the previous concept of a Continual Skimming outfall would create large shadows for predator holding, whereas the drilled shafts of the Adjustable Cantilever would not. The design and construction of an Adjustable Cantilever would also be less complex than the Continual Skimming outfall. The chute profile of the Adjustable Cantilever could be developed to provide a similar entry trajectory to the VTC at low tailwater levels over the entire tailwater range. However the adjustment feature would prevent formation of a quasi-hydraulic jump within the walls of the structure at mid- to high-tailwater levels, a potential problem with the VTC that was observed during testing on the 1:30 model. This action is illustrated in Photograph 4-1 of a quasi-hydraulic jump in the VTC outfall on the 1:30 model. The Adjustable Cantilever also has advantages over a two level outfall (Multiple Cantilever), such as the elimination of: 1) a plunge pool requirement, 2) two side by side conveyance channels, and 3) a 'switch gate' to change flow between outfalls. The Adjustable Cantilever would be applicable at both Ranges D and F. However, at Range D at the lower invert elevation settings, the jet entry velocity would again exceed the 50 fps guideline.

Photo 4-1 Quasi-hydraulic jump in VTC outfall on 1:30 model

4.6.6 Elimination of Outfall Types from the Primary Evaluation

It appears reasonable to eliminate the Multiple Cantilever, VTC and Continual Skimming outfall types from further evaluation. Both the Multiple Cantilever and Continual Skimming outfall would require a longer construction time, and thus a longer time to achieve operation than the remaining alternatives. The outfall types are essentially equivalent in terms of jet entry velocity magnitude and the resulting shear forces and energy dissipation. Furthermore, any advantages these outfall types offer in terms of jet entry angle and the associated juvenile and construction benefits are duplicated by the Adjustable Cantilever. Although the VTC was originally thought to be a potentially viable alternative, recent modeling work on the ENSR 1:30 hydraulic model indicates that it has undesirable hydraulic characteristics at higher tailwater elevations, as well as potential predator shadow issues.

4.7 Outfall Type Selection

Selection from the remaining three outfall types, Classic Cantilever, Mid-Level Cantilever, and Adjustable, which should be carried forward to Stage 2 for final hydraulic model testing and evaluations in combination with the outfall ranges, required further analyses on the 1:30 scale outfall model. These were performed in preparation for and during a workshop at ENSR attended by members of the Design Team, District Staff, and the Regional Agencies and Tribes. Details of these studies are found in the ENSR trip report found in Appendix A.

Studies of the Adjustable Cantilever defined preliminary ranges of operating invert elevations of the cantilever versus project tailwater elevation that would result in acceptable levels of jet impact on the river bottom. The studies of the Classic and Mid-Level Cantilevers developed preliminary designs of plunge pools that would limit the bottom impact to acceptable levels.

4.7.1 General Methods

4.7.1.1 Model Configuration

The basic designs of the Classic and Mid-Level Cantilever outfalls were similar, as pictured for the Mid-Level type in Plate 6. The design for the Adjustable outfall type is also pictured in the same plate.

The general 1:30 model river bed was modified by removal of a section of the model floor and building a containing box, that was then filled with crushed rock with a d_{50}, which simulated movement of approximately 6 inch diameter prototype material. This provided a movable bed section that was used for plunge pool development. For tests of the adjustable cantilever, this movable bed was covered with a fixed bed of plywood sheathing at a constant elevation, similar to that used for the preliminary tests described in Section 4.6.2.

4.7.1.2 Test Conditions and Procedures

The ambient flow field, tailwater elevation, and outfall flow conditions were established using the same methods described in Section 4.6.2.

4.7.1.2.1 Bottom Impact Measurements

The pressures generated on the river bed by contact of the outfall jet were measured using pressure transducers. Adequate data were recorded to calculate a statistically significant average pressure. The pressures were corrected for depth of the transducer by recording data without the outfall operating and subtracting the average pressure for this condition. The resulting average pressure was assumed to be indicative of the velocity head of the component of the flow velocity vector near the bed that was perpendicular to the bed. Through this assumption the pressure data were converted to an average velocity component perpendicular to the bed. This technique was employed for recording and reporting bottom impact for tests both of the Adjustable Cantilever outfall on the flat river bed and of the Mid-Level and Classic Cantilever outfalls with the plunge pools.

4.7.2 Adjustable Cantilever Operating Ranges

Operating ranges for the Adjustable Cantilever outfall were developed by first establishing the outfall flow and an invert elevation, and then lowering tailwater elevation by increments and measuring impact pressures on a pre-determined grid on the downstream river bed. Testing began with the invert at El. 16 feet and with the tailwater at the upper end of the design range (El. 28 feet.). Testing with decreasing tailwater level continued until the criterion for impact pressure on the riverbed was exceeded. Then the outfall invert was lowered and the test sequence repeated, starting with a tailwater level above the lower end of the acceptable range for the previous invert elevation.

Operating ranges were established for two criterion levels:

- Impact pressure equivalent to a velocity component perpendicular to the riverbed of 10 fps.
- Negligible impact pressure, below an equivalent perpendicular velocity component of 5 fps (This has proven in subsequent analysis to be near the instrument noise/precision threshold).

Table 4-4 presents the provisional operating ranges developed during these tests.

Table 4-4 Provisional Adjustable Cantilever Outfall Operating Ranges

Equivalent Bottom Impact Velocity *(fps)*	**Outfall Invert Elevation** *(ft)*	**Operating Tailwater Elevation Range** *(ft)*
10	10	8.5 – 21
	16	13 – 28
Negligible	7	8.5 – 18
	11.5	13 – 21.5
	16	17 – 28

For tailwater El. 35 feet a mild hydraulic jump formed in the downstream end of the outfall channel. This washed out of the outfall when tailwater was lowered to El. 28 feet. In this mode and whenever the relationship between tailwater level and outfall invert was the same (tailwater El. 12 feet higher than the outfall invert) a rooster tail shock wave formed at the water surface downstream where the shockwaves converged, which formed at the sides of the jet where supercritical flow from the outfall contacted the slower ambient flow. The jet remained in the skimming mode and did not contact the riverbed. Upon lowering the tailwater to the invert elevation, the jet mixed with the ambient flow and spread to the bottom several hundred feet downstream. When the water level was lowered below the minimum recommended value for a given outfall invert elevation, the jet visibly contacted the riverbed.

4.7.3 Plunge Pool Development

It was proposed that the plunge pool design be developed by using scour of movable model riverbed materials as a surrogate to develop a stable pool design with fairly uniform turbulent energy at its boundary. Information presented in the literature by Izbash and Khaldre [12] indicate the material size chosen (equivalent to 0.5-0.6 foot diameter prototype rock) would be stable in a highly turbulent environment at velocities up to about 6-10 fps. This velocity level did not constitute an engineering criterion for the allowable bottom impact pressure to meet the biological guideline of preventing fish contact and/or injury. In an attempt to develop an engineering criterion, the conditions for biological field tests at the Bonneville 2nd Powerhouse Ice and Trash chute in the fall of 2001 [13] were reproduced in the 1:30 model and bottom impact pressures were measured. The maximums were equivalent to a velocity component perpendicular to the riverbed of 16 fps. There were no significant injuries of test fish for these conditions. However, the biological tests were not designed to determine injury due to bottom impact, since fish were injected into the periphery of the outfall jet, which may not reach the bottom. Since there was still no engineering criterion, the proposed scour design development approach was used. All tests were performed with the minimum design tailwater El. 8.5 feet.

Progress of scour with time was documented, with the bar of materials deposited downstream removed at intervals to prevent it from locally increasing tailwater and limiting scour progress and depth. Bottom impact pressures were measured during the course of tests. Once a stable scour depth was achieved, impact pressures in the hole also stabilized.

The shape of the scour hole was documented. Typical side slopes of the hole were in the range of 1.5 horizontal: 1 vertical and upstream slopes 3 horizontal: 1 vertical. Pressures on the downstream slope, which was also about 1.5 horizontal: 1 vertical, were higher than in the deep point of the hole because there was still active scour of this slope.

The downstream slope was then excavated at incrementally flatter slopes until the pressures on it were no longer greater than on the pool invert. This required downstream slopes in the range of 4 to 5 horizontal : 1 vertical. Egress of dye from the hole was observed. No dye tended to stay in the hole, but flushed directly on release.

Following this development, the shape of the hole was stabilized in the model using cement to form an engineered plunge pool with regular excavated slopes. Pressure distributions in the hole were documented. This procedure resulted in a 35 foot deep plunge pool for the

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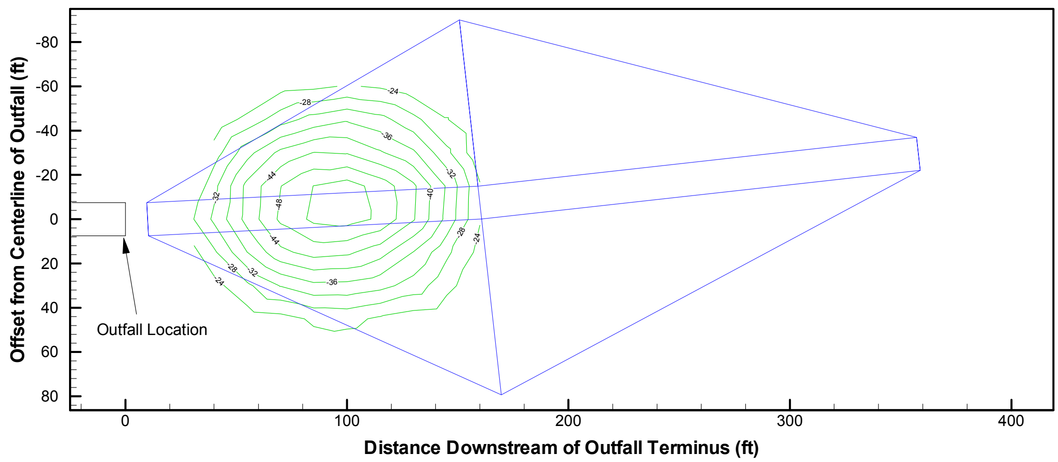
[12] Izbash, S. V. and Khaldre. 1970. Hydraulics of River Channel Closure. Published by Butterworth.

[13] ENSR and INCA Engineers, Inc. October 2001 (under preparation). Bonneville Second Powerhouse Corner Collector Outfall 1:30 Scale Physical Hydraulic Model Research for High Flow Outfall Guidelines, 90 % Draft Report. ENSR Document No. 3697-002-430. Prepared for USACE Portland District.

mid-level cantilever design (outfall invert El. 16 feet) and a 40 foot deep plunge pool for the classic cantilever outfall design (outfall invert El. 26 feet.). Average equivalent bottom impact velocities were on the order of 10 fps for both of these designs. Because there was no engineering criterion for the acceptable level of impact pressure and visual assessment of the jet contact with the pool bottom was considered unacceptable an additional design option was sought.

During the course of testing, geotechnical information became available that indicated the maximum practical depth for the plunge pool would be 50 feet below the existing riverbed El. –20 feet. The plunge pool was deepened to 50 feet, with the same location and slopes as the other holes and pressure data were acquired for the Mid-Level Cantilever Outfall design. This plunge pool configuration is shown in Figure 4-14. The average bottom impact pressures for this design were equivalent to a velocity component of 5-6 fps. The level of jet contact with the plunge pool bottom for this design was reduced, but still visually apparent at the low tailwater level of El. 8.5 feet. Bottom impact pressures would be higher and the jet contact with the bottom more pronounced for a Classic Cantilever outfall with a 50 feet deep plunge pool.



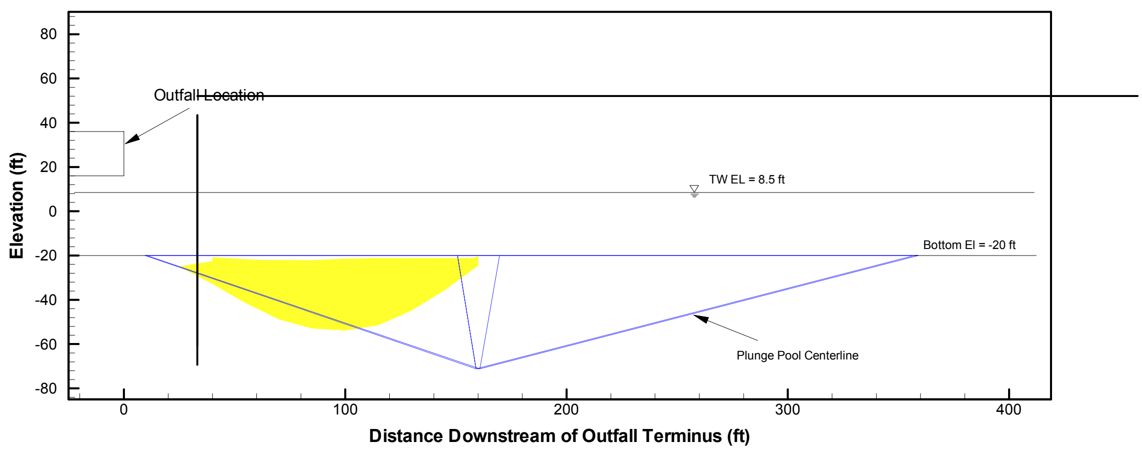


Figure 4-14 Mid-Level Cantilever Outfall with 50 Feet Deep Plunge Pool

4.7.4 Selection of Outfall Types

It is possible to operate an Adjustable Cantilever outfall over the entire design range of tailwater levels from El. 28 to 8.5 feet using only three different outfall invert elevations, 16, 11.5 and 7 feet, without recording any significant bottom impact pressures. The performance of this design was considered acceptable by the design team and the Regional Agency and Tribes representatives for further development.

A Mid-Level Cantilever outfall, with its invert at El. 16 feet and with the maximum practical 50-foot deep plunge pool, will result in bottom impact pressures equivalent to a 5-6 fps velocity component at the minimum design tailwater of El. 8.5 feet. The bottom impact and jet contact will be reduced with increasing tailwater levels. The performance of this design was considered acceptable by the design team and the Regional Agency and Tribes representatives for further development.

A Classic Cantilever outfall with its invert at El. 26 feet with the maximum practical 50-foot deep plunge pool will result in greater jet contact and bottom impact pressures than the Mid-Level Cantilever outfall. The performance of this design was considered unacceptable with the 40-foot deep plunge pool. As a result, the Classic Cantilever design was removed from further consideration.

4.8 Conclusion

As a result of this Stage 1 process, the Design Team, District Staff, and Regional Agencies and Tribes agreed that the Adjustable Cantilever and Mid-Level Cantilever would be the two outfall types to be carried forward to Stage 2 for final hydraulic model testing and evaluation.

5 OUTFALL RANGE/SITE SELECTION

5.1 Introduction

A major concern in the design of an outfall is the conveyance and discharge of the flow to the site. For a given project operation, the difference between headwater and tailwater establishes a net driving head for the outfall flow. This head drives the flow through the conveyance channel, to the terminal structure, and into the tailrace. The amount of head lost to friction and bend losses in the conveyance channel dictate the amount of residual energy delivered to the terminal structure. Different outfall sites will place different requirements on the terminal structure to meet the design guidelines. Therefore, siting and design of the outfall cannot be undertaken independently, ultimately must be designed in combination.

Initially, however, general areas or 'ranges' for a potential outfall can be identified and evaluated independent of the outfall type. As part of the Stage 1 study, several ranges were identified, screened, evaluated and selected utilizing the HiQ Outfall guidelines, the Bonneville 1:100 General Model, and professional judgement. This process is depicted in Figure 5-1, and the two selected outfall ranges were carried forward for more detailed analysis in Stage 2, which is covered in Section of this report.

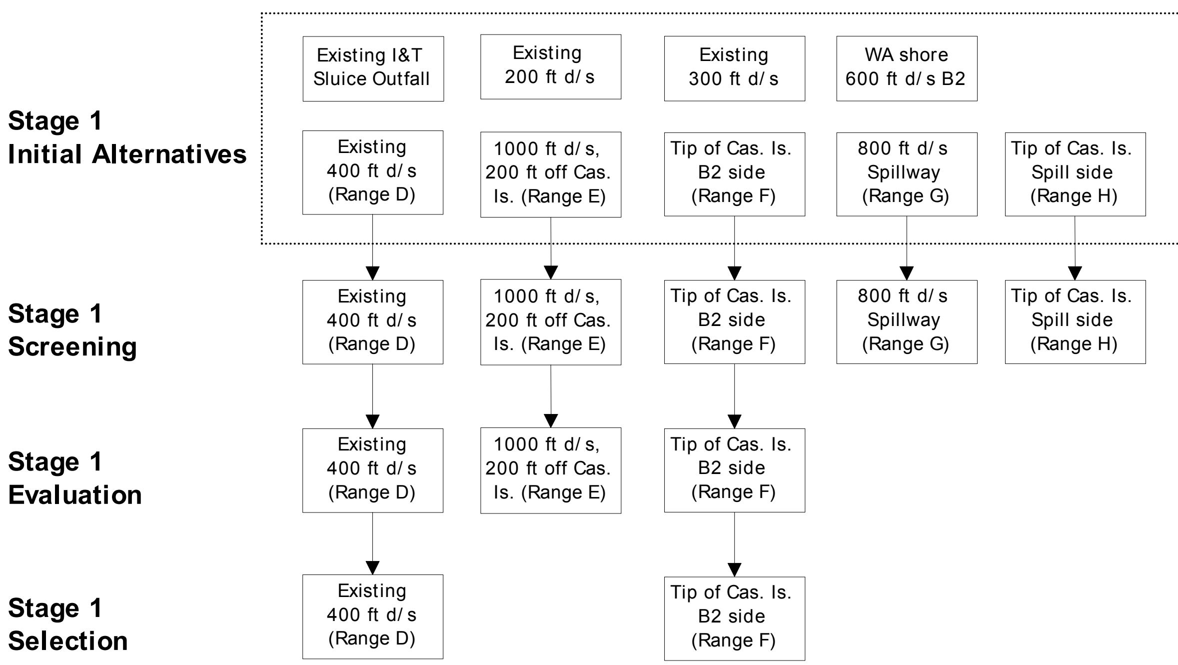


Figure 5-1 Progression for Range/Site Selection – Stage 1

5.2 Initial Range Identification Procedures

Initial development of potential outfall sites required the identification of areas in the tailrace with hydraulic conditions suitable to site HiQ outfalls. These outfall areas, called 'ranges', were identified below B2 using dye studies on the Bonneville 1:100 General Model at ERDC. The outfall ranges were identified over the course of two trips to ERDC (August to September 1999). The first trip was used as a reconnaissance type of effort. The second trip was used to fully interrogate the previously identified sites, investigate other possible sites, and finalize the ranges. Detailed summaries of each trip are provided in Appendix A. (These trips were also utilized to identify potential HiQ outfall sites for the B1 tailrace. Thus, B1 is also discussed extensively in the trip reports included in the appendices. A report titled "Bonneville First Powerhouse High Flow Outfall Site Selection Study", dated October 2000 documents the work performed for B1.)

5.2.1 Model Description[14]

The 1:100 model reproduces approximately 3.7 miles of the Columbia River channel, extending approximately 5,400 feet upstream of the dam, including the adjacent overbank area. Also included were the 76-foot-wide by 500-foot-long lock, a ten-unit powerhouse adjacent to the lock, a spillway containing eighteen bays, and an eight-unit powerhouse along the right descending bank. The model was of the fixed-bed type, with the channel and overbank areas molded in sand-cement mortar to sheet metal templates. The lock, dam crest, powerhouses, piers, and guard walls were fabricated out of sheet metal and/or Plexiglas. The dam gates were simulated schematically with sheet metal, slide-type gates. Model bathymetry was remolded to conform to data from a recent hydrographic survey.

5.2.2 Operating Conditions

Outfall sites must provide acceptable plume dynamics over the range of expected project operations during juvenile migration season. This range of operations was represented in a set of flow scenarios. The operations were based upon low, medium, and high river flows of 150 kcfs, 250 kcfs, and 330 kcfs, respectively. The District, in coordination with the Agencies, specified the flow scenarios that are summarized in Tables 3-2. Spill flow and patterns, and turbine operations were based on the 1999 Fish Passage Plan.

[14] U.S. Army Waterways Experiment Station, (1998). Navigation Conditions at Bonneville Locks and Dam, Columbia River. Technical Report CHL-98-6, p.5.

5.2.3 Test Procedures

5.2.3.1 Baseline Dye Releases

Baseline dye releases (no outfall flow) were used to characterize the general flow patterns of each powerhouse tailrace. This permitted potentially acceptable high-flow outfall ranges to be identified. These releases were performed for the low flow scenarios only because tailrace receiving water conditions are generally favorable at medium and high project flows.

5.2.3.2 Outfall Plume Tracking

Potential outfall ranges were evaluated by tracking the discharge plume from a portable outfall mock-up as the discharge plume progressed downstream. This was accomplished by dying the outfall flow with potassium permanganate crystals. The crystals were dropped into the headbox of the mock-up where they dissolved completely before exiting the outfall structure. The portable mock-up outfall was constructed to simulate a simple cantilevered plunging type outfall.

Water was supplied to the mock-up from the model forebay with a small pump. At B1, the model outfall flow was approximately 13,400 cfs discharging at a width of 40 feet. At B2, the model outfall flow was approximately 5,000 cfs discharging at a width of 15 feet. For both outfalls, the depth of flow ranged from 7 to 10 feet and simulated plunge entry velocities on the order of 35 to 50 fps.

The mock-up outfall was roughly oriented such that the plume was directed downstream and away from shorelines. A Polaroid camera was used to document the orientation of the outfall at each range. This enabled the orientation of the outfall to be approximately the same at different flow scenarios.

Outfall invert elevation will have an impact on the near-field plume hydraulics. Elevations that are high relative to tailwater level may result in jet impact on the river bottom and greater initial spread of the plume. Elevations that submerge the outfall with tailwater and force a hydraulic jump in the outfall will also have different near-field plume characteristics. Elevations near the local tailwater level will tend to skim flow along the surface of the water. Since outlet invert elevation has not been determined yet, it was fixed at a constant elevation of 25.0 feet above tailwater for comparative purposes for most of the tests. Some tests performed with an invert elevation of 17.0 to 19.0 feet to evaluate the sensitivity of plume performance to invert elevation.

The outfall type, orientation, and elevation will all have an impact on plume dynamics. Therefore, the following characterizations should be considered general and preliminary.

5.3 Initial Outfall Range Alternatives

Nine potential HiQ outfall site ranges were investigated at B2. These sites are shown against a plan view of the project in Figure 5-2, page 73. The general behavior of the outfall plume at each of these release sites was evaluated for the low flow scenario. The results were follows:

1. Existing Sluice Chute: The outfall plume acted as a barrier to the powerhouse flow while entraining the flow from the southern units. This created a small area of stagnant water between the plume and Cascades Island. The outfall plume hugged Cascades Island until it merged with the spillway flow and was deflected to mid-channel.

2. 200-foot Sluice Chute Extension: The outfall plume acted as a barrier to the powerhouse flow while entraining the flow from the southern units. This created a low/no velocity region across the bank of Cascades Island. The plume was oriented near the banks of Cascades Island until it merged with the spillway flow and was deflected to mid-channel.

3. 400-foot Sluice Chute Extension: The outfall was extended far enough away from the powerhouse and Cascades Island that some of the flow from the southern units was able to get in behind the outfall. This flow behind the outfall eliminated the low/no velocity region noted previously. However, the outfall plume extended over to Washington shoreline and maintained close proximity to it throughout its travel downstream.

4. 300-foot Sluice Chute Extension: The outfall was extended far enough to prevent the formation of a low/no velocity region along Cascades Island but not far enough for the plume to reach the Washington shoreline. This area seemed preferable to the 200-foot or the 400-foot areas for this flow scenario.

5. 1,000 feet downstream and 200 feet off Cascades Island: Entrainment of the southern powerhouse unit flow along Cascades Island generated a eddying area on its banks. The plume traveled near the shore of Cascades Island until it was deflected to mid-channel by the spillway flow.

6. Tip of Cascades Cascades Island, B2 Side: A very small back eddy was created at the tip of island. The outfall plume moved to mid-channel as it merged with the spillway flow.

7. 800 feet downstream of spillway and 150 feet off Cascades Island: The outfall plume travels along banks of Cascades Island and then behaves very similar to previous release site.

8. <u>Tip of Cascades Island, Spill Side</u>: Plume behavior was nearly identical to that of previous release site.

9. <u>North Shore Release</u>: The outfall plume spread laterally across the north half of channel while maintaining close proximity to the Washington shoreline.

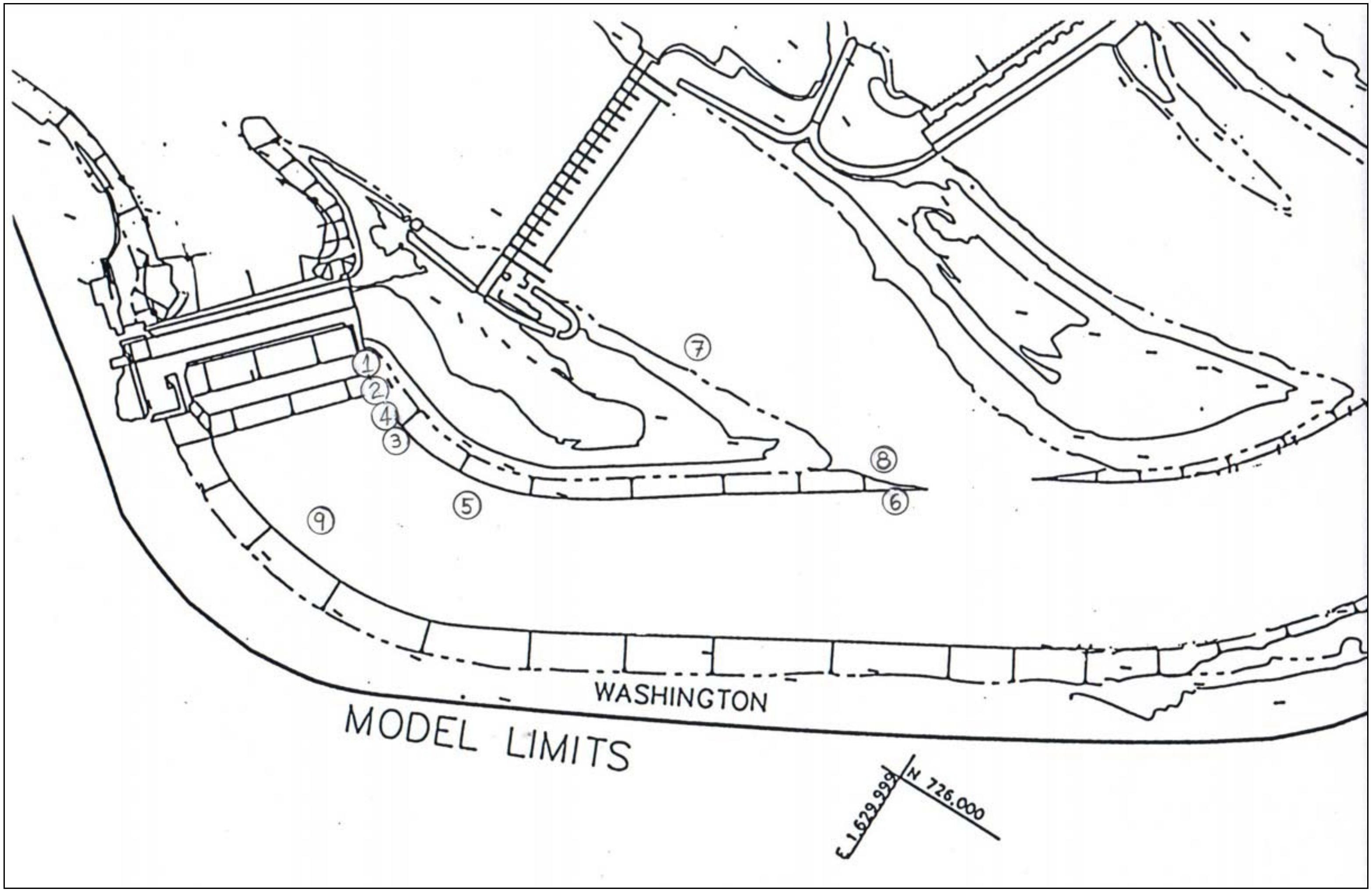


Figure 5-2 Initial Potential B2 Outfall Ranges

5.4 Screening

Site ranges 3, 5, 6, 7, and 8 were selected as preliminary outfall site ranges at B2 based upon their general plume behavior as noted above[15]. To avoid confusion, the selected sites were relabeled D, E, F, G, and H respectively (see Figure 5-3, page 75). Plume tracking studies were then performed at the remainder of the flow scenarios (medium and high). No significant decrease in outfall plume performance was noted. Figure 5-4 through Figure 5-8 (pages 76 - 80) summarize the general dye plume limits for ranges D, E, F, G, and H respectively for flow scenarios with spillway flow (B2O-a, B2O-c, B2O-d, and B2O-e) per Table 3-2. Figure 5-9 through Figure 5-11 (pages 81 - 83) show the general plume limits for ranges D, E, and F respectively for flow scenarios without spillway flow (4b).

The investigations also enabled some general conclusions to be drawn regarding the plume behavior of HiQ outfalls located in the B2 tailrace. In general:

- Sites located near shorelines have a tendency to entrain flow away from the bank and create a small region of stagnant/eddying flow next to the plume immediately downstream of its entry point.

- Sites oriented in the southern part of the tailrace, near Cascades Island, produce plumes that have the least chance of coming into close proximity of the Washington shoreline. In general, the plumes were oriented mid-channel upon merger with the spillway flow. As the flow from both B1 and the spillway increases, the plumes will likely be more northerly located.

- Sites oriented in the northern part of the tailrace, near the Washington shore, produce plumes that come into close proximity of the Washington shoreline. These are forced further towards the shore upon merger with the spillway and B1 flows.

- There may be a certain distance downstream of the powerhouse along Cascades Island at which outfall plume egress characteristics are always acceptable. If so, detailed model work would be required to determine that 'certain distance'.

[15]As mentioned in Section 5.2, two visits were made to ERDC to perform model testing and initial screening on the nine originally identified B2 release sites, one trip in August 1999 and one in September 1999. The August trip included only staff from the District and the INCA team, while the September trip also included staff from the Regional Agencies and Tribes. A close review of the trip reports for these two trips, which are included in Appendix A, shows that a 300-foot sluice chute extension is preferred in August, while a 400-foot sluice chute extension is preferred in September. Although the testing approach was generally the same for both trips, there were some "improvements" made to the overall model testing (such as a better model of the outfall structure) for the September trip. Thus, when the September modeling was completed, it was agreed by all participants that a 400-foot sluice chute extension performed best among the various Range D sites.

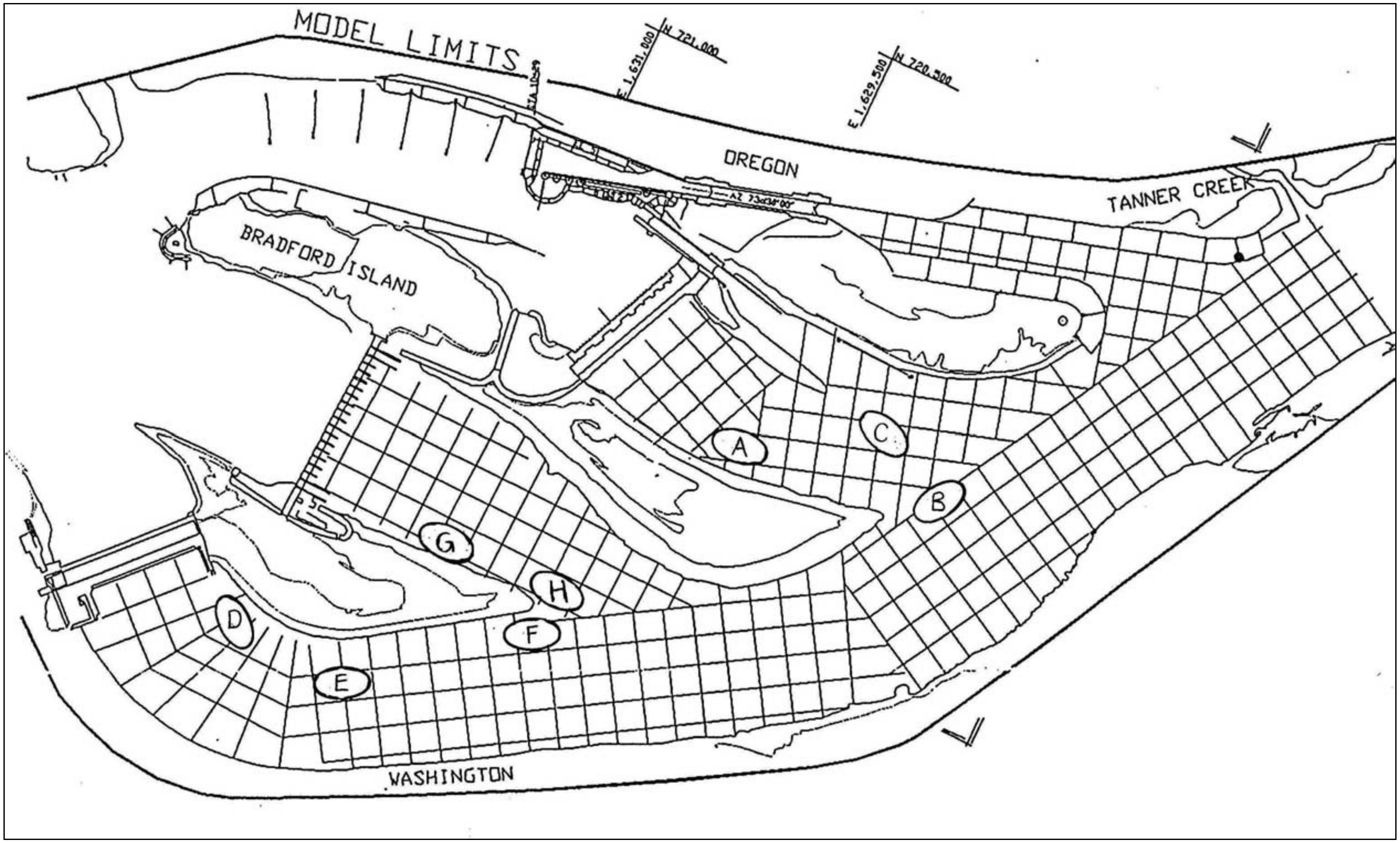


Figure 5-3 Primary Release Sites Examined – Bonneville Dam Tailrace

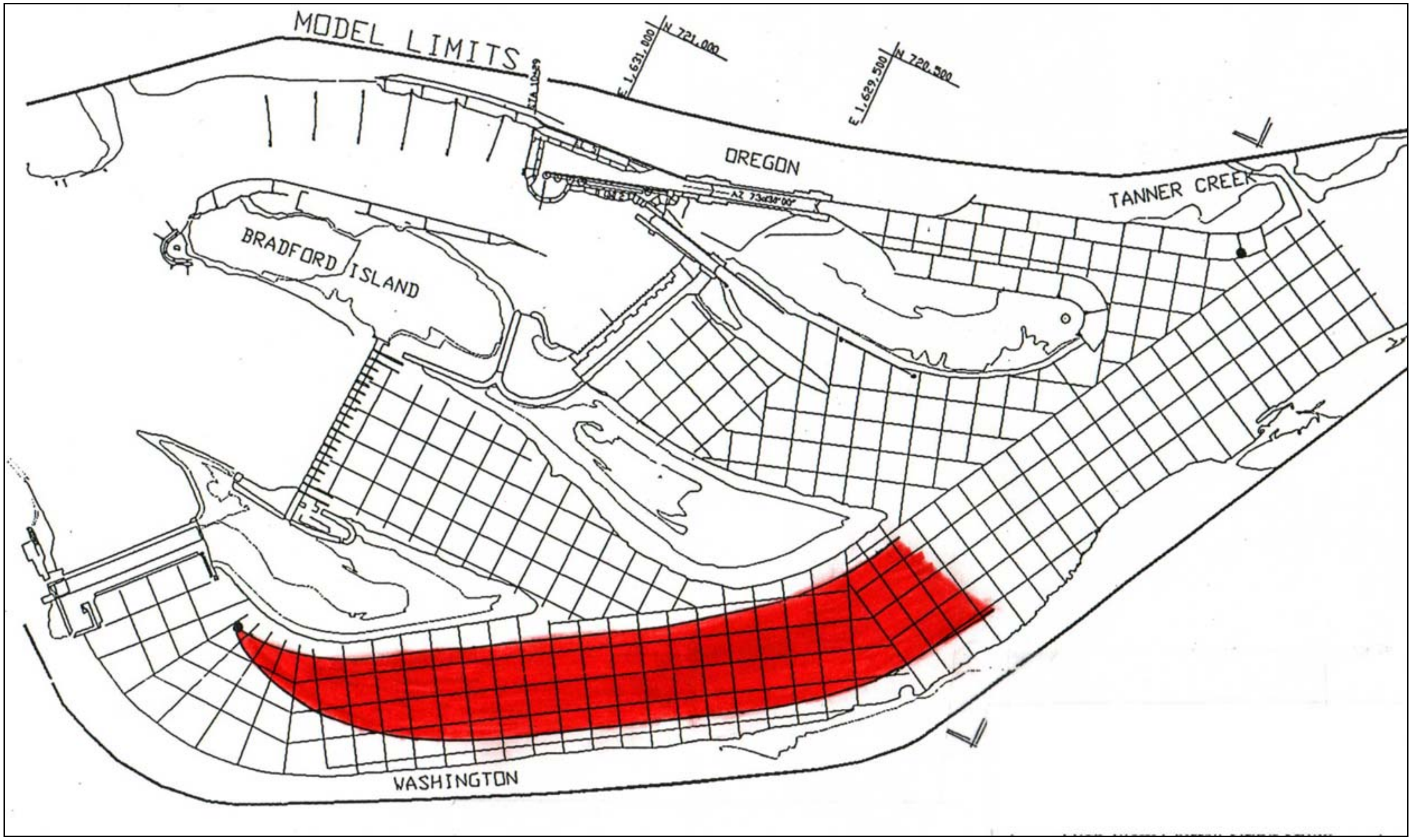


Figure 5-4 Range D Plume Extent Summary – Flow Scenarios B2O-a, B2O-c, B2O-d, and B2O-e

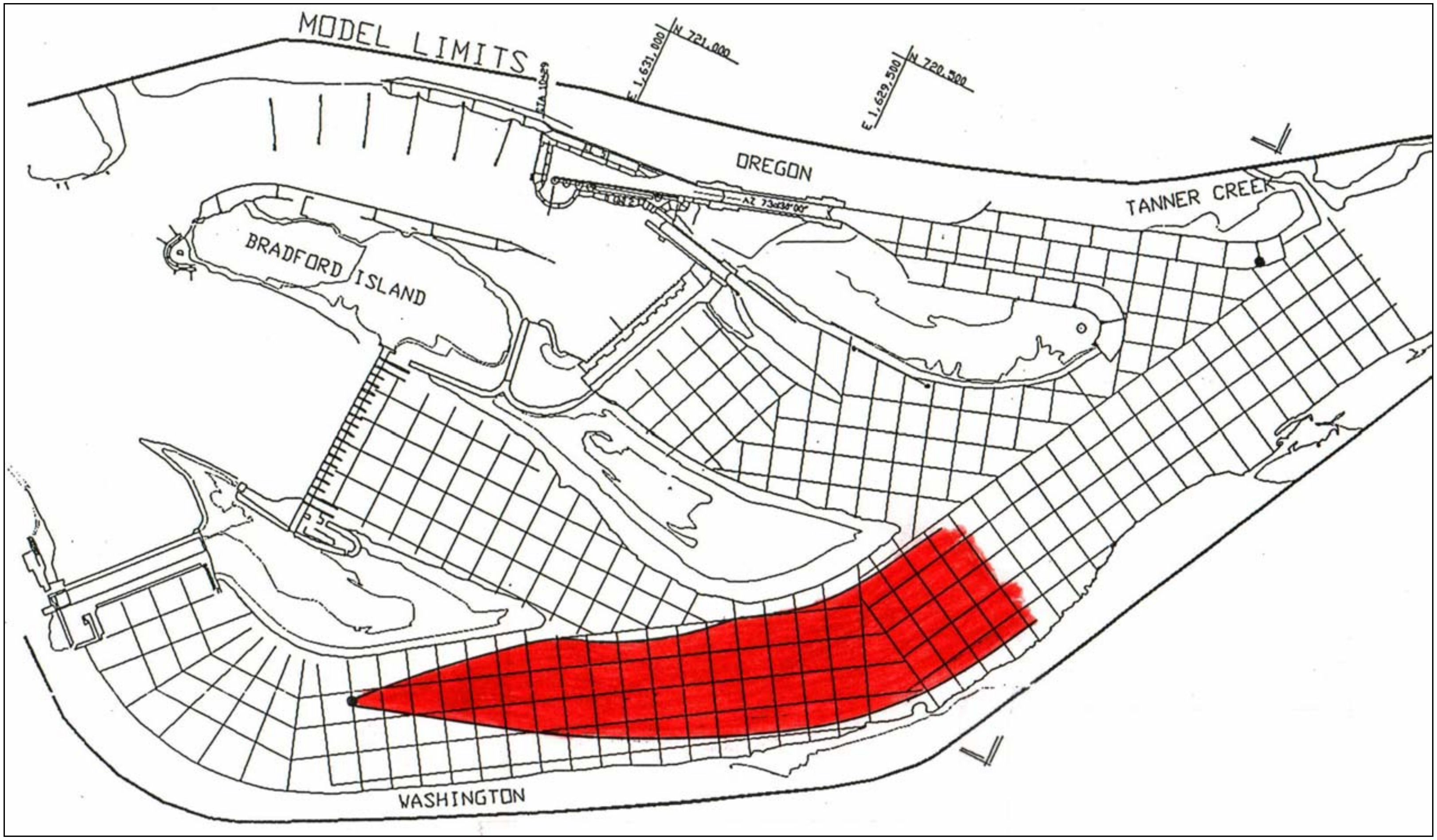


Figure 5-5 Range E Plume Extent Summary – Flow Scenarios B2O-a, B2O-c, B2O-d, and B2O-e

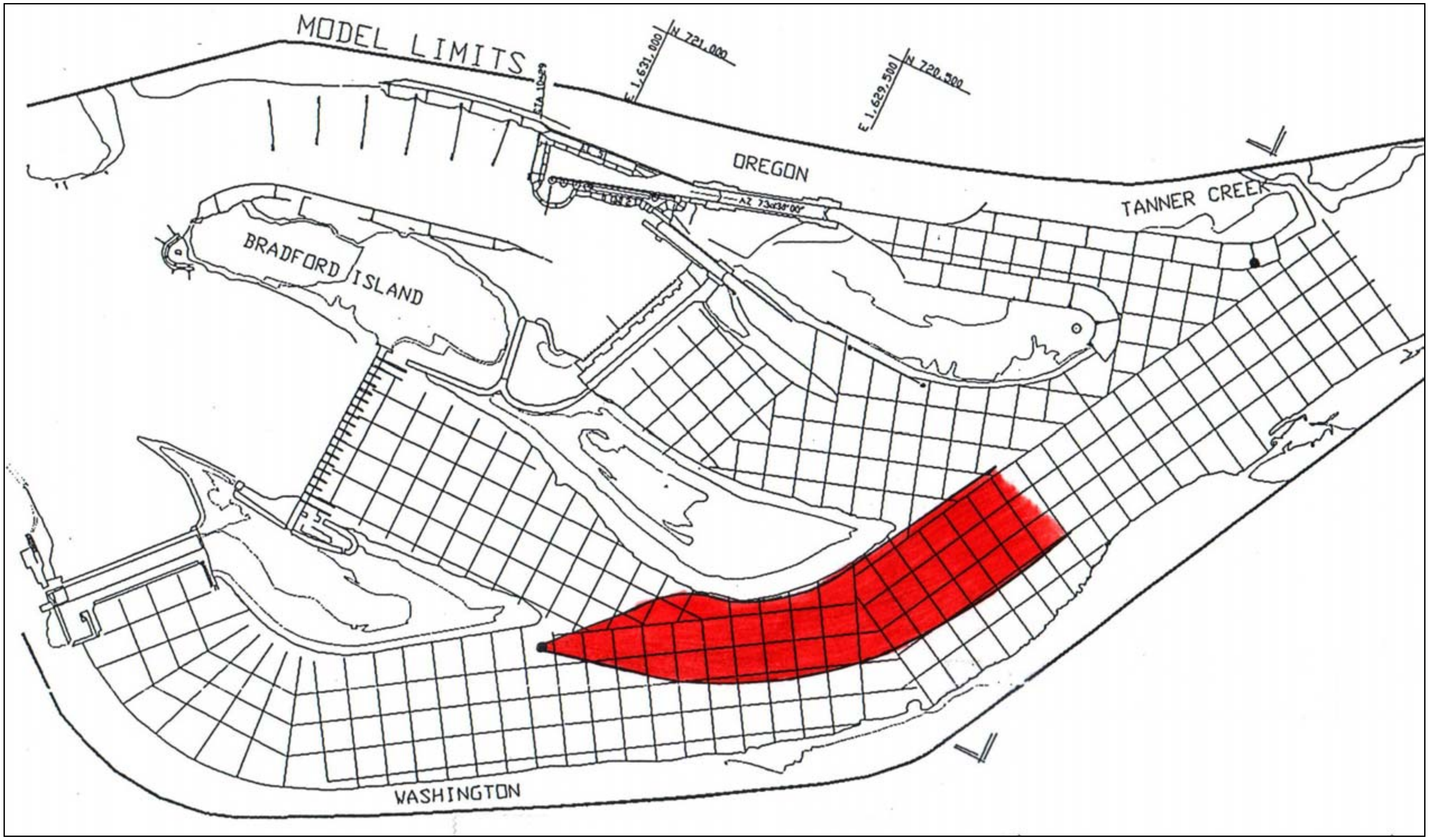


Figure 5-6 Range F Plume Extent Summary – Flow Scenarios B2O-a, B2O-c, B2O-d, and B2O-e

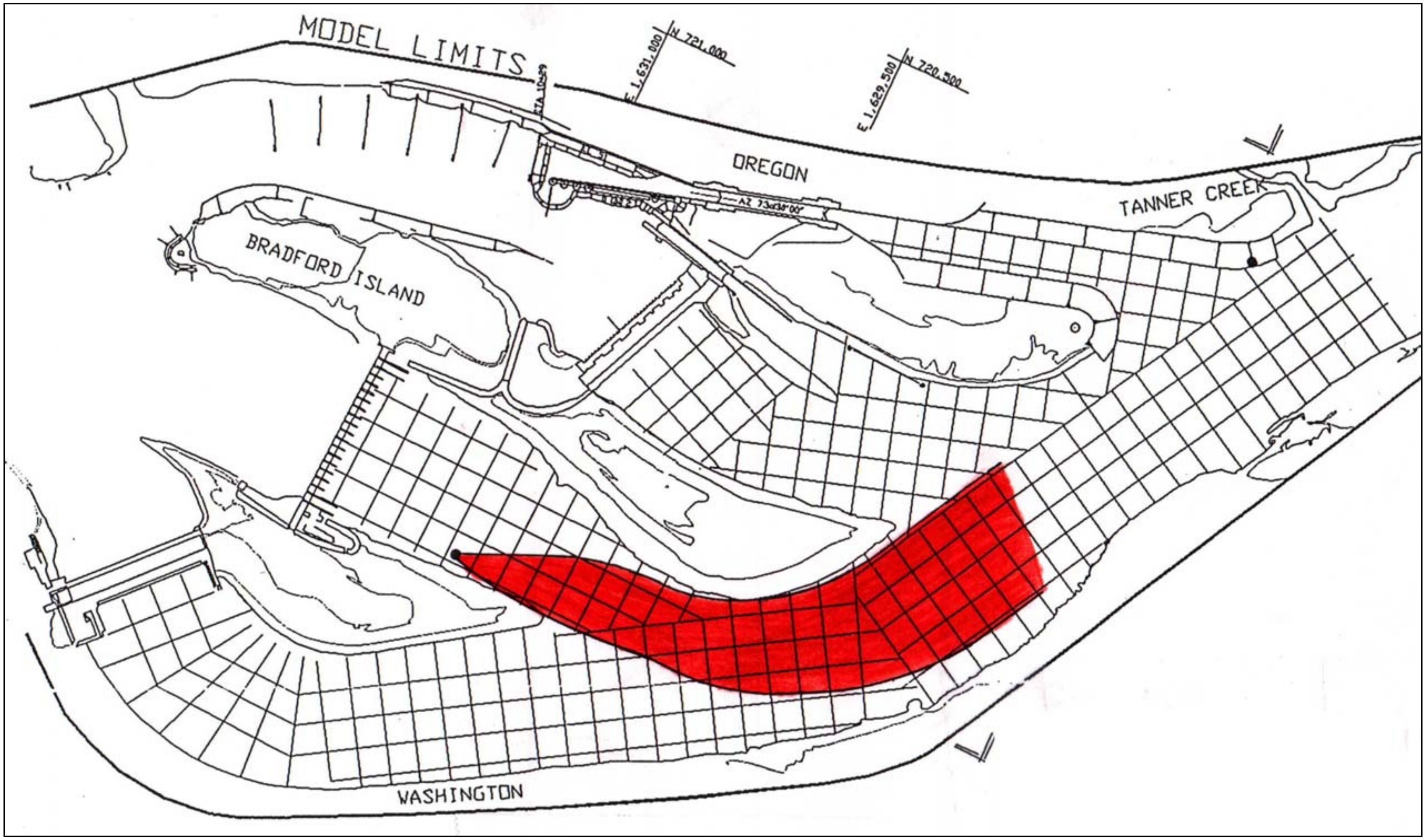


Figure 5-7 Range G Plume Extent Summary – Flow Scenarios B2O-a, B2O-c, B2O-d, and B2O-e

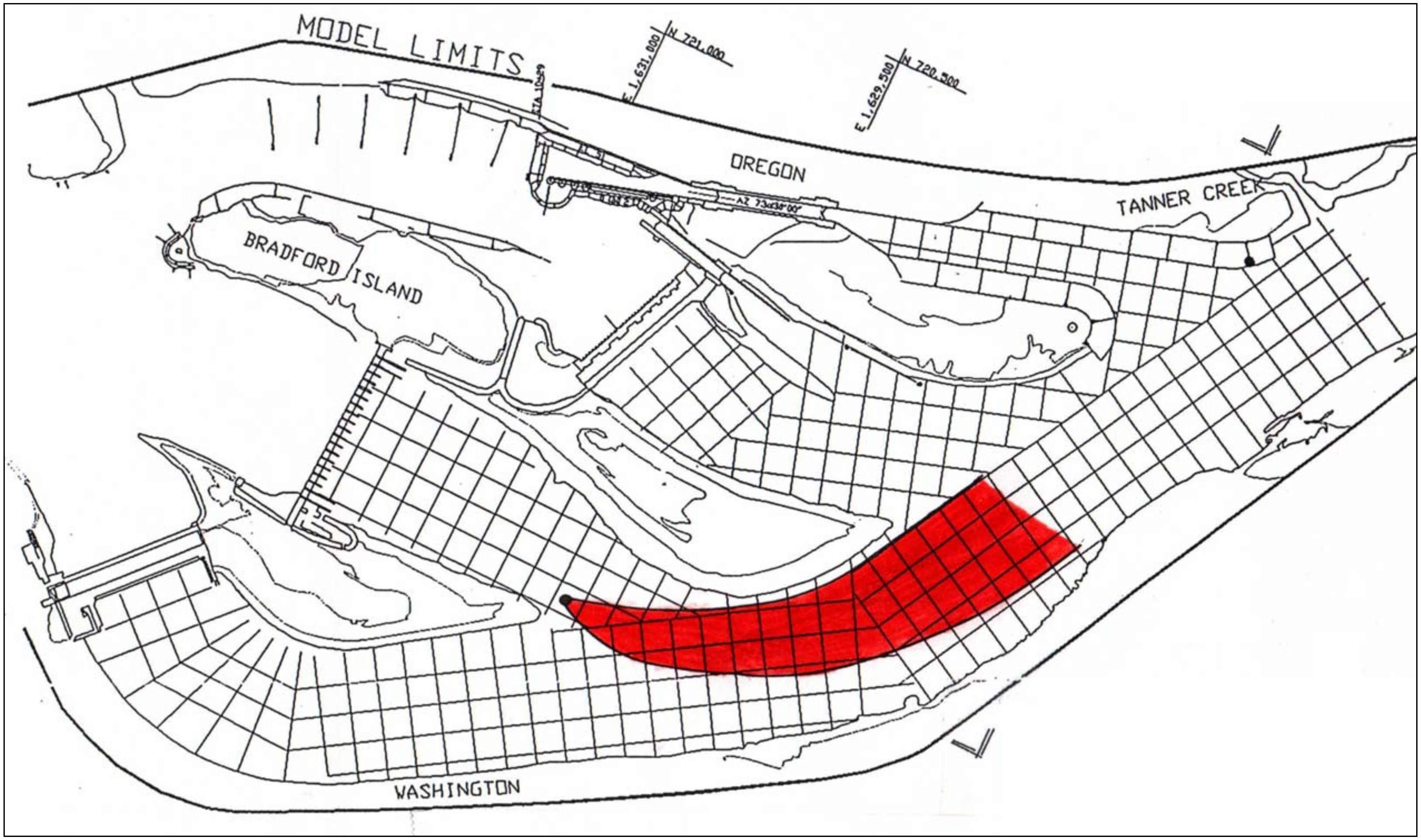


Figure 5-8 Range H Plume Extent Summary – Flow Scenarios B2O-a, B2O-c, B2O-d, and B2O-e

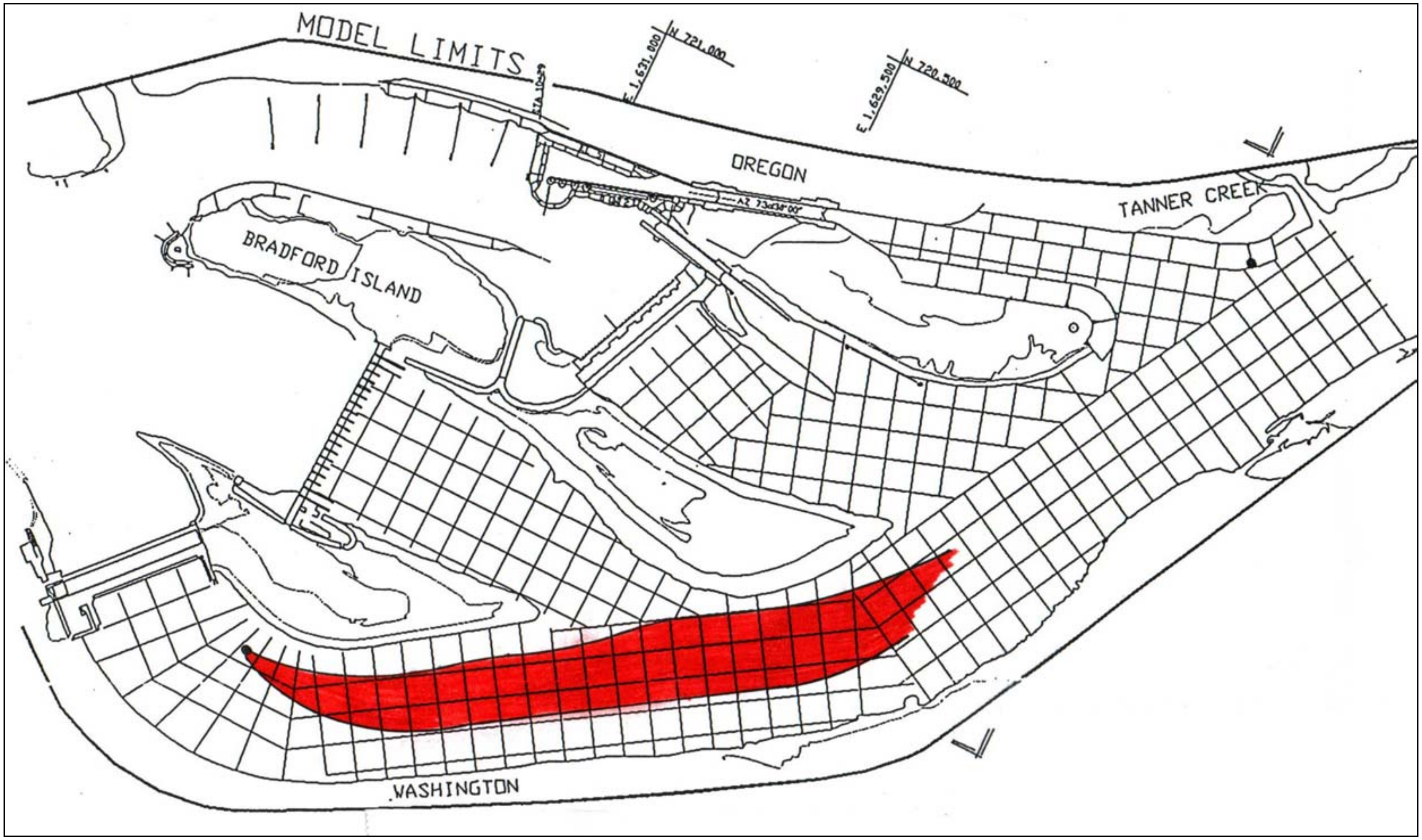


Figure 5-9 Range D Plume Extent Summary – Flow Scenario B2O-b

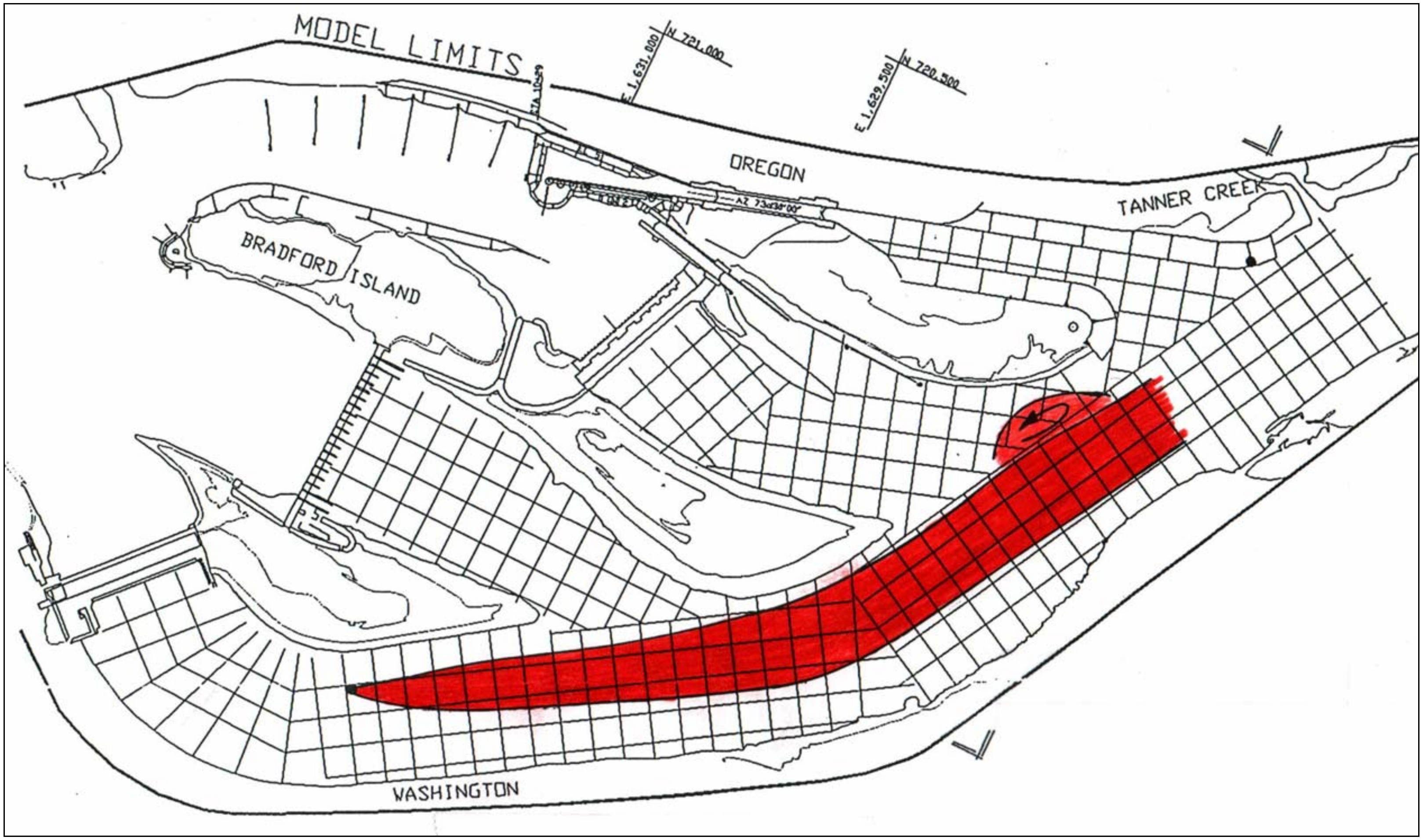


Figure 5-10 Range E Plume Extent Summary – Flow Scenario B2O-b

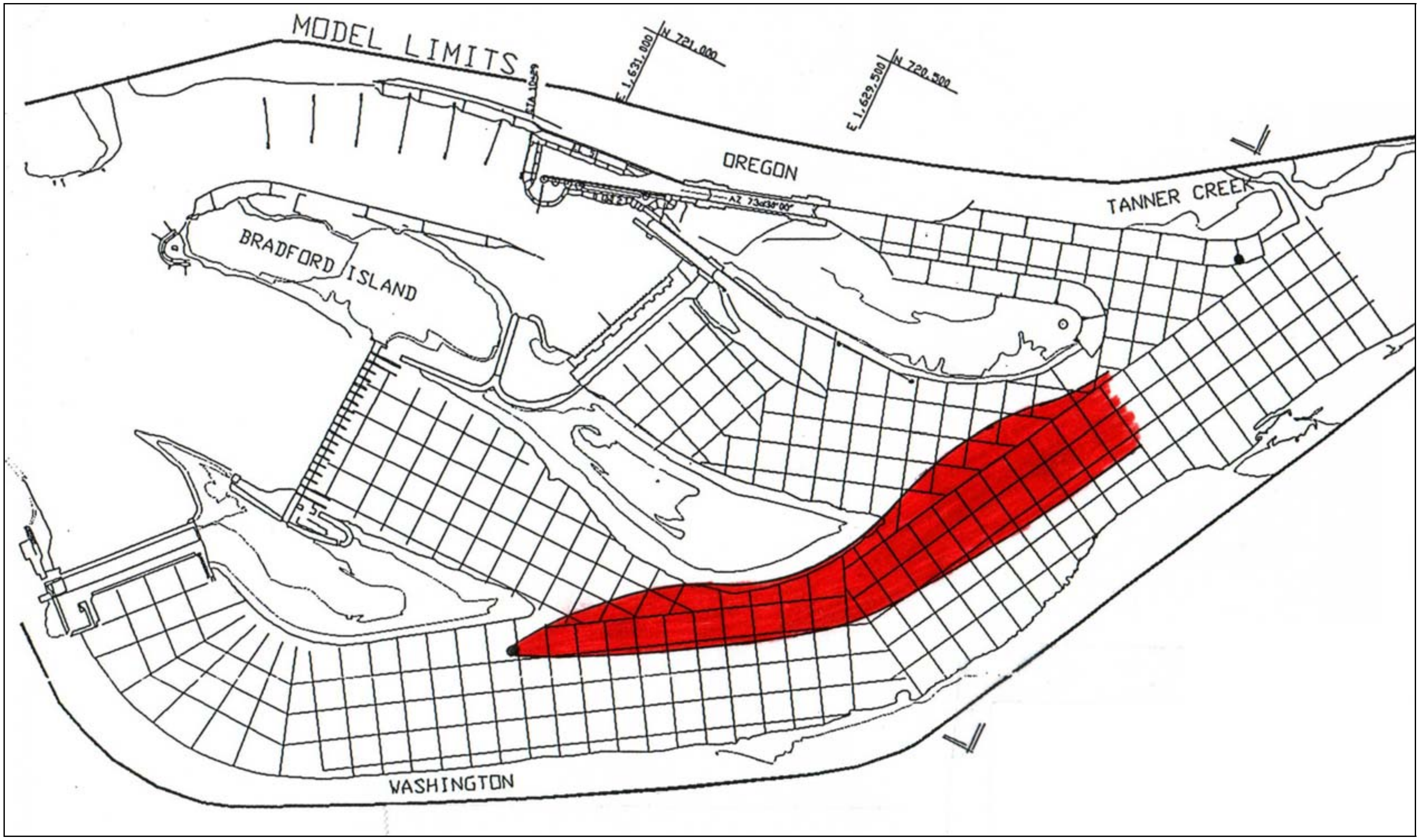


Figure 5-11 Range F Plume Extent Summary – Flow Scenario B2O-b

5.5 Primary Evaluation

5.5.1 Evaluation Criteria and Scoring

Once the necessary hydraulic modeling has been accomplished, the various tailrace ranges could be evaluated utilizing an evaluation matrix. The following criteria were developed based upon the preliminary guidelines and other important factors that will potentially affect outfall performance. It should be noted that there are no 'structural' criteria. Two structural criteria were originally considered: (1) distance and length of the potential conveyance channel, and (2) distance of the range from the shore. However, these factors primarily affect the construction costs, so including these criteria duplicated the cost criteria.

The appropriateness of weighting the criteria was considered. However, after a review of the merits of each of the criteria, it was decided that all the criteria were of approximately equal importance and that no one criterion should have a more significant influence in the evaluation than the others.

The ranges were evaluated versus the criteria in a matrix, presented at the end of this section. The team experts in a particular area proposed a score for each range under each criterion; e.g. the team biologists proposed scores for the biological factors, etc. The proposed scores were reviewed and approved by the entire team. A score of 1 indicates the performance of the range for that criterion is expected to meet the preliminary guidelines and/or be comparable to that of other regionally accepted outfalls. A score of 2 indicates the range performance is expected to be less than that required by the preliminary guidelines, but with minimal impact to smolts. A score of 3 indicates the range performance is neither expected to meet the preliminary guidelines nor to be acceptable with respect to potential injury to smolts.

Evaluation Criterion No. 1 -- Predators are not known to be concentrated near the nearfield outfall plume and substrate does not provide potential predator cover near the nearfield outfall plume.

Explanation - Smolts may be vulnerable to predation if they are impaired in any way during passage. This criterion helps protect smolts from predation while they recover from any passage stress. It is generally intended to decrease predation rates on bypassed smolts. This criterion also emphasizes not siting outfalls in locations where the bottom materials may provide potential cover so that predators may hold near or immediately downstream from the outfall location. The evaluation was made on the basis of the substrate materials, with PSA material types at and immediately downstream from the outfall site range being considered acceptable and BLD materials unacceptable. Note that predators may redistribute after a new smolt outfall is constructed. (Preliminary Guideline No. 4.)

Scoring - Predators have been observed along the Cascades Island and Washington shorelines and near the I&T chute. Ranges D and F coincide with known predator habitat and thus received a score of 3. Ranges E and G are further from shore and in deeper water than Ranges D and F, but still near some known predator areas. Thus, Ranges E and G scored a 2. Range H scored a 2 because it was where predators have been previously located.

Evaluation Criterion No. 2 -- Predators are not known to be concentrated near the farfield outfall plume.

Explanation - This criterion is similar to No. 1 except the emphasis is on the farfield plume. Near- and farfield plumes are distinguished to provide a detailed examination, as plume and predator characteristics may differ between these regions. (Preliminary Guideline No. 4.)

Scoring – All ranges received a rating of 2 since it is assumed that current predator locations will change as soon as the outfall is operated and that there is no reason to believe that predator concentrations after outfall operation will be significantly different between the ranges.

Evaluation Criterion No. 3 -- Adult migration (fishway entrances, shorelines, or known adult migration paths) will not be deleteriously affected by the high flow outfall discharge and plume.

Explanation - This criterion addresses false attraction and masking issues. The idea is to locate the high flow outfall where it will not interfere with existing adult attraction flows and fishways. It will be important to not impact adult migration in any adverse manner. (Preliminary Guideline No. 5.)

Scoring - None of the ranges examined seemed to present adverse conditions for adult migration. The primary migration paths are along the Oregon and Washington shorelines. Thus, outfalls in the candidate ranges should not impede migration because the ranges are away from the main state shorelines. The ranges are also away from fishway entrances. Thus, all ranges under consideration at B2 scored a 1 because Preliminary Guideline No. *5* was upheld.

Evaluation Criterion No. 4 — Receiving water velocities exceed 4 fps, or site-specific velocities with an operating high flow outfall are acceptable.

Explanation - This criterion helps protect smolts from predation by providing local velocities near the outfall that will prevent predator holding. No outfall site will satisfy' this criterion if river flows or flows to the channel the outfall site is located in are too low. Therefore the potential outfall site ranges are rated on a relative rather than absolute basis, as operational restrictions may be necessary to meet the guidelines. (Preliminary Guideline No.1.)

Scoring - Table 5-1 presents the variation in ambient velocities in each of the ranges for the flow field data presented in Figures 3-4 through 3-6. A score of 1 indicates that velocities essentially meet the 4 fps criterion for all of the test scenarios relevant to the particular range, e.g. scenarios where B 2 was operating for ranges in the B 2 tailrace. If some velocities dropped below the 4 fps criterion in a range, but the magnitude of the higher velocity streamlines in the range still met the criterion for all scenarios, a score of 2 was given. A score of 3 indicates that for some of the scenarios, there were no velocities within the range, even on the highest velocity streamline, which met the 4 fps criterion. Range G in the spillway tailrace received a score of 3, even considering only scenarios where the spillway was operating (B2O-A and B2O-C).

Table 5-1 Ambient Tailrace Flow Velocities versus Outfall Range

Flow Scenario[1]	Ambient Flow Velocity In Range (fps)[2]							
	A	B	C	D	E	F	G	H
B1 – a	3.9-5.4	1.6-5.0	4.7-5.0	0	0	0.3-0.5	5.0-6.4	4.3-6.1
B1 – b	4.8-5.9	2.7-6.2	5.1-6.9	0.3-0.5	1.0	0.3	1.0	0.6-1.1
B1 – c	4.7-5.4	5.3-5.7	4.8-5.1	0	0.9-1.1	0.3-0.6	2.7-3.9	4.5-5.2
B2 – a	0	3.4-5.9	0	1.7-5.8	3.0-4.2	2.3-3.5	2.1-2.9	4.3-6.0
B2 – b	0	2.6-4.2	0	4.9-6.6	5.5-6.1	4.4-5.5	0.4-0.6	0.9-0.7
B2 – c	0	1.1-6.7	0	5.0-6.8	4.8-6.1	3.8-3.9	3.7-5.0	3.5-4.4

Grey shading – indicates non-design conditions for an outfall at the particular range.

1 – See Table 3-2 .for flow scenario information; data in this table are for the identified scenarios without outfall flow.
2 – Velocities were derived from float data from 1:100 general model, see Figures 3-4 through 3-6.

Evaluation Criterion No. 5 -- Plume dispersion characteristics in the far field do not result in the plume nearing shorelines or entering eddies or slack water areas.

Explanation - This criterion is intended to prevent the smolts in the plume from being carried near locations where predators may easily hold. Evaluation relative to this criterion was guided by the scores in the summary table of the results of the preliminary outfall siting studies performed at ERDC[15]. (Preliminary Guideline No. 4)

Scoring - Ranges D and E in the B2 tailrace received scores of 1 because, on the average, the bulk of the outfall plume stayed well offshore. Ranges F, G, and H received a score of 2 because, on the average, the bulk of the plume remained greater than 100 feet offshore, but with periodic excursions of its edges near shore. None of the ranges received a score of 3, which would indicate plumes that tended to hug the shoreline or deflect off shoreline projections.

Evaluation Criterion No.6 — Plume dispersion characteristics are relatively constant over a range of operating scenarios.

Explanation - While the outfall plume from a site range may have excellent characteristics for one particular operation, it may not at another. This criterion is intended to downgrade sites that may require operational restrictions to satisfy the guidelines. Evaluation was guided by the consistency of performance indicated in the summary table of the results of the preliminary outfall siting studies performed at ERDC[16]. (Preliminary Guideline No. 6.)

Scoring - Ranges D and E in the B2 tailrace received a score of 1 because they demonstrated consistently acceptable performance for the scenarios tested at ERDC. Range F in the B2 tailrace received a score of 2 because it performed poorly for one test scenario. This might be addressed by placing a limitation on project operations. Ranges G and H in the spillway tailrace received a score of 3 because they will only provide acceptable performance if the spillway is operating.

[15] INCA Engineers, Inc. October 1, 1999, Bonneville 2nd Powerhouse Corner Collector Site Selections – Trip Report from the ERDC Trip of September 14-16, 1999. Prepared for USACE Portland District.

[16] INCA Engineers, Inc. October 1, 1999. Bonneville 2nd Powerhouse Corner Collector Site Selections - Trip Report from the ERDC Trip of September 14-16, 1999. Prepared for USACE Portland District.

Evaluation Criterion No.7 — Relative construction costs.

Explanation - Although cost information is not necessarily a criterion for range selection, the various potential ranges were rated on a conceptual cost basis. The cost of various outfall ranges is controlled primarily by the length of the conveyance channel. Other variables affecting the cost of the system include the type of the outfall and whether the channel is constructed on land or over the tailrace. Any required orientation of the outfall may affect the route of the conveyance channel.

Scoring - Range D has the shortest length of channel from the corner collector intake and was given a rating of 1. Range E and F were rated a 3 due to the longer channel length. Range G is appropriate for a channel beginning at the spillway. It was rated 2 due to the short length of channel required. Range H was rated 3 due to the long length of channel required.

The evaluation matrix is shown on the next page (Table 5-2).

Table 5-2 B2CC Site Selection Range Evaluation Matrix

Outfall Site Range	Description	Biological Factors			Hydraulic Factors		Proj Oper	Costs	Eval Summary	
		Predator concentrations near plume, near field	Predator concentrations near plume, far field	Impacts on adult migration	Receiving water exceeds 4 fps	Plume dispersion characteristics, far field	Plume characteristics over a range of operating scenarios	Relative construction costs	Total Score	Copmparative Ranking
A	South shore of Bradford Island, 1000 feet downstream of PH	3	3	1	1	2	2	2	14	3
B	Bradford Island Tip	1	2	1	2	1	2	3	12	1
C	Tower Island Tip	1	2	1	1	2	3	3	13	2
D	I&T Chute, 400 feet downstream of Exit	3	1	1	2	1	1	1	10	1
E	North shore of Cascade Island, 1000 feet downstream of PH	2	1	1	2	1	1	2	10	1
F	Cascade Island Tip, PH side	3	2	1	2	2	2	3	15	4
G	South shore of Cascade Island, 1000 feet downstream of Spillway	2	2	1	3	2	3	2	15	4
H	Cascade Island Tip, Spillway side	2	2	1	2	2	3	3	15	4

1 = Range performance meets the preliminary guidelines and/or is comparable to the performance of other regionally acceptable outfalls.
2 = Range performance is below the optimum preliminary guideline standards but anticipated impact to smolts is minimal.
3 = Range performance does not meet the preliminary guidelines and/or is anticipated to be unacceptable due to higher potential injury to smolts.

5.5.2 Primary Evaluation Recommendations

Based upon the evaluation matrix, Ranges D and E had the highest ratings. Assuming that each of the criteria are of approximately equal importance and that the individual scoring is an accurate reflection of the relative merits of the ranges, there is a significant difference in the final ranking between Ranges D and E and Ranges F, G, and H. However, since the Regional Agencies and Tribes (A/T's) believe strongly that a downstream site should be evaluated, it was decided at this point to carry Ranges D, E, and F forward for additional analysis and evaluation.

5.6 Outfall Site Selection

Following several meetings between the Portland District and the INCA Design Team on the B2CC Site Selection Study, it was agreed that the Team should consider and recommend whether Range E could be eliminated from further evaluation and modeling. The Regional Agencies and Tribes indicated a preference for ranges that are located as far as possible from the powerhouse. Also, a preliminary cost estimate indicate that the total cost of a system that discharges at Range E is more expensive than one that discharges at Range F because a conveyance channel to Range F can be constructed at grade whereas a channel to Range E cannot. It therefore seems appropriate to consider this possibility.

Based upon a review and discussion of the work to date (ERDC trips, modeling results, analysis of relevant criteria, and meetings with the District and A/T's), the Team recommended that Range E be eliminated from further consideration. The justifications for this recommendation were:

- In a side by side comparison of Range D and Range E using the evaluation criteria, Range D is the preferred range. There are no situations or conditions where Range E appears as a more favorable range for a HiQ outfall than Range D. Even though the evaluation matrix scores the two ranges equally in all criteria except cost, preliminary ERDC modeling indicated that Range D plume dynamics are preferable over those of Range E, although only by a slight amount.

- There is a significant difference in cost between Range D and Range E. This cost is primarily related to the longer conveyance channels for Range E.

- Range E presents a significantly higher risk of not being able to be constructed in two In-Water-Work periods, due to the extensive over-water construction it requires. Thus, selecting Range E jeopardizes the 2004 construction completion date with no discernable advantages.

- Range E is basically a 'twin' outfall range to Range D. There is no justification to carry both ranges forward for additional evaluation when Range D displays better model results, has a higher likelihood of construction completion in 2004, and is significantly less costly. The only 'advantage' that Range E has over Range D is that it is further downstream. If that were an overriding criterion, then Range F, which is even further downstream, would be preferred over Range E.

Given the significant cost advantage of Range D over Range E and the A/T's preference to have Range F included in future studies, the Team recommended that the District carry only Ranges D and F on for integration with the Outfall Type Selection.

5.7 Conclusions

As a result of this Stage 1 process, the Design Team, District Staff, and Regional Agencies and Tribes agreed that the Range D and Range F would be the two outfall ranges to be carried forward to Stage 2 for final hydraulic model testing and evaluation and integration with outfall types.

6 INTEGRATION OF OUTFALL TYPE AND RANGE SELECTIONS

6.1 Introduction

Selecting one outfall type/site combination is the fundamental goal of this project. To this point in the study, however, ranges and structure types have been treated separately. Previous plume testing was done in the 1:100 scale general model using a cantilever outfall discharging on the existing river bed. In this integration section, ranges and structure types are examined in combination, which is also important because of synergistic effects. Certain outfall types may be more effective at certain locations than others. The integration work was the first time the effects of detailed outfall structures, with conveyance channel obstruction of ambient flow plus plunge pools, were considered in plume performance.

Two outfall types, Mid-Level (MLC, Plate 6) and Adjustable Cantilever (AC, Plate 6), and two outfall ranges, D and F (Plate 5), were considered (see Sections 4 and 5, respectively). Thus, four possible outfall range/type combinations were investigated.

- Range D/Mid-Level Cantilever (MLC)
- Range D/Adjustable Cantilever (AC)
- Range F/MLC
- Range F/AC

During the course of the studies a specific outfall site within Range F, designated F-Tip, was introduced.

The integration work entailed three phases each entailing an ERDC trip (see list below). Detailed results may be found in the respective trip reports in Appendix A. After the results of the integration work are presented, this section closes with a recommendation for a preferred outfall range/type.

- Phase 1 – Initial Investigation of Combinations – ERDC March 2001
- Phase 2 – Fine-Tuning – ERDC May 2001
- Phase 3 – Fishery Agency Review – ERDC June 2001

6.2 Methods

To evaluate range/type combinations, dye plume characteristics were examined in the 1:100 scale general model of Bonneville Dam at ERDC. Methods common to all three phases are presented here, including model improvements, flow scenarios and model test runs, and a plume rating system. Methods specific to each of the three trips are explained below with reporting for that trip (Sections 6.3, 6.4, and 6.5, respectively).

6.2.1 Model Improvements

The Bonneville 1:100 scale general model was improved for the outfall range/type combination tests. The B2 powerhouse was replaced with an entirely new model structure with newly calibrated flow meters controlling the flow through individual units. The spillway structure was also re-calibrated. A calibration check of tailrace velocity profiles was performed by comparing model velocity data with recently acquired ADCP field data. Detailed physical models of outfall structures and plunge pool designs were developed on the 1:30 scale model (see Section 5) and added to the 1:100 scale model. The outfall structures included conveyance channels. The alignments of the conveyance channels were based on preliminary engineering performed for a routing feasibility study, the details of which are presented in Appendix B. Plunge pool inserts were molded in fiberglass and installed in receiving boxes filled with aquarium gravel, which provided capability to adjust plunge pool location. Outfall flow for Range D was provided through a pump and hopper set-up, not by opening the sluice chute as during testing in 1999. The flow depth and velocity set at the outfall exit were based on full water surface profile calculations for the planned conveyance channels to the outfall location, not on approximations based on normal depth calculations as used for previous tests. Thus, the simulated flow conditions were more accurate during the integration tests than previous model tests.

6.2.2 Flow Scenarios and Model Test Runs

Flow scenarios were tested for various project operations under the following three total river flows: Low = 115 kcfs, 80 percent exceedence; Medium = 215 kcfs, 50 percent exceedence; and, High = 300 kcfs, 25 percent exceedence. Individual flow scenarios were developed by the Fisheries Agencies and the Design Team. They established river flows and project operations, developed possible scenarios, and picked the top priorities (Table 6-1). The critical scenarios were for the low and medium flows because plume dynamics were generally more problematic than at high flows. (Note also that the scenarios were modified during testing, e.g., Scenario 1A indicated a different combination of B2 units operating for a given B2 flow than the base scenario. See Table 6-8 on page 105 for a list of B2 operating units associated with each flow scenario.)

Table 6-1 Flow Scenarios for Outfall Range/Type Combinations

Flow Scenario	Spill *(kcfs)*	B2 PH *(kcfs)*	B2 Outfall *(kcfs)*[A]	B1 PH *(kcfs)*	River Flow *(kcfs)*	% Exceed	TWL *(ft)*
1	50	60	5	0	115	75	11.5
2	75	35	5	0	115	75	11.5
3	50	35	5	125	215	50	18.4
4	50	140	5	20	215	50	18.4
5	75	135	5	0	215	50	18.4
6	120	90	5	0	215	50	18.4
7	0	110	5	0	115	75	11.5
8	0	140	5	70	215	50	18.4
9	150	60	5	0	215	50	18.4
10	50	140	5	105	300	25	23.6
11	75	140	5	80	300	25	23.6
12	120	55	5	120	300	25	23.6
13	120	140	5	35	300	25	23.6
14	150	35	5	110	300	25	23.6
15	150	145	5	0	300	25	23.6

(A) Actual flows in the 1:100 scale model may have been lower in previous model tests for this study than the values reported here.

A model test run was an outfall range/type combination set up in the 1:100 model and tested at a particular flow scenario. Outfall flow was scaled using depth and velocity from conveyance calculations. The values used for the range/outfall type combinations are presented in Table 6-2. (These calculations assumed a Manning's 'n' value for concrete of 0.012. Later, during development of the 90 percent Design Documentation Report (DDR), the sensitivity of flow depth to 'n' values ranging up to 0.020 was analyzed. It was determined that for 'n' value of 0.015 a weak hydraulic jump will occur in the conveyance channel and flow depths and velocities will change accordingly.) During a given model test run, multiple dye releases were made to document plume dynamics. A 'slug-type' injection provided a consistent volume of dyed outfall water. The number of injections was increased when plume dynamics appeared to vary with time in order to capture a representative range of plume performance. Outfall flow level and invert elevation were routinely checked during modeling.

Table 6-2 Flow Depths and Velocities for Outfall Range/Type Combinations

Outfall Range/Type Combination	Outfall Invert Elevation *(ft)*	Flow Depth *(ft)*	Flow Velocity *(ft/sec)*
D/MLC	16	7.7	46.3
D/AC	7	6.7	52.8
F/MLC	16	13.4	26.5
F/AC	7	9.0	39.6

6.2.3 Plume Documentation and Rating System

Each model test run was rated and documented in writing and on video. A rating or score was assigned at the conclusion of each run then, in most instances, verified later by reviewing the videotape. Potential predator habitat was defined as zones within 100 feet of either bank line or areas with large back eddies and/or coves or irregularities in the bank lines, because predators have a tendency to stage near structures where water velocity is relatively low; shorelines act as structures. Shorelines from the outfall locations to approximately two miles downstream were examined. Note that, although these ratings simply reflect outfall plume dynamics as revealed by dye in a hydraulic model and not probability of smolt survival, they are the best data available to assess outfall range/type combinations for the B2CC smolt bypass. Dye plume excursion into areas of potential predator habitat was estimated visually using guide markings on the model bed as a guide. Multiple (3 or more) dye releases were made to establish repeatability of results. Ratings were made by Gary Johnson of Bionalysts, Chick Sweeney of ENSR and Karen Kuhn of CENWP-HD and a composite rating was developed. The rating system used was a scale of '1' to '5' as follows:

> '1' – *Excellent* -- Five percent or less (≤ 5%) of the outfall plume entered any areas of potential predator habitat or became stalled in any lower velocity zones.
>
> '2' – *Good* -- Only 5-15 percent (> 5 and ≤ 15%) of the outfall plume entered areas of potential predator habitat or became stalled in lower velocity zones.
>
> '3' – *Satisfactory* -- Between 15 and 25 percent (> 15 and ≤ 25%) of the outfall plume entered areas of potential predator habitat or became stalled in lower velocity zones.

'4' – *Poor* -- Between 25 and 40 percent (> 25 and ≤ 40%) of the outfall plume entered areas of potential predator habitat or became stalled in lower velocity zones.

'5' – *Unacceptable* -- More than 40 percent (> 40%)of the outfall plume entered areas of potential predator habitat or became stalled in lower velocity zones.

6.3 Phase 1 – Initial Investigation of Combinations – ERDC March 2001

6.3.1 Purpose

The purpose of this trip during March 12-20, 2001 was to compare outfall plume dynamics for the four 'finalists' of outfall range/type combinations: Range D/MLC, Range D/AC, Range F/MLC, and Range F/AC. The intent was to rank the combinations. The trip report in Appendix A (ERDC Trip No. 4) contains details of this Phase 1 work.

6.3.2 Methods

In addition to the methods described in Section 6.2, the rating data were analyzed to produce 'final scores' for the purpose of evaluating the range/type combinations. The raw data for this analysis were the consensus ratings for each run. Each range/type combination was analyzed separately as follows. For a given flow scenario, the ratings from the runs were averaged to produce one rating for each scenario. For a given total river flow (115, 215, or 300 kcfs), the averages for each flow scenario were then themselves averaged to produce one rating for each flow scenario, called the 'composite' rating. Thus, at this point in the analysis composite ratings for each range/type combination for each total river flow were developed.

The composite ratings were then tabulated and the scores for each flow scenario weighted. Performance, i.e., composite ratings, under the low (115 kcfs) and medium (215 kcfs) flows was assumed to be more important than the higher flow because egress conditions are presumably worse at lower flows. Therefore, the weighting factors were 0.4 for low and medium flows and 0.2 for the high flow. The composite ratings were multiplied by the weighting factors to produce 'weighted ratings' for each total river flow for each outfall range/type. Finally, the three weighted ratings for each outfall range/type were averaged to produce the 'final score'.

6.3.3 Results

During the March 2001 ERDC trip, 105 'runs' were made on the 1:100 scale model. Early in the trip it was evident that the Range D/AC combination was unworkable because the bulk of the outfall plume nearly always moved onto the Washington shore. Therefore, this combination was dropped from further investigation. The ratings for the Range F combinations were 'satisfactory' primarily because of relatively poor performance during the low flow scenarios (Tables 6-3 to 6-5). Examination of the other three range/type combinations showed that Range F/MLC and Range F/AC were clearly favorable to Range D/MLC (Table 6-6).

Table 6-3 Results for Range D/Mid-Level Cantilever

Total River Q	Scenario	No. Runs	Min.	Max.	Ave.	Composite
Low=115 kcfs	1A	2	4	5	4.5	
	2	1	5	5	5.0	4.2
	7	2	2	2	3.0	
Medium=215 kcfs	5	1	2	2	2.0	
	6	3	3	5	4.7	3.7
	8	1	2	2	3.0	
	9A	1	5	5	5.0	
High=300 kcfs	11	1	5	5	5.0	
	13	2	2	4	3.5	4.2
	15	1	4	4	4.0	

Table 6-4 Results for Range F/Mid-Level Cantilever

Total River Q	Scenario	No. Runs	Min.	Max.	Ave.	Composite
Low=115 kcfs	1A	3	3	5	4.3	
	2	3	2	4	4.3	4.4
	7	2	4	5	4.5	
Medium=215 kcfs	5	2	1	1	1.0	
	6	2	2	2	2.0	2.6
	8	2	3	4	3.5	
	9A	2	3	4	4.0	
High=300 kcfs	11	2	1	1	1.0	
	13	2	1	1	1.0	1.3
	15	1	1	1	2.0	

Table 6-5 Results for Range F/Adjustable Cantilever

Total River Q	Scenario	No. Runs	Min.	Max.	Ave.	Composite
Low=115 kcfs	1A	1	3	3	4.0	
	2	5	3	5	4.8	4.4
	7	4	4	5	4.5	
Medium=215 kcfs	5	1	1	1	1.0	
	6	1	2	2	3.0	2.8
	8	1	3	3	3.0	
	9A	1	3	3	4.0	
High=300 kcfs	11	1	1	1	1.0	
	13	1	1	1	1.0	1.0
	15	1	1	1	1.0	

Table 6-6 Summary of Outfall Range/Type Combinations
(during ERDC trip March 2001)

Range	Outfall Type	Total River Q (kcfs)	Composite Rating	Weighting	Weighted Rating	Final Score
D	Mid-Level	115	4.2	0.4	1.7	
		215	3.7	0.4	1.5	4.0
		300	4.2	0.2	0.8	
F	Mid-Level	115	4.4	0.4	1.8	
		215	2.6	0.4	1.1	3.1
		300	1.3	0.2	0.3	
F	Adjustable	115	4.4	0.4	1.8	
		215	2.8	0.4	1.1	3.1
		300	1.0	0.2	0.2	

For several runs, some range/type combinations were reoriented and tested under different scenarios. The purpose of reorientation was to refine or improve plume travel to reduce contact with tailrace shoreline. Reorienting the outfall structures sometimes resulted in a little improvement, but the improvements were not considered significant enough to change the run's rating. When the final range/type combination is selected for further design, its orientation should be studied and refined to maximize its performance (see Phase 2, refinement, in Section 6.4).

In another attempt to improve plume dynamics, outfall discharge was lowered from 5,300 cfs to 2,600 cfs. The intent was to reduce the amount of dye plume intersecting shorelines. Surprisingly, this decrease in discharge resulted in minimal improvement on plume performance. Accordingly, an outfall discharge of 5,300 cfs continues to be the design standard.

Lastly, an important result of the Phase 1 ERDC trip in March 2001 was preliminary investigation of a modified Range F called 'F-Tip' in combination with a Mid-Level Cantilever. Range F was modified by moving the outfall to the downstream tip of Cascades Island instead of off the B2 tailrace side of the end of the island so that sheet pile might be used to support the outfall instead of drilled shafts. (See Section 6.4 for a detailed explanation of F-Tip.) For special tests at 'F-Tip,' a quasi-plunge pool was excavated in aquarium gravel because the plunge pool receiving box in the model did not allow the plunge pool insert to be positioned correctly. Also, Cascades Island tip bathymetry did not allow the outfall to be placed at the precise location/orientation desired for F-Tip tests during the

March 2001 trip. The preliminary results for plume dynamics from F-Tip in Phase 1were encouraging and were not extensively tested until Phase 2 (Section 6.4).

6.3.4 Conclusions and Recommendations

Range F/MLC and Range F/AC had similar plume egress performance and both had noticeably better plume egress than Range D/MLC or Range D/AC. Range F/MLC, not Range F/AC, was recommended for further study because of concerns about the technical feasibility of such a large movable structure, which would also be exposed to substantial lateral loading from spillway flows. The AC structure also would have the potential for very high operation and maintenance costs plus environmental concerns associated with lubrication of the movable structure bearing.

In addition, it was recommended that a modified Range F called F-Tip be investigated in Phase 2.

6.4 Phase 2 – Fine-Tuning – ERDC May 2001

6.4.1 Purpose

The purpose of this trip May 1-3, 2001 was to fine-tune the Mid-Level Cantilever at Range F. Specific objectives were to (1) compare performance of a Mid-Level Cantilever at the original Range F to new F-Tip site, and (2) compare different MLC configurations at the chosen site, including orientation (azimuth) and distance from shore. The trip report in Appendix A (ERDC Trip No. 5) contains details of the Phase 2 work.
The F-Tip modification (Plate 5) was based on the idea that the entire conveyance channel could be built at grade on the north side of Cascades Island, rather than constructing the conveyance channel on drilled shafts over water from the north shore of Cascades Island to Range F. The west (downstream) tip of the island would be extended in a triangular shaped sheet pile enclosure, with the sides of the triangle parallel to the north and spillway channel shorelines of the island. The outfall would extend from the apex of the triangle and the plunge pool would be dredged in the intersection between the exit of the spillway tailrace channel and the B2 tailrace channel. The construction methods described would result in cost savings as described in Section 8 of this report.

6.4.2 Methods

Two changes for Phase 2 work were made to the 1:100 scale general model of Bonneville Dam. First, bathymetry at the tip of Cascades Island was lowered (the tip of the island was removed down to El. – 20 feet) to allow easier adjustment of the outfall azimuth and length. The shoreline of the island tip was simulated with bricks and sand bags. Lastly, the plunge pool receiving box was enlarged to allow plunge pool installation at the appropriate location for the F-Tip site.

Water levels and velocities were set at the exit of the outfall structure using the Range F/MLC values from Table 6-2. The difference in total conveyance channel length to the F-Tip location as compared to the original F location would not cause a noticeable change in the flow depth.

To compare plume performances for F-Tip and original F, real-time Phase 2 model runs of F-Tip were examined in conjunction with videotape of Phase 1 runs of original F. Thus, the comparison design team members made was essentially 'side-by-side'.

To fine-tune the chosen outfall range/type combination, eleven flow scenarios that emphasized B2 as the priority powerhouse (2, 4, 5, 6, 7, 6, 8, 9A, 11, 13, and 15) were run at 0 degrees rotation relative to the longitudinal axis of the outfall in Plate 5. This provided comparable data to those already collected for other ranges and outfall structures. Rotations of 5 and 10 degrees counterclockwise (CCW) rotation were also tested to examine the effect of changes in outfall orientation. For the 5-degree rotation, the only scenarios tested were those for which there had been a rating difference for the 0 and 10-degree CCW rotation cases, or for which the plume had impinged on both banks and therefore the performance rating might be affected by small incremental changes in plume trajectory.

6.4.3 Results

In 8 of 11 flow scenarios, plumes from the F-Tip location had slightly less intrusion on or near shorelines than those from the original F location. The differences in plume dynamics between F-Tip and original F were not dramatic but they were noticeable. Therefore, F-Tip with a Mid-Level Cantilever was the preferred outfall range/type combination. The next step was to fine-tune outfall configuration.

The F-Tip/MLC outfall combination was fine-tuned by rotating the outfall axis and shortening the outfall. Rotation of 10-degree CCW gave comparable results to the 0 degree case with the exceptions of Scenarios 6, 8, and 9A (Table 6-7). Overall, the 0-degree rotation was slightly better than the 10-degree rotation (average rating 2.46 and 2.55, respectively, Table 6-7). As expected, a 5-degree rotation CCW did not improve plume dynamics. Rotation of the outfall counterclockwise seemed to improve performance for scenarios with heavy spill, but at the expense of the low or no spill scenarios. Reducing the outfall length 100 feet, provided no apparent advantage in plume performance. In addition, shortening the outfall would require modification of the island shoreline, which could offset any cost savings associated with outfall shortening.

Table 6-7 Ratings and Areas for Outfall Rotations

Scenario	0 Degrees		10 Degrees CCW	
	Rating	Plume Intrusion	Rating	Plume Intrusion
2	4	WA	4	WA
4	1	WA	1	Bradford
5	1	WA	1	Bradford
6	3	Both	2	Both
7	5	Bradford	5	Bradford
8	4	Bradford	5	Bradford
9A	3	WA	4	WA
10	2	WA	2	Both
11	1	None	1	WA
13	1	WA	1	WA
15	2	Both	2	Both
Average	2.46	---	2.55	---

Counterclockwise (CCW) rotation of the outfall axis for the F-Tip/MLC configuration.
Rotation is relative to the longitudinal axis of the outfall shown in Plate 5.

6.4.4 Conclusions and Recommendations

In conclusion, the F-Tip site had better plume egress than the original Range F because slightly less or comparable plume encroachment occurred within 100 feet of the shorelines for eight of the 11 scenarios tested. Fine-tuning by rotating and shortening the outfall did not improve plume performance. Therefore, the 'best' outfall range/type combination to carry forward for review by the Fisheries Agencies was a Mid-Level Cantilever at F-Tip with no rotation or shortening (Plate 5).

6.5 Phase 3 – Fishery Agency Review –June 2001

6.5.1 Purpose

During Phase 3, representatives of National Marine Fisheries Service (NMFS) and the Columbia River Intertribal Fisheries Commission (CRITFC) joined the Design Team at the 1:100 scale general model at ERDC to observe performance of the preferred outfall range/type configuration, a Mid-Level Cantilever Outfall at the F-Tip location. The objectives were to (1) obtain consensus whether the preferred combination would provide acceptable plume performance, and (2) determine if the presence of outfall structures at the tip of Cascades Island might have a deleterious influence on juvenile fish approaching that area from the spillway. Specific methods employed, results, and conclusions and recommendations are summarized in the following sections.

6.5.2 Methods

The trip objectives were addressed with tests of plume performance, outfall structure effects, and dam operations effects. Plume performance tests used the same methods described in Section 6.2. In addition, the bathymetry of tip of Cascades Island that was removed to facilitate movement of the outfall during Phase 2 refinement work was re-installed in the model. The plunge pool was positioned for the F-Tip location and the outfall was installed in a sheetpile enclosure (Plate 5). Tests were performed for flow scenarios 1-15 (Table 6-1).

Tests of the effects of the outfall structure on the spill plume were performed with and without the outfall structure. Also tests were undertaken with the outfall structure in conjunction with a training wall that guided flow from the spillway tailrace along the bank line toward the outfall. The configuration of the training wall, illustrated in Plate 5, was developed by ERDC prior to the demonstration tests through an iterative test procedure. Wall configurations were placed in the model, dye was observed, and the wall adjusted until the eddy, which formed adjacent to the spillway side of the outfall, was essentially eliminated. The wall configuration proved to be independent of the flow scenario. The effect of the outfall structure on potential egress of juvenile fish passing through the spillway was evaluated by observing the area extent of the eddy, which formed at the tip of the island and the rate of egress of flow from the spillway tailrace. The outfall effect tests were performed for flow scenarios 4, 5, and 6 (Table 6-1).

6.5.3 Results

Plume performance results for all fifteen test scenarios, which are presented in Table 6-8, Figures 6-1 and 6-2, and Photographs 6-1 through 6-15, (Appendix A, Trip Number 6 – 1:100 Scale Model) show the variability in plume dynamics depending on dam operations. Plume performance for the low, 155 kcfs, river flows appears to be good when three B2 units are operated. For total river flows of 215 kcfs, plume performance is excellent if B2 is fully loaded and spill is 55 kcfs or more. For tests with total river flows of 300 kcfs, if B2 is fully loaded and spill flows are 55 kcfs or more, plume performance is good.

Table 6-8 Plume performance for Mid-Level Cantilever outfall at F-Tip location.

Flow Scenario No.	B2 Units Operating	Flow *(kcfs)*					Dye % within 100 ft of Shore				Rating[A]
		B2	Outfall	Spill	B1	Total River	BIS	WA U/S	WA D/S	Total	
1A	11,12,18	51	5.3	55	3.8	115	0	1	10	11	2
2	11,18	34	5.3	72	3.8	115	0	2	23	25	3
3	11,18	35	5.3	55	120	215	0	5	23	28	4
4	11 thru 18	140	5.3	55	15	215	0	0	0	0	1
5	11 thru 18	127	5.3	79	3.8	215	0	0	0	0	1
6	11,12,13,17,18	82	5.3	124	3.8	215	0	1	20	21	3
7	11,12,13,14,15,17, 18	100	5.3	4.9	3.8	114	36	0	0	36	4
8	11 thru 18	140	5.3	4.9	65	215	50	0	0	50	5
9A	11,12,18	51	5.3	155	3.8	215	0	1	23	24	3
10	11 thru 18	140	5.3	55	100	300	0	0	3	3	1
11	11 thru 18	140	5.3	79	76	300	8	0	7	14	2
12A	11,12,18	54	5.3	124	117	300	0	3	40	43	5
13	11 thru 18	140	5.3	124	31	300	5	0	0	5	1
14	11,18	35	5.3	155	105	300	0	4	47	51	5
15	11 thru 18	136	5.3	155	3.8	300	0	0	20	20	2

[A] Ratings are indicative of plume performance, but are based on observations by one member of the design team, rather than the consensus model used for previous tests.

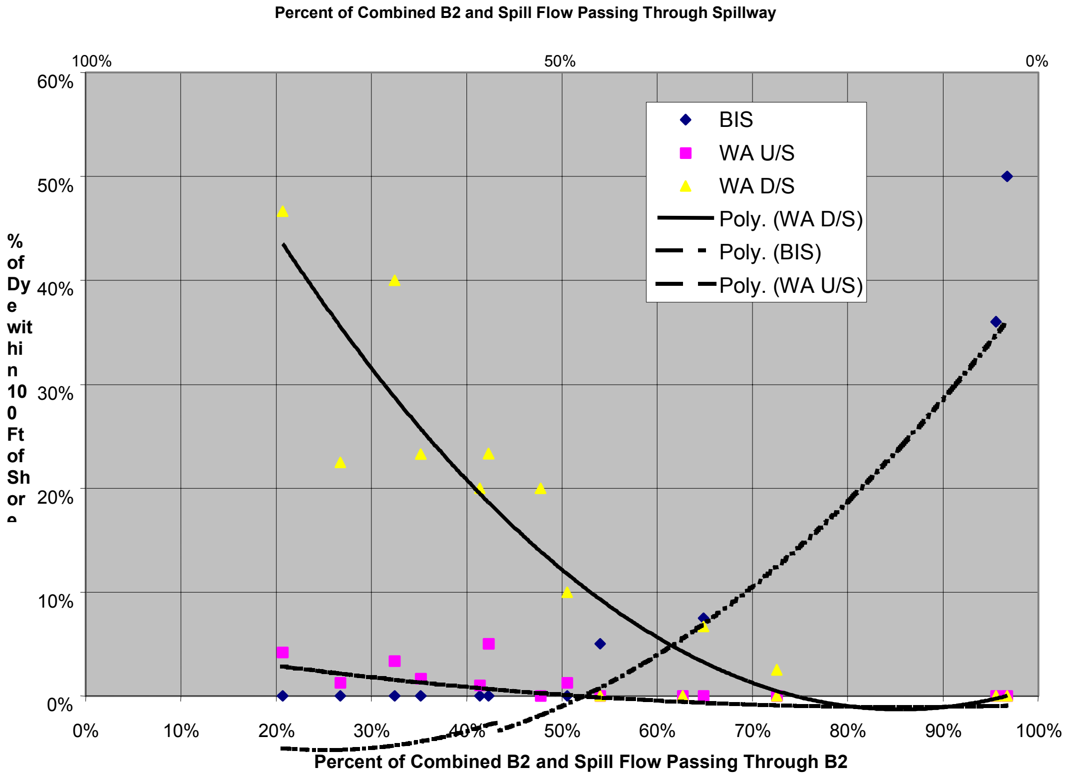


Figure 6-1 Percentages of Dye Intrusion

Percentages of dye intrusion (within 100 feet) at Bradford Is. (BIS), the Washington shore upstream of the cove (WA U/S), and Washington shore downstream of the cove (WA D/S) for varying distributions of the combined B2 and spill flow. Derived from tests of the MLC at F-Tip during the June 2001 ERDC trip.

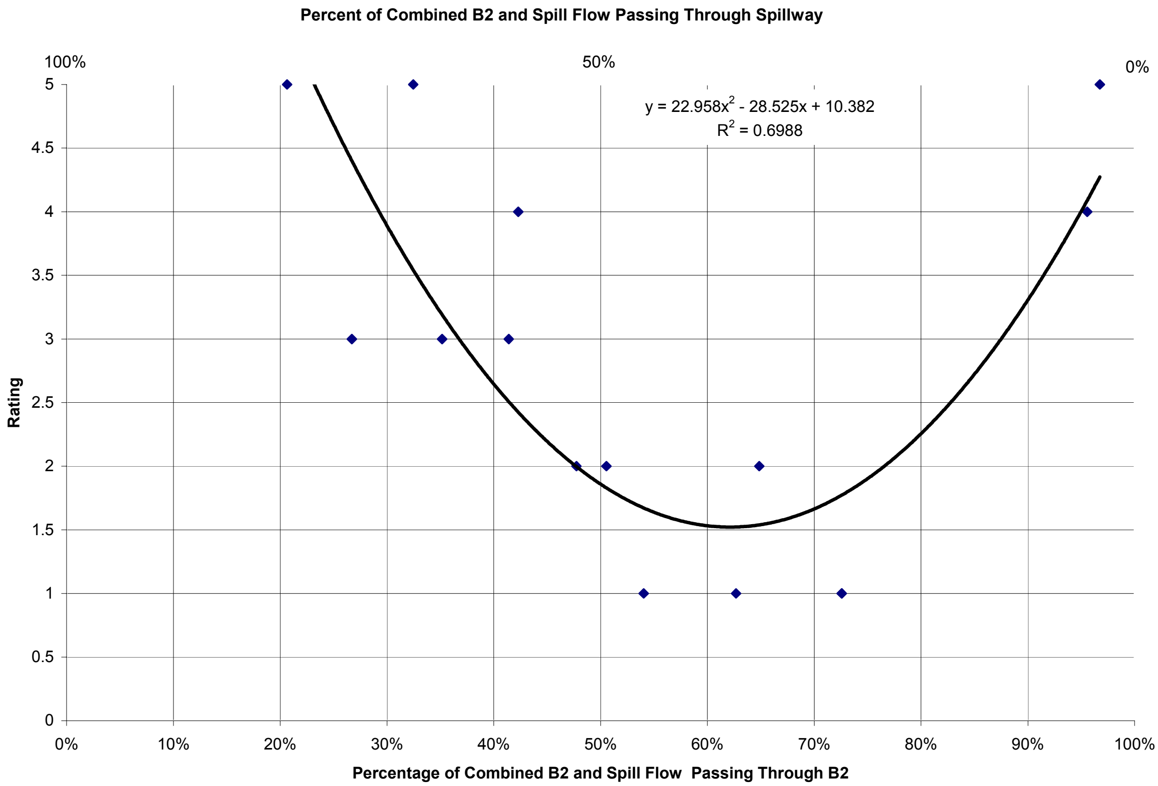


Figure 6-2 Egress Rating vs. B2 Flow

The relationship between egress rating varying distributions of the combined spillway and B2 flow. Derived from tests of the MLC at F-Tip during the June 2001 ERDC trip.

The results of tests of outfall structure effects showed that the eddy at the downstream tip of Cascades Island did not extend as far downstream without the outfall present, about two-thirds as long, but was wider toward the Washington shore side of the tip. The overall area extent of the eddy appeared to be about the same. Addition of the training wall eliminated the eddy, but velocities were low adjacent to the wall and injected dye was retained in this area for extended periods of time, though no longer than without the outfall. The training wall was eliminated from consideration because it did not appear to improve conditions relative to predator holding potential. Contribution of the outfall to localized flow separation and eddying at the downstream tip of Cascades Island was quite small and barely discernible.

Based on the results in Table 6-8, operation of three or more B2 units with 55 kcfs spill seemed to achieve acceptable outfall plume dynamics, if two of the operating units are at the WA side of the powerhouse. For 124 kcfs spill, operation of five or more B2 units appeared to achieve acceptable outfall plume dynamics. Finally, for full B2 load, spill of 30 kcfs seemingly is required to achieve acceptable outfall plume dynamics. The issue of B2 and spill flows for optimal plume performance was also examined using the split between B2 and spill flow.

The relationship between both dye plume incursion near shore and egress rating versus the B2 to spill flow split shows that optimum egress (rating = 1) occurred when approximately 62 percent of the combined B2 and spill flow is routed through B2 and 38 percent through the spillway. Ratings were poor when the flow through B2 was less than 48 percent (spill flow more than 52 percent) and conversely when B2 flow was more than 72 percent (spill flow less than 28 percent) (Figure 6-2). This suggests the need for a better understanding of minimum B2 flows, in conjunction with spill, for acceptable plume dynamics. Subsequent studies will need to identify specific B2 and flow distributions required to operate the B2CC.

6.5.4 Conclusions and Recommendations

The Fisheries Agencies and the Design Team agreed that the proposed F-Tip/Mid-Level Cantilever outfall is the optimum outfall site/type combination for the B2 tailrace. There may be, however, conditions when river flows are low with no or little spill and/or if B1 is the priority powerhouse where B2 high flow outfall plume performance might be less than optimum. In these instances, the outfall may have to be shut down, or at the least, fish survival should be evaluated.

6.6 Conclusion from Integration of Outfall Types and Ranges/Sites

The F-Tip site with Mid-Level Cantilever type outfall is the optimum outfall range/type combination for the B2 tailrace. It should be carried forward to the next design phase for ultimate construction.

7 COST ESTIMATES

An interim cost estimate was prepared based on revised alignments and information developed during the initial DDR effort for this project. This work was performed by the Portland District and the INCA team.

The costs are conceptual in nature and are most useful for comparing alternative designs. Some of the alternatives have changed since the last cost estimate was issued. These changes are based on more complete design and site information. Also, contingencies have been applied to individual cost items rather than the overall project. This has resulted in some overall project reductions.

Five Alternatives have been estimated:

- Mid-level Cantilever at Range D
- Mid-level Cantilever at Range F
- Adjustable Cantilever at Range F
- Mid-level Cantilever at Range F at the tip of Cascade Island
- Adjustable Cantilever at Range F at the tip of Cascade Island

The construction cost for the Mid-level Cantilever at Range D increased approximately $4.5M. This is due to more detailed structural design information by the Portland District for the over water channel. These cost increases also apply to the over water structures at the F location. The plunge pool size was increased to 50 feet based on the recent 1:30 model testing and the required reduction in bottom strike velocities.

The cost for the Mid-level Cantilever at Range F has decreased substantially. This is due to a number of factors. The cost of the plunge pool is lower based on information from the test program performed recently, which indicates that the gravel materials in that area will stand on 2:1 slopes without the use of structural walls installed underwater. Also, the alignment is shorter due to utilization of the I&T sluiceway for a portion of the alignment across the tailrace deck, and locating the outfall to the north side of Cascades Island. The previous assumption was that the new channel would exit the powerhouse just downstream of the entrance gate. This is very close to the cutoff wall at the forebay, and resulted in larger costs for the Range F. The current hydraulic design utilizes the existing 45 degree bend in the I&T sluiceway. Recent work by the Portland District also indicates that less earthwork will be required for construction of the channel at on grade portions. The estimate was decreased approximately $26M. Verification of these design changes needs to be accomplished during the DDR effort to confirm these cost savings. One concern is construction over water at the lower tailrace elevation required at the F location.

The cost for the Adjustable Cantilever at Range F decreased by approximately $21M. This decrease was less than that for the Mid-level since no plunge pool was involved.

Previous estimates for the outfalls at the tip of Cascades Island were not done. The costs when compared to the F location are about $8M less. This is due to the elimination of overwater construction involving drilled shafts. The F Tip alternative utilizes sheetpile walls, which are significantly less expensive even though the total length of channel is somewhat longer.

Below is a summary for costs for the five alternatives studied. Additional cost information is included in the Cost Appendix.

Table 7-1 Cost Summary

Cost Item	D Mid-level	F Mid-level	F Adjustable	F Tip Mid-level	F Tip Adjustable
Gate Modifications	$ 900,000	$ 900,000	$ 900,000	$ 900,000	$ 900,000
ITS Modifications	$ 300,000	$ 300,000	$ 300,000	$ 300,000	$ 300,000
Sheetpile Infill @ ITS	$ -	$ 625,000	$ 625,000	$ 625,000	$ 625,000
Sheetpile @ Island Tip	$ -	$ -	$ -	$ 4,750,000	$ 4,750,000
Elevated Channel	$ 8,580,000	$ 17,374,500	$ 15,147,000	$ -	$ -
At Grade Channel	$ -	$ 9,900,000	$ 9,900,000	$ 13,500,000	$ 13,500,000
Plunge Pool	$ 14,820,000	$ 1,892,800	$ -	$ 1,892,800	$ -
Adjustable Substructure	$ -	$ -	$ 1,950,000	$ -	$ 500,500
Adjustable Channel	$ -	$ -	$ 994,500	$ -	$ 845,000
Adjustable Mechanical	$ -	$ -	$ 845,000	$ -	$ 845,000
Subtotal	$ 24,600,000	$ 30,992,300	$ 30,661,500	$ 21,967,800	$ 22,265,500
Profit and Overhead	$ 6,150,000	$ 7,748,075	$ 7,665,375	$ 5,491,950	$ 5,566,375
Total Construction Cost	**$ 30,750,000**	**$ 38,740,375**	**$ 38,326,875**	**$ 27,459,750**	**$ 27,831,875**
E & D*	$ 5,962,000	$ 7,606,000	$ 7,606,000	$ 5,880,000	$ 5,880,000
S & A	$ 1,584,000	$ 2,013,000	$ 2,013,000	$ 1,419,000	$ 1,419,000
Total Project Cost	**$ 38,296,000**	**$ 48,359,375**	**$ 47,945,875**	**$ 34,758,750**	**$ 35,130,875**
* Includes DDR, P&S, EDC, and 10% Contingency					

8 CONCLUSIONS AND RECOMMENDATIONS

The F-Tip site with Mid-Level Cantilever type outfall is the optimum outfall range/type combination for the B2 tailrace. It should be carried forward to the next design phase for ultimate construction.

Appendix A

Hydraulic Modeling Site Visit Memorandums

Trip Number 1 – 1:100 Scale Model

Trip Number 2 – 1:100 Scale Model

Trip Number 3 – 1:30 Scale Model

Trip Number 4 – 1:100 Scale Model

Trip Number 5 – 1:100 Scale Model

Trip Number 6 – 1:100 Scale Model

Photo 1A

Photo 1B   Plume for Scenario # 1A

Photo 2A

Photo 2B Plume for Scenario # 2

Photo 3A

Photo 3B Plume for Scenario # 3

Photo 4A

Photo 4B Plume for Scenario # 4

Photo 5A

Photo 5B **Plume for Scenario # 5**

Photo 6A

Photo 6B Plume for Scenario # 6

Photo 7A

Photo 7B Plume for Scenario # 7

Photo 8A

Photo 8B Plume for Scenario # 8

Photo 9A

Photo 9B Plume for Scenario # 9A

Photo 10A

Photo 10B Plume for Scenario # 10

Photo 11A

Photo 11B Plume for Scenario # 11

Photo 12A

Photo 12B   Plume for Scenario # 12A

Photo 13A

Photo 13B Plume for Scenario # 13

Photo 14A

Photo 14B Plume for Scenario # 14

Photo 8-15A

Photo 14B   Plume for Scenario # 14

Appendix B

F-Tip Routing Study

Introduction

During the Site Selection Study work concentrated on Ranges D and E. This work was done in case Range F was selected. It was intended to highlight issues that would be dealt with in the construction of a channel to Range F. Based on earlier work done, this work assumed that the elimination of the 45 degree bend in the upstream end of the ITS would be eliminated, and the conduit would emerge from the end of the powerhouse.

Utility conflicts are highlighted and issues involving the required deep excavation very close to the cutoff wall considered. The construction next to the cutoff wall was considered to be very difficult and problematic. When the Portland District staff began design work for the DDR, they elected to pursue a design which utilized the upper portion of the ITS and emerged from the ITS much farther downstream. Further hydraulic design was able to correct the flow issues involved with keeping the 45 degree bend in the ITS.

Outfall Channel

The outfall channel is a buried concrete box, 15-feet wide by 20-feet high (inside dimensions) that begins at Sta. 0+55 and continues to Sta. 1+50. Upstream of Sta. 0+55 the channel is inside the Ice and Trash Sluiceway (ITS), at Sta. 0+55 the channel breaks through the Service Bay wall. The upstream end of the channel is at invert El. 29.0 and slopes down at 0.0042 ft/ft. At Sta. 1+50 the buried channel transitions to an open channel. A Composite Utility Plan for the upstream end of the channel is shown on Plate 1. A sectional view is shown on Plate 2.

Intercepted Utilities

There are a number of utilities that are intercepted by the outfall channel, including electrical, sanitary sewer, communication, and water. The Composite Utility Plan, Plate 1, was developed using record drawings and field inspection. A number of the drawings were lacking horizontal and vertical control for the features, so the utilities were scaled off and appropriate elevations assumed. The beginning of the channel, Sta. 0+00, is consistent with the previous studies, the downstream edge of the ITS gate slot. Station location of the utility conflicts were established by scaling off the composite utility plan.

Most of the utilities can be maintained in service during construction and accommodated in the final layout. The utility conflicts have been addresses in order of their appearance on the profile.

Electrical

There are four conflicts between the outfall channel and the electrical utility. The first is a luminaire and buried supply conduit at Sta. 0+70. This luminaire will be removed during construction and could be replaced after final grading.

The second conflict is the most problematic of all utility conflicts. This is the conflict between the 3x3 electrical duct bank that currently contains the powerhouse intertie and the channel, at Sta. 1+50. Construction impact on this duct bank was the subject of much discussion during the Bonneville First Powerhouse JBS Improvements project. This duct bank is shown as being 3 feet-4 inches square with nine 4-inch diameter ducts arranged in three columns of three rows. The record drawing shows a telephone and communication duct bank located adjacent to the electrical duct bank. There are two ducts in this bank and it is located either above or alongside the electrical duct bank, the drawing indicated that either location was acceptable. The powerhouse intertie exits CV2-1 at El. 33 (approximate) and enters CV2-2 at El. 72 (approximate). This results in the duct bank falling approximately mid-channel at the location where the buried channel becomes the open channel at-grade. It may be possible to provide a temporary routing that maintains service between the two powerhouses. It may also be possible to permanently relocate the intertie as an aerial span over the at-grade channel. Alternately, the conductors could cross the channel in a closed conduit, similar to a pipe bridge. Permanent relocation would require modification of the conductor in CV2-1 and possibly a new cable routing that enters PH2 at a higher elevation. On the left side of the channel it would be necessary to raise the grade to accommodate the new, higher, intertie and it would be necessary to install new duct bank from CV2-1 to CV2-2.

The third conflict is a crossing of the channel with a luminaire supply conduit at Sta. 2+40. This is a 3/4-inch diameter EMT with two 12 ga conductors and one 8 ga conductor. This is easily relocated for construction and for the permanent installation.

The last conflict is a luminaire, at Sta. 3+70, that is near enough to the channel that it will be disturbed during the construction. This luminaire can be re-set upon completion of the channel.

Sanitary Sewer

A sanitary sewer line crosses the outfall channel at Sta. 1+20. During construction the effluent could be pumped around the trench or temporarily collected in a reservoir and disposed of by a septic tank pumping contractor. It is anticipated that the line could be permanently reinstalled in the fill over the outfall channel.

Communication

A two duct bank crosses the outfall channel at Sta. 1+50. This duct bank was built adjacent to the electrical duct between CV2-1 and CV2-2, and is discussed with the electrical crossing above.

Water

At Sta. 2+00 there is an 8-inch water stub out to the powerhouse. During construction this water service could be maintained using a variety of different methods, depending on the water use. It is also possible that this stub out is a redundant connection to the water system and it could be inactivated for the duration of construction. As the outfall channel has become an open at-grade channel, there will be no soil cover to reinstall this line in. It is anticipated that this water service would be permanently re-established using a steel pipe bridge to cross the channel.

Affected Structures

There are a number of structures which are not in direct conflict with the location of the outfall channel but will be affected by the construction. These structures are shown on the Composite Utility Plan, Plate 1.

Cutoff Wall

Open cut excavation at the upstream end of the outfall channel will temporarily eliminate support for the cutoff wall, creating a condition where it also must function as a retaining wall. Record drawings indicate that the wall extends down to bedrock, approximate El. -30 feet, at this location. This creates a 30-foot high retaining wall out of an impermeable diaphragm. In addition to the soil loads there will be the combination of porewater pressure (this is a cutoff wall) and construction surcharge (due to the equipment used for excavation and underpinning the USM channel). The geometry of the excavation eliminates the possibility of using the wall of the excavation to brace the cutoff wall. It is anticipated that an extensive system of walers and tiebacks will be required to support the cutoff wall. This support will create its own set of problems, specifically the tiebacks will perforate the waterproof membrane thus destroying the function to save the geometry. Steps can be taken to mitigate this condition but it may be better to select an excavation method that avoids impacting the cutoff wall entirely. Tunneling the outfall channel from the powerhouse wall to Sta. 0+75 would eliminate the need to support the cutoff wall. Tunneling is addressed further in the discussion of the impact of the outfall channel on the USM channel, below.

USM Channel

As the outfall channel exits the ITS wall it passes under the USM Channel. At this location the crown of the outfall channel is approximately at El. 49 feet and the invert elevation of the USM Channel is approximately El. 61 feet. The outfall channel should not directly affect the USM Channel. There is a slight possibility that the ITS wall will require reinforcing around the penetration, particularly in the soffit of the outfall channel. If this reinforcement includes additional reinforcing steel or post-tensioning then the depth of embedment should be carefully considered. There is a slight plan overlap in the two channels and it would be possible to drill up through the soffit of one channel and into the invert of the other. It would take effort and desire to create a problem, it is not likely to happen by accident.

While physical interference between the two channels is not expected to be a problem, excavation for the outfall channel will significantly affect the USM Channel. Construction of the outfall channel may be accomplished by open cutting the entire alignment or tunneling the upstream end. Open cutting the upstream end creates extensive problems for both the USM channel and the cutoff wall (see discussion above). Open cutting the entire alignment will leave the USM Channel unsupported, requiring underpinning to preserve its structural integrity.

Tunneling through the ITS wall and under the USM Channel is an alternative. The record drawings of the cutoff wall show that the top of rock is at elevation approximately -30 feet. It is expected that the backfill material was an unclassified, but fairly clean and well graded, fill. It is also expected that the material was placed with some amount of compaction and moisture control. While this material is not suitable for driving a tunnel beneath or near an existing structure, it should be possible to improve this material to make it suitable. Ground improvement would consist of permeation grouting, using a cement based grout, installed by drilling horizontally through the south wall of the powerhouse. The grout 'columns' would be arranged in a semi-circle above the soffit of the future outfall channel excavation and below the USM channel base slab. The intent of the ground improvement is to develop arching capacity in the soil above and adjacent to the outfall tunnel which would then allow excavation of a soft ground tunnel. It is anticipated that the tunnel would extend a distance necessary to eliminate impacting the cutoff wall and the USM channel. This is approximately 50 feet. From this point on, downstream of Sta. 1+05, the channel would be constructed in an open cut excavation.

Access Road

The method of excavation for the outfall channel may significantly impact the access road to the Forebay Deck at El. 90 feet. This road continues north across the Forebay Deck and eventually connects to the Washington SR14. This road also continues south across Cascades Island, the Spillway, Bradford Island, and Bonneville 1st Powerhouse to eventually connect with Oregon SR84. If the excavation for the upstream portion of the outfall channel is open cut, then this access will be interrupted. Adjacent to the south end of the powerhouse, extending approximately 50 feet, there will be a benched excavation with benches at El. 26 and El. 60. Unless provisions are made for a temporary access bridge there will be no traffic access from the 2nd powerhouse to Cascades island. Additionally the gantry crane used to install the TIES will not be able to travel south of the powerhouse to retrieve the TIES from their storage location on Cascades Island.

If the upstream portion of the outfall channel is tunneled then there will be no appreciable impact on the road access or carne function.

Fire Rescue House

Currently there is a single story building located at Sta. 4+75 that will be impacted by construction of the open at-grade channel. This is the structure shown in Photo 1, below. Business use of this structure has been discontinued and it has recently been used for fire rescue training for local fire departments. The windows have been boarded up to make the house smoke tight. The fire departments place dummies in the structure, fill the structure with smoke, and then perform rescue training. Bonneville Operations staff have surplused the building and as of January of 2001 are soliciting agencies, organizations, and individuals to remove the building from the site.

Cost Estimate

The estimated cost for the utility conflicts are based on tunneling the outfall channel excavation from the powerhouse to approximately Sta. 1+05 and open cut excavation from that point downstream. If the entire alignment of the outfall channel is open cut excavation the cost to mitigate the utility conflicts will increase 100 to 200 percent (they will double or triple). Also if the upstream end of the outfall channel is open cut excavation then it will be necessary to add the costs for road mitigation, that is, installing a temporary access bridge.

The estimates costs for mitigating utility conflicts and affected structures are tabulated below. The costs have been broken out for each item discussed above, in dollars, for the temporary and permanent mitigation.

Utility/Item	Number	Temporary	Permanent	Total
Electrical	1	1,583	1,950	3,533
Electrical	2	18,379	35,068	53,446
Electrical	3	3,780	3,780	7,561
Electrical	4	1,725	1,430	3,155
Sanitary Sewer		16,000	15,756	31,756
Communication		2,217	16,782	18,998
Water		25,195	15,451	40,646
Cutoff Wall		0	0	0
USM Channel		130,000	19,500	149,500
Fire Rescue House		0	0	0
			Total	308,595

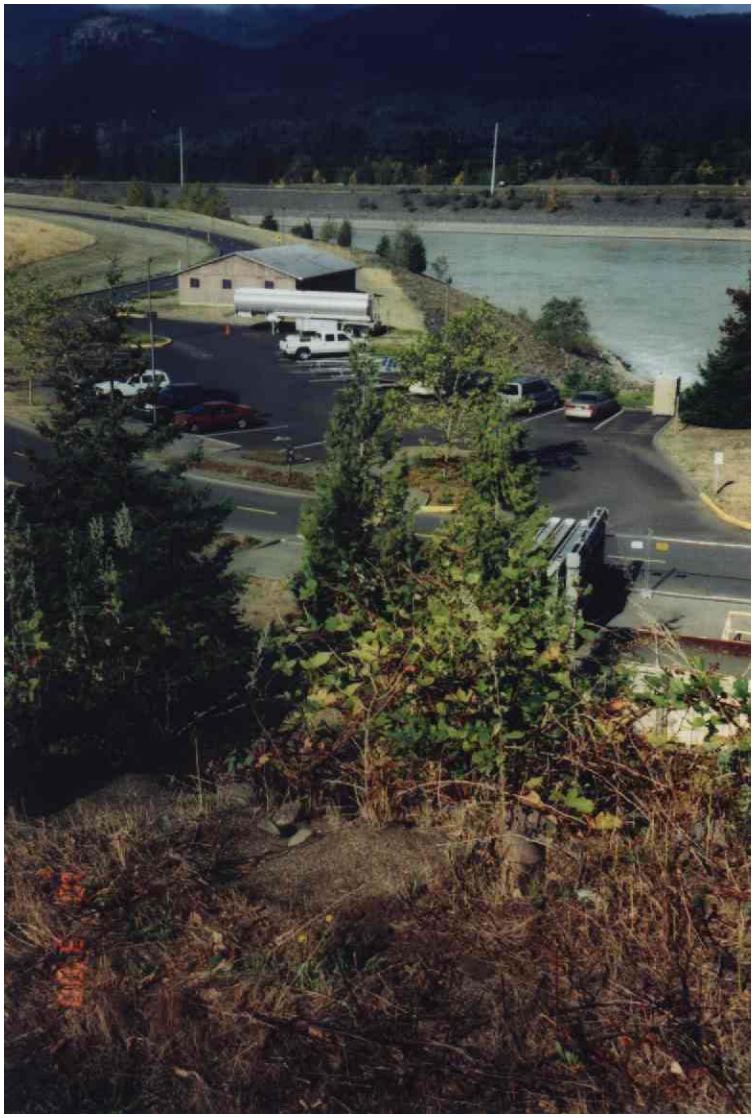


Photo B1 - Looking Southwest at Parking Lot and Fire Rescue House

Appendix C

Crane Access

Bonneville Dam Second Powerhouse
Corner Collector

Draft Evaluation of Crane Access

Objective

The objective of this letter report is to document the evaluation of crane access to the Bonneville Dam Second Powerhouse (B2) tailrace. There are a number of issues that have initiated this study including: size of cranes required to construct the outfall substructure and superstructure, size of cranes to install the B2 flow deflectors, clearance under the Bonneville First Powerhouse (B1) Flume Bridge, and timing of the various construction activities.

Crane Capacities

Corner Collector Construction - Outfall Substructure

It is anticipated that the equipment selected for the construction of the Flume Bridge substructure would also be appropriate for the construction of the Outfall substructure. The selected crane is a Manitowoc 888, which is a 230 ton capacity crawler crane (track mounted). This crane will be barge mounted and has a required clearance of approximately 20 feet, barge and crane. The crane will be de-rated depending on what size barge it is used on. At this time the final size of the substructure elements is not known. Construction of the substructure is possible, but the issue should be considered during design.

Corner Collector Construction - Outfall Superstructure

The estimated lift requirement for the superstructure is 500 tons for each end of the adjustable cantilever if it is constructed of concrete. This significantly exceeds the capacity of the any floating equipment available.

Two alternatives are possible. One alternative is to construct a temporary work platform rather than barge mounting a crawler crane. This opens up the possibility of using a ringer crane or a guy derrick crane. Manitowac Cranes, Inc. builds Ringer® attachments for their cranes with capacities up to 1433 tons. Terex American Crane Corporation builds guy derrick cranes with capacities up to 600 tons. Both the ringer and derrick cranes are variations of a crawler crane, which means they are transportable by truck. This in turn means that the required clearance is approximately 14 feet for the crane elements plus 6 feet for the barge, for a total of 20 feet.

Another alternative is a custom designed heavy lifting system, similar to the VSL Heavy Lift used to install the Winfield guide walls. The outfall substructure, drilled shafts, could be used to support the lifting mechanism. This is the most likely and cost effective method for erecting the adjustable cantilever.

The adjustable cantilever may also be constructed of steel, which would reduce the required lifts.

The fixed portion of the superstructure construction may be cast-in-place concrete or utilize some pre-cast components. For cast-in-place concrete, a barge mounted crawler crane would be adequate. If precast elements are used it is more likely that the contractor would build a work platform next to the structure.

Flow Deflector Installation

Mr. Lance Helwig, P.E., CENWP-EC-DS, provided information regarding the anticipated crane capacities needed for installing flow deflectors at B2. The two heavy lifts are the saddle frame lift, at 35 tons, and the bulkhead lift, at 70 tons. It is expected that it would be possible to locate a barge mounted crane adjacent to the lift and set location therefore a 125 ton barge mounted crane could handle the lifts. Lance suggested that the D. B. Pacific, with a 165 ton capacity, or the D. B. Columbia, with a 125 ton capacity, would be appropriate. Both of these cranes are floating cranes owned by General Construction, the construction contractor that built the B2 outfalls. The D. B. Pacific has a draft of 8 feet and a gantry height of 75 feet, measured from the deck; total height above water is 83 feet. The D. B. Columbia has a draft of 5 feet and a gantry height of 64 feet, measured from the deck; total height above water is 69 feet.

Mr. Greg Simmons, P.E. of General Construction, reported that they do have the capability of collapsing the gantry of the D. B. Columbia, producing a gantry height of 59 feet. This results in a total clearance requirement of 64 feet. He also reported that a standard track mounted crane working off a barge should fit under the complete Flume Bridge. The crane/barge combination would require load rating for the specific lift configuration to be accomplished.

The capacity of a track mounted crane is severely limited by working off a barge. Greg estimated that a 225 ton capacity crane, de-rated to approximately 135 tons, would be required for installation of the flow deflectors.

Flume Bridge Clearance

Bridge Elevation

The elevation of the bottom of the girders, less camber and deflection, can be calculated by subtracting the girder bottom thickness and the pipe wall thickness from the invert elevation of the pipe. All of these dimensions are provided in the Bonneville First Powerhouse, Juvenile Bypass Systems Improvements drawings. This set of construction documents has recently (15 March 2001) been issued as Final. Flume invert elevations, at either end, are 51.20 and 49.34 feet. The girder bottom thickness is shown as 1 foot-1 inch (1.08 feet), and a 48-inch O.D. SDR 32.5 HDPE pipe has a wall thickness of 1.5 inches (0.13 feet). Once the Flume Bridge is constructed the minimum girder bottom elevation will be 48.13 feet.

Water Surface Elevation

Water surface elevation taken from the *Design Memorandum Supplement No. 2 to Design Memorandum No. 37, Bonneville First Powerhouse, Juvenile Bypass System Improvements*, Published 15 January 1999 by CENWP are tabulated below.

Event	**Water Surface Elevation (feet)**
Maximum Flood (Spillway Design Flood)	51.5
Standard Project Flood	48.9
Base (100-yr) Flood	38.5
Maximum Operating Tailwater	33.1
Minimum Tailwater	7.0

The water surface elevation in this reach of the river is measured by the USGS Gage No. 14-128870, located 50 feet upstream of Tanner Creek. The *Invitation for Bid, DACW57-97-B-0027, Bonneville Lock and Dam, Skamania County, Washington, Construct High and Low Level Juvenile Bypass Outfall Structures, Construction Solicitation and Specifications*, August 1997 contained river stage and discharge data (current to April 1997). During the In Water Work Period (IWWP), defined below, the tailwater surface elevations that have a 50 percent probability of exceedance is the following:

Month	**Tailwater Surface Elevation (feet)**
December	15.04
January	15.44
February	17.02

Required Clearance

Crane Type	Required Clearance (feet)
Floating (165 ton)	83
Floating (125 ton)	69, reducible to 64
Crawler (track mounted)	20
Ringer	20
Guy Derrick	20

Provided Clearance

Subtracting the 50 percent probability of exceedance tailwater surface elevation from the minimum Flume Bridge undergirder elevation results in the following clearance values.

Month	Provided Clearance (feet)
December	33.09
January	32.69
February	31.11

Schedule

In Water Work Period

The IWWP begins 1 December and continues to the last day of February. No construction that affects fish behavior is allowed outside of the IWWP. In the past, there have been rare instances of time extensions of the IWWP. These are at the discretion of the National Marine Fisheries Service (NMFS) and it cannot be assumed that NMFS will grant an extension. It is a general assumption that NMFS supports construction activity dedicated to the protection of endangered and listed species and that they evaluate requests for IWWP time extensions by weighing the short-term negative impacts against the long-term benefits.

B1 Flume Bridge

Construction of the Flume Bridge is currently scheduled to begin on 20 July 2001 and continue through 2 April 2003.

Substructure

Construction of the substructure is currently scheduled to begin 3 December 2001 and continue through 24 June 2002.

Superstructure

Construction of the superstructure is currently scheduled to begin 1 July 2002 and continue through 16 October 2002.

B2 Corner Collector Outfall

The current schedule for the Corner Collector Outfall anticipates tendering construction bids in the Spring of 2002. In Water Work could begin 1 December 2002, after the Flume Bridge is substantially complete.

B2 Flow Deflectors

Mr. Lance Helwig, CENWP-EC-DS, also provided information regarding the anticipated schedule for installing the B2 Flow Deflectors. There may be two different Flow Deflector installations; the first is the installation of the six mandatory Flow Deflectors. This will occur before March 2002 and is expected to occur after the construction of the substructure for the Flume Bridge.

There is the possibility of the installation of twelve optional Flow Deflectors. These twelve would be installed at a later date, anticipated to be after the superstructure of the Flume Bridge is complete and would therefore require that the construction crane fit beneath the Flume Bridge girders.

Conclusion

- Construction of the Flume Bridge will prevent passage of a floating crane.
- As the schedule for Flow Deflector and Flume Bridge construction is currently known, installation of the mandatory Flow Deflectors are not affected by the Flume Bridge construction.
- As the schedule for Flow Deflector and Flume Bridge construction is currently known, installation of the optional Flow Deflectors will be affected by construction of the Flume Bridge. It will limit the lifting equipment to barge mounted crawler cranes.
- It may be possible to construct the Corner Collector Outfall substructure using a barge mounted crawler crane. If the Flume Bridge were constructed prior to the Corner Collector Outfall substructure, it would prevent use of a floating crane.

- A concrete adjustable cantilever structure cannot be erected from a barge loaded crane if the Flume Bridge is constructed. This is not the most practical method in any event. The most likely method of erecting this structure is with a lifting system supported by the structure. The other superstructure components and substructure can be constructed with either barge mounted crawler cranes or a temporary working platform. These issues need to be considered as design proceeds.

Appendix D

Tailrace Mortality Study

Analysis of B2 Tailrace Mortality and Consequences for High Flow Outfall Site Selection

BioAnalysts, ENSR, HDR, and INCA March 8, 2001

INTRODUCTION

Siting for a new high flow outfall (~5,300 cfs) is underway for the Bonneville Dam Second Powerhouse corner collector (B2CC). The B2CC, a surface flow bypass for smolts, will be created by modifying the existing I&T chute at B2. The outfall siting process has narrowed the candidate locations down to two sites, called Ranges D and F (Figure 1 and Photograph 1). Range D is located 400 ft directly off of the existing outfall. Range F is located adjacent to the B2 tailrace side of the tip of Cascades Island. Range F is about 0.4 miles downstream from Range D in the B2 tailrace. Both locations have favorable characteristics for a high flow outfall, although Range D was selected by the Corps of Engineers because of apparently better egress characteristics. Range F would be more costly because of its greater length. But, the level of predation, and hence mortality, in the stretch of the B2 tailrace between Ranges D and F is a concern by some for Range D.

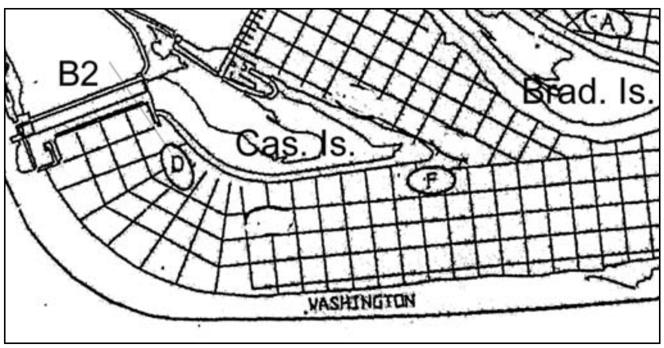


Figure 1. Map of B2 tailrace showing high flow outfall Ranges D and F. Grid spacing is 200 ft.

The purpose of this report is to use existing data and reports to assess the expected level of predation in the 0.4-mile segment of the B2 tailrace between Ranges D and F. Specifically, our objectives are as follows:

1. Smolt Survival -- Review reports on Bonneville Dam smolt survival studies and characterize operational scenarios, including flow conditions and release locations.

2. Predation -- Describe basic biology of northern pikeminnow, map known and historical concentrations of predators in the B2 tailrace, and assess the impact of the Northern Pikeminnow Removal Program.

3. Expected Predation Impacts on Survival at Ranges D and F -- Review and analyze CFD data from past and present conditions relative to smolt survival in the B2 tailrace and assess possible future smolt survival rates given new hydraulic conditions when the high flow outfall is operational.

Photograph 1. Bonneville dam tailrace.

This review will focus on the B2 tailrace between Ranges D and F. However, B2 tailrace is defined to extend from the powerhouse downstream ~1.6 miles, which is in the vicinity of a survival study release site (see next section). Direct survival studies using balloon tags will not be discussed because, at least for work at B2, they did not include tailrace mortality. This review includes sections on survival studies, predation, analysis of expected impacts, and conclusions.

SURVIVAL STUDIES

Smolt survival studies pertinent to the topic of B2 tailrace mortality were performed by NMFS in 1987-1992 as part of an assessment of relative survival through turbine and intake screen bypass routes at B2. They employed a mark-recapture technique. Each year, except 1991, about 1.5-2.0 million juvenile fall chinook salmon were marked with freeze-brands, tagged with coded wire tags, and released at various locations at the dam (explained in detail below under experimental conditions). Marked fish were recaptured at Jones Beach about 100 miles downstream from B2. Adult returns were expected to provide the "ultimate" assessment of survival differences, but poor return numbers made this analysis equivocal. Therefore, recapture rates at Jones Beach were the basic data from these survival studies.

There are several important caveats to the NMFS survival studies at B2. The results for the 1987-1992 studies apply only for the species and conditions tested, i.e., hatchery subyearling chinook salmon (*Oncorhynchus tshawytscha*) of size 70-120 mm during moderate to low river flows. As

Ledgerwood et al. (1990 p. 54) said, "Passage survival of subyearling chinook salmon taken directly from the hatchery may not be representative of highly smolted, river-run migrants or yearling-sized fish." Also, during the 1987-1989 studies, discharges at the Second Powerhouse and spillway were limited, often only occurring for the purpose of test fish releases.

Experimental Conditions

The survival studies used subyearling fall chinook salmon as the test species (Table 1). Test fish were obtained from the Bonneville Hatchery, except in 1992 when they had to be acquired from Little White Salmon National Fish Hatchery. Fish sizes were representative of subyearling migrants. Fish were released in late June and July (Table 1).

Release locations (Figure 2) during tests in 1987-1992 varied depending on specific study objectives (Table 1). Releases in the B2 intake screen bypass, the primary objective of the survival studies, are not useful for our review because of the unique effects the bypass/outfall had on smolt survival. Likewise, the turbine releases were intended to examine effects of turbine passage. Spillway, B1, and egress releases were all special efforts for specific objectives. Of the nine different release locations, the front-roll and downstream locations are most germane to our review of B2 tailrace mortality (they were used concurrently in only 1988 and 1989). Differences in recovery of marked fish released form the front roll and downstream sites should reflect the effect on survival of passage through the 1.6 miles of B2 tailrace between these study sites. This stretch of river includes the 0.4-mile tailrace segment between high outfall Ranges D and F (Figure 1). Thus, although we present some data from 1990, our review of survival work will emphasize 1988 and 1989 results from the front roll and downstream release locations because it is the most relevant B2 survival data to high flow outfall site selection.

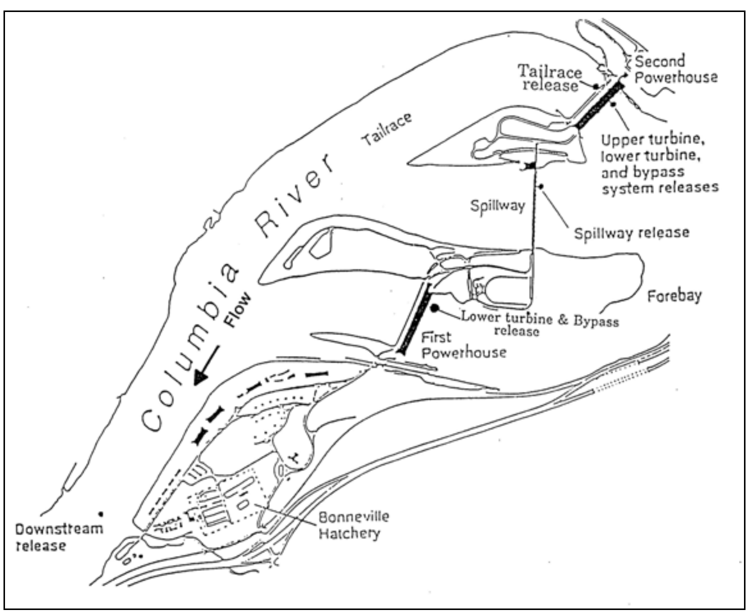


Figure 2. Schematic of Bonneville Dam and vicinity showing release locations during 1987-1992 survival studies. The "tailrace release" site is the same as the front roll site. Obtained from Figure 2 in Dawley et al. (1998).

Table 1. Summary of conditions during subyearling chinook survival studies at Bonneville Dam.

		1987	1988	1989	1990	1992
	Reference	Dawley et al. (1988)	Dawley et al. (1989)	Ledgerwood et al. (1990)	Ledgerwood et al. (1990)	Ledgerwood et al. (1990)
Biological Characteristics	Fish Size (g)[17]	4.4-4.6	6.5-9.1	6.1-10.2	5.6-10.1	6.4-8.1
	Fish Length (mm)[18]	73.6-82.7	89.4-108.7	83.4-99.4	nd	nd
	Fish Source	BON	BON	BON	BON	LWS
	Release Period	6/24-7/19	6/27-7/24	6/22-7/22	6/30-8/3	6/18-7/9
Release Locations	Bypass	XX	XX	XX	XX	nt
	Upper Turbine	XX	XX	XX	nt	nt
	Lower Turbine	XX	XX	XX	XX	XX
	Front-roll	nt	XX	XX	nt	
	Downstream – Hamilton Island	XX[19]	XX	XX	nt	XX
	Bypass Outfall -- Egress	nt	nt	nt	XX	nt
	Spillway – Bay 5	nt	nt	XX	nt	nt
	B1 Bypass	nt	nt	nt	nt	XX
	B1 Turbine	nt	nt	nt	nt	XX

Hydraulic Conditions

During the NMFS survival tests at B2 in the late 1980s and early 1990s, B1 was the priority powerhouse because of poor guidance by the B2 intake screens for subyearling chinook salmon. B2 operations were restricted to daytime when spill reached the 75 kcfs cap or when energy load could not be met from elsewhere in the system. For the survival research, B2 was operated at night from about 0000-0800 h only on nights when test fish were released. One-half of the powerhouse (Units 11, 16, 17, and 18) was operated during tests in 1987-1989 and 1992; during 1990 tests, full B2 powerhouse was operated.

Since test fish were released in June and July, flows and tailwater elevations were typically low (Table 2). Test periods in 1987-1989 had lower flows in June-July than those in 1990 and 1992. During ½ powerhouse operation, B2 flow was about 50-60 kcfs. There was no spill during 1987 and 1988 tests. During 1989, the only spill was that for test fish releases at the spillway. In 1990 and 1992, some spill occurred. Annual mean tailwater elevation during fish releases ranged from 11.8 to 17.2 ft. Annual mean water temperature during fish releases ranged from 66.0 to 68.2 °F.

[17] Measured at release.

[18] Measured at capture, except for 1989 when FL was measured at release.

[19] In 1987, the downstream release at Ham. Is. was near the shoreline; in subsequent years this release was in mid-river.

Table 2. River conditions during test-fish release dates for B2 survival studies (typically in June-July).

		Total Q*	B2 Q	Spill Q*	B2 TW El.	Temp.	B1 Q
1987	Mean	117.5	57.0	0.0	11.8	67.4	na
	Min	94.3	54.8	0.0	10.0	64.0	na
	Max	139.2	60.7	0.0	13.5	69.0	na
1988	Mean	124.0	56.5	0.0	12.3	66.7	na
	Min	96.0	51.2	0.0	9.6	65.0	na
	Max	164.2	62.4	0.0	15.0	69.0	na
1989	Mean	133.3	61.5	53.0	16.9	66.0	na
	Min	104.6	56.8	53.0	16.4	62.0	na
	Max	158.8	67.5	53.0	17.3	69.0	na
1990	Mean	186.0	121.2	26.8	17.2	68.2	na
	Min	132.9	112.7	0.0	14.7	66.0	na
	Max	278.3	131.3	89.1	21.4	72.0	na
1992	Mean	179.7	54.8	65.6	15.6	67.5	48.4
	Min	107.0	52.9	13.0	11.7	64.0	40.0
	Max	247.0	59.7	129.0	19.0	69.0	54.8

* 1990 total and spill and 1989 spill data obtained from DART.

Four sources of data were reviewed to develop a hydraulic characterization of the B2 tailrace during the NMFS survival studies (Table 3):

1. Physical model data acquired on the 1:100 scale general model of the Bonneville Project at the USACE Engineering Research and Design Center (ERDC), formerly known as Waterways Experiment Station (ERDC). These data were generated with floats submerged to a depth of 14 feet. The data were acquired by videotaping the float movement from above, digitizing the float tracks, and calculating the float velocities at various points in the tailrace. Since several float releases were required to cover the entire tailrace, these data may not be representative of average conditions, nor are they synoptic over the entire area of coverage.

2. Field data acquired by ERDC staff in 1995, using Acoustic Doppler Current Profilers (ADCPs). These data were acquired from a boat which was continuously moving across measurement cross sections. Therefore, these data are not synoptic over the survey area and may not represent average velocities either.

3. Field data acquired by ENSR last year (ENSR 2000). These data were also acquired using ADCPs, but the survey boats maintained position over a measurement station for a long enough time period to characterize the average velocities at the station.

4. Computational Fluid Dynamics (CFD) calculations performed at the U.S. Department of Energy's Pacific Northwest National Laboratory (PNNL). These calculations were performed using the three-dimensional CFD model Flow-3D.

Table 3. **B2 Tailrace Mortality Study Flow Conditions and Available Velocity Data**

Case/Figure	Model or Field Data	Date of Collection	Measurement Technique	Purpose	Flow Conditions								
					Forebay Level *(ft)*	Tailwater Level *(ft)*	Total River Flow *(kcfs)*	B1 Flow *(kcfs)*	B2 Flow *(kcfs)*			Spill Flow *(kcfs)*	Miscellaneous Flow *(kcfs)*
									Total	Outfall	Unit Operations		
8.1.1.1 Mortality Case 1 (M-1) and comparable model and field data – Partial B2 Load													
M-1	B2 Mortality Study				N/A	N/A	130.0	74	56.0	0	11, 16 - 18	0	0.0
1/Fig3	Field	1995	ADCP Moving Transects	N/A	74.7	10.9	128.3	61	58.8	0	11, 12, 17, 18	2.5	6
A/Fig6-7	CFD	N/A	N/A	B2 Mortality Study	74	11.5	130.0	64.1	61.0	0	11, 16 - 18	4.9	0.0
8.1.1.2 Mortality Case 2 (M-2) and comparable model and field data --- Full B2 Load													
M-2	B2 Mortality Study				N/A	N/A	186.0	66	120.0	0	11 - 18	0	0.0
2	Field	7-Feb-00	ADCP: 13, 23, & 29 ft depths	1:100 Model Verification Study	74	16.4	179.3	32.8	141.1	0	11 - 18	0	5.4
3/Fig5	1:100 Model (1)	1980's	Floats submerged 14 ft	Navigation Lock Study	N/A	14.2	162.4	0	160.0	0	11 - 18	2.4	0.0
8.1.1.3 CFD data with adjustable cantilever outfall at Range D													
B/Fig8-9	CFD	N/A	N/A	B2 Mortality Study	74	11.5	115.0	0	60.0	5.3	11,12,17,18	50.0	0.0

Following review of the first three data sources, three data sets were selected, which came reasonably close to reproducing the mortality test hydraulic conditions. These are listed in Table 3 as Cases 1, 2, and 3. Case 1 was representative of mortality test M-1 (partial B2 load) and Cases 2 and 3 were representative of mortality test M-2 (full B2 load).

Two CFD runs, Cases A and B, were made specifically for this review. Case A simulated the low B2 discharge (56 kcfs) flow conditions. Case B modeled the adjustable cantilever at Range D discharging 5,300 cfs. Plots of the two field data sets (Cases 1 and 2), the 1:100 physical model data (Case 3) and one CFD run (Case A) are included as Figures 3-7. The two field and CFD plots presented are at elevations -4.3m (-14.1 ft), 1 m (3.4 ft), and 5 feet (1.5 m), respectively, which are representative of near surface velocities where smolts were likely distributed.

This approach was taken both to review data which were consistent with the float data acquired on the physical model and to characterize conditions in the upper part of the water column where the fish were probably located. The data sets for similar conditions conform reasonably well with each other. Figures 7 and 9 expand the same data shown in Figure 6 and 8, but with a focus on the vicinity of Range D.

When B2 was partially loaded, there were two distinct areas of slightly higher velocity downstream from the operating turbines (Figures 3, 6, and 7). However, within approximately 400 ft downstream, about at Range D, flow was fairly uniformly distributed across the B2 tailrace with magnitudes ranging from approximately 0.5 to 1.0 m/s (1.6 to 2.8 fps). The higher velocities, about 1 m/s (3.2 fps), were located along the north (Washington) shore, except for the narrow plume of higher velocity caused by operation of Unit 11. Velocities near the Cascades Island shore were typically about 0.8 m/s (2.4 fps). Velocities along neither shoreline, nor anywhere in the B2 tailrace, were above the 1.2 m/s (4 fps) threshold required to prevent predator holding. Test fish for the mortality study were released in the front-roll of the turbine boil at Unit 17. The flow vector directions indicate that surface flow from the release point may have expanded into the area downstream from the non-operating center units, where velocities were lower, on the order of 0.2 to 0.8 m/s (0.6 to 2.4 fps).

When B2 was fully loaded, Case C, surface velocities were fairly uniform across the width of the B2 tailrace all the way downstream past the tips of Cascades and Bradford Islands to Tanner Creek (Figures 4 and 5). The only exceptions were the zones of circulation that were established in the spillway and B1 tailraces and at the entrance channel to the new navigation lock. Velocities were on the order of 1.5 to 2.4 m/s (5 to 8 fps), in the B2 tailrace upstream from Range F. These are well above the predator holding threshold. The exception was immediately along the shore of Cascades Island, where they dropped to 0.6 to 1 m/s (2 to 3.2 fps) from about midway to the end of the island. Flow lines from the test fish release location at Unit 17 progressed readily downstream through a region of velocities varying from 1.4 to over 2.4 m/s (4.6 to over 8 fps).

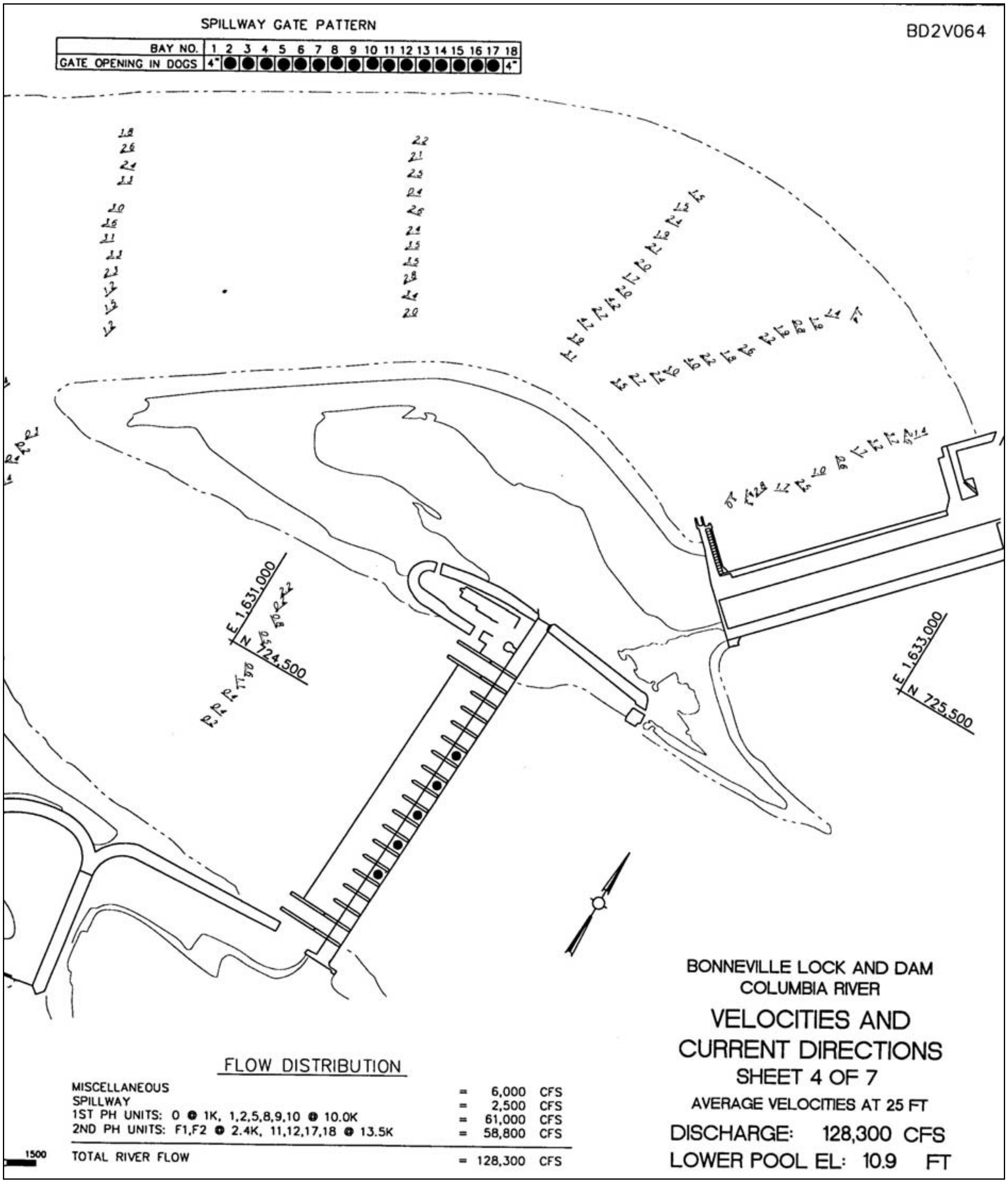


Figure 3. Case 1 – 1995 ERDC Field ADCP Data for Partial B2 Loading

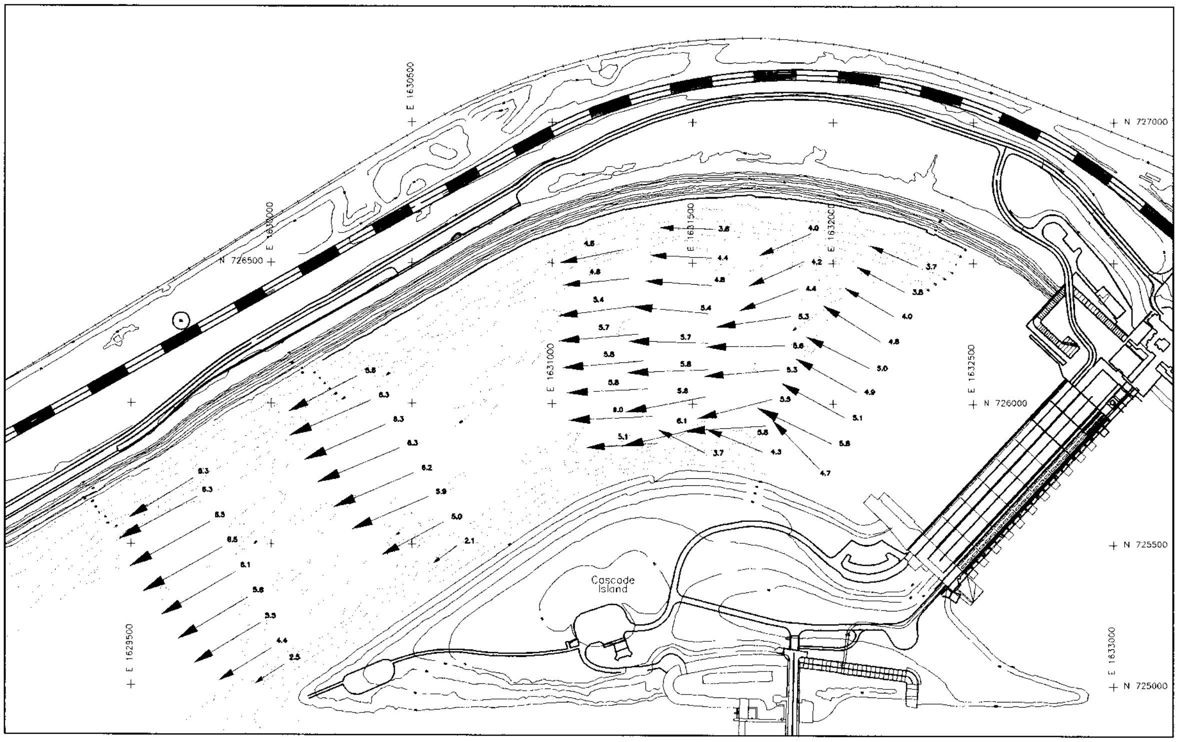


Figure 4. Case 2 – 2000 ENSR Field ADCP Data for Full B2 Loading.

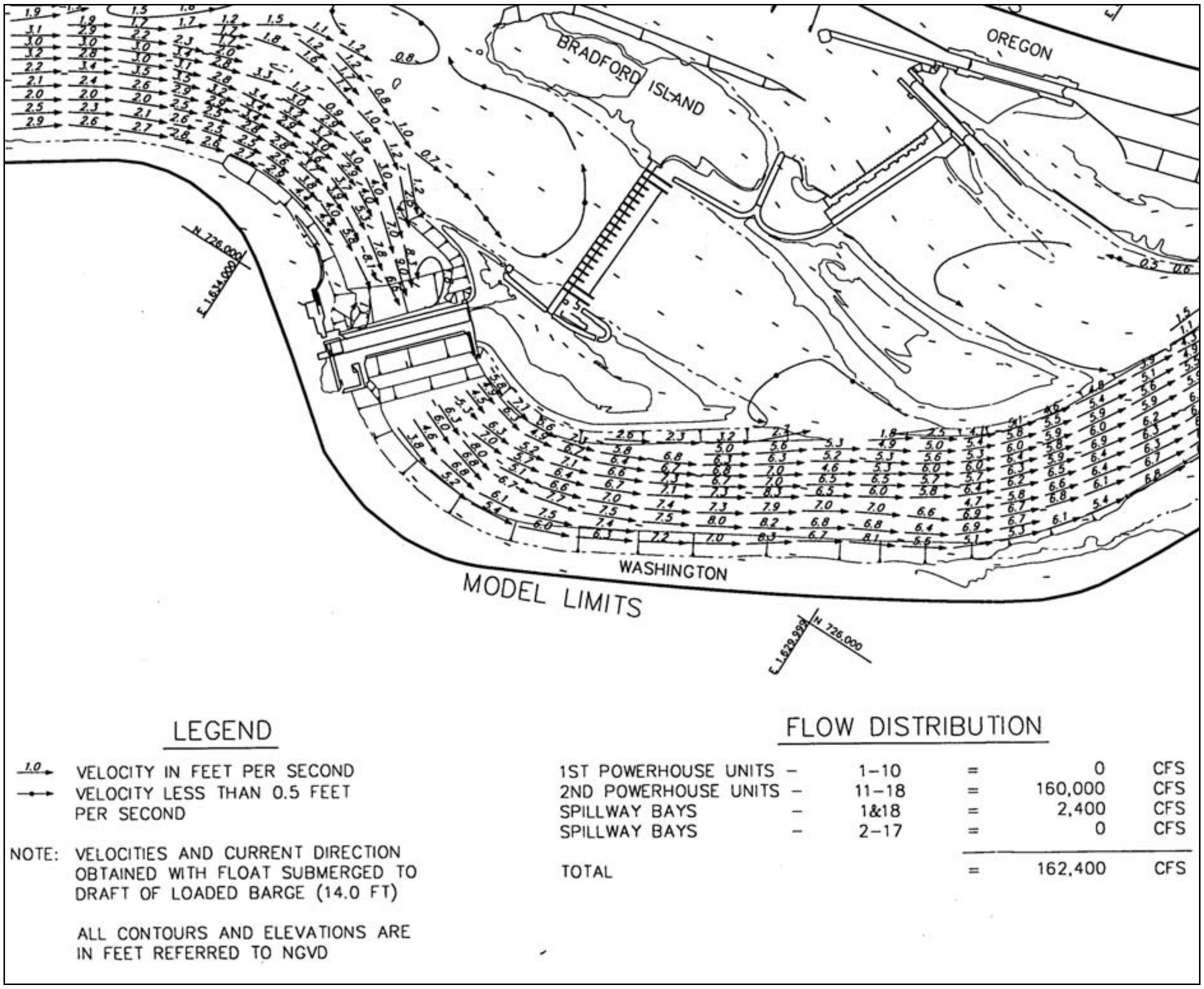


Figure 5. Case 3 – 1980's ERDC 1:100 Model Float Data for Full B2 Loading

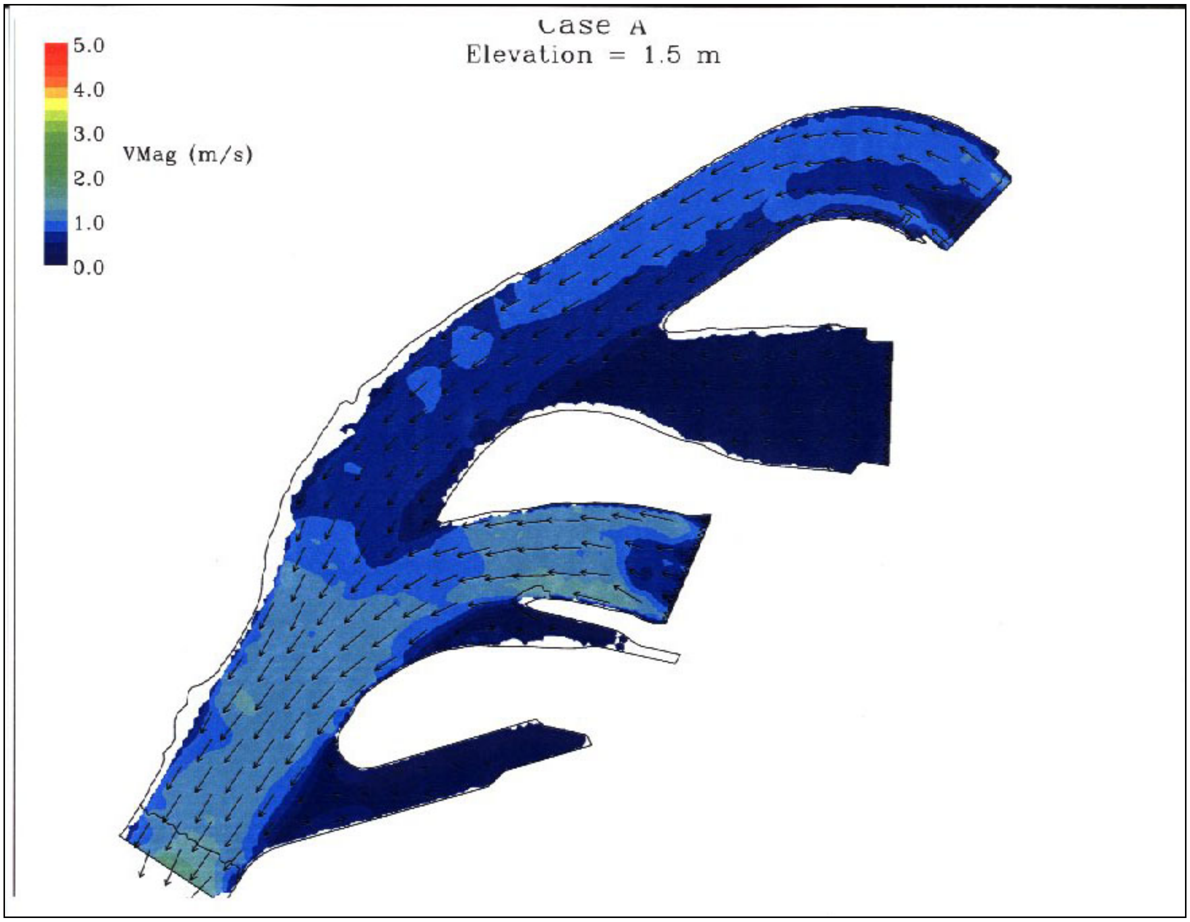


Figure 6. Case A – 2001 PNNL CFD Data for Partial B2 Loading.

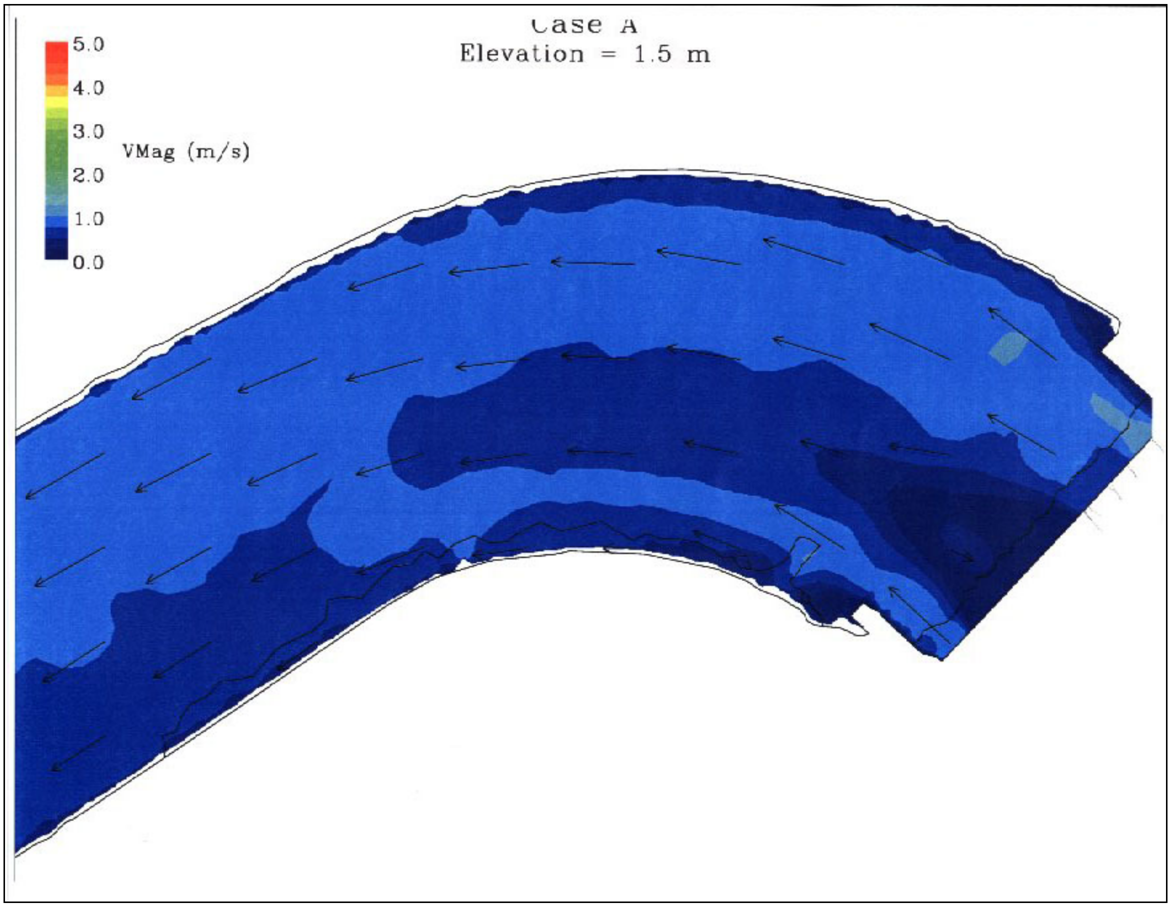


Figure 7. Case A – 2001 PNNL CFD Data for Partial B2 Loading (Close-up at Range D).

Summary of Survival Study Results

Differences in recovery percentages at the estuary (Jones Beach) reflected passage survival differences because tagging, release, and recovery methods did not appear to affect recovery rates. But, an XX% difference in recovery percentage does not equate to an XX% difference in survival (see Appendix A for an example calculation). Therefore, we must analyze the impacts of predation in the B2 tailrace without an estimate of the absolute survival rate in this piece of river.

Recovery rates were higher in 1989 than 1988 or 1990 (Table 4), presumably because of improved capture techniques compared to 1988 and relatively low river flows compared to 1990. During 1988 and 1989 studies, recovery rates at Jones Beach were highest for downstream release site than the others, except for the spillway release in 1989 (Table 4). The front roll release site was the next most successful release (Table 4). The percentage difference in recovery rate between the front roll and the downstream release sites was -10% in 1988 and –5% in 1989 (Table 4). Over the two study-years, this difference was –7%. Let it be clear that this is a relative difference in recovery rate and does not represent an absolute difference in survival.

Table 4. Results from BON survival studies in1988 and 1989. Data for 1988-1989 from Table 5, p. 47 Ledgerwood et al. (1990). Data for 1990 from last 10 releases (lower turbine releases not successful before) from Table 4, p. 35 Ledgerwood et al. (1991).

	Release Location	1988	1989	1988-1989	1990
Recovery Rates (%)	Bypass	0.4376	0.8007	0.6191	0.5577
	Upper Turbine	0.5024	0.8298	0.6732	nt
	Lower Turbine	0.5104	0.8256	0.6680	0.5721
	Front-roll	0.5095	0.8637	0.6866	nt
	Downstream – Ham. Is.	0.5690	0.9061	0.7376	nt
	Bypass Outfall -- Egress	nt	nt	nt	0.5686
	Spillway – Bay 5	nt	0.9604	---	nt
Percentage Difference from Front Roll	Bypass	-14	-7	-10	---
	Upper Turbine	-1	-4	-2	---
	Lower Turbine	0	-4	-3	---
Percentage Difference from Downstream	Front-roll	-10	-5	-7	---
	Spillway	nt	6	---	---

Ledgerwood et al. (1990 p. 50) stated, "…differences between recovery percentages of the front roll groups and the downstream groups provide an estimate of the effects of passage through the 2.5 km tailrace and river downstream from the Second Powerhouse on survival. The 1988 and 1989 combined mean recovery percentages of front roll-released fish was about 7% lower than the combined mean recovery percentages of downstream released fish." This observation indicates that there was probably predation in the B2 tailrace during survival studies in 1988 and 1989.

When prorated, the seven percent difference in Jones Beach recovery rates between the front roll and downstream release sites corresponds to a 1.75% difference for the tailrace segment between Ranges D and F. This is a worst-case estimate of the relative difference in survival between the two locations. Test fish released in the front roll likely egressed along the Washington shore where predation was heavy because partial powerhouse loading then was concentrated at the northern-most units and velocities were low (< 4 fps). These data should not be applicable to a new high flow outfall in the B2 tailrace because it will be designed so that the outfall plume does not intersect shorelines.

In 1989, fish were also released at a spillway location (Bay 5). Recovery percentages for these fish were 6% higher than those for the downstream release site (Ham. Is.). However, this may have been an artifact of the study conditions. Spill (~53 kcfs almost doubled ambient river flow), which only occurred during spill releases for the study (2.5 h prior and 5.5 h after release), could have cleared out predators in the region downstream of the downstream release site during spill tests because water velocities were increased dramatically. In addition, the spill release as designed as a best-case scenario and was tested in only one study year.

In conclusion, based on the collective survival data, Dawley et al. (1998) stated the following reasons for poor performance of the old B2 bypass outfall:

- Bypass passage appeared to cause significant stress, which led to increased vulnerability to predation in the tailrace.
- The outfall was a point source for predators to congregate near, even though ambient velocities were about 3.3-5.3 fps and exit velocity was about 25 fps.
- Egress was through the relatively low velocity B2 tailrace which provided a large amount of habitat suitable for northern pikeminnow.
- The bypass outfall was located on the northern side of a tailrace which angles to the south, thereby directing migrants shoreward toward rip-rap areas that are prime habitat for northern pikeminnow. [This also pertains to the front roll release at Unit 17.]

Dawley et al. (1998) stated the following conclusions for good performance of the downstream release site off Hamilton Is.:

- High ambient velocity – 5 to 7 fps.
- Far from shore – about 325 ft.
- Rapid downstream dispersal resulting in decreased smolt density and increased recovery time prior to encountering predators.
- Current direction (i.e., outfall flow) parallel to shoreline.
- Lack of predator attraction from a continuous egress of smolts at a single location [specific condition during the study].
- Night releases minimized avian predation [specific condition during the study].

When applied to Ranges D and F in the high flow outfall siting study, Dawley et al.'s (1998) parameters show similarities between the two sites, except for one parameter, egress distance (Table 5). To summarize, it all comes down to the 0.4 mile distance between the two sites and how much predation is expected to occur in this specific stretch of tailrace.

Table 5. Summary of Range D vs F using Dawley et al.'s (1998) parameters. * signifies an additional parameter from B2 outfall site selection process.

Parameter	Assessment	D	F
Bypass Stress	similar levels of stress expected	=	=
Point Source	both broadcast sources	=	=
Ambient Velocity	GET DATA but probably similar	=	=
Egress Distance	F is ~0.65 km (0.4 miles) farther d/s than D	--	+
Orientation	similar, toward center of river, parallel to shore	=	=
Distance from Shore	similar, both about 200 ft	=	=
*Resiliency to op's	F depends on spill	+	--

PREDATION

Northern Pikeminnow Biology

Life History

Northern pikeminnow [formerly northern squawfish] (*Ptychocheilus oregonensis*) is a large, widely distributed, native cyprinid of the Columbia River Basin. Adult northern pikeminnow (NPM), typically range in size from 300-550 mm fork length (FL). They are opportunistic feeders, with fish >200 mm FL becoming increasingly piscivorous (Poe et al. 1991). This species is known to be long-lived with mature adults living up to 20 years or longer (Parker et al. 1995). Reproduction and early life history of NPM were described by Barfoot et al. (1999) and Gadomski et al. (2000).

Feeding Habits

In the lower Columbia River (below McNary Dam), NPM feed primarily on invertebrates until they reach about 250 mm FL (Poe and Rieman 1988). Once they reach 250 mm FL or so, their diets are highly dominated by fish and crayfish (Poe et al. 1991) and they become major predators of juvenile salmonids in the Columbia Basin. Predation is especially heavy at mainstem dam tailraces (Vigg et al. 1991; Ward et al. 1995). Losses of juvenile salmonids to predators can be quite significant. In one study in John Day Reservoir, predation was estimated to be about 2.7 million juvenile salmon per year for 1983-1986, with monthly predation mortality ranging from 7% in June to 61% in August (Rieman et al. 1991). In that same study, the tailrace boat restricted zone (BRZ) for McNary Dam was by far the most concentrated area of predation by the northern pikeminnow; this relatively small area accounted for over 20 % of the losses.

Consumption of juvenile salmonids by northern pikeminnow in the Bonneville Dam tailrace BRZ (B1, B2, and spillway) has been documented to be among the highest in the system (Petersen et al. 1991; 1993). Reported predation levels ranged from about 2.5 smolts/predator /day in spring to about 7.8 smolts/predator/day in the summer. Predation rate was positively correlated with increasing water temperatures and decreasing smolt size in summer.

During the NMFS survival studies in 1990 and 1992, predation indexing occurred in the B2 tailrace BRZ concurrently on several dates (Petersen et al. 1991 and 1993). Several hundred coded wire tags (CWT) from survival study fish were recovered in the digestive tracts of NPM collected by electrofishing in both years. During two of the dates of survival study releases, July 24 and 25, 1990, the US Fish and Wildlife Service collected 58 NPM in two 15-min electrofishing samples. Fifteen NPM were randomly selected for food habits analysis from each date. Twenty of the 30 NPM ingested a total of 92 juvenile salmonids and 55 of those juveniles had CWTs. After the reading tag codes, it was determined that 17 were from the lower turbine release, 29 were from the bypass release, and 9 were from the egress release. Although the sample sizes were small, these data indicated that vulnerability to predators is related to release site and that a release site higher in the water column and presumably in higher velocity water may have reduced this vulnerability.

Swimming Performance/Limits

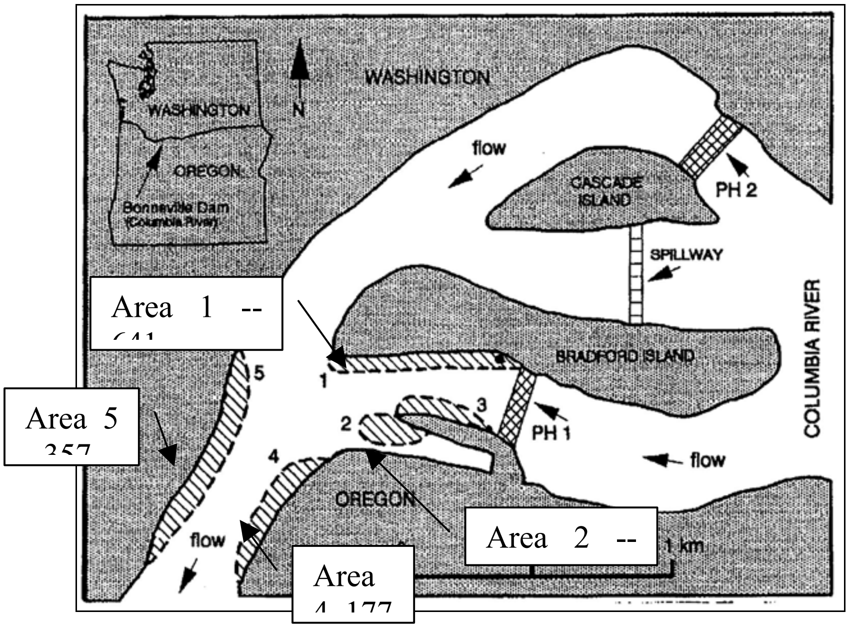
In 1991 and 1992, studies of NPM were conducted to determine their endurance/fatigue limits for prolonged swimming which could be used to assist in setting biological criteria for juvenile outfalls (Mesa and Olson 1993). Prolonged swimming speeds are those that fish can maintain for up to 200 min before fatiguing. Two size classes (medium = 300-390 mm and large = 400-490 mm) of predaceous-sized NPM were tested at 12 and 18 °C. At 12° medium-sized fish fatigued at 1.00 m/s and large-sized fish fatigued at 1.04 m/s. At 18° medium-sized fish fatigued at 1.07 m/s and large-sized fish fatigued at 1.12 m/s. All fish fatigued faster at 12° than 18°. Mean times to fatigue were 12 min for medium-sized fish and 28 min for large-sized fish. All fish fatigued at 1.30 m/s (4.3 fps). The researchers recommended that outfalls be sited in areas of high water velocity (1.3 m/s or higher) so NPM predation may be reduced, because these predators would not be able to hold position to efficiently feed on juveniles exiting a point source outfall.

Preferred Habitat

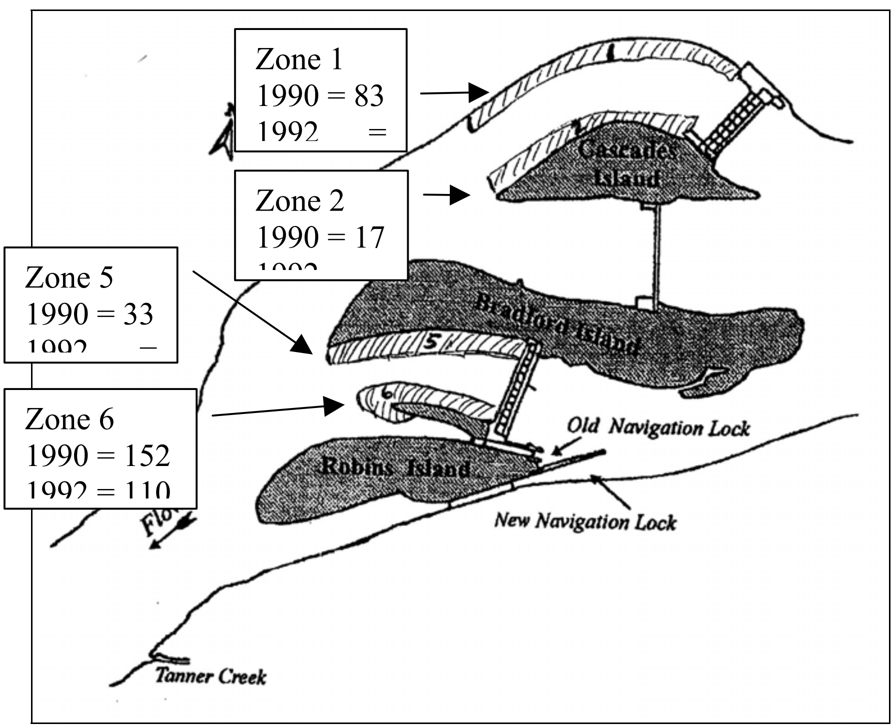
An extensive radio telemetry study of the NPM was conducted May through December in 1993 and 1994 to examine the seasonal distribution, movements, and habitat preferences of this species in several lower Columbia River reservoirs (Martinelli and Shively 1997). During the study, 335 radio-tagged NPM were tracked by boat, plane, and fixed station receivers. Results indicated that NPM were commonly associated with mean water depths of 4.8 m, water velocities of < 1 m/s, and mean distance from shore of 43.6 m. Cobble-boulder or cobble-bedrock substrates strongly dominated the habitats where NPM were found; sand substrate was uncommon for NPM in the B2 tailrace. In another example of habitat preference, Dawley et al. (1988) surmised that relatively low recapture rates for marked fish released at the downstream release site (Figure 2) nearshore in 1987 were due to predation in the shallow nearshore waters; they moved the releases to mid-river after 1987.

Historical Predator Distribution

The primary predator of juvenile salmon in Bonneville Dam tailwaters, northern pikeminnow, is apparently widely distributed (Maps 1-3). Catch of northern pikeminnow using electro-fishing was highest near rocky shoreline areas. In the B1 tailrace, abundance of northern pikeminnow was greatest near the low flow outfall for the submersible traveling screen bypass system (Maps 1-3). They were also prevalent along Robbins and Bradford islands. In the B2 tailrace, northern pikeminnow seemed to be concentrated in the tailrace at the north and south ends of the powerhouse (Maps 2-3). High densities were also observed in areas near the rip-rap along the Washington shore (Map 2).



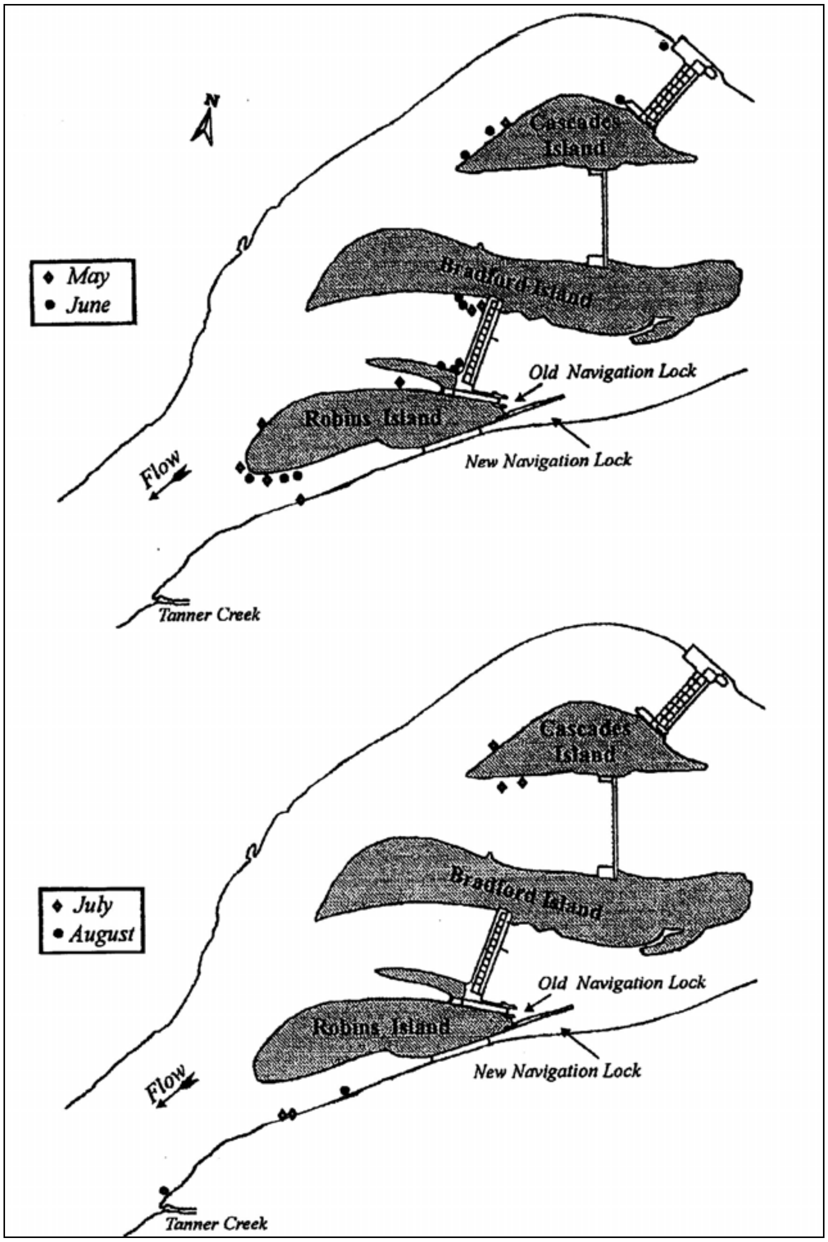
Map #1. Catch of northern pikeminnow (equal effort) at four sampling areas (1, 2, 4, 5 diagonal hatching) at Bonneville Dam tailrace and location in Area 1 where tagged salmon were released. Area 3 was not able to be sampled for 4 of the 6 sample dates. Modified from Figure 1 in Peterson et al. (1994).



Map #2. Catch of northern pikeminnow (equal effort) for Zones 1, 2, 5, and 6 during the predator indexing study in 1990 (Petersen et al. 1991).

In another radio telemetry study, the behavior and distribution of 60 radio-tagged northern pikeminnow were monitored from May through August, 1996 in the forebay of Bonneville Dam (Knutsen et al. 1997) to determine their distribution and movements in response to project operations and the presence of juvenile salmonids. During the early period of the study (May) 12 of the 60 tagged pikeminnow moved into Bonneville Dam tailrace and stayed there until August. As shown in Map #3, especially in spring, pikeminnow at B2 were found at the north and south ends of the powerhouse and near the downstream end of Cascade Island. At powerhouse I tailrace NPM were found near the submerged bypass outfall exit and the sluiceway outfall and at the tip of Robbins Island.

Predators were not distributed completely across entire tailwater channels; they were mostly distributed near shorelines where there was cover. In general, northern pikeminnow prefer littoral, relatively shallow areas in reservoirs and dam forebays and tailraces (Ward et al. 1995; Martinelli and Shively 1997). Therefore, outfall sites will have to be away from shorelines (> ~100 ft perhaps). Also, receiving water velocities will be higher away from shore than near shore, improving outfall conditions.



Map #3. Mobile tracking contacts of radio-tagged northern pikeminnow in the tailrace area of Bonneville dam in May (n=8), June (n=12), July (n=5), and August (n=2) in 1996. Modified from Figure 10 in Knutsen et al. (1996).

Northern pikeminnow are very mobile fish (Knutsen et al. 1996). They can move readily from one area to another in search of prey. For this reason, the "known" distribution of these fish may change as the distribution of smolts changes due to new outfalls and project operations. In addition, predators have been observed in the vicinity of all potential outfall sites. Predators in the Bonneville Dam tailrace are prevalent and cannot be easily avoided. However, we can locate and design high flow outfalls for B1 and B2 such that conditions conducive to predation are minimized (e.g., see the preliminary high flow outfall guidelines).

Impacts of NPM Removal Program

In 1999, an evaluation of the impact of the NPM predator control program was conducted by the Oregon Department of Fish and Wildlife (Zimmerman et al. 2000). Two objectives of this study were to: (1) estimate the reductions in predation on juvenile salmonids since implementation of the predator control fisheries, and (2) compare 1999 NPM population parameters (relative abundance, consumption, size and age structure, growth, and fecundity) to estimates of these parameters made in 1990-1996. We focus on results for the Bonneville Dam tailrace population of NPM. (Note that only the tailrace downstream of the BRZ could be compared because no sampling of the BRZ could be done in 1999 due to high spill levels.)

There are distinct differences in NPM population characteristics between 1990-1996 and 1999 sample periods (Table 6). Compared to mean relative abundance of NPM in Bonneville Dam tailrace from 1990-1996 (CPUE= 4.45), relative abundance in 1999 was CPUE =3.5. The Bonneville tailrace predation index average for spring 1990-1996 was 3.78 and for the spring of 1999 it was 0.4. The summer index average for 1990-1996 was 3.5 and for 1999 the average was 0.6. Based on the 1999 survey data, it appears that both the abundance of NPM and their consumption rate of juvenile salmonids was lower than pre-program levels, especially in summer. Unfortunately, the BRZ in the Bonneville tailrace was not able to be sampled in 1999, so there is some uncertainty if the sub-population close to the dam would reflect similar reductions. However, since most radio telemetry studies of northern pikeminnow indicate that there is frequent and rapid movement within tailrace areas of dams (Hansel et al. 1995), it is likely that the BRZ and remainder of tailrace sub-populations are well mixed and the parameters would be very similar.

Table 6. NPM population characteristics between 1990-1996 and 1999 sample periods.

	1990	1992	1993	1994	1995	1996	Average 1990-1996	1999
Density Index								
Tailrace	5.8	3.4	9.6	2.9	2.2	2.8	4.45	3.5
Tailrace BRZ	13.7	12.9	14.5	18.9	4.6	5.8		N/a
Predation Index Spring								
Tailrace	5.5	1.4	6.1	7.4	1.4	0.9	3.78	0.4
Tailrace BRZ	8.0	2.8	3.5	2.5	1.7	0.8		N/a
Predation Index Summer								
Tailrace	2.3	5.7	9.1	1.0	1.6	1.3	3.5	0.6
Tailrace BRZ	16.4	21.8	3.2	1.3	1.2	4.0		N/a

Analysis of Expected Impacts

Expected Environmental and Hydraulic Conditions

Bathymetry in B2 Tailrace Re: Predator Habitat – Bathymetry is fairly uniform (El. –18 to –25 ft) from the end of the draft tube excavation to the tip of Cascades Island. Bank slopes in the B2 tailrace are approximately 2.5H to 1V on the south shore and approximately 4H to 1V on the north shore.

Thus, Ranges D and F and the stretch between them should be in water about 30 ft deep (TW at El. 10 ft). This is deep water for NPM and is not preferred habitat.

Bottom Type in B2 Tailrace Re: Predator Habitat – Bottom materials in the B2 tailrace from the dam downstream to the tip of Cascades Island are mostly pre-slide alluvium (PSA). PSA is primarily gravel, sand, and silt deposits which result in a fairly smooth and uniform bottom. The south and north shorelines of the B2 tailrace are comprised of rip-rap. Thus, if the outfall is located far enough from shore (~200 ft), i.e., away from the rip-rap, then the bottom type will not be favorable habitat for NPM.

Expected Velocities from CFD for the New B2 High Flow Outfall -- The CFD model was used to calculate the tailrace flow field with and without the effect of the adjustable cantilever outfall at Range D. The conditions without the outfall were described in the section reviewing tailrace hydraulics for the mortality studies. B2 loading for this case reflects the present B2 operating scheme for four units. The case with the outfall flow is the final one listed in Table 3, Case B. The outfall invert was set at elevation 2.1 m (7 ft). The resulting flow field at elevation 1.5 m (4.9 ft) is plotted in Figures 8 and 9. Figures 9 expands the data shown in Figure 8, with a focus on the vicinity of Range D.

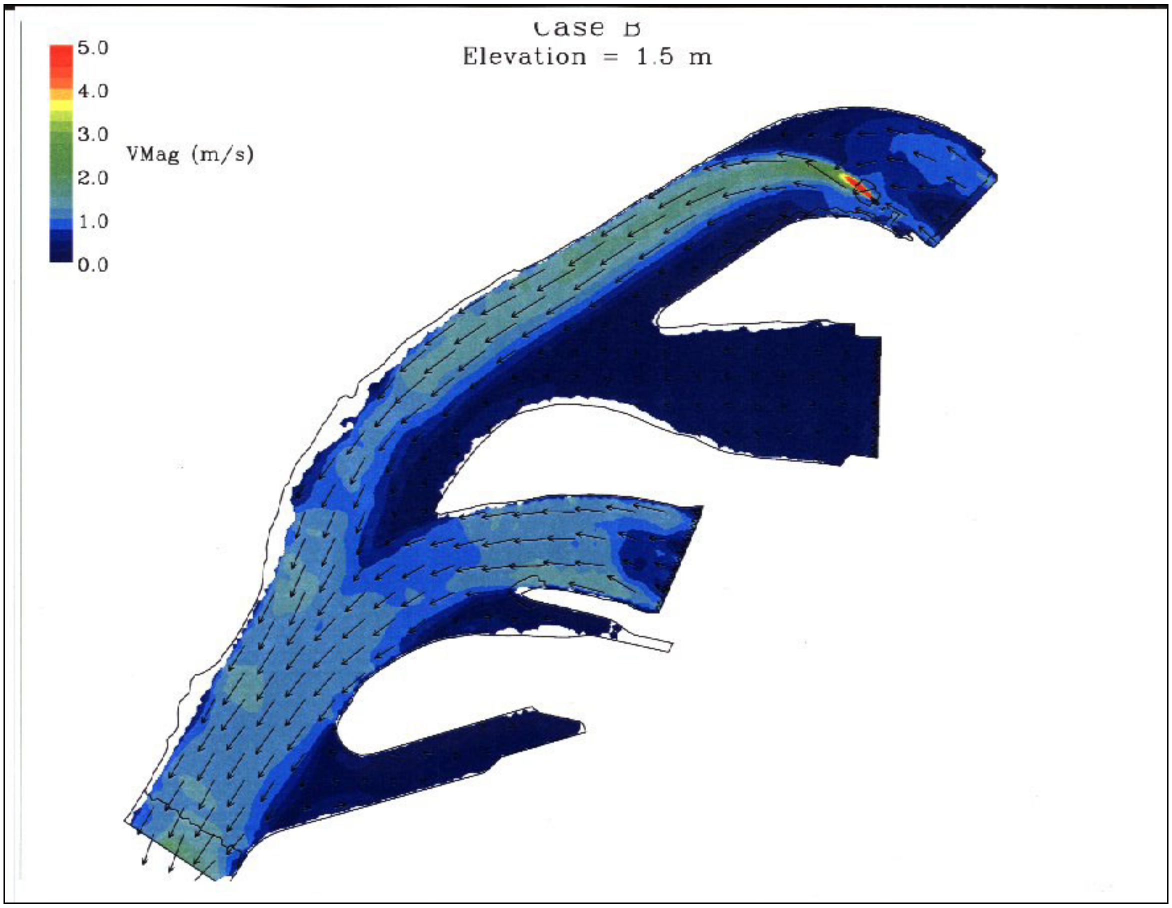


Figure 8. Case B – 2001 PNL CFD Data for Partial B2 Loading with Adjustable Cantilever Outfall at Range D

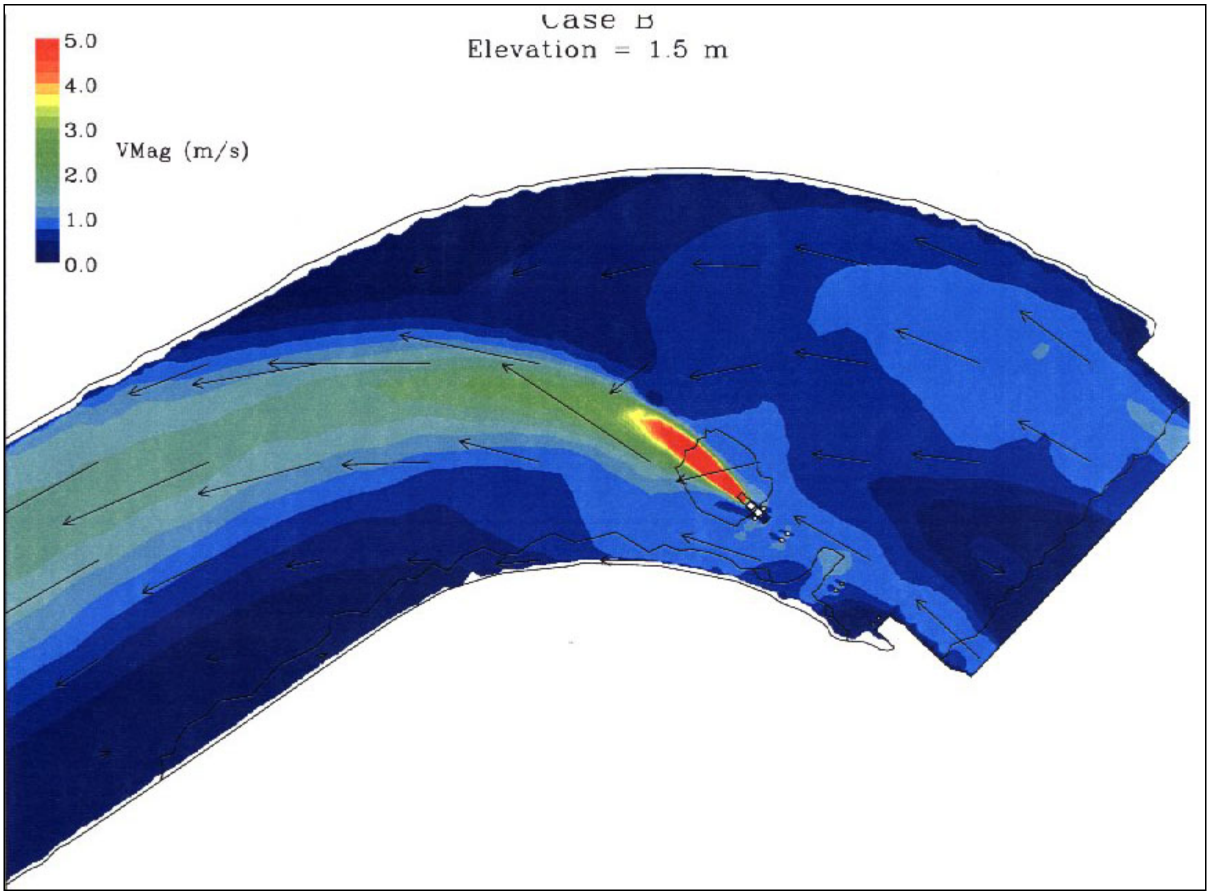


Figure 9. Case B – 2001 PNL CFD Data for Partial B2 Loading with Adjustable Cantilever Outfall at Range D (Close-up at Range D)

Comparing conditions with and without the outfall with B2 partially loaded (Figures 6 and 7 compared to Figures 8 and 9), the impact of the outfall is distinct. Where there had been no velocities greater than 1 m/s (3.3 fps anywhere in the B2 tailrace upstream from the tip of Bradford Island, the outfall will create a plume of 2 m/s (6.4 fps) and higher velocities that will extend throughout the same reach.

Along the Cascades Island shoreline, the velocities will be reduced to approximately 0-0.2 m/s (0-0.6 fps) as compared to the 0.8 m/s (2.4 fps) velocities, which occurred in this region without the outfall. Neither of these ranges of velocity magnitude, with or without outfall, would be adequate to prevent predator holding.

For the partial B2 loading case that was calculated, the presence of the outfall will create a distinct plume of higher velocities, greater than the 1.2 m/s (4 fps) required to prevent predator holding, that will be maintained downstream throughout the B2 tailrace. This will likely still be the case with different powerhouse loads distributions and re-orientation of the outfall to better center the plume in the B2 tailrace.

In conclusion, although the stretch of tailrace between Ranges D and F may have favorable NPM habitat at the shorelines, predator habitat is unfavorable in mid-river. The net effect of the adjustable cantilever outfall will be to create a distinct plume of higher surface velocities throughout the B2 tailrace between proposed outfall ranges D and F than would occur without the presence of the outfall discharge. Therefore, if the outfall at Range D can be designed so that its discharge plume stays away from shoreline, predator holding areas between ranges D and F will be avoided.

Expected Predation Levels and Survival Differences in Tailrace between Ranges D and F

We expect predation and, hence mortality, to be minimal in the 0.4 miles of B2 tailrace between Ranges D and F, as long as the outfall plume from Range D remains in mid-river and does not intersect shoreline areas. Relative or absolute differences in survival between Ranges D and F should be minimal if the discharge plume for Range D can be designed appropriately.

SUMMARY AND CONCLUSIONS

Survival – B2 tailrace survival data from studies in the late 1980s and early 1990s indicated that juvenile salmon died, probably from predation, in the B2 tailrace. Absolute levels of mortality, however, could not be estimated. The relative difference in recapture rates between test fish released in the front roll and those released at the site 2.0 miles downstream was –7%. When applied to Ranges D and F in the high flow outfall siting study, Dawley et al.'s (1998) parameters show similarities between the two sites, except for one parameter, egress distance. To summarize, it all comes down to the 0.4 mile distance between the two sites, out of 100 miles to the estuary, and how much predation is expected to occur in this specific stretch of tailrace.

Predation – Predators inhabit the B2 tailrace, especially shallow, rocky, shoreline areas. But, predator abundance and consumption of juvenile salmon at B2 tailrace have decreased since inception of the Predator Control Program. To be successful, the high flow outfall discharge must create a high velocity plume that does not come within at least 100 ft of B2 tailrace shorelines.
Expectations – We expect predation and, hence mortality, to be minimal in the 0.4 miles of B2 tailrace between Ranges D and F, as long as the outfall plume from Range D remains in mid-river and does not intersect shoreline areas.

LITERATURE CITED

Barfoot, C.A., D.M. Gadomski, and R.H. Wertheimer. 1999. Growth and mortality of age-0 northern pikeminnow.

Dawley, E. M. L. G. Gilbreath and R. D. Ledgerwood. 1988. Evaluation of juvenile salmonid survival through the Second Powerhouse turbines and downstream migrant bypass system at Bonneville Dam, 1987. Report to Portland District, U.S. Army Corps of Engineers ed.; 1988 Jan.

Dawley, E. M. L. G. Gilbreath R. D. Ledgerwood P. J. Bentley B. P. Sandford and M. H. Schiewe. 1989. Survival of subyearling chinook salmon which have passed through turbines, bypass system, and tailrace basin of Bonneville Dam Second Powerhouse, 1988. Report to Portland District, U.S. Army Corps of Engineers ed.; 1989 Aug.

Dawley, E. M. L. G. Gilbreath R. D. Ledgerwood P. J. Bentley and B. P. Sandford. 1998. Effects of bypass passage at Bonneville Dam Second Powerhouse. Report to Portland District, U.S. Army Corps of Engineers ed.; 1998 Dec.

ENSR. 2000. Final Report – Bonneville Second Powerhouse Tailrace ADCP Field Data Collection. Prepared for USACE Portland District through a subcontract to INCA Engineers. ENSR Document 3697-003-106A. March 2000.

Gilbreath, L. G. E. M. Dawley R. G. Ledgerwood P. J. Bentley and S. J. Grabowski. 1993. Relative survival of subyearling chinook salmon that have passed Bonneville Dam via the spillway or Second Powerhouse turbines or bypass system: adult recoveries through 1991. Report to Portland District, U.S. Army Corps of Engineers ed.; 1993 Jan.

Knutsen, C. J, R. H. Reeves, and K.A. Leader. 1996. Distribution and behavior of northern squawfish in Bonneville Dam forebay, 1996. Draft of Annual Report to the Bonneville Power Administration.

Ledgerwood, R. D. E. M. Dawley L. G. Gilbreath P. J. Bentley B. P. Sandford and M. H. Schiewe. 1990. Relative survival of subyearling chinook salmon which have passed Bonneville Dam via the spillway or the Second Powerhouse turbines or bypass system in 1989, with comparisons to 1987 and 1988. Report to Portland District, U.S. Army Corps of Engineers ed.; 1990 Jul.

Ledgerwood, R. G. E. M. Dawley L. G. Gilbreath P. J. Bentley B. P. Sandford and M. H. Schiewe. 1991. Relative survival of subyearling chinook salmon that have passed through the turbines or bypass system of Bonneville Dam Second Powerhouse, 1990. Report to Portland District, U.S. Army Corps of Engineers ed.; 1991 Oct.

Ledgerwood, L. G. E. M. Dawley L. G. Gilbreath L. T. Parker B. P. Sandford and S. J. Grabowski. 1994. Relative survival of subyearling chinook salmon at Bonneville Dam, 1992. Report to Portland District, U.S. Army Coprs of Engineers ed.; 1994 Dec.

Martinelli, T.L., and R.S. Shively. 1997. Seasonal distribution, movements and habitat associations of northern pikeminnow in two lower Columbia River reservoirs. Regulated Rivers: Research and Management 13: 543-556.

Mesa, M.G., and T.M. Olson. 1993. Prolonged swimming speed of northern pikeminnow. Transactions of the American Fisheries Society 122: 1104-1110.

Parker, R.M., M.P. Zimmerman, and D.L. Ward. 1995. Variability in biological characteristics of northern pikeminnow in the lower Columbia and Snake rivers. Transactions of the American Fisheries Society 124: 335-346.

Petersen, J.H. and five co-authors. 1993. System-wide significance of predation on juvenile salmonids in Columbia and Snake river reservoirs. Annual Report of Research for 1992 by the U.S. Fish and Wildlife Service to the Bonneville Power Administration, 42p.

Petersen, J.H. and six co-authors. 1991. System-wide significance of predation on juvenile salmonids in Columbia and Snake river reservoirs. Annual Report of Research for 1990 by the U.S. Fish and Wildlife Service to the Bonneville Power Administration, 53p.

Petersen, J.H., D.M. Gadomski, and T.P. Poe. 1994. Differential predation by northern squawfish on live and dead juvenile salmonids in the Bonneville Dam tailrace (Columbia River). Canadian Journal of Fisheries and Aquatic Science 51: 1197-1204.
Pikeminnow rearing in shoreline habitats of a Columbia River reservoir. Environmental Biology of Fishes 54: 107-115.

Poe, T. P., M.G. Mesa, R.S. Shively, and R. Peters. 1994. Development of biological criteria for siting and operation of juvenile fish bypass systems: implications for protecting juvenile salmonids from predation. Pages 169-176. K. Bates (ed.) In: Fish Passage Policy and Technology. Symposium proceedings. American Fisheries Society Annual Meeting. Portland, Oregon. September 1993.

Poe, T.P., H.C. Hansel, D.E. Palmer, and L.A. Prendergast. 1991. Feeding of predaceous fishes on out-migrating juvenile salmonids in John Day Reservoir, Columbia River. Transactions of the American Fisheries Society 120: 405-420.

Rieman, B.E., R.C. Beamesderfer, S. Vigg, and T.P. Poe. 1991. Estimated loss of juvenile salmonids to predation by northern pikeminnow, walleyes, and smallmouth bass in John Day Reservoir, Columbia River. Transactions of the American Fishetries Society 120: 448-458.

Vigg, S., T.P. Poe, L.A. Prendergast, and H.C. Hansel. 1991. Rates of consumption of juvenile salmonids and alternate prey fish by northern pikeminnow, walleye, smallmouth bass, and channel catfish in John Day Reservoir, Columbia River. Transactions of the American Fisheries Society 120: 421-438.

Ward, D.L., J.H. Peterson, J. J. Loch. 1995. Index of predation on juvenile salmonids by northern squawfish in the Lower and Middle Columbia River and the Lower Snake River. Transactions of the American Fisheries Society 124:321-334.

Zimmerman, M.P., T.A. Friesen, D.L. Ward, and H.K. Takata. 2000. Development of a systemwide predator control program: indexing and fisheries evaluation. Annual Report of Research for 1999 by the Oregon Department of Fish and Wildlife to the Bonneville Power Administration.

Appendix A – Relationship between Recovery Percentage and Survival Rate

The following example shows that the recovery percentage of marked fish at Jones Beach does not equate to an estimate of the absolute survival rate. Recovery percentage does reflect differences in relative survival as it appears the assumptions for equivalent recovery effort (detectability) at Jones beach were met. Thus, for the purpose of the B2 survival studies, recovery percentage was useful to determine the differential in survival between release locations, but not to show absolute survival rates, which the researchers clearly understood. The following example makes this point.

Assume 100,000 marked fish were released at the B2 front-roll and 100,000 at the downstream Hamilton Island release site. Also assume that the absolute survival rate to Jones Beach was 20% for the front-roll fish and 25% for the downstream fish – a 5% difference in absolute survival rate. This means 20,000 front-roll fish and 25,000 downstream fish survived to Jones Beach.

Next, assume equivalent recovery effort (detectability) of 1%. So, 1% x 20,000 = 200 front-roll fish and 250 downstream fish were recovered. Therefore, the recovery percentages would be:

Front-roll: (200/100,000) x 100 = 0.20%

Downstream (250/100,000) x 100 = 0.25%

Then, the percentage difference of front-roll from downstream would be:

(FR-DS)/DS x 100 = (0.20-0.25)/0.25 x 100 = -20%

In conclusion, a difference in recovery percentage reflects relative survival differences, but cannot be used to estimate absolute survival.

Appendix E

Adjustable Cantilever Conceptual Design

B2 Corner Collector
Adjustable Cantilever Update
Draft Letter Report

1. Introduction

The primary objective of this letter report is to investigate the feasibility of an adjustable cantilever outfall for the Bonneville Second Powerhouse Corner Collector Juvenile Bypass System. The secondary objective of this letter report is to revise the estimate costs for the eight configurations tabulated below. New geotechnical information complicates the construction of plunge pools. These costs have been addressed as well as further definition of the outfall alternatives.

Outfall Cantilever	Location	Location	Location
Adjustable (IE 11 - 22)	D	E	F
Classic (IE 26)	D	E	
Mid-Level (IE 16.5)	D	E	F

2. Outfall Description

The adjustable cantilever is an outfall element that connects to the transportation channel, an elevation view is shown in the Sketches portion of this report. This element may be used as the outfall at any of the discharge locations, without modification. The element is 105 feet long and is comprised of a 70-foot backspan and a 35-foot cantilever. The ratio of backspan to cantilever length was selected to balance the global positive and negative bending moments. As the design is further developed, this configuration will be investigated further.

Two structures were investigated for this report, a concrete section (described further in the Section 3 of this report) and a steel section (described further in Section 4 of this report). The construction and installation of the steel adjustable section is easier due to its light weight. The concrete design has cost advantages and may be less susceptible to dynamic and fatigue problems. It is recommended that both designs be developed to the DDR stage in order to evaluate which system has the best overall value.

The upstream end of the adjustable cantilever is supported by a hinge on the transportation channel. This hinge will be capable of resisting the vertical load due to the self weight of the structure, the internal live load, and the longitudinal component of the stream flow load, while providing 6 degrees of rotation capacity. A seal, as described in Section 5, would provide watertight integrity between the transportation channel and the outfall section.

The downstream end of the adjustable cantilever is supported by a hoisting system. For this report, two systems were investigated, one that uses tension cylinders and one that uses compression cylinders. Both hoist systems and associated dogging provisions are discussed in Section 5 of this report.

3. Concrete Outfall

The concrete adjustable outfall has a channel width of 15 feet and a channel depth of 20 feet. The invert, crown and walls are 18-inches thick and are shown in Section 8 of this report. The structure is a post-tensioned box beam with grouted tendons. Mild reinforcement was used to resist the out-of-plane bending in the invert, walls, and crown. Temperature and shrinkage steel was included in the longitudinal direction.

The design was investigated in accordance with the requirements of EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures*, and EC 1110-2-XXXX, *Structural Design of Precast and Prestressed Hydraulic Concrete Structures*. At this level of investigation the design was not optimized; the structure shown is structurally feasible and provides the opportunity for potential cost reductions.

The 28-day compressive strength, f'c, of the concrete used in the design is 5,000 psi. The yield strength of the mild reinforcement is 60 ksi and the pre-stressing is 1/2-inch diameter, 270 ksi, and low relaxation strand.

4. Steel Outfall

The steel adjustable outfall also has a channel width of 15 feet and a channel depth of 20 feet. The invert, crown and walls are 5/8-inch CRES skin plate with carbon steel stiffeners spaced 2-foot on center. The stiffeners consist of 5/8-inch by 12-inch webs and 1-inch by 6-inch flanges, welded in a T configuration.

The design was investigated in accordance with the requirements of EM 1110-2-2105, *Design of Hydraulic Steel Structures*. As is the case for the concrete outfall structure, the design was not fully optimized. The structure shown in the Sketches section of this report is structurally feasible and provides opportunity for structural optimization.

The following issues need to be addressed in future design phases:

- The size of the channel's cross section is required for hydraulic reasons not structural. The 15-foot wide and 20-foot high box-type channel is very strong for the relatively short span length of the structure; therefore, the global flexure does not govern the design. The high velocity of flow through the channel, and the hydraulic forces due to submergence in the tailrace are issues that will have to be addressed in the DDR. The dynamic behavior investigation of the structure was deferred to the DDR. In the feasibility phase, the selection of the skin plate thickness and the spacing of the ring stiffeners was governed by the local load cases.

- It appears advisable to investigate the dynamic behavior of the structure closely in the final design stage, and to adjust the design for low vibratory response of the structure. Since physical (i.e., hydraulic and structural) modeling may not capture all circumstances of the actual structure and loads, the final design of the channel should adapt features that would allow the fine-tuning of the structure in the final phase of the construction.

- If the fisheries and hydraulic issues can be reconciled with the structural needs, a circular section would require significantly less plate material and welding than the current box-type cross section.

- The use of carbon steel for the interior surfaces of the flume may be problematic due to corrosion of the surface from high water velocities. It is anticipated that future design will use all stainless steel (perhaps 300 series) material since stainless steel plate material of 1/2-inch thickness is obtainable with good mechanical properties and requires a minimum of maintenance. As an alternative to the all stainless steel, the merit of cladded steel plates for use as skin plate was investigated. For details see below. "Cladded" means that 1/8-inch thick stainless steel is roll-bonded to carbon steel in a specialized mill process. The benefit of cladded steel is best appreciable when corrosive internal conditions and high internal pressure exist. These conditions require a combination of a stainless steel lining and high tensile strength (up to 6-inch plate thickness) of carbon steel. These conditions are not present here, and the use of the cladded steel would not eliminate the need for corrosion inhibiting coatings and future maintenance efforts. In addition, welding with different weld consumables (stainless and carbon steel at the same joint) is cumbersome and not cost efficient.

5. Mechanical Systems

5.1 Summary of Movable Cantilever Outfall Mechanical Systems

In this section, the feasibility and cost of mechanical systems necessary to raise and lower the cantilever outfall structure is addressed. All mechanical system concepts were based on a movable outfall structure 105 feet long, which is simply supported by the fixed outfall structure on one end and by a hydraulic cylinder hoist system on the other. The span between supports was selected as 70 feet with the additional 35 feet of movable outfall structure cantilevered past the hoist system. Both steel and concrete outfall structures were investigated.

Two different hoist concepts were developed, the first using a pair of cylinders operating in tension, and the second using a pair of cylinders in compression. Both concepts included a separate structure component and dogging pin system to hold the full dead and live load of the outfall when the system is not being moved.

Based on the concepts developed for this report, a hydraulic cylinder hoist system can be developed that will move either a concrete or steel cantilever outfall structure. The critical lift cylinders that form the core of these hoist systems are large, but are well within the capabilities of most specialty hydraulic cylinder manufacturers. Similar hydraulic hoist systems are currently used in movable bridges, locks and dams across the United States and Europe. Washington State Ferries currently operates three overhead passenger-loading systems that utilize similar lift cylinders and an automated dogging pin system. One of these systems has been in nearly continuous operation since about 1990 with minimal maintenance issues.

5.2 Assumptions and Design Basis

The following paragraphs summarize the assumptions and preliminary design basis that were used to develop the two hoist system concepts contained in this report.

Assumptions:

- Hydraulic cylinders will be used as the principal actuation means of the outfall.
- Flow through the outfall will be reduced to a depth of approximately 6 inches when the movable cantilever section is to be adjusted. This will require closing the gate.
- Total range of motion (stroke) of the hydraulic cylinder hoist will be approximately 10 feet.
- The cantilever outfall section may be fabricated using steel or concrete.
- Total hoisted load for the cylinders will be on the order of 588 kip (steel outfall) or 1,391 kip (concrete outfall)

Design Basis:

- Structural components will be designed to the requirements of EM 1110-2-2105, *Design of Hydraulic Steel Structures*.
- Hydraulic system design shall be per AASHTO *Load and Resistance Factor Design for Movable Highway Bridges* (with modifications).
- Full hoist system redundancy will be provided -- no single failure of a hoist system component (structural or hydraulic) shall cause the outfall to be dropped in an uncontrolled manner.
- Rated pressure for all hydraulic components shall be 4,500 psi.

- System design pressure will be 3,000 psi (for all normal operating conditions).
- System normal working pressure shall be two-thirds the design pressure or 2,000 psi.
- Maximum system pressure with one hydraulic cylinder failed shall be rate pressure (4,500 psi).

5.3 Outfall Hoist System

Two hoist system concepts were developed based on the assumptions and design basis presented in the previous section. Both concepts use a pair of hydraulic lift cylinders to move the cantilever outfall structure and rely on a separate structural component to support the complete dead and live load of the outfall when the structure is not being moved. Each concept is explained in detail below and sketches of the concepts are provided in Section 8 of this report.

5.3.1 Concept 1 -- Tension Lift Cylinders

This concept uses lift cylinders in tension to hoist the outfall structure. Each cylinder is capable of supporting the full load of the moving outfall structure (and any transient dynamic loads) in order to provide full structural redundancy in the hoist system. The cylinders are positioned on the centerline of the outfall such that a cylinder failure will not apply torsion to the outfall structure. The cylinders are trunnion mounted and supported by a compact trunnion support frame (bracket) affixed to the lift platform structure.

Two simple hanger bars provide the independent structural component to support the full live and dead load of the outfall when the system is not being repositioned. The hanger bars are attached to the outfall structure and are kept in position by guide shoes mounted to the trunnion bracket. Dog pin assemblies mounted to the trunnion bracket contain pins that extend and engage holes in the hanger bars to support the live and dead loads. The dog pins are actuated with small hydraulic cylinders.

The principal advantage of this system over the compression cylinder concept is that the hydraulic cylinder rod can be kept small (no buckling concerns) and that the hanger bar and its associated guide system can be quite simple. The primary disadvantages of this concept are that catastrophic fatigue failure of the tension cylinder rod can occur, and the required cylinder bore will be larger than an equivalent compression cylinder due to the cylinder rod area.

5.3.2 Concept 2 -- Compression Lift Cylinders

This concept uses compression lift cylinders to hoist the outfall structure. The cylinders operate on a large lift frame, which is supported and guided by a trunnion mounted guide structure affixed to the lift platform. Structural redundancy for this system can be provided either by sizing the cylinders to carry the full load of the moving outfall structure (and any transient dynamic loads) or by designing the guide frame bearing system to be self-locking if large eccentric loads are applied by the cylinders (i.e. one cylinder failed).

The lift frame in this concept supports the full live and dead load of the outfall when the system is not being repositioned. Dog pin assemblies mounted to the guide structure contain pins that extend and engage holes in the lift frame to support the live and dead loads. The dog pins are actuated with small hydraulic cylinders.

The principal advantage of this system is that catastrophic fatigue failure of a hydraulic cylinder rod is not a concern (rod is in compression) and that a smaller overall cylinder bore is possible. The primary disadvantages are that the lift frame and guide structure are more complicated than the simple hanger bars used in Concept 1 and cylinder buckling is possible and must be accounted for in the cylinder design.

5.4 Alternate Hoist Systems

During the preparation of the DDR it would be prudent to further explore alternative hoist systems. The USACE has a long history of successful use of winch and wire rope systems, usually on tainter and vertical lift gates. While preliminary investigation of a winch and rope hoist concluded that a hydraulic system was more advantageous the winch/rope hoist system should be evaluated in the DDR. Other systems such as sector gears, rack and pinion hoists or an ACME screw hoist configuration could be further investigated during the preparation of the DDR.

5.5 Outfall Pivot Bearing

The upstream support and pivot for the movable outfall structure can be handled in a number of different ways. One simple approach is to attach a pair of yokes similar to those used on tainter gates to the fixed (non-movable) portion of the outfall structure and install a mating pair of trunnion bearings to the end of the movable outfall structure. Thrust bearings between the yoke and the trunnion structure can be used to provide positive lateral alignment of the fixed and moving portions of the outfall structure. Given the expected loads, trunnion pin diameter will be around 8 to 10 inches. Self-lubricating composite bearings (or other maintenance free bearing types) may be advisable because of the remote installation location.

5.6 Outfall Seal System

The joint between the fixed and movable portions of the outfall structure can also be approached several different ways. This seal must prevent leakage of the outfall flow, provide for smooth flow transition from the fixed to the movable portion, and must be durable enough to resist ice and trash that is transported through the outfall. One simple approach to this problem is to utilize fiber reinforced polymer or elastomer sheet to form blade seals. These seals are attached to the upper (non-moving) portion of the outfall and overlap onto the movable outfall structure. The bottom seal would be allowed to deform and follow the contour of the outfall floor. The side seals would be supported by the vertical wall of the movable outfall. Both seals would work much like the shingles on a roof.

6. Construction Considerations

6.1 Shaft Construction

Shaft construction is limited by the amount of work that can be accomplished during an In Water Work Period (IWWP). An outfall at location D requires the construction of eight drilled shafts (four pairs) which can be accomplished during a single IWWP. At location E there are too many shafts to complete in a single IWWP, this creates scheduling problems if the outfall must be operational in 2004.

6.2 Outfall Construction

The DDR should address the installation of the adjustable cantilever. If a steel design is selected the lifting weights are lower. For the concrete design the weights are quite high. In addition to lifting the structure, it must be maneuvered between the support shafts at the downstream end. It may be possible to bulkhead the ends of the structure and float it into position.

6.3 Plunge Pool Construction

The presence of a sand and silt layer, approximately 20 feet thick, containing of mica sands will require special handling. These sands are also called crystal sands and are characterized by their plate-shaped particles, extremely high sensitivity, and lack of residual strength. It is anticipated that extensive measures will be required to prevent this layer becoming mobilized. To prevent this from happening, a configuration of sheet piling has been developed to facilitate a staged excavation. This configuration is shown in Section 8 of this report.

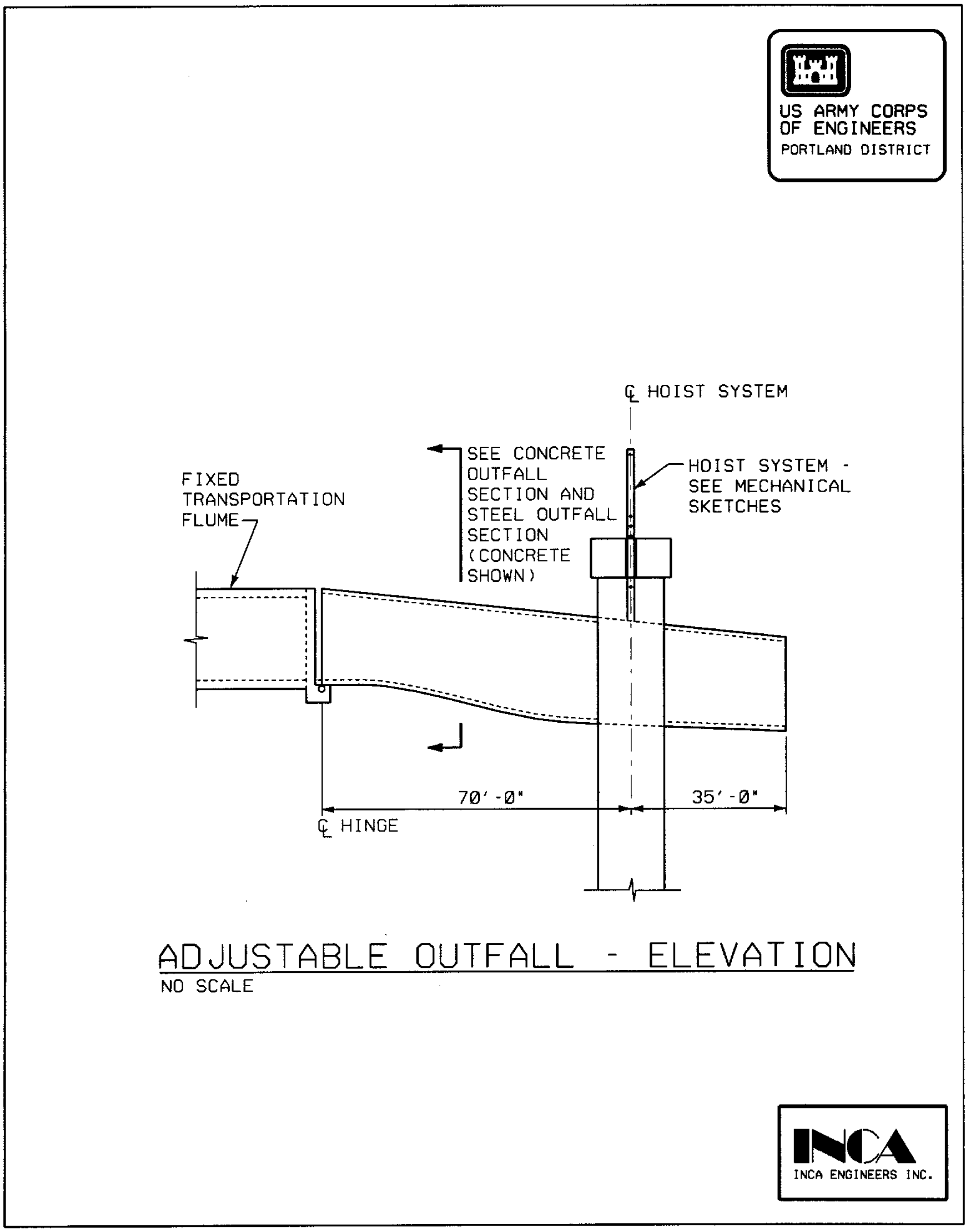
The general plan is to drive a set of parallel sheet pile walls to retain the potentially flowing material. The native material would be excavated with a row of sheet piling providing temporary support. Riprap fill would then be installed to create a final slope of 2 horizontal to 1 vertical. A second row of sheet piling would be installed and the process repeated. The current thinking is that vertical steps, approximately 20 feet high, could be established and backfilled using this method. The number of steps required depends on the total depth of the plunge pool. As the Corner Collector Outfall is further developed the depth and spacing of the sheet pile walls would be developed in conjunction with the plunge pool requirements.

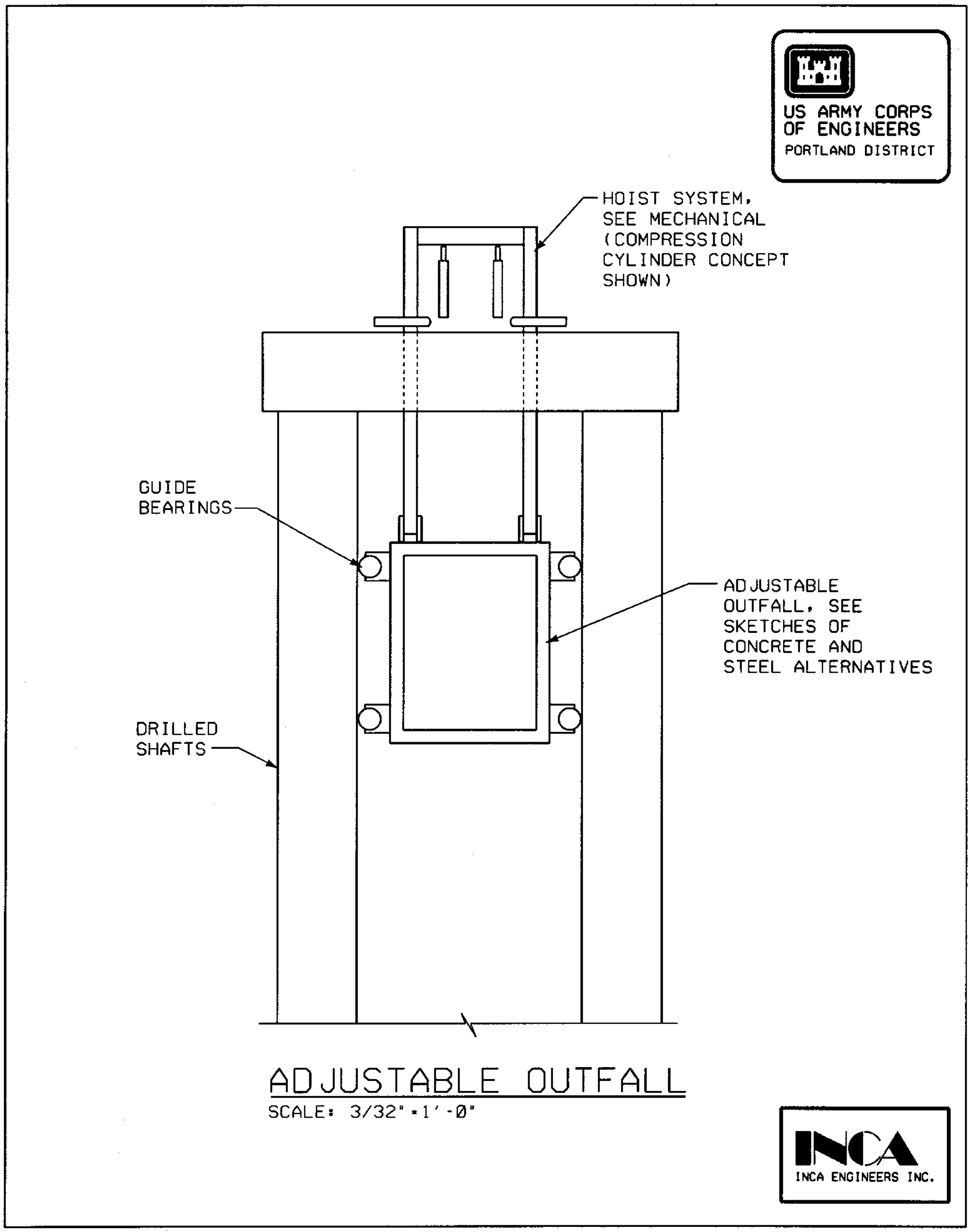
7. Estimated Costs

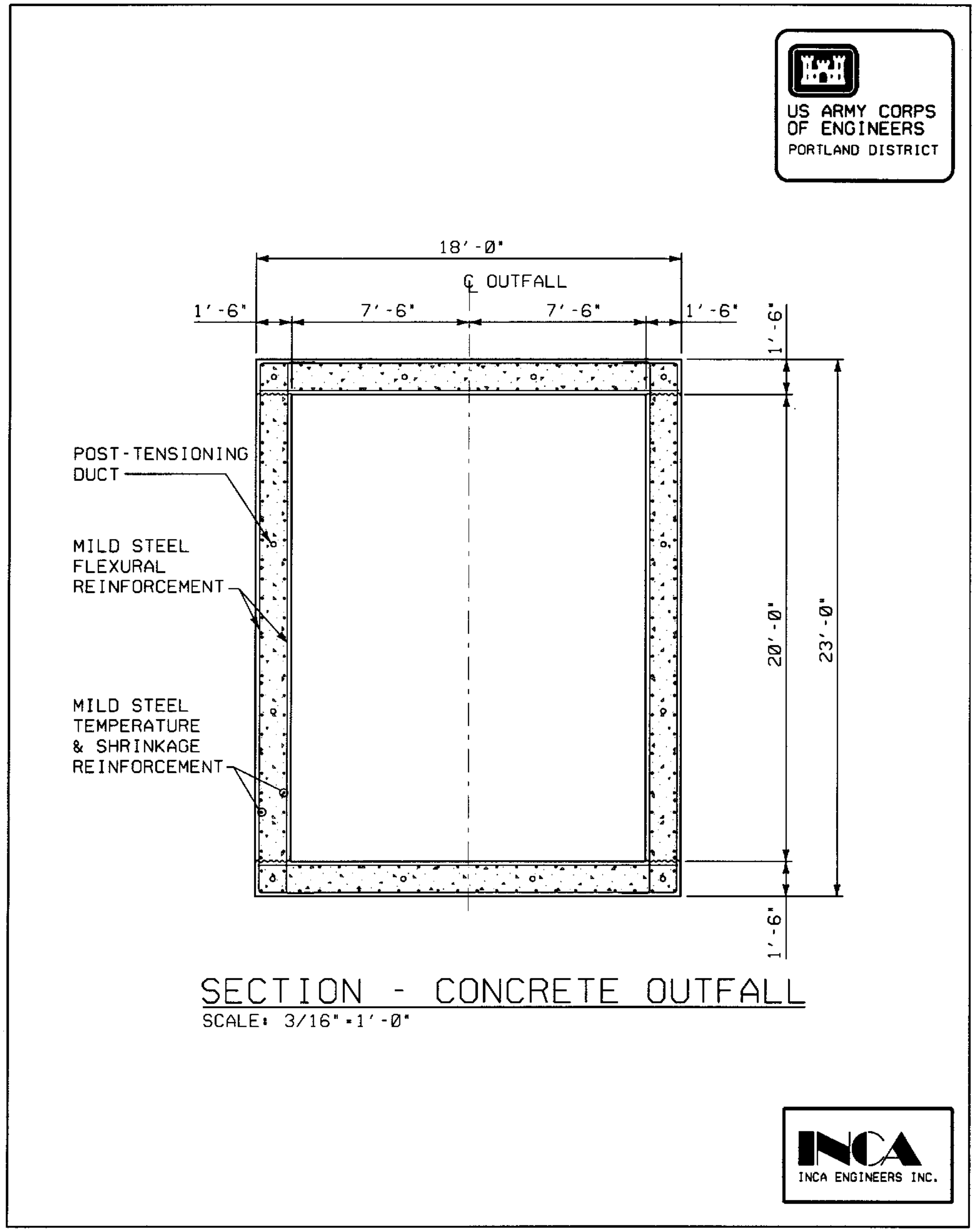
The cost estimate from the B2 Corner Collector Outfall Alternatives Study has also been updated. The biggest change in costs is for the plunge pools. Recent geological investigations indicate that sheet pile will be necessary to retain the dredged slope at the required angle. These sheet piles need to be driven underwater. Also additional structural fill will be required to provide the 2 horizontal to 1 vertical slope. An additional cost issue for the plunge pools is the disposal of dredge spoils. The cost estimate is based on $10 per cubic yard for disposal in the river. If upland disposal is required the cost is about $80 per cubic yard. This is a $7 million cost impact for a 50-foot deep plunge pool without profit or contingencies. The issue of disposal should be addressed in the DDR. Cost estimates for alternatives assume a 40-foot plunge pool depth for the classic cantilever. The midlevel is 30 feet and the adjustable is 15 feet. These depths are subject to change as hydraulic testing continues.

A new cost estimate was developed for the adjustable cantilever and for the fixed concrete structure at Range D. These are the first costs developed for the adjustable cantilever. The unit cost developed for the fixed concrete structure is very close to those used in previous estimates (B2 Corner Collector Outfall Alternatives Study). The unit costs developed for Range D were used to develop the estimated costs for Ranges E and F. A summary of the costs for the outfalls is presented in Table 1.

8. Sketches


US ARMY CORPS OF ENGINEERS
PORTLAND DISTRICT
℄ HOIST SYSTEM
SEE CONCRETE OUTFALL SECTION AND STEEL OUTFALL SECTION (CONCRETE SHOWN)
HOIST SYSTEM - SEE MECHANICAL SKETCHES
FIXED TRANSPORTATION FLUME
70'-0"
35'-0"
℄ HINGE
ADJUSTABLE OUTFALL - ELEVATION
NO SCALE
INCA ENGINEERS INC.

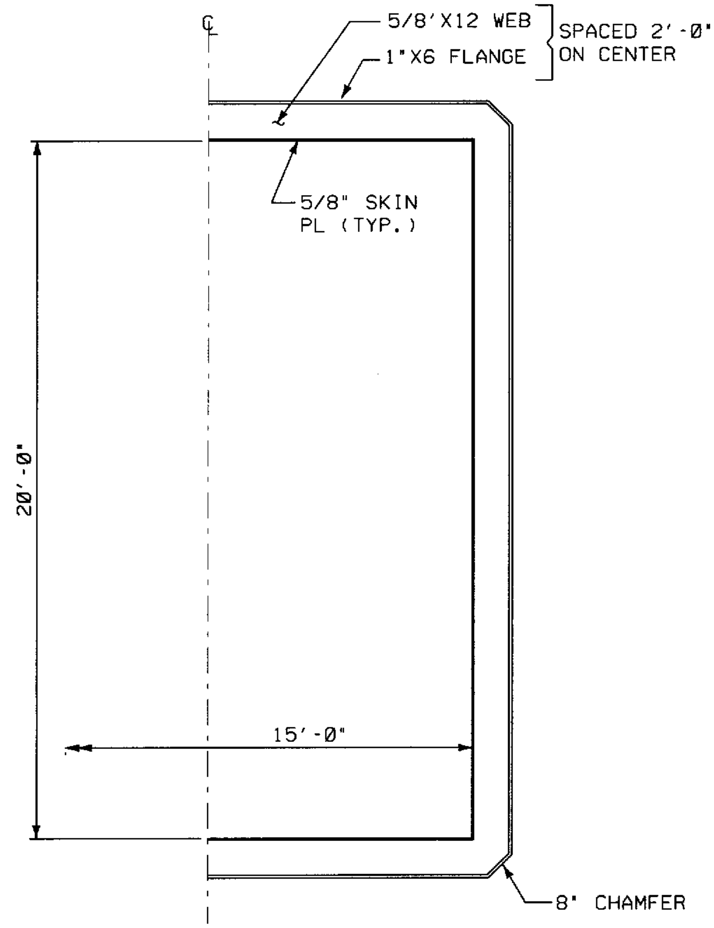

US ARMY CORPS
OF ENGINEERS
PORTLAND DISTRICT
HOIST SYSTEM,
SEE MECHANICAL
(COMPRESSION
CYLINDER CONCEPT
SHOWN)
GUIDE
BEARINGS
ADJUSTABLE
OUTFALL, SEE
SKETCHES OF
CONCRETE AND
STEEL ALTERNATIVES
DRILLED
SHAFTS
ADJUSTABLE OUTFALL
SCALE: 3/32"=1'-0"
INCA
INCA ENGINEERS INC.



SECTION - CONCRETE OUTFALL

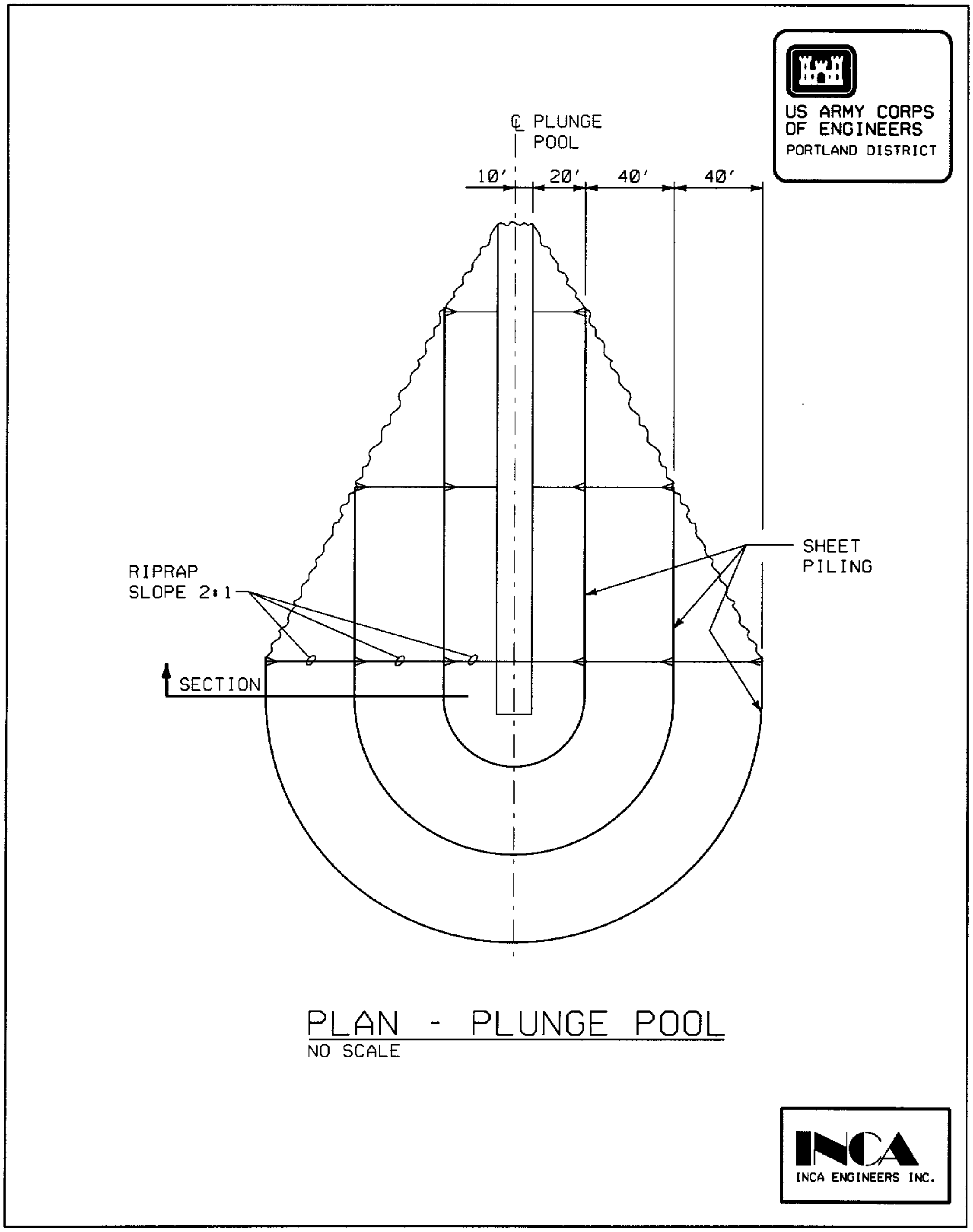
SCALE: 3/16"=1'-0"

US ARMY CORPS
OF ENGINEERS
PORTLAND DISTRICT

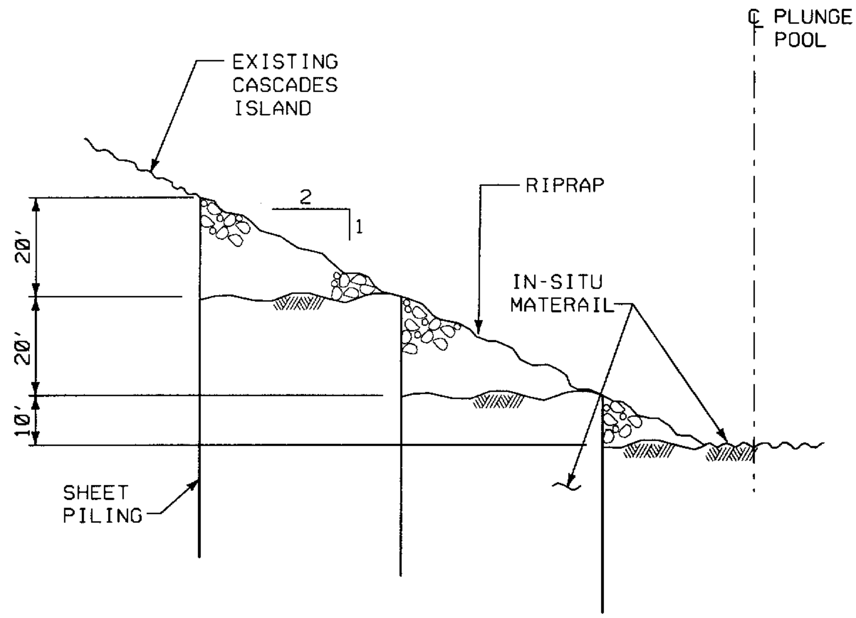


SECTION - STEEL OUTFALL
SCALE: 1/4"=1'-Ø"

INCA
INCA ENGINEERS INC.


US ARMY CORPS
OF ENGINEERS
PORTLAND DISTRICT
℄ PLUNGE
POOL
10'
20'
40'
40'
SHEET
PILING
RIPRAP
SLOPE 2:1
SECTION
PLAN - PLUNGE POOL
NO SCALE
INCA ENGINEERS INC.

US ARMY CORPS
OF ENGINEERS
PORTLAND DISTRICT



SECTION - PLUNGE POOL
SCALE: 3/32" = 1'-0"

INCA ENGINEERS INC.

HARTFORD
ENGINEERING

PROJECT: **PROJ. #** 1024a

CONTRACT / CLIENT: DACW 57-97-D-0003 / CENWP

DESIGN TOPIC: LIFT SYSTEM - TENSION CYL

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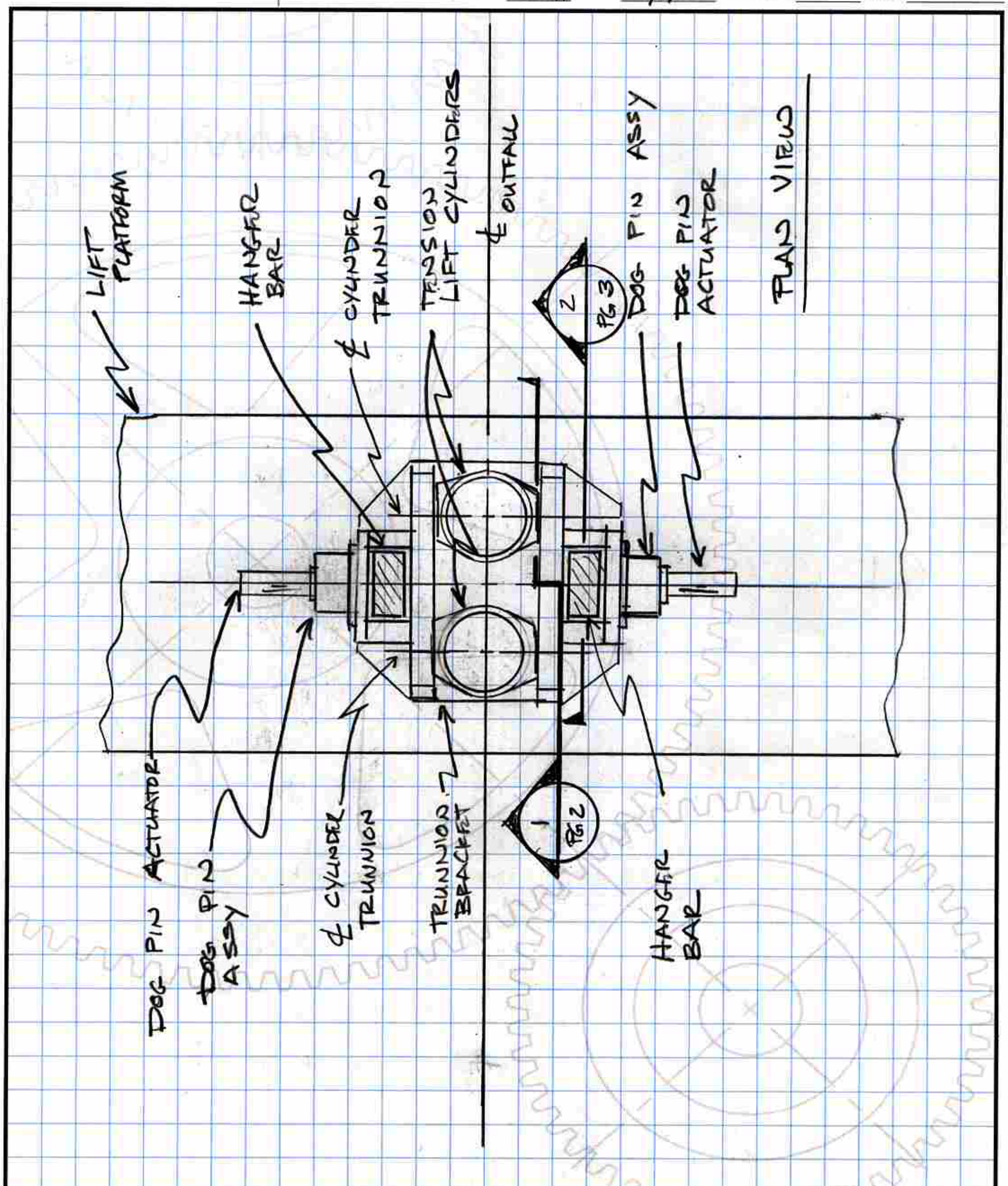

PAGE 1 OF 3

HARTFORD ENGINEERING

PROJECT: ______ PROJ. # ______

CONTRACT / CLIENT: DACW57-97-D-0003 /CENWP

DESIGN TOPIC: LIFT SYSTEM - TENSION CYL

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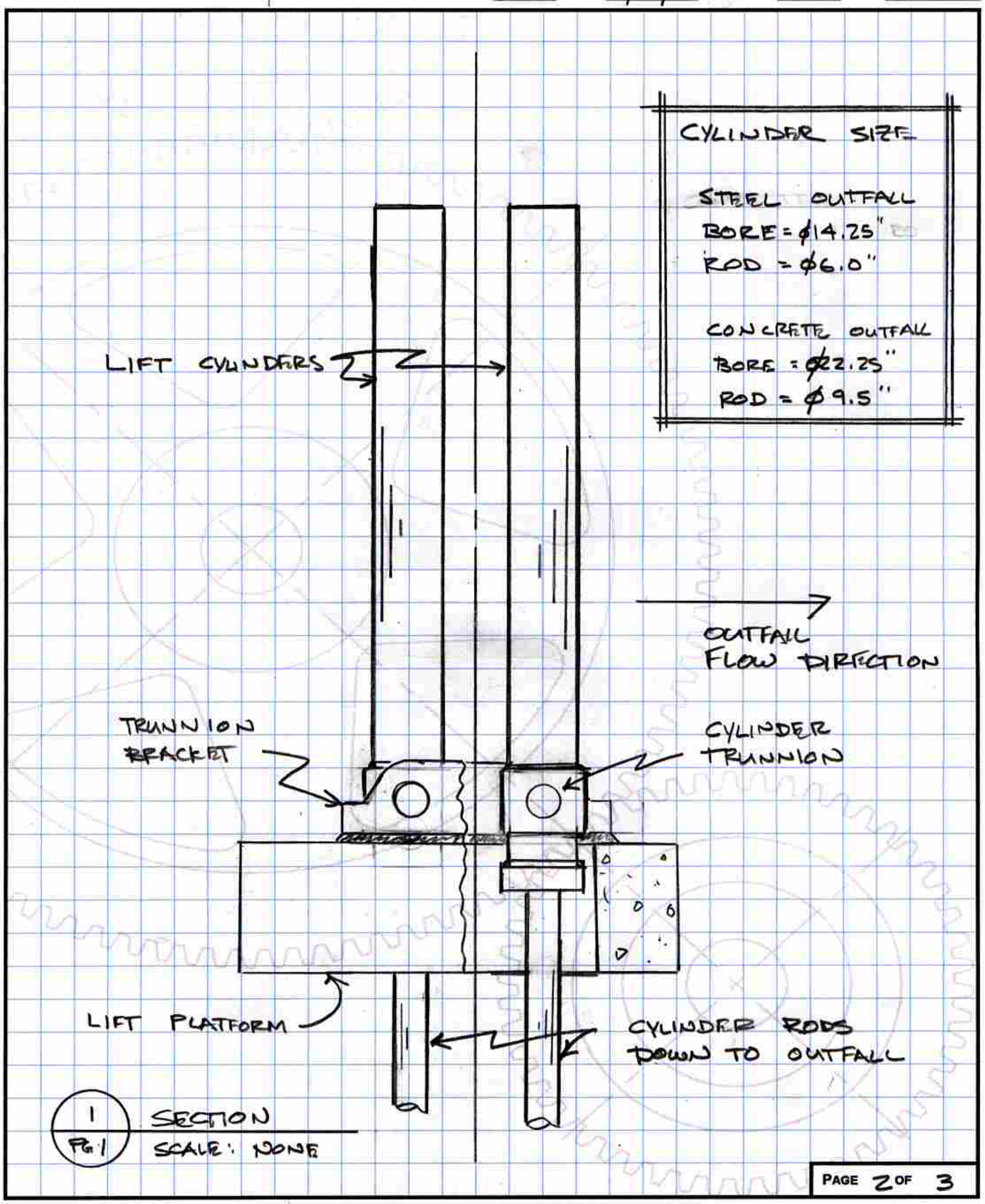

PROJECT: PROJ. #

CONTRACT / CLIENT: DACW 57-97-D-0003 / CENWP

DESIGN TOPIC: LIFT SYSTEM - TENSION CYL.

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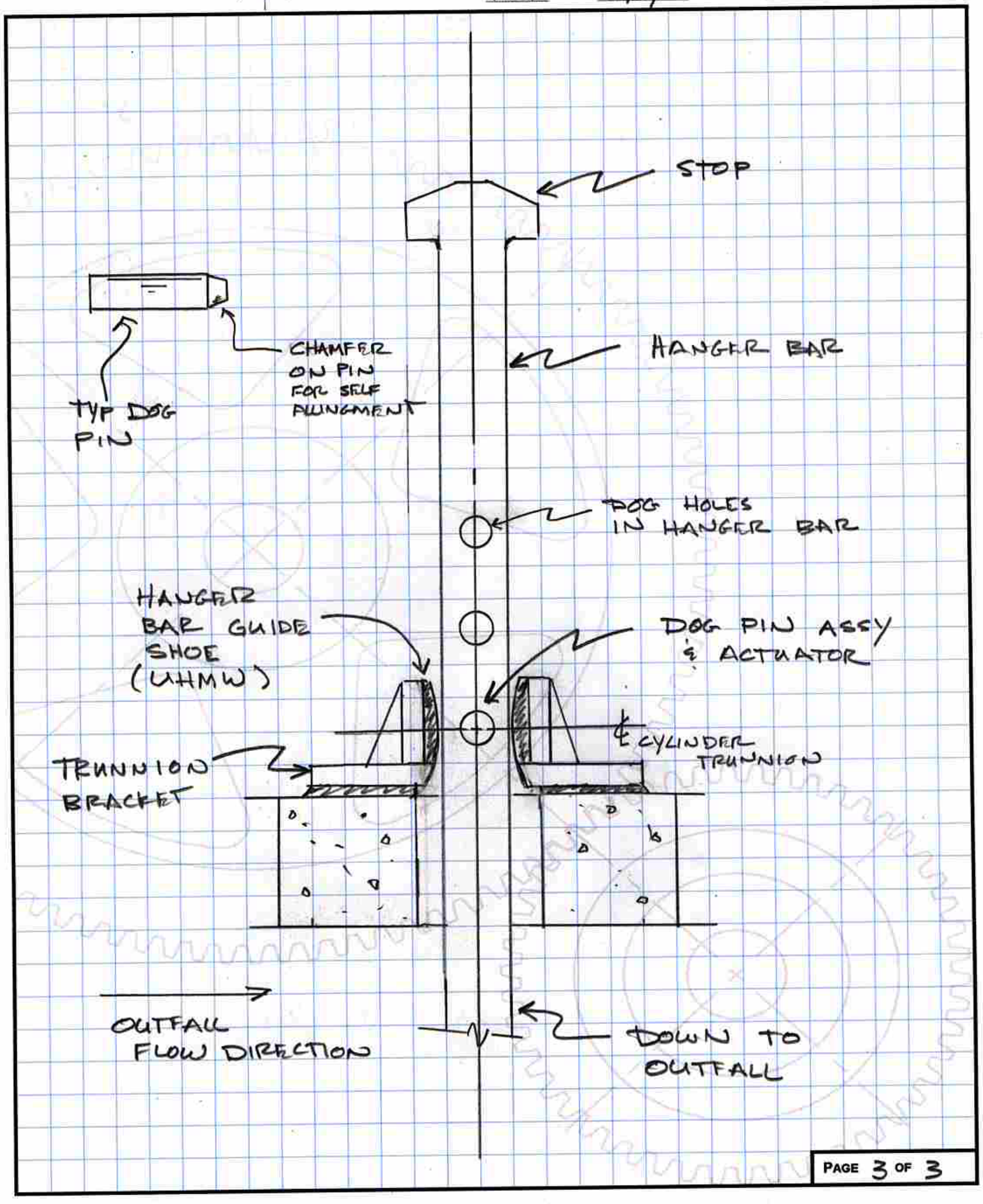


PAGE 3 OF 3

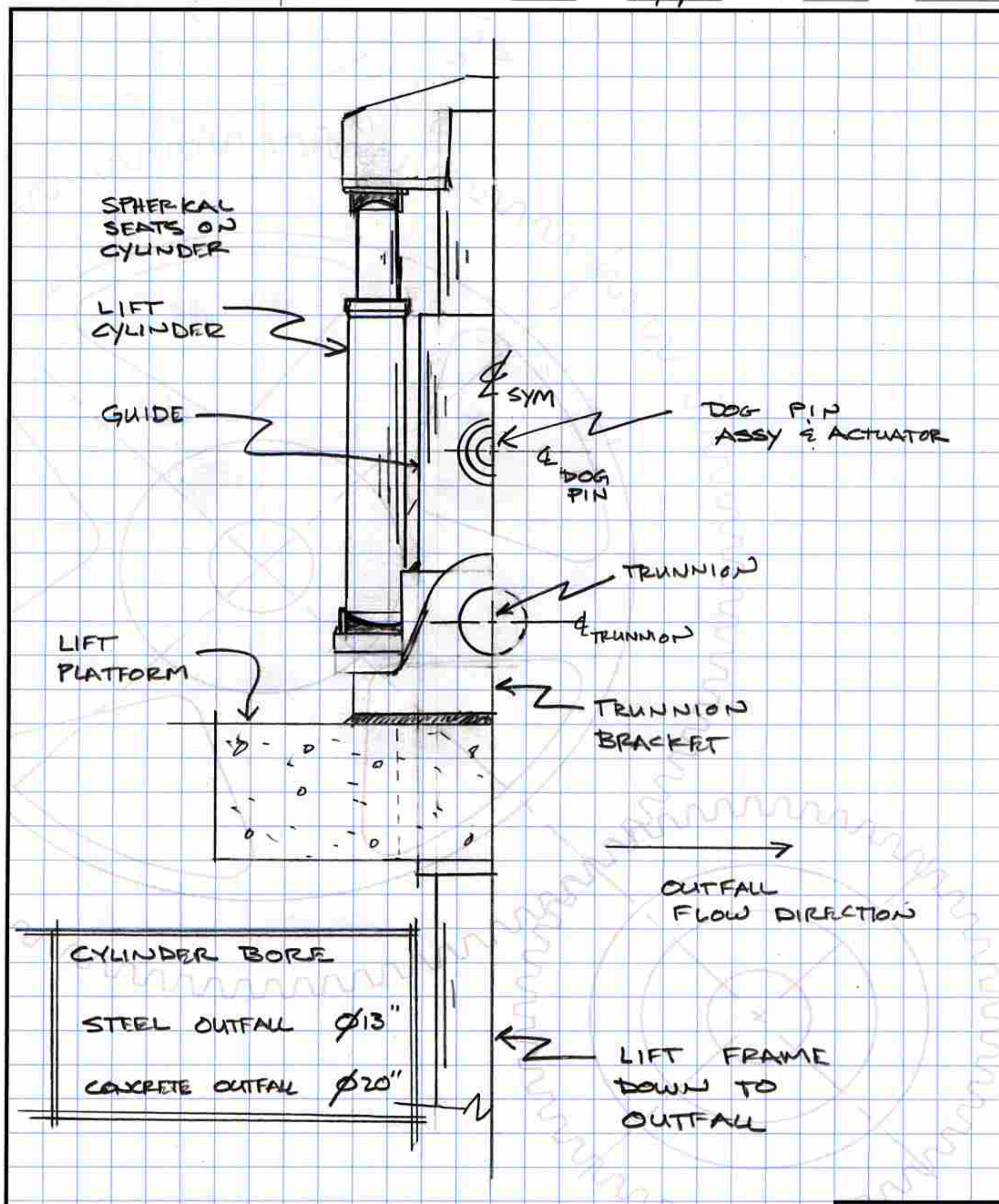
HARTFORD
ENGINEERING

PROJECT: PROJ. #

CONTRACT / CLIENT: DACW 57-97-D-0003 / CENWP

DESIGN TOPIC: LIFT SYSTEM – COMPRESSION CYL

MADE BY: DWH DATE: 12/8/00 CHECK BY: DATE:



PAGE 1 OF 2

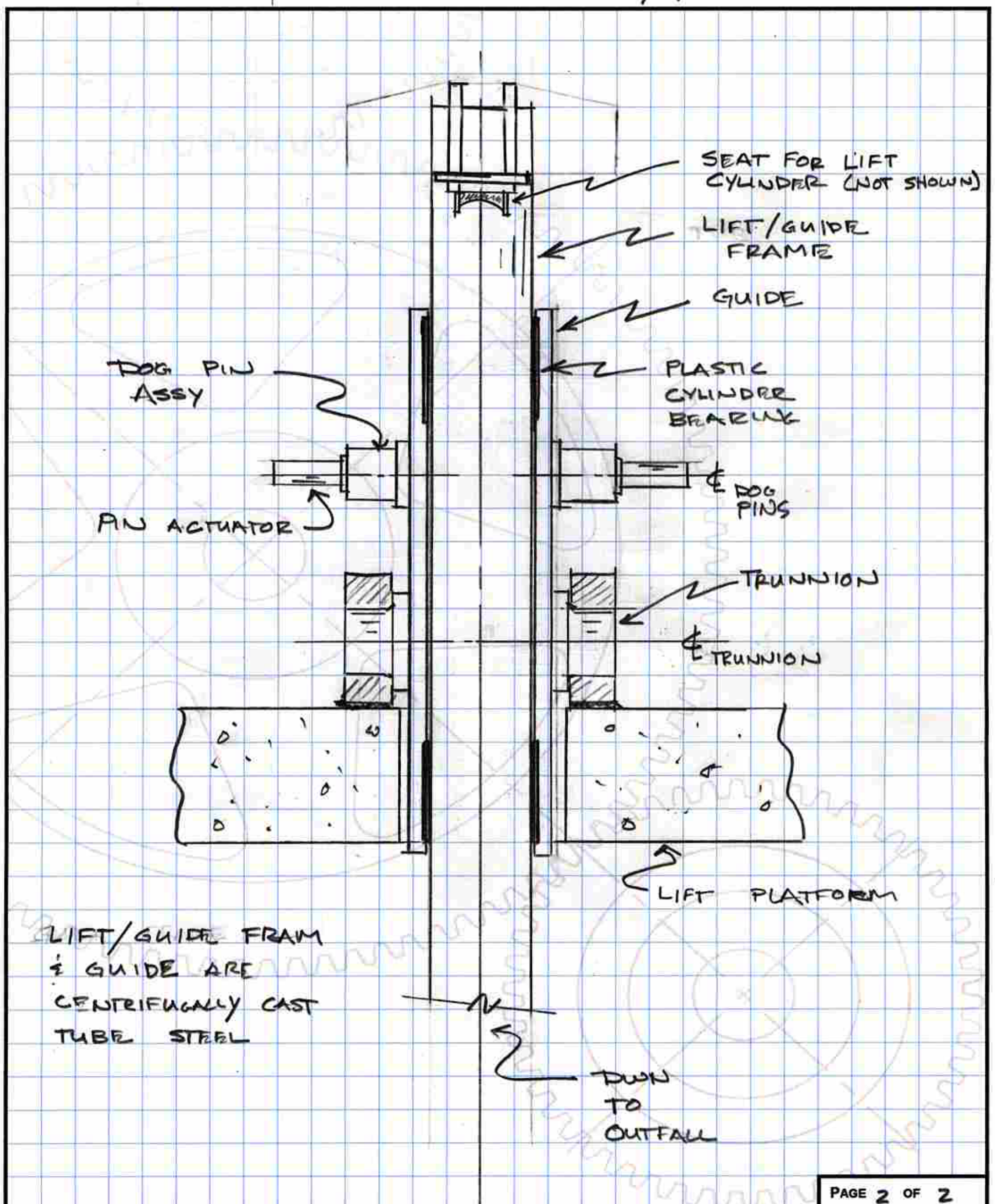
HARTFORD ENGINEERING

PROJECT: PROJ. #

CONTRACT / CLIENT: DACW 57-97-D-0003 / CENWP

DESIGN TOPIC: LIFT SYSTEM - COMPRESSION CYL.

MADE BY: DWH DATE: 12/08/00 CHECK BY: DATE:



PAGE 2 OF 2

Appendix F

Cost Analysis

Appendix G

Comments and Responses