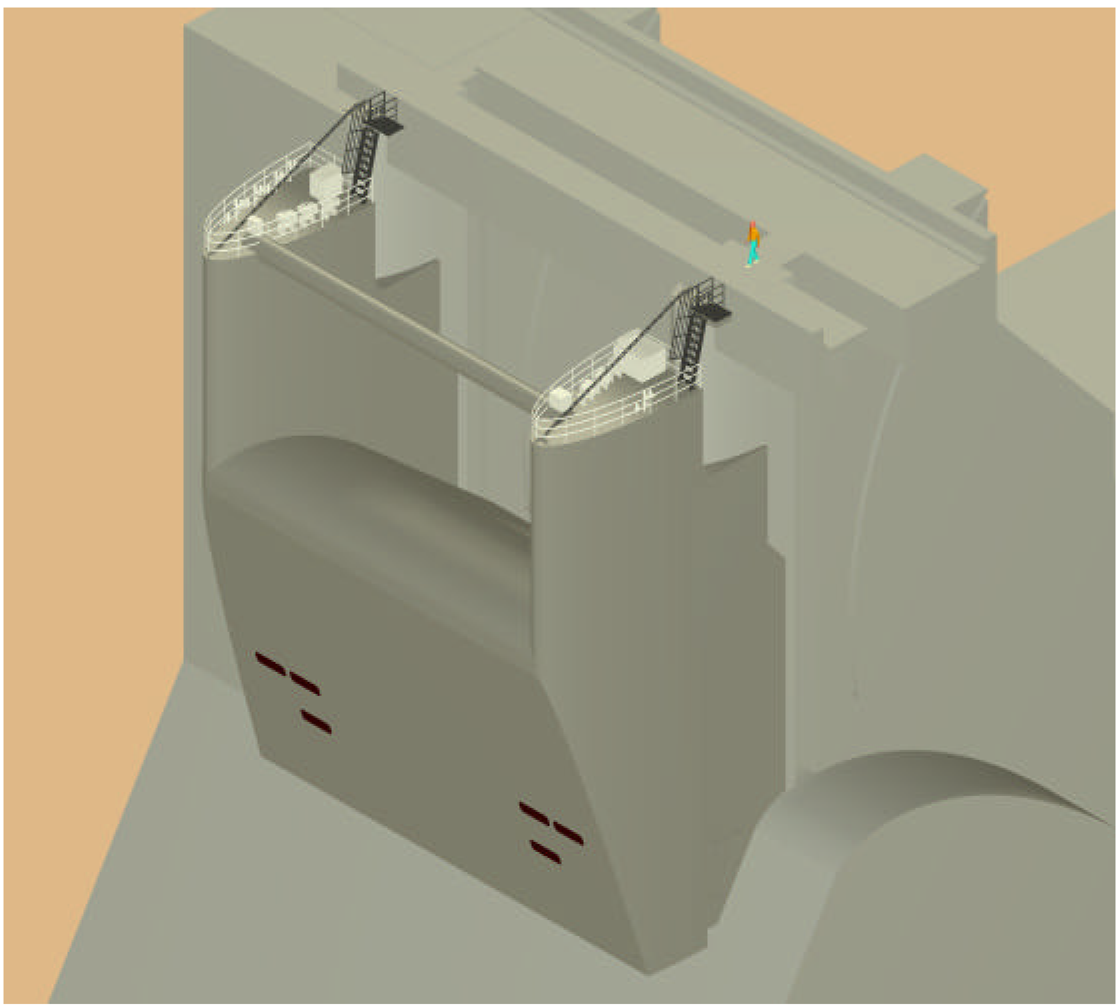
US Army Corps
of Engineers ®
Portland District

Design Documentation Report
No. 53

John Day Lock and Dam Removable Spillway Weir



October 2001

JOHN DAY SURFACE BYPASS
REMOVABLE SPILLWAY WEIR

DESIGN DOCUMENTATION REPORT
NO. 53

OCTOBER 2001

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EXECUTIVE SUMMARY

This Design Documentation Report (DDR) contains information from design work through the 90% point of plans and specifications development. At this design point it was decided to divide the project into two pieces. The first contains the spillway deflector and its dewatering stop logs. The second contains the RSW Main Structure, Main Structure support and tailpiece. This Design Documentation Report is for the RSW and its pertinent features. The spillway deflector described in Supplement No. 1 to Design Memorandum No. 50.

The John Day Dam Project, operated and maintained by Portland District, U.S. Army Corps of Engineers (CENWP), is located approximately 95 miles east of Portland, Oregon, at River Mile 215.6 on the Columbia River. The John Day Powerhouse began operation in 1968. The powerhouse has been modified in recent years to enhance the downstream migration of juvenile salmonids. Enhancements to the existing Juvenile Bypass System (JBS) in addition to surface collection are being studied concurrent with the work done in this report.

In addition to screened bypass systems the region has initiated studies into spill programs to enhance non-turbine passage of juvenile salmonids. Beginning in 1995 the Corps initiated a Surface Bypass Program. In 1998 a feature design memorandum was prepared. This report investigated rebuilding of the skeleton bays at the John Day powerhouse to add surface bypass spillways. Due to the high cost of the Skeleton Bay spillways, the region decided to investigate the installation of a Removable Spillway Weir (RSW) at Spillway Bay 20 to test the skeleton bay surface bypass spillway concept. Initially, the RSW was required to be installed or removed in less than two days. During development of the DDR and input from National Marine Fisheries Service, the RSW would be installed and removed once per year, at most. This DDR outlines the result of that study and the changes in the design from the 90% plans and specifications.

A RSW is a steel structure positioned on top of a spillway. It is operated in a free overflow condition with no spilling gate control and it provides a surface oriented outlet for downstream migrating fish. Water passes over the RSW and down the spillway chute and is then directed horizontally by a spillway deflector.

Seven RSW spillway geometries were studied, and four were tested in the sectional model at the Northwest Hydraulic Consultants laboratory. Two were tested in the general project model at the Engineering Research Development Center (ERDC). Based on analysis and model testing, a preferred geometry was selected. The selected alternative has a crest 39.7 feet upstream of the Construction Base Line (CBL). The RSW piers would extend upstream 45.9 feet of the CBL. The preferred alternative has the same spillway geometry as Proof of Concept Alternative 5 and Optimum RSW Alternative C. However, the upstream face of the RSW has been modified for structural purposes.

In order to insure optimum hydraulics on the spillway chute and flow detector, the RSW extends downstream beyond the radial gate. This requires that the RSW be installed and removed in two pieces; the main structure upstream of the bulkhead slots and the tailpiece downstream of the slots. The main structure is the largest piece, will be designed to float, and installed by a tug. The tailpiece will remain in place during the three-year life of the project. The RSW will be fabricated from steel at an off-site location, and the pieces will be transported to the site for installation.

The cost of the RSW is estimated to be $10,140,000. This includes the main structure, tailpiece, attachment brackets, and installation of these items. This estimate does not include design and construction management.

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*	Transcript of Public Hearings on John Day	1960
*	Cemetery Relocations, Final Report, Relocations Irrigon Cemetery	1963
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*	Cemetery Relocations - Final Report Riverview Cemetery at Boardman	1965
*	Metallurgical & Weld Investigation	1963
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*	Report on Concrete Operations & Tests	1969
*	Supplemental Report - Concrete Operations and Tests	1970
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*	Bridge Inspection Report # 1 - Navigation Lock Spillway	1971
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JOHN DAY LOCK AND DAM
COLUMBIA RIVER, OREGON AND WASHINGTON

PERTINENT DATA

1. General

Stream mile from mouth	215.6
River miles from The Dalles Dam	23
River miles from McNary Dam	76.4
Drainage area, square miles	226,000
Length at crest, feet	5,900±
Normal height headwater to tailwater, feet	105
Discharges in cubic feet per second:	
Minimum of record	30,500
Mean annual low flow	60,800
Mean annual flow	188,500
Mean annual peak flow	583,000
Maximum of record, June 1894	1,230,000
Standard project flood:	
At dam site	1,060,000
Through reservoir	1,050,000
Spillway design	2,250,000
Tailwater elevations, The Dalles pool elevation 160:	
100,000 cfs	160.6
198,000 cfs, 12 powerhouse units	162.0
330,000 cfs, 20 powerhouse units	164.8
700,000 cfs, maximum for strict adherence to fishway design criteria	173.6
1,060,000 cfs, standard project flood	182.0
2,250,000 cfs, spillway design flood	205.3

2. Reservoir

Elevation, normal pool	265
Elevation, minimal pool for power (without flood control drawdown)	262
Elevation, minimum pool for flood control	257
Elevation, Maximum controlled pool for flood control	268
Flood storage (between backwater profiles for 800,00 cfs), Acre-Feet	500,000
Reservoir length - miles	76.4
Reservoir area at normal pool (flat), acres	52,000
Reservoir pondage below pool elevation 265:	
1-foot drawdown, Acre-Feet	50,000
2-foot drawdown, Acre-Feet	100,000

3-foot drawdown, Acre-Feet	150,000
Relocations	
Union Pacific Railroad, miles	59
Spokane, Portland & Seattle Railway, miles	80
Railroad Branch lines, miles	4
Oregon highways, miles	32
Washington highways, miles	40

3. Spillway

Number of bays	20
Bay width, feet	50
Pier width, feet	12
Over-all width, feet	1,252
Crest elevation	210
Gate size, with by height above crest, feet	50 x 58.5
Stilling basin length, feet	210
Over-all length, feet	340
Deck elevation	281
Deck width, clear, feet	30

4. Powerhouse

Length over-all, feet	1,921
Width over-all (transverse section), feet	243
Intake deck elevation, feet m.s.1.	281
Draft tube deck elevation, feet m.s.1.	185
Maximum height (draft tube invert to intake deck), feet	217
Spacing main units, station service, and assembly bay, feet	87
Turbines:	
Type	Kaplan-6 blade
Runner diameter, inches	280
Revolutions per minute	92.3
Rating, horsepower	171,100
Generators:	
Rating (name plate), kilowatts	108,700
Power factor	0.95
Kilovolt ampere rating	114,420
Overload capability at 0.95 power factor, kilowatts	125,000
Units installed complete	16
Skeleton units provided	4
Total number of units definitely provided for	20
Initial plant capacity - rated, kilowatts	1,304,400
Initial plant capacity - overload capability, kilowatts	1,500,000
Ultimate plant capacity - rated, kilowatts	2,174,000
Ultimate plant capacity - overload capability, kilowatts	2,500,000

5. Navigation Lock

Net clear length, feet	675
Net clear width, feet	86
Minimum water depth over sills, feet	15
Normal upper water surface elevation in chamber	265
Maximum upper water surface elevation in chamber	268
Top of lock walls, elevation	273
Minimum water surface elevation in chamber	155
Upstream sill block elevation	242
Downstream sill block elevation	140
Upstream gate type: Submersible lift	
Height, effective, feet	27
Downstream gate type: Vertical lift	
Height, feet	114
Maximum possible lift, feet	113
Normal lift, feet	105
Length of guard walls, feet	700

6. Navigation Channels

Temporary upstream channel for second-stage construction:	
Width, feet	150 to 300
Bottom elevation	145
Permanent downstream channel:	
Width, feet	250
Bottom elevation	139

7. Concrete Non-overflow Sections

Clear deck width, feet:	
Left abutment	30
Between powerhouse and spillway	30
Between spillway and lock	32
Deck elevation	281

8. South Shore Abutment Embankment

Deck elevation	281
Deck width, clear, feet	30
Freeboard embankment elevation	286
Freeboard embankment top width, feet	10
Material: Rock fill with central impervious core	
Slopes, upstream and downstream	1 on 2

9. North Shore Abutment Embankment

Crest elevation	286
Crest with, feet	30
Material Rock fill with inclined impervious core	
Slopes:	
Upstream	1 on 2.5
Downstream:	
Above elevation 276	1 on 2
Below elevation 273	1 on 1.5

10. Fish Facilities

Fish ladders:	South Shore	North Shore
Maximum design river flow, cfs	700,000	
Slope	1 on 10	
Regulation for pool fluctuation	Vertical Slot Control Section	
Fixed weir height, feet	6	
Normal ladder flow, cfs	125	126
Diffusion chambers:		
Numbers	9	
Velocity thru gratings, feet per second:		
Gross area	0.25	
Net area	0.50	
Powerhouse collection channel:		
Optimum transportation velocity, fps	2	---
Entrances:		
Submerged orifice slots	42	---
Overflow weirs	4	3
Velocities, feet per second:		
Through orifices	8	---
Over weirs	4	---
Diffusion chambers, number	40	---
Auxiliary water requirements, cfs:		
Minimum tailwater, elevation 155	2,565	450
Maximum design tailwater, elevation 173.6	3,216	1,068

11. Cofferdams

Number	3
Type:	
River legs	Steel Cells
Shore connections	Earth & Rock
Design river flow for overtopping, cfs:	
First and second step steel cells	700,000

SECTION 1 INTRODUCTION

1.1 General

This DDR contains information developed through the preliminary design (DDR) phase of work and during development of the plans and specifications up to the 90% level. This DDR describes the development of the design of the RSW Main Structure, Tailpiece and Main Structure Attachment. Through the 90% submittal, the DDR also contained a description of the extended length spillway flow deflector. However, at the 90% point in development of the plans and specifications, it was determined to biologically test the extended deflector prior to the testing of the RSW. Thus, separate plans and specification packages were developed for the RSW and spillway flow deflector.

The John Day Dam Project, operated and maintained by Portland District, U.S. Army Corps of Engineers (CENWP), is located approximately 110 miles east of Portland, Oregon, at River Mile 215.6 on the Columbia River. The John Day Powerhouse began operation in 1968. Plate 1 shows the major features of the John Day Project. The powerhouse has been modified in recent years to enhance the downstream migration of juvenile salmonids. Studies for enhancements to the existing Juvenile Bypass System (JBS) in addition to surface collection are concurrent with the work done in this report.

As screened bypass systems were evaluated, the regional goals of non-turbine passage were not always met. To enhance the screened bypass systems, the region has also initiated spill programs to further enhance non-turbine passage and presumably survival of juvenile salmonids. There are presently three possible passage routes for downstream migrant juvenile salmonids past the John Day Project. They can either be guided by the existing mechanical screened bypass system, be passed through voluntary (or non-voluntary) spill, or pass through the turbines. Several new juvenile salmonid passage programs are either being implemented or are in the planning phase for possible implementation at John Day. Although all of the existing or planned programs are intended to improve juvenile salmonid survival, several issues have yet to be verified. The primary goal for fish passage at each project is to provide a minimum of 80% Fish Passage Efficiency (FPE) with 95% minimum survival. It has been suggested that a Surface Bypass passage route could either increase efficiency of the spill program or enhance the screened bypass system. Also, with the emphasis on increased spill, the region is subjected to increased total dissolved gas in the river and the impacts to lost power production are being realized.

Beginning in 1995 the Corps of Engineers along with input from regional fishery managers began a Surface Flow Bypass (SFB) program (Harza Northwest 1996, Harza Northwest 1995). This program was intended to look at possible ways to bypass downstream migrant salmonids with surface oriented flows. As different SFB concepts were evaluated, the primary focus for the John Day project was the possible use of one or more skeleton bays located between the operational powerhouse turbines and spillway. In 1998 the Corps completed a Feature Design

Memorandum (Corps 1998a) outlining the use of the skeleton bays as a possible Surface Bypass Spillway (SBS) for juvenile salmonids. After review of the SBS FDM, the regional System Configuration Team (SCT) decided the cost of constructing the skeleton bay SBS was too high given the uncertainty of "proof of concept". In 1999 the SCT requested that the Corps evaluate two possible directions for SBS at John Day. One was to evaluate the use of four skeleton bays as a possible SBS; the second was to evaluate the use of a spillway bay to see if a less costly test of the SBS concept was possible. As the Corps evaluated the use of a spillway bay for a possible prototype SBS, an issue of spillway capacity and loss thereof also was raised. This possible loss in spillway capacity made the permanent modification of a spillway bay unpractical. Capacity would be required elsewhere if it is removed from the spillway, and estimated costs for additional capacity are high.

A Removable Spillway Weir (RSW) was conceptually designed as an option in the Surface Bypass Collection System Combinations Report, Lower Snake River (Corps 1998b). A RSW is a hollow steel structure that is filled with air for floating and towing into place. In the vicinity of the spillway bay, selective filling of the structure would occur to rotate the structure to the proper angle. Then, the RSW would be moved into place and further submerged until it rested on support brackets permanently mounted on the spillway. The existing spillway tainter gate would still accomplish flow control. This design has applications for use at John Day Dam, and could serve extremely well as a prototype test of the efficiency of a high flow surface bypass spillway. To serve as a "proof of concept" for a Skeleton Bay SBS, the RSW should be located as close to the skeleton bays as possible in Spillway Bay 20, and should be designed to have similar flow attraction characteristics as the skeleton bay surface bypass spillway. Being removable, an alternative means of passing the spillway design flood would not have to be considered.

The RSW could be designed as either a "proof of concept" for the Skeleton Bay SBS or could be designed to be a permanent bypass. The potential for a different geometry is possible if the RSW is not required to mimic the Skeleton Bay SBS.

The RSW should be designed so that there is minimal work on the spillway to attach the weir to the existing spillway bay concrete. The RSW would be designed to be totally removed from the spillway bay for maintenance and so that the spillway design flood capacity is not affected. Given the hydrologic and meteorologic characteristics of the Columbia River Basin, ample warning time of potential high spillway usage exists. Therefore, removal time does not impose a constraint on the RSW design.

Presently, the Walla Walla District Corps of Engineers is constructing a RSW for installation in 2001 at Lower Granite Dam. The unit discharge of this proposed RSW is less than that proposed for John Day, however other design criteria are the same. A draft pre-engineering report (JE Sverdrup 2000) and a draft hydraulic modeling study report (JE Sverdrup & ENSR 2000) have been published. Results from this work will be utilized in the John Day Design Documentation Report (DDR) work.

In 1997 spillway deflectors 12.5 feet long were installed in Spillway Bays 2 through 19. An extended deflector type of geometry was incorporated into the design of the Skeleton Bay Surface Bypass Spillway (SBS) to enhance performance and tested in the John Day general model at the Waterways Experiment Station. The deflector and its performance are discussed in Supplement No. 1 to Design Memorandum No. 50.

1.2 Purpose

This DDR considers alternatives and recommends specific RSW designs to meet surface oriented spillway requirements at the John Day Dam. The recommendations include:

- A RSW Proof of Concept Design to analyze the concept of the Skeleton Bay SBS
- Optimum or Permanent Surface Spillway Design

1.3 Scope

This DDR presents the results of the following work tasks:
- Model testing of the selected RSW geometry.
- Structural and mechanical engineering development of the selected proof of concept RSW design.
- Visits to the Northwest Hydraulic Consultants' hydraulics laboratory to view the operation of selected RSW geometries.
- Development of five "Optimum" RSW geometries. The term "optimum" means that these RSW's would be suitable for use as permanent surface bypass spillways at the John Day Project.
- Update of the RSW design based on development of the plans and specifications to the 90% point.

After the 90% submittal of the DDR, the DDR was divided into two documents. This DDR describes the RSW and its development. A second document, *Supplement No. 1 to Design Memorandum No. 50*, describes the design development of the spillway deflector.

The scope of work was amended between the 30% and 60% DDR submittals. These amendments affected the contents of the previous submittal in the following ways:
- An option to test more RSW geometries was exercised. In the initial modeling the two geometries selected for testing proved to be unsatisfactory. Therefore, other geometries were tested. As a result final testing of the selected geometry was not complete in time to be included in the 60% submittal.
- Due to further sectional modeling requirements and repairs to the general model at the Engineering Research Development Center (ERDC), the second model visit to ERDC was not completed.
- Due to the testing of other geometries as stated above, it was decided to generate the five "optimum" RSW geometries for the 60% submittal instead of the 30% submittal. These "optimum" geometries are described in the Appendix F of this DDR.

After the 60% submittal, a modification to the scope of work was developed. This modification called for additional hydraulic modeling and development of a summary report on deflectors, including design, testing, and field effectiveness of the deflector as a TDG reduction measure. The additional work included modeling several flow deflector geometries, testing the RSW

Tailpiece on the spillway by itself, development of a full rating curve for the RSW, and building a sectional model of the Skeleton Bay Spillway to compare it to the selected RSW configuration. The modeling report will not be completed by the submittal date of the DDR. The modeling report will be submitted later as an appendix to this DDR. However, all modeling results were available to the designers prior to final design and the preparation of this DDR.

1.4 Authorization

This study is an element of the Columbia River Fish Mitigation Program (CRFMP) and is being conducted under the Rivers and Harbors Act of 1945, Public Law 79-14, dated March 2, 1945. This document is being developed under Contract No. DACW57-97-D-0004, Delivery Order No. 21, with modifications through Modification No. 0007, between the CH2M Hill/Montgomery Watson Joint Venture and the U.S. Army Corps of Engineers, Portland District.

1.5 Agency Coordination

Coordination with fisheries agencies has been on-going throughout the development of this DDR. Agencies including National Marine Fisheries Service (NMFS), Oregon Department of Fish and Wildlife (ODFW), and representatives of Indian Tribes have attended the 10%, 30%, 60% and 90% review meetings. Reports of these meetings are included in Appendix B. In addition, agency representatives have attended model review visits to Northwest Hydraulic Consultants' laboratories and one visit to the Engineering Research Development Center to view the John Day general model.

SECTION 2 BIOLOGICAL CONSIDERATIONS

2.1 Introduction

Juvenile salmonid bypass system development on the lower Columbia River has evolved considerably over the last five years. Initiated by the listing of Snake River chinook and sockeye salmon as endangered under the Endangered Species Act, the Corps of Engineers, Portland District has been reevaluating every aspect of existing juvenile bypass system technology. Understanding the fundamentals of fish behavior and incorporating these responses into system designs is critical for a successful surface bypass system. The Portland District regards juvenile salmonid approach patterns in project forebays, spatial and temporal distributions, and responses to hydraulic flow fields as critical design components for surface bypass technology. Further, the conveyance of flow and fish, and the disposition of fish once bypassed into the tailrace, are also critical to any successful bypass system.

The surface flow bypass (SFB) program utilizes juvenile salmon migration behaviors to design and construct more effective bypass systems. Existing biological information collected over the last three decades at each of the lower Columbia River hydroelectric projects has therefore been reviewed, and where shortcomings were noted additional research has been initiated. Dam forebay and tailwater flow characteristics are being analyzed in the field and on hydraulic models, and fish responses to flow parameters are being investigated. The preliminary information collected through this effort has been utilized to construct scale size bypass systems on hydraulic models. As additional biological information is ascertained, the models are manipulated to refine system designs. Construction of complete prototype systems may occur at lower Columbia River dams once enough information has been collected through biological and hydraulic investigations to evaluate the likely efficiency of the system.

The concept of surface flow to bypass downstream migrating salmonids arose from the realization that sluiceways and sluice chutes pass a disproportionate number of fish for the amount of flow passed. With this information the Wells project located on the mid Columbia River, successfully constructed a functional production juvenile bypass using the surface flow concept. It should be noted that while sluiceways and sluice chutes are highly efficient (number of fish per unit volume of flow), they did not meet regional goals of 80% Fish Passage Efficiency (FPE, the percentage of fish passing through non-turbine routes). Therefore, the Wells project utilized a deep slot concept, across the face of the powerhouse, to successfully meet passage goals. With the success of SFB at the Wells project, the Corps and regional fishery managers have evaluated SFB concepts at Corps operated facilities.

The focus of SFB at the John Day project, was essentially an evaluation of the use of the existing skeleton bays for a surface flow bypass alternative. One reason that the skeleton bays were evaluated at John Day is that other SFB "concepts" were being evaluated at other projects. The primary concepts the Corps evaluated, as the onset of surface flow bypass was the deep slot, corner collectors, and "high flow" surface spill designs like the skeleton bay. Regional decisions at the inception of SFB evaluations were not to evaluate the same concepts at different locations. It should be noted that different SFB concepts use different fish behavior, and hydraulic conditions to accomplish fish passage goals. The deep slot concept is generally a stand alone bypass concept that could replace the existing screened mechanical bypasses, with slots across the entire face of a powerhouse. The deep slot concept uses the knowledge that juvenile salmonid generally do not prefer to sound, and given an acceptable hydraulic condition will pass into a deep slot SFB to be routed around the powerhouse. The corner collector concept, uses specific fish behavior and forebay hydraulics that bring large numbers of downstream migrants within a specific location, and in general are thought to enhance the FPE along with existing screened bypass systems. The skeleton bay concept uses a large volume of flow that presumably will (1) creates a substantial attraction flow, and (2) is located in an area that large numbers of downstream migrants will be attracted to.

2.2 Skeleton Bay versus Removable Spillway Weir

As stated earlier, the primary focus for the John Day SFB program was use of the skeleton bays as a surface flow passage route. With the completion of the Skeleton Bay (SB) Design Memorandum and associated hydraulic model studies, the SB concept was deemed too expensive for prototype construction without further biological information as to the potential for success. Regional direction to the Corps was to evaluate a Removable Spillway Weir (RSW) as a surrogate for prototype testing the SB concept that would be less expensive than construction of the skeleton bay SFB, but allowing for a "test of concept" for the SB.

2.3 Biological Evaluations

There are many issues that need to be addressed either prior to, or concurrently with new downstream migrant passage design and/or construction at John Day. Although this report is intended to evaluate surface bypass spill options, it is also necessary to remember that other possible competing juvenile fish passage options are being considered for John Day. Besides the surface bypass spill option being considered for John Day, there is also a screened bypass program evaluating replacing the existing 20 foot screens with 40 foot screens, and a spill program looking at spill efficiency, and the possibility of 24 hour spill as well. The only other option not presently being considered for study at John Day is a powerhouse Surface Flow Bypass option deep slot or corner collector).

Past studies of downstream migrant salmonids have provided some of the rationale for surface bypass spill. There have been two years of study of the deep slot powerhouse SFB at the Bonneville first powerhouse. Efficiency data collected in 1998 was in the 80 to 90% ranges. Limited 1999 data show efficiency in the 70 to 80% efficiency range, and preliminary data from 2000 show an efficiency in the 80% range. The Bonneville deep slot SFB program has expanded, and the "final" test in 2000 was to determine if SFB, or extended length screens will be pursued as the primary juvenile salmon passage technology of the future for Bonneville first powerhouse. At the Bonneville second powerhouse, a SFB Corner Collector was studied in 1998. This SFB design was deemed so efficient and effective that the Corps is moving forward with design of a permanent Corner Collector SFB for Bonneville second powerhouse. At The Dalles dam, the sluiceway has long been operated as the primary juvenile salmon passage route in conjunction with voluntary spill. The Dalles dam is continuing to study the SFB concept, and blocking of trashracks in an effort to increase the efficiency of the system in 2001.

As discussed earlier, other options are being studied at John Day to increase juvenile salmon non-turbine passage. Extended Length Submersible Bar Screens (ESBS) has been evaluated at John Day. Although the guidance efficiency was promising, gatewell hydraulics associated with the ESBS caused unacceptable mortality. Although this is a powerhouse system, surface flow spill could either enhance the ESBS system, or prove to be effective enough to continue the use of the existing Submersible Traveling Screen (STS) system.

Another juvenile passage strategy at John Day has been voluntary spill for fish. Past spill for fish at John Day has focused on nighttime spill. Recently, the concept of 24-hour spill has been tested to see if FPE could be enhanced. In 1999, a spill for fish test was conducted to investigate the benefits of daytime spill for fish. The study design called for 30% daytime spill, 0% daytime spill, and 60% nighttime spill. Due to high river flows in 1999, and the need to limit total dissolved gases (TDG), most nighttime spill was in the 45% range. Two evaluation techniques were used to evaluate the changes in FPE between spill treatments. Radio telemetry evaluation of spill passage detected no differences in FPE between spill treatments for either juvenile steelhead or yearling chinook salmon. However, spill effectiveness increased with 24-hour (daytime) spill, but the increase in spill passage was a result in a decrease in passage through the screened bypass system (hence, no increase in FPE). Hydroacoustic evaluation of the 24 hour versus 12-hour spill could not indicate increase/decrease in FPE since fish guided into the screened bypass system were not enumerated. The hydroacoustic evaluation did show an increase in spill passage similar to the radio telemetry (Johnston & Nealson, 1999 in prep). The primary benefit for the daytime spill was decreased residence time for spring chinook salmon (Hansel & Beeman, 1999 in prep).

A second year of 24-hour versus 12-hour spill was conducted in 2000 at the John Day project. For radio telemetry, the 2000 test again showed the same trend for spring migrants, no significant increase in FPE for daytime spill. However, with new coded transmitters for summer migrants, a significant increase in FPE was detected for subyearling chinook salmon for 24-hour spill. Further, as with 1999, radio telemetry in 2000 showed an increase in spill passage efficiency for yearling and subyearling chinook salmon. The other potential benefit for daytime (24-hour) spill as seen in 1999, decreased forebay residence time, was similar to 1999 tests. There was a decrease in forebay residence time for yearling and subyearling chinook salmon, however the decrease was only for migrants arriving at the project during daylight hours (no decrease during nighttime hours). One further piece of information from the 24-hour versus 12-hour spill evaluation needs consideration. Route specific survival had not been tested at the John Day project prior to 2000. Using radio telemetry technology, although not statistically significant, survival for yearling chinook salmon and steelhead was higher for 12-hour spill than 24-hour.

2.4 Biological Considerations

Given the uncertainty of the different potential methods to increase FPE at John Day dam, and the success realized with some of the SFB programs within the region, it appears reasonable to continue evaluating surface spill as a means of increasing FPE and survival at John Day dam. It must be stressed, there are still three main issues associated with SFB and all salmon passage technologies. Fish must not only be collected safely, but the conveyance around the dam is an important component, and safe re-introduction back into the river following conveyance is also an important issue. It is believed these issues can be evaluated in part from hydraulic modeling. Further, the SFB program at Bonneville is conducting field, laboratory, and hydraulic model research on high volume discharges to insure these higher volumes of water carrying fish can be safely re-introduced back into the tailrace. Preliminary data from these high volume outfall studies in 2000 show great promise for safe re-introduction into the tailrace of flow and fish for SFB structures.

The Skeleton Bay and Removable Spillway Weir both have extended flow deflectors as part of their designs. One issue that will need to be addressed in addition to RSW factors, is fish condition and survival passing over the extended deflector. To date there are no extended length flow deflectors within the region. Without hydraulic modeling, and possibly field tests for fish injury/condition this could remain an unknown prior to installation and testing presently scheduled for spring, 2002. Another fish condition/injury issue has been raised downstream of the extended flow deflector. In hydraulic model studies on a 1:30 scale sectional model, some members from the regional fishery managers have expressed potential "concerns" given the high discharge, and associated energy dissipation and turbulence in the tailrace. Although spill for fish passage is considered safe in most conditions, the volume of flow through the RSW and associated hydraulics downstream may be more severe than most spillways. Additional considerations may be necessary concerning the extended flow deflector. Presently the RSW will be constructed with an extended length flow deflector. If a decision is made to build the Skeleton Bay as the permanent Surface Bypass, how will the extended flow deflector perform during "normal" spill?

Another issue is downstream tailrace conditions for fish egress. Spill for fish passage programs require spill patterns be evaluated to insure positive downstream egress with a minimum of eddies and to insure flow does not encounter potential predator habitat areas. With the high volume of flow associated with the RSW, it is apparent that some amount of flow on either side of the RSW (powerhouse and spillway) will be necessary to minimize eddies and reverse flow patterns. This requirement raises at least two issues, what volume of flow is required through the spillway and powerhouse, and whether spill for downstream egress would be required 24 hours a day. One potential benefit for the RSW or SB would be that they are more efficient than standard spill, thereby decreasing the volume of normal spill required meeting passage goals. This could, in turn, have a positive benefit to total dissolved gas levels, and provide for increased hydropower generation. However, if a high volume of spill were required for good downstream egress conditions, then this would defeat one of the main purposes of the RSW/SB. The second issue for spill is that the region is currently evaluating the use of daytime spill at John Day to enhance FPE and survival. It has not been determined to date that 24-hour spill will be the normal practice. However, if spill is required to balance tailrace conditions for good downstream egress, and daytime spill is deemed not beneficial, then the RSW/SB may only be operated during daytime hours.

Adult passage issues also need to be evaluated for the RSW given the volume of flow to be discharged. Location of these types of juvenile fish bypasses need to be evaluated in hydraulic model studies to insure false attraction of adults causing delay, and the over riding of attraction flows at adult entrances. The issue of over riding adult attraction flows can be evaluated (at least in part) with the use of hydraulic models. However, without an ongoing adult salmon passage evaluation, it may be either very expensive or impracticable to evaluate the false attraction issue.

One issue for the RSW is the potential production of dissolved gas due to the high volume of discharge. The region presently relies heavily on spill as a juvenile salmon passage strategy. As the volumes of spill increased, dissolved gas levels from high spill became apparent. High levels of total dissolved gas can be lethal to not only migrating fish, but also to resident fish and other organisms within the ecosystem. It is therefore necessary that the RSW does not add substantially to total dissolved gas production.

Another issue is the potential for cavitation on the downstream face of the ogee where the RSW connects to the existing spillway crest. Although this issue is being evaluated as part of the engineering and design components, it could also have additional impacts from a fish condition/injury point. Although this may not be as severe a potential problem, it is one additional issue that will have to be considered during design, and possibly field testing.

2.5 DRAFT Study Plan

The original concept for a biological study plan for the RSW was to operate it under a randomized block treatment in an in/out configuration. This would allow for a scientifically sound and statistically valid test of the RSW prototype. This would have been a simple and effective evaluation given that the RSW was "removable". However, during the design process it has become apparent that removal of the RSW, although possible for spill capacity reasons, is not feasible for a biological test. Further, with the removable of the RSW, the amount of flow through a "normal" spill bay would need to equal that of the RSW for a true test of concept, and fish conditions and gas supersaturation likely would be problematic.

It is obvious that just installation and enumeration of fish passage through the RSW will not provide a true and scientifically sound biological evaluation. Therefore, some manner of testing the RSW in an on/off condition will be necessary. Further, in order to test the RSW in a sound scientific manner, conditions must be similar for both the on and off configurations. With this in mind, we would propose to test the RSW prototype with the same amounts of surface flow and normal spillway spill for each treatment, in a randomized block treatment design. This would call for a decrease of spillway spill volume equal to the volume passing the RSW during the on treatment, and an increase of spillway spill volume equal to the volume passing the RSW during the off treatment. With this simple study design the same amount of flow is passing the spillway/RSW for both treatments and a true treatment study of the surface spill/SK concept would be possible. Given the proposed study design, some agreed upon level of increased FPE with the RSW on would be necessary for a positive result (this needs to be discussed and agreed upon within the region prior to testing).

One more consideration may be required as the RSW is evaluated. As discussed above, there are concerns with fish condition/injury with both the extended flow deflector, and hydraulic conditions downstream of the deflector. If these issues persist, this would require an evaluation fish condition/injury, preferably prior to the major portion of the outmigration. One potential consideration is to construct the extended flow deflector prior to construction of the RSW, and test fish condition/injury without the RSW.

At this point the efficiency/effectiveness evaluations are considered to be the paramount components in the biological evaluation of the RSW. There have been several studies of Surface Flow Bypass in which three dimensional behavioral data was collected, with the idea of coupling the data with hydraulic data to evaluate why a certain prototype either worked, or did not work. At this point in the planning process we feel the efficiency/effectiveness data will be the most useful for decisions by the Corps, and regional fishery managers.

SECTION 3 HYDRAULIC DESIGN

3.1 Design Criteria

The hydraulic design criteria used in developing the RSW are listed below:

1) Crest head of 22.5 feet at maximum normal operating forebay elevation of 268-ft msl.
2) RSW shall be located upstream of the existing spillway gate, if possible.
3) Existing spillway gate flow control capability shall not be compromised by RSW.
4) If possible RSW entrance flow acceleration shall be less than 0.1 feet per second per foot, up to point where capture velocity of 7 fps is reached (NMFS criteria).
5) RSW discharge capacity about 14,000 cfs (similar unit discharge capacity as the Skeleton Bay Surface flow Bypass Spillway (SBS) design) at normal operating pool elevation range of 262-265 ft.
6) RSW shall develop approximate similar zone of flow attraction influence in the forebay as the Skeleton Bay SBS.
7) RSW shall draw primarily from upper 30 to 50 feet of water column.
8) RSW shall exhibit hydraulic characteristics similar to the Skeleton Bay SBS.
9) RSW shall be located near powerhouse to approximate skeleton bay location.
10) RSW pier nose shape shall be optimized to minimize flow separation and pier drawdown effects.
11) RSW entrance shall be as near to the spillway crest as practical.
12) The RSW shall be designed to be structurally stable once in place.

In general, the RSW is likely to be most effective when the proportion of total spillway flow passing over the RSW is high. When spill volume over the RSW falls below a certain percentage of total spillway discharge, the effectiveness of the RSW is expected to be diminished, since the flow through adjacent spillway bays will reduce the relative strength of the flow field entering the RSW. The consensus of the team members following the initial discussion during the first project site visit was to design the RSW to approximate the proposed unit discharge through the Skeleton Bay SBS in order to develop a similar attraction flow field in the reservoir. Preliminary computations indicated that a design head of 22.5 ft was required to achieve this similarity. Model data later showed that the unit discharge for the RSW with 22.5 ft of head was less than the unit discharge of the SBS at a comparable pool elevation. Because the 22.5 ft head criteria had already been established with the agencies, and because subsequent testing in physical models revealed that the surface withdrawal characteristics of the RSW were not as sensitive to unit discharge as previously thought, the 22.5 ft head criteria was retained for final design.

More detailed criteria are provided below.

3.1.1 Forebay / Tailwater Range

1. Maximum forebay elevation is 276.3 ft (NGVD) for the Spillway Design Flood Event (SDF).
2. Maximum operating forebay elevation 268 ft (NGVD) (for fish passage also).
3. Normal operation high forebay elevation 265 ft (NGVD).

4. Normal operation low forebay elevation 262 ft (NGVD).
5. Minimum operating forebay elevation 257 ft (NGVD).
6. Maximum tailwater elevation during (SDF) is 205-ft msl (NGVD).
7. Maximum tailwater elevation during Standard Project Flood (SPF) event is 182-ft msl (NGVD).
8. Maximum tailwater elevation during normal high flow operation is 165-ft msl (NGVD).
9. Maximum tailwater elevation during normal low flow operation is 155-ft msl (NGVD).
10. Minimum tailwater elevation during normal operation is 160-ft msl (NGVD).

3.1.2 Discharge Capacity

1. Maximum SDF discharge 2,250,000 cfs.
2. SPF discharge 1,060,000 cfs.
3. Number of Spillway Bays: 20.
4. Maximum per-bay discharge: 112,500 cfs (SDF event).
5. Maximum per-bay discharge: 53,000 cfs (SPF event).
6. RSW discharge at minimum operating pool elevation 257 ft msl (NGVD): 7,000 cfs
7. RSW discharge at typical low operating pool elevation 262.5 ft msl (NGVD): 14,000 cfs
8. RSW discharge at typical high operating pool elevation 264 ft msl (NGVD): 15,500 cfs
9. RSW discharge at maximum operating pool elevation 268 ft msl (NGVD): 21,000 cfs

3.1.3 Hydraulic Loads on Structures

1. Maximum hydraulic loads on RSW structure shall be hydrostatic and/or hydrodynamic throughout the range of forebay operating pool elevations from elevation 257.0 through 268.0.

3.1.4 General Geometry & Miscellaneous Criteria

1. RSW shall be located upstream of the existing spillway gate, if possible.
2. RSW shall be located near powerhouse, to approximate skeleton bay location.
3. RSW pier nose shape shall be optimized to minimize flow separation and pier drawdown effects.
4. Design of the RSW and interface zone with existing spillway shall be developed to eliminate, or at least minimize, potential for cavitation damage.

3.2 Design Guidance and References

Hydraulic design of the various alternatives discussed in this report utilizes design guidance from the Corps of Engineers' EM series, as appropriate, the Bureau of Reclamation Design of Small Dams, and the Corps of Engineers' Hydraulic Design Criteria. Design of all features of each alternative follow established guidance from the Corps of Engineers and the Bureau of Reclamation sources.

3.2.1 Design References

The references listed in the following sections were used to establish hydraulic criteria. In addition, many of the reports and studies listed in the References section were also used to develop background and justification for adoption of these criteria.

3.2.2 Engineer Manuals (EM) / Other

EM 1102-2-1602 Hydraulic Design of Reservoir Outlet Works
EM 1102-2-1603 Hydraulic Design of Spillways
Hydraulic Design Criteria (US Army Corps of Engineers, 1988)

3.2.3 Computer Programs

CORPS H1102 "Standard High Spillway Crest Coordinates"
CORPS H1103 "Stage-Discharge Relation for Standard Spillway (ungated)"
CORPS H1105 "Stage-Discharge Relation - Spillway Crest Uncontrolled Flow"
CORPS H1107 "Stage-Discharge Relation for Elliptical Crest Spillway"
CORPS H1108 " Crest and Upper Nappe Profiles for Elliptical Crest Spillways"
CORPS H1109 "Pressure Distribution for Elliptical Crest Spillways"
CORPS H1110 "Spillway Energy Losses"
CORPS H1111 "Standard Spillway Crest Water Surface Elev. – High Dams"
CORPS H1116 "Standard Shape Spillway Crest Pressure – High Dam"
CORPS H1170 "High Spillway Crest Coordinates – 3-1 Upstream Face"
CORPS H1180 "High Spillway Crest Coordinates – 3-2 Upstream Face"
CORPS H1190 "High Spillway Crest Coordinates – 3-3 Upstream Face"
HEC-RAS
CORPS Hydraulic Design Computer Aids
HEC-2

3.2.4 Texts

Internal Flow Systems, Miller
Handbook of Hydraulics, King and Brater
Engineering Hydraulics, Rouse
Design of Small Dams, US Bureau of Reclamation
Open Channel Flow, Henderson
Fluid Mechanics, Roberson & Crowe
Fluid Dynamics, Daily & Harlemann
Fisheries Handbook of Engineering Requirements and Biological Criteria, US Army Corps of Engineers, Portland District, 1991

Portland District Corps of Engineers Publications

John Day Lock and Dam Surface Bypass Spillway, Feature Design Memorandum No. 52. US Army Corps of Engineers, Portland District, September 1998.

North Pacific Division Corps of Engineers Publications

Spillway Deflectors at Bonneville, John Day and McNary Dams on Columbia River, Oregon-Washington and Ice Harbor, Lower Monumental and Little Goose Dams on Snake River, Washington. Seattle District. September 1984.

Walla Walla District Corps of Engineers Publications

Lower Snake River: Surface Bypass and Collection System Combinations Conceptual Design Report, US Army Corps of Engineers, Walla Walla District, December 1998.

Other Related Reports and Studies

Surface Bypass Alternatives at Bonneville, The Dalles, and John Day Spillways. Final Report. Harza Northwest, Inc. May 1996 (2 Volumes).

Surface Bypass Alternative Study at John Day Powerhouse. Final Report. Harza Northwest, Inc. December 1995 (2 Volumes).

Lower Granite Lock and Dam Surface Bypass and Collection Removable Spillway Weir Pre-Engineering Report. 90% Submittal. JE Sverdrup. March 24, 2000.

John Day Dam Sluice Model, Hydraulic Model Study. Final Report. Northwest Hydraulic Consultants. May 1999.

Hydraulic Model Study of Removable Spillway Weir for Juvenile Fish Passage at Lower Granite Dam. 60% Draft Report. Sverdrup Civil, Inc., and ENSR. April 2000.

3.3 Similar RSW Design for Lower Granite Dam

Walla Walla District is evaluating a similar concept at Lower Granite Dam, which will be prototype-tested in 2001. Plans and Specifications for the Lower Granite RSW are to be completed sometime during 2000. The Lower Granite RSW is being designed for 6,000 cfs, while the John Day design flow is greater than 14,000 cfs. The John Day RSW is designed to simulate the flow conditions through the Skeleton Bay SBS. In addition, the configuration of the spillway RSW sections at the two projects are somewhat different, and the John Day skeleton bay discharge objective is greater than that for the Lower Granite RSW.

Four different Lower Granite RSW designs were developed, but the general crest geometry for all were similarly shaped and all were to be placed entirely upstream of the existing spillway radial gate. Three of the four concepts were designed for about 15 feet of submergence below the maximum normal operating pool elevation, and the fourth concept was designed for submergence of 21 feet below the maximum normal operating pool elevation. All four concepts include an ogee crest which transitions to the existing spillway face through a radius bucket, with the true tangent intersection occurring upstream of the existing spillway radial gate seal beam. The reader is referred to the 90% submittal document for the Lower Granite RSW design (JE Sverdrup March 2000) for a more detailed discussion of the Lower Granite RSW structure.

3.4 Skeleton Bay Surface Flow Bypass (SBS) Spillway Design

The design of the Skeleton Bay SBS was developed by Montgomery Watson to the Feature Design Memorandum (FDM) level (US Army Corps of Engineers, September 1998) in a previous study completed for the Portland District. Portland District, Corps of Engineers Hydraulic Design Section staff accomplished the hydraulic design with input from the National Marine Fisheries Service and other regional fisheries resource agencies.

The SBS design was intended to convert one or more of the existing powerhouse skeleton bays into a surface spill fish bypass by removing the top of the powerhouse and constructing three large open channel spillway chutes with deflectors through each powerhouse skeleton bay monolith. Each of the 3 chutes per skeleton bay would have a broad crest extending about 30 ft across the full breadth of the upstream upper powerhouse structure, then a chute extending over the existing turbine pits and down to a deflector below the existing downstream lower powerhouse deck. Two interior piers within each Skeleton Bay SBS would be 7 ft wide, and the exterior piers at the joint between skeleton bay units would be 13 ft wide. Total width of each skeleton bay is 90 ft and each of the three chutes would be 21 ft wide.

Model studies were conducted with the SBS geometry at WES to confirm hydraulic performance. The SBS was tested in both the 1:80 scale John Day general model, and in the 1:40 scale sectional model. Results were favorably reviewed by regional fisheries resource agencies. Data collection included velocity measurements in the forebay approaching the entrance to the SBS, on the chute, and in and around the discharge jet from the deflector into the tailwater. Qualitative observations made in the physical models included the upstream zone of influence of the SBS, approach velocity, and downstream egress characterization.

3.4.1 Surface Bypass Spillway 'Proof of Concept'

From 1995 to 1998 the Corps of Engineers, along with input from the regional fishery resource agencies, developed a Skeleton Bay Surface Bypass Spillway design for John Day Dam which utilized the four skeleton bay units in the powerhouse. However, following development of the FDM (US Army Corps of Engineers, September 1998), which found that the Skeleton Bay SBS was more expensive than originally anticipated, the Corps and fishery agencies decided to pursue a means of verifying the anticipated performance of such an SBS system. During the resulting exploration for alternatives, the concept of a Removable Spillway Weir (RSW) was conceived. This RSW would be installed temporarily in Spillway Bay 20 and would be used to determine the potential effectiveness of a large surface collector prior to funding the large capital costs of the Skeleton Bay SBS. The "Proof of Concept" RSW is intended to perform this function without compromising the existing spillway design flood discharge capacity.

The RSW concept was first developed for the Walla Walla District's Lower Granite Dam project, where similar fish passage issues exist. The RSW is generally described as a hollow steel structure that is filled with air for floating and towing into place. In the vicinity of the spillway bay, selective filling of the structure would occur to rotate the structure to vertical. Once vertical, the RSW would be moved into place and further submerged until it rests on support brackets permanently mounted on the spillway. The RSW would be designed to flow free of gate control during operational testing. The Lower Granite design, even though having different fish passage goals, flow criteria and design considerations, was considered to have application at John Day Dam and was considered a good candidate for testing the surface collection success at the John Day project. To serve as a "proof of concept" for a Skeleton Bay SBS, the RSW should be located as close to the skeleton bays as possible in Spillway Bay 20, and should be designed to have similar flow attraction characteristics as the Skeleton Bay SBS. Being removable, an alternative means of passing the spillway design flood would not have to be considered. The RSW could be designed as either a "proof of concept" for the SBS or could be designed to be a permanent bypass. The potential for a different geometry is possible if the RSW is not required to mimic the Skeleton Bay SBS.

3.5 RSW Design Process

A three phased design process was developed for the John Day Dam RSW. The initial phase of design consisted of conceptual hydraulic design of six alternatives that were considered to have some potential of emulating the hydraulic characteristics of the Skeleton Bay SBS. The next design phase consisted of preliminary physical model testing of some of the conceptual alternatives to serve as a "Proof of Concept" that the designs selected would in fact emulate the SBS hydraulic performance. As a further refinement in the design process, Optimum RSW alternatives were developed which were considered to not only emulate the SBS performance, but to actually have potential to improve upon the performance exhibited by the SBS. A detailed discussion of this design process is included in Appendix F.

3.6 Initial RSW Concepts and Model Alternatives Report

A total of six alternatives were developed to the conceptual level of hydraulic design. The six were presented to the National Marine Fisheries Service, Columbia River Inter-Tribal Fish Commission staff, and Corps personnel at an Alternatives Selection Meeting on 9 May, 2000 held at the John Day Project. The level of hydraulic design was limited to development of configurations based on previous Skeleton Bay SBS collector work, Lower Granite Dam RSW work, and rough calculations of approximate velocities, discharge capacities, and water surface profiles in the vicinity of the RSW structure. The selected design/s have been developed more fully in this DDR study, and the performance of the selected design/s have been documented in a 1:25 scale sectional physical hydraulic model of the spillway and a larger 1:80 scale general model of the John Day project located at the ERDC.

Alternative 1 – Skeleton Bay Geometry w/ Piers 88' Upstream
Alternative 2 – Vertical Face RSW w/ Piers 46' Upstream
Alternative 3 – Sloping Face RSW w/ Piers 129' Upstream
Alternative 4 – Vertical Face RSW w/ Piers 41' Upstream, step at spillway ogee interface
Alternative 5 – Vertical Face RSW w/ Piers 46' Upstream w/ semi-permanent Lower Crest
Alternative 6 – Sloping Face RSW w/ Piers 124' Upstream, step at spillway ogee interface

The Model Alternative Report, presented to the District during the site visit/kickoff meeting on 9 May 2000, is furnished in Appendix D of this DDR.

3.7 Final RSW Design

The RSW is designed for 22.5 ft of head at maximum normal operating pool elevation 268-ft msl (NGVD). The RSW will have a finished, in-place width of 50 feet, the same as the existing spillway crest. The RSW is comprised of the crest and piers on either side and will be floated into place. Bulb seals will provide positive head seal between the RSW and existing piers. Bulb type seals will also provide positive closure against reservoir head at the downstream toe of the crest section.

The RSW is designed as a two-piece installation, with a much smaller, lower, tailpiece installed downstream of the existing stoplog slots and under the spillway radial gate. The larger RSW crest section (main structure) is installed upstream of the existing spillway radial gate and would be installed adjacent to the tailpiece. The purpose of this design is to eliminate any radius transition between the RSW and existing spillway that would cause disruption to flow. The Tailpiece would be semi-permanently installed on the spillway crest. After the upper crest section is removed, removal of the small tailpiece section could be accomplished in the dry by placing bulkheads in the existing bulkhead slot and raising the spillway tainter gate. The tailpiece section would not be easily and rapidly removable. However, evaluation of project operating conditions indicate that rapid removal of the tailpiece section either for high flow or biological testing requirements will not be necessary.

The RSW geometry was selected following evaluation of various alternative designs in the sectional physical model. Unstable and oscillating surface waves and roostertails existed on the RSW and spillway chute and extended deflector resulting from the reverse curvature bucket transition with both Alternative 2 and Alternative 4. However, with the RSW and existing spillway joined by a continuous tangent sloping chute, those undesirable conditions were eliminated. The final geometry accomplished this objective by providing the desired smooth transition from the RSW crest section to the existing spillway chute.

3.7.1 Crest Design

The large upstream main section of the RSW crest shape was designed in accordance with guidance found in the COE design manual EM 1110-2-1603 “Hydraulic Design of Spillways”. The RSW is designed for a maximum operating head (He) of 22.5 ft at normal high operating pool elevation 268 ft to emulate the head on the Skeleton Bay SBS. As such, the crest elevation is 245.5 ft and the crest length is 50 ft. See Figure 3-1 for a diagram of the RSW geometry. The axis of the RSW crest is located 39.7 ft upstream from the axis of the existing spillway crest (also designated as the Construction Base Line (CBL)). The downstream quadrant of the RSW has a design head (Hd) of 22.5 ft so that He/Hd equals 1.0 and the crest equation is:

$$Y=(0.03545)(X)^{1.85}$$

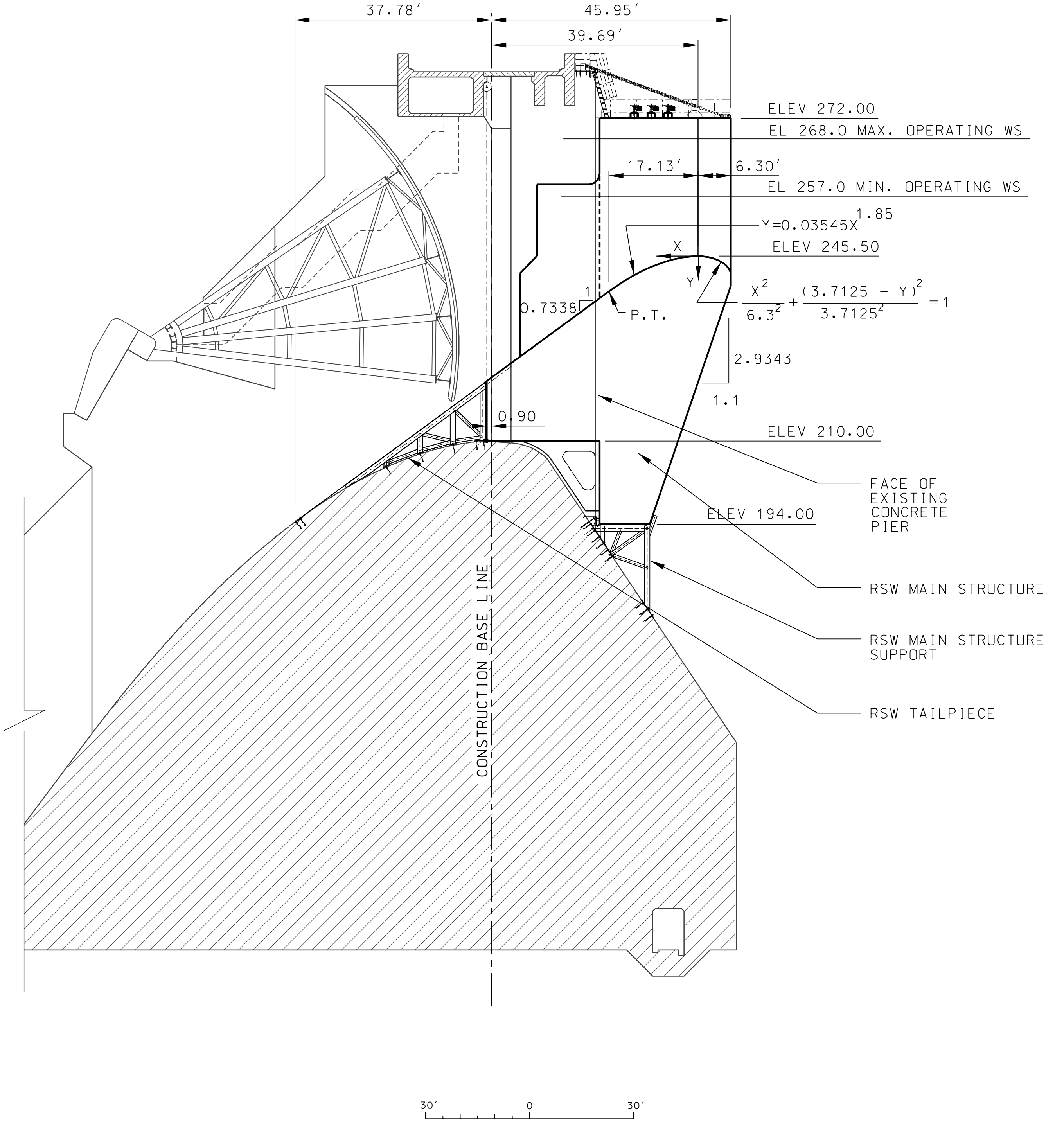


Figure 3–1
RSW Geometry

The upstream quadrant is the standard elliptical shape defined by the EM with a major axis of 6.3 ft and a minor axis of 3.7125 ft. The RSW discharge determined from model studies varies from 7,000 cfs at pool elevation 257 ft to 21,000 cfs at pool elevation 268 ft. The discharge at typical low and high operating pool elevations 262.5 ft and 264 is 14,000 cfs and 15,500 cfs, respectively. The RSW crest and piers extend about 46 feet upstream of the CBL. The downstream quadrant of the RSW ogee becomes tangent to the downstream face of the existing spillway about 37 ft downstream from the CBL where the spillway face slope is 0.7338 ft/ft. The piers would extend to near the upstream vertical face of the dam. Capture velocity of 7 fps is reached upstream of the entrance section. Consequently there is no need to meet the maximum acceleration criteria of 0.1 fps/ft within the RSW entrance.

The downstream end of the RSW ogee crest joins the tailpiece section at a true tangent point just upstream of the radial gate seat on the RSW on the same 0.7338 slope. The downstream end of the main RSW section will be designed to have a minimum gap to eliminate any irregularities across the joint. Water surface elevations and depths along the bay centerline and left pier measured in the physical model are shown in Table 3-1. Pressures measured on the RSW in the 1:25 scale sectional physical model are shown in Tables 3-2 and 3-3. The computed velocities over the RSW crest are shown in Table 3-4 and the RSW rating curve is shown on Figure 3-2. Cavitation potential was evaluated using information from EM 1110-2-1603 and the USBR Engineering Monograph No. 42, "Cavitation in Chutes and Spillways". The cavitation index (C.I.) at any location is defined as:

$$C.I. = (P - h_{vapor}) / h_v \quad \text{where}$$

P = local pressure (absolute), ft water
h_{vapor} = vapor pressure of water (absolute) = 0.4 ft at 50 degrees F
h_v = velocity head, ft

The minimum dynamic pressure measured in the physical model at the point where the Main Structure meets the Tailpiece was 3.2 ft and computed velocities are about 55 fps. The computed C.I. for those pressure and velocity conditions is 0.77 and the maximum allowable abrupt into-the-flow irregularity from Plate 2-7 of the EM is 0.05-inch. Therefore, this joint must be very smooth.

The tailpiece section is a separate section designed to fit in the approximately 8-ft space between the downstream edge of the stoplog slot and the spillway gate seat on the face of the spillway to allow installation and removal.

Table 3-1. RSW Water Surface Profile With Pool Elevation 268 ft

Location in Feet		Crest Elevation	Water Surface Elevation		Depth of Flow	
			Chute Centerline	Left Pier	Chute Centerline	Left Pier
Feet From CBL	Feet From Pier Nose	ft	ft	ft	ft	ft
-	-	-	264.9	266.3	-	-
-45.2	0.8	243.6	-	263.1	-	19.5
-44.4	1.6	244.3	-	262.1	-	17.8
-43.5	2.5	244.7	-	261.5	-	16.8
-42.7	3.3	245.0	-	260.9	-	15.9
-41.9	4.1	245.3	264.4	259.8	19.2	14.5
-37.8	8.2	245.4	262.6	257.1	17.3	11.7
-33.7	12.3	244.5	261.0	254.9	16.5	10.3
-29.6	16.4	242.9	259.1	252.8	16.2	9.9
-25.5	20.5	240.7	256.5	250.3	15.8	9.6
-21.4	24.6	237.8	253.8	247.0	15.9	9.1
-17.3	28.7	234.8	250.0	243.4	15.2	8.6
-13.2	32.8	231.8	246.4	241.6	14.6	9.8
-9.1	36.9	228.8	243.3	237.9	14.5	9.1
-5.0	41.0	225.8	239.6	235.2	13.8	9.4
4.0	50.0	219.2	233.0	230.1	13.8	10.9
11.4	57.4	213.8	225.0	224.6	11.2	10.8
19.6	65.6	207.8	217.1	218.7	9.4	10.9
37.0	83.0	195.7	204.1	205.7	8.4	10.0

Notes: 1) Discharge = 594.6 m^3/s (21,000 cfs)

Table 3-2. Static Pressure on RSW

Pressure Tap Location	Elev	Pressure Head: Forebay WSE = 264 ft, Bay Centerline	Near the Pier	Forebay WSE = 268 ft, Bay Centerline	Near the Pier
(ft)	(ft)	(ft)	(ft)	(ft)	(ft)
- 4.7	244.3	9.0	12.3	8.6	12.7
0.0	245.7	4.5	0.8	4.1	-2.5
3.8	245.1	3.3	10.0	3.3	11.9
15.5	239.5	3.7	no readings	4.5	no readings
25.0	232.4	7.0	4.5	8.6	6.6
44.1	218.9	6.2	4.9	7.8	2.5
45.9	217.6	4.9	no tap	7.4	no tap
48.6	215.8	6.6	7.4	6.6	9.8
52.3	212.9	7.8	4.9	7.8	7.4
60.5	206.9	5.3	4.1	7.0	4.1
71.8	198.6	6.2	2.1	6.2	2.9
83.8	189.8	-1.2	no tap	-1.2	no tap
94.0	179.9	1.2	no tap	0.4	no tap
104.6	169.1	-1.6	no tap	-2.1	no tap
71.38[4]	198.6	-5.3	no tap	-4.5	no tap
70.98[5]	198.6	0.4	no tap	-0.4	no tap
34.25[2]	227.2		6.2		10.7
34.25[3]	232.2		0.4		2.9

Note : 1) Pressures presented are relative to elevation of the pressure tap
2) Pressure tap located downstream of the pier offset 1 ft above the crest
3) Pressure tap located downstream of the pier offset 6 ft above the crest
4) 1 inch (prototype) discontinuity was added 0.4 ft upstream of Pressure Tap 11
5) 1 inch (prototype) discontinuity was added 0.8 ft upstream of Pressure Tap 11
6) Pressure tap location 0.0 is at RSW crest axis

Table 3-3. Dynamic Pressure on RSW

Pressure Tap		Pressures (ft)							
		Forebay WSE = 264 ft				Forebay WSE = 268 ft			
Location	Elev (ft)	Mean	Max	Min	St. Dev.	HGL	Max	Min	St. Dev.
15.4	239.9	3.7	6.7	-0.8	1.2	4.5	6.1	2.9	0.7
25.0	232.5	7.0	9	4.7	0.7	8.6	13.1	7.4	1.1
41.4	220.9	6.2	9.6	3.2	1.4	7.8	10.8	7.4	1.2

Note: Pressure tap location in feet downstream from RSW crest axis.

Table 3-4 Computed Velocities On RSW

Distance From CBL (ft)	Distance From RSW Crest Axis (ft)	PE 262 Velocity (fps)	PE 265 Velocity (fps)	PE 268 Velocity (fps)
-150.00	-110.3	1.78	2.28	2.87
-60.00	-20.3	1.79	2.31	2.90
-55.00	-15.3	1.80	2.31	2.91
-50.00	-10.3	1.80	2.32	2.92
-45.00	-5.3	13.65	15.50	17.56
-40.00	-0.3	18.02	19.89	22.06
-35.00	4.7	20.67	22.47	24.65
-30.00	9.7	23.65	25.38	27.58
-25.00	14.7	27.03	28.56	30.72
-20.00	19.7	32.34	33.92	35.92
-15.00	24.7	36.64	38.20	40.10
-10.00	29.7	40.24	41.76	43.59
-5.00	34.7	43.41	44.90	46.68
0.00	39.7	46.31	47.76	49.49
5.00	44.7	48.99	50.41	52.09
10.00	49.7	51.50	52.89	54.53
15.00	54.7	53.87	55.23	56.83
20.00	59.7	56.12	57.46	59.03
25.00	64.7	58.27	59.59	61.12
30.00	69.7	60.34	61.63	63.13
35.00	74.7	62.34	63.60	65.07
40.00	79.7	64.26	65.50	66.94

Note: CBL station 0.0 is 39.7 ft downstream of the RSW crest axis. Positive distances are downstream.

The tailpiece is roughly triangular in shape with a vertical upstream face and a downstream sloping face of 0.7338 V : 1.0 H to fit tangent to the sloping downstream face of the RSW main section. The tailpiece crest elevation is 221.3 ft. Therefore, at normal high operating pool elevation 268 ft, the free flow head is 46.7 ft. The physical model was used to evaluate hydraulic pressures on the tailpiece both with and without gate control (Table 3-5). With pool elevation 268 ft, static pressures as low as minus 11.1 ft were measured on the tailpiece with a gate opening of 8 ft. At the PMF pool elevation of 276 ft, static pressures as low as minus 29.6 ft existed. If only the tailpiece itself were in place under free flow conditions, local dynamic pressures as low as absolute zero could be expected on the tailpiece section. Such conditions could lead to failure of the tailpiece section and significant damage to the concrete on the face of the existing spillway. Therefore, free flow operation of the RSW with only the tailpiece section in place is not acceptable. The physical model data also indicates that gate controlled operation with only the tailpiece in place would be risky at gate openings greater than about 5 ft. Therefore, any operation with only the tailpiece in place should not be attempted except in remote, emergency conditions. Testing of an ogee-shaped tailpiece section in the physical model did not reveal any improvement in pressures on the tailpiece (Table 3-6). With the more streamlined design shape, minimum pressures of about minus 12 ft existed with a gate opening of 8.9 ft and pressures under free flow conditions with a pool elevation of 276 ft were absolute zero (prototype).

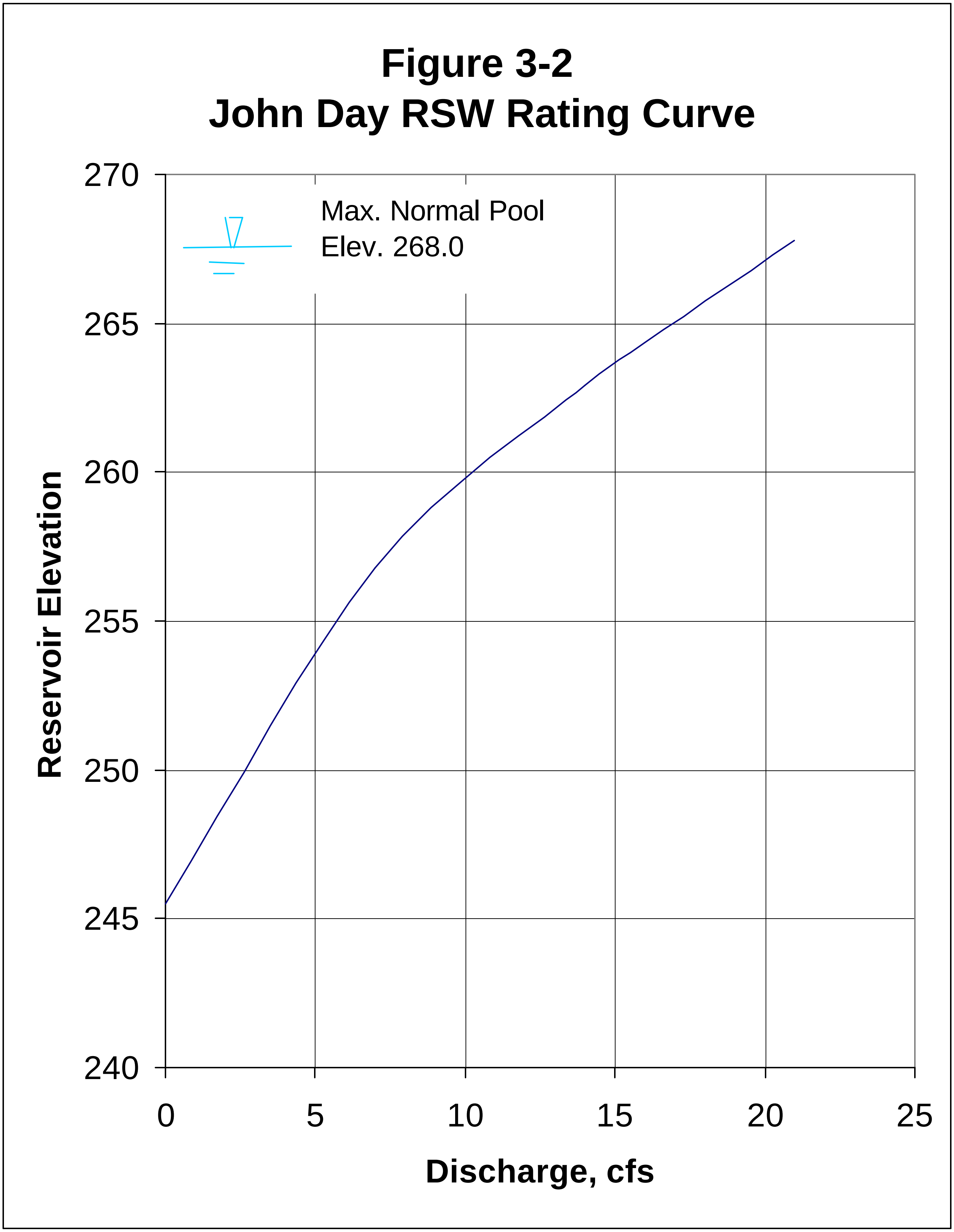


Figure 3-2
John Day RSW Rating Curve
Max. Normal Pool
Elev. 268.0
270
265
260
255
250
245
240
0
5
10
15
20
25
Reservoir Elevation
Discharge, cfs

Table 3-5. Static Pressure on Sharp-Crest Tailpiece

Pressure Tap		Pressure Head (ft)													
		Gate Opening (Go) = 5.6		Gate Opening (Go) = 8.0		Gate Opening (Go) = 9.6		Gate Opening (Go) = 10.9		Gate Opening (Go) = 11.6		Gate Opening (Go) = 12.7		Ungated Flow	
Dist. (ft)	Elev ft	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier
- 1.2	210.0	58.1		57.7		57.3		56.9		56.9		56.9		62.6	
0.0	214.8	53.1		52.9		52.9		52.5		52.1		52.1		57.0	
1.9	219.8	5.3	4.0	-11.1	-10.7	-15.2	-16.1	-14.4	-14.8	-14.8	-13.6	-15.2	-15.6	-25.5	-29.6
3.9	218.4	21.8		-11.0		-15.1		-14.3		-14.7		-15.1		-24.5	
8.8	214.8	11.5	11.5	12.3	13.1	10.7	13.1	-4.5	-2.4	-9.8	-7.4	-12.1	-14.7	-20.5	-18.8
13.0	211.7	-1.9	-3.5	3.1	0.6	10.4	7.2	12.5	3.9	7.6	2.2	6.7	8.0	-18.7	-16.6
23.0	204.4	3.2	2.4	4.6	4.0	6.3	4.9	9.5	6.5	12.0	12.2	12.4	15.5	-13.4	-13.2
32.5	197.4	-1.9		-0.6		0.2		1.4		2.2		3.1		1.0	
44.3	188.3	0.3		1.1		1.5		1.9		2.3		2.7		24.9	
55.0	178.0	0.3		1.1		2.8		3.2		3.6		4.0		33.9	

Notes: 1) Pressures are relative to elevation of the pressure tap

2) Forebay pool elevation 268.0 ft during gated tests and 276.0 ft during ungated tests.

3) Gate Opening (Go) is the minimum gate opening defined by the distance from the gate sill to a point perpendicular to the tailpiece crest.

4) Distance 0.0 is at axis of tailpiece section crest. Pressure tap at that location was in the vertical, upstream face of the tailpiece section.

Table 3-6. Static Pressure On Ogee-Crest Tailpiece

Pressure Tap		Pressure Head (ft)													
		Gate Opening (Go) = 6.1		Gate Opening (Go) = 7.7		Gate Opening (Go) = 8.9		Gate Opening (Go) = 9.3		Gate Opening (Go) = 9.7		Gate Opening (Go) = 10.8		Ungated Flow	
Dist. (ft)	Elev (ft)	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier	Bay C.L.	Near the Pier
- 3.1	210.0	54.8		56.9		56.1		56.5		56.5		56.1		60.2	
- 1.9	214.8	51.4		51.6		50.8		50.8		50.8		49.2		52.9	
0.0	218.4	8.3	6.6	-4.4	-5.7	-11.8	-12.3	-13.4	-14.7	-17.1	-18.0	-26.1	-27.0	< -75	< -75
2.0	218.1	1.2	0.5	-8.2	-9.0	-8.2	-9.0	-14.8	-15.5	-16.8	-17.6	-23.4	-24.1	-57.9	-61.1
4.2	216.8	-1.6	-0.9	-4.0	-3.8	-4.9	-5.4	-5.3	-5.9	-6.5	-7.9	-8.6	-8.3	-59.8	-59.6
6.9	214.8	0.1	1.7	-0.4	1.3	-0.2	1.5	0.1	1.7	-0.2	1.5	-0.8	0.9	-29.1	-27.4
11.1	211.7	0.6	2.3	1.4	3.1	1.9	3.5	1.9	3.5	2.3	3.9	2.5	4.1	-11.3	-9.6
21.1	204.4	5.5	7.4	5.9	7.8	6.7	8.6	6.7	8.6	7.1	9.0	7.5	9.4	6.7	8.6
30.6	197.4	3.3		5.5		5.5		6.4		6.0		6.4		17.9	
42.4	188.3	4.4		5.2		5.2		6.0		5.6		6.0		26.5	
53.1	178.0	4.4		5.2		5.6		6.4		6.9		6.9		not recorded	

Notes:

(1) Distance 0.0 is at axis of tailpiece crest

(2) Measured pressures less than about minus 33 ft in model reflect absolute zero in prototype

3.7.2 Interface at Existing Spillway Surface

The downstream end of the tailpiece section will terminate somewhat upstream of the true tangent point with the spillway to provide adequate plate thickness (about ¼-inch) to meet structural requirements. Dynamic pressures as low as 0 ft were measured in the physical model at this location with a smooth transition. With an abrupt 1-inch (prototype) offset built into the model, pressures decreased to as low as about –12 ft. The computed velocity at this location is 60 fps. The computed C.I. at this location is 0.31, therefore care must be taken to ensure that surface irregularities are controlled to ensure that cavitation conditions will not be initiated. Figure 2.4 of the USBR Engineering Monograph No. 42 indicates that a transition slope of 1 vertical on 6 longitudinal should be acceptable to minimize cavitation potential under such conditions. The downstream end of the steel tailpiece section will be welded to a separate steel plate to be imbedded in the spillway face. The 0.25 inch offset will be filled with weld material that will be ground smooth at a slope of 1 vertical to 6 longitudinal to provide a smooth transition between the tailpiece and the spillway.

3.7.3 Pier Design

The final design pier nose shape is slightly different from the design shape of the existing John Day Dam spillway piers. The existing pier nose was developed during extensive physical model testing prior to construction of the dam. The existing spillway pier nose was shaped in the form of an ellipse having a minor axis of about 6.3-ft, slightly larger than the 6-ft half-width of the spillway pier. With this geometry, the intersection of the downstream end of the pier nose and the face of the pier itself introduced a separation point that contributed to maximizing spillway discharge efficiency. However, as illustrated in the physical model tests accomplished as part of the design process, this flow separation initiated standing waves and resulted in unstable hydraulic "ridges" and roostertails that extended down the entire face of the spillway and flow deflector. These hydraulic conditions were considered to be unacceptable with respect to safe fish passage. In an attempt to eliminate, or minimize the formation of standing waves off the pier nose, the RSW pier nose ellipse has a minor axis of 6 ft, the same as the half width of the pier. This geometry forms a true tangency at the intersection of the downstream end of the pier nose and the pier face and is expected to improve flow characteristics as compared to those existing with the existing spillway pier nose geometry. Physical model testing of the final design pier shape indicates that flow conditions over the crest with this design geometry are consistent with those occurring with the existing pier nose geometry.

In order to provide sufficient structural integrity, an abrupt away-from-the flow offset of 3-inches will exist at the connection between the downstream end of the RSW pier and the face of the existing spillway pier. Velocities of about 45 fps exist at that location, and the minimum pressure measured in the physical model just downstream from the pier offset is about 0.4 ft. The C.I. for those conditions is about 1.0. Plate 2-6 of the EM suggests that the C.I. for incipient cavitation with a 3-inch abrupt away-from-the flow offset under those pressure and velocity conditions is about 0.9, therefore, cavitation should not occur at the downstream end of the RSW pier.

Physical model testing also illustrated that a large (about 4-ft) drawdown existed around the pier nose with the pier length extending 46 ft upstream of the existing spillway crest. The initial thought was that elimination of this drawdown might reduce the standing wave phenomenon observed emanating from the pier nose and, subsequently, improve hydraulic characteristics down the face of the spillway. Model testing with the piers extended approximately 18 ft further upstream (about 64 ft upstream of the spillway crest) into the forebay where lower velocities existed illustrated that the drawdown around the piers could be decreased to about 1 ft and appeared to decrease the standing wave formation. However, contrary to initial thoughts, this decrease in drawdown had no improvement on the hydraulic conditions existing down the face of the spillway. Observation of approach flow conditions in the 1:80 scale general model indicated that piers extended into the forebay about 41 ft upstream of the existing spillway crest resulted in somewhat better approach conditions than existed with piers extending about 65 ft upstream. Therefore, extending of the piers upstream beyond 46 ft from the existing spillway crest was not considered beneficial.

Surging of the water surface in spillway bays upstream from tainter gates has been observed with gate-controlled operation in some models. This condition is attributed to critical combinations of spillway bay geometry and gate opening. Such surging is undesirable because it creates unbalanced loading on the piers, gates, etc. The RSW will not be operated with gate-controlled conditions; therefore surging should not normally be an issue. Guidance in EM 1110-2-1603, paragraph 3-7 was used to evaluate the potential for surging with the RSW in place. The pier length to spillway bay width ratio is about 1 and the spillway bay width to maximum head at which the gate controls ratio is about 3.5. Based on the EM, surging should not be expected for such conditions. However, transition between discharge control at the gate and at the crest will exist for short periods of time when the RSW is being put into, or taken out of, service. Observations in the physical model did indicate the existence of turbulent, surging conditions upstream of the gate through a small change in gate opening as the discharge control shifts between the gate and crest of the RSW. See Figure 3-3. Project operations will need to consider this potential condition and ensure that the RSW is not operated for any long duration under control shift conditions (see paragraph 3.10).

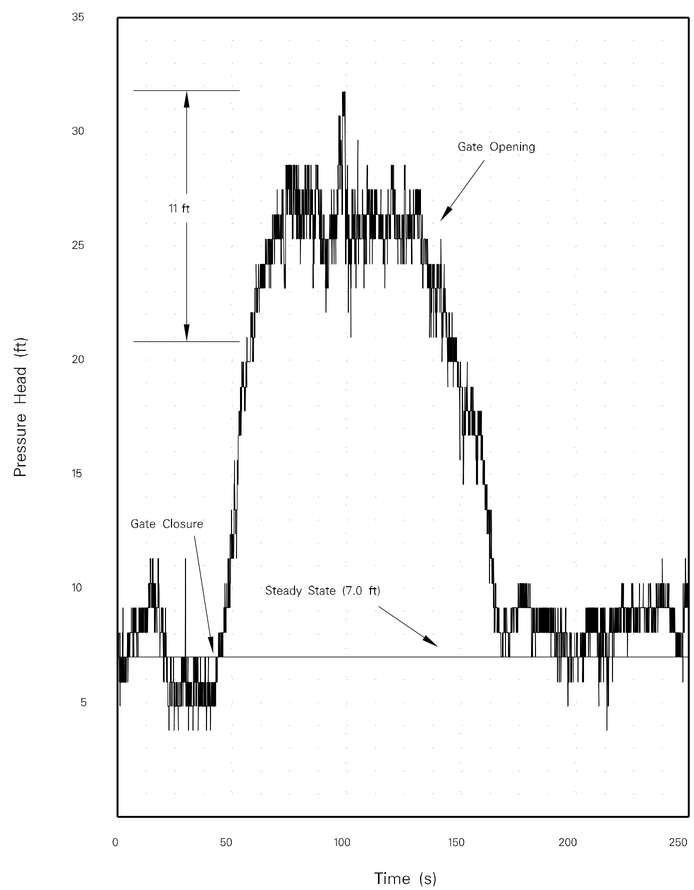
3.8 Hydraulic Model Studies

Various RSW configurations for this study were tested in physical hydraulic models located at Northwest Hydraulic Consultants' Vancouver, British Columbia laboratory and at the COE's Engineering Research Development Center (ERDC) in Vicksburg, Mississippi. Both models were used to develop hydraulic data and to characterize performance of selected RSW configurations, and to help the District and the regional fisheries resource agencies select the most successful design.

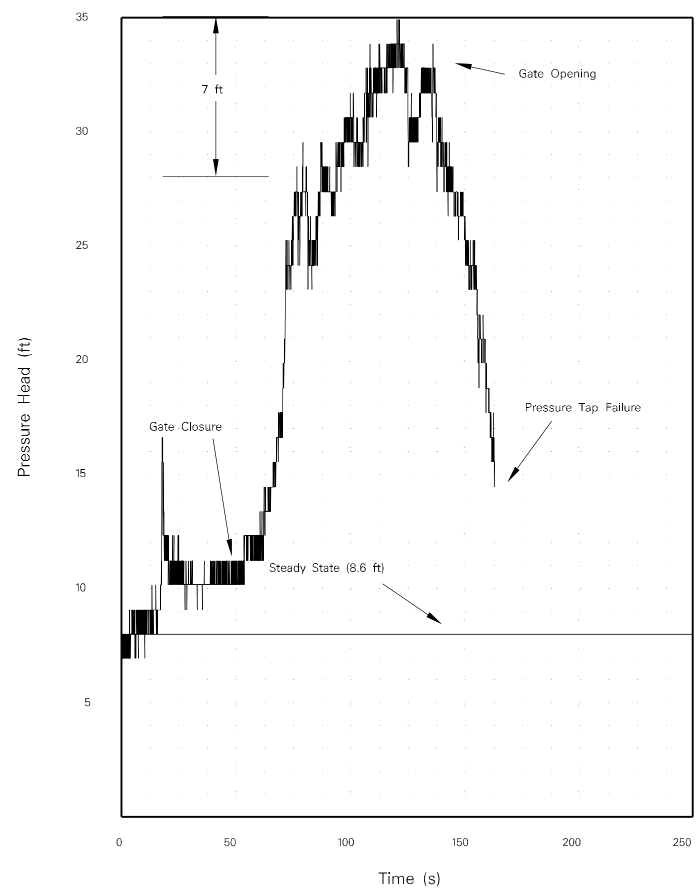
3.8.1 Sectional Model Studies

Sectional model studies were accomplished at Northwest Hydraulic Consultants' Vancouver, B. C. laboratory facilities. The 1:25 scale model simulated John Day Dam Spillway Bay 20, half of Bay 19, and the non-overflow dam and a portion of the Skeleton Bay 20. Selected RSW configurations were constructed to insert into the model for evaluation purposes. In addition to

Forebay WSE = 264.0 ft



Forebay WSE = 268.0 ft



Notes
1) Time in prototye seconds
2) Pressures measured on RSW approximately 20 ft upstream of RSW gate seat

John Day Dam RSW

Hydraulic Model Study

Dynamic Pressures on RSW
During Gate Closure and Opening

FIGURE 3–3

the final design, Proof of Concept Alternatives 2, 4 and a simplified version of Alternative 5 were evaluated in the model facility. Standing waves were initiated by the drawdown around the pier nose with RSW Proof of Concept Alternatives 2, 4 design. These waves were then amplified by the reverse curve transition between the end of the RSW and the existing spillway face. The amplified waves generated unacceptable flow "ridging" and large roostertails that traveled down the face of the spillway and impacted on and downstream from the spillway deflector. Preliminary testing of a concept with extended piers resulted in considerably less drawdown around the pier nose, but did not improve hydraulic conditions downstream from the transition bucket between the RSW and the existing spillway. The abrupt step feature at the downstream end of the RSW with Proof of Concept Alternative 4 also had little significant improvement on hydraulic conditions downstream of the RSW. The only design concept providing acceptable hydraulic conditions downstream from the RSW was Proof of Concept Alternative 5, which eliminated the reverse curve transition between the RSW and the existing spillway face. Discussion of the initial RSW configuration testing and observation is provided in the Physical Model Alternatives Report in Appendix E. Physical model studies used in verification of the final RSW design have been completed, however, the final draft Physical Model Study Report will not be completed until after submittal of the final DDR.

3.8.2 General Model Studies

The 1:80 scale general model consists of a large portion of the forebay and reservoir for the John Day Dam project, the dam (including the powerhouse, spillway, navigation lock, and abutments), and a large portion of the tailwater channel and downstream river. Proof of Concept Alternative 2, a design having extended piers and the final RSW design were evaluated in the model facility. The approach conditions to the Proof of Concept Alternative 2 RSW were quite similar to those existing with the Skeleton Bay SBS. The design having piers extending upstream further into the forebay than does Alternative 2, illustrated no improvement over the Alternative 2 configuration, and in fact had more areas of "dead" water behind the exterior faces of the approach piers. Observations in the general model indicated that the overall "zone of influence" or the attraction flow net to the RSW decreases as overall spillway flow increases.

3.9 RSW Operation

The purpose of the RSW is to test the potential effectiveness of surface collection against the presently preferred method of spill through the existing spillway gates. Although the actual RSW/Spillway testing plan has not been fully defined at this time, the District anticipates that RSW tests will likely be made several times each month during the juvenile migration period. The testing period would likely begin in early spring, as river flows begin to rise in response to mountain snowmelt. The RSW would be attached to the spillway and sampling of passing fish would be conducted for several days. Then data for the spillway passage of fish would be recorded. As river flows increase during spring, the juvenile migration will also increase. The RSW testing might be conducted as frequently as once a week during the peak migration period. Spillway flows will also likely peak during this time period.

The RSW is expected to be fairly successful at capturing and passing juveniles during periods of low to moderate river flows, such as early spring or late summer. During these times, the unit discharge, and consequently the attraction flow net in the forebay of the RSW will be proportionally greater than that for the smaller flows passing simultaneously through the remaining 18 or 19 spillway bays. However, as river flows and overall spillway flows increase to the maximum attained during the peak migration season, the relative attraction to the RSW is expected to be less, since the unit discharge through the spillway bays will be nearly equal to or greater than that through the RSW. There are currently varying estimates as to the proportional discharge limit at which the RSW will become ineffective. In general the RSW efficiency for surface flow collection and withdrawal is thought to become largely ineffective at total spillway flows (including the RSW) greater than about 150,000 cfs. The physical modeling work on the 1:80 scale model at WES confirms that the overall "zone of influence" or areas of attraction flow net to the RSW is diminished above these higher spillway flows. However, it remains to be seen in the actual field tests whether the fish behavior and attraction effectiveness can be directly inferred from the hydraulic performance results observed in the model.

See Section 4 for detailed discussion of the RSW structural design and Section 8 for installation of the ballasted RSW structure.

3.10 Tainter Gate Operation

The existing function of the spillway control tainter gate will not be adversely affected by the RSW structure. Presently, normal operation of the spillway gate requires that the gate be lifted (or closed) in incremental amounts, or "stops", to control spill bay discharge. Typical spillway flows normally don't require that the spillway gates are opened any more than perhaps ten "stops", or around 10 to 15 feet above the gate seat. The relative openings of adjacent spillway gates are dictated by the "spill pattern" prescription, coordinated with by the National Marine Fisheries Service, to aid in fish passage. However, during extremely large flood events, the spillway gate openings are quite large. The spillway gates are only lifted clear of the flow during the most extreme hydrologic events, such as the Spillway Design Flood. The existing spillway ogee crest shape was designed for these free flow spillway discharges.

The RSW crest shape was designed specifically for free flow conditions. When the RSW is in place, the spillway gate will be lifted clear of the flow passing down the RSW chute. This will protect any passing fish from injury due to impact with the gate or turbulent shear in the gate approach area. However, very unsatisfactory hydraulic conditions were observed in the sectional model upstream of the spillway gate during gate opening and closing with the RSW in place as the discharge control shifted between crest and gate control. A maximum peak-to-peak pressure fluctuation of 11 ft was measured in the physical model on the RSW upstream from the gate during gate opening and closing operations (paragraph 3.7.3). Extreme turbulence and vortices which would likely result in vibration of the gate existed. Under such conditions, the gate and RSW would be subjected to rapidly varying, uneven pressures. Therefore, project operations will need to be aware of such undesirable operations and ensure that the RSW is not operated for any significant duration of time under shifting discharge control conditions

The RSW structure will include a tailpiece section that will rest on top of the existing spillway gate seat. As discussed in paragraph 3.7.1, operation with only the tailpiece section in place will create significantly low pressures on the tailpiece. These low pressures compromise safe operation of the spillway, therefore operation with only the tailpiece in place must be avoided. Except in extremely remote emergency conditions, i.e., upstream dam failure, the spillway bay will not be operated with only the tailpiece in place.

3.11 Flow Deflector

A deflector on the spillway below the RSW will be constructed to simulate the Skeleton Bay SBS and reduce TDG generation in the stilling basin downstream of the Spillway Bay 20 RSW. Design of the Spillway Bay 20 deflector is contained in John Day Dam Supplement No. 1 to Design Memorandum 50, Spillway Flow Deflectors.

SECTION 4 STRUCTURAL DESIGN

4.1 General Description

The shape and location of the RSW on the spillway is dictated by the RSW hydraulic performance in the model studies. Additional RSW geometries were evaluated before the "Proof of Concept" Alternative 5 was selected. A structural analysis for the DDR was performed and the details of the seals and connections have been developed. There are three main structures involved in the selected proof of concept alternative: 1) RSW Main Structure, 2) RSW Tailpiece, and 3) RSW Main Structure Attachment. These structures are described in this section.

The design life of the RSW Main Structure, Main Structure Attachment, and Tailpiece is three years which is the testing period for the RSW.

4.2 Structural Design Criteria

4.2.1 References

1. EM 1110-2-2105 Design of Hydraulic Steel Structures, updated 5-31-94
2. ANSI/AWS D3.5-93, Guide for Steel Hull Welding,
3. EM 1110-2-2200 Gravity Dam Design
4. EM 1110-2-1806 Earthquake Design and Evaluation for Civil Works Projects

4.2.2 Removable Spillway Weir (RSW) Steel Component Design

The design of the steel components of the RSW conforms to the requirements of References 1 and 2.

4.2.3 Dam Operating Parameters

Forebay Pool Elevations:	
Design Pool:	
Maximum Operating Forebay Elevation:	268 ft (NGVD)
Minimum Operating Forebay Elevation: (Minimum Pool with RSW in Place)	257 ft (NGVD)
Maximum Flood Pool:	276 ft (NGVD)
Minimum Pool with RSW in Place:	257 ft (NGVD)
Range of Tailwater Elevations:	
Minimum:	155 ft (NGVD)
Normal Operating:	160 ft (NGVD)
Normal High Flow:	165 ft (NGVD)
Spillway Base Elevation:	100.0 ft (NGVD)
Spillway Base Width:	62 ft
Spillway Base Length:	164 ft

For this report the base length is the horizontal length of the spillway at the plane of the stability analysis.

4.2.4 Material Weights

Water: 62.4 PCF
Steel: 490 PCF
Portions of the RSW are assumed to be full of water.

4.2.5 Foundation Uplift

For this report uplift is applied uniformly over 100 percent of the base area at the spillway base elevation.

4.2.6 Hydrostatic Forces

The hydrostatic forces experienced by the RSW vary linearly with depth. Depth ranges are from 11 feet in the transit condition to 74 feet when installed.

4.2.7 Ice Pressure

Ice build-up is not expected at John Day Dam for the anticipated spring and summer months of operation for the RSW. It has been assumed ice loads are not a factor in the design.

4.2.8 Seismic Forces

The definition of the Operating Basis Earthquake (OBE) and the Maximum Credible Earthquake (MCE) shown in Load Case 3 and Load Case 4 in Section 4.3 follows:

> **Operational Basis Earthquake (OBE):** The OBE is an earthquake that can reasonably be expected to occur within the service life of the project, that is, with a 50 percent probability of exceedance during the service life. This corresponds to a return period of 144 years for a project with a service life of 100 years. The associated performance requirement is that the project function with little or no damage, and without interruption of function. The OBE for this project has been determined to be 6 percent of the acceleration of gravity.
>
> **Maximum Credible Earthquake (MCE):** The MCE is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source, on the basis of seismological and geological evidence. The MCE for this project has been determined to be 19 percent of the acceleration of gravity.

4.2.9 Load Cases

Separate load cases were developed for the three structures analyzed in this DDR. These structures are 1) The RSW Main Structure, 2) the RSW Tailpiece, and 3) the entire spillway with RSW installed. The load cases for each of these structures are given in the discussion of these structures later in this section.

4.2.10 Material

Steel:

Wide Flange Sections
Plate: ASTM A36 or ASTM A572
Other shapes: ASTM A36
Structural Tubes: ASTM A500 Grade B
Structural Pipe: ASTM A53 Grade B or API 5LX42
Bolts: ASTM A325
Anchor Bolt Grout: Two component epoxy, ASTM C-881

4.3 Stability Analysis

The stability analysis of the spillway with RSW in place is described in this submittal. Criteria used in the analysis are shown below and in Section 4.2.

4.3.1 Foundation Design Parameters

Allowable Bearing Strength	150 kips/sq ft
Coefficient of friction for sliding	Tan 40 degrees (concrete/rock interface)

Sliding factor of safety

1. Case 1 = 2.0
2. Case 2 = 1.7
3. Case 3 = 1.7
4. Case 4 = 1.3

Location of Resultant for Overturning

1. Case 1 = Within Middle 1/3 of Base Width
2. Case 2 = 75% Base in Compression
3. Case 3 = 75% Base in Compression
4. Case 4 = Within the Base Width

Load Conditions

1. Case 1 = Normal Operating Condition: Reservoir at elevation 264.0 feet with tailwater at elevation 161.4 feet, with uplift.
2. Case 2 = Flood Discharge Condition: Maximum pool elevation 276 feet with tailwater at elevation 205.3 feet, with uplift.
3. Case 3 = Normal Operating Condition and Operating Basis Earthquake (OBE). Combine Case 1 with a 6%g (OBE) earthquake.

4. Case 4 = Maximum Credible Earthquake (MCE). Combine Case 1 with a 19% g (MCE) earthquake.

Overturning Stability Assumptions

1. The structure is rigid (non-deforming) structure that bears on an elastic foundation.
2. The structure rotates about the centroid of its base area in the global Z direction.
3. Any base area not considered rigid will not be included in base properties.
4. Moment arms for horizontal loads are measured to the base centroid elevation.
5. Final foundation bearing pressures are determined by adding the uplift pressures to the base pressures.

4.3.2 Stability Analysis and Results

The purpose of this analysis was to determine if the addition of the Surface Bypass Removable Spillway Weir to Spillway Bay 20 would affect the stability of the bay compared to the original design of the dam. In order to perform the analysis Design Memorandum Number 16 by the U. S. Army District, Walla Walla Corps of Engineers dated 11, August, 1959 was reviewed. In addition, Feature Design Memorandum Number 52 dated September 1998 and Supplemental Stability Calculations for Feature Design Memorandum Number 52 were reviewed.

Section 6 and Plate numbers 46 through 52 of Design Memorandum Number 16 show the design criteria and results of the original stability analysis for a typical spillway bay of the dam. It should be noted that spillway bay number 20 is not identical to a typical spillway bay for the dam. The difference between a typical spillway bay and bay number 20 is that the piers for bay number 20 are much larger and have better stability characteristics than those for a typical bay. However, the spillways themselves are identical. Therefore, it was decided that if the Removable Spillway Weir were placed at a typical bay and it did not significantly affect the stability of that bay, the stability of bay number 20 would not be affected by the weir. Consequently, the stability analysis is compared to the original analysis for a typical spillway bay.

If you compare the design criteria in Design Memorandum Number 16 to the design criteria shown in Section 4.3.1 of this report there is one major difference. Load Cases 3 and 4 of this report include a loading combination of the Normal Operating Levels for the reservoir and the tailwater, which includes seismic loads. In Load Case 3 the seismic condition includes loads for a 6% g or Operating Basis Earthquake (OBE) and Load Case 4 includes loads for a 19% or Maximum Credible Earthquake MCE). In the original analysis there was only one seismic loading condition and that included loads for a 10% g earthquake. Since the purpose of this investigation is to compare the affect of the Removable Spillway Weir on the original stability of the spillway, the seismic loads from the original design were used for the analysis. It should also be noted that, during the flood discharge condition (Load Case 2 of this report), the Removable Spillway Weir will be removed from the spillway and Load Case 2 becomes identical to the original design. Consequently, Load Case 2 was eliminated from the analysis and Load Cases 3 and 4 were combined leaving only two load cases for the analysis. Please note that the only other significant difference in loading between the original analysis and this analysis is how the hydraulic lateral pressures are applied to the top of the spillway at the Removable Spillway Weir.

These lateral pressures were provided by Glosten Associates, Inc. and were substituted for the lateral pressures in the original design.

The results of the analysis for Load Case 1 showed that the actual bearing pressures were well within the allowable bearing strength of 150 ksf shown in Section 4.3.1 of this report. It also showed that the location of the resultant for overturning is within the middle 1/3 of the spillway base width, which also meets the criteria for design. However, the safety factor for sliding was approximately 1.94, which is slightly below the required safety factor of 2.0. The reason that the safety factor for this analysis is slightly lower that 2.0 is that, with the Removable Spillway Weir in place and the existing radial gate in the fully open condition, there is an area just upstream of the radial gate that has less water flowing over the weir than the original design. The affect that this has on the analysis is that there is slightly less weight which can be relied on to resist sliding. However, since the existing piers at spillway bay number 20 are significantly larger that a typical spillway bay, the extra weight of the piers will make up for the weight of the water and the safety factor will exceed 2.0. Therefore, it appears that all of the design criteria for the analysis have been met for spillway bay number 20 and the Removable Spillway Weir will not affect the stability of the dam.

The results of the analysis for the seismic load case showed that the actual bearing pressures were well within the allowable bearing strength of 150 ksf shown in Section 4.3.1 of this report. It also showed that the location of the resultant for overturning is within the middle 1/3 of the spillway base width, which also meets the criteria for design. However, the factor of safety for sliding was approximately 1.26, which is slightly below the required safety of 1.3 for Load Case 4 and well below the safety factor of 1.7 for Load Case 3. At the time of the original analysis there was only one load case assumed for seismic design and no distinction between an Operating Basis Earthquake and a Maximum Credible Earthquake. The required safety factors for design for seismic at that time were likely based on the worst expected reasonable design earthquake. This is more applicable to Load Case 4, which has a required factor of safety of 1.3 for the Maximum Credible Earthquake. Therefore, for comparison purposes, a required factor of safety of 1.3 was used. The slight difference between the safety factor in the new analysis and the existing analysis for this load case can be attributed to the difference in water depth above the weir as described above. Consequently, it appears that all of the design criteria for the analysis have been met for spillway bay number 20 and the Removable Spillway Weir will not affect the stability of the dam.

In summary, it appears that, based on the above analyses, the Removable Spillway Weir will not affect the stability of the dam.

4.4 RSW Selection

When developing the 30% Report, it was envisioned that the floating RSW would be located entirely upstream of the spillway tainter gate. With this assumption, it was possible to close the tainter gate in Spillway Bay 20 (and if required, Bay 19) and carry out the operation of attaching or detaching the floating RSW to the dam in the relatively calm water. However, during the hydraulic studies, it was determined that the shape and size of the RSW would be such that the downstream portion would extend downstream of the tainter gate. The reasons for selection of this RSW geometry are based on hydraulics and are explained in Section 3 Hydraulic Design.

Once the RSW geometry was set three aspects of the RSW design had to be selected. These aspects are: 1) design of the structure, 2) its method of attachment to the dam, and 3) its method of installation and removal. This section describes the selection of the RSW in relation to these aspects. Later sections describe the design and the installation and removal procedure.

Since the tainter gate rests on the RSW the entire RSW structure cannot be removed without allowing some flow past the tainter gate. Removing the RSW against up to 60 feet of head would be dangerous, if not impossible, and was not considered a viable option. Therefore, the RSW would have to be divided into two or more separate pieces.

After investigating several options it was decided to divide the RSW into two pieces to facilitate installation and removal. The best way to accomplish this was to divide the RSW just downstream of the existing bulkhead slots and upstream of the tainter gate. See Plates 2, 3, and 4. This would allow the downstream piece to be installed or removed in the dry behind the existing stop logs. The downstream piece, called the RSW Tailpiece, would be bolted to the dam. The tainter gate will close and rest on the tailpiece. The other portion of the RSW, called the Main Structure, would be located upstream of the tainter gate and tailpiece. The main structure is designed to float and be submersible for installation and removal. The two pieces will be connected to each other , with a Closure Plate, so that there is a smooth spillway surface.

In the 30% Submittal Report of June 2000, Three Options for attaching the RSW to the dam were investigated. RSW Option 1 was Float In and Out, in which a floating structure is moved in and out of position, by using a tugboat or similar vessel. This option was selected as the most viable. Option 2 called for hinging the RSW on its upstream side and rotating it into and out of position atop the spillway crest. This option was deemed too difficult to perform since the RSW would have to be flooded in order to rotate it into the stowed position. We believe that the flooding could not be controlled to insure that rotation of the one million pound structure could be slowed enough to prevent impact and damage upon reaching the stowed position. Option 3, moving the RSW on a track, was discarded for the same reasons. In the 60% and 90% submittals only Option 1 was examined further.

In the 60% Submittal Report of September 2000, installation and removal of the RSW Main Structure was proposed to be accomplished with support barge which was anchored in the forebay. This type of readily available support for removal was necessary because of a design requirement to remove the RSW with a very short period of time, time that would not allow for a tugboat to be brought to the site. During the 60% PRM, concern was raised about the support barge providing an area for fish and avian predators to gather. To address this concern, the two-day removal design criteria was changed. This 90% submittal proposes using a tugboat for installation and removal of the RSW Main Structure. Using the tugboat eliminates the need for the support barge anchored in the forebay.

The relaxation of the short removal period affected the design of the tailpiece. The design of the tailpiece is based on a one-time installation and one-time removal (at the beginning and end of the testing time frame). For this 90% submittal the tailpiece was redesigned to propose more conventional steel construction techniques.

4.5 RSW Design

4.5.1 RSW Main Structure

Load Conditions:

Case 0	Full Main Structure Weight	
	Tainter Gate Closed	
	Tanks Equalized to Forebay Level	
Case 1	Normal Operating	
	Design Pool Elevation	268 ft (NGVD)
	Dead Load (weight of RSW main structure)	
	Tainter Gate Open	
Case 2	Removal Operation	
	Design Pool Elevation	268 ft (NGVD)
	Dead Load (weight of RSW main structure)	
	Tainter Gate Closed	
Case 3	Normal Operating with OBE Earthquake	
	Design Pool Elevation	268 ft (NGVD)
	Dead Load (weight of RSW main structure)	
	Tainter Gate Open	
	Operating Basis Earthquake (0.06 “g”)	
Case 4	Normal Operating with MCE Earthquake	
	Design Pool Elevation	268 ft (NGVD)
	Dead Load (weight of RSW main structure)	
	Tainter Gate Open	
	Maximum Credible Earthquake (0.19 “g”)	
Case 5:	Maximum Wave Load	
	Design Pool Elevation:	268 ft (NGVD)
	Dead Load (weight of RSW main structure)	
	Tainter Gate Closed	
	5.2-ft significant wave height	
Case 6:	Stability during earthquake	
	Design Pool Elevation:	268 ft (NGVD)
	Deal Load (weight of RSW main structure)	
	Tainter Gate Closed	
	Maximum Credible Earthquake (0.19 “g”)	
	Hydrodynamic pressure due to MCE	

Since the maximum credible earthquake is an extreme event, a 33% increase in allowable stress will be used for evaluating the capacity of the RSW structure for Case 4.

Note the RSW main structure has not been designed to support the stop logs.

4.5.1.1 Structural Arrangement

The RSW main structure will consist of a 50-foot wide spillway section and piers on either side of the spillway. See Plates 2 through 5. The piers will extend up to the height of the existing piers at elevation 281.0 feet msl. The bottom of the main structure will rest on a platform described in section 4.5.3 below and on the existing spillway crest. When in position on the dam, the main structure is 78 feet high, 47 feet long, and 74 feet wide. In total it will weigh 530 long tons or 1,187,200 pounds when dry.

The main structure will be constructed of stiffened steel plate and steel framing. See Plates 6 through 9. Welded construction will be used throughout the structure (ANSI/AWS D3.5-93). Plate, formed to match the desired geometry, acts as the outside shell of the RSW. Internal compartments, needed for installing and removing the RSW, are created from stiffened plate bulkheads/diaphragms.

Global loads will be resisted by the entire structure (much as a ship's hull resists the forces and moments associated with seas and cargo). The principal global loading will be due to eccentric hydrostatic loads experienced during normal operation of the RSW. This load must be transmitted through the RSW structure to the dam pier noses and spillway monolith behind the lower sealing surface.

Hydrostatic loads will be applied against empty compartments in the Main Structure. The plate, stiffeners, frames and diaphragm bulkheads will resist these loads. Plates 6 through 9 show the structural members, flats, frames and bulkheads. Wherever practical, the compartments will be free-flooded so as to minimize the hydrostatic loading. Free flooding involves opening flood valves and vent valves to allow full upstream hydrostatic head to be applied to the inside of the compartments. Tanks 3N and 3S (those used to rotate the RSW for removal) will be designed to resist hydrostatic heads of up to 48 feet. Tanks 1, 2N, and 2S will be designed to resist a hydrostatic head of 29 feet.

The centerline bulkhead, which separates tank pairs 2 and 3, should not experience significant differential pressure in any of the RSW transit, installation, operation, and removal conditions. However, the centerline bulkhead was designed to the relief valve pressure setting (21 psig) for tanks 2 and 3.

4.5.1.2 Structural Design

The main structure has been designed using first-principle methods for sizing of plate, stiffeners, and frame members to withstand local hydrostatic pressure. In addition, a global finite element model was built and analyzed to design the structure to withstand the global structural loads.

The model, shown in Figure 4-1 below, was constructed in the MAESTRO (**M**ethod for **A**nalysis **E**valuation and **STR**uctural **O**ptimization) finite element analysis software, produced by Proteus Engineering of Stevensville, MD. MAESTRO is an industry-leading software package for the global analysis of stiffened-plate structures.

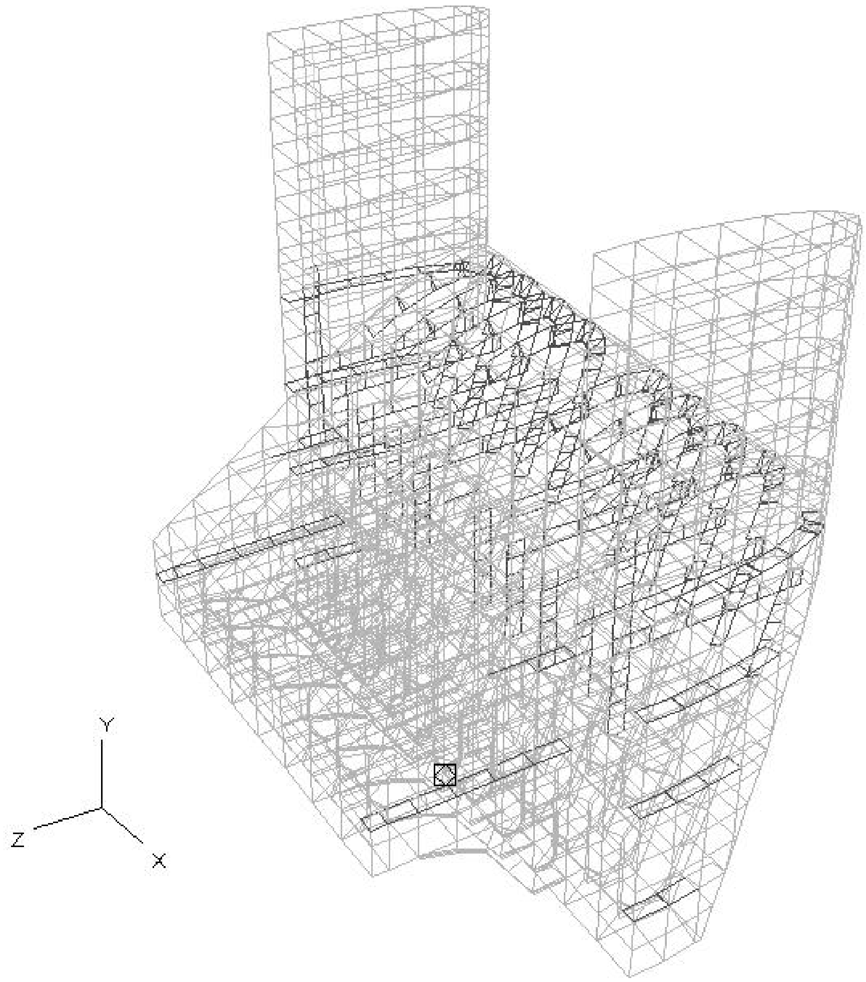


Figure 4-1
MAESTRO Global Finite Element Model

The following text describes each of the individual loadcases specifically considered in the structural design.

Load Case 0: Full Main Structure Weight
Tainter Gate Closed
Tanks equalized to forebay level

This case represents the full weight of the RSW Main Structure resting on the Main Structure Attachment. The only load included in this case is the self weight of the structure. This case corresponds to a condition in which the tainter gate is closed, and the tanks are flooded, or equalized to the pool elevation in the forebay. Because of the lack of any significant global loads, this case does not impact the structural design. However, the case provides the magnitude and distribution of the support reactions at the bottom of the structure. The load distribution per frame is provided in the Structural Design and Finite Element Analysis report. The total reactions are 674 kips on the upstream edge of the main structure attachment and 513 kips on the downstream edge.

Load Case 1 Normal Operating
Design Pool Elevation: 268 ft (NGVD)
Dead Load (weight of RSW main structure)
Tainter Gate Open

This load case represents the maximum static global load on the structure in which allowable stresses are at their minimum. For this case, the MAESTRO model was used to analyze the global structural loads, and local hydrostatic pressures were considered in the design of local stiffening members.

The model was loaded to a maximum hydrostatic head of 74 feet of water, corresponding to the 268-foot design pool elevation. For simplicity, hydrostatic pressures downstream of the spillway crest, and on all surfaces enclosed by the seal on the dam face, were assumed to be zero. This ensures that no downstream pooling of water is required for operation of the RSW.

Finite element analysis indicates that all stresses in the model are below allowable levels. In addition, the model was assessed for resistance to buckling and collapse failure modes. While the results indicate that buckling and collapse will not occur, local plate buckling due to high shear loads should remain a consideration during further design development. These areas include the plating of the 16', 28', and 40' flats, at the seal connections in the pier nose ends, and the bottom plating adjacent to the centerline bulkhead.

Analysis of the seal reactions in this case indicates that the displacement of the structure tends to pull the bottom corner of the RSW away from the dam face seal. While this deflection is small, and well within the flexible range of the seal, stiffness of the perimeter structure around the dam face seal should remain a consideration during further design development.

Load Case 2 Removal Operation
Design Pool Elevation: 268 ft (NGVD)
Dead Load (weight of RSW main structure)
Tainter Gate Closed

This load case represents the maximum local pressure loads on the pier nose structure. In this case, tank pairs 2 and 3 are pressurized to remove water. As a result, the differential pressure at the top of the tank structure is large, because the internal tank pressure remains constant while the external hydrostatic head decreases with increasing elevation.

Load Case 3 Normal Operating with Earthquake
Design Pool Elevation: 268 ft (NGVD)
Dead Load (weight of RSW main structure)
Tainter Gate Open
Operating Basis Earthquake (0.06 "g")

This load case represents the maximum global load on the structure in which allowable stresses are at their minimum. However, a finite element analysis was not performed in the DDR phase to verify the structural adequacy for this load case. Because this case is identical to Load Case 1, with a 0.06g acceleration superimposed on the hydrostatic load, a simplistic approach was taken to check the adequacy of the structure. The worst case direction for this acceleration is into the spillway; therefore, the acceleration can be approximated as an increase in the static load, equal to 6% of the RSW weight.

Superimposing 6% of the RSW weight on the total reaction at the dam face results in a 4% increase in load. The highest stress in the RSW structure resulting from Load Case 1 is 14.9 ksi. Increasing this stress by 4% yields a value of 15.5 ksi – still below the allowable limit. Therefore, based on this preliminary assessment, the structure is adequate to withstand this load case. More accurate assessment of the dynamic load was considered in subsequent design phases.

A complete discussion of the finite element analysis of this load case can be found in the Final Structural Design Report.

Load Case 4: Normal Operating with Earthquake
Design Pool Elevation: 268 ft (NGVD)
Dead Load (weight of RSW main structure)
Tainter Gate Open
Maximum Credible Earthquake (0.19 "g")

Since the maximum credible earthquake is an extreme event, a 33% increase in allowable stress will be used for evaluating the capacity of the RSW structure for Case 4. As with Load Case 3, this case was not explicitly modeled for finite element analysis in the DDR phase. However, using the same approach applied above, if a load equal to 19% of the RSW weight is superimposed on the static load, the overall load increases by 13%. Since the allowable stress for this case increases by 33% over the Load Case 1, the increase in load will not produce stresses in excess of allowable.

Again, as in Load Case 3, dynamic loads in this case were assessed to a greater extent as the design developed.

A complete discussion of the finite element analysis of this load case can be found in the Final Structural Design Report.

Load Case 5: Maximum Wave Load
Design Pool Elevation: 268 ft (NGVD)
Dead Load (weight of RSW main structure)
Tainter Gate Closed
Wave Height due to 90 mph fastest mile windspeed

Structural loads in this case are lower than those for any of the above cases. However, consideration in this case was given to the potential of wave loads causing the RSW to rotate or lift off its support pedestal. For these calculations, all tanks are assumed to be fully flooded.

The first task in this analysis was to hindcast the significant wave height (Hs) produced by 90 mph winds. From NOAA chart 18535, the limiting fetch at the site was measured to be about 4.4 nautical miles. Based on this fetch-limited condition, the wave height was calculated using the US Army Corps of Engineers' Shore Protection Manual (1984). It was found to be approximately 4.4 ft.

The second task was to determine approximate upper bounds on the wave loads on the weir. The wave loads were estimated using linearized potential flow theory, with the following additional assumptions. The loads were calculated for the statistical extreme wave height in one thousand cycles with a 10% probability of exceedance. This extreme wave height is 9.4 feet. The free-surface elevation was taken to be 257 feet, and the forebay bottom was assumed to be at 137-foot elevation. The weir was approximated as a vertical wall extending down – to about 187' elevation – from the free surface, allowing the velocity potential to be approximated by that due to a standing wave.

The fluid pressure on the RSW was evaluated using Bernoulli's equation, and the horizontal force exerted by the waves was computed by integrating the pressure over the projected frontal area of the RSW. The horizontal wave force acting on the RSW was estimated to be 295 kips. Without additional restraint, this force is adequate to cause the RSW to tip off the pedestal into the forebay. Therefore, tie-backs must be in place at the top of the RSW pier noses to hold the RSW in position during extreme wave conditions. These tie-backs must be able to resist a horizontal force of approximately 100 kips.

The uplift, or vertical force on the RSW, was also computed to determine if the RSW could be lifted from the pedestal. This force was calculated by assuming the above horizontal force to be a resultant of the pressure acting on the inclined forebay face. Taking the vertical component of this normal force, the uplift was estimated to be 168.3 kips. This value is significantly less than the weight of the RSW main structure. Therefore, the RSW cannot be lifted from the pedestal by the wave forces.

Load Case 6: Stability during earthquake
Design Pool Elevation: 268 ft (NGVD)
Deal Load (weight of RSW main structure)
Tainter Gate Closed
Maximum Credible Earthquake (0.19 “g”)
Hydrodynamic pressure due to MCE

The effects of an MCE on the stability of the RSW Main Structure resting on the Main Structure Attachment were considered. The tendency for the main structure to become dislodged from its support was assessed. For this load case, all tanks were assumed to be fully flooded.

Rough-order estimates show that if the tainter gate is closed in an MCE the dynamic loads, which include accelerations and hydrodynamic pressures, can be expected to dislodge the RSW Main Structure, causing it to lift and tip off of the Main Structure Attachment. The large magnitude of the earthquake loads make it impractical to restrain the structure. The CG does not go outside the mainstructure supports so, it is expected that at the conclusion of the earthquake, the structure will fall back to rest on the Main Structure Attachment. During an MCE, it is expected that the vertical range of motion will be less than one inch and the rotation, or tipping, range of motion will be about two degrees. This two degree tipping motion translates to a displacement at the pier nose tops of about 2.7 feet.

During an MCE the Main Structure Attachment must support large vertical and horizontal reactions. Estimates show that upstream horizontal forces on the Main Structure Attachment will be as high as 2,660 kips, and downward vertical forces will be as high as 2,560 kips.

4.5.1.3 RSW Naval Architecture

It is anticipated that the main structure will be constructed as one floating element that is towed to the John Day Dam through the navigation locks at the lower three dams on the Columbia River. It is envisioned that it will be installed and removed without the use of heavy lift equipment. This means that the main structure, in its transport condition, must have a draft no greater than 11 feet to pass through the locks. Its orientation with respect to the waterplane can be controlled. This is accomplished by altering ballast in certain internal compartments of the main structure. Figure 4-2 illustrates these compartments. Plate 18 illustrates the anticipated orientation of the RSW Main Structure for towing.

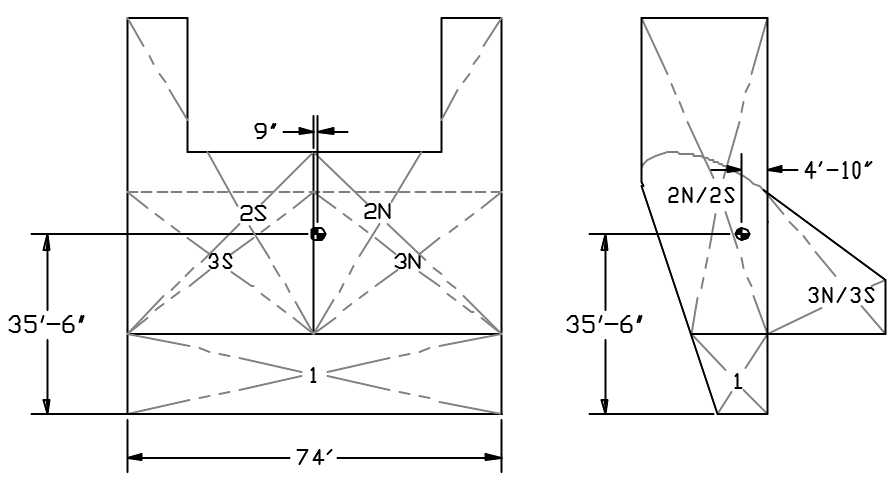


Figure 4-2
Tank Arrangement

The dry weight of the RSW main structure, without waterballast, is calculated to be 1,187,200 lbs, and the vertical center of gravity in the installed orientation is 35.55 feet above the bottom of the main structure at elevation 194 feet msl. The longitudinal center of gravity is 0.734 feet from center of the spillway towards Pier 19 and the transverse center of gravity 4.81 feet upstream of the dam face bulkhead. Based on its geometry, weight and center of gravity, the main structure's draft and trim were calculated. Calculations were performed using the General Hydrostatics Software suite created by Creative Systems of Port Townsend, Washington. The calculated maximum draft during transport, when tanks are dry, is 8'-0". In this condition, the main structure has adequate stability for towing on the protected waters of the Columbia River when in the transport condition (metacentric height of 60.9 feet). Metacentric height is a measure of a floating body's stability that reflects its resistance to overturning. It is calculated based on the body's waterplane inertia, volume of displacement, vertical center of buoyancy and vertical center of gravity.

If transit takes place with a trace amount of water in the tanks, the RSW can be expected to rotate more toward upright, and draw a deeper draft. If 1% of ballast capacity is left in each tank, the draft increases to about 8'-3" feet and the metacentric height decreases to 59.2 feet.

After transport, compartments will be flooded to reorient the main structure for installation. The minimum draft of the reoriented RSW is about 63 feet. This minimum was selected based on maintaining a minimum practical level of stability (metacentric height of 0.9') during the installation process. The draft allows for installation of the structure at the minimum pool level of 257'. Additional discussion of RSW main structure operations can be found in Section 8 of this report.

Once mounted on the dam the Main Structure is sealed at existing pier noses and along the Main Structure Attachments. See Plate 10 for details of the sealing arrangement. Prior to preparation of the plans and specifications the mounting and sealing surfaces on the dam should be surveyed and inspected.

4.5.1.4 Fatigue

Fatigue should not be an issue based on the magnitude and frequency of the loads measured in model testing. However, given the inherent uncertainties of the model test results, vibration sensors will be mounted at several locations in the main structure to periodically record and assess the level of structural vibration.

4.5.2 RSW Tailpiece

Load Conditions:
Four separate load conditions are considered.

Load Case 1: Closed condition (No flow over spillway)
The Tailpiece is fixed in place and the tainter gate is closed. The stoplogs are opened so that the RSW can be guided into position.

Max. Normal Operating Pool (ft)	268
Top of Spillway (ft)	210
The height of water pressure to be resisted (ft)	58

Weight of tainter gate resting on the tailpiece is 250 kips over the 50 foot width.

Load Case 2: Normal Operating Condition
The tailpiece and RSW are fixed in position and the tainter gate is open.

Max. Normal Operating Pool (ft)	268
Top of Spillway (ft)	210

Weight of water acting on the Tailpiece.
The hydrostatic forces experienced by the tailpiece vary linearly with depth.

Load Case 3: Opening & closing of tainter gate
Dynamic hydraulic loads acting on the Tailpiece.

Load Case 4: Closed condition (No flow over spillway)
Hydrostatic pressure of 58 feet
Weight of tainter gate resting on the tailpiece is 250 kips over the 50 foot width.
Maximum credible earthquake with one-third stress increase.

The most critical loading case is Case 1, when maximum water head acts on the Tailpiece.

The Tailpiece will be fabricated using stiffened steel plates and structural steel members. Most of the members will be welded. The plate on the top of the framing will match the desired hydraulic profile of the spillway. The plate on the front face (upstream), along with the corresponding structural steel frames will resist the hydraulic loads. These loads will be transmitted to the spillway concrete structure via bolted connections between the frames and the spillway. See Plates 11 and 12.

The top plate of the tailpiece has a variation in the hydraulic head, which was determined from the model testing. These variations do not result in significant stress changes.

The interface between the upstream edges of the Tailpiece and existing concrete surfaces will require a watertight seal. A rubber seal will run from the tainter gate contact point down along the vertical face of the Tailpiece and along the horizontal interface between Tailpiece and spillway surface. A standard J seal, which will use the hydrostatic head to help close the gap, will be used. The J seal will be attached to a steel fabrication that will allow field adjustment for variations between the tailpiece and concrete surfaces. See Plate 12.

We anticipate that elements of the Tailpiece will be shop fabricated and assembled in the field. These elements will consist of frames and cover plates.

A typical frame of the Tailpiece will be fixed to the spillway on base plates, which are bolted to the existing concrete. Anchor bolts are drilled in and set with epoxy.

The cover plate will stabilize the frames laterally to the concrete surfaces on each side. Cover plates will be welded to the frames. The downstream tip of the cover plate will be welded to an embedded plate to resist any tendency to vibrate. The embedded plate will be grouted flush with the spillway surface and attached to the spillway concrete with embedded anchors. See Plate 12.

When the Tailpiece is removed all anchor bolts will be cut off leaving only the one embedded plate that is flush with the spillway surface. This will keep the flow of water, over the spillway, undisturbed.

4.5.2.1 Changes during Development of the Plans and Specifications.

The shape of the upstream edge of the Tailpiece was changed from a rounded profile to a sharp point. The change was made because hydraulic modeling showed that undesirable hydraulic conditions would exist with both the sharp and rounded crests. The sharp edge dramatically simplified the closure plate between the Tailpiece and the Main Structure. The closure plate would be part of the Main Structure instead of a separate piece.

4.5.3 RSW Main Structure Attachment

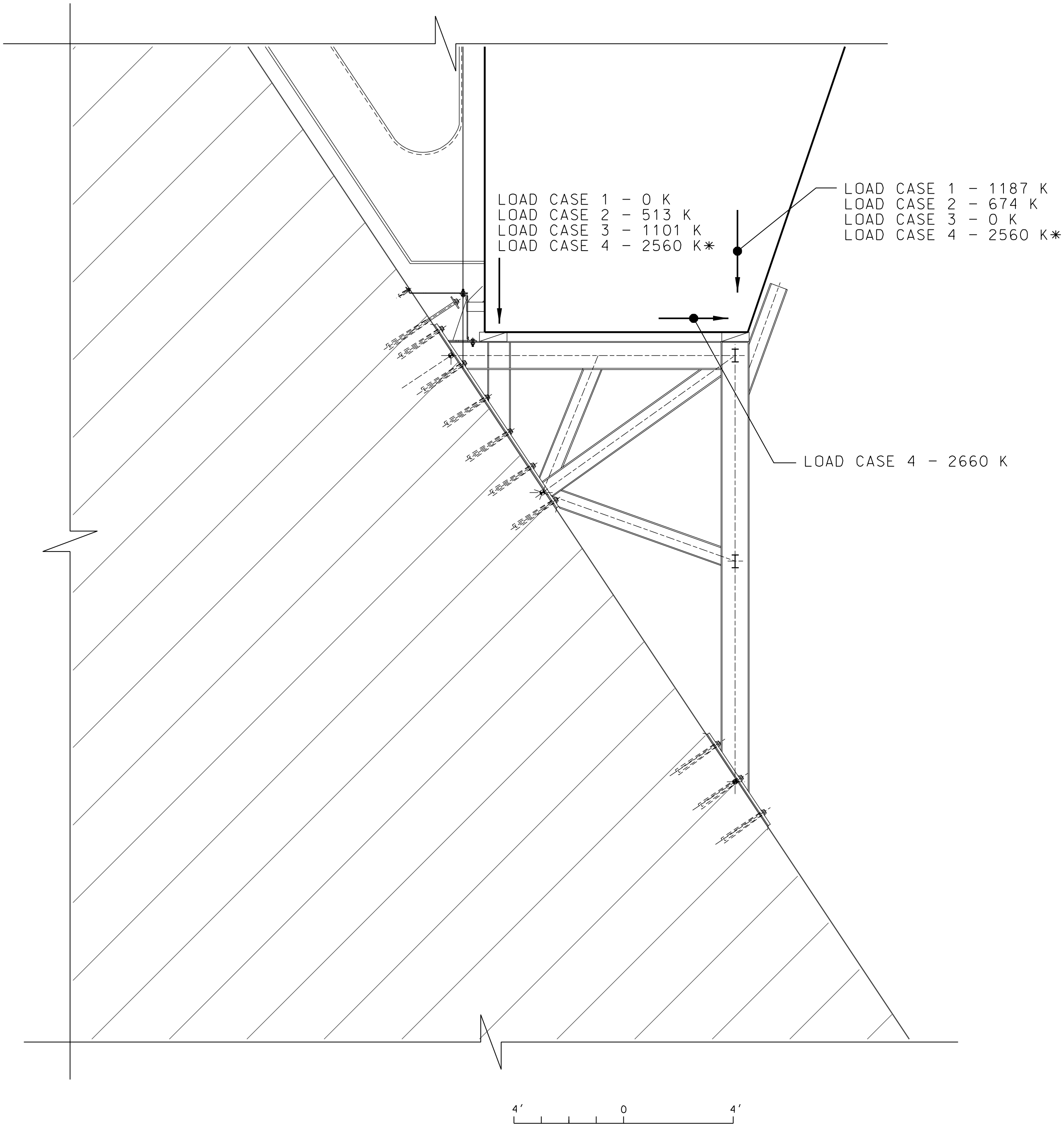
Load Conditions:
Four separate load conditions are considered. See Figure 4-3

Load Case 1: Closed Condition (No flow over spillway), during installation of RSW Main Structure.
Submerged weight of Main Structure entirely on the upstream edge of 13 frames.

During the installation process, the RSW is rotated from the horizontal transport orientation to a vertical orientation. As the RSW is lowered and rotated into position, the first point of contact of the RSW on the support frames will be on the upstream edge. Therefore, the entire submersed weight of 1,187 kips is resisted by the upstream column members of the frames. The load is distributed equally to all 13 frames.

Load Case 2: No operation. Dead Load only distributed to 13 frames.

At the completion of the installation, the entire submerged weight of the RSW (1,187 kips) is distributed to all 13 frames at the upstream and downstream points. These are 26 points of vertical deadload support. There is no mechanical connection between the RSW and the support frames.



NOTES:

LOADS ARE DISTRIBUTED TO 13 FRAMES EQUALLY.

* THIS LOAD IS THE MAXIMUM AND WILL SHIFT FROM ONE SIDE TO THE OTHER AS THE MAIN STRUCTURE ROCKS FROM SIDE TO SIDE DURING THE MCE.

Figure 4–3
Main Structure Attachment Loads

Load Case 3: Normal Operation. Dead Loads distributed to 13 frames.

During normal operation, the tainter gate is open and water flows over the RSW. The introduction of the hydraulic loads changes the distribution of the deadloads to the support frames. The hydrostatic loads are resisted by beam action of the Main Structure spanning between the dam faces on each side of the spillway. There are no loads on the attachment frames from the hydrostatic loads.

Load Case 4: No Operation with Earthquake.

The normal operations will continue for about 6 months (April through September) with about 6 months of no flow (October through March). The no flow condition is most critical with an earthquake load. Therefore, load case 4 adds the MCE earthquake to the no flow load case. The vertical reactions are increased and redistributed. The vertical reactions resisted by the 13 frames for this load case is 2,560 kips. The point of application changes from the upstream to the downstream point on the frames as the earthquake cycles. In addition, there is a horizontal shear force of 2,660 kips applied to the frames. There is no mechanical connection between the support frames and the RSW. The shear force is resisted by bearing on the beam which cradles the base of the RSW.

The floating RSW Main Structure will be towed into position in Spillway Bay 20 by a tugboat. It will finally rest on a seating arrangement called “Main Structure Attachment” (see Plate 13). The Attachment will consist of 13 structural steel frames, at 6’-3” on centers across the spillway. This spacing will match the spacing of the typical web frames of the floating RSW Main Structure. The frames will be connected to each other by means of structural steel sections to formulate a complete three dimensional structure approximately 70’-0” long, between Piers 19 and the non overflow dam. This Main Structure Attachment will be fabricated off site in a shop and barged to the project. Divers will attach the frames to the upstream concrete surface of the existing spillway using drilled in anchor bolts set in epoxy that can be used underwater. There is no mechanical connection between the RSW main structure and the supporting attachment structure. The main structure will be wedged into the mating attachment frame. This wedging action will create a force to resist movement at the bottom of the RSW main structure. There is, however, a connection between the existing concrete dam structure and the RSW at approximate elevation 272. This attachment will consist of a chain and turnbuckle.

The main structure will be set on shims attached to the attachment frames. These shims will allow for vertical grade adjustment to take up misalignments.

There will be a sealing surface across the full width of the support frame and vertically along the face of the dam. The surface will provide a smooth steel surface on which the seals from the Main Structure will seat. The details for this sealing surface allow for installation adjustments to accommodate as-built conditions. However, the contractor should survey the existing conditions to ascertain the magnitude of as-built variation. Grout will be pumped into the space between the existing dam and the steel sealing surface structure to complete a watertight connection. See Plate 13.

4.5.3.1 Changes during Development of the Plans and Specifications.

Vertical steel sealing surfaces on the existing pier noses were added to insure a reliable smooth surface on which the Main Structure seals could seat around the entire interface.

SECTION 5 MECHANICAL DESIGN

5.1 RSW Main Structure Floatation & Ballasting System

Installation and removal of the RSW main structure requires that water be added and removed from some of its internal compartments. Addition of water is accomplished by flood openings and vents. Removal of water is by application of compressed air (vents closed) to force water out of a compartment through a flood opening. Compartments 1, 2N, 2S, 3N and 3S (N indicates the north side and S indicates the south side) are equipped with vents, which double as compressed air lines, and with flood openings controlled with remotely actuated butterfly valves. Plate 14 describes the systems and components in more detail.

The design uses electric motors and reach rods to operate the flood valves. Electric solenoids are used to operate the smaller vent and deballast air valves. This arrangement will allow the valves to be operated from the river tug so that personnel need not be on the RSW when it is rotated between transit-to-installation and installation-to-transit attitudes. The river tug supplies the electric power and control signals to the solenoids.

There are five vent valves, one for each of the compartments. These valves are electrically actuated and are located on top of the RSW Main Structure piers. The design includes five relief valves, with pressure limits set to protect the structure.

There are five deballast air valves, one for each of the compartments. These are remotely operated valves located on the deballast air manifold on top of the RSW Main Structure piers. The relief valves are set at 13 psig for tank 1 and 21 psig for tanks 2N, 2S, 3N, and 3S. In addition pressure reducers are incorporated in the deballast air manifold design.

There are six flood valves, two for compartment 1 and one for each of the remaining compartments. These are butterfly valves with reach rods and electric motors. The valves are located in the lowest levels of each compartment in the RSW main structure. The motors are located on the top of the pier noses. Reach rod vendors report a 75 percent mechanical efficiency for each reach rod stuffing box or bevel gear. The operators will have to be sized to account for this.

An uninterruptable power supply (UPS) is provided to close the valves in the event of a power loss. The UPS does not provide enough power to close the flood valves simultaneously; therefore, the closing sequence must be controlled. First, the vent valves are closed simultaneously. Then each flood valve is closed sequentially.

Compressed air for blowing the tanks will be provided from a portable air compressor mounted on the river tug. A portable diesel driven compressor capable of generating 350 cfm free air discharge at 100 psi has been selected for the design. This capacity will allow the RSW main structure to be rotated from the installed orientation to the transport orientation in 6 hours.

5.2 Corrosion Control

The design life of the Proof of Concept RSW is expected to be relatively short (3 years), given that it is a prototype for evaluating the Skeleton Bay SBS. With this in mind, relatively simple and inexpensive corrosion control measures are appropriate. Passive cathodic protection, in the form of zinc anodes, will act as the primary means of corrosion protection for the internal compartments. The interior spaces will also be coated with a zinc rich primer.

It is not practical to use zincs to protect exterior surfaces given the hydraulic demands placed on the RSW. A coating system is the most reasonable solution for protecting these surfaces. An epoxy-based system is recommended given the anticipated service.

It may be advisable to use a more abrasion resistant system for the spillway surfaces and full epoxy coating of the interior compartments, if a longer service life is desired. However, it is expected that the spillway surface, where high flow velocities, occur will abrade to expose bare steel.

5.3 Seal Design

Several different perimeter sealing schemes were evaluated during the design process. The different arrangements considered the use of: J-Type seals, neoprene block seals, and inflatable seals. This section describes the final J-seal arrangement used. See the following section for a discussion of the other options evaluated.

The J-seals were chosen as the best solution to facilitate the RSW installation / removal process and to satisfy the given geometry of the RSW / dam interface.

The J-Type seals rely on an initial deflection against the sealing surface. The hydrostatic head then presses the seal against the opposing surface forming a seal. The J-Type seals need to be arranged such that the entire sealing surface is in the same geometric plane. The J-Type seals are relatively rugged, have a tolerance for misalignment, and do not require a large force to preset the seal into position. However, the intersection of the RSW, the dam's spillway, and pier nose are not in the same geometric plane. A horizontal sealing surface would be installed on the same plane as the sealing surfaces on the pier nose. Bearing blocks are used to transfer the majority of the hydrostatic force to the dam. See Plate 10.

5.4 Changes during Development of Plans and Specifications

The original concept considered for control of the flood valves used pneumatic actuators. The pneumatic actuator concept was changed to hydraulic actuators due to concerns over the depth of submergence of the actuators. Hydraulic power for the actuators would be provided by an accumulator tank. The design pressure of 3000 psi (reserved) would be achieved using compressed nitrogen, which would be stored in bottles located on the top of the RSW Main Structure pier noses.

Electric solenoids were used in the design to control the hydraulic actuators for the flood valves. This arrangement allowed the valves to be operated from the river tug so that personnel need not be on the RSW when it is rotated between transit-to-installation and installation-to-transit attitudes. The river tug would supply the electric power and control signals to the solenoids.

During development of plans and specifications, the hydraulic actuator concept was discarded due to concerns about leaking hydraulic fluid. The hydraulic system had the advantage that all valves would close in the event of a loss of external power. The electric operators were then selected. To safeguard the Main Structure an uninterruptible power supply (UPS) was included in the design. The UPS would provide power to drive all valves closed in case external power is lost.

Initially, seals inflated by water at a higher head than the forebay were considered to provide the seal between the dam and RSW Main Structure. The inflatable seals remain deflated during the installation of the RSW. When the seals are deflated, they would be protected by the bearing blocks. Once the RSW is in its final resting position, the seals would be inflated using water from a head tank. The inflatable seals have a good tolerance, and require no preset force during installation. The full hydrostatic force acting on the RSW is too large to be supported by the inflatable seals by themselves. Bearing blocks would be used to transfer the majority of the hydrostatic force to the dam. The layout of this option provides a good bearing area as well as a good sealing surface. The inflatable seal option was discarded due to concerns about the reliability of the inflatable seals and a lack of operational experience with seals of this type.

SECTION 6 ELECTRICAL DESIGN AND CONTROLS

6.1 General

Installation and removal of the RSW main structure requires that a Local Control Panel (LCP) would be temporarily installed on the tugboat, and three electrical cables would be attached between the tugboat LCP and the LCP permanently installed on the RSW. The cables would have quick disconnects for easy connection and disconnection, especially in an emergency. The cables would have flotation devices attached along their length to reduce cable strain and easy handling. The LCP on the tugboat would stand on legs connected to a solid bottom plate. This allows the LCP to be quickly secured by simply loading weights such as sandbags onto the bottom plate if desired. Plates 15 and 16 show the tugboat LCP and control layout in more detail.

The electrical equipment permanently installed on the RSW would consist of the following:

1. A motor operated valve actuator for each valve, sixteen total. The five vent valves and five deballast air valves are smaller and require low power actuators. The six flood valves are larger and require higher power actuators.

2. Each valve actuator has the following features: a Local/Off/Remote (LOR) switch, an Open pushbutton, a Close pushbutton, mechanically and electrically interlocked reversing contactors, a thermal overload protection device, torque switches, and limit switches.

3. A Local Control Panel (LCP) with a removable Uninterruptible Power Supply (UPS). The UPS would supply power to automatically close all the valves if a connecting cable breaks. The vent and deballast air valves are wired with control relays to all close at once. The higher power flood valves are wired with individual time delay relays to close one at a time. This closing sequence will take about three minutes.

4. A vibration monitoring system consisting of:

    a. A removable, portable analyzer and display. It would be battery powered and only brought to the RSW periodically when readings are desired. Because it is not permanently installed no vibration history or automatic alarming capabilities are provided.

    b. Four permanently installed vibration transducers. A transducer would be positioned at each of the locations of most concern. These are the downstream end of the Main Structure spillway skin and downstream end of the pier skin.

The electrical equipment temporarily installed on the tugboat consists of the following:

1. LCP, number CP-1, the LCP receives power from the tugboat and has the following components;

    a. Two circuit breakers; one for the flood valves and UPS power; one for the vent valves, deballast air valves, and controls,

    b. Each of the 16 valves have the following controls: an OPEN/OFF/CLOSE switch, a REMOTE position blue indicating light, an OPEN position red indicating light, a CLOSED position green indicating light. See Plate 15.

Three cables connect the RSW LCP and CP-1. Each cable has floatation devices attached along its length to reduce cable strain. Each cable has a quick disconnect coupling with strain relief fittings at each end. The cables have the following requirements:

1. Cable #1 requires 3 #8 American Wire Gauge (AWG) conductors for 120VAC power the flood valves and UPS,

2. Cable #2 requires 51 #14 AWG conductors for 120VAC valve control,

3. Cable #3 requires 34 #14 AWG conductors for 120VAC valve position indication.

SECTION 7 CONSTRUCTION

7.1 Construction Considerations

It is anticipated that the RSW Main Structure will be constructed as one, buoyant structure. This will allow it to be towed to the installation site. Several fabricators on the lower Columbia River can construct and launch this size of a structure into the river.

Alternatively, a fabricator could construct the Main Structure in smaller components and assemble them on a barge. At the dam site, the barge could be submerged to float the RSW off.

Given that this structure must have watertight integrity, welding should conform to ANSI/AWS Guide for Steel Hull Welding. See Section 4.2.1.

Several areas of the spillway would be surveyed to insure that the RSW components would properly fit onto the spillway crest. This survey would be conducted by the Contractor immediately after the Notice to Proceed is issued. The survey data would be available to guide production of the shop drawings or to engineers in case design changes are necessary. Much of the survey would be conducted underwater and would probably require that some type of guide or frame be installed on the spillway or its piers. Some items that would require a survey are listed below:

- Distance between the piers
- The evenness of the pier noses from the top to below the bottom of the Main Structure
- The shape of the spillway crest to insure that the Main Structure has enough clearance
- The spillway crest to obtain data for the sizing of the Tailpiece

The Main Structure Attachment would be constructed underwater. First, the anchors would have to be drilled into the dam. Then the base plates for the frames would be bolted to the dam. The frames could be prefabricated and then bolted to the anchors. The contractor would decide the amount of prefabrication. Once the Main Structure Attachment is installed, it would need to be surveyed to fix the proper height of shims at each frame. The shims should be tack welded to the frames.

The Tailpiece would be built in the dry behind the spillway crest stop logs. In order to install the Tailpiece on the spillway all tools and materials would have to be lowered from above, either through the stop log deck opening or between the spillway bridge and tainter gate in the full-up position. The opening between the spillway bridge and the top of the stop logs would be about four feet. The opening between the fully opened tainter gate and west side of the spillway bridge would be about five feet. Whichever path is chosen wood sheathing or some other type of padding would have to be installed to protect the stop logs or tainter gate.

Another alternative is to lift the materials from a barge anchored in the stilling basin of Spillway Bay 20. Lift will be about 50 feet from the tailwater at elevation 160 to the spillway crest at elevation 210. The horizontal reach required would be about 85 feet from the barge to the location of the Tailpiece.

7.2 Construction Schedule

Most of the construction effort will be the fabrication of RSW Main Structure, Tailpiece, and Main Structure Attachment. The Main Structure will float and be towed to the site by a tug boat. A tug will also be used to install the RSW Main Structure on Spillway Bay 20. The Tailpiece and Main Structure Attachment will probably be transported to the site on a barge. Once the Main Structure Attachment and Tailpiece is in place, the Main Structure will be installed by methods described in Section 8.1. A schedule for these activities is shown on Figure 7-1.

The schedule is ambitious. Bids for the RSW cannot be sought until biological testing of the extended length deflector is complete and indicates that the deflector is safe for fish passage. The installation and evaluation is not yet scheduled. Assuming that the testing is completed in March 2003, the earliest date to advertise for bids would be in early April 2003. The estimated time for fabricating the Main Structure is 170 days. Depending on the availability of yards to perform this work, it could take longer to fabricate the Main Structure. The bid solicitation for the Lower Granite RSW called for a four-month fabrication schedule and several bids were received. Based on the bid results it appears that the yards can meet short fabrication schedules. However, the Lower Granite RSW fabrication took longer than anticipated mainly due to the painting requirements. It is anticipated that the simpler interior painting specification for the John Day RSW will not cause a schedule delay.

The Tailpiece and Main Structure Attachment would be brought to the site and installed simultaneously. This will take about 45 days. The Main Structure would be brought to the site after the Tailpiece and Main Structure Attachment have been installed. This allows some more time to fabricate the Main Structure. See the Operations and Maintenance Manual in Appendix H for a more complete description of the installation sequence.

The critical path for the project lies along fabrication and installation of the RSW Main Structure. Items, which could delay the project, involve longer than anticipated biological evaluation of the deflector or delays in deciding whether to proceed with the RSW fabrication and installation.

ID	Task Name	Duration	Start	Finish
1	**SPILLWAY DEFLECTOR**			
2	Notice to Proceed	1 day	Mon 8/5/02	Mon 8/5/02
3	Deflector Construction	207 days	Tue 8/6/02	Fri 2/28/03
4	Deflector Testing	31 days	Sat 3/1/03	Mon 3/31/03
5	**BAY 20 RSW**			
6	NTP RSW	7 days	Tue 4/1/03	Mon 4/7/03
7	BCOE Update	59 days	Sat 2/1/03	Mon 3/31/03
8	Bid and Award RSW	55 days	Tue 4/1/03	Sun 5/25/03
9	Notice to Proceed	7 days	Mon 5/26/03	Sun 6/1/03
10	Spillway Survey	28 days	Mon 6/2/03	Sun 6/29/03
11	**Main Structure**			
12	Shop Drawings	50 days	Mon 6/30/03	Mon 8/18/03
13	Fabrication	170 days	Tue 8/19/03	Wed 2/4/04
14	Transportation	7 days	Thu 2/5/04	Wed 2/11/04
15	**Tailpiece Structure**			
16	Shop Drawings	40 days	Mon 6/30/03	Fri 8/8/03
17	Fabrication	115 days	Sat 8/9/03	Mon 12/1/03
18	Transportation	7 days	Tue 12/2/03	Mon 12/8/03
19	**Main Structure Attachment**			
20	Shop Drawings	40 days	Mon 6/30/03	Fri 8/8/03
21	Fabrication	115 days	Sat 8/9/03	Mon 12/1/03
22	Transportation	7 days	Tue 12/2/03	Mon 12/8/03
23	**INSTALLATION**			
24	Main Structure Attachment	60 days	Tue 12/9/03	Fri 2/6/04
25	Tailpiece	60 days	Tue 12/9/03	Fri 2/6/04
26	Install, Fit & Measure Main Structure	17 days	Thu 2/12/04	Sat 2/28/04
27	In-Water Work Period	90 days	Mon 12/1/03	Sat 2/28/04
28	**Biological Testing**	**196 days**	**Thu 4/1/04**	**Wed 10/13/04**
29	Balloon Tag/Survival Testing	5 days	Sat 10/9/04	Wed 10/13/04
30	Efficiency Testing, FPE	105 days	Thu 4/1/04	Wed 7/14/04

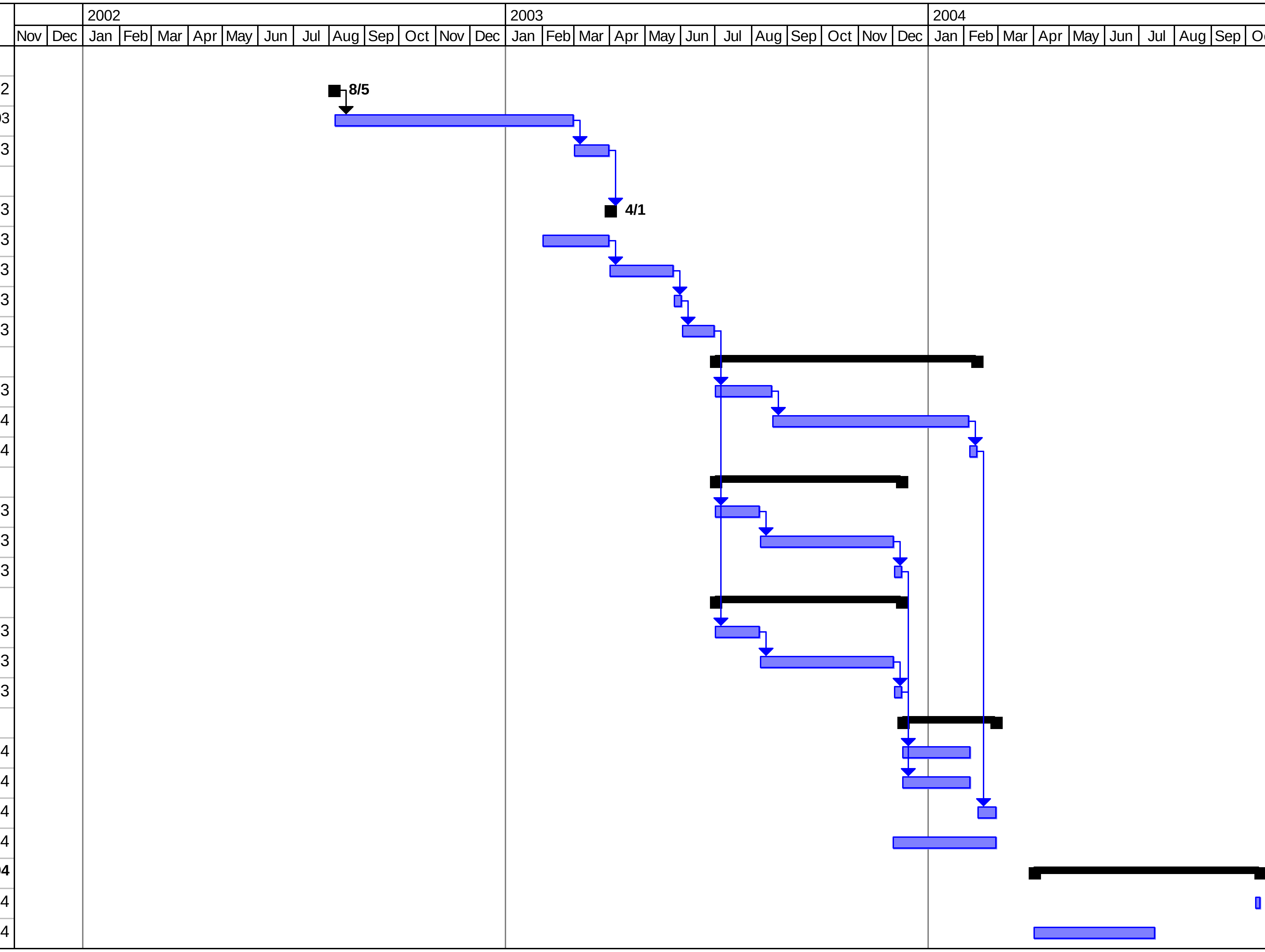


FIGURE 7-1
John Day RSW Schedule

SECTION 8 OPERATION AND MAINTENANCE

8.1 Installation and Removal

The installation and removal procedures in this section are those anticipated at the time of finalizing the DDR. The contractor or those completing the plans and specifications could change the procedures given below.

For hydraulic reasons it is critical that the RSW spillway surface at the line where the Tailpiece and Main Structure join is as smooth as possible. It will be difficult to achieve a close fit. Therefore, it is recommended that an underwater survey of the spillway be conducted prior to fabrication. To insure a good fit the Main Structure Attachment would be installed first, and the Main Structure temporarily installed on it. Divers can then measure the location of the lower end of the Main Structure spillway. The Main Structure would then be removed, and stop logs installed to dewater the upper spillway. The tainter gate would then be raised to its full height position and dogged off. Using the survey information the Tailpiece would then be installed to the proper height to insure a good fit with the Main Structure. After the tailpiece is installed the Main Structure would mounted on the Main Structure Attachment to complete the installation.

8.1.1 RSW Main Structure

Installation. Installation of the Main Structure will require a river tug that can supply compressed air and can position the Main Structure in Spillway Bay 20. See Plates 17 and 18. Basic air manifolds for blowing the compartments and for actuating the flood valves would be located on the RSW Main Structure.

After towing to the pool above the John Day Dam, flood and vent valves must be opened to rotate the RSW into the installation position. Its draft can then be altered by adding more water to compartments 2N, 2S, 3N and 3S or by applying compressed air to these compartments to expel water.

The four steps to rotate the main structure to its installation position are illustrated on the top portion of Plate 18. See the Operations and Maintenance Manual in Appendix H for more installation details. Lines will be painted on the Main Structure to indicate the correct position of the Main Structure at the end of each ballasting step described below:

Step 1 Open flood and vent valves to Tank 1 by means of electric power and signals from the Tug Local Control Panel. The structure will settle as shown in the Step 1 drawing on Plate 18.

Step 2 Open the flood valves to tanks 2N and 2S. Once the structure reaches the position in the Step 2 drawing on Plate 18 (Tanks 2N and 2S will be 73% filled), vents and valves are closed to prevent additional flooding.

Step 3 Open the flood and vent valves to Tanks 3N and 3S and close again when the structure has floated upright and settled to the correct depth. The Step 3 drawing on Plate 18 shows the Main Structure during filling.

Step 4 The left drawing in Step 4 on Plate 18 shows the structure ballasted for installation at the end of Step 3. Most of the time the forebay pool will be higher than elevation 257. If this is the case, Tanks 2N, 2S, 3N, and 3S will continue to be flooded to allow the structure to settle to the proper elevation. The tug would then push the Main Structure into position. See Plate 17. Tanks 2N, 2S, 3N, and 3S would continue to be flooded to lower the Main Structure until it rests on the Main Structure Attachment. The fully flooded position is shown on the right drawing in Step 4 on Plate 18.

Divers would then measure the fit between the Main Structure and Tailpiece at the RSW spillway surface and the gap between the shims on the Main Structure Attachment and the Main Structure. If these measurements do not meet tolerances the Main Structure would be raised and the shims adjusted. This procedure would be repeated until installation tolerances are met. Once seated in the spillway, holdbacks are added at each piernose and the flood vent valves are left open. The electric cables and air hose would be disconnected and the tug moved away from the spillway. Then, the tainter gate would be opened to begin operation.

Removal. Removal of the RSW is accomplished by closing the tainter gate, positioning a tug, removing the holdbacks, tying the Main Structure to the tug, applying compressed air to compartments 2N, 2S, 3N and 3S until the RSW floats free and then pulling the RSW away from the dam using the tug. The Main Structure would then be deballasted in reverse order of the ballasting procedure. These operations are graphically depicted in Plate 17 and the bottom half of Plate 18. A more detailed description is contained in Appendix H.

The current concept for providing the necessary support for RSW Main Structure removal/installation is to use a tug outfitted with a compressor and the Tug Local Control Panel to actuate the water and air valves. The structure and tug would be positioned with the tug's engines and tethers to the dam to help with lateral positioning.

Storage of the RSW Main Structure could be provided at the north side of the navigation lock floating guidewall. This location is shown on Plate 1. The RSW would be de-ballasted into its towing position (draft of about 7.5') and towed to a mooring buoy located north of the navigation channel. The mooring point would be installed by the Contractor. The design of the mooring would be reviewed by the engineer prior to installation.

8.1.2 RSW Tailpiece

To remove or install the Tailpiece, the stop logs need to be in place and the tainter gate fully open. We anticipate that the tailpiece will be fabricated in major segments offsite and erected on the spillway. The structure consists of a series of girders that can be set in place and attached to bearing plates. First, the embedded end plate and bearing plates would be installed on the spillway. The end plate would be installed, leveled, and grouted in place flush with the spillway surface. From this foundation the two outer girders would be brought into place. The girders would be about 34 feet long by 11.5 feet high. See Plate 11. The access route and rigging would be left up to the contractor based on project operational restrictions. The two outer frames would be set in place and the tainter gate lowered until it rests on the frames. The height of one of the frames would be adjusted until the gate rests evenly on the two frames. The rest of the frames would be installed so their tops are in line with the tops of the two outer frames. The skin plates would then be welded onto the girders and the seals installed. In this manner a good seal between the tainter gate and the Tailpiece can be achieved.

The Tailpiece can be removed by disassembling the pieces. It is anticipated that the Tailpiece will only be removed once at the end of the test period and will not be used again. However, if extreme high flows are anticipated, which require the use of Spillway Bay 20 to pass flood flows, the Tailpiece would be removed by cutting the steel into pieces and removing. It is anticipated that about three weeks warning of the impending flood would allow time to hire a contractor and complete the emergency removal.

8.1.3 Main Structure Attachment

Divers would install the main structure attachment. Once scenario is to use a template. A template frame would be fabricated, lowered into place, and fixed to the piers. This frame would be used to drill holes for the anchors. Once the anchors are in place, the frames for the Main Structure Attachment would be lowered and bolted to the dam. Then the cross bracing and seal bearing plate assembly would be installed and grout injected. The template would be removed after installation of the anchors.

8.2 Maintenance Requirements

Significant maintenance efforts are not anticipated over the design life of this structure. The flood and vent valve actuators would require periodic maintenance over the 3 year design life.

Bolted manholes on the back face of the RSW Main Structure would provide access to the various compartments when it is in its transport condition. This will allow for inspection and maintenance of flood valves. Access to the interior of the Main Structure is only possible when it is in the transport position.

8.3 Operational Requirements

Vibration sensors would be installed at critical locations on the RSW. Data from these sensors are obtained by connecting a portable computer to cables at the top of the north pier. Data can only be obtained when the portable computers are connected. Due to the large amount of data, vibration data would be collected periodically. It is envisioned that vibration data would be collected during startup and periodically thereafter.

SECTION 9 COST ESTIMATES

9.1 Project Description – Removable Spillway Weir

The John Day Dam Removable Spillway Weir is located at Spillway 20 at the John Day Dam on the Columbia River. The project site is located about 110 miles east of Portland, near Rufus, Oregon and is easily accessible by barge, rail and Interstate 84. The project is on the Washington State side of the Columbia River, in Klickitat County.

The primary feature of the project is a 625 ton steel spillway weir that is prefabricated, floated into place and can be removed when needed. Other features include the steel Tailpiece and a Main Structure Attachment.

9.2 Summary of Costs

This section summarizes the cost and describes some of the major design features. See Appendix G for the MCACES Cost Summary Tables and an Excel Total Cost Summary sheet.

The total estimated construction cost of the Removable Spillway Weir project at John Day Dam is $10.14 million. The base cost before contingency is $8.34 million. A 15% contingency adds $1.25 million and escalation to midpoint of construction is $0.54 million. This estimate does not include costs for engineering and design or construction management. With construction management added, the total cost is $10.75 million.

Including contingency and escalation the costs of the main features of the project are as follows:

(Costs in Thousands)

	Contract	Contingency 15%	Escalation 5.6%	Total
• Mobilization-Demobilization	$257	$39	$17	$313
• Tailpiece	$718	$108	$47	$873
• Main Structure Attachment	$527	$79	$34	$640
• Main Structure	$6,794	$1,019	$442	$8,255
• Optional - Dive Crew	$36	$5	$2	$43
• Optional – Welder-Machinist	$11	$2	$1	$14
TOTALS	**$8,343**	**$1,251**	**$542**	**$10,138**
Construction Management				**$ 608**
TOTAL w/CM				**$10,746**

Mobilization and demobilization will require mobilizing/demobilizing a large barge crane to help install the Tailpiece, and Main Structure Attachment.

The tailpiece is a 50.4 ton steel structure that is installed at the top of the existing spillway directly underneath the existing tainter gate. It meets the Main Structure to form single unit when both are in place. It has eleven prefabricated frame pieces and a ½ - inch thick steel skin that was assumed to come in 5 foot by 8 foot sections. The tailpiece will be attached to the concrete spillway by forty-four steel mounting base plates, each bolted to the spillway with 1-1/4" x 18" long anchor bolts. Existing stop logs will be installed to dewater the spillway during construction. The tailpiece will be lowered into place by a crane and will be shipped to the dam via tugboat and barge or truck.

The Main Structure Attachment is a 20.5 ton steel support structure that will be built in place underwater by divers in depths up to 90 feet. The structure is comprised of steel structural members that are bolted together and attached to the face of the dam with 206 anchor bolts (1-1/4" diameter by 18" long and ¾" by 10" long). This structure supports the Main Structure when it is filled with water and lowered into place against the face of the dam.

The Main Structure is a prefabricated buoyant steel structure that weighs 625 tons. It will be built according to welding standards for barges and will be capable of being towed to the John Day Dam by tugboat. It has chambers that can be filled with compressed air or water to create the proper draft to float it to the dam and submerge it in place at Spillway 20. The coatings for the Main Structure will include priming inside and out and final coats on the outside only. The cost of coatings is included in the price per pound of steel. The Main Structure is the primary component of the project.

A tugboat will be chartered to maneuver the Main Structure into place and remove it when required. A skid mounted compressor must be purchased or rented to assist the tugboat with the Main Structure placement and removal.

9.3 Basis of the Estimate

The basis of the estimate are the 90% John Day Dam Removable Spillway Weir Design Development Report and drawings submitted May 2001. (Contract No. DACW57-97-D-004, Task Order Case No. 21)

9.3.1 Construction Schedule

Construction is expected to take place between May 2002 and March 2003. Restrictions on in-water work apply between March and November of each year. Spilling in the adjacent spillway will be restricted during construction.

No overtime is anticipated during construction, but some double shifts may be necessary during in-water work periods.

The project will be accomplished using one construction acquisition plan.

9.3.2 Subcontracting Plan

Not applicable.

9.3.3 Project Construction

Project site access will be by paved road to the John Day Dam site and by barge on the Columbia River.

For the purpose of estimating costs, the fabricated steel structures are expected to be built in the Portland area.

Construction Methodology: Construction will require mechanical and structural work to be performed in a sequenced and coordinated fashion.

Unusual Conditions: Cold winter weather when in-water work is allowed, high winds and rough water are conditions that make working on and adjacent to the Columbia River and John Day Dam extremely difficult.

Unique construction activities include maneuvering a semi submerged 625 ton steel weir into place, attaching a 50 foot wide steel main support structure by divers in up to 90 feet of water, and constructing and attaching a 50.4 ton steel tailpiece structure to a sloped spillway.

Equipment/Labor Availability and Distance Traveled: construction equipment will be mobilized and demobilized by the general construction firm securing the contract. It is anticipated that the firm will be from the Oregon/Washington area.

Labor was assumed to be available without restriction considering the close proximity to the Portland area.

9.3.4 Environmental Concerns

Restrictions on when in-water work can occur during the months of December through March. Other concerns are the normal minimization of fuel and oil leakage from heavy equipment during construction and temporary storage of equipment at the project site.

9.3.5 Contingency, Sales Tax and Escalation

The estimate includes a 15% contingency and 5.6% escalation factor. The escalation factor was based on the midpoint of construction for the project of January, 2003. The indices for escalation were based on the latest Civil Works Construction Cost Index System indices for Fish and Wildlife Facilities. The project is expected to take ten months to construct and require one in-water work period from December to February of 2002-2003. A sales tax of 7.8% was included in the estimate.

9.3.6 Effective Dates for Labor, Equipment, Material Pricing

Davis Bacon Decision Number WA010001, dated 4/06/01 was used for labor rates. The 1999 equipment rates for the Northwest region were used for equipment costs except barge cranes and barges which were priced based on quotes from general contractors. Material pricing was based on information from the fabricators and contractors. Some costs were based on past bids and other past quotes. Some pricing in the 1999 MCACES unit price book was revised to reflect the estimator's experience and historical cost information.

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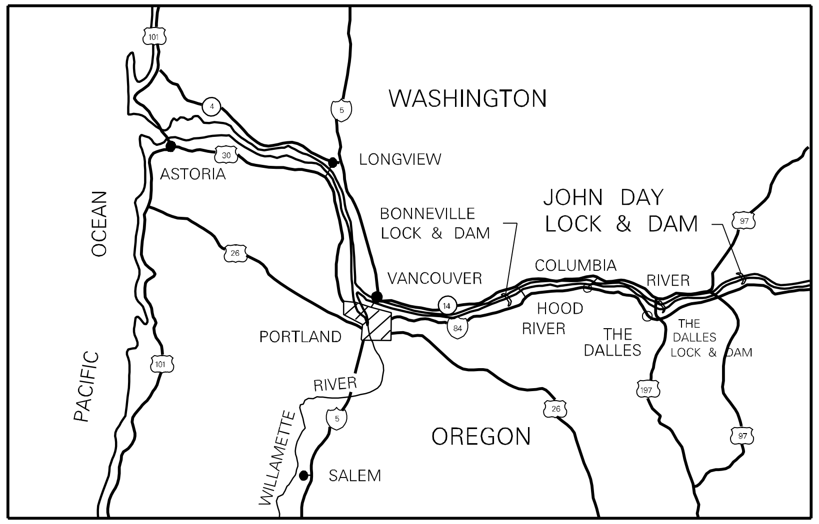
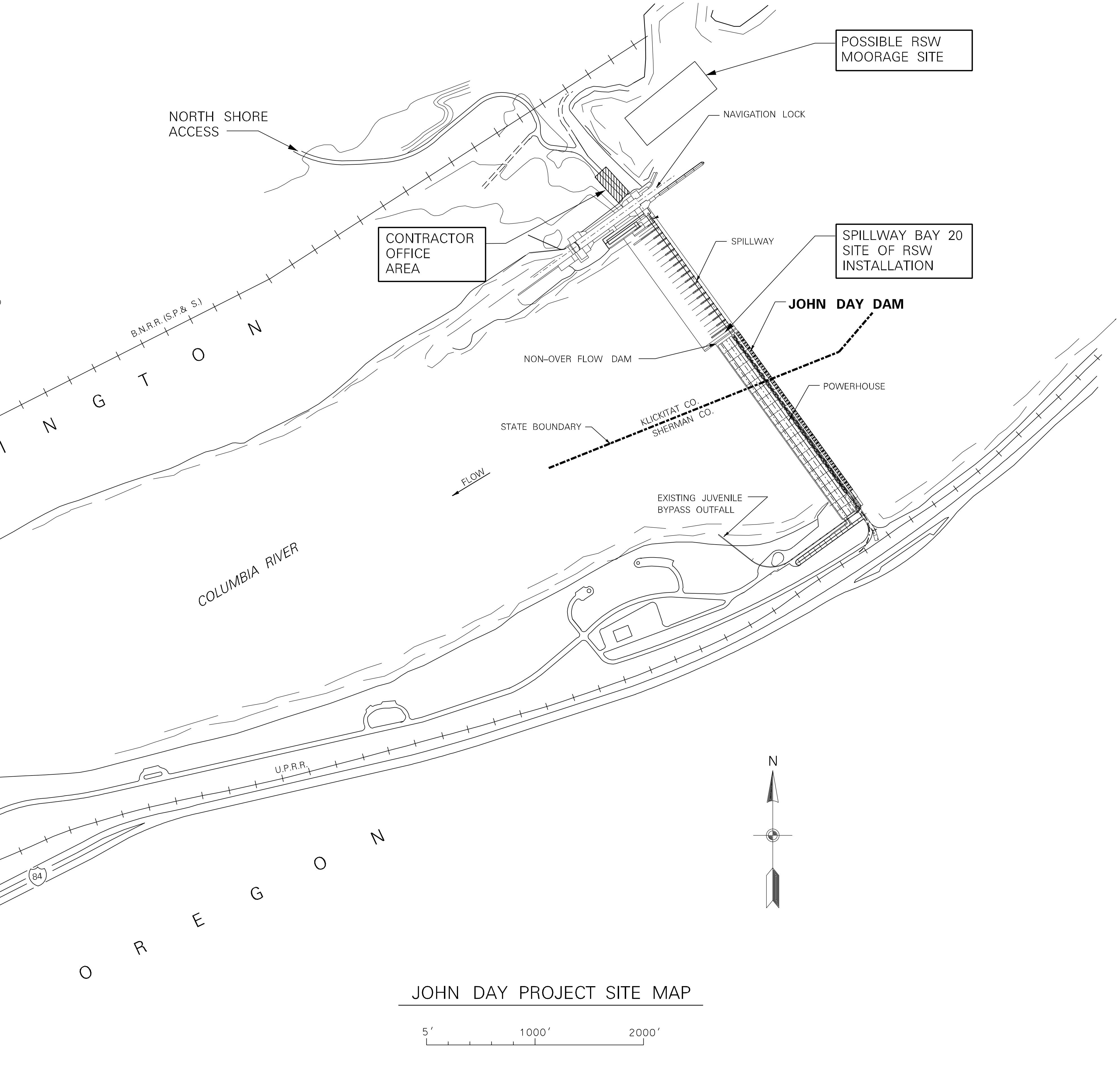
U.S. Army Corps of Engineers, *Lower Snake River: Surface Bypass and Collection System Combinations Conceptual Design Report*, December 1998b.

PLATES



VICINITY MAP



JOHN DAY PROJECT SITE MAP

US Army Corps of Engineers
Portland District

Revision	Date	Description	By

Designed by: BARTON
Drawn by: SCHULZ
Checked by: DORRATCAGUE
Submitted by: DONALD R. CHAMBERS, P.E. Chief, Structural & Arch. Design
Date: OCT 2001
CADD File Name: JDS001F01L01.DGN
Technical Manager: MATTHEW HANSON

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REMOVABLE SPILLWAY WEIR
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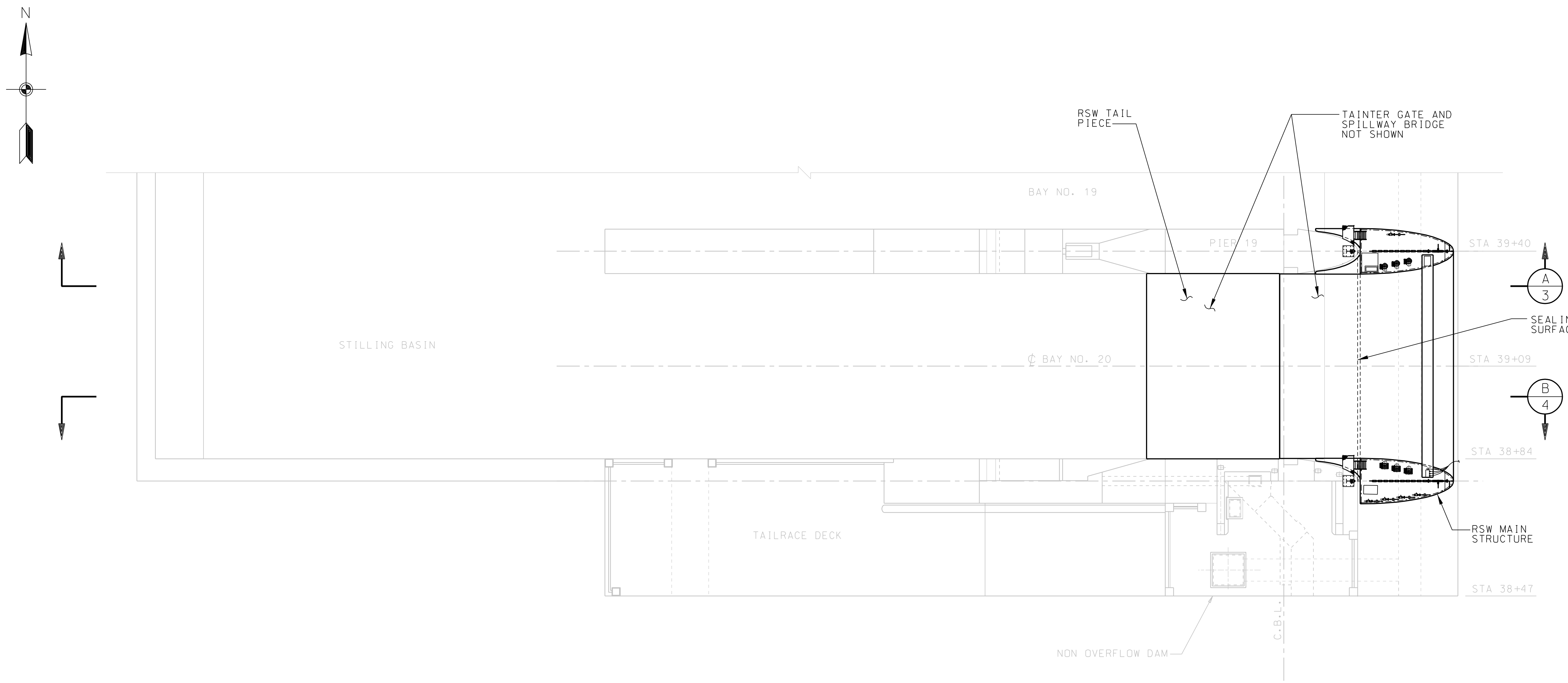
PROJECT SITE PLAN

DRAWING STATUS:

DRAWING NO.

PLATE
1



PLAN - SPILLWAY BAY 20

30' 0 30'

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SPILLWAY BAY 20 - PLAN

DRAWING STATUS:

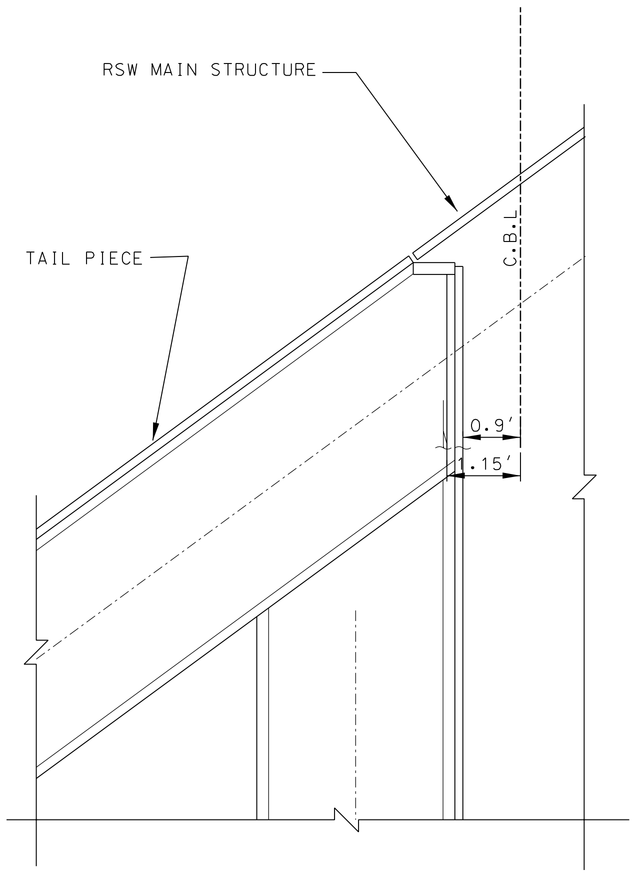
DRAWING NO.

PLATE
2

NOTES:

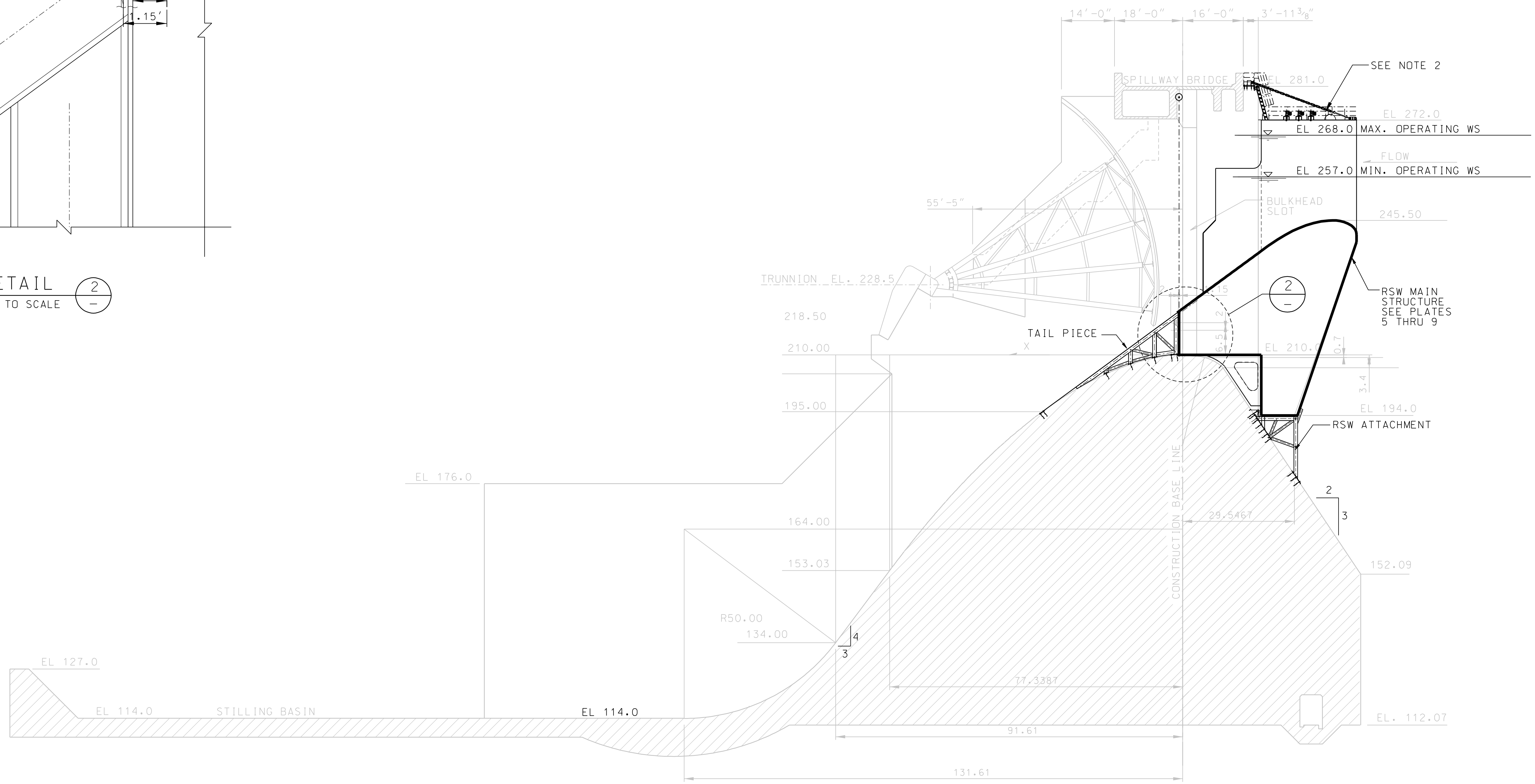
1. DEFLECTOR SHOWN IS AN ASSUMED SIZE AND ELEVATION FOR ESTIMATING PURPOSES. ACTUAL DIMENSONS WILL BE DETERMINED BY HYDRAULIC MODELING.

2. 150K CAPACITY RESTRAINT CABLE FROM EXISTING CONCRETE PIER NOSE TO RSW STEEL STRUCTURE.



DETAIL 2/–

NOT TO SCALE



TYPICAL SPILLWAY CROSS SECTION - LOOKING NORTH A/2

30' 0 30'

US Army Corps of Engineers Portland District

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SPILLWAY CROSS SECTION LOOKING NORTH

DRAWING STATUS:

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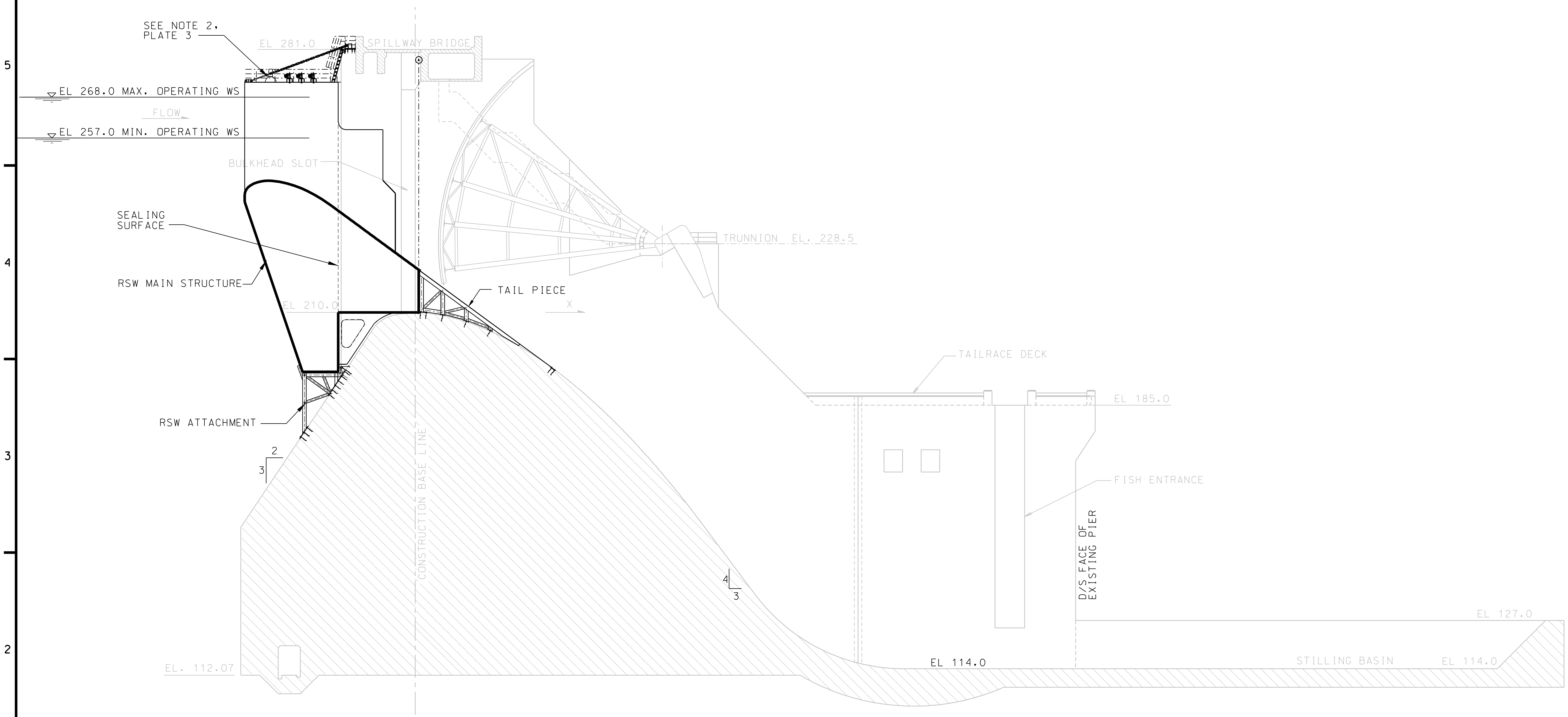
PLATE 3



TYPICAL SPILLWAY CROSS SECTION – LOOKING SOUTH (B/2)

30' 0 30'

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SPILLWAY CROSS SECTION
LOOKING SOUTH

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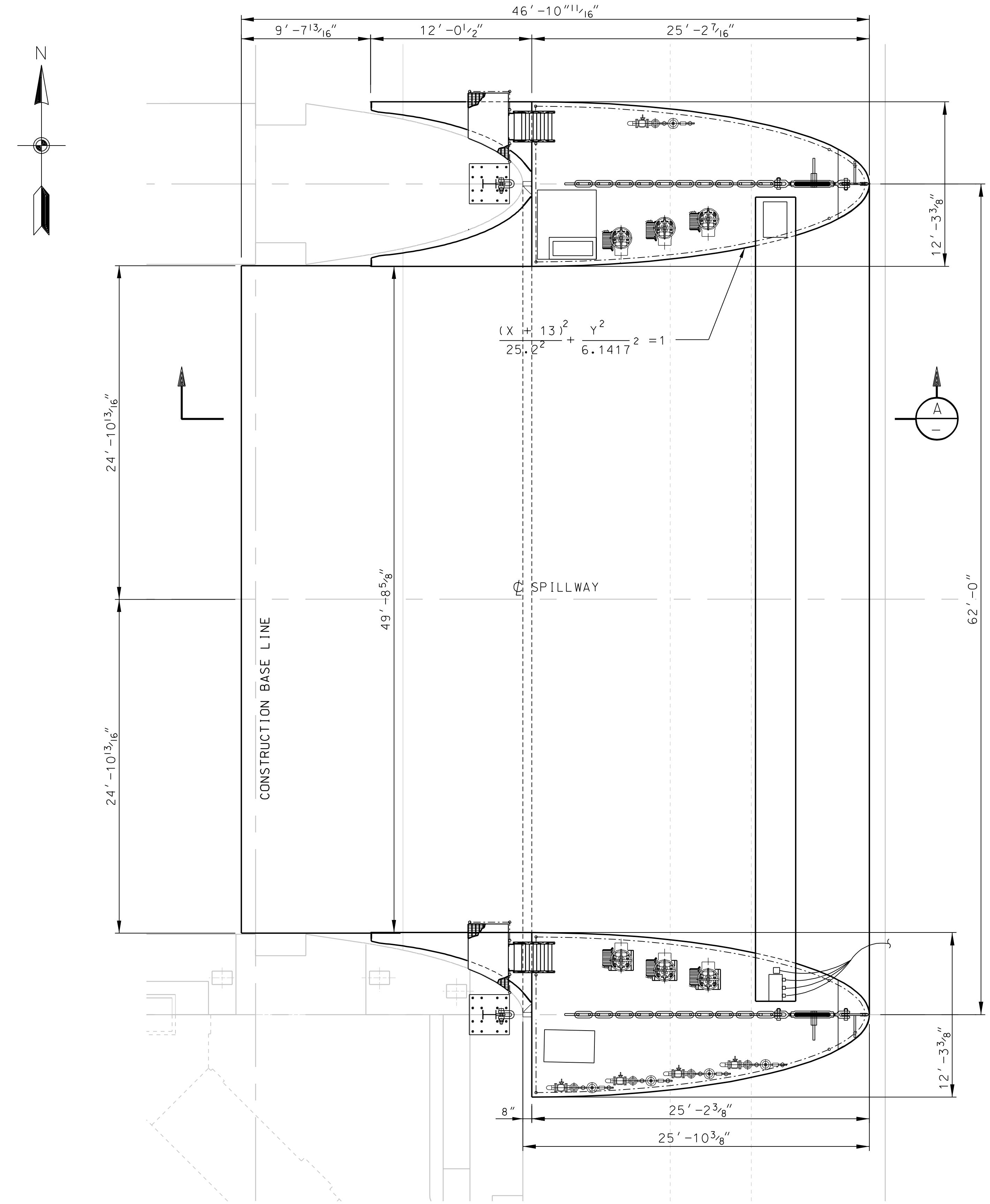
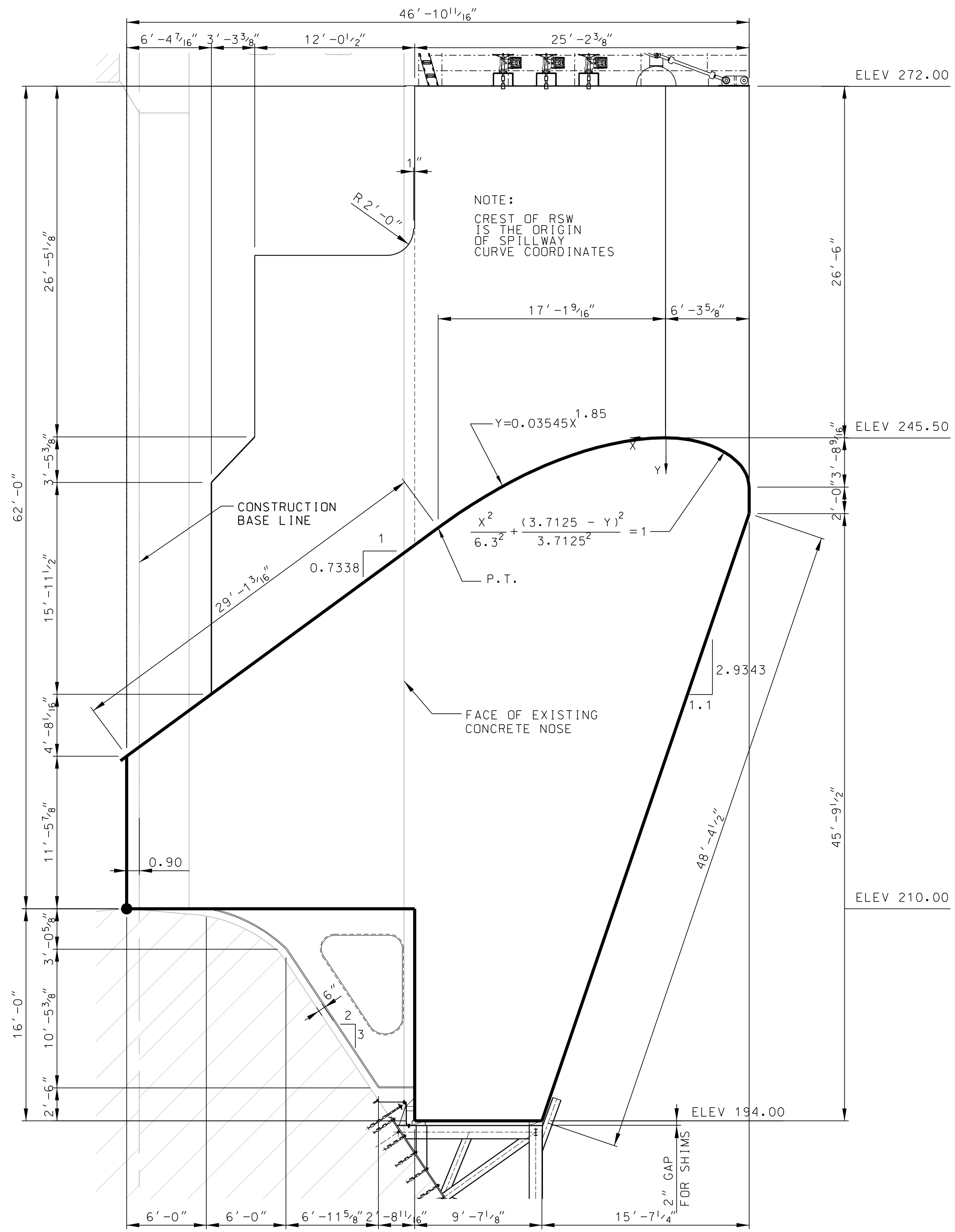
PLATE
4



RSW PLAN



SECTION A

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RSW MAIN STRUCTURE PLAN AND SECTION

DRAWING STATUS:

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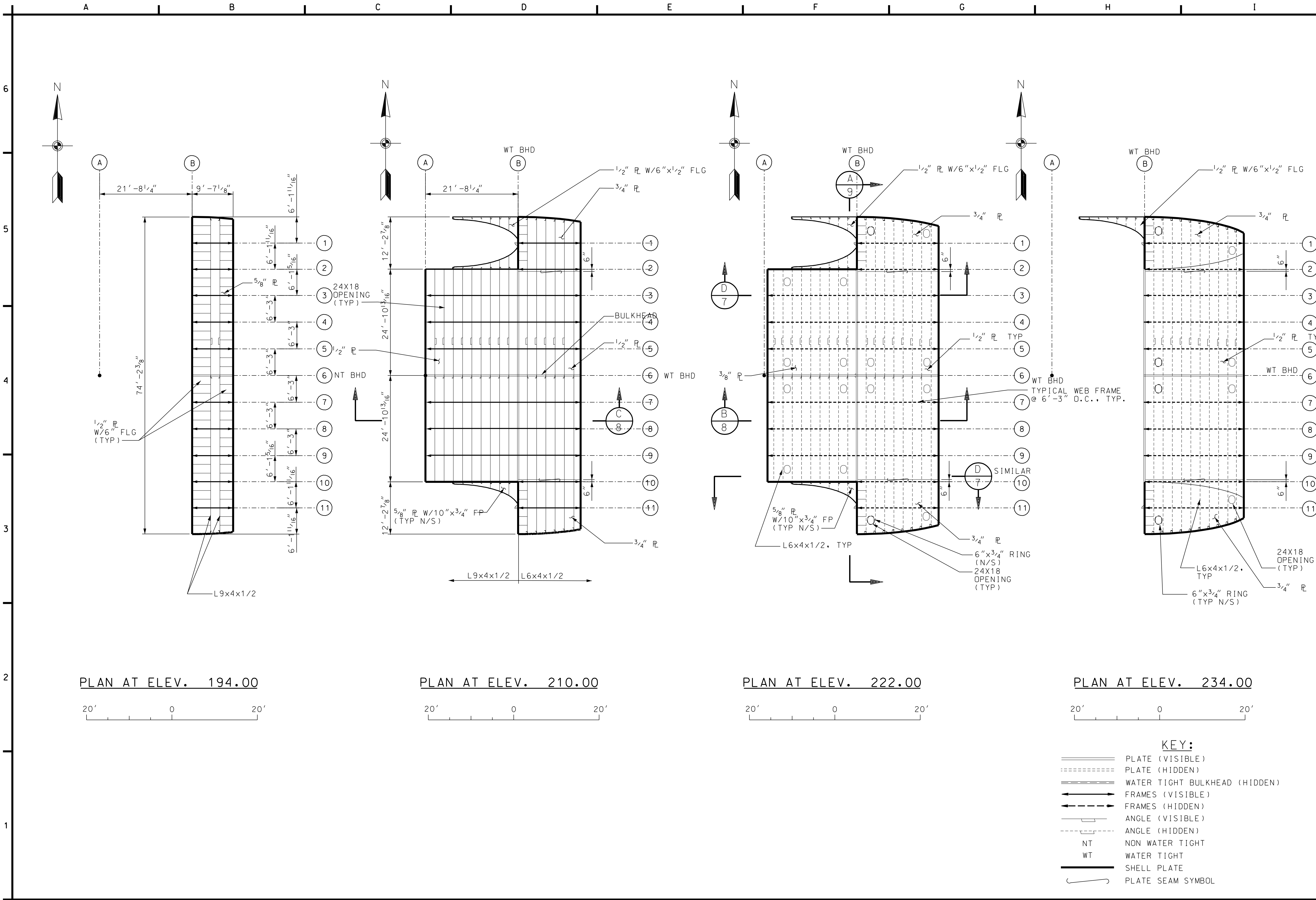
PLATE 5



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REMOVABLE SPILLWAY WEIR
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MAIN STRUCTURE DETAILS

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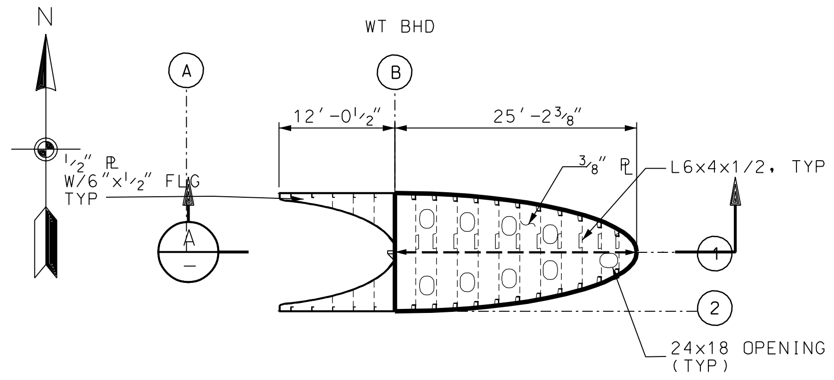
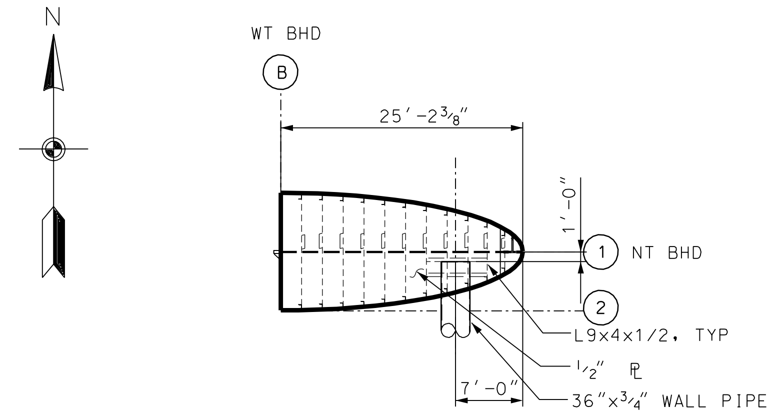
PLATE
6



PLAN AT ELEV. 42.50, 49.00
ELEV. 36.00 SIMILAR

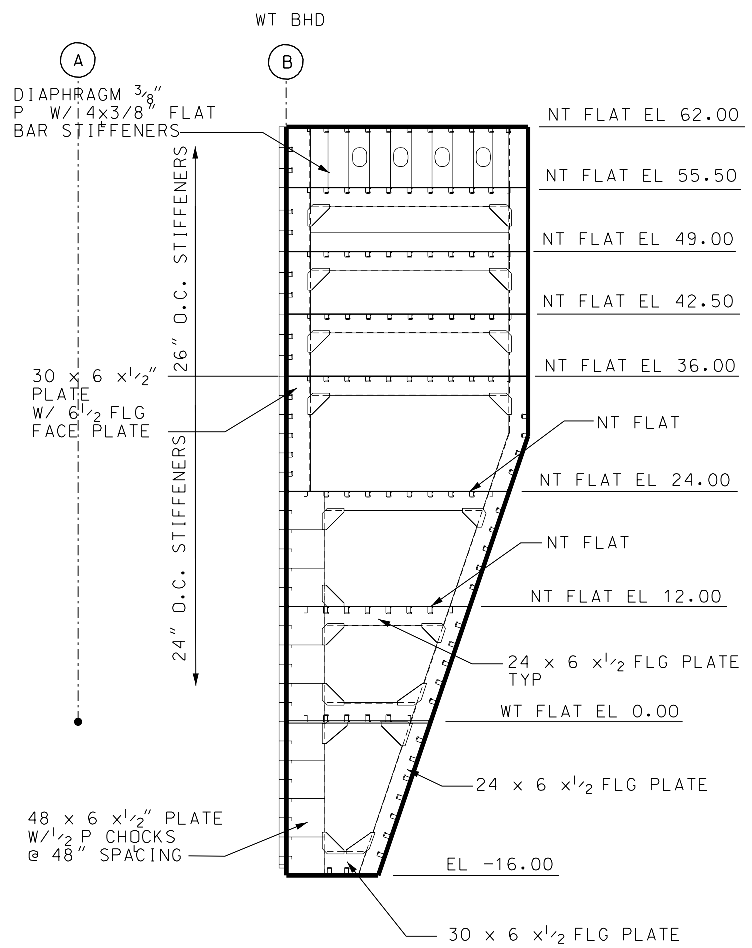
20' 0 20'



PLAN AT ELEV. 62.00
ELEV. 55.50 SIMILAR

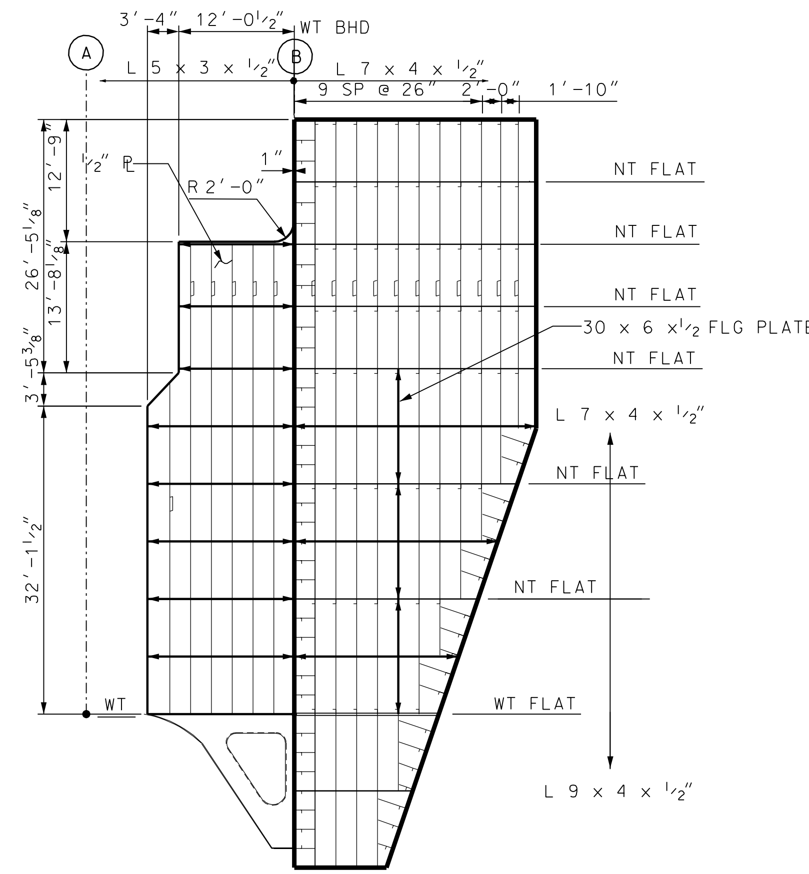
20' 0 20'

NORTH SIDE SHOWN
SOUTH SIMILAR



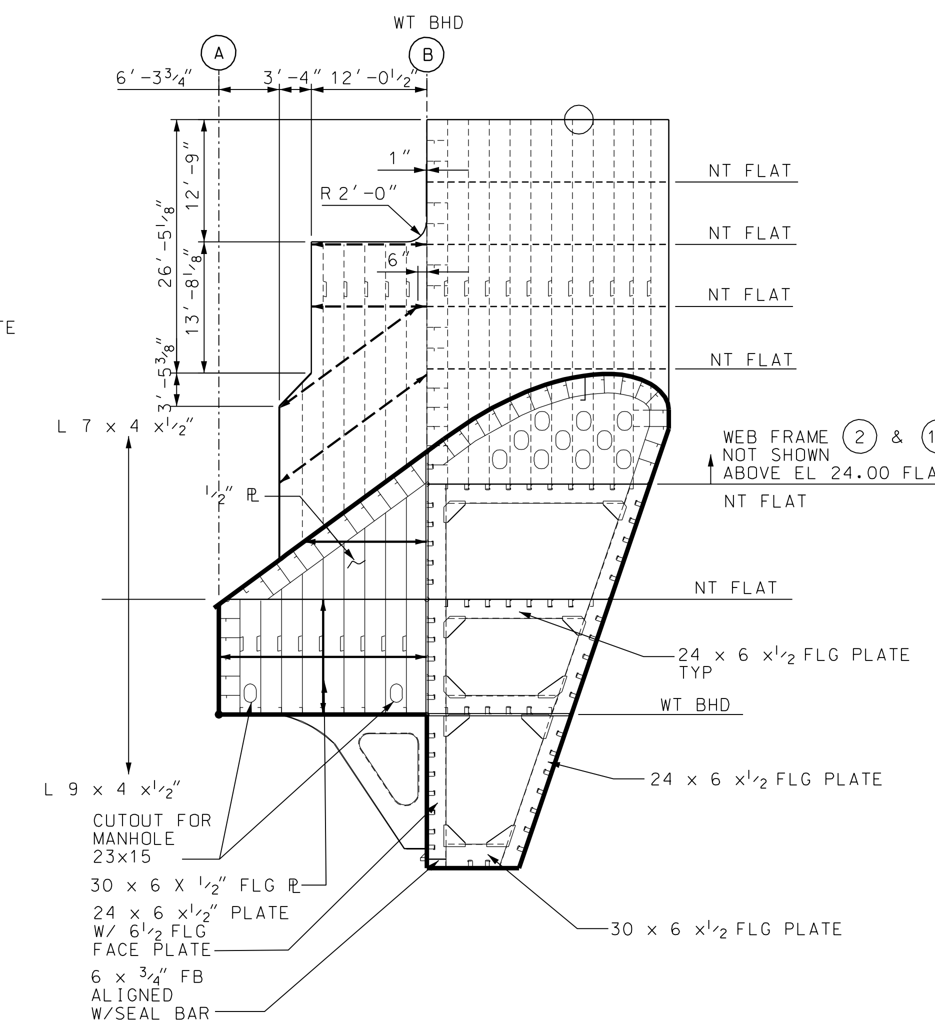
SECTION AT PIER CENTERLINE (A)

20' 0 20'



PIER 19
N. PIER BULKHEAD AND FAIRING

20' 0 20'



S PIER 19, N PIER 20
PIER BULKHEAD AND FAIRING

20' 0 20'

SEE KEY ON PLATE 6

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MAIN STRUCTURE
SECTIONS 1

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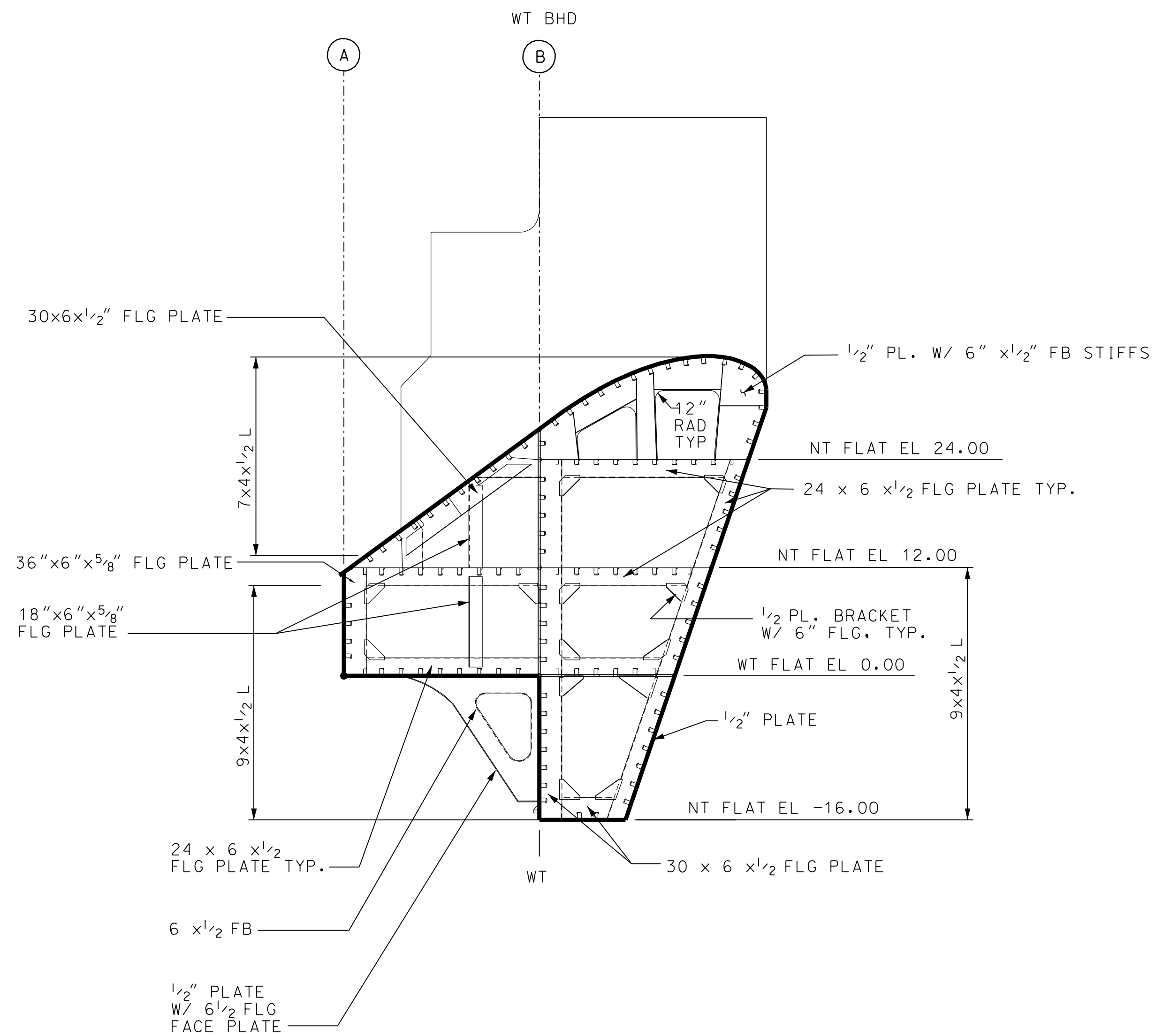
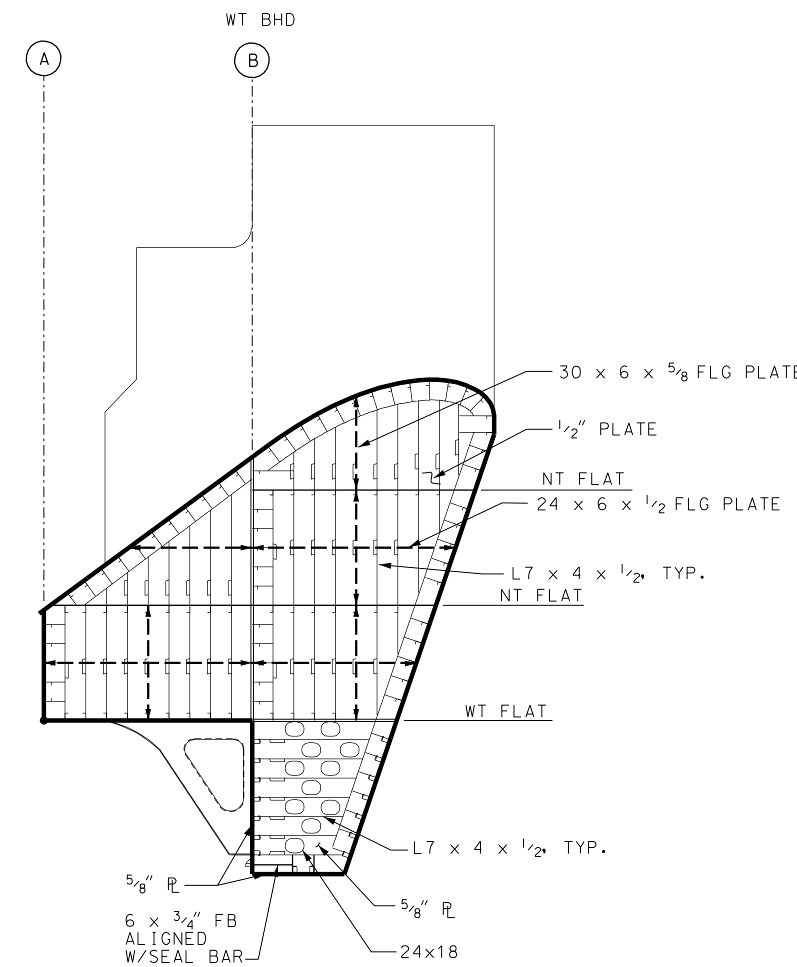
PLATE
7



TYPICAL WEB FRAME B/6

(6'-3" SPACING)

20' 0 20'



BULKHEAD AT SPILLWAY CENTERLINE C/6

20' 0 20'

SEE KEY ON PLATE 6

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JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
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MAIN STRUCTURE SECTIONS 2

DRAWING STATUS:

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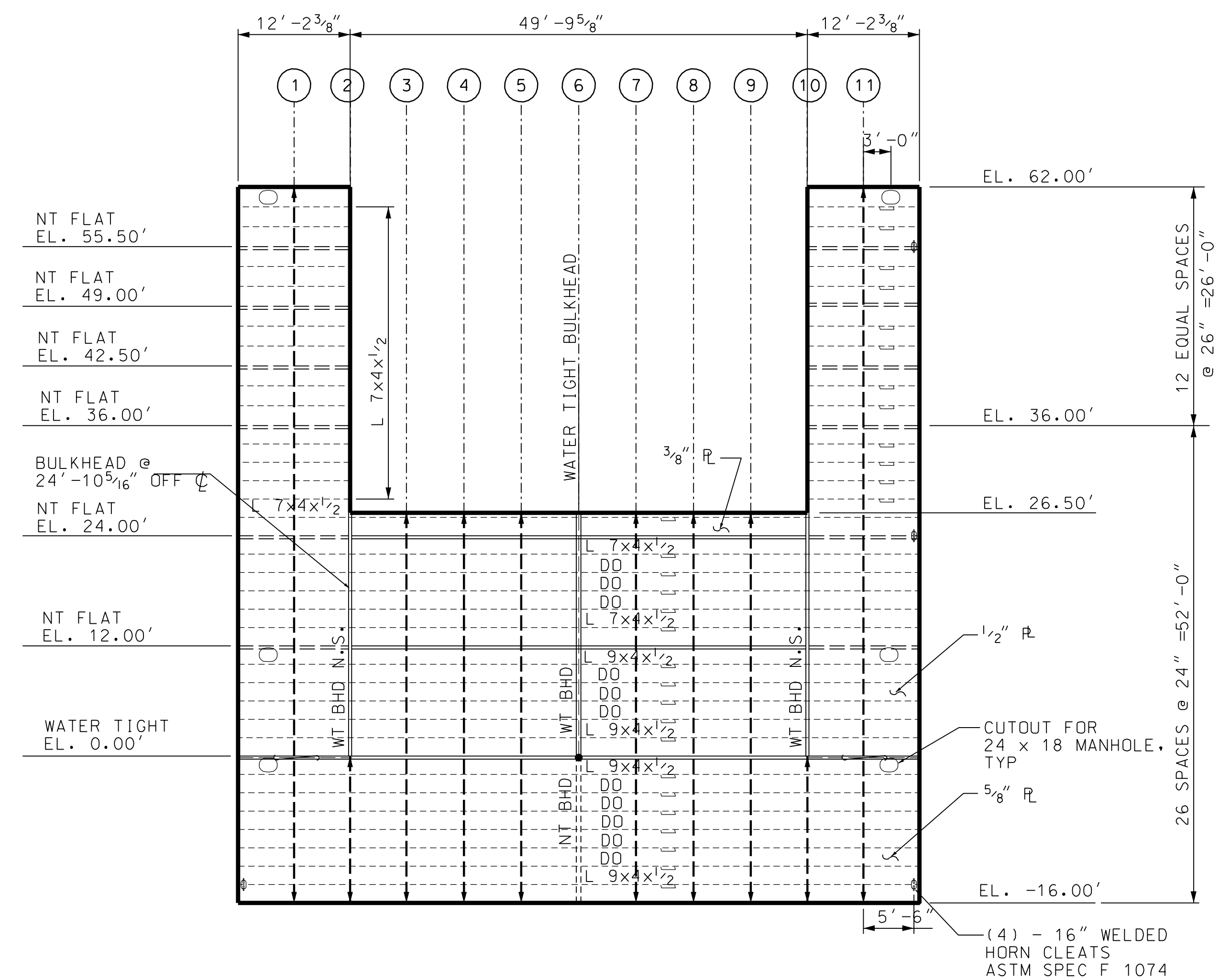
PLATE 8



BULKHEAD SECTION A/6

20' 0 20'

SEE KEY ON PLATE 6

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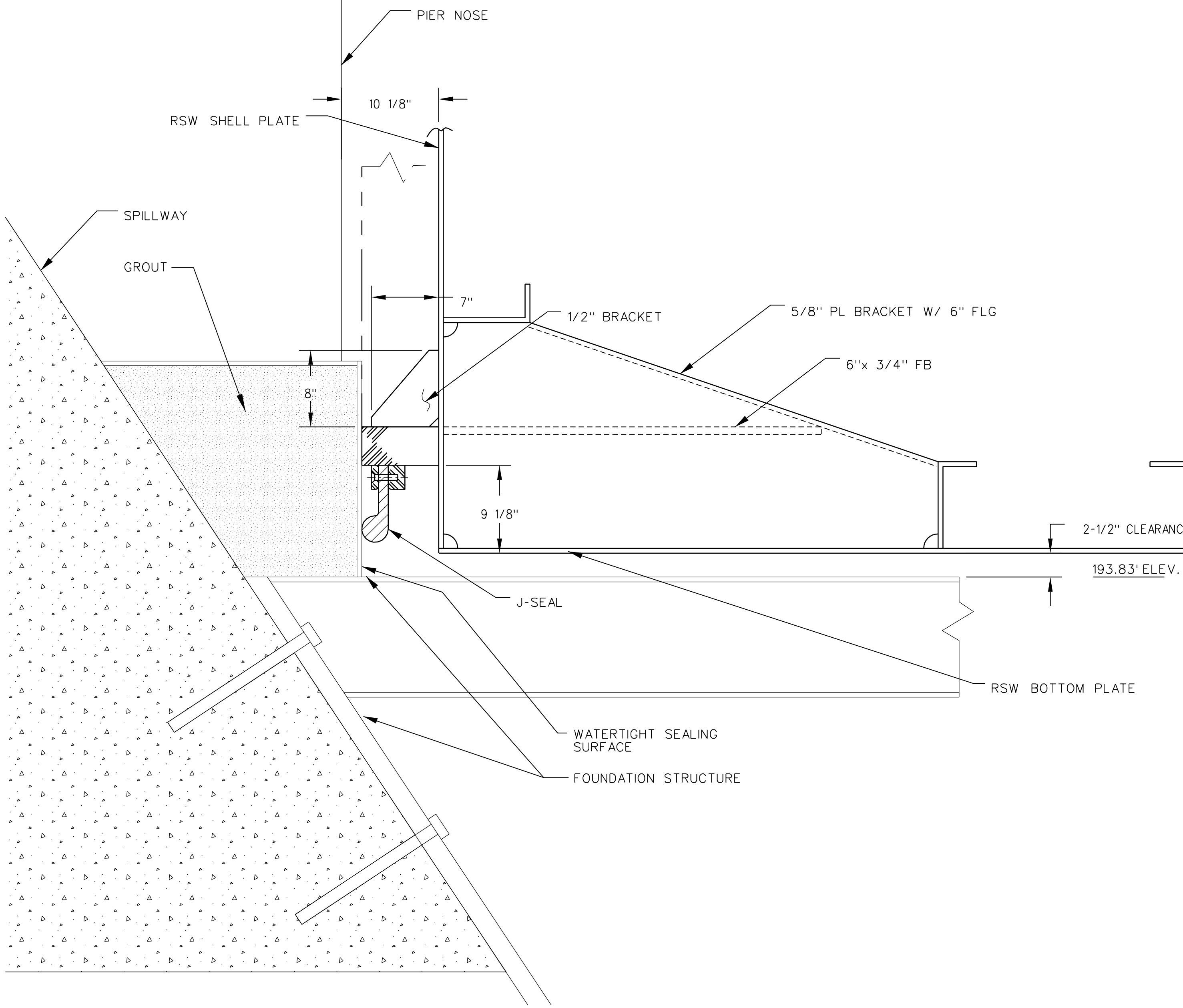
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT

MAIN STRUCTURE
BULKHEAD

DRAWING STATUS:

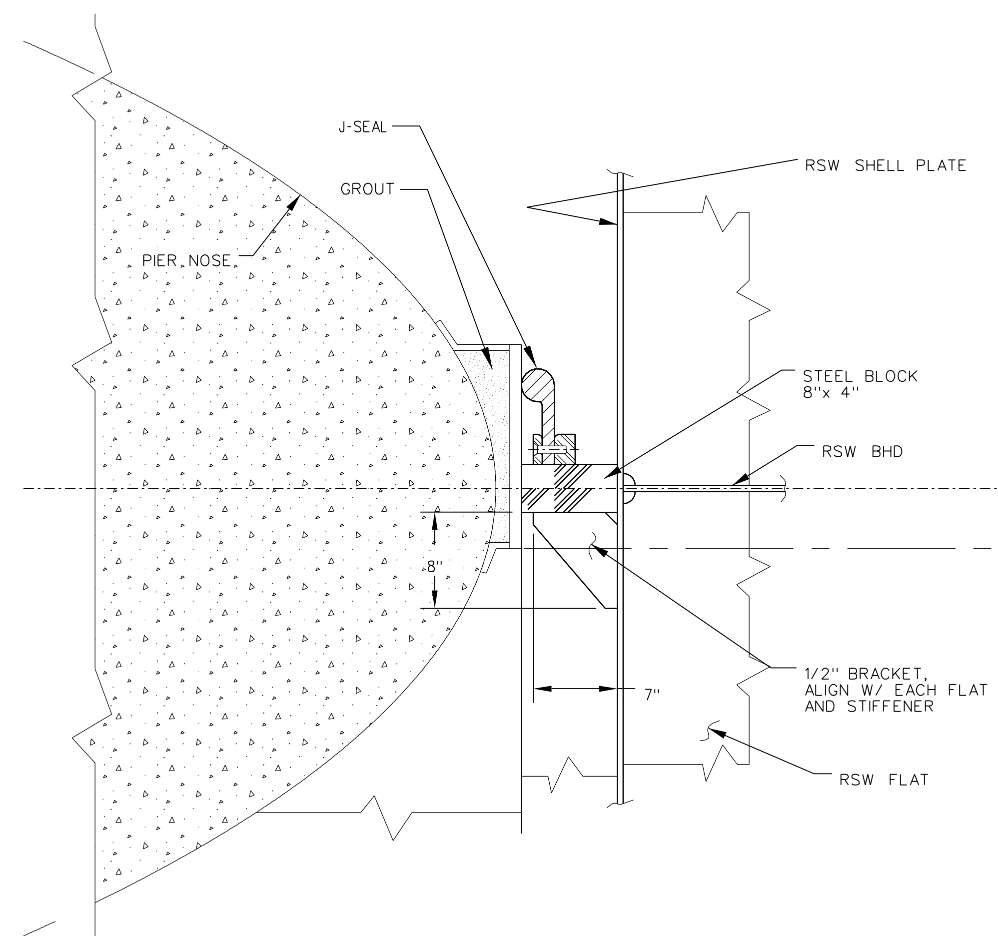
DRAWING NO.

PLATE 9



SECTION C/S4
BOTTOM SEAL
SCALE: 1"=1'-0"

12" 6" 0 1' 2'



SECTION G/S10
NORTH PIER SEAL
SOUTH PIER SEAL (SIMILAR/OPPOSITE)

12" 6" 0 1' 2'

US Army Corps of Engineers
Portland District

Revision	Date	Description	By
			xxx

Designed by: TGA
Drawn by: IBS
Checked by: JAS
Submitted by: DONALD R. CHAMBERS, P.E.
Chief, Structural & Arch. Design
Date: OCT 2001
CADD File Name: JDF0010G04.DGN
Technical Manager: MATTHEW HANSON

CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE

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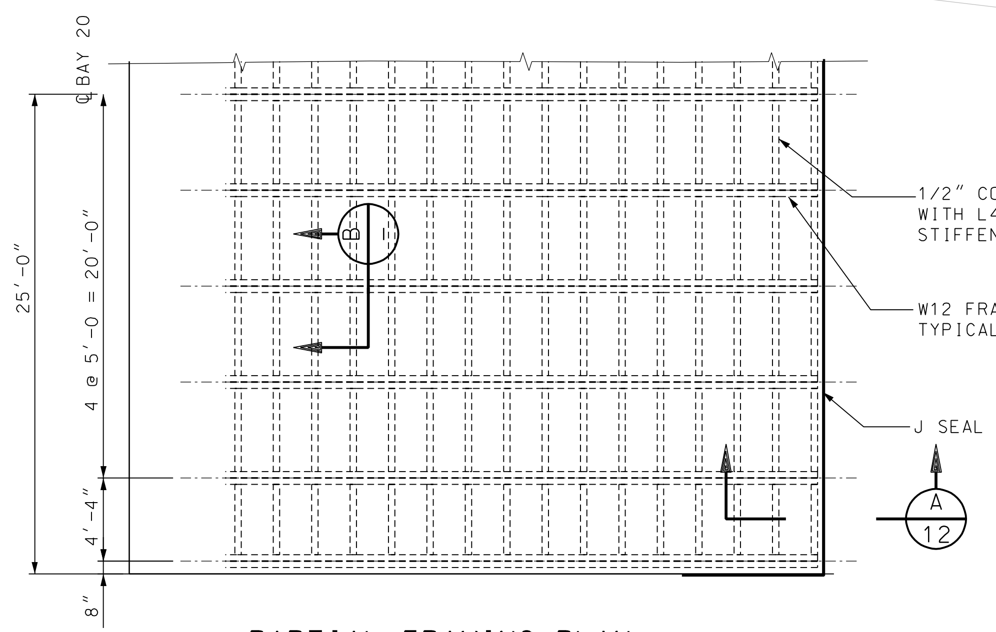
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT

PERIMETER SEAL
ARRANGEMENT

DRAWING STATUS:

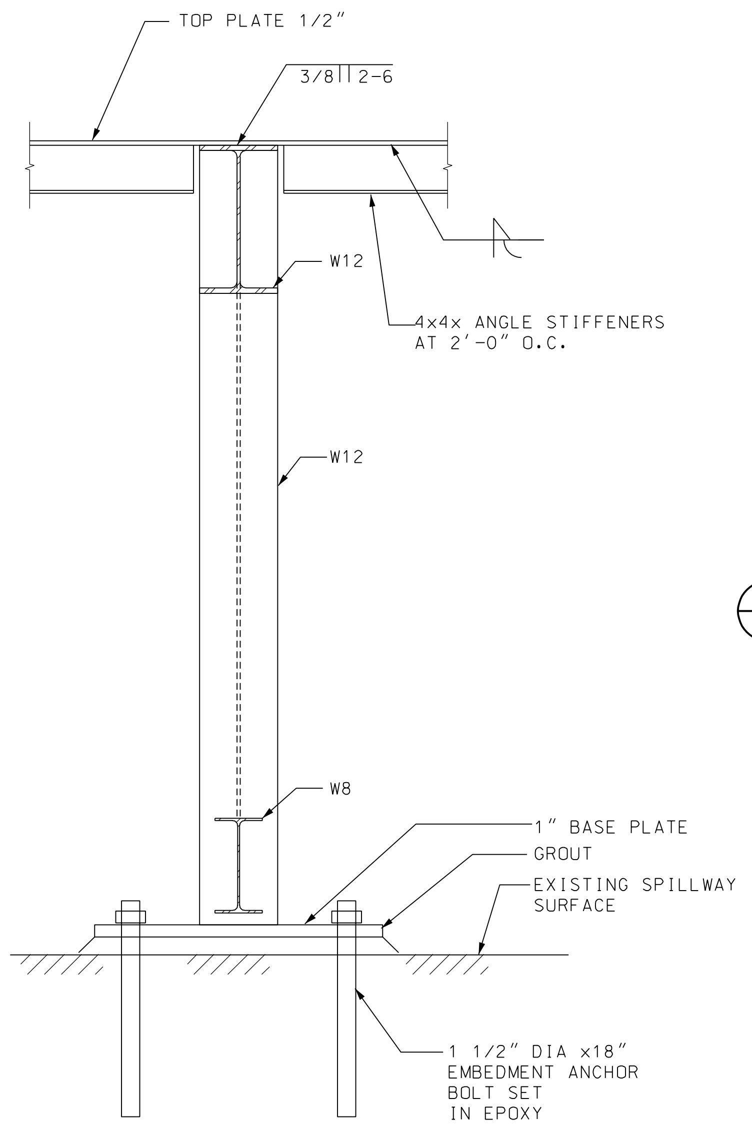
DRAWING NO.

PLATE
10



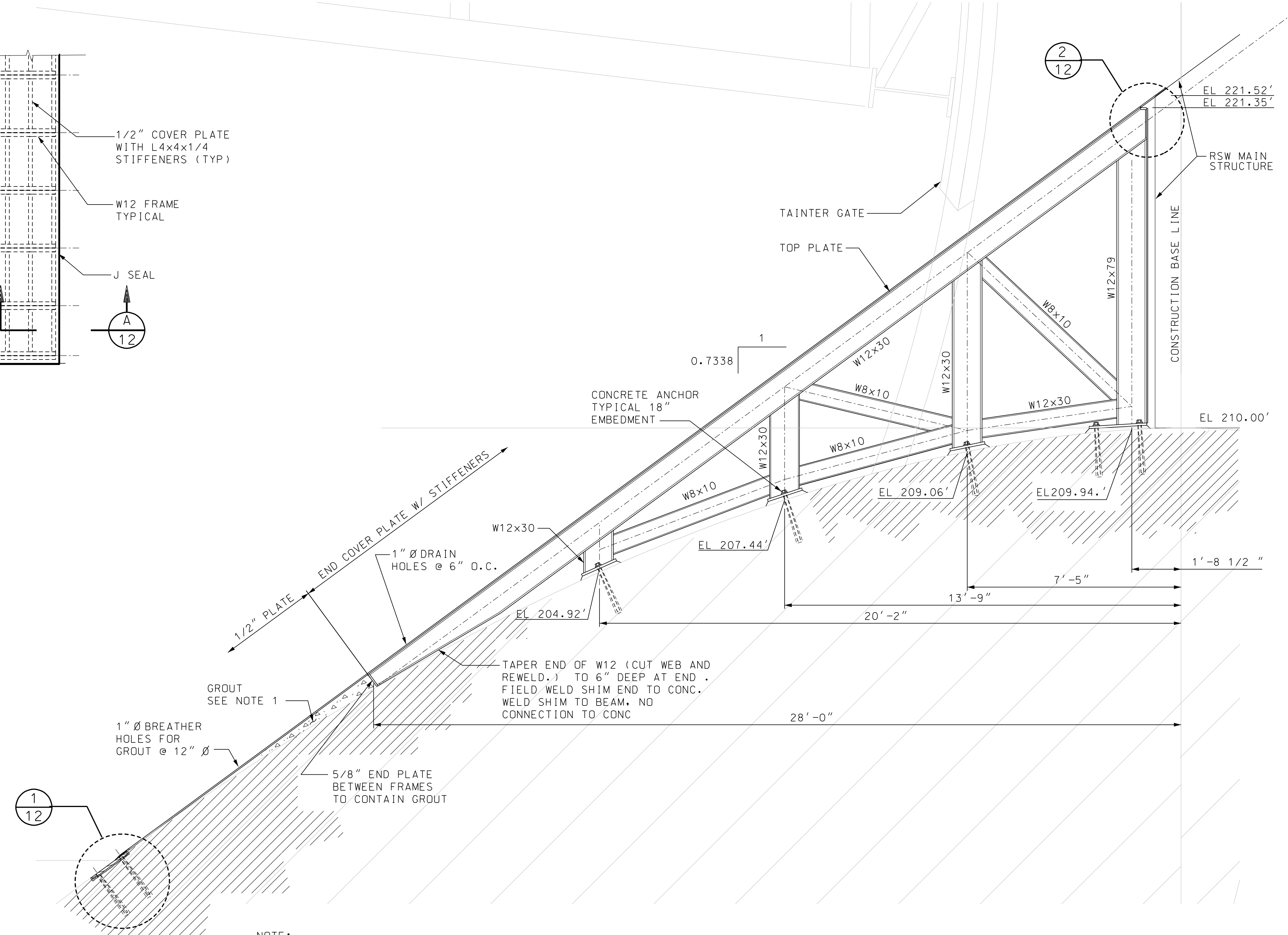
PARTIAL FRAMING PLAN

10' 0 10'



TYPICAL FRAME SECTION B/–

12" 6" 0 1' 2'



NOTE:
1. PUMP GROUT INTO SPACE BETWEEN COVER PLATE AND EXISTING CONC SURFACE, PROVIDE BOND BREAKER SO GROUT CAN BE REMOVED AT DECOMMISSIONING.

TYPICAL FRAME ELEVATION

12" 0 2' 4' 6'

US Army Corps of Engineers
Portland District

Revision	Date	Description	By

Designed by: MIESBAUER
Drawn by: ROBERTS
Checked by: HUANG
Submitted by: DONALD R. CHAMBERS, P.E. Chief, Structural & Arch. Design
Date: OCT 2001
CADD File Name: JDF00I011G04.DGN
Technical Manager: MATTHEW HANSON

CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE

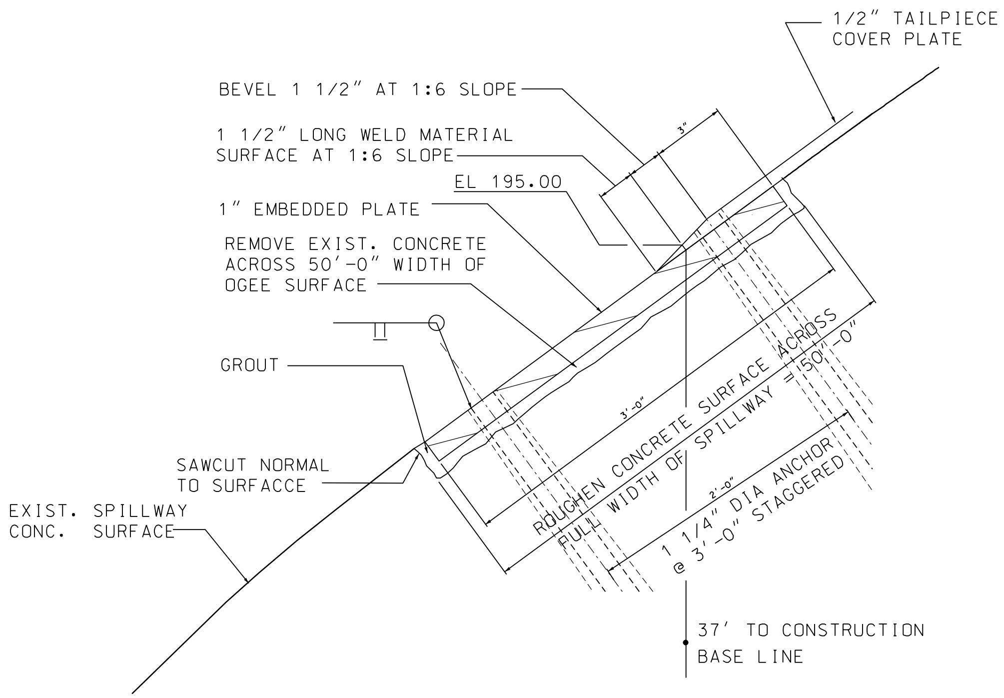
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON

COLUMBIA RIVER — JOHN DAY DAM — OREGON - WASHINGTON
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT
TAILPIECE
PLAN AND SECTION

DRAWING STATUS:

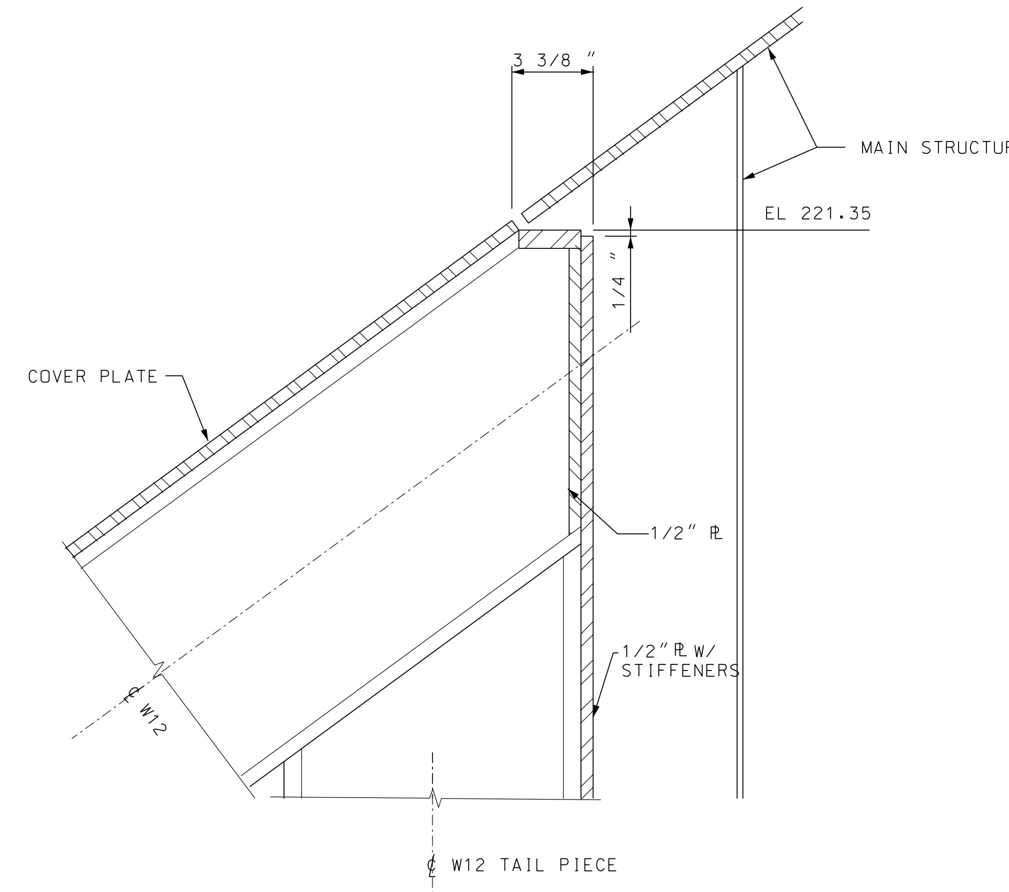
DRAWING NO.

PLATE
11



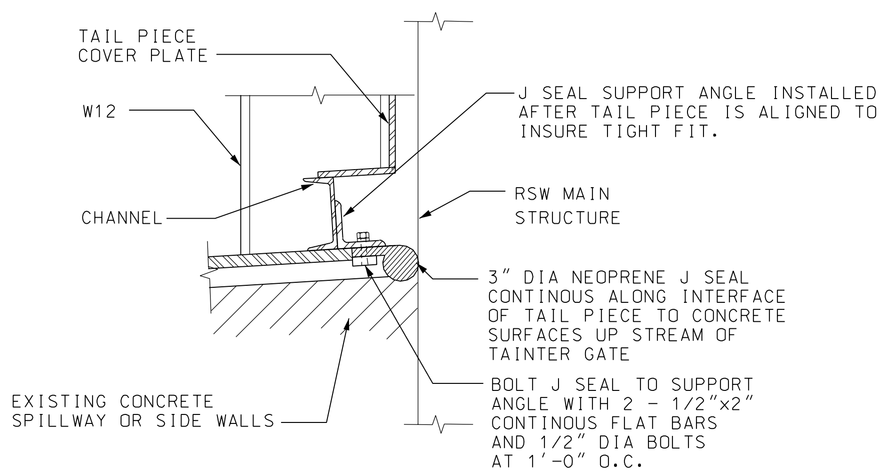
TAIL PIECE TO SPILLWAY CONNECTION 1/11

SCALE: NOINE



CLOSURE DETAIL 2/11

12" 9" 6" 3" 0 1'



J SEAL SECTION A/11

12" 6" 0 1' 2'

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Portland District

Revision | Date | By

Designed by: MIESBAUER
Date: OCT 2001
Drawn by: ROBERTS
CADD File Name: JDF0012G04.DGN
Checked by: HUANG
Technical Manager: MATTHEW HANSON
Submitted by: DONALD R. CHAMBERS, P.E. Chief, Structural & Arch. Design

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PORTLAND, OREGON

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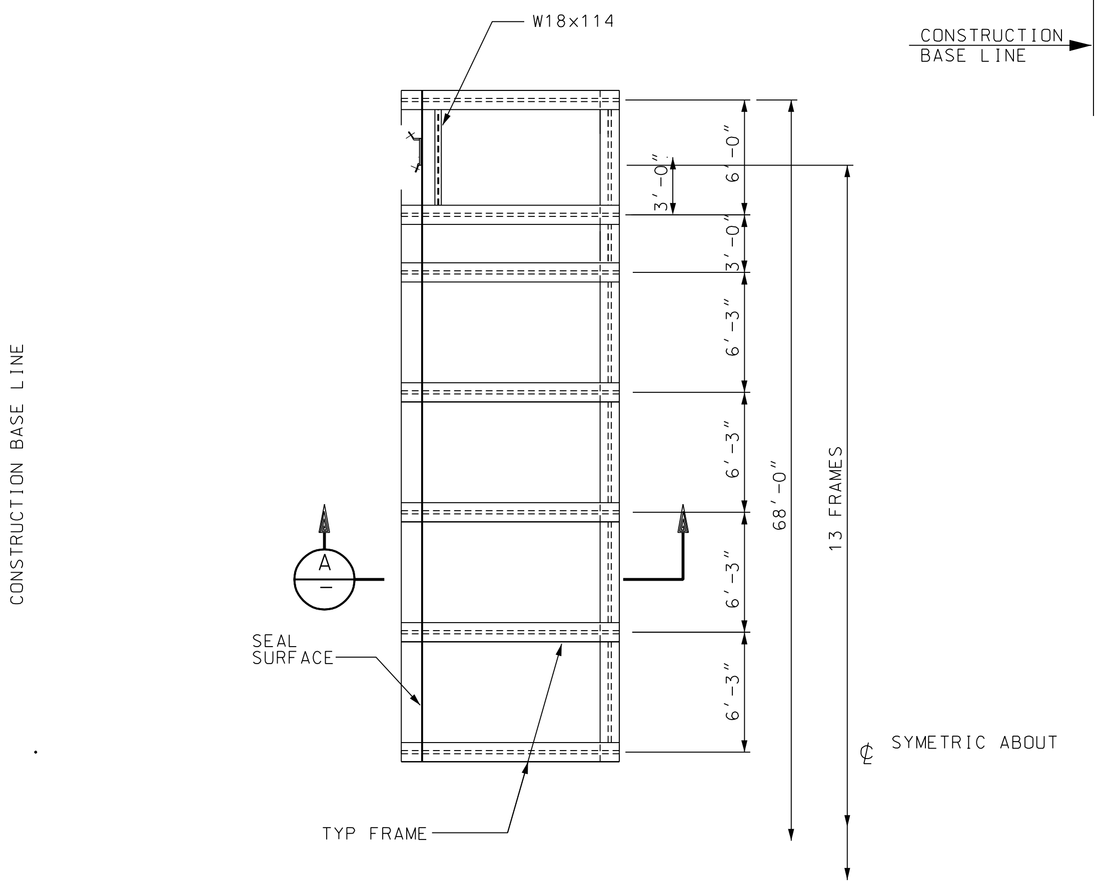
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT

TAILPIECE DETAILS

DRAWING STATUS:

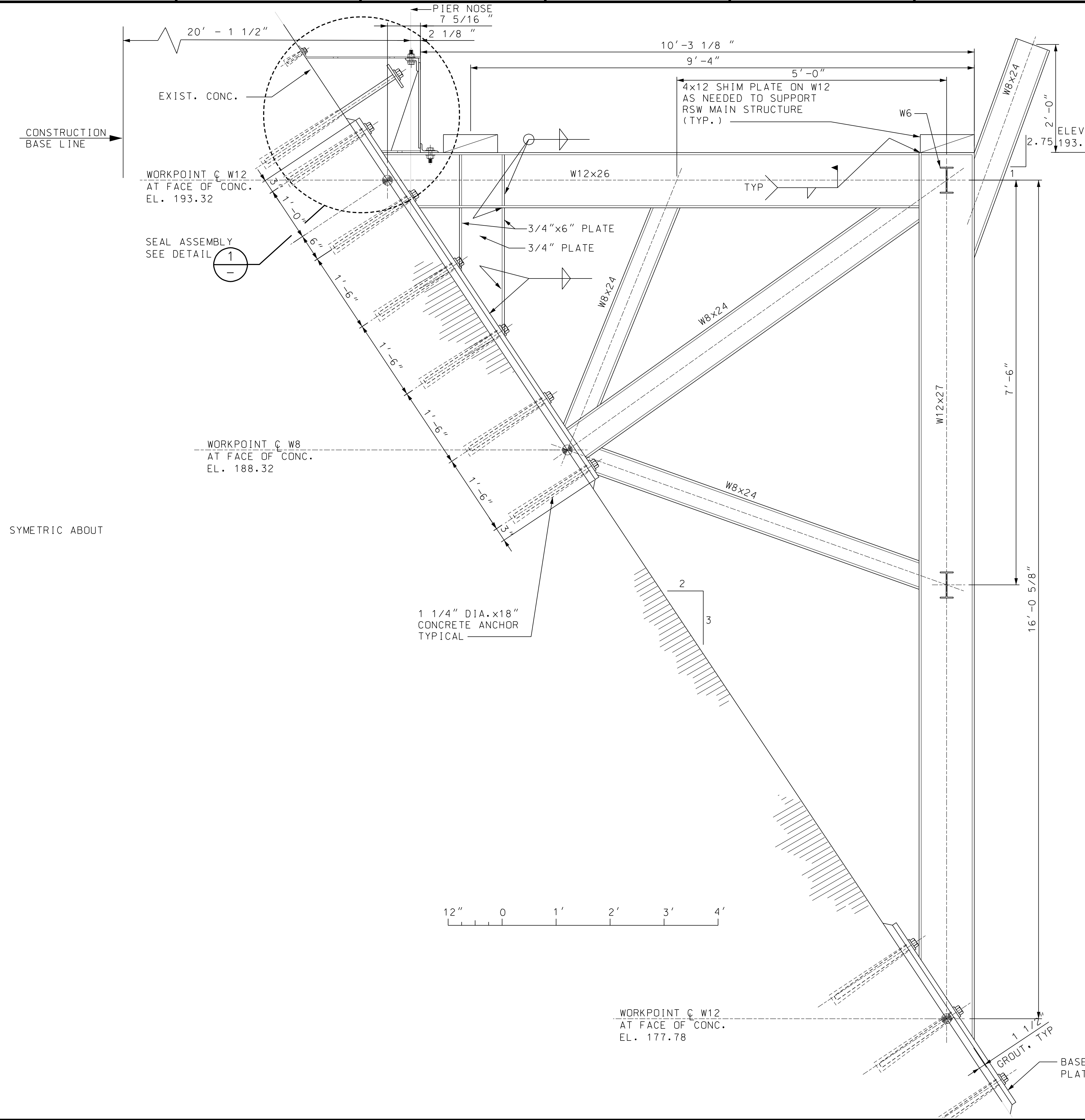
DRAWING NO.

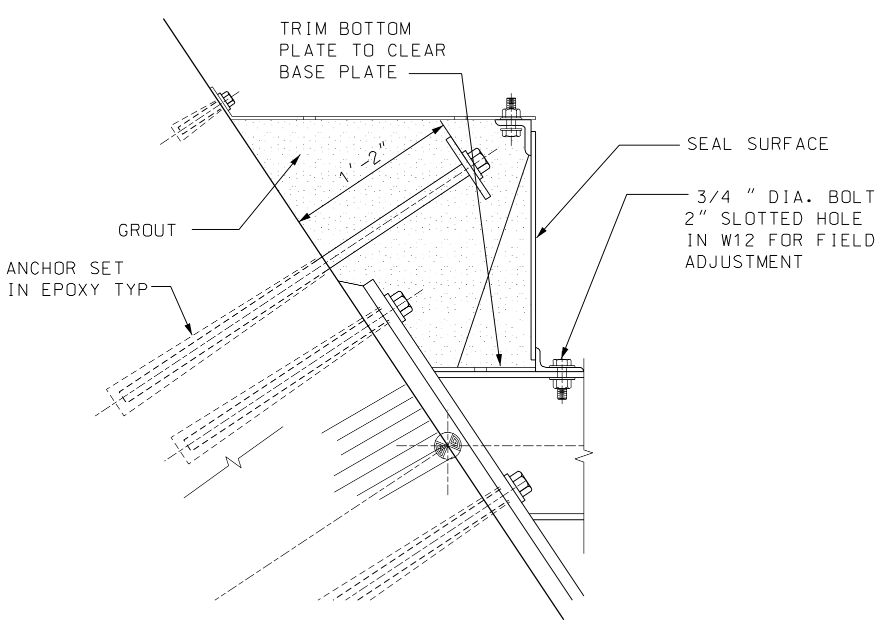
PLATE 12



PLAN - MAIN STRUCTURE ATTACHMENT

5' 0 5'





SEAL ASSEMBLY DETAIL 1

12" 6" 0 1' 2'

US Army Corps of Engineers
Portland District

Revision	Date	Description	By

Designed by: MIESBAUER
Drawn by: ROBERTS
Checked by: HUANG
Submitted by: DONALD R. CHAMBERS, P.E. Chief, Structural & Arch. Design
Date: OCT 2001
CADD File Name: JDF00I013G04.DGN
Technical Manager: MATTHEW HANSON

CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON

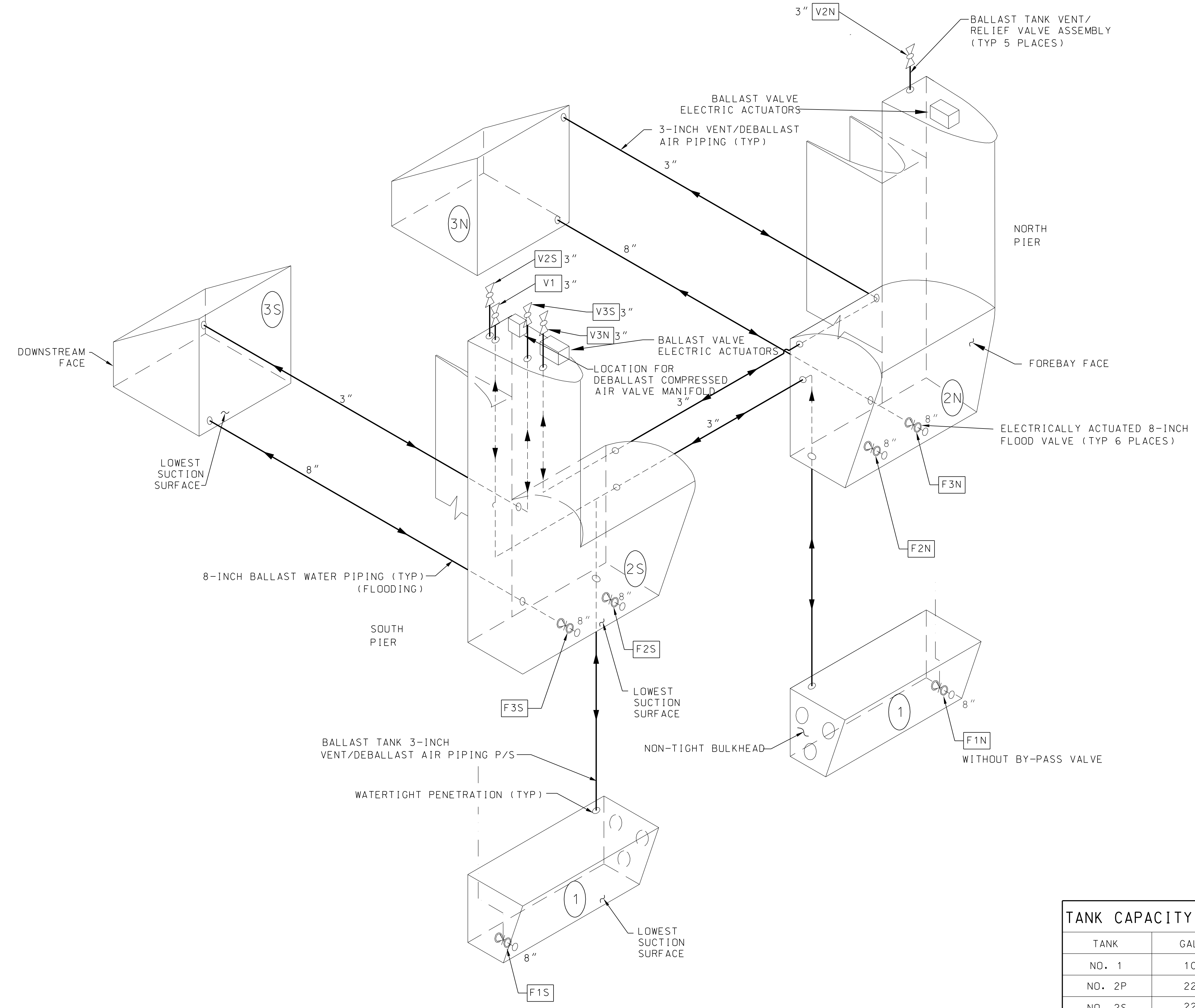
COLUMBIA RIVER JOHN DAY DAM OREGON - WASHINGTON
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT

MAIN STRUCTURE ATTACHMENT

DRAWING STATUS:

DRAWING NO.

PLATE 13



SCHEMATIC

RSW MAIN STRUCTURE BALLAST SYSTEM
NO SCALE

GENERAL NOTES

1. THE PIPING USED FOR FLOODING AND VENTING THE VARIOUS TANKS ARE CONSIDERED AS PART OF THE BALLAST SYSTEM. THE COMPRESSED AIR PIPING FROM THE SUPPORT TUG CONNECTION TO THE VARIOUS TANK VENT ASSEMBLIES ARE CONSIDERED TO BE THE DEBALLAST AIR SYSTEM.
2. ALL FLOOD VALVES SHALL BE TERMINATED WITH SUCTION BELLMOUTHS HAVING AT LEAST 1.5 TIMES THE SUCTION PIPE FLOW AREA, ARRANGED TO ACHIEVE MINIMUM RESIDUAL WATER LEVEL. CONCENTRIC REDUCERS WITH PROPER DECK CLEARANCE COULD BE SUBTITUTED.
3. ONLY THE FLOOD VALVES AND VENT VALVES ARE SHOWN ON THIS DRAWING. THE DEBALLAST AIR VALVE AND THE RELIEF VALVE FOR EACH TANK ARE NOT SHOWN FOR CLARITY.
4. ASTM F1166, STANDARD PRACTICE FOR HUMAN ENGINEERING DESIGN FOR MARINE SYSTEMS, EQUIPMENT AND FACILITIES, SHALL BE OBSERVED FOR THE DETAILED DESIGN OF SYSTEMS.
5. PIPING SUPPORTS SHALL BE LINED WITH NEOPRENE. THE SELECTION AND INSTALLATION OF PIPE HANGERS SHALL BE IN ACCORDANCE WITH ASTM F708, DESIGN AND INSTALLATION OF RIGID PIPE HANGERS; STAUFF TWIN CLAMP HANGER, FIGURE 9, FOR TUBING AND SPLIT CAP HANGERS; FIGURE 1, FOR PIPING.
6. GAUGE PIPING ASSEMBLIES SHALL BE IN ACCORDANCE WITH ASTM F721, GAUGE PIPING ASSEMBLIES.
7. HULL PIPING PENETRATIONS SHALL BE IN ACCORDANCE WITH ASTM F994, DESIGN AND INSTALLATION OF OVERBOARD DISCHARGE HULL PENETRATION CONNECTIONS, TYPE II, CLASS 1.
8. VALVES SHALL BE IDENTIFIED BY LABEL PLATES IN ACCORDANCE WITH ASTM F992, VALVE LABEL PLATES, EXCEPT THAT ALUMINUM LABEL PLATES ARE NOT TO BE USED ON VALVES INSIDE THE RSW MAIN STRUCTURE.
9. PIPE WELDING SHALL BE IN ACCORDANCE WITH ASTM F722, WELDED JOINTS FOR SHIPBOARD PIPING SYSTEMS.
10. PIPE ROUTING AND HULL PENETRATIONS ARE SHOWN FOR ILLUSTRATION PURPOSES ONLY. REFER TO STRUCTURAL ARRANGEMENT FOR EXACT LOCATIONS OF HULL PENETRATIONS.

TANK CAPACITY TABLE

TANK	GALLONS
NO. 1	107,000
NO. 2P	229,000
NO. 2S	229,000
NO. 3P	81,973
NO. 3S	81,793

SYMBOL LIST

PIPE
PIPE (HIDDEN)
REACH ROD
BALL VALVE
BALL VALVE, VARIABLE PORT TYPE
GATE VALVE
RELIEF VALVE
PIPE CAP
B BASKET STRAINER
3S INDICATES BALLAST TANK NUMBER
F3N INDICATES VALVE DESIGNATION

PREFIX:
F - FLOODING
V - VENT
A - AIR (DEBALLASTING)

SUFFIX:
N - NORTH TANK
S - SOUTH TANK

US Army Corps of Engineers
Portland District

Revision	Description	Date	By

Designed by: DAI
Date: OCT 2001
Drawn by: KUTZ
CADD File Name: JDF001014G04.DWG
Checked by: DAI
Technical Manager: MATTHEW HANSON
Submitted by: DONALD R. CHAMBERS, P.E. Chief, Structural & Arch. Design

CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON

COLUMBIA RIVER JOHN DAY DAM OREGON - WASHINGTON
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT
RSW MAIN STRUCTURE
FLOODING/DEBALLAST SYSTEM
GENERAL ARRANGEMENT

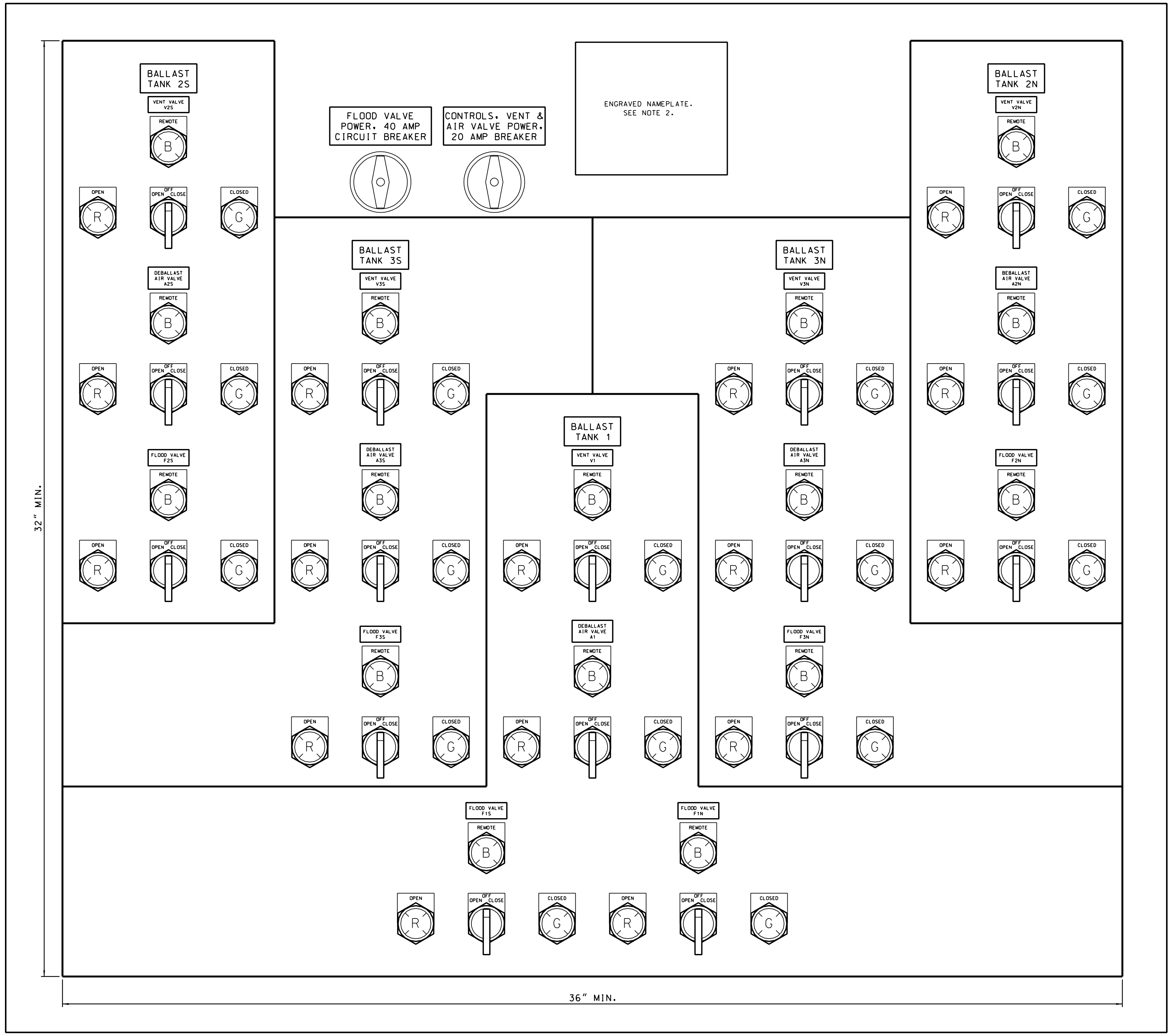
DRAWING STATUS:

DRAWING NO.

PLATE
14

NOTES:

1. ALL WORK SHALL BE DONE IN ACCORDANCE WITH THE NATIONAL ELECTRICAL CODE (NEC).
2. RED NAMEPLATE WITH WHITE LETTERS.
   FIRST LINE (3/8" LETTERS): CAUTION:
   OTHER LINES (1/4" LETTERS): DO NOT MOVE MORE THAN 1 FLOOD VALVE AT A TIME, F2S, F2N, F3S, F3N, OR F1. OTHERWISE IT MAY TRIP THE "FLOOD VALVE" CIRCUIT BREAKER. IF THE CIRCUIT BREAKER TRIPS PLACE ALL FLOOD VALVE SWITCHES IN THE "OFF" POSITION, THEN OPEN ONLY THE 1 VALVE CALLED FOR IN THE OPERATING PROCEDURE.



TUG LOCAL CONTROL PANEL
(TEMPORARY MOUNTING ON TUG-BOAT)

US Army Corps of Engineers
Portland District

Revision	Date	Description	By

Designed by: DEERKOP
Drawn by: DEERKOP
Checked by: DORRATCAGUE
Submitted by: DONALD R. CHAMBERS, P.E. Chief, Structural & Arch. Design
Date: OCT 2001
CADD File Name: JDF001015G04.DGN
Technical Manager: MATTHEW HANSON

CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
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JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT

CONTROL PANEL LAYOUT

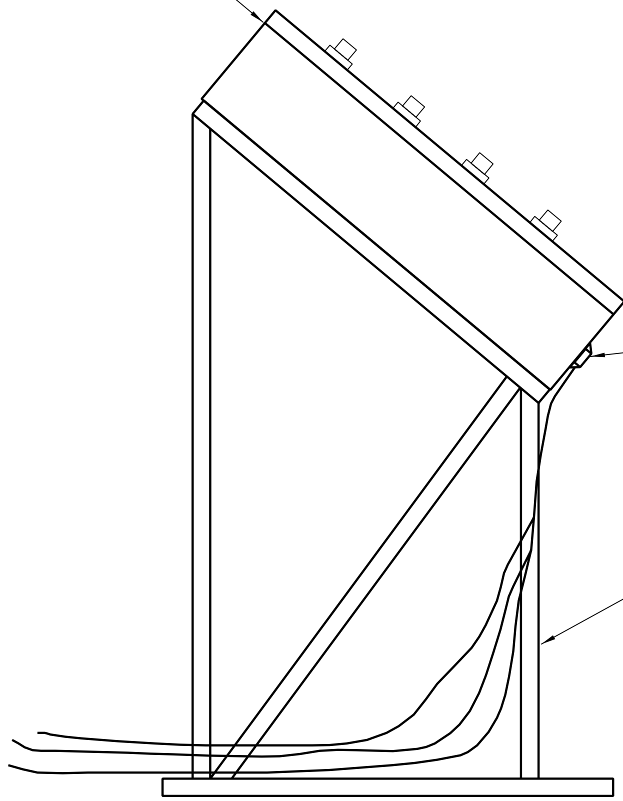
DRAWING STATUS:

DRAWING NO.

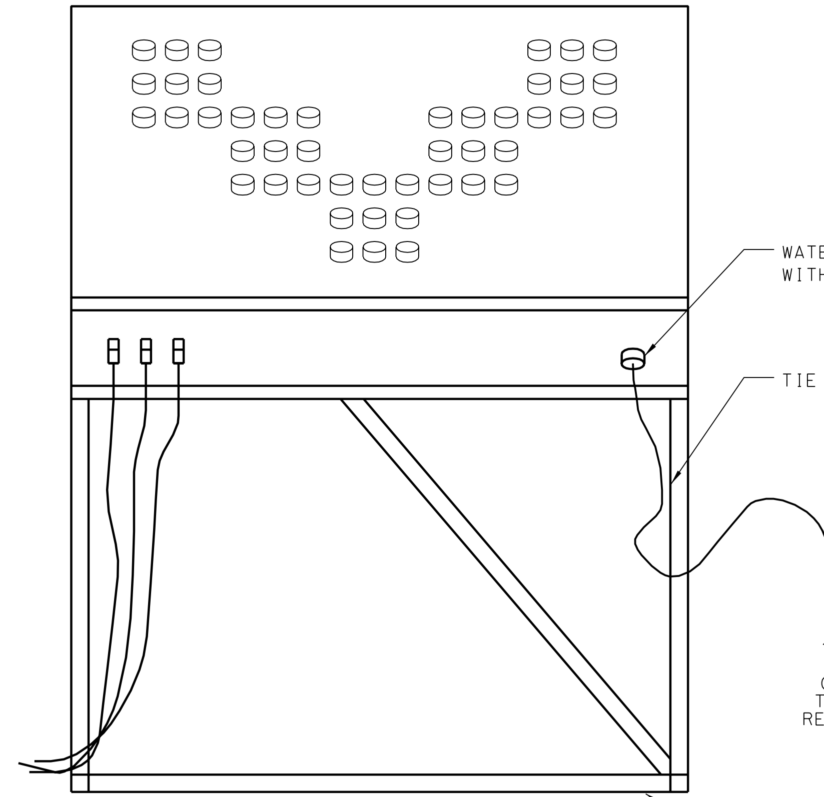
PLATE
15

NOTES:

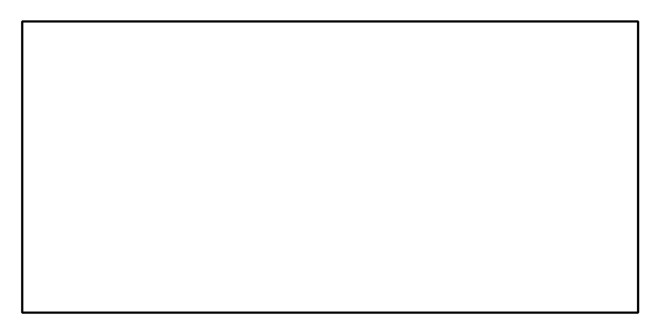
1. TEMPORARILY MOUNT/SECURE TUG LCP TO DECK OF TUG BOAT IN A MANNER APPROVED BY THE TUG OPERATOR, SAND BAGS PLACED ON BOTTOM PANEL, ETC.
2. SECURE INTERCONNECTING CABLE WITH CORD GRIPS AT THE RSW AND TUG. NO STRAIN SHALL BE PLACED ON THE QUICK-DISCONNECT CABLE CONNECTORS.



TUG LOCAL CONTROL PANEL
(LEFT SIDE ELEVATION)



TUG LOCAL CONTROL PANEL
(FRONT ELEVATION)



RSW LOCAL CONTROL PANEL
(FRONT ELEVATION)

US Army Corps of Engineers
Portland District

Revision	Date	Description	By

Designed by: DEERKOP
Drawn by: DEERKOP
Checked by: DORRATCAGUE
Submitted by: DONALD R. CHAMBERS, P.E. Chief, Structural & Arch. Design
Date: OCT 2001
CADD File Name: JDF10E002C01.DGN
Technical Manager: MATTHEW HANSON

CH2M HILL MONTGOMERY WATSON JOINT VENTURE

U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS PORTLAND, OREGON

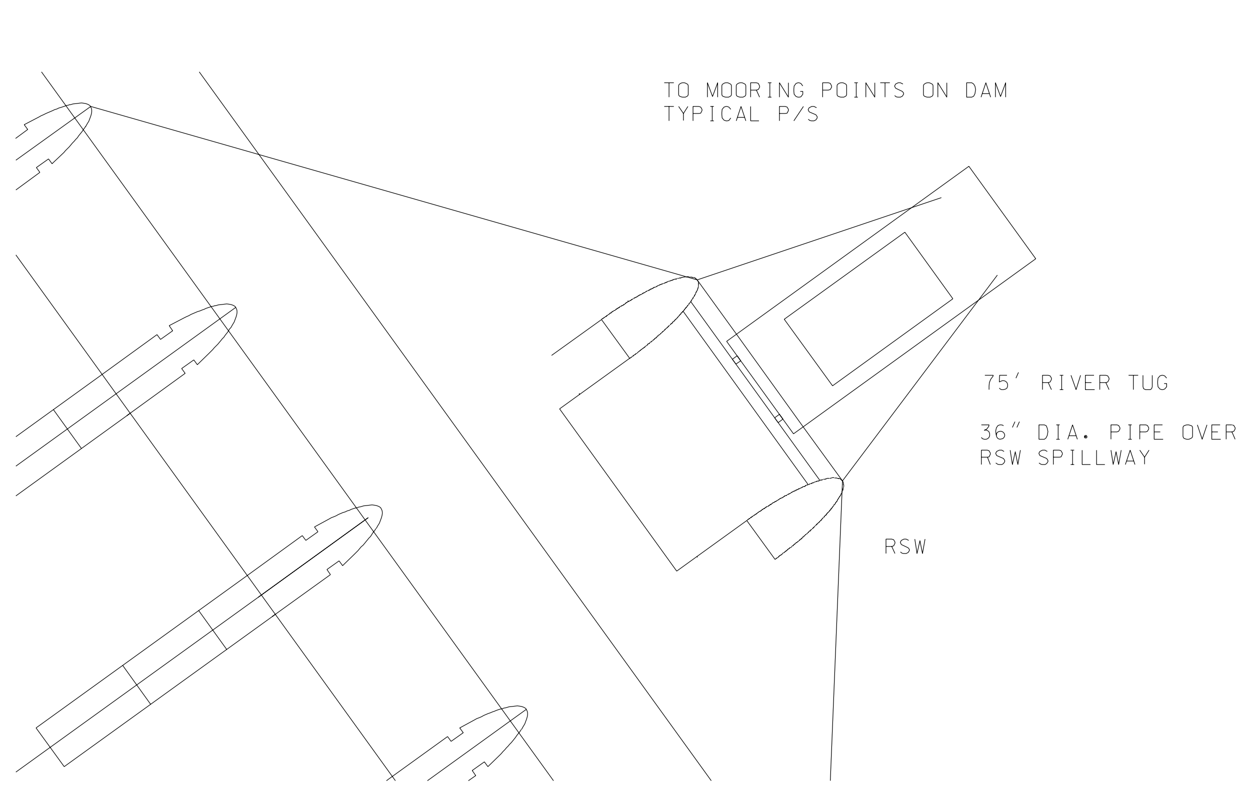
COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT

WIRING ARRANGEMENT AND DETAILS

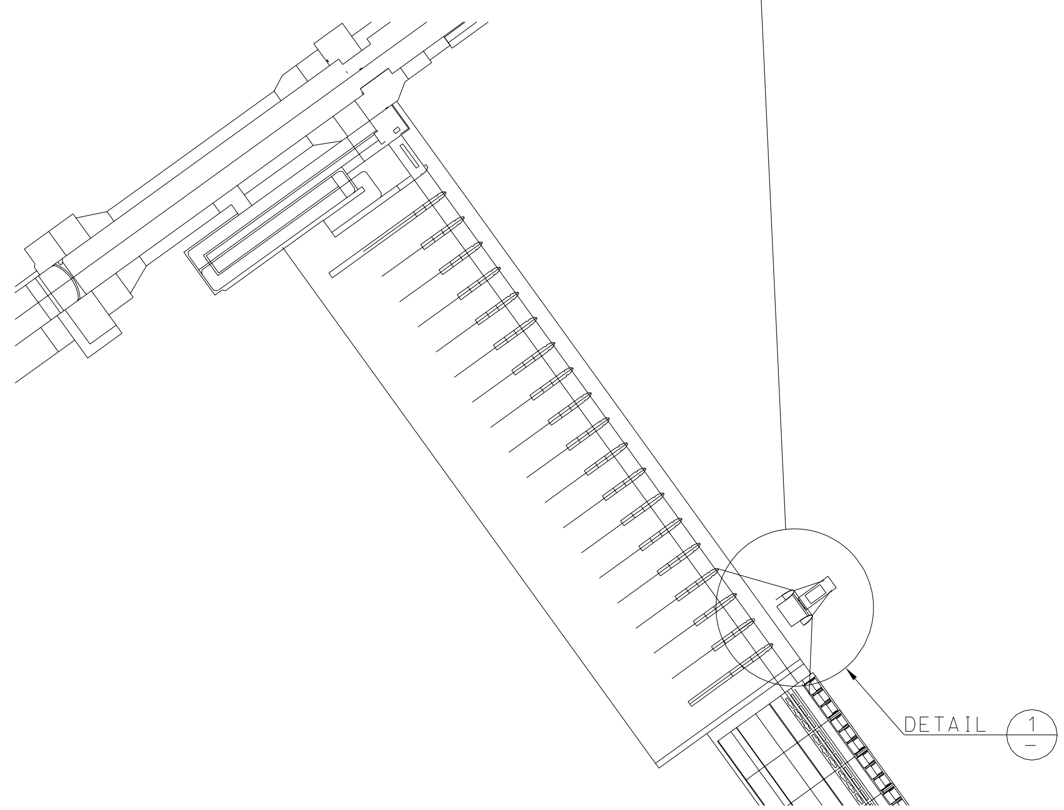
DRAWING STATUS:

DRAWING NO.

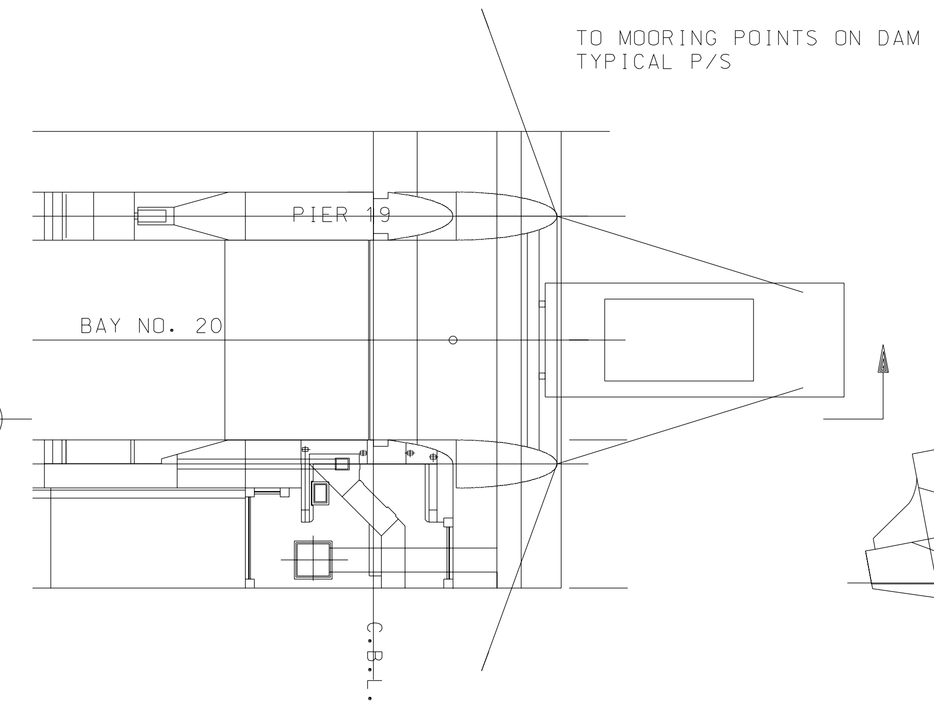
PLATE
16



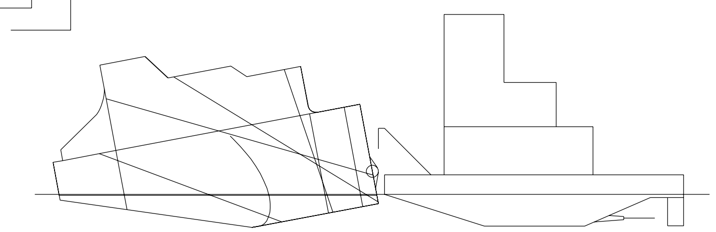
DETAIL 1
RSW & TUG
APPROACHING SPILLWAY



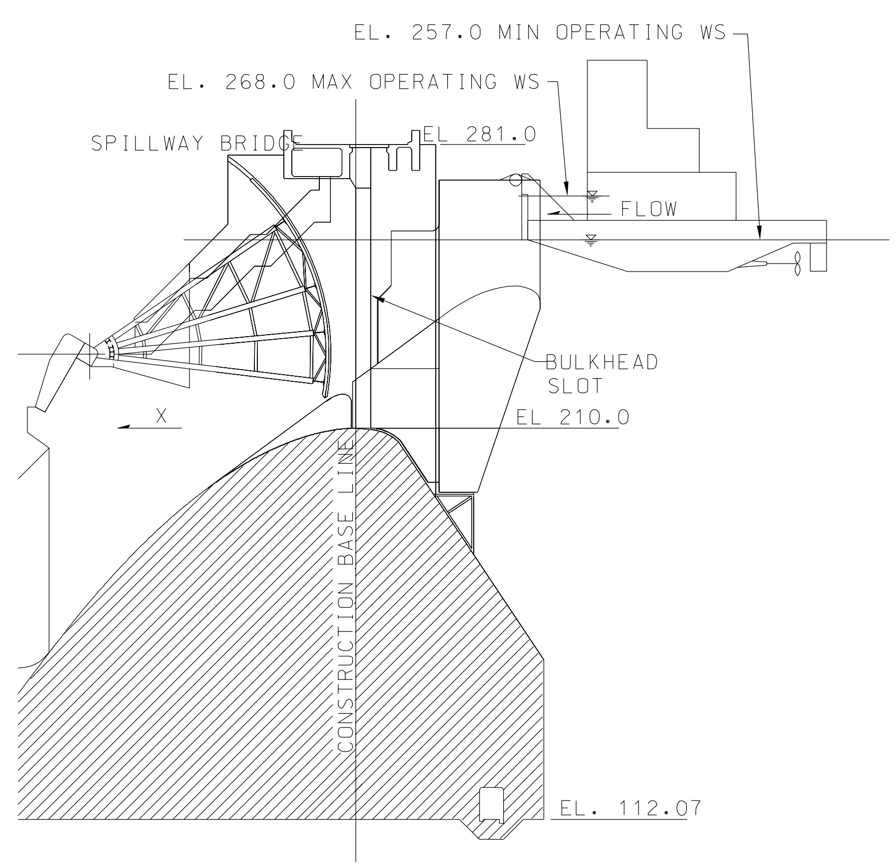
SITE PLAN



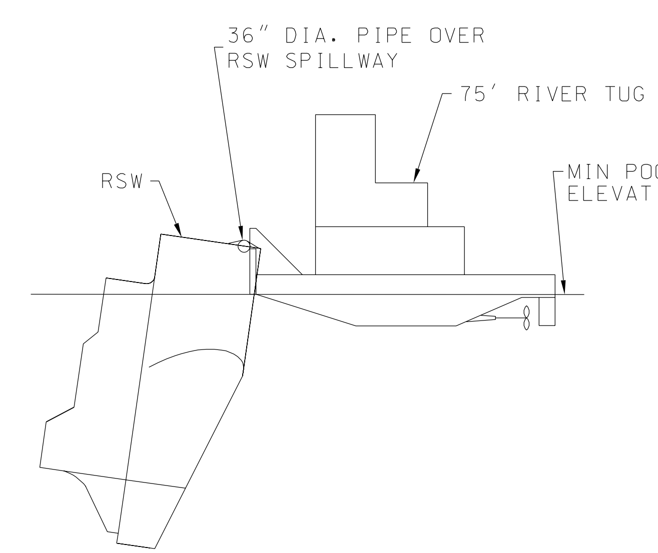
DETAIL 2
SITE PLAN
WITH RSW MAIN STRUCTURE IN PLACE



RSW & TUG
TRANSPORT ARRANGEMENT



SECTION A
SITE
ELEVATION
WITH RSW MAIN STRUCTURE IN PLACE



RSW & TUG
INSTALLATION/REMOVAL ARRANGEMENT

US Army Corps
of Engineers
Portland District

Revision | Date | Description | By

Designed by: WHITE
Date: OCT 2001
Drawn by: KUTZ
CADD File Name: JDF00101TG04.DGN
Checked by: SPRINGER
Technical Manager: MATTHEW HANSON
Submitted by: DONALD R. CHAMBERS, P.E.
Chief, Structural & Arch. Design

CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON

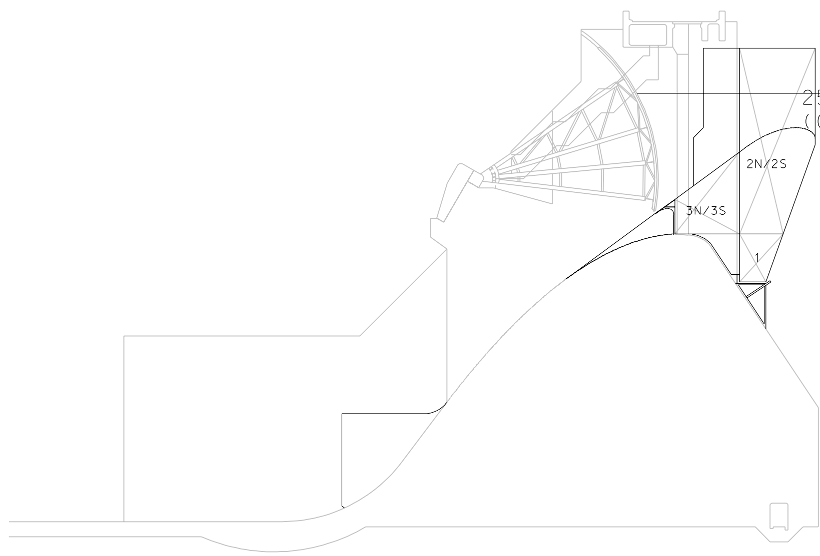
COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT
RSW MAIN STRUCTURE
INSTALLATION OPERATIONS

DRAWING STATUS:

DRAWING NO.

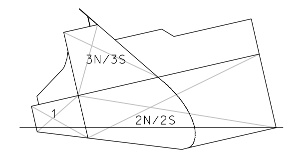
PLATE
17

INSTALLATION PROCEDURE

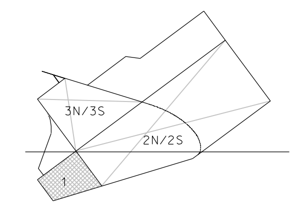


257' POOL INSTALLATION
(GM = 0.90 FT)

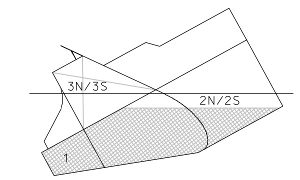
TRANSPORT POSITION:



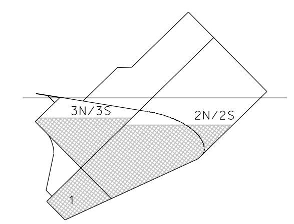
STEP 1 - FLOOD TANK 1



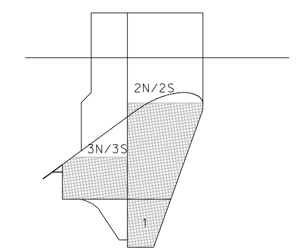
STEP 2 - FILL TANK 2 73%



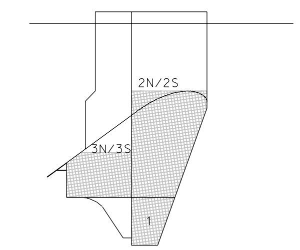
STEP 3 - FILL TANK 3 TO RIGHT RSW



STEP 4 - FILL TANKS 2 & 3 TO ADJUST DRAFT

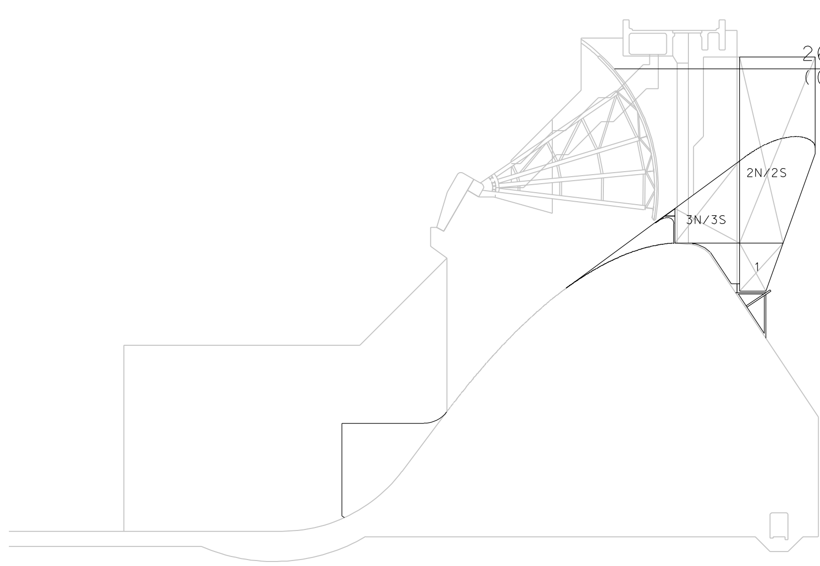


257' POOL INSTALLATION
(GM = 0.90 FT)



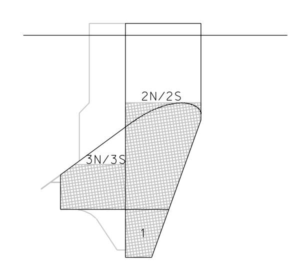
268' POOL INSTALLATION
(GM = 2.47 FT)

INSTALLATION PROCEDURE

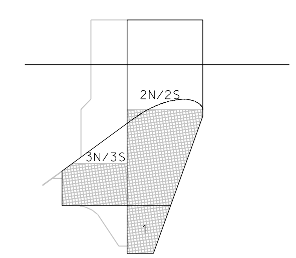


268' POOL INSTALLATION
(GM = 2.47 FT)

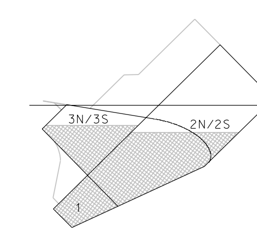
INSTALLED POSITION
268' POOL ELEVATION



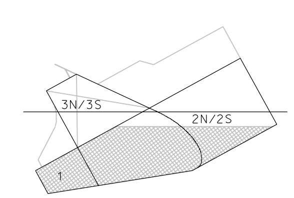
STEP 1 - BLOW TANKS 2&3
TO FLOAT RSW



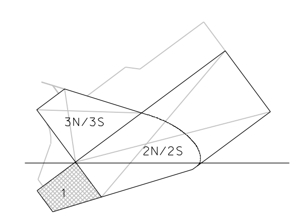
STEP 2 - BLOW TANK 3
TO ROTATE RSW



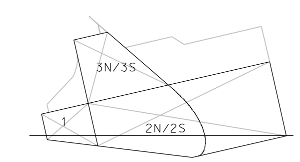
STEP 3 - BLOW TANK 2



STEP 4 - BLOW TANK 1



TRANSPORT POSITION:



REMOVAL PROCEDURE

NOTE: GM = METACENTRIC HEIGHT, A MEASURE OF STABILITY

US Army Corps of Engineers
Portland District

Revision	Description	Date	By

Designed by: DAI
Drawn by: KUTZ
Checked by: DAI
Submitted by: DONALD R. CHAMBERS, P.E. Chief, Structural & Arch. Design
Date: OCT 2001
CADD File Name: JDF0010I8G04.DGN
Technical Manager: MATTHEW HANSON

CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON

COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
DESIGN DOCUMENT REPORT
RSW MAIN STRUCTURE
BALLASTING PROCEDURES

DRAWING STATUS:

DRAWING NO.

PLATE
18

APPENDIX A

Technical Review Documentation

CENWD-NP-ET-E (BRASSFIELD) Create Date –21 Dec 00

MEMORANDUM FOR: Pat Jones CENWP-EC-DX Print date 28-Sep-01

Revised 22 Dec 00

SUBJECT: ITR Cost Estimate for 90% DDR for John Day Dam Bay 20 Removable Spillway Weir

1. Major recommended changes to the estimate – Estimated increase in total cost from $9.4 million to $13.4 million. ***A/E Response: Estimate is now $14.3 million w/o accounts 30 and 31 (Construction Management and Engineering & Design).***

.1 Add cost for 30 and 31 accounts ***A/E Response: Added $866,000 for SIOH (Construction Management – Account 31; did not add dollars for account 30 (Engineering & Design). Usually the Corps estimator adds these costs to an Excel cost summary sheet.***

.2 Revise contingencies to: 15% on the fabrication cost of the RSW and flow deflector bulkhead. Use 25% contingency on the remainder of the work. ***A/E Response: Calculated what 15% contingency on the RSW fabrication and 25% on the rest of the work added up to. It came out to 19.2% overall on everything. Used 19.2% contingency on the overall project. (This was done in the next estimate, which was done for the 60% plans and specs. However, for the final DDR, which reflects the estimate for the 90% plans and specs submittal, a 15% contingency was used.)0***

.3 Change RSW fabrication cost from $2.50/LB to $3.25/LB. NWW had a cost of $3.80/LB in their Lower Granite RSW which bid a month or so ago. They had an accelerated manufacturing schedule so I lowered cost to $3.25. ***A/E Response: Talked to the low bidder for the Lower Granite Project. They suggested $3.50 /LB would be a good cost for the RSW fabrication. I used $3.50/LB in the estimate.***

2. Minor cost revisions:

.1 Prime OH - Change project manger time to 2 months; delete assist PM; delete one field engineer and revise other to 2 months; increase survey crew to 30 days; delete secretary; increase toilets to 20 months; revise total pickup time to 20 months (two pickups full time); delete continuous cleanup; delete payroll clerks. Add stairs to access tail section installation; add fork lift full time; add lights for night work. ***A/E Response: I added all of these changes to the cost estimate.***

.2 Mob cost of 150 ton barge crane looks too high ***A/E Response: Reduced Mobilization of the barge crane to $75,000.***

.3 tail piece – change fab cost to $3.25 (see cmt 1.3 above); double stop log installation time to allow for removal and check to see who hauls stoplogs – gov't or contractor; add cost for tail piece to spw connection – see detail 2/12 on plate 13; use UPB item 02071 0020 with revised production rate of 7 CF/HR for chipping concrete; use UPB item 02069 0825 for cutting concrete on spy weir – too much slope to use slab saw; add core drilling for expansion anchor – UPB item 02011 4400 with output of 1.6; install anchor plate revise crew rate to 1 plate per hour; install tail piece – piece will be in several pieces to get items through gate slot so revise installation time and add field welding; ***A/E Response: Made all of the above changes, except used $3.50/LB for fabrication costs.***

.4 main structure support – change fab cost to $3.25/LB; anchor bolt installation add 3 days of crew time to allow for layout of holes; use rate for 5 man diving crew of $6000/day; ***A/E Response: Used $3.50/LB for fabrication cost and added all of the other changes described in this comment.***

.5 FAB RSW – check Goldsten cost for piping their labor cost for pipe installation is way to high looks like total cost should be around $60k rather than $100K; change fab cost to $3.25/LB; ***A/E Response: Agree. Made these changes, but used $3.50/LB for fabrication costs.***

.6 Install RSW – add diving crew to installation to adjust shims and check for proper seating on support. ***A/E Response: Agree. Made these changes.***

3. A&E SOW for production of P&S:

.1 Make sure A&E performs ITR of their cost estimates

.2 Make sure A&E knows proper format for cost estimate

.3 Make sure A&E signs statement of confidentiality when they prepare the government estimate for bid opening.

.4 Develop bid schedule at 30% P&S so 60% cost estimate format is correct.

A/E Response: Please provide statement of confidentiality for signature. Please inform A/E of any special formats required for this estimate. A bid schedule was developed for the 60% P&S.

4. Update to current labor rates.
A/E Response: Revised estimate to reflect current labor rates.
5. Write-up states using NAT97 equipment rates. Should be using NAT99 Region 8 rates. ***A/E Response: Used NAT99 Region 8 rates for this estimate.***

6. For escalation use the Civil Works Construction Cost Index System (CWCCIS) rather than Military factor. ***A/E Response: Used the CWCCIS for escalation for this estimate.***

7. Need to change the direct cost columns of the estimate. Delete the "other" column and replace it with a "Unit Cost" column. Use the unit cost column for subcontract cost items like the diving crew at $6000 per day. This column does not receive any sales tax markup. See the Gold user manual page 5-14 subcontractor (this has been re-named unit cost. The other reason for using the unit cost column is that the UPB uses this column for some costs. If you use an UPB item with cost in this column the estimate must also have the unit cost column otherwise the cost is not added to the estimate.

A/E Response: Revised the "Other" column as per the Corps comment to reflect a "Unit Cost" column that was renamed "Subcontractor". Used this column for items like the $6,000 per day diving crew.

Wallace W. Brassfield, P.E.,
Certified Cost Engineer

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REVIEW COMMENTS

PROJECT: LOCATION:

☒ CENWP-EC-HD ☐ Air Force ☐ Army ☐	Date: 28-Sep-01 Reviewer: **C.Goodell** Phone: (503) 808-4896	Design Document ☒ D.Memo ☐ Concept ☐ P & S ☐ Prelim. ☐ ☐ Final	☐ Bio/Env. ☐ Civ./San. ☒ Hydraulic ☐ Struct.

Item No.	Drawing Sht. Spec. Para.	COMMENTS	Action taken on comment by: REVIEW CONFERENCE A - Comment Accepted W - Comment Withdrawn (If neither, explain)	DESIGN OFFICE C - Correction made (If not, explain)	Back check by: (INITIALS)
1	Page 3-1	Design Criteria. Provide basis behind these critieria. For example, why 22.5 ft for the head on the crest at max operating pool?	A.	The following discussion (or some version thereof) will be added to the DDR text: The original intent of the study was to approximate the unit discharge of the SBS. Preliminary calculations showed that, for normal operating pool elevation 268 ft, the RSW crest would have to be at about 22.5 ft of submergence to accomplish this. This 22.5 ft figure was presented to the agencies during the 10% meeting, and they adopted it as a design criteria. More accurate calculations later, combined with model measurements, showed that the unit discharge would be slightly different than that for the SBS. However, the 22.5 ft criteria had already been established, and model results showed promising results, thus the 22.5 ft criteria was retained permanently.	

2	Page 3-1 2nd para	"…to attempt to approximate the proposed discharge through the Skeleton Bay Surface Bypass Spillway (SBS) with the RSW in order to develop a similar attraction flow…" My understanding was that unit discharge similarity, not overall discharge, was achieved. (14000 cfs/50 ft vs 18900 cfs/63 ft). If that is indeed the case, why???	A.	See response to Comment #1 above. Text of the DDR will be changed to clarify that the goal was to try to develop the same unit discharge as the SBS. At that time, it was believed that by matching the unit discharge, the same surface withdrawal characteristics could be produced. Since that time, we have found that the surface withdrawal characteristics are not as sensitive as once thought to the unit discharge of the RSW.	
3	Page 3-7	Section 3.7. I'm a bit confused about the difference between Proof of Concept Alternatives and Optimum Alternatives. Can you provide a brief explanation here?	A.	Briefly, the POC alternatives would at least emulate the SB performance. Optimum alternatives would improve upon the SB performance.	

4	Page 3-9 Sect. 3.8.1 Para 1	Would it not be better to design the RSW ogee crest for normal pool 265 as opposed to maximum operating pool 268? In my understanding, the pool rarely exceeds 265 during a normal year. This would provide better spillway discharge efficiency over the most prevalent pool elevations. Could still design air supply to the crest for pool 268 to account for the increased cavitation risk if this change is implemented.	A.	Agree that the suggested design could increase discharge capacity over that of the DDR design. The under-design when operating at PE 268 would only be about 15% and is not expected to present a cavitation risk. However, the increase in discharge capacity with the suggested design revisiion would only be about 2% (about 350 cfs). I do not believe this small increase to be significant enough to warrant a re-design at this late date. In addition, model tests showed that surface withdrawal characteristics were not as sensitive to unit discharge as originally thought. On the other hand, downstream considerations of flow down the spillway chute became more of a concern, Therefore, as the design evolved the importance of maximizing unit discharge decreased.	
5	Para 2	"The crest section is designed…" "crest section" should be replaced with "RSW".	A	Text will be revised accordingly.	
6	Para 3	The computed cavitation index of 0.7 is indicated to be "sufficiently high." Sufficiently high compared to what? The critical cavitation index? If so, what value for the critical cavitation index are you using in this case, and why?	A	Text will be clarified. Per Plate 2-6, EM 1110-2-1603, incipient cavitation exists at an abrupt into the flow offset of about 1/16-inch for a cavitation index of 0.7	

7	Page 3-10 Para 1	The ogee equation here is used under what criteria? The tailpiece alone is a somewhat unconventional spillway design. I'm wondering if this was accounted for when designing the ogee shape. I don't suppose the standard crest design can be used here. What do you think?	A	Agree that the operating conditions at free flow on this small of section are anything but "standard". Due to the extremely large operating head to design head achievable in the available space on this small section, I doubt that any satisfactory design can be achieved for free flow under the conditions existing on this section. Design work attempted to optimize the RSW crest and the tailpiece section shape, given that the goal was to locate the RSW crest as close to the existing CBL as possible, and yet minimize the elevation of the tailpiece section so that the tainter gate and bulkheads could still be used as originally designed. Based on physical model tests, the design has been revised and the District policy will be to not operate with the tailpiece alone in place. DDR text will be clarified.	
8	Page 3-12 Para 2	"The existing spillway bays are capable of passing up to 6500 cfs per bay…without exceeding TDG limits…" Where does this come from? Provide reference. Mike Schneider may have some input to this.	A	Wording was based on data availbe at the time. More recent field studies have been accomplished by the COE and that information will be included in the final DDR. Text will be revised as appropriate.	

9	Sect. 3.9	Again, what constitutes an “Optimum design?”	A	“Optimum design” is not uniquely quantified, but rather included things such as (1) increased discharge efficiency, (2) improved hydraulic characteristics around piers, down face, etc., (3) improved forebay draw and tailrace egress conditions, etc.	
10	Para 2	The skeleton bay deflector elevations are 157.0 and 160.0.	A	Noted.	

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
[ ] DESIGN MEMO [ ] CONCEPT [ ] FINAL [ ] PLANS & SPECS [ ] PRELIMINARY			19-Jun-01

REVIEWER			
[ ] CENWP-PE- [ ] AIR FORCE [X] ARMY	NAME: Matthew D. Hanson PHONE NUMBER: (503) 808-4934	[ ] ARCHITECT [ ] LAND ARCHITECT [ ] CIVIL [ ] SANITARY	[ ] MECHANICAL [ ] ELECTRICAL [ ] STRUCTURAL [X] Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	ACTION TAKEN ON COMMENT: REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	Cover	The cover will need to be revised to match the COE standard. I will provide it to MW for the final printing. Also, the DDR will be number 53 to follow the FDM sequencing		We will change cover based on the one you will send to us.	
2.	Inside cover	Change DDR title to include #53. When the final copy is produced, the technical review certification sheet will be included		We will make this change.	
3.	ES 1.1	Revise the last two sentences in the second paragraph to read: "The primary goal for fish passage at each project is to provide a minimum of 80% Fish Passage Efficiency (FPE) with 95% minimum survival. It has been suggested that a Surface Bypass passage route could either increase efficiency of the spill program or enhance the screened bypass system. Also, with the emphasis on increased spill, the region is subjected to increased total dissolved gas in the river and the impacts to lost power production are being realized."		This will be done.	
4.	ES, Pg 1-2	Fifth line change "rests" to "rested" Fourth para, change "beginning design of an" to "constructing a" Fifth Para, modify it to reflect the following thinking: remove references to the affect of the flow deflector on the fishway entrance. Bay 1 and the fishway entrance has no relation to the design at bay 20. State that the extended deflector was investigated to enhance deflector performance and was installed in the skeleton bay. An extended deflector is being investigated in spillway bay 20 to test the skeleton bay concept and to test the effectiveness of an extended deflector.		These modifications have been made.	
5.	ES page 1-4	Top para. It is not necessarily true that the bulkhead would be the same regardless of the deflector design. If the deflector were short (say 12.5 feet) the bulkhead would not need to be as deep and a contractor likely would opt for a smaller bulkhead. Please revise the sentence to reflect that a dewatering bulkhead was designed for the longest flow deflector design.		This change has been made.	

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
[ ] DESIGN MEMO	[ ] CONCEPT	[ ] FINAL			19-Jun-01
[ ] PLANS & SPECS	[ ] PRELIMINARY				

REVIEWER				ACTION TAKEN ON COMMENT
[ ] CENWP-PE-	NAME: Matthew D. Hanson	[ ] ARCHITECT	[ ] MECHANICAL	
[ ] AIR FORCE		[ ] LAND ARCHITECT	[ ] ELECTRICAL	
[X] ARMY	PHONE NUMBER: (503) 808-4934	[ ] CIVIL	[ ] STRUCTURAL	
[ ]		[ ] SANITARY	[X] Technical Review	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
6.	Section 3	I discussed with Dennis D. and my primary comment on this section is that it needs to reflect the final design only. The alternatives can be mentioned such as in Paragraph 3.6, but the information in the main body of the text should only discuss the final design, and should only show information related to the final design such as velocities, water surface profiles, etc. The information presented is good, but the alternative process was confusing and does not assist the reader in the information he needs on the final design. All the alternative discussion needs to be moved to the Appendix D. The new appendix D will contain the Model Alternative report, an Alternative development discussion (this will contain primarily section 3.7 including figures, , and all the information from the Tables 3-1,2,3 and figures that pertain to all the alternatives that are not the final geometry) and the Optimum RSW Alternative Discussion (Section 3.9 and associated tables and figures) There does not need to be any discussion of the Optimum RSW in the main body of the DDR. Dennis and I agreed that this change and the fact that the hydraulic model report may make it necessary to remove the Appendix D and E and put them in a separate Appendices volume to the DDR..		DDR structure will be revised as suggested.	
7.	Section 3	My 60% comment #10 was only partially complied with. In the final DDR in the main text, I would like to see the stationing for all the figures and tables **negative upstream** of the CBL and **Positive downstream.** In the tables in the appendix, the stationing can be the way it is now.		DDR will be revised as suggested.	
8.	3.8.1	Bottom paragraph. In the middle of the paragraph it is stated that the downstream toe has not yet been determined, and on Plate 13, the thickness is 5/8". This detail and statement must be reconciled.		Text will be revised to clarify the final design.	
9.	3.8.3 page 3-10 top para	Before the end of the second sentence revise "…gate seat on the face of the spillway." To "……gate seat on the face of the spillway to allow installation and removal."		Text will be revised as suggested.	
10.	3..8.2, page 3-11	Second para change "warranted" to "beneficial", also remove "with" on the 10^{th} line of the same para.		Text will be revised as suggested. Discussion of surging will be clarified in the DDR. Surging has been observed in	

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
DESIGN MEMO / CONCEPT / FINAL / PLANS & SPECS / PRELIMINARY			19-Jun-01

REVIEWER			
CENWP-PE- / AIR FORCE / X ARMY	NAME: Matthew D. Hanson; PHONE NUMBER: (503) 808-4934	ARCHITECT / LAND ARCHITECT / CIVIL / SANITARY	MECHANICAL / ELECTRICAL / STRUCTURAL / X Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
		Third para: state where the surging of the water surface upstream from the tainter gate occurred. Was it in the model in the prototype, the John Day model? Also, what is the cause of the surging and what are the effects of the surging: will it be a fish issue, or is there a problem?		various models (not John Day). Function of bay geometry and discharge. Surging would be a structural loading issue, not a fish issue.	
11.	3.8.3, page 3-12	3rd para; revise the working on the 100,000cfs statement. State that the RSW will likely not be operated at higher spillway flow ranges. This is because the efficiency of the RSW will decrease as more flow is passed through the spillway.		DDR text will be revised to clarify.	
12.	3.9	Move this section to the appendix as stated in comment 6		Will do for final DDR.	
13.	3.9.1	Page 3-14, top paragraph, third line from the bottom: state what kind of high strength material will be used? cementitious? Steel? UHMW?		Steel, the final design will be clarified in the DDR text. At the time of the 90% draft this detail was not yet finalized.	
14.	Table 3-1,2,3	See comment no 7		Will do for final DDR.	
15.	Page 3-24	Top paragraph: coordinate this statement with the resolution of comment 11 above		Will do for final DDR.	
16.	3.13	I could not find where the information on the curvature of the upstream nose of the tailpiece was discussed and where it came from. Please add a statement regarding its geometry. If we are not going to operate the bay without the RSW, there is no reason to have the piece rounded to make it better for flow. This also needs to be stated.		The Tailpiece design has been revised to eliminate both the upstream and downstream curved sections. Wording in the final DDR will be revised to incorporate the final design triangular shaped tailpiece section	
17.	Page 4-6	The statement right above P4.4, "…it appear that…" should be "…it appears that…"		The change has been made.	

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(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL			19-Jun-01
PLANS & SPECS	PRELIMINARY				

REVIEWER				ACTION TAKEN ON COMMENT		
CENWP-PE-	NAME: Matthew D. Hanson	ARCHITECT	MECHANICAL	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
AIR FORCE		LAND ARCHITECT	ELECTRICAL	(A = Comment accepted)	(C = Correction made. List drawing or paragraph number whwere correction made)	
X ARMY	PHONE NUMBER: (503) 808-4934	CIVIL	STRUCTURAL	(If not accepted explain)	(If not corrected, explain)	(Initials)
		SANITARY	X Technical Review			

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
18.	Page 4-8	Second para from bottom, define “free-flooded”. This is a term not normally used by the COE		A sentence has been added to explain this.	
19.	Page 4-10	4th line from the bottom, “preformed” should be “performed”		The change has been made.	
20.	Page 4-14	State the weight of the tainter gate and the weight of the gate resting on the tailpiece Have dynamic frequency loading been evaluated with respect to the tailpiece cover plate? Is there a maximum span that the plate should be subjected to?		The reaction of the tainter gate on the tailpiece is 250 kips over the 50-foot width. This information will be added to the report. The model testing showed only slight variations in loads, which should not create a dynamic problem. The dynamic loads varied over a head of about 11 feet This was judged to be well within the capability of the structure. The plate spans on the tailpiece are less than on the main structure.	
21.	Page 4-16	The curved closure plate is not discussed and how it is attached and designed. Refer to reasons in the hydraulic section as to why it is curved		The shape on the upstream face of the tailpiece has been changed to a sharp edge verses the rounded edge. The hydraulic testing showed that the spillway can’t be operated with only the tailpiece in place.	
22.	Figure7-1	Add a bullet to the schedule that the DDR is to be complete prior to the final approval of Plans and specifications		The change has been made.	
23.	Page 9-2	State the painting assumptions used in the cost estimate		The painting of the RSW is included in the price per pound of steel. The painting assumptions will be included in Section 9 for the final DDR.	

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(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL			19-Jun-01
PLANS & SPECS	PRELIMINARY				

REVIEWER				ACTION TAKEN ON COMMENT		
CENWP-PE-	NAME: Matthew D. Hanson	ARCHITECT	MECHANICAL	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
AIR FORCE		LAND ARCHITECT	ELECTRICAL			
X ARMY	PHONE NUMBER: (503) 808-4934	CIVIL	STRUCTURAL			
		SANITARY	X Technical Review			

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
24.	General	Are there any concerns or any recommendations on testing the RSW, and after a short period of time inspecting it for potential fatigue or flow damage? Should this be recommended after the season, to have it removed for inspection? My primary concern is for areas of flow where the water may be causing load reversals due to vibration, or surging, or anything. The particular areas of concern are the tailpiece toe (cavitation damage), tailpiece welding at the toe and the head (load reversals due to flow), and for the support structure of the tailpiece (load reversals due to flow)		The RSW Main Structure will remain in p[lace throughout the testing period. To inspect inside of the RSW, it would have to be removed and brought to the transport position for access to the inside. At this time there is no plan to remove it for testing. The Tailpiece would be less likely to sustain fatigue problems. It could be inspected by providing an access hatch on its front side. Dewatering stop logs would be inserted after the Main Structure is removed.	
25.	Plate 2	On the plan, show in dashed lines, the location of the sea0 ling surface		The change has been made.	
26.	Plate 5	Add the dimension of the RSW width between the piers		The change has been made.	
27.	Plate 10 and 11	I am concerned about the integrity of the seal and the potential cost and special fabrication of the seal required for installation. My fears may be alleviated if you provide prior applications, and manufacturer information (a website or some catalog cuts) If the seal has a hole in it, there will be no seal against headwater. You stated in the meeting that the reason a bulb seal was not used, was due to the concern about sealing against concrete. If that is so, why do you use a bulb seal in section A Plate 11? I think a crush seal is a better design for this application that was you have headwater working against a plate that forces the seal to seal. Other than that, a bulb seal is still preferable to an inflatable seal. Section A should include the bearing blocks and shims		The seal design will be further investigated in the Plans & Specifications effort. (The sealing arrangement has been subsequently changed to a J seal.)	

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
[ ] DESIGN MEMO [ ] CONCEPT [ ] FINAL [ ] PLANS & SPECS [ ] PRELIMINARY			19-Jun-01

REVIEWER		
[ ] CENWP-PE- [ ] AIR FORCE [X] ARMY	NAME Matthew D. Hanson PHONE NUMBER (503) 808-4934	[ ] ARCHITECT [ ] MECHANICAL [ ] LAND ARCHITECT [ ] ELECTRICAL [ ] CIVIL [ ] STRUCTURAL [ ] SANITARY [X] Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
28.	Plate 13	There is concern over the plate span of about 4 feet and the potential for vibration of the closure plate. Also, how is the tee section at the midpoint of the closure plate attached to the rounded surface. I am questioning whether we need the closure plate or not. I am also concerned about bolting both the top and bottom for the closure plate		In the final design, the closure plate only spans 6 inches and is part of the Main Structure. This makes the structure much stiffer in this area. In addition, vibration sensors will be placed on the structure to monitor vibrations.	
29.	Page D-1	Footer correct "Sudmittal", next page correct TADLE		The change has been made.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

PAGE OF

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO ☐ CONCEPT ☐ FINAL ☐ PLANS & SPECS ☐ PRELIMINARY ☐	John Day Surface Bypass Removable Spillway Weir 90% DDR		09/28/01

REVIEWER			
☐ NPW-EC-D ☐ AIR FORCE ☐ ARMY	NAME David Illias PHONE NUMBER X4901	☐ ARCHITECT ☐ LAND ARCHITECT ☐ CIVIL ☐ SANITARY	☐ MECHANICAL ☐ ELECTRICAL ☐ STRUCTURAL

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	ACTION TAKEN ON COMMENT: REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	Paragraph 1.1	The executive summary should be a separate section after the cover page and not part of the introduction.		The Executive Summary will be moved ahead of the Table of Contents and will immediately follow the title page.	
2.	Paragraph 1.1	In the introduction it states that additional structural analysis and design will be completed on the main structure and tailpiece. This is in conflict with paragraph 4.1 which states that the structural analysis for the DDR has been completed. If the structural analysis is not complete state specifically what analysis still need to be done.		Paragraph 4.1 has been changed to read that "a structural analysis was performed" and not completed. It will be completed during development of Plans & Specs.	
3.	Paragraph 4.3.1	Load cases should include wind, and ice.		The stability calculations were a comparison of the original stability calculated in DM 16 and the stability with the RSW in position. There was no ice load case evaluated in DM 16. In DM 16 the only load case with wind was the construction load case. Therefore, these two factors were not included in the comparative stability analysis described in Section 4.3..	
4.	Section 4	Load forces should include impact forces on the tailpiece when spillway bay is open and the main structure is removed. Possible impact loads should include logs and ice. The tailpiece anchorage should be checked for impact loads.		The spillway will not be operational with only the tailpiece in place. This was decided during subsequent hydraulic modeling with only the tailpiece in place.	
5.	Paragraph 4.1	In the second sentence of the second paragraph, remove "to last as long as the dam, which is at least". Dam structures are typically designed for at least 100-year life.		"…which is at least 50 years." has been deleted.	
6.	Structural Design	The RSW will lower the hydraulic gradient along the existing piers and increase the differential loading along the piers. The piers need to be checked for this loading condition.		The hydrostatic differential due to spill drawdown along the piers was ignored in the FEM as a simplifying assumption. This assumption is reasonable considering the maximum drawdown along the pier is about 15 feet. This small differential pressure over a relatively small area has a negligible impact on global structure.	
7.	Paragraph 8.1.3	Need to note whether the support frame will be left in place after testing.		If you are talking about the "template frame", it would be removed after drilling and installing	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

PAGE OF

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE 09/28/01
X DESIGN MEMO / CONCEPT / FINAL PLANS & SPECS / PRELIMINARY	John Day Surface Bypass Removable Spillway Weir 90% DDR		

REVIEWER			
NPD-EC-D AIR FORCE ARMY	NAME David Illias PHONE NUMBER X4901	ARCHITECT LAND ARCHITECT CIVIL SANITARY	MECHANICAL ELECTRICAL STRUCTURAL

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	ACTION TAKEN ON COMMENT: REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
				the anchors..	
8.	Paragraph 9.3.3	Fabrication may not necessarily take place in Portland.		For the cost estimate the location of fabrication must be assumed. The Portland area is the most likely location for fabrication.	
9.	Plate 10	Show the surface intersection line between the pier nose the spillway surface.		Plate 10 has been corrected.	
10.	Plate 11	Provide supplier information and specifications for the inflatable seals. These types of seals have not been used on our dam projects.		The inflatable seals have been abandoned in the Plans & Specs development.	
11.	Plate 12 and 13	The tailpiece and the lower end of the main structure will be subject to extreme dynamic loading during spill. Critical elements of these structures and connections need to be designed for potential vibration and fatigue loading. The 5/8 inch plate between the tailpiece and main structure, and the tailpiece transition to the existing spillway along with its connections need to be checked for cyclic loading. Instrumentation may need to be considered.		After the hydraulic study, it was decided that the spillway can't be operated with the tailpiece in place. Therefore, the upstream edge of the tailpiece has been changed so there is only 3 inches between the tailpiece and main structure. The closure plate spans the three inches and pressure transducer data from hydraulic testing showed that there is significant load variation.	
12.					

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE: DESIGN MEMO [ ] CONCEPT [ ] FINAL [ ] PLANS & SPECS [ ] PRELIMINARY [ ] x **DDR**

PROJECT: **John Day Removable Spillway Weir DDR**

90% ITR Review

LOCATION: John Day Project

DATE: 12/18/00

REVIEWER

X CH2M/MW JV; AIR FORCE; ARMY

NAME: See Item No. 1

PHONE NUMBER: 425 453-5000 (Pete Wiedemann)

ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY; MECHANICAL; ELECTRICAL; STRUCTURAL; Technical Review

ACTION TAKEN ON COMMENT

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	-----------	The reviewers are indicated by their initials as follows: PFW – Pete Wiedemann General/Mechanical Review RR – Dick Regan Hydraulic Review TPD – Tom Delaney Structural Review			---------
2.	Page iv, List of Figures	Number 3-6: typo in first line (“Pierce”)		The change has been made.	PFW
3.	Para 1.3	The 4th bullet talks about the development of five “optimum” RSW geometries; “optimum” being defined as “suitable”. Why not just call them “suitable RSW geometries” in the first place? “Suitable” is a better description than “optimum”.		This term was used in the scope of work and therefore, we believe that it should be kept in the report. This nomenclature is confusing. The RSW descriptions that were not selected will be placed in an appendix.	PFW
4.	Para 3.10	In the 4th line, “District” needs to be defined.		This has been defined.	PFW
5.	Para 5.1	Terminology needs to be consistent with the Plates. (e.g.: the second paragraph refers to “electric solenoids”, while Plate19 refers to them as “4-way, 3-position control valves.”Also the same paragraph refers to “flood valves” while Plate 19 refers to them as “water valves”.		Will clarify text and make terminology consistent. Plate 19 should read “Flood valves.” Also “support vessel” should change to “river tug” on Plate 19.	PFW
6.	Para 5.1	In the 3rd paragraph, “RSVD” needs to be defined.		RSVD = reserved. Will clarify.	PFW
7.	Para 5.1	The last paragraph refers to the “support vessel”, while Plate 20 calls it a “75’ River Tug.”		Will change support vessel to river tug in text.	PFW
8.	Plate 18	The schematic is difficult to follow. The legend indicates the dashed lines are to show hidden pipe, but they are also used to show the hidden structure.		The dashed piping lines have direction arrows. Hidden structure lines don’t. Will clarify.	PFW

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	
PLANS & SPECS	PRELIMINARY	x DDR	90% ITR Review		12/18/00

REVIEWER				ACTION TAKEN ON COMMENT
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	MECHANICAL	
AIR FORCE		LAND ARCHITECT	ELECTRICAL	
ARMY	PHONE NUMBER: 425 453-5000 (Pete Wiedemann)	CIVIL	STRUCTURAL	
		SANITARY	Technical Review	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
9.	Plate 18	The piping callouts "P/S" need to be defined.		P/S = Port/Starboard (Left/Right) Will clarify text.	PFW
10.	Plate 21	The elevation callout "GM" needs to be defined.		This will be defined on Plate 21. GM is the metacentric height which is a measure of the stability of a floating object.	PFW
11.	Pg. PD-4 10. Fish ladders	Both ladders are 1:10 slope, Regulation for pool fluctuation is with a vertical slot control section. The normal flow in both ladders is the same, and there are more diffusers on the south side than the north. There are no submerged orifice entrances. All this data must be checked.		These have been corrected.	RR
12.	Pg.3-8, 4th para.	Sentence near end states that the tailpiece extends upstream of the stoplog slots. The dwg. Show it extending downstream from the stoplog slots. This should be clarified.		Text will be clarified as appropriate.	RR
13.	Pg. 3-9 1st para.	This para. States that unacceptable hydraulic characteristics existed. The undesirable characteristics should be explained.		Information will be included in the final DDR.	RR
14.	Pg. 3-11 Section 3.8.3	This para. Discusses filling the location between the end of the tailpiece and the existing spillway concrete with a high strength material to form a smooth transition. I presume that this material would be some type of epoxy grout. The drawings show a 5-ft. long plate welded between the tailpiece and a beam embedded into the spillway crest. This should be coordinated. If a steel plate is used it must be anchored to the existing concrete at frequent intervals, and the space between the bottom of the plate and the existing concrete pressure grouted. If a plate is used there can not be any more than 1/16" offset at the downstream termination of the plate.		Final design eliminates the abrupt offset shown in the draft report. The final DDR text will be revised to describe the final design.	RR
15.	Pg. 3-24 Section 3.13 2nd para.	This para. states that the spillway gate is not subjected to rapidly varying or uneven pressures. This is not the case when the gate is being closed or opened. Also during the opening and closing of the gate the RSW is also subjected to rapidly varying or uneven pressures of a fairly large magnitude. Has this type of		The gate was opened and closed in the hydraulic model. Observations showed some sloshing between the gate and RSW. Final DDR text will be revised to	RR

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
DESIGN MEMO / PLANS & SPECS / CONCEPT / PRELIMINARY / FINAL / x DDR	John Day Removable Spillway Weir DDR 90% ITR Review	John Day Project	12/18/00

REVIEWER	NAME	PHONE NUMBER		
X CH2M/MW JV AIR FORCE ARMY	See Item No. 1	425 453-5000 (Pete Wiedemann)	ARCHITECT LAND ARCHITECT CIVIL SANITARY	MECHANICAL ELECTRICAL STRUCTURAL Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
		loading been considered in the design?		qualitatively describe potential dynamic loading during gate opening/closure.	
16.	Sect 4.2 Design Criteria	1. Wave loads are discussed as caseload 5 on pg. 4-11. The wind wave conditions, wave height, wave period, design wind velocity etc. should be discussed in this section. 2. There is no discussion of other hydrodynamic loads, such as that discussed in Comment No. 5, and the loads hydrodynamic loads generated by the flow over the RSW.		1. See 2nd Paragraph of load case 5 discussion. 2. Transducer results from the hydraulic modeling showed that the maximum difference between the maximum and minimum pressures on the RSW spillway was 7.5 feet. We judge that this variation is sufficiently low to not cause any structural or fatigue problems.	RR
17.	Pg. 4-12 Section 4.5.1.3	1. Should discuss the orientation of the RSW when in the towing position. 2. Should state that the weight of the RSW is 1,095,000 lbs. Is the Dry weight or the weight of the structure in position on the spillway, wet weight.?		We will add sentence to clarify. “Plate 21 illustrates the anticipated orientation of the RSW Main Structure for towing.” Pg. 4-13, first paragraph, dry weight	RR
18.	Pg. 4-16 Section 4.6	The 1st paragraph discusses items that will be discussed in the 90% report. Since this is the 90% report this paragraph should present these items.		The deflector geometry will be selected after modeling is complete. The text was changed to reflect this.	RR
19.	Plate 6 & 7	Define what the arrow heads indicate.		See key in lower right corner. Dashed line w/arrows = web frames.	RR
20.	Plate 11	Define what the circular line to the left of the seal indicates on both sections.		This indicates the maximum extension of the inflatable seal. This has been replaced with a j type seal.	RR
21.	Plate 13	1. Is the 5/8” closure plate sufficiently stiff to with stand the hydrodynamic loads when the RSW is operating? The future model studies of the tailpiece should make every effort to find a tailpiece design that would eliminate the tailpiece ogee		The closure plate detail has been changed. The plate only spans 6 inches in the final design. The ogee shape has	RR

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
DESIGN MEMO / CONCEPT / FINAL; PLANS & SPECS / PRELIMINARY / X DDR	John Day Removable Spillway Weir DDR — 90% ITR Review	John Day Project	12/18/00

REVIEWER				ACTION TAKEN ON COMMENT
X CH2M/MW JV; AIR FORCE; ARMY	NAME: See Item No. 1; PHONE NUMBER: 425 453-5000 (Pete Wiedemann)	ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY	MECHANICAL; ELECTRICAL; STRUCTURAL; Technical Review	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
		shape so that the RSW structure can but directly against the tailpiece with out a closure plate. 2. See Comment No. 14.		been eliminated. Reference to comment 14: The cover plate is welded to an embedded plate that is flush to the concrete surface. The cover plate is beveled to smooth the transition.	
22.	General	Is the RSW anchored to the existing spillway in any manner or is it just supported on the base support structure? This should be discussed in more detail.		More discussion will be added to the text in section 4.5.3.1 to explain the support of the Main Structure.	RR
23.	Plate 14	Attaching the support frame at exactly the correct elevation under about 60-ft of water will be extremely difficult if not impossible. Has consideration been given to providing mechanical jacks on either the support frame or the RSW structure to assist in leveling the RSW structure?		The intent of the design is to provide shims from the support frames to the proper elevation. This will allow the contractor the opportunity to compensate for misalignment of the frames.	RR
24.	4-1	In paragraph 4.2.1, add TM 5-809-10 for dynamic earthquake induced forces produced by water acting on walls (sloshing).		TM 5-809-10 is not applicable. EM 1110-2-2200 addresses water pressure on dams during earthquakes.	TPD
25.	4-2	In paragraph 4.2.8, the information that I have is that the OBE seismic event should be 6 percent of gravity.		This will be corrected in the final submittal.	TPD
26.	4-3	In paragraph 4.2.8, the information that I have is that the MCE seismic event should be 19 percent of gravity.		This will be corrected in the final submittal.	TPD
27.	4-3	In paragraph 4.3.1, change the OBE seismic event in Load Case 3 to 6 percent of gravity.		This will be corrected in the final submittal.	TPD
28.	4-3	In paragraph 4.3.1, change the MCE seismic event in Load Case 4 to 19 percent of gravity.		This will be corrected in the final submittal.	TPD

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT John Day Removable Spillway Weir DDR	LOCATION John Day Project	DATE
DESIGN MEMO	CONCEPT	FINAL	90% ITR Review		12/18/00
PLANS & SPECS	PRELIMINARY	X DDR			

REVIEWER				ACTION TAKEN ON COMMENT
X CH2M/MW JV AIR FORCE ARMY	NAME: See Item No. 1 PHONE NUMBER: 425 453-5000 (Pete Wiedemann)	ARCHITECT LAND ARCHITECT CIVIL SANITARY	MECHANICAL ELECTRICAL STRUCTURAL Technical Review	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
29.	General	The design of the RSW should take into account both earthquake induced forces caused by sloshing of water and from the structure itself in Load Cases 3 and 4.		Agree. The design accounts for earthquake induced water pressure on the RSW main structure in accordance with EM1110-2-2200.	TPD
30.	4-8	In paragraph 4.5.1, change the OBE seismic event in Load Case 3 to 6 percent of gravity.		This will be corrected in the final submittal.	TPD
31.	4-8	In paragraph 4.5.1, change the MCE seismic event in Load Case 4 to 19 percent of gravity.		This will be corrected in the final submittal.	TPD
32.	4-10	In Load Case 3 change the OBE seismic event to 0.06 “g”.		This will be corrected in the final submittal.	TPD
33.	Plate 15	How much tension is being relied on to develop shear friction in the deflector reinforcing? I don’t think you will be able to fully develop number eleven bars with grout.		The actual size and geometry of the deflector was not designated until after the 90% submittal. So calculations to answer this comment were not made. However, the calculations will be made and described in the final DDR.	TPD
34.	Plate 15	In the deflector reinforcing detail at the top of the deflector, a number eleven bar has a pretty large radius on a 180 degree hook. There may be some clearance problems with reinforcing in that area.		The actual size and geometry of the deflector was not designated until after the 90% submittal. The rebar clearances and anchoring design will be described in the final DDR.	TPD
35.	Plate 17	In section A there is a 3/8” plate shown on the bottom section of the bulkhead. Should this be a 5/8” plate?		Plate 17 will not be in the final submittal since the bulkhead has been replaced with a scheme employing the existing stop logs.	TPD

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	**John Day Removable Spillway Weir DDR**	John Day Project	
PLANS & SPECS	PRELIMINARY	X **DDR**	**90% ITR Review**		12/18/00

REVIEWER			
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	MECHANICAL
AIR FORCE		LAND ARCHITECT	ELECTRICAL
ARMY	PHONE NUMBER: 425 453-5000 (Pete Wiedemann)	CIVIL	STRUCTURAL
		SANITARY	Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
36.	Plate 17	The line work for the end of the bulkhead as shown in Detail 1 doesn't match the end of the bulkhead as shown in the Dewatering Bulkhead Plan Detail.		The dewatering bulkhead has been abandoned in the final design. The existing stop logs at the dam will be used in constructed slots for dewatering. Therefore, this plate will not be in the final submittal.	TPD
37.	Plate 17	On the Partial Elevation, where the 62'-0" dimension actually goes to should be clarified.		This plate will not be in the final submittal since the bulkhead has been replaced with a scheme employing the existing stop logs.	TPD
38.					

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
DESIGN MEMO / CONCEPT / FINAL / PLANS & SPECS / PRELIMINARY	90% Submittal for John Day RSW DDR		27 Nov 00

REVIEWER		
AIR FORCE / X ARMY	NAME: Diana Modini / PHONE NUMBER: 503-808-4868	ARCHITECT / LAND ARCHITECT / X CIVIL/Hydraulic / SANITARY / MECHANICAL / ELECTRICAL / STRUCTURAL / Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	P3-9, 3.8.1 Par 3	"The minimum computed cavitation index…". Can you add formulas used.		Text will be clarified to add CI equation	
2.	Figure 3-5	This figure shows the tailpiece and the low crest of the RSW. Is the drawing correct?		The initial POC design did include both sections.	
3.	P3-14, par 3.9.2	"Optional tasks in the scope of work must be exercised to continue with investigation of the Optimum RSW concept." Looking over Proof of Concept 5 and Optimum RSW alt C, it appears to be the same except a low crest versus the tailpiece. Is it true that the optional items are to be exercised due to the change in tailpiece? Please clarify paragraph? Or drawings?		Final text will be clarified regarding the "Optimum" design.	
4.	P3-15, par 3.10.1	Is Proof of Concept 5 formally called alternative 2 with fillet (but with out the low crest/tailpiece)? And is Optimum RSW alternative B what used to be called Simplified alt 7?		POC 5 has a slightly different geometry than your reference alternative 2 with fillet. Optimum Alt B is the same as an earlier described alternative 7.	
5.	P3-15, par 3.10.1	"…Proof of Concept geometry, Alt 5, currently is underway." We just went to WES and my understanding is that the optimum RSW alt C was built and modeled. In the previous paragraph, it states that Optimum RSW alt C is being built in the sectional model. Please clarify what was built for both models. Also is drawing (s) correct, in the general model the backside of the RSW was undercut. Please clarify.		Both the WES general model and the NHC sectional model are constructed with the same design. The final design being modeled includes an upstream face undercut for structural reasons. Final DDR text will be clarified.	
6.	General	There is some confusion between Proof of Concept and Optimum. Maybe to keep clear for others, add a paragraph (or the like) to explain the goals of the two categories. One suggestion or idea is to number/letter all designs, but state which are proof of concept and others are optimum and why.		This information is already included in paragraph 3.5	
7.	Figures 3-11 etc	Shows symbols for the three forebay elevations. Maybe clarify that in either the figure or in the written document.		Legend will be revised.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
[ ] DESIGN MEMO [ ] CONCEPT [ ] FINAL [ ] PLANS & SPECS [ ] PRELIMINARY	90% Submittal for John Day RSW DDR		27 Nov 00

REVIEWER

	NAME		
[ ] AIR FORCE	Diana Modini	[ ] ARCHITECT	[ ] MECHANICAL
[X] ARMY	PHONE NUMBER 503-808-4868	[ ] LAND ARCHITECT	[ ] ELECTRICAL
		[X] CIVIL/Hydraulic	[ ] STRUCTURAL
		[ ] SANITARY	[ ] Technical Review

ACTION TAKEN ON COMMENT

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
8.	P3-16, par 3.11.1	The paragraph states what was done, but no conclusions. What do these water surface profiles tell us? Please clarify.		Data was developed for documentation and design purposes and are considered to be self-explanatory as presented.	
9.	P3-21, par 3.11.2	The paragraph states what was done, but please include a conclusion of what the velocities are telling us about these alternatives.		Data was developed for documentation and design purposes and are considered to be self-explanatory as presented.	
10.	P3-21, par 3.11.3	If using Proof of Concept as POC, then use throughout out to be a little more consistent.		Terminology POC will be deleted.	
11.	P3-21, par 3.11.3	"…3.6 at low forebay elevations to about…" and on page 3-22 its states that "…3.6 at forebay elevation 257…" Is this essentially at repeat? If so maybe combine.		Will consider comment in final revision of DDR.	
12.					
13.					
14.					

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO / PLANS & SPECS; CONCEPT / PRELIMINARY; FINAL / 30%	John Day RSW 60% DDR		October 18, 00

REVIEWER			
CENWP- / AIR FORCE / ARMY	NAME: Matthew D. Hanson; PHONE NUMBER: 503-808-4934	ARCHITECT / LAND ARCHITECT / CIVIL / SANITARY	MECHANICAL / ELECTRICAL / STRUCTURAL

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	Executive Summary	Second Para, Last sentence is not true anymore, this needs to state that this was an initial guideline that through design development is not considered practical any longer	A	C. Revision made to text.	
2.	P3.4.1	2nd Para, 5th sentence, the existing spillway tainter gate is intended to be either open or closed. There has been no discussion of throttling, and no desire to include it with the RSW. The tailpiece however, will be tested with throttling, but it is likely it will never be operated that way due to undesirable flow characteristics. Also, the first sentence of this second paragraph should state that the NWW RSW design has different fish passage goals, which resulted in, different flow criteria, different project needs and different design considerations.	A.	C. Removed the sentence regarding gate control and added suggested language regarding the Lower Granite design.	
3.	P3.5	Last sentence. The 10% meeting, should be called the Site visit/ kickoff meeting, or alternative discussion, but not a 10% PRM	A.	C. Change made.	
4.	Fig 3-4	The original title I thought had “aeration step” in it.	A.	C. Added ‘aeration’ to figure caption.	
5.	P3.7.1	Inflatable bulb seals will not be allowed for sealing of the RSW. I understand that an inflatable seal would rely on the inflatability of the seal to hold back headwater which is not reliable over a sustained period of time. In any Corps structures that I have been involved in, the seals are either crush, bulb, or flap seals which use headwater to seal. If you have previous examples of Corps structures that allow an inflatable seal, I would be interested in seeing the details. Currently crush seals are a good solution to temporary structure sealing, and can be used without a seal plate embedded into the concrete, or any continuous supply of compressed air for inflation.	A.	C. Seal design is presently being developed. The word inflatable may be confusing. The seal envisioned in the hydraulic design is a bulb-type, actuated by water pressure from the reservoir. Whatever type of seal is selected for final design, actuation will be accomplish using either reservoir headwater or water pressure from an external source.	
6.	P3.7.1	Last paragraph: This paragraph should reference a figure, because the description is too complex. Split this paragraph to discuss the lower RSW crest issues separately	A.	C. This paragraph was rewritten to make it more clear.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO / PLANS & SPECS; CONCEPT / PRELIMINARY; FINAL / 30%	John Day RSW 60% DDR		October 18, 00

REVIEWER		
CENWP- / AIR FORCE / ARMY	NAME: Matthew D. Hanson; PHONE NUMBER: 503-808-4934	ARCHITECT / LAND ARCHITECT / CIVIL / SANITARY / MECHANICAL / ELECTRICAL / STRUCTURAL

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
7.	P3.7.3	Second sentence, remove the word “extensive”, and add that the pier shapes were designed using COE Hydraulic criteria.	N.A.	. The pier shape as originally designed was developed through extensive physical model testing. The HDC data were developed following these tests.	
8.	P3.8.1	Bottom P of page 3-10, second line: we have not used the term “velocity flux” It has been referred to as a flow acceleration criteria	A	C. Terminology “velocity flux” deleted.	
9.	Page 3-11	Second paragraph from bottom. The lower portion of this paragraph discusses filling something with “high strength epoxy grout” This should be removed from this paragraph because this is speculation. The hydraulic section should state that the section would be “permanently attached”, or “filled with concrete” or “designed to be structurally stable once in place” or “not easily removed”	A.	The proposed design requires a high strength cementatious material to smoothly transition between the RSW and the concrete of the existing spillway to eliminate cavitation potential. The term “high strength epoxy grout” will be replaced with “high strength cementatious material”. Statement that the RSW section will be designed to be structurally stable once in place has been added to the criteria.	
10.	Figure 3-11, Tables 3-1, 3-2	Figures 3-12 through 3-20 stationing needs to be referenced off of the CBL so that there is some consistent rationale to the stationing . Stationing should be negative upstream of the CBL and positive downstream. Once the alternatives are compared, a single stationing system based on the RSW Crest centerline could be established, but it may be easiest to use the CBL stationing throughout the DDR.	A	The 0.0 distance in all figures is the CBL (existing spillway axis) and will be clarified on the figures. The stationing on tables 3-1 through 3-4 have been revised to be consistent with the stationing on the figures, i.e., positive upstream of the CBL and negative d/s of the CBL..	
11.	P3.10.3	State the coefficients and the HDC charts used to complete the rating curves	A	C. Requested information added to text.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO; PLANS & SPECS; CONCEPT; PRELIMINARY; FINAL; **30%**	John Day RSW 60% DDR		October 18, 00

REVIEWER			
CENWP-; AIR FORCE; ARMY	NAME: Matthew D. Hanson; PHONE NUMBER: 503-808-4934	ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY	MECHANICAL; ELECTRICAL; STRUCTURAL

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
12.	P3.11	First Para. May not be true anymore. I think it best to delete it and start with the second para, or refer to the bio study plan (Which Blaine will write)	A	C. First paragraph discussing rapid and frequent removal deleted.	
13.	Page 3-21	Top para: Spill patterns are specifically designed to enhance **juvenile** fish passage, not adult. And the spill patterns are not necessarily “enforced” but coordinated with the NMFS	A	C. Revisions made as suggested.	
14.	P4.1	Discuss design life of the various systems, structures.	A	C. A paragraph is added to Section 4.1.	
15.	P4.2.3	Min pool is el 257	A	C. Elevation changed.	
16.	P4.2.8	The seismic coefficients for Dalles and John Day projects is 0.1g for the OBE and 0.19g for the MCE from Jim Griffiths in Geotech	A	C. These values are incoprorated.	
17.	P 4.4	First paragraph, second from last sentence uses “beyond” the tainter gate. Change this to “downstream” of the tainter gate Last paragraph on page 4-4, second sentence change to read: “Removing the RSW against up to 60 feet of head….”	A	C. Text changed as requested.	
18.	P4.5.1	Are any mis – filling load conditions considered? Is it likely, and should one be included in the design of the attachment? Last paragraph of this section on page 4-6: define the word “free flooded” Does this mean it is open to forebay level? Also note somewhere that the ports for filling will have to be screened for juvenile fish.	A	Various filling sequences have been investigated. Visual marks will be added to the outside of the RSW to indicate when a sequence step has been achieved. Yes. Screens can be added to the flood openings. However, for a once-per-year filling, it probably would not be required.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO / PLANS & SPECS; CONCEPT / PRELIMINARY; FINAL / **30%**	John Day RSW 60% DDR		October 18, 00

REVIEWER			
CENWP- / AIR FORCE / ARMY	NAME: Matthew D. Hanson PHONE NUMBER: 503-808-4934	ARCHITECT / LAND ARCHITECT / CIVIL / SANITARY	MECHANICAL / ELECTRICAL / STRUCTURAL

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
19.	Figure 4-1	Note below the figure 4-1 on page 4-7 that the calculated weight of the RSW is empty, out of water.	A	C. This will be clarified.	
20.	P4.5.2	Case 4: add “tainter gate resting on tailpiece”	A	C. Added.	
21.	P4.5.3.1	Top of page 4-9, explain or use a better term than “fixing” method of attachment?	A	C. Explanation has been reworded.	
22.	P4.5.1, 4.5.2	No discussion of seals was included in either design of the RSW or tailpiece. This should be explained in the 90%. Also note that the tailpiece must include a seal system, as it will resist headwater pressure before the RSW is installed.	A	C. A seal design and supporting text has been added for both the Main Structure and the Tailpiece.	
23.	Fig 5-1	Include text descriptions on this figure	A	C. The function of this figure was to illustrate the RSW geometry. It will be clarified	
24.	Section 6	Any instrumentation proposed? Any filling piezometers or indicators proposed or will it all be by air pressure?		Due to the change in attitude, there is no effective gauging system for the RSW tanks. Also, in our experience, gauging systems are major maintenance items. As noted earlier, we propose a system of external marks that will allow the operator to achieve the desired flooding sequence.	
25.	Page 8-1	Second to last sentence: explain what a “holdback” is	A	The holdbacks will be clarified. Steam boat ratchets and lashing chain are one of the options being considered.	
26.	Section 9	Costs should be in thousands. Split out flow deflector concrete and dewatering bulkhead Split our RSW fabrication, transportation and installation	A	C. These will be done.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE						PROJECT	LOCATION	DATE
X	DESIGN MEMO		CONCEPT		FINAL	John Day RSW 60% DDR		October 18, 00
	PLANS & SPECS		PRELIMINARY		30%			

REVIEWER							ACTION TAKEN ON COMMENT		
	CENWP-	NAME Matthew D. Hanson		ARCHITECT		MECHANICAL	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
	AIR FORCE			LAND ARCHITECT		ELECTRICAL			
	ARMY	PHONE NUMBER		CIVIL		STRUCTURAL	(A = Comment accepted)	(C = Correction made. List drawing or paragraph number whwere correction made)	
		503-808-4934		SANITARY					

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	(If not accepted explain)	(If not corrected, explain)	(Initials)
27.					
28.					

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

PAGE OF

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE 9/26/00
X DESIGN MEMO / CONCEPT / FINAL / PLANS & SPECS / PRELIMINARY	John Day Surface Bypass Removable Spillway Weir 60% Report		

REVIEWER

NWP-EC-D_______ / AIR FORCE / ARMY

NAME David Illias

PHONE NUMBER X4901

ARCHITECT / LAND ARCHITECT / CIVIL / SANITARY / MECHANICAL / ELECTRICAL / X STRUCTURAL

ACTION TAKEN ON COMMENT

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	Table of Contents	Section 1 should include a paragraph on agency coordination that summarizes the coordination that has taken place and references Appendix C. The project description and changes since the last report are typically separate paragraphs in Section 1. Appendix E should include deflector model study result or included in another appendix.	A	A new section "1.5 Agency Coordination" has been included in the 90% submittal. The project description is included in section 1.1 and the changes since the last submittal are in section1.3. Further modeling will be done on the spillway deflector. The results of this work will be added later when modeling is complete.	
2.	General	A design report for deflectors in Spillway bay 20 is presently being initiated. Need to address in this report how that will interface.	A	New paragraphs have been added to section 1.3 to describe how the deflector work and other items will interface with the main body of work. The project schedule in Section 7 will also address this.	
3.	Paragraph 1.3	If a deflector elevation and geometry is determined in this report then the structural design should also be included.	A	The deflector geometry has not been determined. It will be included later when it is known. A generic deflector design has been included for the 90% submittal.	
4.	Paragraph 1.4	Need to address the correct authority for this work. The delivery order contract is not the authority.	A	Wording has been added to section 1.4 to reflect the correct authority for the work.	
5.	Paragraph 3.5	The alternatives descriptions in paragraph 3.6 are different then those in the alternatives in appendix D.	N. A.	The alternatives are the same. The titles have been edited to reflect comments made on the 30% submittal.	
6.	Paragraph 3.12	Has the model been tested with the lower RSW in place. The impacts with this section in place during spill need to been addressed.	A	Performance with only the tailpiece section in place (both gated and free flow operation) will be evaluated in the physical model. Results of the model tests will not be available for the 90% submittal.	
7.	Paragraph 4.3.1	Load conditions need to include vibration, spillway bay 20 open and 19 closed, and spillway bay 19 open and 20 closed. Are their operating restrictions on the original piers. There was a rumor that due to insufficient anchorage of the reinforcing pier adjacent bay could not be closed and fully open. Is the terminology load cases or load conditions.	A	Electronic pressure cells will be used in select locations on the physical model to obtain some dynamic pressure information which may be useful for vibration evaluation. Load Cases is the terminology used.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

PAGE OF

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO; CONCEPT; FINAL; PLANS & SPECS; PRELIMINARY	John Day Surface Bypass Removable Spillway Weir 60% Report		9/26/00

REVIEWER	
NWP-EC-D_______; AIR FORCE; ARMY	NAME David Illias; PHONE NUMBER X4901
ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY; MECHANICAL; ELECTRICAL; X STRUCTURAL	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
8.	Paragraph 9.2	Consider tying up the RSW up against the skeleton powerhouse bays when not in use. This will eliminate a support barge.	A	Based on the 60% PRM discussion. It looks like a tug boat rented for the infrequent installations and removals might be a better choice. We will also look at tying the RSW up to the Skeleton Bays.	
9.	Paragraph 9.2	A 20% contingency is too high for a DDR level cost estimate. The cost estimate statement is redundant in paragraph 9.3.5.	½ A	Will delete the redundant statement. Since there is no design for a major feature and this was the first estimate of this project, the estimator feels 20% is appropriate.	
10.	Paragraph 9.3.1	Need to include any restrictions on spilling in the adjacent spillway bays during installation of the main structure attachment, the RSW tailpiece, and the RSW main structure. Diving operations will restrict opening the adjacent bay or bays.	A	Agree. Will include these restrictions.	
11.	Paragraph 4.5.1	Will RSW be anchored down and will vibration be a problem, particularly with adjacent bay operation.		The RSW will be held in place primarily by gravity and differential hydrostatic loads. Hold backs at the pier noses could be added for additional restraint when the tainter gate is closed (no differential head).	
12.	Paragraph 3.13.1	What does "TDG" stand for.	A	TDG is the acronym for Total Dissolved Gas. This acronym is defined in paragraph 1.1	
13.	Figure 7-1	The construction schedule should show model studies, in-water work restrictions, and any other construction restrictions.	A	These items as well as plans and specs and bidding periods will be added to the schedule.	

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
DESIGN MEMO; CONCEPT; FINAL; PLANS & SPECS; PRELIMINARY; X DDR	John Day Removable Spillway Weir DDR 60% Review	John Day Project	9/29/00

REVIEWER			
X CH2M/MW JV; AIR FORCE; ARMY	NAME: See Item No. 1 PHONE NUMBER: (425) 453-5000	ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY	X MECHANICAL; ELECTRICAL; STRUCTURAL; Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	-----------	The reviewers are indicated by their initials as follows: PFW – Pete Wiedemann General/Mechanical Review RR – Rick Regan Hydraulic Review TPD – Tom Delaney Structural Review			---------
2.	General	A fly sheet should be provided in front of the Table of Contents.	A	This will be included in the 90% Submittal.	PFW
3.	Executive Summary, para 4	The preferred geometry that was selected should be stated.	A	Sentences were added to the 4th paragraph in the Executive Summary.	PFW
4.	Executive Summary, para 5	What will determine whether the tailpiece stays in place or is removed by crane? This should be discussed.	A	It was decided that the whole tailpiece would not be removed by crane and that the tailpiece would be semi-permanently installed and left in for the 3-year life of the project. A sentence was added to the 5th paragraph in the Executive Summary.	PFW
5.	Para 1.1, last para.	The reason that the 30-feet deflector length might not be the best solution should be stated.	A	C. Additional wording was added to paragraph 1.1	PFW
6.	Para 1.3	The 4th bullet talks about the development of five "optimum" RSW geometries. How can all five be the best?	A	"Optimum means that these RSW geometries would be suitable for permanent installation in the spillway bays.	PFW
7.	Para 3.6	The first paragraph states that five of the six initial alternatives are presented below. What happened to the sixth alternative? It should be mentioned.	A	C. The sixth initial alternative is presented in the report as Optimum Alt A. Statement added to Hydraulic Design Section para 3.7 to clarify.	PFW
8.	Para 3.8.1	The differentiation between Alternatives 1 thru 6 and Alternatives A thru E needs to be clarified. The text is confusing.	A	C. A new paragraph 3.5 has been added in the Hydr Des section to clarify the process of the RSW design development.	PFW

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	
PLANS & SPECS	PRELIMINARY	X DDR	60% Review		9/29/00

REVIEWER				ACTION TAKEN ON COMMENT		
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
AIR FORCE		LAND ARCHITECT	ELECTRICAL	(A = Comment accepted)	(C = Correction made. List drawing or paragraph number whwere correction made)	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	(If not accepted explain)	(If not corrected, explain)	(Initials)
		SANITARY	Technical Review			

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
9.	Tables 3-1, 3-2	The tables have been inserted <u>before</u> the text references. Also, they could use page numbers.	A	C. Page numbers will be added to the tables. The tables do follow references in the text. See Hydraulic Design Section para 3.10.1	PFW
10.	Tables 3-3, 3-4	The tables could use page numbers.	A	C. Page numbers added to tables.	PFW
11.	Section 5	The inflatable side and bottom seals on the main structure should be discussed in this Section.	A	Text will be added to describe seals.	PFW
12.	Section 5	The air compressors and engine drives and winches and drives should be described and capacities indicated in this section and shown on Plate 11.	A	Text will be added to describe the details of outfit items.	PFW
13.	Para 5.1	The various vent, blow, flood, control, etc valves should be called out on Fig 5-1.	A	This figure will be replaced with a more legible diagram.	PFW
14.	Para 5.1	Is 7 psig adequate pressure to deballast the RSW under all forebay elevations?	A	This value is being corrected for the calculated pressure.	PFW
15.	Para 5.1	The locate of the two air manifolds is not mentioned in the text.	A	The manifolds will be located on the RSW Pier Noses and the location shall be documented in the text.	PFW
16.	Para 5.1	The support raft, referred to in the last paragraph, is called out as support barge on Plate 11.	A	The support raft will be replaced by a vessel of opportunity.	PFW
17.	Figure 5-1	The figure needs callouts. Also, the 2-dimensional isometric is difficult to read.	A	This figure will be replaced with a more legible diagram.	PFW

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	9/29/00
PLANS & SPECS	PRELIMINARY	x DDR	60% Review		

REVIEWER				ACTION TAKEN ON COMMENT
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	
AIR FORCE		LAND ARCHITECT	ELECTRICAL	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	
		SANITARY	Technical Review	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
18.	Section 6	The first paragraph refers to a portable diesel engine compressor. Plate 11 shows two compressors.	A	A single compressor will be recommended.	PFW
19.	Section 6	The first paragraph states the compressed air will operate vent and flood valves. Page 5-1, second paragraph, states that the vent valves are to be plug type relief valves and implies that only the flood valves are pneumatically actuated.	A	Due to depth considerations, hydraulic actuators will be used. With remote operation, both vent and flood valves will require actuators.	PFW
20.	Section 6	Paragraphs one and two refer to an installation barge. Plate 11 calls it a support barge.	A	The support barge is being replaced by a vessel of opportunity (towboat).	PFW
21.	Para 7.1, 2nd para.	The second paragraph could use some rewording.			PFW
22.	Para 7.2, 2nd para.	Some words are missing at the end of the last sentence.	A	The last sentence was altered to make the meaning clear.	PFW
23.	Para 8.1.1, 2nd para.	The raft referred to is called out at a support barge on Plate 11	A	The support barge is being replaced by a vessel of opportunity (towboat).	PFW
24.	Para 8.1.1, 3nd para.	The various procedural steps refer to percentage fill levels for flooding the tanks. How will these levels be determined during operation?	A	Marks will be painted on the outside of the RSW to indicate when it has reached an acceptable condition for a given step.	PFW
25.	Para 8.1.1, 4nd para.	"Piernose" should be two words.	A	Correction made.	PFW
26.	Para 8.1.1, 5th para.	The referenced Plate 13 should probably by Plate 11.	A	This section is being rewritten in the 90% and a new plate is being developed.	PFW

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	9/29/00
PLANS & SPECS	PRELIMINARY	x **DDR**	60% Review		

REVIEWER				ACTION TAKEN ON COMMENT		
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
AIR FORCE		LAND ARCHITECT	ELECTRICAL	(A = Comment accepted)	(C = Correction made. List drawing or paragraph number whwere correction made)	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	(If not accepted explain)	(If not corrected, explain)	(Initials)
		SANITARY	Technical Review			

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
27.	Para 8.1.2, 2nd para.	The referenced four feet six inch dimension is shown on Plate 2 to be only four feet zero inches.	A	The inconsistent values will be corrected.	PFW
28.	Para 8.1.5	The paragraph is unclear in its description as how the tailpiece is to be installed.	A	The installation has been decided and this alternative has been dropped.	PFW
29.	Para 8.1.6	There's a "typo" in the second sentence.	A	This alternative has been dropped and won't appear in the 90% submittal.	PFW
30.	Para 9.3.4	The first sentence is unclear. Usually <u>no</u> in-water work is permitted from Mar 1 through Dec 31.	A	Sentence was changed to indicate no in-water work, but the dates of March through November are correct.	PFW
31.	Plate 9	This plate provides important design information, but seems of questionable value in this report	A	This plate will be replaced by a schematic of the systems.	PFW
32.	Plate 10	The control air manifold view would be more clear is the air operated knife gate valves were shown.	A	The valves will be indicated.	PFW
33.	Plate 10	The control air manifold view refers to a "specialty equipment list" Is this the "symbol list" shown at the right?	A	This callout will be clarified.	PFW
34.	Plate 10	Both manifold views refer to shore connections for air. Air is to be provided from the support barge.	A	Air will now be provided from a vessel of opportunity (towboat)	PFW
35.	Plate 11	The plate is confusing. Suggest sequential steps be shown, similar to Plate 12	A	The plate will be clarified.	PFW
36.	Plate 12	The abbreviate "GM" should be spelled out and dimensioned on the various views.	A	The metacentric height (GM) will be defined and identified on each view.	PFW

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	
PLANS & SPECS	PRELIMINARY	X DDR	60% Review		9/29/00

REVIEWER				ACTION TAKEN ON COMMENT
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	
AIR FORCE		LAND ARCHITECT	ELECTRICAL	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	
		SANITARY	Technical Review	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
37.	Pertinent data Page PD-4 Fish Facilities	2 ladders – one on North shore other on South shore	A	The North and South shore ladder column headings have been relocated for clarity..	RR
38.	Pertinent data Page PD-4 Fish Facilities	North ladder – 3 submerged weir entrances, only 2 normally operated	A	Changed	RR
39.	Pertinent data Page PD-4 Fish Facilities	South ladder – 4 submerged weir entrances, only 3 normally operated	A	Changed	RR
40.	Pertinent data Page PD-4 Fish Facilities	Number of submerged orifice entrance is incorrect		These were the numbers provided in original design. The number has changed since then.	RR
41.	Pertinent data Page PD-4 Fish Facilities	Number of diffuser chambers needs to be checked – looks high. All fish ladder pertinent data should be checked.	A	Drawings were checked. There is 40 difusion gratings and associated chambers (2 per Unit).	RR
42.	Page 1-2 Paragraph 1.1	General – removal of RSW in a couple of days to prepare for a flood that would require full SW use is unnecessary – baring an upstream total dam failure – the knowledge that full capacity will be required will be known months ahead of the time that the requirement is needed. I don't believe that this is the reason for requiring rapid removal of the RSW. This should be clarified.	A	C. Agree, rapid removal of RSW for either flood passage or biological testing is no longer a requirement. References to rapid removal have been deleted from the DDR.	RR
43.	Page 1-2	Suggest that the two Walla Walla District – Lower Granite reports should become appendix in this report.		These reports are quite voluminous. Therefore, they should remain as references.	RR
44.	Page 1-2 and Page 1-3	On page 1-2 states 30' deflector "<u>might not be the best solution</u>". On page 1-3 states 30' deflector was "<u>judged to be unacceptable</u>" – quite a difference.	A	C. The text on pages 1-2 and 1-3 has been revised to be consistent.	RR
45.	Page 3-2 Section 3.1.4	A cavitation index of 0.6 is not a "magic number". The geometric shape with respect to the cavitation number needs to be considered – a cavitation number of 0.6 with an offset away from the flow of 0.06 ft would not experience cavitation damage but an offset of 0.1 ft would. Refer to plates 2-6 to 2-9 EM1110-2-1603.	A	C. The text has been revised to delete reference to a specific cavitation index value.	RR

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	
PLANS & SPECS	PRELIMINARY	x DDR	60% Review		9/29/00

REVIEWER			
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL
AIR FORCE		LAND ARCHITECT	ELECTRICAL
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL
		SANITARY	Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
46.	Pages 3-3 Section 3.2.4	Seattle District reports, these are not Seattle District Reports. They are North Pacific Div Reports, which were accomplished by Seattle District personnel.	A	C. Correction made.	RR
47.	Page 3-4 Paragraph 3.3	How much greater is John Day Dam discharge objective than the discharge objective for Lower Granite Dam? The report should state these two values.	Not Accepted	The requested information is already included in the paragraph.	RR
48.	Page 3-7 2nd Paragraph	States RSW crest shape designed to maximize discharge and efficiency – if this is the case the crest must be underdesigned (He/Hd >1.0) however this paragraph also implies the crest is designed for 22.5' of "submergence" which should be "head" as this crest is far from being submerged. If the crest shape is designed for 22.5' then He=Hd and the shape does not maximize the discharge/efficiency. See plate 3-3 of EM1110-2-1603 – Suggest a discussion of the development of the ogee crest shape be included in report.	A	C. Agree. The paragraph has been re-written to discuss the development of the ogee crest shape.	RR
49.	Page 3-7 5th Paragraph	The design of the lower crest shape should be discussed in more detail i.e. what is He/Hd ratio, what is the $C=Q/LHe^{3/2}$ design value, what is the additional surcharge to reservoir if lower portion is in place during the spillway design flood, what crest pressures would be expected during a full open gate event. The report must provide sufficient documentation for <u>not</u> using the lower crest as stated in section 3.7	A	C. Discussion of hydraulic design of the tailpiece section has been added to Hydraulic Design Section paragraph 3.8.1	RR
50.	Figure 3-5	The upper crest shape equation may have a typo error in the constant. It ends in 5 where as the Alt 2,3, and 4 constants end in 6. This should be checked.	A	C. Correction made to figures. The correct coefficient is 0.03545.	RR
51.	Page 3-8 Section 3.7 Paragraph 3	Indicates that gated flow could be passed over the lower portion of crest <u>while</u> upstream portion was removed. This sounds quite risky, but don't you mean <u>when</u> upstream portion is removed. Also, if any flow is to be passed with lower portion in place, a detailed discussion of the hydraulic characteristics and limitations must be presented in the report and verified by model study.	A	C. Agree, text has been revised. Performance of the tailpiece with gated and un-gated flow will be evaluated in the physical model. The shape of the tailpiece section has been theoretically designed to minimize gate-controlled performance problems, however, acceptable performance can not be	RR

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	
PLANS & SPECS	PRELIMINARY	X DDR	60% Review		9/29/00

REVIEWER				ACTION TAKEN ON COMMENT
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	
AIR FORCE		LAND ARCHITECT	ELECTRICAL	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	
		SANITARY	Technical Review	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
				guaranteed until model results are available.	
52.	Page 3-9 Section 3.7.1 4th Paragraph	Explain how “severe” the underdesign of the lower crest shape is and what effects this degree of underdesigned crest will cause – and provide an estimate of how low pressures might be expected to reach and compare these estimated pressures to acceptable low pressure. Although not exactly the same geometry, it might be worthwhile to compare to cavitation potential of the geometry shown on plate 2-8 EM1110-2-1603. These pressures and other hydraulic characteristics should be investigated in the model. Granted frequent overflow should never occur but one never knows what will occur in the field.	A	C. The text has been revised to discuss the degree of under-design and potential pressures with free flow over the tailpiece section. Actual pressures and hydraulic performance will be measured and tested in the physical model. Based on theoretical estimates, free flow over the tailpiece section will result in extremely low pressures and is considered to be unacceptable.	RR
53.	Page 3-9 Section 3.7.2	Section 3.7 EM1110-2-1603 discusses spillway bay surge. This should be discussed in the context that surging might occur under gated flow condition if more than one bay is fitted with these extended piers. Also surging in the spillway bay should be checked for and reported using the single bay design model operating with various gate openings (Note at Chief Joseph Dam surging was found to be so extreme that surge suppresser devices had to be developed and installed on the piers). Surging with the one bay RSW might be a severe problem if the gate is used to control the flow through this bay. Model observations of surging should be discussed. Again I realize that the RSW is only to operate at free flow conditions but who knows what will happen in the future.	A	C. Test has been revised to address comment. Based on criteria in EM 1110-2-1603, surging within the spillway bay should not occur. The model will be used to verify this conclusion.	RR
54.	Page 3-9 Section 3.7.3 2nd Paragraph	Without knowing the geometry of the intersection of steel and concrete how can you predict that the intersection will be cavitation free (See comment 5 above)	A	C. Agree. The paragraph has been revised. The intersection of the steel RSW and the spillway concrete will be designed to eliminate surface irregularities.	RR

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
DESIGN MEMO / CONCEPT / FINAL; PLANS & SPECS / PRELIMINARY / X DDR	John Day Removable Spillway Weir DDR 60% Review	John Day Project	9/29/00

REVIEWER			ACTION TAKEN ON COMMENT		
X CH2M/MW JV AIR FORCE ARMY	NAME: See Item No. 1 PHONE NUMBER: (425) 453-5000	ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY; X MECHANICAL; ELECTRICAL; STRUCTURAL; Technical Review	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
55.	Page 3-10 Section 3.7.4	Earlier in the report the 30' deflector was stated to be unacceptable (page 1-3). Here the report states that the deflector might be as long as 30 ft. these conflicting statements should be removed from the report. This subject also occurs in section 4.6	A	C. Text has been revised throughout the report to remove conflicting information.	RR
56.	Page 3-12 Section 3.9.1	Does the investigations of the hydraulic characteristics with the 1:25 scale sectional model include the measurement of crest pressure using both piezometers and electronic pressure cells. This data should be collected in order to assess cavitation potential and hydraulic loading and associated hydraulic induced vibrations.	A	C. Agree, both electronic pressure cells and piezometers will be used in the physical model to measure pressures in select areas of the RSW and the existing spillway.	RR
57.	Page 3-13 Section 3.10.1	A more useful presentation of the hydraulic characteristics of the flow over the crest would be for a max pool elevation develop the centerline profile and the profile along the pier and also report velocities for these two profiles – This could be done for max & min pool only.	Not Accepted	Information from the models illustrates that flow depth/velocity varies across the entire width of the RSW. Computed values along piers and the centerline of the bay are not considered any more meaningful than the average conditions presented. Physical model data showing measured depths at various locations on the RSW will be included in the 90% document.	RR
58.	Page 3-19	Both curves should define the y-axis as Reservoir Elevation.	A	C. The 'y' axes will be re-labeled to read Reservoir Elevation.	RR
59.	Page 3-21 Section 3.12 2nd Paragraph	Should state that RSW is designed for free flow conditions <u>only</u> not <u>also</u>.	A	C. The word 'also' has been deleted from the text.	RR
60.	Page 3-21 Section 3.12 3rd Paragraph	The statement is made that the lower crest is designed in a manner that precludes undesirable hydraulic conditions. I believe this statement should be left unsaid until model study results are analyzed.	A	C. Agree, the statement has been deleted from the text.	RR

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	9/29/00
PLANS & SPECS	PRELIMINARY	X DDR	60% Review		

REVIEWER				ACTION TAKEN ON COMMENT		
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
AIR FORCE		LAND ARCHITECT	ELECTRICAL	(A = Comment accepted)	(C = Correction made. List drawing or paragraph number whwere correction made)	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	(If not accepted explain)	(If not corrected, explain)	(Initials)
		SANITARY	Technical Review			

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
61.	Page 4-1 Section 4.1 2nd Paragraph	Again 30' long deflector determined to be unsatisfactory but hydraulic discussion (see comment 55) states deflector might be as long as 30 ft. This needs to be clarified.	A	C. Agree, see response to items 44 and 55. After modeling the deflector geometry will be set.	RR
62.	Section 4.2	There is no mention of hydrodynamic loads. The RSW is a relatively light structure, which might react to rapidly varying pressure fluctuations on the crest. See comment 53. Also, there is no mention of wave loading on the structure. Also see comment 66	A	C. Electronic pressure cells will be used at various locations in the physical model to obtain data regarding dynamic pressures. The RSW will be filled with water and have a very large mass. Wave loads will be determined and evaluated.	RR
63.	Section 4.5.1.1.	Should check min lock sill-depth. I thought it is somewhat greater than 11 ft.	A	The minimum of depth over sill is greater than this. However, we want to maintain a clearance of about 5 feet for this level of study.	RR
64.	Plates 4 thru 8	There is <u>vast</u> differences between the RSW design shown on these plates and the "proof of concept design" discussed and shown on figures in the Hydraulic Design Section of the report, Section 3. The hydraulic design and model studies are with a crest that have a vertical up stream face the drawings depict an overhanging crest. The hydraulic characteristics of the two are quite different. Also, the tailpiece is shown with a sharp upstream edge contrary to the ogee shape described in the hydraulic text. This sharp edge design would result in severe damage to the tail piece and to the concrete of the existing spillway with uncontrolled flow and may result in tainter gate vibration during controlled flow. If the design depicted on the Plates is to be recommended the physical model studies must incorporate this design shape and the hydraulic text of the report must discuss this shape. **These differences are so great that further comments on the plates by this reviewer will be withheld until the conflicting designs are resolved.**	A	C. Corrections made to plates. The recommended design incorporates the upper crest shape shown on plates 2-4 with the tailpiece section shown on Figs 3-5 and 3-8, not the sharp edge design shown on plates. The recommended shape will be tested in the physical model. These problems will be addressed in the 90% submittal.	RR

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	
PLANS & SPECS	PRELIMINARY	X DDR	60% Review		9/29/00

REVIEWER				ACTION TAKEN ON COMMENT		
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
AIR FORCE		LAND ARCHITECT	ELECTRICAL	(A = Comment accepted)	(C = Correction made. List drawing or paragraph number whwere correction made)	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	(If not accepted explain)	(If not corrected, explain)	(Initials)
		SANITARY	Technical Review			

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
65.	Section 4.5.1.1	Details should be presented on where and what size the ballast compartments are. How the flooding is going to be controlled should be presented. The rate of turning should be determined and presented. The material presented in this section of the 60% report does not provide a convincing argument that the RSW can be tipped from horizontal to vertical and placed into a spillway bay. I did notice that there is more argument for the tipping process in Section 8; however, the above comment is still valid.	A	Flooding is controlled by means of the flood valves and vents. We are designing the system for a slow rotation (2 hours) from horizontal to vertical.	RR
66.	Section 4.5.3.1	This section does not discuss how the main structure is attached to the existing structure. If it to be held in place by gravity force this should be explained in detail. Wave and other hydrodynamic loads are expected to contribute significantly to the instability of this structure, and may preclude a pure gravity attachment.	A	Gravity and hydrostatic pressure are the main restraining forces. Hold backs will also be provided at the pier noses. Calculations show that the main structure is stable for all loads.	RR
67.	Section 4.5.3.2	This section discusses cutting recesses into the spillway crest parallel to the crest. These recesses are not detailed on the drawing. The text goes on to say there will not be any protrusions above the spillway surface in order to prevent the formation of cavitation. It should be understood that an offset away from the flow (recess) could trigger cavitation just as well as protrusions into the flow. The geometry and cavitation index for the proposed geometry must be investigated, See EM1110-2-1603. Hydrodynamic loads on the tailpiece must be examined in detail using the physical model.	A	C. Agree, the design will preclude into-the-flow or away-from-the flow irregularities in the existing spillway or at intersections between the RSW and the existing spillway. Electronic pressure cells will be located at various places on the tailpiece section in the physical model to obtain dynamic pressure data.	RR
68.	Section 5.2	A painted surface that has the correct preparation should withstand the high velocity flow and not abrade. Painting of the sluice steel-liner a t Libby Dam withstand velocities of 130 fps.	A	Last sentence deleted.	RR
69.	General	Given the wind conditions at John Day Dam a significant wave of up to 4 or 5 feet might occur. The flow conditions, with reservoir waves, on the crest should be investigated. Waves would result in surging flow over the crest. High flow when a wave crest arrives at the spillway and lower flow when the trough occurs. This condition could result in significant hydrodynamic loads. The physical model should be used to investigate this phenomenon.	A	C. Based on information in John Day Dam GDM 3, the significant wave is about 5.2 ft. RSW stability was analyzed using a design wave of 2 times the significant wave, or 10.4 ft, with a period of 4.1 seconds. The RSW is located below 11.5 to 22.5 feet below the surface.	RR

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	9/29/00
PLANS & SPECS	PRELIMINARY	X DDR	60% Review		

REVIEWER				ACTION TAKEN ON COMMENT		
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
AIR FORCE		LAND ARCHITECT	ELECTRICAL	(A = Comment accepted)	(C = Correction made. List drawing or paragraph number whwere correction made)	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	(If not accepted explain)	(If not corrected, explain)	(Initials)
		SANITARY	Technical Review			

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
				The wave forces are less at depth. Pressures on the RSW will be measured in the physical model using electronic pressure cells and piezometers at select locations with this wave regime simulated.	
70.	Table of contents	There is no reference to the location of the plates between Section 9 and the references.	A	This has been added.	TPD
71.	Page 1-3	In the second line of the second bullet for the amendments, “the” is spelled with a capital “T”.	A	Corrected.	TPD
72.	Page 4-1	In paragraph 4.2.3 under the “Range of Tailwater Elevations” shouldn’t the first line item read “Normal Low Flow” instead of “Minimum”?		Minimum is the correct term since the elevation not only depends on flow but also on the water level set by operation of The Dalles Project.	TPD
73.	Page 4-2	Has the spillway base elevation of 112.07 ft (NGVD) ever been verified?	A	For the purposes of the stability analysis the base elevation to be used is the one used in the initial analysis. This value will be entered on page 4-2.	TPD
74.	Page 4-3	Add the material for the high strength bolts connecting the RSW to the piers to paragraph 4.2.10.	A	Added information.	TPD
75.	Page 4-3	Add the material for the bulb seals to paragraph 4.2.10.		The material for the bulb seal is a motter of design rather than criteria. The bulb seal material will be entered elsewhere in Section 4.	TPD
76.	General	Where is the RWS stored while it is not in use?		It is anticipated that it will be stored north of the floating lock guidewall, near the floating maintenance bulkhead.	TPD

09/28/01REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	John Day Removable Spillway Weir DDR	John Day Project	9/29/00
PLANS & SPECS	PRELIMINARY	x DDR	60% Review		

REVIEWER				ACTION TAKEN ON COMMENT		
X CH2M/MW JV	NAME: See Item No. 1	ARCHITECT	X MECHANICAL	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
AIR FORCE		LAND ARCHITECT	ELECTRICAL	(A = Comment accepted)	(C = Correction made. List drawing or paragraph number whwere correction made)	
ARMY	PHONE NUMBER: (425) 453-5000	CIVIL	STRUCTURAL	(If not accepted explain)	(If not corrected, explain)	(Initials)
		SANITARY	Technical Review			

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	BACK CHECK BY
77.	Plate 5	In Alternative 1 the 3/8 inch plate appears to span almost 10 feet. How is it supported?		The design of this Tailpiece structure has been changed and will be shown in the 90%.	TPD

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO; CONCEPT; FINAL; PLANS & SPECS; PRELIMINARY; X **60%**	John Day Removable Spillway Weir - 60% DDR Cost Est	John Day Lock & Dam, Columbia River Basin	

REVIEWER

CENWP-EC-DX	NAME Pat Jones	ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY; MECHANICAL; ELECTRICAL; STRUCTURAL; X Cost Estimate
AIR FORCE	PHONE NUMBER (503)808-4790	
ARMY		

ACTION TAKEN ON COMMENT

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	MCACES Cover sheet	It appears that this job will be accomplished in the state of Washington, so sales tax needs to be applied.	A	Will include sales tax in next estimate.	
2.	Summary page 12	Percentages given for Field Overhead, Prime Contractor's Profit and Prime Contractor's Bond are not correct. Instead of the 12%, 5%, and 1.0% given, they calculate to be 7.2%, 8.9% and 0.9%.	A	Will revise headings to reflect actual percentages in next estimate.	
3.	Detail page 10	Item 6-0506, Anchor Bolts/Install Steel. Need to check if a 3 man crew is adequate. May need a 5 man crew. Might want to check with Mike Colesar at The Dalles/John Day project, (541)298-7567.	A	Will check with Mike Colesar before next estimate.	
4.	Detail page 12	Item 6-0703, Tow RSW to John Day Dam. Mis-spelled word "float".	A	Will correct.	
5.	Detail page 15	Top item, Support Barge. Last sentence. Should be "waiting", not "waited".	A	Will correct.	
6.	Crew Backup	Need to show at a lower level, so actual crews show up. Of course, I can get these off the floppy, but they should be in the printout too.	A	Will print out crews at lower level in next estimate.	
7.					
8.					
9.					

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO; CONCEPT; FINAL; PLANS & SPECS; X PRELIMINARY	Removable Spillway Weir 60%	John Day Dam	10/4/00

REVIEWER			
X NP EC-DM; AIR FORCE; ARMY	NAME: Rick Mettler; PHONE NUMBER: 808-4925	ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY	X MECHANICAL; ELECTRICAL; STRUCTURAL

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1	Section 5	Provide text in the mechanical section describing the operation of the ballast/deballast system. For instance, which valves are opened, and how are they actuated, to fill the tanks? Also describe the deballasting procedure.	A	General descriptions of the system operations will be added to the text.	
2	Section 5	Add the estimated fill/emptying time and how much air is required to meet the emptying time. Provide calculations.	A	This information will be added to the text.	
3	Section 5	What are preliminary compressor characteristics? Pressure, CFM, motor HP, etc.	A	This information will be added to the text	
4	Section 5 and Plate 10	How was 7psi chosen as the deballast air manifold relief valve setting? Are there calculations to back this up? Provide calculations.	A	This value is incorrect and has been revised in the text and plates.	
5	Plate 10	Label the valves on the hydraulic schematic to match the text in Section 5.	A	The text will be clarified.	
6	Plate 10	I couldn't find the pneumatically operated gate valve (flood valves) on this sheet. Presumably they are operated by the cylinders, controlled by the 4W3P valves.	A	The plate will be clarified.	
7	Plate 10	Shouldn't the cylinders be shown inside the compartments? And connected to the flood valves?	A	The plate will be clarified.	
8	General	Who is providing the description of the support barge and the equipment for it, such as winches and diesel generator?	A	The barge has been replaced by a support tug.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	60% Submittal for John Day Surface Bypass RSW DDR	Oregon	28 Sept 00
PLANS & SPECS	X Prelimnary				

REVIEWER				
AIR FORCE	NAME: Diana Modini	ARCHITECT	MECHANICAL	
X ARMY	PHONE NUMBER: 503-808-4896	LAND ARCHITECT	ELECTRICAL	
		X CIVIL/Hydraulic	STRUCTURAL	
		SANITARY	Technical Review	

ACTION TAKEN ON COMMENT

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	P 3-1, Par 3.1	Do we want to add that the RSW is also suppose to be as close as we can get it to the spillway?	A	C. Added to criteria	
2.	Par 3.1.1	PMF is 276.3. Is SDF same as PMF?	A.	C. Spillway Design Flood pool elev changed to 276.3 ft in text. SDF and PMF are the same for JD.	
3.	P3-4, Par 3.3	"…flow previously tested…". This was only tested in the general model. Maybe insert the word "conditions".	A	C. Clarification will be made in the text.	
4.	P3-4, Par 3.3	Do we need this much discussion on Lower Granite? I understand your point, but maybe state why you are discussing it.	A	Comment accepted, however, the relatively brief amount of discussion regarding Lower Granite is considered warranted and is required by the SOW.	
5.	P3-6, Par 3.5 and 3.6	The two paragraphs are kind of confusing. Can they be combined somehow so there isn't a lot of repetition?	A	The paragraphs were written separately to describe the design process per the SOW. As such, they should not be combined.	
6.	P3-6, Par 3.5	What happened to the simplified alt 7? I see that it listed as an Optimum alt B. Why is it considered Optimum?	A	The SOW only required that five Proof of Concept alternatives be developed. Therefore, the POC Alternative 7 is being presented in the DDR as Optimum Alternative B.	
7.	P3-6, Par 3.6	"…and the Physical Hydraulic Model Study Report…" Is it the same as the Model Alternatives Report in Appendix D?	A	They are two separate reports and are included as separate appendices in the DDR.	
8.	Fig 3-1 and related	Is the "Existing Crest Centerline" the existing spillway axis stated in velocity (etc) tables?	A	Yes, the existing crest centerline and the existing spillway axis are one and the same.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE			PROJECT	LOCATION	DATE
DESIGN MEMO	CONCEPT	FINAL	60% Submittal for John Day Surface Bypass RSW DDR	Oregon	28 Sept 00
PLANS & SPECS	X Prelimnary				

REVIEWER				
AIR FORCE	NAME: Diana Modini	ARCHITECT	MECHANICAL	
X ARMY	PHONE NUMBER: 503-808-4896	LAND ARCHITECT	ELECTRICAL	
		X CIVIL/Hydraulic	STRUCTURAL	
		SANITARY	Technical Review	

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
9.	Fig 3-5	Is this figure supposed to show the low crest? Because Optimum C has it. Please clarify. Otherwise its paragraph should state there are three portions to the RSW.	A	C. The original POC RSW Alt 5 did have three pieces as shown on Fig 5. The intermediate piece was deleted in the final POC design (aka Opt Alt C). The text will be revised to clarify.	
10.	P3-8, Par 3.7 and 3.7.1	Clarify that the low crest is now called a tailpiece, or other through report.	A	C. Clarification made in text.	
11.	P3-9. Par 3.7.2	Is there any discussion about why one pier length of the RSW alternatives is acceptable over another?	A	C. Additional text presented in Hydraulic Design Section para 3.8.2 to discuss pier length.	
12.	P3-9, Par 3.7.3	"As discussed in Section 3.6.1". Its 3.7.1	A	C. Correction made in report.	
13.	P3-9, Par 3.7.1 and 3.7.3	Seems like part of the paragraphs repeat themselves, can it be condensed some?	A.	C. These paragraphs have been re-written which may satisfy the reviewers comment.	
14.	P3-8, Par 3.8.1	Section 3.5.1 doesn't exist. Try 3.5?	A	C. Correction made in report.	
15.	Par 3.7, 3.8.1, and 3.8.2	These paragraphs are confusing. Assume you are trying to say that the final POC is Alt 5, but we change to the low crest (tailpiece, not the intermediate), so assume this is the final optimum RSW?	A	Assumption is correct. The final POC design is Alternative 5 with a two-piece RSW. That is also considered to be Optimum Alt C.	
16.	P3-12, Par 3.9	:…and at the WES in…". Minor. Delete "the".	A	C. Correction made in text.	
17.	Figures 3-11 etc	Suggest naming them also as Water Surface Profiles. Also is the RSW only shown? I don't see the double ogee on some of them. If it is only the RSW, why not include past the double inflection (RSW to existing spillway ogee)?	A	C. Correction made to figures.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
DESIGN MEMO; CONCEPT; FINAL; PLANS & SPECS; X Prelimnary	60% Submittal for John Day Surface Bypass RSW DDR	Oregon	28 Sept 00

REVIEWER			
AIR FORCE; X ARMY	NAME: Diana Modini; PHONE NUMBER: 503-808-4896	ARCHITECT; LAND ARCHITECT; X CIVIL/Hydraulic; SANITARY	MECHANICAL; ELECTRICAL; STRUCTURAL; Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	ACTION TAKEN ON COMMENT: REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
18.	P3-12, Par 3.9.1	"…Construction is now…final Proof of Concept RSW alternative 5". Do you mean Optimum C?	A	Comment acknowledged. The final POC design (Alt 5) is the same as the Optimum design Alt C.	
19.	P3-19, Fig 3-22	Should title be "Optimum" not POC? Also scale looks different from figure 3-21. If so, make them the same for easier comparison. Also suggest adding figure number to labeling (ex. Opt D is figure 3.9).	A	C. Corrections made to figure.	
20.	P3-20, Par 3.11, 3	Discussion of WES work, should some or all of this also be included under 3.9.2 General Model Studies?	A	C. Results of model testing to date will be summarized in appropriate sections of the report.	
21.	P 3-21, 1st par	"…large flood events, such as might occur…". Should "such as" be deleted or other wording?	A	C. Text revised as suggested.	
22.	P 3-21, Par 3.13.1	Should a discussion include that the optimum deflector will be obtained with and without the RSW.	A	C. Will include this clarification in the text.	
23.	P 3-22, Par 1	"…elevation 149…". Not familiar, but do you mean 148? Bays 2-19 are at 148?	N.A.	The original model study from the Bonneville Lab gives an elevation of 149 ft. We understand that the as-built geometry is elevation 148 ft with a 15 ft radius transition.	
24.	P 3-22, Par 1	"…spillway at elevation 153." Is this elevation correct? Original contract said 153, yet during previous trips, I was told 148. Also says radius was 11ft. I was told it was 12.5. Please clarify.	A	C. For the initial Proof of Concept model tests, the deflector was set at elevation 148 ft, had a radius of 15 ft and was 30 ft in length. Clarification made in text.	
25.	Appendix B	Missing trip reports from the 20-22 June and 19-20 July by MW.	A	The trip reports were done by Northwest hydraulics and are contained in Appendix D as part of their model report. A sheet will be placed in Appendix B directing the reader to these trip reports.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE

	DESIGN MEMO		CONCEPT		FINAL
	PLANS & SPECS	x	Prelimnary		

PROJECT: 60% Submittal for John Day Surface Bypass RSW DDR

LOCATION: Oregon

DATE: 28 Sept 00

REVIEWER

	AIR FORCE	NAME: Diana Modini		ARCHITECT		MECHANICAL
X	ARMY	PHONE NUMBER: 503-808-4896		LAND ARCHITECT		ELECTRICAL
			x	CIVIL/Hydraulic		STRUCTURAL
				SANITARY		Technical Review

ACTION TAKEN ON COMMENT

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
26.	Appendix C	COE trip report is included in correspondence. I don't think it needs to be included. If you want to include out trip reports than maybe they should go with nhc's trip reports.	A	C. The trip report has been moved to Appendix B.	
27.	Appendix D	Page numbers read "B-1" rather than D-1 etc	A	C. Pages will be re-numbered.	
28.	Appendix E	Should trip reports be distinguished from the Physical Model Alternatives Report? It all runs together. Also trip report from 7 August trip is missing. Jim wrote up the last trip report.	A	The trip reports referenced are appended to the Physical Model Alternatives Report and can not be removed. The 7 Aug trip report is not included in the PMAR as that lab visit occurred after the report was published. However, that trip rpt will be included in App C.	
29.	Main Text and App D	Move drawings together, separate from text.		Design type drawings are referred to as plates and are therefore, in a separate section. These will be used later in the plans & specs phase of work. The figures are not design drawings but are graphics to clarify explanations in the text. This is a common Corps report organization and we think it should be kept the way it is.	

Response to NMFS Comments on the 60% Submittal, John Day Surface Bypass Removable Spillway Weir DDR. The comments are added below each question:

To: Willis, NWP
FR: Ruff

Subject: John Day Removable Spillway Weir (RSW)
60% Design Documentation Report (DDR)

Dear Mr. Willis:

We appreciate the opportunity to review the subject report, and have the following comments:

1. Page 1-2, paragraph 5 - We stated in comment #4 of our August 3, 2000 letter on the 30% DDR that we had not seen the final end-bay deflector report, and were not involved in selection of a 30 ft deflector length for spill bay #1, on the basis of hydraulic conditions near the fishway entrance. This decision should be made on the basis of technically interactive model observations and discussions with the agencies. We believe that the deflector length for spill bay #1 should be 12.5 ft, based on a voluntary spill discharges of up to 10 kcfs. We also repeat our request for the final 1999 end-bay model study report by Northwest Hydraulics, Inc.

Response: Note, the forth-coming model study will not be looking at deflector performance in bay #1. Bay 1 has been determined to be a normal length deflector set at elevation 148. This design was accomplished during 1999 testing and is documented in NHC's model report written August '99. The bay 20 length and elevation will be determined through model testing and coordination with appropriate fisheries agencies. A copy of the 99 NHC report will be forwarded to NMFS.

2. Page 1-3 - We understand that there will be an additional scope of work negotiated with your contractor. We agree this is necessary, since there are still outstanding technical issues to be addressed. We request the opportunity to interface on the additional scope of work, to assure consensus on the final list of activities.

Response: The scope of the work has been coordinated with NMFS.

3. Page 3-5, Paragraph 1 and Page 3-21, Section 3.13.1 - Modeling activities prior to completion of Feature Design Memorandum No. 52 (FDM), Section 3.5, addressed combinations of one to four skeleton bays. Chute exit discharge invert elevations (IE) of 157 and 160 were selected , particularly in the context of multiple skeleton bay development. Since the skeleton bay FDM is the basis against which the spill bay #20 prototype RSW will be compared, we recommend narrowing and refining the most economically probable skeleton bay configuration - the single skeleton bay #20, three-chute option. Refinement of the optimum skeleton scenario, while not necessarily binding, can occur within the additional scope of work referenced in comment #2. This will allow a more specific comparative assessment of each alternative. We also

recommend that the single skeleton bay have the same chute IE's, and that selection be based on a low (summer) design project discharge and tailwater elevation (for both the skeleton bay and RSW) of 150 kcfs and el 159. This assumes that both the single skeleton bay and RSW jets will induce unavoidable tailrace eddies at lower project discharge and tailwater elevations, and will not be operated below these thresholds. (Exact low design discharge and tailwater elevation will have to be confirmed in the general model.) Attaining optimum total dissolved gas abatement during this period suggests the lower FDM skeleton bay chute (I.E. 157) should be selected for future skeleton bay vs RSW comparisons.

Response: Observations of hydraulic performance of one skeleton bay surface flow bypass chute in the 1:25 scale sectional model at NHC is included in the scope of work included in the contract modification proposal presently being negotiated between COE and it's contractor. The initial skeleton bay configuration to be tested in the NHC model will be the same as what was designed in the FDM No. 52 (this has a deflector elevation of approximately El. 157.) The final extended deflector geometry to be constructed will be determined through a coordinated design effort between the District and the Fisheries Agencies.

4. Page 3-12, Section 3.8.2 - "...physical model testing of two Optimum RSW configurations will be conducted." At the August, 2000 meeting with fisheries representatives at Northwest Hydraulics, Optimum Alternative C was selected as the RSW design alternative with which to proceed into the next design development phase. Unless there is an overriding reason for doing so, we recommend development of the second option, referenced above, be dropped.

Response: Perhaps, the text in paragraph 3.8.2 is a bit misleading. The AE's scope of work calls for presenting 5 "Optimum" RSW geometry's. At the discretion of the Corps up to two of these can be selected for further study. This further study would require a modification of the contract by exercising a contract option. It so happened that one of the "Optimum" RSW geometry's (Alternative C) was the one selected as the final alternative for the "proof of concept".

5. Page 3-20, Section 3.11 - As expressed at the September 26, 2000 RSW meeting, we believe a satisfactory test of RSW biological performance at the John Day spillway can occur with the RSW installed for the entire passage season in spill bay #20. Although tailoring a blocked study design to allow frequent placement and removal of the RSW would be ideal, we don't believe it is realistic or necessary.

Response: Concur. Removal may prove to be too time consuming, complex and costly. A draft study design for the 2002 test of the JDA RSW will be provided to the regional fishery managers for review and comment. It is imperative that if a decision is to be made concerning the success/failure of the RSW or Skeleton Bay, that a true "test" of the concept be conducted.

6. Page 3-21, Section 3.13.1 - We recommend the high RSW design operating conditions be at project discharge 350 kcfs and tailwater el. 165. Combined with the low design operating conditions described in comment #3, the extended deflector for spill bay #20 should assure skim and undular flow conditions within the design project discharge and tailwater range.

Response: The scope for the deflector testing includes a wide range of tailwater elevations, including elevation 165. We intend to evaluate hydraulic performance throughout this entire range during testing to determine the most acceptable operating conditions.

7. Plate 2 - This RSW configuration extends downstream of the spill gate on-seal location. Therefore, a more complex, three-piece RSW will be installed for prototype testing. We are concerned that installation of the RSW main structure must result in the smoothest possible ogee surface transition to the RSW tailpiece, if fish are to be adequately protected. The potential for gaps and/or ridges at that transition is great, since both existing concrete and to-be-fabricated RSW steel components could have tolerance imperfections that would compound transition imperfection. Therefore, we recommend all possible precautions, including comprehensive field measurements, be implemented. We also recommend that a several day activity be planned for the construction schedule that is for the purpose of modifying one or both adjoining pieces to assure a satisfactory fit.

Response: The details of the fit of the RSW main structure and tailpiece are being studied now and will be addressed in the 90% submittal. Field measurements are being contemplated and will be discussed in the 90% submittal. A more detailed schedule is being developed of the 90% submittal also. The design for the RSW will accommodate the concern for smooth transition. A smooth transition is necessary from both a fish passage perspective and from a hydraulic design perspective. Rough transitions in an installation such as this can lead to unacceptably low pressures and potential damage to both the RSW and the existing spillway and must be avoided. Pressures will be measured in the physical model, and any indication of unacceptable conditions will be apparent.

8. Plate 11 - The support barge should be completely removed during prototype testing and during the entire juvenile passage season.

Response: Since the RSW will be installed and removed no more than once per year a tug will be used to perform this function. Therefore, a barge will no longer be necessary. This change will be reflected in the 90% submittal.

We appreciate the opportunity to comment on this document. If there are questions or comments, please contact Steve Rainey, 503-230-5418, or Gary Fredricks, 503-231-6855.

Sincerely,

Jim Ruff

cc	Ebberts, NWP
	Modini/Buchholz, NWP

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

PAGE OF

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE 09/28/01
X DESIGN MEMO / CONCEPT / FINAL / PLANS & SPECS / PRELIMINARY	JDA RSW 60% DDR		

REVIEWER			
NPW-EC-D / AIR FORCE / ARMY	NAME Rock Peters CENWP-PM-E PHONE NUMBER 808-4777	ARCHITECT / LAND ARCHITECT / CIVIL / SANITARY	MECHANICAL / ELECTRICAL / STRUCTURAL / Biological XX

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number whwere correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	Page 1-1 2nd paragraph	A more detailed story of spill needs to be discussed. This should include the daytime vs nightime test. Also, that we are currently evaluating project survival for spill and other passage routes.	A	This will be discussed in Section2 Biological Considerations, which is being written by the COE.	
2.	Section 1.1 on pages 1-1 and 1-2	Need a discussion in this area that clearly defines **why** we are testing. Also providing linkage to the FDM on skeleton bay as this laid out critical uncertainties and bilogical issues that needed to be addressed. Some of the primary issues for biological test include potential spill efficiency gains and need to evaluate the survival in response to an extended deflector. Also need to measure TDG. These are some of the key unknowns. Also in this section, I suggest a discussion of flow rates and potential gas exchange be discussed.	A	This will be discussed in Section2 Biological Considerations, which is being written by the COE.	
3.	Page 2-1	Biological considerations section has yet to be written. This section needs to be done prior to my review.	A	Section2 Biological Considerations is being written by the COE and will be included in the 90% submittal.	
4.	Page 3-5 Section 3.4.1 First paragraph	Again the description of what we are doing needs to have more information and linked back to the skeleton bay FDM. This section again needs more information on why we are doing the test and what we hope to accomplish.	Not Accepted	A link to the skeleton bay spillway is included in section 1.1. Other discussion will be contained in the Biological Considerations section	
5.	Page 3-9 3.7.3 2nd paragraph	After reading this paragraph, I am unsure whether we have potential to cavitate or not near the interface with the RSW. Obviously, this is not a desirable condition and should be avoided.	A	C. The text has been revised for clarification. The hydraulic conditions are conducive to initiating cavitation unless surface irregularities are eliminated. The design will be developed to eliminate, or minimize, the occurrence of surface irregularities. Very close tolerances will need to be specified, and adhered to, in the construction to prevent irregularities from occurring.	
6.	Page 3-10 3.7.4 Deflector design	This section needs a better explanation of TDG including levels of TDG anticipated and a rationale that discusses the need for TDG evaluation during the test program. Based on the existing deflectors, we anticipate we can spill approximately 6500 cfs per bay without exceeding the TDG waiver criteria. A discussion of TDG performance is needed based on the 15,000 flow rate we will be testing. The down side is we may build the structure and not be able to test if TDG performance is in excess of the waiver limits. This needs to be discussed.	A	C. Added additional text discussing TDG.	
7.					
8.					
9.					
10.					
11.					
12.					

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X 60%DESIGN PLANS & SPECS; CONCEPT PRELIMINARY; FINAL	John Day Surface Bypass RSW	District Office, Portland, OR	26-Sep-99

REVIEWER			
AIR FORCE; X ARMY	NAME: Natalie Richards, PE Hydraulics; PHONE NUMBER: 503-808-4879	ARCHITECT; LAND ARCHITECT; x CIVIL; SANITARY	MECHANICAL; ELECTRICAL; STRUCTURAL; Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph Number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	Pg. 3-1	Alternative D- Skeleton Bay is the standard that is being designed to. Why would it be considered Alternative D? 4) 0.1 feet per second per foot and 7 fps capture velocity- (Design Criteria set by NMFS) Only by digging in the back was I able to figure out the significance of this information. It might be helpful to clarify where it came from.	A	C. The RSW Alt D design has been developed as a means to closely resemble the geometry of the skeleton bay SBS, but with a significantly lower cost than the actual SBSBS. As such, it is considered to be a legitimate design to be presented. Reference to NMFS will be made in the criteria.	
2.	Pg. 3-6	Alternative 6 is discussed but no drawing is provided.	A	C. Initial Alt 6 is the same as Optimum Alt A shown by figure 3-6. The text will be revised to refer the reader to section 3.8 for a discussion of Alt 6.	
3.	Pg. 3-6, Pg. 3-10 and memo (May 9, 2000)	"Proof of Concept" versus "Optimum RSW" versus "Options". Being new to the project, having Alternatives 1-5 and Alternative A-E and in the kick off meeting memo Alternatives 1-6 and Options 1-6, is all somewhat confusing. Best I can tell, the Alternatives minus Alternative 6 is the same as Proof of Concept. What has occurred between Alternatives (1-6) to Proof of Concept (1-5) to Optimum (A-E)? There are really 11 different designs?	A	The design process was specified in the SOW. The POC designs are the preliminary technical evaluations of the six alternatives initially identified as having potential to emulate performance of the SBS. The Optimum design alternatives incorporate features which are considered to potentially improve upon the hydraulic characteristics exhibited by the SBSBS.	
4.	Pg. 3-8	Under 3.7, paragraph one seems like it should be the closing paragraph for the section which then leads into 3.7.1 Crest Design.	A	Comment accepted, however, it seems that the sequence in the 60% report is adequate.	
5.	Pg. 3-10	Paragraph 3-"Alternative A is the same as the initial proof of concept RSW Alternative 6"- Again Alt 6 is not provided.	A	See response to item 2.	

REVIEW COMMENTS

(For use of this form, see NPD Suppl 1, ER 1110-1-12.)

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X 60%DESIGN PLANS & SPECS; CONCEPT PRELIMINARY; FINAL	John Day Surface Bypass RSW	District Office, Portland, OR	26-Sep-99

REVIEWER			
AIR FORCE; X ARMY	NAME: Natalie Richards, PE Hydraulics; PHONE NUMBER: 503-808-4879	ARCHITECT; LAND ARCHITECT; X CIVIL; SANITARY	MECHANICAL; ELECTRICAL; STRUCTURAL; Technical Review

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph Number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
6.	Pg. 3-17	Figure 3-19 Skeleton Bay Hydraulics- Standard- Same comment as Pg. 3-1.	A	See response to item 1.	
7.	Pg. 3-13 and 3-17	Figure 3-11 versus Table 3-1- The station scale on the graph does not coincide with the "Location U/S of Existing Spillway Axis." Station 0.00 is at the end of the spillway in Figure 3-11 and Station 0.00 is at the broad crested weir in Figure 3-19. If they coincided or if there was a common defined point on the graphs (Spillway axis) versus the Tables, it would help get some perspective as to velocities and depths while looking at specific points along the RSW configurations. (Also – 150 on the Tables probably corresponds to a + station.)	A	C. The figures and tables will be revised to clarify and/or make stationing consistent.	
8.	PLATE 2	The design shows an angle on the forebay side of the RSW. This is not consistent with Alternative C. Is another piece being added? Are they designing the right RSW? Or are we modeling the wrong RSW?	A	C. Designs shown on plate 2 and Fig 3-5 were, unfortunately, not coordinated for the report. Alternative C has been modified to reflect the Plate 2 design	
9.	Pg 3-21	Second paragraph- "When the RSW is in place, the spillway gate will be lifted clear of the flow passing down the RSW chute." During the meeting yesterday, NHC stated that this was not the case any more. This is an issue for NMFS and may need to be included in the "Design Criteria" on page 3-1. Can this issue be addressed? Are there hydraulic issues?	A	C. Text added to clarify that the RSW will only be operated with the gate clear of flow. Reference made in the mtg was regarding free flow operation with only the tailpiece section in place. Such operation is not considered acceptable at this time (will be evaluated in the model to confirm).	
10.					
11.					

From: Russell, Joseph B NWP
> Sent: Wednesday, October 04, 2000 1:28 PM
> To: Hanson, Matthew D NWP
> Subject: JD Removable Spillway Weir
>
> Matt: Looking at the 60% Submittal........It appears I've missed the
> Review meeting by a couple of weeks. In any event, I have a couple of
> concerns .....
>
> You've indicated that the structure must have watertight integrity. What provisions will you have in testing the structure during fabrication?

- *There are standard tests for watertightness that can be performed during modification (ABS Rules for Building and Classing Steel Vessels).*

>
> You've stated in Section 5 that the floatation and Ballasting System will have only single valves to control each compartment. What provisions will be made to filter the small diameter discharge lines to keep from plugging (with mill scale, grinding dust, slag chippings, etc).

- *These are not small diameter discharges (6" and larger).*

- Would it be advisable to individually gauge the compartments for leveling, plumbing, etc during the in-place installation of the structure? This would also allow for leak-testing prior to floatation for transport.
- *A gauging system can be added at an additional expense and increased maintenance demand. The mechanical system is based on that used for the floating maintenance bulkheads. These bulkheads have about the same operating frequency and do not have gauging systems.*

- What is the estimated service life of this structure?
- *At this time, we are assuming a 3 year life, given that this is only a test fixture for evaluating more permanent modifications to the skeleton bays.*
-
- Should protective coating be applied internally as well as externally? *Since the life of the structure is planned to be only 3 years, no internal protective coatings are envisioned. If the life is to be extended these coatings can be added later.*

> Some external anchorage points should be included to compensate for "full sail" effort of the floating structure. Probably will be a multiple tug rigging....or a (hydraulic or screw) jack type fine movement anchorage. This will definitely require close weather coordination with wind and wave phenomena at this site.

Mooring points will be added to the design for connection of tug. Hold backs at the pier noses can be used for fine tuning horizontal location during installation. Due to the large vertical loads, shims will be required for vertical position alignment.

APPENDIX B

Project Review Meeting Reports

90%PRM REPORT

Project Name : John Day Surface Bypass Removable Spillway Weir Design

Contracts : DACW57-97-D-0003, TASK ORDER NO. 21

Meeting Date : November 28, 2000

Location : Corps of Engineers, Portland District Offices, Summit Room

Subject : 90% Project Review Meeting

Attendees: Steve Rainey, NMFS; Jim Stow, USFWS; Tom Lorz, CRITFC; Kim Fodrea, BPA; Matt Hanson, Dave Illias, Blaine Ebberts, Randy Lee, Chris Goodell, Diana Modini, Corps of Engineers; Ed Zapel, Northwest Hydraulic Consultants; Lee Miesbauer, CivilTech; Justin Morgan, The Glosten Associates; Dennis Dorratcague, Montgomery Watson;

Conclusions and action items reached in the meeting are underlined in the report below.

The meeting started at 9:35 AM.

After introductions Dennis Dorratcague described work to date. Dennis stated that the objective of the meeting was to obtain comments to finalize the DDR. This is important since the next step is to start plans and specifications in early January.

Comments from the Corps and ITR on the John Day RSW DDR 90% submittal are due in by the end of the week, December 1. The agency comments are due before Christmas. Blaine handed out Section 2, Biological Considerations. This section was not included in the 90% bound DDR.

Hydraulics

Ed Zapel stated that the hydraulic model tests of the selected alternatives are now underway. In addition to water levels and velocities, pressures on the spillway and RSW at critical locations will be taken. This will aid in analyzing cavitation potential.

Steve Rainey mentioned the need to have CFD modeling done so that velocities and accelerations in the flow could be compared to fish movement obtained by the 3-D sonic tag tracking. Blaine mentioned that there needed to be some discussion between the Corps and the Fishery Agencies regarding the value of the CFD and sonic tag integration. NWW will be using this technology at the Lower Granite RSW, and the value of it will be taken into account when considering the biological testing for the John Day RSW. Steve also stated that if the CFD was performed on the RSW, that it could also be performed on the skeleton bay design for comparison. Ed Zapel said that the velocity collection program in the forebay of the general model must be designed to accommodate calibration of a future CFD model (I thought that ED said that the velocity measurements will be performed so as to calibrate a CFD model if necessary)

Ed Zapel mentioned modeling to be performed under a contract modification would define the height, length, and entry radius of the flow deflector. Steve Rainey asked that the operating tailwater elevations and spillway flow patterns be discussed and the elevation of the flow deflector be reevaluated. Steve Rainey also is concerned about the range of River flows and tailwater elevations that the deflector will be configured for in the modeling effort. He thinks that an appropriate range is from about 150,000 to 300,000 cfs. The Corps stated that this information will be discussed at another meeting and the Agencies will be involved in the decision making process.

Jim Stow suggested that the downstream end of the tailpiece could be designed to provide an air supply to eliminate cavitation. Ed Zapel stated that the cavitation potential will be evaluated and air supply could be added if necessary.

Jim Stow also questioned whether the loss in spillway capacity has been coordinated with NWD. At this time, NWP was waiting for the information that NHC will provide on the capacity of the tailpiece section, the loss in capacity due to the RSW, and a determination on the time it takes to remove the RSW and tailpiece. Once all this information is ascertained, NWD will be contacted about temporary spillway capacity reduction. It is still likely that NWD will allow the test due to the fact that there is adequate time to remove the RSW and tailpiece prior to a PMF event. NWP Hydraulics will perform the coordination once the information is available.
Dennis Dorratcague gave an overview of the structural design showing the three main parts of the RSW – main structure, tailpiece, and main structure attachment.

RSW Main Structure
Justin Morgan described the design of the main structure. He described the new items designed since the 60% submittal. These included the seal around the perimeter of the main structure. He also described the installation and removal using a tug since the barge, proposed in the 60% submittal, is no longer required. It was decided that the RSW will likely be installed once and will be maintained in place for the 3-year test life of the structure. A 24-inch diameter pipe would be installed across the top of the main structure to help with installation. The centerline of the pipe would be at elevation 272.0 with the bottom at elevation 271.0. After some discussion it was decided that the pipe would be high enough out of the water to avoid being snagged by floating debris, since the forebay pool is maintained between about 262 and 264.

During installation the main structure will be rotated into position for installation on the spillway by flooding tanks in succession. Dennis and Justin said that the rotation process would be completed in 1/2 to 1 hour. This is incorrect it will take about 6 hours to complete the rotation of the main structure.

During installation divers would be working directly in front of the closed tainter gates. Leakage at the gates could be a possible source of danger for divers. It was agreed that this could be an issue and should be addressed as concern for the Construction Contract.

It was mentioned that the design life of the RSW is three years and that painting the interior at the end of the three-year test could extend the life considerably, if it is decided to continue to use the RSW. Steve Rainey said that the cost of the structure with interior painting done during initial fabrication versus done later should be compared. The decision to paint the interior of the RSW should also be discussed in a forum due to the potential for using the RSW after the 3 year test life and the high cost of painting after installation.

Tailpiece

Lee Miesbauer described the tailpiece design. It consists of girders spaced at 5 feet along the spillway and anchored to it. Steel plates will be welded on the girders to form the spillway. Seals will be placed along the upstream bottom and sides of the tailpiece, so that the tainter gate and tailpiece will form a watertight seal. Modeling work will be performed under MOD 3 to test the ability of the spillway with tailpiece installed to pass floods. The tailpiece can be removed in a timeframe of about one week if its removal is required to pass floods.

Main Structure Attachment

Lee Miesbauer described the main attachment, which will be installed about 90 feet underwater by divers. It will have its own seal system and will provide the face for sealing the bottom of the main structure. Dennis Dorratcague mentioned that the DDR recommends an underwater survey of some type prior to final design to ascertain any irregularities which might affect sealing any part of the RSW.

Spillway Deflector

The spillway deflector shown in the 90% DDR is an estimated geometry. The actual height, length, and entry radius will be determined in modeling to be performed under MOD 3. It is there to provide a reasonable cost estimate of the deflector.

A large bulkhead will be necessary to dewater the area for construction of the deflector. Since the deflector expected to be long (about 30 feet), the spillway must be dewatered almost to the floor of the stilling basin. As an alternate to a new bulkhead, Dennis mentioned that Mountings on the back of the powerhouse can be installed and the existing spillway stop logs could be used. Matt Hanson mentioned that under the contract envisioned, the contractor would be responsible for dewatering. A method of dewatering must be demonstrated in the DDR to show that it could be done and to develop a cost.

Schedule

Dennis Dorratcague described the overall schedule for completion of the project by April 1 2001. He said that in order to meet the deadline plans and specifications of the main structure would have to begin in early January. A scope of work for the plans and specifications would soon be issued and the work order negotiated. Plans and specifications for the deflector and tailpiece could not begin until modeling for these items is completed. The schedule shows that information for final design of the deflector and tailpiece would be available in 75 and 90 days after start of modeling, respectively. Dennis mentioned that the DDR would not be completed until the modeling is completed, so that the modeling report could be incorporated into the final DDR. This would take to about June of 2001. Matt mentioned that the DDR must be completed and signed by the Corps before advertising for bidding. So, perhaps the DDR could be completed earlier and the modeling report added later as an addendum, or it could be submitted as a separate document.

It was agreed to that the installation of the RSW should be completed before April 1 to allow testing of the RSW before it is put into service. This testing would consist of release of balloon tagged fish to see if they sustain injury in passing the RSW. The contract end date will be set to be at 1 April for the RSW installation.

The meeting adjourned at about 1:00 PM.

MONTGOMERY WATSON

60%PRM REPORT

Project Name : John Day Surface Bypass Removable Spillway Weir Design

Contracts : DACW57-97-D-0003, TASK ORDER NO. 21

Meeting Date : September 26, 2000

Location : Corps of Engineers, Portland District Offices, Summit Room

Subject : 60% Project Review Meeting

Attendees: Gary Fredericks, Steve Rainey, NMFS; Matt Hanson, Dave Illias, Mark Dasso, Bob Buchholz, Natalie Richards, Diana Modini, Blaine Ebberts, Cal Sprague, Corps of Engineers; Ed Zapel, Northwest Hydraulic Consultants; Lee Miesbauer, CivilTech; Justin Morgan, The Glosten Associates; Dennis Dorratcague, Montgomery Watson;

Conclusions reached in the meeting are underlined in the report below.

The meeting started at 9:35 AM.

After introductions Dennis Dorratcague described work to date. The 30% submittal was mainly hydraulics except for description of three RSW mounting options. This 60% submittal is the first one that contains structural information and does not contain design details. Dennis stated that the objective of the meeting was to define the project in sufficient detail to proceed to the 90% submittal.

Matt Hanson gave an update on the Lower Granite RSW work. Bids were opened last week and the low bidder was Dick Corporation with Thompson Metal Fabrication at $9.5 million. The next bid was at $13 million.

Comments from the Corps and ITR on the John Day RSW DDR 60% submittal are due in by the end of the week, September 30. The agency comments are due in two weeks on October 10.

Ed Zapel reviewed the proof of concept and optimum RSW alternatives presented in the 60% submittal. Steve Rainey was concerned about the direct comparison of unit discharge between the selected Alternative 5 (Optimum RSW Alternative C, Figure 3-8) and the Skeleton Bay (Optimum RSW Alternative D, Figure 3-9). Ed stated that the scope of work of the modification under review allows for comparison since the RSW and skeleton bay surface spillway will be in different flumes allowing for a direct comparison. Steve was also concerned that the deflector elevation and length and the possible entrainment of water from the powerhouse tailrace be modeled. These will also be evaluated in the scope modification.

Steve Rainey also said that the NMFS' WES model trip report should be included in the submittal. Steve was concerned that biological considerations were not in the submittal. Matt stated that this was not part of the scope of work for the AE. Steve and Gary Fredericks said that they were concerned that the high unit discharge (15,000 cfs in Bay 20) might prove to be injurious to fish.

Structural Design

Dennis Dorratcague gave an overview of the structural design showing the three main parts of the RSW – main structure, tailpiece, and main structure attachment. The upstream face is cut under (not vertical) for structural purposes. This is different than the hydraulics drawing on Figure 3-8. Steve Rainey stated that this feature seemed to improve the hydraulics for the Lower Granite RSW. Dennis pointed out that the details such as seals, tolerances, and the interface between the main structure and tailpiece has not been developed yet. These features will be shown in the 90% submittal.

Justin Morgan described the main structure and how it is to be installed and removed. Gary Fredericks said that the installation barge cannot remain anchored in the pool above the dam. After some discussion it was decided that the RSW would remain in place for the three years of testing envisioned at this time. If it is to be removed at all, only the main structure will be installed and removed once per year, and the tailpiece will be left installed on the spillway for the entire 3 year duration of the testing program. Dennis indicated that a tugboat rather than a barge might be the best means of installing the main structure. At present, the RSW main structure design assumes that it would not be rigidly attached to the dam, but that its weight would be sufficient to hold it in place. This assumption may be modified as more detailed design effort is accomplished. Justin said that a compressor would feed pressurized air to a manifold, and valves on the manifold would direct air to open vent and filling valves to ballast the main structure. Justin also described the remaining design issues that will be addressed in the 90% submittal.

Lee Miesbauer discussed the design alternatives for the tailpiece. The structures were similar, but the alternatives differed based on the method that would be used to remove and reinstall it. Since the tailpiece would remain in place for the three years of testing, it was decided that it would not have to be designed to be removed within a two-day period, which was the original criterion. If an emergency situation arose it could be cut up and removed in one to two weeks. Bob Buchholz said that he would check with the hydrologists to get an estimate of the flood warning time that would be available for RSW removal.

Steve Rainey was concerned about sealing the RSW and water flow through the stoplog slot. Concern was also expressed regarding the ability of the two RSW pieces to match up and if any means of adjustment could be used. The approach to these details will be addressed in the 90% submittal.

Schedule

Dennis Dorratcague described the overall schedule for completion of the project by April 1 2001. Dave Illias said that the BCOE and bidding period, up to notice to proceed for construction, would take 7 to 8 months, not the time shown on the schedule in the 60% report. This would only allow about 2 months for final design. Dave Illias indicated that final design would likely take up to 5 months, not 2. Dennis said that the schedule would have to be redone taking into account the proposed modeling scope modification. The critical path runs from the modeling through plans & specifications and the main structure fabrication. Dennis said that the schedule should be revisited soon in light of the new modeling scope.

The DDR schedule is as follows:

- November 15 – 90% submittal turned in to Corps
- November 28 – 90% PRM

The meeting adjourned at about 2:00 PM.

MODEL VISIT AND 30%PRM REPORT

Project Name : John Day Surface Bypass Removable Spillway Weir Design

Contracts : DACW57-97-D-0003, TASK ORDER NO. 21

Meeting Date : August 7 & 8, 2000

Location : Northwest Hydraulic Consultants Laboratory, Vancouver, B. C.

Subject : Site Visit and Model Alternative Meeting

Attendees August 7 & 8: Gary Fredericks, Steve Rainey, NMFS; Tom Lorz, CRITFC; Chuck Tracy, ODFW; Matt Hanson, Diana Modini, Blaine Ebberts, Corps of Engineers; Jim Lencioni, Northwest Hydraulic Consultants; Dennis Dorratcague, Montgomery Watson;

Attendees August 8 only: Lee Miesbauer, CivilTech; John Springer, The Glosten Associates;

Conclusions reached in the meeting are underlined in the report below.

August 7, 2000

The meeting started at 1:00 PM.

Dennis Dorratcague presented a brief discussion of the modeling work completed to date and the model nomenclature. The nomenclature used for the various alternatives is described in the table below. The alternative numbers follow those in Appendix B of the 30% submittal.

Alternative	Discussion
Alternative 2	Alternative chosen at the Site Visit & Alternatives Meeting on May 9, 2000 for study in the hydraulic model.
Alternative 7	Alternative added for model study subsequent to the May 9 meeting. It is the same as Alternative 2 with the piers extended upstream and a ramp upstream of the RSW ogee.
Alternative 5	This is the same Alternative 5 as in Appendix B of the 30% report.
Alternative 2 with fillet	This is Alternative 2 with a straight length of spillway installed tangent to the RSW and existing spillway. This was done to take away the recurved section where the RSW met the existing spillway. It also happens to be almost identical to the RSW spillway shape of Alternative 5 in Appendix B of the 30% report.

The rest of the meeting was spent viewing the operation of models of Alternative 2 and Alternative 2 with fillet.

August 8, 2000

The meeting began at 8:00 AM.

A brief discussion of the model runs and the remaining issues to be investigated in the lab was held in the conference room. Then the group went to the lab to look at the model of Alternative 2 with fillet again to inspect the disturbances around the spillway piers and how they propagate down the spillway and onto the 30-foot spillway deflector. Another reason for re-running the model was to look at the flow off the deflector over a range of tailwaters deemed to be more likely under RSW operating conditions.

After viewing the model the meeting re-convened in the conference room for the Project Review Meeting (PRM). Dennis Dorratcague stated that the objective of the meeting is to select an RSW geometry so that the schedule of operating an RSW in April 2002 could be met.

Everyone agreed that Alternative 2 with fillet is the selected geometry.

Gary Fredericks and Tom Lorz were concerned with the turbulence that they observed off the end of the spillway deflector in the model. Much of the discussion centered on the turbulence at the operating tailwaters that the RSW would experience and whether the disturbance coming off the spillway piers adds to this turbulence. In the model runs on the morning of August 8 cross sections of the water surface elevation on the 30-foott deflector were taken with the Alternative 2 RSW both with and without upstream pier extensions. Although the extensions reduced the disturbance around the piers (1-foot versus 4-foot maximum drawdown), little difference in the water surface profile on the deflector was measured. So, it was concluded that upstream pier extensions were not necessary, and that the Alternative 2 with fillet could proceed without modification.

The need for more modeling of the deflectors at both Spillway Bays 1 and 20 were discussed. Diana Modini said that she is writing a scope of work to provide more modeling of the deflectors. It was agreed that the model should investigate the elevation, length, and upstream radius of the deflectors. Increasing the upstream radius might decrease the effects of standing waves from upstream disturbances that appear to become magnified on the deflector. It was decided that a decision on the deflector design would have to wait until modeling data could be collected on various deflector designs.

There was also a discussion on how well the present spillway/deflector represented the geometry of the skeleton bay spillway. Dennis will provide a drawing showing profiles of both the Alternative 2 with fillet and skeleton bay spillways superimposed on each other. Gary Fredericks and Steve Rainey both thought that it was necessary to have a sectional model of the skeleton bay spillway since the skeleton bay is only modeled in the general 1:80 scale model. All Babb of NHC said that if skeleton bay modeling is authorized within the next two to three months the flume housing the McNary spillway could be used. This flume would have a scale of 1:25 slike the present RSW model. This would allow both the RSW and skeleton bay models to be viewed simultaneously.

Lee Miesbauer described the connection alternatives in the 30% report. Dennis Dorratcague described various scenarios for removing the RSW of Alternative 2 with fillet. This discussion of the structural issues is for informational purposes only at this point. The 60% submittal will discuss the structural alternatives including the hydraulics of leaving a small piece of the RSW in place semi-permanently.

Schedule

The schedule for completion of the DDR was discussed. About six weeks have been lost from the schedule in modeling the four alternatives mentioned above. The DDR would be completed in December. See the revised schedule below. The 60% submittal would be submitted to the Corps on September11. The 60% PRM was scheduled for 9:30 AM on September 26 at the Corps offices in Portland. The next trip to WES to view the general model cannot take place until early October since the bathymetry is being reinstalled and the model re-calibrated.

The meeting adjourned at 12:00 PM.

John Day RSW DDR
Revised Schedule

Task	Date
30% Submittal PRM	8-Aug-00
60% Submittal	12-Sep-00
60% PRM	26-Sep-00
90% Submittal	7-Nov-00
90% PRM	21-Nov-00
Submit 100% DDR	4-Dec-00
Final Documentation	10-Dec-00

SITE VISIT AND KICKOFF MEETING REPORT

Project Name : John Day Surface Bypass Removable Spillway Weir Design

Contracts : DACW57-97-D-0003, TASK ORDER NO. 21

Meeting Date : May 9, 2000

Location : John Day Dam

Subject : Site Visit and Model Alternative Meeting

Attendees : Ed Meyer, Gary Fredericks, Steve Rainey, NMFS; Tom Lorz, CRITFC; Matt Hanson, Bob Buchholz, Diana Modini, Blaine Ebberts, Dick Leatherbury, Corps of Engineers; Lee Miesbauer, CivilTech; John Springer, The Glosten Associates; Ed Zapel, Northwest Hydraulic Consultants; Dennis Dorratcague, Montgomery Watson.

The meeting started at 9:45 AM on May 9, 2000. Action items to be performed by the indicated individuals are underlined in the meeting report description below.

Project Overview

After introductions, Matt Hanson and Dennis Dorratcague provided an overview of the surface collection and bypass efforts at the John Day Project. This removable spillway work is being performed as a result of the skeleton bay surface bypass spillway Feature Design Memorandum No. 52. This design memorandum was completed and estimated Skeleton Bay modifications for a surface bypass spillway to be: $54 million for one bay and $85 million for 2 bays. Prior to making such a large capital investment in the skeleton bay concept, the John Day RSW was conceived to test the effectiveness of a surface spillway outlet. . The ultimate goal of this work order is to determine RSW geometry to simulate a skeleton bay surface bypass spillway which includes an ogee type crest and a 30 ft long flow deflector in Bay 20 of the John Day spillway.

Walla Walla District is pursuing a similar concept at Lower Granite Dam which will be tested in April 2001. The Lower Granite RSW is being designed for 6,000 cfs, while this John Day design is for upwards of about 14,000 cfs. The Lower Granite RSW Plans and Specifications are to be completed in July 2000.

The objective of the meeting was to familiarize the team with the John Day site and to select the top 2 goemetries from the 6 alternative Removable Spillway Weir (RSW) alternatives. The number one choice would be modeled and the second choice would be modeled if necessary. Prior to any discussion a tour was held at Spillway Bay 20 at the north end of the powerhouse. Following this, the alternatives were evaluated and geometries were selected.

Criteria

Ed Zapel led a discussion of the initial criteria used in developing the RSW alternatives. The criteria are listed below:

1. A submergence of 22.5 feet. (i. e. spillway crest to water surface is 22.5 feet)
2. To be located upstream of spillway gate (use existing spillway gate for flow control)
3. Flow acceleration to be 0.1 feet per second per foot, if possible
4. Flow would be around 14,000 cfs (similar flow rate per foot of width as the skeleton bay surface bypass spillway design)
5. Remove or install the RSW in less than 24 hours
6. Maximum operating pool is 268.0. (Normal operating pool during the fish passage season is elevation 264.0)
7. Minimum operating pool is 257.0

Steve Rainey (NMFS) indicated that RSW discharge as % of total spillway Q is an important factor in design of the RSW configuration. RSW spill during low spillway flow is most important, since the RSW would pull a significant portion of the total spill for the project at that time. Steve Rainey anticipates that when spill volume through the RSW is less than about 40% of total spill, the RSW will no longer be effective, since other spillway bays will mask the flow field entering RSW.

Design criteria may change to fixed discharge capacity, not necessarily fixed submergence. The group debated whether it was more important to match the discharge capacity of the Skeleton Bay concept rather than the submergence. The consensus following the discussion was to attempt to match the discharge through the skeleton bays with the RSW because the discharge is what develops the attraction flow field in the reservoir.

Alternative Discussion and Selection

Ed Zapel then presented the six RSW Alternatives that he prepared. These are stated below:

Alternative 1 John Day Skeleton Bay Geometry
Alternative 2 Lower Granite Flow Efficient Design
Alternative 3 Lower Granite Fish Efficient Design
Alternative 4 Lower Granite Flow Efficient Design with Step at Transition to Existing Crest
Alternative 5 RSW with Semi-permanent Lower Crest
Alternative 6 Lower Granite Fish Efficient Design with Step at Transition to Existing Crest

The hydraulic discussion centered on two issues: the flow field created in the forebay and the hydraulics on the spillway particularly at the juncture of the RSW and the existing spillway. It was decided that the RSW design should pull proportionally more flow from the surface, preferably from within the top 30 feet of reservoir. The RSW should emulate the Skeleton Bay as much as possible in that it draws from near the surface, and is as near to the existing crest of the spillway as possible. Concepts that extended well out into the forebay were undesirable as they do not appear to emulate the flow field of the skeleton bay surface bypass spillway.

Once preliminary tests and evaluations are completed the modeling work will be used to refine the RSW design. It is estimated that refinement of the upstream section of the RSW crest can optimize the amount of flow from the surface (top 30 feet of the water column

The shape of the RSW is also important for floatation purposes. The geometry will have to be evaluated with respect to how the RSW will be floated through the navigation locks prior to installation in Bay 20. The draft restriction from going through the locks may limit the geometry of the upstream face of the RSW.

A concern was raised that, if the RSW piers are too close to the crest, there will be drawdown and turbulence near them. Ed Zapel stated that the piers shape will be determined following the selection of the crest location. The pier shape is dependant upon the velocity of flow past them and their respective location to the crest of the RSW. The pier shape will be calculated according to hydraulic criteria and evaluated in the model. At this time, the pier shape is a refinement that will come later as the design progresses.

A question was raised as to the percent of the time that the pool elevation is at certain elevations especially during the migration period from April through August. The Corps will supply data to Montgomery Watson for analysis and development of stage frequency curves.

NMFS did not have a favorable opinion of Alternatives 4 and 6, which had an aeration step. Their opinion was that it might lead to unsafe passage conditions for downstream migrants. However, Ed Zapel said that the flow at the end of the RSW is fast enough to produce cavitation and that providing air might be necessary to limit this potential. At this time, Alternatives 4 and 6 will not be evaluated further unless the cavitation potential from model evaluation is high.

To select the alternative to be tested in the hydraulic model some criteria were put forward. These were:

1. Zone of influence
2. Flow field intensity
3. Pull more flow from top 30' of pool
4. Match skeleton bay hydraulics
5. Same discharge capacity vs same depth of submergence
6. Close to powerhouse
7. Pier nose shape

By process of elimination, Options 1, 3, 4, 5 and 6 were dropped. Options 1, 3 and 6 were dropped because of the distance they reached out into the forebay. Options 4 and 6 were dropped because of the unacceptable aeration step. Option 5 was dropped because it was felt that the semi-permanent decrease in spillway capacity would be unacceptable. Alternative 2 was selected for further study.

Schedule

The schedule was discussed. Bob Buchholz said that scheduling due to bathymetry changes to the John Day model at WES will affect the overall schedule of the trips to WES. The model schedule and the

RSW design schedule will be coordinated and the timing for the trips to WES will be evaluated as to when the trips can occur. The initial concept was to view the upstream zone of influence in the general model prior to the bathymetry changes because the tailrace bathymetry will not have an effect on the upstream ZOI. Following the model bathymetry modifications, the downstream tailrace and spillway flow patterns will be evaluated. The table on the following page shows the schedule as defined in the scope of work. The schedule may change in some fashion following coordination of WES modeling concerns.

The meeting adjourned at 2:30 PM.

John Day RSW DDR
Schedule

Task	Date
NTP	12-Apr-00
Develop Alternative Evaluation Submittal	9-May-00
Model Alternative Meeting	9-May-00
COE Vancouver Model Visit	6-Jun-00
COE First WES Model Visit	13-Jun-00
30% Submittal	13-Jun-00
30% Submittal PRM	27-Jun-00
Second Vancouver Model Visit	27-Jul-00
Second WES Model Visit	1-Aug-00
60% Submittal	1-Aug-00
60% PRM	15-Aug-00
90% Submittal	26-Sep-00
90% PRM	10-Oct-00
Submit 100% DDR	23-Oct-00

APPENDIX C

Correspondence

MEMORANDUM FOR RECORD

FROM: Diana Modini

SUBJECT: John Day RSW alternative 2 – NHC Trip Report for 20-22 June 2000

DATE: 5 July 2000

Attendees: Diana Modini, NWP
Brad Bird, NWP
Matt Hanson, NWP
Blaine Ebberts, NWP
Ed Zapel, NHC
Al Babb, NHC
Dennis, MW
Dick Regan, NHC
Jim Lencioni, NHC
Ken, NHC

Trip Objectives:
The objectives of this trip was to observe RSW alternative 2 and the deflector (L=30ft, R=30ft, El=148) performance in the 1:25 sectional model and to determine whether to pursue this alternative further.

Observations & Discussions:
In general the model was operated with a forebay of 262.2 to 264 with a maximum at 268, while the tailwater ranged between 158 to 168.

When observing the flows upstream for alternative 2, the water surface was depressed at the pier nose which appeared to caused a "rooster tail" effect along the side of the spillway. It was also hypothesized that the double ogee (double inflection) accentuated the roostertail. At this time biologist speculate that this "rooster tail" may cause injury to fish. This was due in part because the roostertail impacted directly on the flow deflector. If this were true then the goal would be to eliminate the "rooster tail". See pictures 1-3.

In an attempt to eliminate or reduce the "rooster tail" various wood pieces were placed at the pier noses at different angles. In addition the half spill bay was opened two stops to observe the flow separation and the "rooster tail". There were no significant changes, therefore a rough alternative 7 (wood ramp and extended piers) was built and installed in the model. It too had "rooster tails" along the sides of the spill bay and in the middle. In addition the ramp was removed from RSW alternative 7. From the "rooster tail" standpoint the rough alternative 7 was worse than alternative 2. In addition, the data points taken for the rating curve, for both alternatives, showed that the rough alternative 7 was no more efficient than alternative 2. Hence it made sense to drop alternative 7, yet this was to be discussed as a team with Bob Buchholz with a decision to NHC.

A discussion followed the observations which RSW alternative 4 (aeration step with 41 ft piers) was thought to reduce or eliminate the "rooster tails" plus reduce the possibility of cavitation down the spillway ogee. Blaine stated that some of the fishery managers did not approve of this design because of the potential of fish injury. However, the potential for fish injury with Alternative 4 is just a hypotheses at this point, and the concept needs to be looked at in the model to better evaluate the hypothesis. In addition a "flat plate" (fillet) could be added to RSW alternative 2 to reduce or eliminate the "rooster tails", but the cavitation issues would need to be addressed for this alternative.

The question arose as to how much general modeling had been completed with the Surface Bypass Skeleton Bay at WES and whether the rooster tail was observed. No notes of a rooster tail have been found.

The following are the data points taken from the sectional model to produce rating curves for RSW alternative 2 and rough alternative 7 which will be used during the WES trip the following week.

For RSW alternative 2:

FB	Q	TW	
267.75	19,000	167	(pictures 4 and 5)
254.1	15,500	167	
262.25	13,100	167	
266	18,000	167	(pictures 6, 7, 8)
263.25	14,700	167	

For RSW rough alternative 7 (pictures 11, 12, 13)

FB	Q	TW
269	22,600	160
267.5	20,800	160
266.5	18,700	160
262.3	13,200	160

RSW rough alternative 7 w/o ramp (pictures 14, 16, 17)

FB	Q	TW
262.0	13,200	160
263.25	14,500	160
267	19,200	160
264	15,200	160
165,500	17,500	160

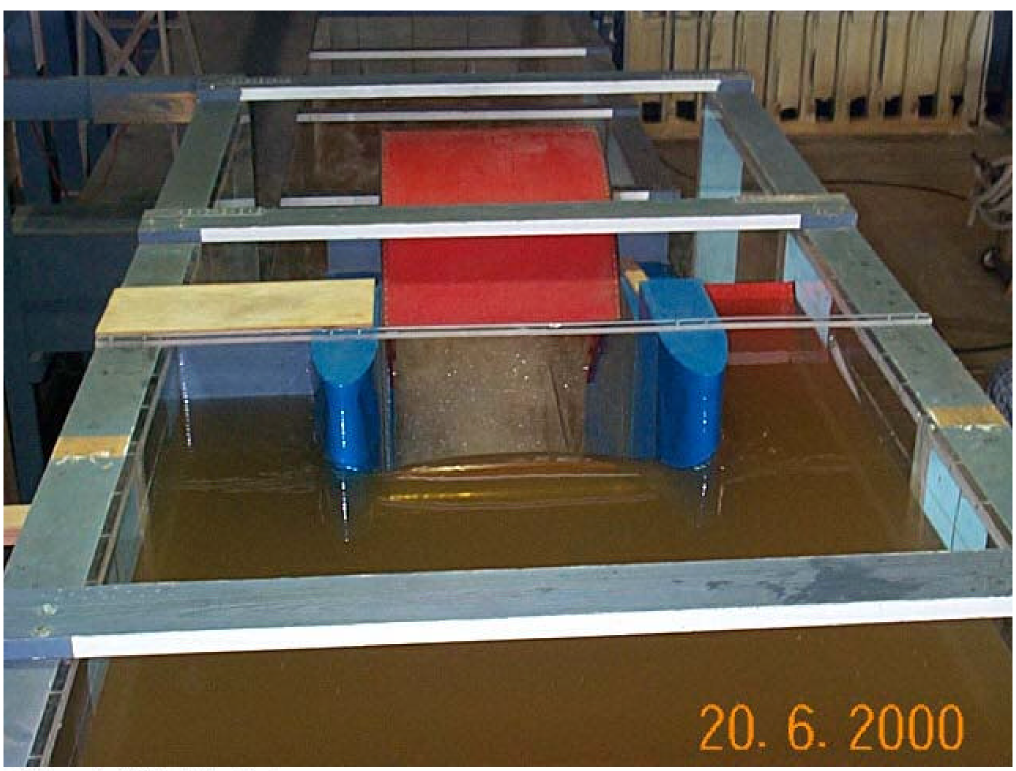
In all cases the deflector performance produced somewhat of a wave, which didn't appear to be appropriate. In addition we roughly approximated normal spill (with tainter gate down) and also found the performance to be worse.

Recommendations and Action Items:
Based on the observations and rating curves, rough alternative 7 should be dropped, since it wasn't more efficient in the forebay and the "rooster tails" were worse. Alternative 2 is efficient and met criteria, yet "rooster tails" were present, in which we may want to eliminate by placing a fillet so that there isn't two inflections where the RSW meets the spillway. From a team meeting (I not present) held the following day after the NHC trip a decision was made to build alternative 4 and a fillet for RSW alternative 2. Cavitation issues should be addressed for alternatives. In addition we should develop a new scope (mod) to determine a proper deflector with and without the RSW in spillway 20. This would include documenting different elevations and lengths (possible radius?) and obtain a recommendation.

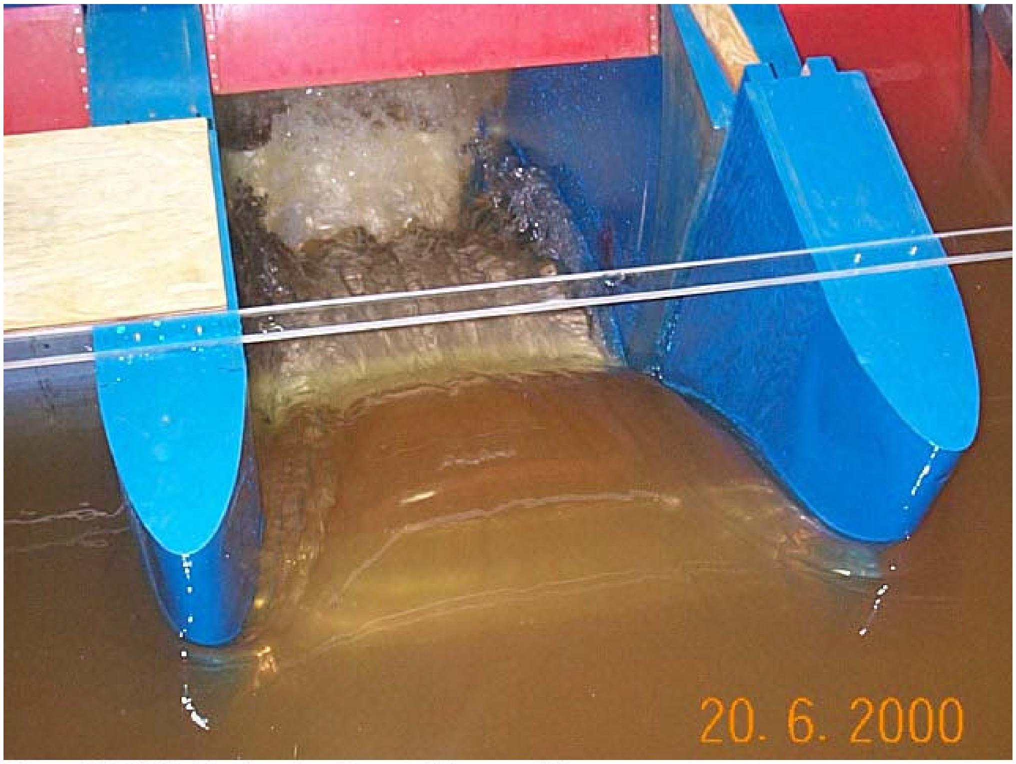
Picture 1. RSW alt 2. Water surface, drawdown around piers.

Picture 2. RSW alt 2. "Roostertails"

Picture 3. RSW alt 2. Deflector performance.



Picture 4. RSW alt 2. Forebay.

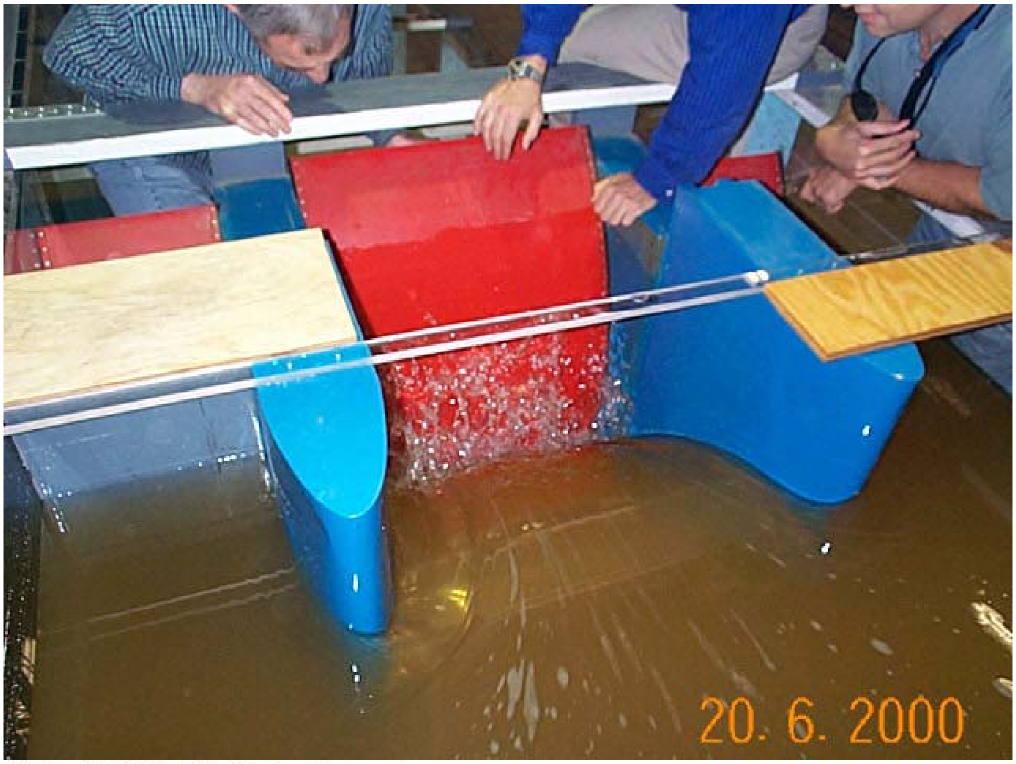


Picture 5. RSW 2. Forebay, drawdown, and "roostertails".

Picture 6. RSW 2. "Rooster tails".

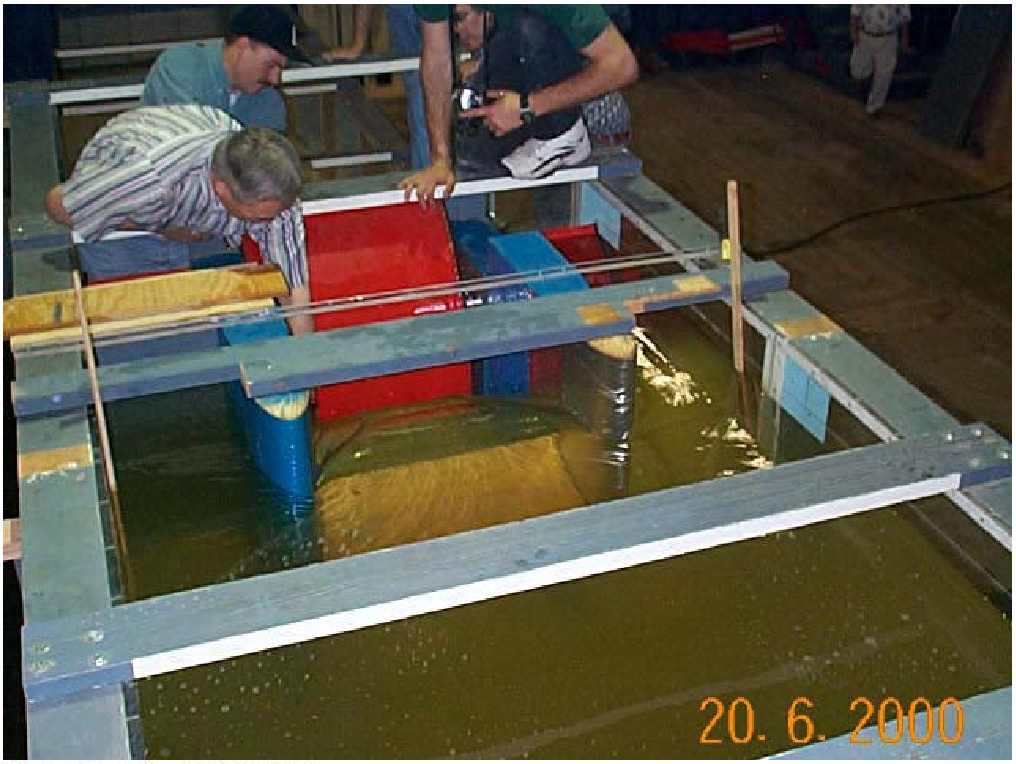
Picture 7. RSW 2. "Rooster tails" and deflector performance.

Picture 8. RSW alt 2. Deflector performance.



Picture 9. RSW alt 2. Closing tainter gate.

Picture 10. Closing tainter gate to observe a normal spill.



Picture 11. "Rough" RSW alt 7.

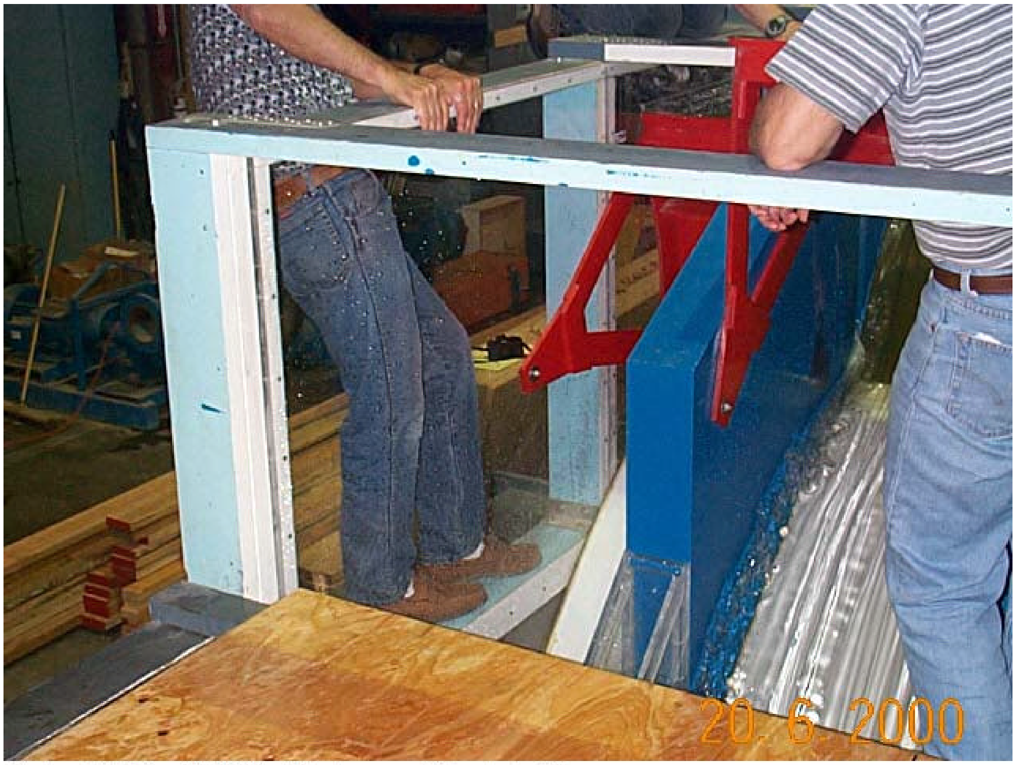
Picture 12. "Rough" RSW 7. Double ogee (inflection) and "rooster tails".

Picture 13. "Rough" alt 7. Deflector performance.

Picture 14. "Rough" RSW alt 7 w/o ramp. "Rooster tails".

Picture 15. "Rough" RSW alt 7 w/o ramp. Forebay.

Picture 16. "Rough" RSW alt 7 w/o ramp. Double ogee and "rooster tails".



Picture 17. "Rough" RSW alt 7 w/o ramp. "Rooster tails".

Cover Letter
To: Davis Moriuchi
From: Jim Ruff

RE: John Day Removable Spillway Weir (RSW)

Dear Mr. Moriuchi:

The purpose of this letter is to present comments and recommendations on the John Day RSW at an early phase of design development. The attached in-house memo describes our comments in more detail. The highlights of that memo include:

1. In 1999, the National Marine Fisheries Service (NMFS) proposed to the System Configuration Team (SCT) the idea of installing a RSW prototype model in spill bay #20 at John Day Dam for testing in 2002. The intent was to determine the encounter rate of migating juvenile fish, and what percentage would pass over the RSW. Evaluation results would then lead to a decision whether to proceed with costly skeleton bay design and construction. The SCT endorsed RSW prototype testing at John Day. Counter to what the district has acknowledged, a decision to proceed with deployment of one or more permanent RSW's across the spillway was also part of that proposal. This will allow the flexibility to decide in late-2002 to proceed with either one or more skeleton bays, one or more permanent RSW's, or to both alternatives. We recommend the district adjust its developmental approach accordingly.
2. We recommend proceeding into sectional modeling with the Northwest Hydraulics, Inc alternative #2, which includes a nominal 26-ft pier nose extension (relative to the existing pier nose) . We also recommend a contrasting pier nose extension of nominal 45-ft length, with a ramp to increase the horizontal distance from the RSW crest to the pier nose, be investigated preliminarily in the sectional model. By comparing and contrasting two different RSW configurations, design development can proceed on a more informed basis.
3. We recommend adding forebay computational fluid dynamic 3-D numerical modeling of the immediate forebay area (as defined in the enclosed memo) to the scope of work. This component is viewed as necessary in the context of evaluating the magnitude of the RSW forebay flow field, and assessment of juvenile response to that flow field. Integration of 3-D juvenile tracking with 3-D numerical modeling of forebay hydraulics is increasing precision of our ability to determine more effectively the performance of prototype surface bypass facilities. We don't propose to delay design and construction until after the numerical model has been prepared. Rather, we propose it be prepared in time for usage in the performance evaluation phase.
4. We also recommend that the design and construction of the spill bay #1 deflector be included in this scope of work. It has become clearly apparent from February, 2000 total dissolved gas field testing and spill operations at John Day this spring that a deflector at spill bay #1 is critically important from the water quality, and adult/juvenile fish passage perspectives.

5. We also recommend that the district design team coordinate more closely with the Walla Walla District, which has already entered an advanced stage of design development for its RSW test at Lower Granite Dam in 2001. While we are not inferring that the John Day design should emulate that of Lower Granite, we believe having greater understanding of issues of concern in that design development process would be instructive, efficient, and would lead to a better RSW at John Day.

In closing, we are hopeful that this letter will aid in defining our views at this early stage of design development of the John Day RSW. We look forward to an interactive process, and additional discussion on comments and recommendations presented in this letter and enclosure.

If there are questions or comments, please contact Steve Rainey, 503-230-5418, or Gary Fredricks, 503-231-6855.

Sincerely,

Jim Ruff

cc Gary Fredricks
Tom Lorz, CRITFC
Bob Buchholz, NWP
Matt Hanson, NWP
Blaine Ebberts, NWP

F/NWR5

MEMORANDUM FOR: Hydro-FCRPS Files

FROM: Steve Rainey

SUBJECT: John Day Removable Spillway Weir (RSW)

This first NWP meeting with the agencies and tribes on development of a prototype RSW occurred at John Day. Tom Lorz (Columbia River Intertribal Fish Commission), Gary Fredricks, Ed Meyer, and Steve Rainey (all of NMFS) attended. Blain Ebberts, Bob Buchholz, Diana Modini, and Matt Hanson were present for Portland District, along with Corps design contractors Dennis Dorratcaque (Montgomery-Watson) and Ed Zapel (Northwest Hydraulics). The naval architect and structural design subcontractors were also present.

NMFS has actively participated in design development of a 6000 cfs RSW at Lower Granite Dam, which will be tested in 2001 and is currently in the detailed design and specifications preparation phase. In contrast, the John Day RSW is scheduled for prototype testing in 2002, with an approximate design flow of 14,000 cfs. Other than the naval architect and myself, none of the other parties present at this kickoff meeting have been actively participating in the NWW RSW prototype project. In order to maximize what has been learned in development of the NWW LGR-RSW design, and factor that into design development at John Day, it is imperative that either the two districts maximize coordination and overlap, or that some entity (possibly NMFS), play a key role in assuring that "re-invention of the wheel" is minimized. This is not to say that the John Day RSW should be identical to that at LGR - it shouldn't. The primary goal is to proceed with design development at John Day on the basis of a full and complete understanding of LGR-RSW design development.

The purpose of this memo is to present comments and recommendations at an early juncture in the John Day RSW design process. These were not presented by National Marine Fisheries Service (NMFS) at the kickoff May 9, 2000 meeting, since we had not received materials for review in advance, and are only now able to respond more comprehensively to what was presented and discussed. These comments and recommendations are preliminary, and are envisioned as a basis for future interactive discussions with NWP that we believe will result in a better prototype design and performance.

Background

John Day RSW - The System Configuration Team (SCT) agreed with a 1999 NMFS proposal to prototype-test a RSW to determine the percentage of juvenile fish approaching the project that would readily perceive the RSW flow field (opportunity for discovery), then successfully pass. The RSW was perceived as a less risky, more cost-effective way to assess whether a single skeleton bay surface bypass system (capacity of 18,900 cfs) should be installed. The prototype cost was expected to be approximately $10 million, compared to the single skeleton bay (1998) estimate of $56 million. At the conclusion of testing, the proposal called for a decision to either build one or more permanent skeleton bay surface bypasses, build one or more permanent RSW's, or abandon the RSW and skeleton bay concepts.

Perceived RSW and Skeleton Bay Surface Bypass Potential

Deep wide surface collector slots in front of powerhouses (similar to those at Wells Dam) have been investigated with prototype facilities during the last six years with mixed results. Of those wide, deep entrances investigated, only Bonneville 1st Powerhouse is still being studied; the others have been either abandoned, or are on hold. With a breadth of prototype evaluation experience, it has become more evident that surface-oriented entrances (such as ice and trash sluiceway entrances, the Wanapum sluiceway opening, and the Bonneville 2nd Powerhouse sluice chute) have passed many fish with relative low flow. (In this case, "surface-oriented" refers to an unimpeded surface drawdown hydraulic profile that transitions over a weir crest and becomes super-critical.) Therefore, the RSW was conceived of to present a strong, surface-oriented forebay flow/passage route. It was reasoned that this device may reduce excessive juvenile forebay delay of fish that may be looking for a passage route that more nearly simulates natural conditions. Delay is considered directly related to predation exposure/losses.

Perceived Limitations of the RSW and Skeleton Bay Surface Bypass

Both devices are perceived to be most beneficial when spill is at low levels (less than 40% of project discharge). At higher spill percentages, research suggests that fish will pass through the spillway (and powerhouse) with limited delay, and benefits of the RSW will be negligible. In the context of John Day, where voluntary day spill levels are currently limited by power needs, juvenile passage testing of 24-hour spill up to 30% of project discharge is currently proceeding. The question of whether there will be sustained voluntary day spill at John Day in the future (and how much) is a hotly contested issue. Therefore, even if day spill levels continue at approximately 30% (including surface bypass flow as spill), both the RSW and skeleton bay surface bypasses present the potential of maintaining a higher spill passage rate (spillway efficiency) than with spill only. Part of the rationale for this thinking relates to the high vertical distribution of juvenile fish in the forebay, relative to the deeper spill gate sill elevation, 53' below normal forebay minimum operating pool (el 262.5).

RSW Testable Hypothesis

As referenced earlier, NWP would do well to fully interface with NWW on design development of the RSW at Lower Granite. Some, though not all, questions being addressed at LGR are the same as at John Day. For perspective, the primary questions to be addressed at the Lower Granite RSW test in 2001 relate to enumeration of the percentage of fish passing into the RSW flow field, and the percentage approaching at discreet incrementally-closer distances (e.g., 30 meters, 20 meters, and 10 meters) from the RSW which pass. Another important issue to be resolved at LGR is whether fish that accumulate upstream of units 5 and 6 (due to the influence of behavioral guidance forebay curtain and upper intake occlusions), and approach the powerhouse without encountering the RSW flow field on their initial approach, will meander and enter/accept/pass over the RSW. These fish will have to transcend approximately 200' in a direction perpendicular to powerhouse approach flow in order to enter the RSW flow field. Paraphrased, important questions pertain to what percentage of fish will find and pass the RSW on their first approach, and what percentage will find and pass after accumulating upstream of the powerhouse.

At John Day, the questions are similar. However, the project forebay is much larger and subyearlings (and other stocks such as sockeye) can also potentially be tested. The question of whether the increase in RSW capacity from 6 kcfs to 14 kcfs is large enough to attract a substantial percentage of fish (especially without forebay curtain or upper intake occlusion devices) remains speculative, as it does for the skeleton bay (18.9 kcfs capacity).

Definition of the precise magnitude of the RSW flow field, and precise 3-D tracking of fish responses in that flow field, are best assessed through emerging acoustic tag and numerical modeling integration. (See below).

Anticipated Operations During 2002 Prototype Testing

While this subject is still wide open at John Day, the topic has been discussed at LGR. We expect that the primary tests of incremental RSW-skeleton bay benefit will be during the spring and summer, both day and night. However, since the dissolved gas spill cap at John Day is now at 140-160 kcfs, and assuming no change, night spill RSW and skeleton bay incremental benefits are expected to be negligible. Day spill conditions are expected to continue at approximately 50-100 kcfs during voluntary spill periods, with project flows in the range of 225 - 375 kcfs during the *spring*. We expect this day-time voluntary spill operation to be the focus of incremental benefits assessments of the RSW-skeleton bay surface bypass, relative to spill-only conditions. Further, we expect the blocked spill tests to be with the RSW operating during one block, and the spill bay #20 tainter gate on seal during the other test condition. A forebay numerical model would allow representative forebay hydraulic conditions during both tests to be contrasted, and would allow a better definition of the relative size of the RSW flow field.

During the *summer*, project discharges are expected to range from approximately 150-275 kcfs, with day spills of (roughly) 50-100 kcfs. Again, it is expected that blocked study operations would be with bay 20 and the RSW operating vs bay 20 off. Contrasting forebay conditions during on-off conditions again would allow the upstream influence (size) of the RSW flow field to be defined, and contrasted. Additionally, forebay fish tracking with the RSW on versus off conditions could be contrasted.

Potential John Day Future RSW or Skeleton Bay Deployment

The ultimate potential of the RSW and/or skeleton bay, assuming positive performance in 2002, relates to the possible use of multiple RSW's or skeleton bay surface bypasses to pass an *equivalent or* *larger* number of fish safely over the spillway, relative to a larger spill without the RSW or skeleton bay surface bypasses. This would not only result in fish benefits, but power and water quality benefits (lower tailrace total dissolved gas levels) as well.

Forebay Hydraulic Modeling for the John Day Skeleton Bay - Is More Required?

NMFS participated in the skeleton bay design development process in 1997-98, to the extent possible with limited manpower resources at that time. We did attend a WES trip to observe skeleton bay(s) general model forebay and tailrace conditions, and have observed forebay and tailwater modeling runs with the skeleton bay and RSW at the John Day 1:80 model on other occasions. After reviewing Feature Design Memorandum No. 52, the primary reference to the influence and upstream projection of the skeleton bay flow field (at 18.9 kcfs for one skeleton bay) is Figure 3.3. The contrasting operations were for a total project discharge of 100 kcfs, with the skeleton bay operating, then shut down. Vector arrows for both conditions are presented to depict the influence of the skeleton bay on the near-field forebay flow field. Unfortunately, John Day flows never drop this low in the spring migration period, and rarely get this low in summer until after the summer migration. I am unaware whether additional forebay modeling in the context of the skeleton bay occurred, or whether it was documented. Even if there is additional documentation of other forebay modeling runs, an important question relates to whether model-tested operations were the same as currently being implemented (e.g., day spill to 30%).

Currently Proposed Hydraulic Modeling for John Day RSW Development

Our understanding is that section modeling and some incremental forebay modeling with the 1:80 John Day forebay model is planned to augment design development of the John Day RSW. We know from experience that, although the John Day general model is an excellent tool for tailrace modeling, it is too large for efficient forebay modeling because forebay velocities are low. Each forebay modeling run would take a relative long period for which to generate outputs. In our experience, excessive reliance on any forebay general model often results in the need for relatively crude integration of fish behavior with subjective dye releases to be primary factors in prototype bioengineering performance assessment, especially during WES visits by the agencies and tribes. If a more quantitative, impromptu assessment is needed from the general model, WES is often not able to provide this without a sizable delay.

Integration of Fish Tracking and Forebay Hydraulics Near the RSW

Therefore, we conclude that additional modeling is required to define probable test conditions during prototype testing. Further, we believe the maximum benefit of prototype testing the RSW at John Day will only occur if precise tracking of individual juvenile fish of varied stocks/species can be integrated with numerical modeling outputs. Integration is currently in developmental stages at Rocky Reach, Lower Granite, and Bonneville 1st Powerhouse, and should be a more powerful and refined evaluation tool by 2002. While a physical hydraulic forebay model is suited for subjective concurrent assessments of dye movement and fish behavior, more precise investigations are required to fully extract and characterize whether new prototype systems are creating desired fish passage performance. In contrast, a near-field numerical model (extending several hundred feet upstream from spill bay #20, and from approximately spill bay 17 to turbine unit 15) is probably satisfactory for the 2002 tests, it may be that a subsequent need for a large forebay numerical model (related to possible multiple RSW installation) will be required at some point. Once prepared, the larger forebay numerical model could be used indefinitely for a range of applications relating to fish passage and other investigative needs.

Other Numerical Modeling Benefits

While computational fluid dynamic numerical models take a longer time to assemble than hydraulic models, they are much more flexible once assembled. They can also be used for a broad range of other applications over time, without the inflexibility of general models of a distinct scale. They are also quicker and more user-friendly than hydraulic models in the context of integration of individual fish tracks with flow-fields. This will allow a broader range of adjacent spillway gate and turbine settings to be more readily evaluated. Rather than biologists and engineers jointly trying to synthesize fish behavior on the basis of subjective observation of dye releases at a general model, individual fish tracks can be superimposed in a color-coded velocity or acceleration flow-field numerical model backdrop that allows magnitudes to be immediately identified. They also allow acceleration flow fields to be readily observed, counter to a physical hydraulic model. (There is some potential that juvenile fish may actually be responding to acceleration, rather than velocity, flow fields.)

NMFS-FCRPS Modeling Recommendations

We recommend 3-D computational fluid dynamics numerical forebay modeling at John Day be added to the scope of work, even though we understand it is not currently planned. The basis for this recommendation is our experience with the Lower Granite RSW design. In that case, the integrated use of a 3-D numerical forebay model, a 1:40 forebay model, and evolving sonic tag tracking technology (in conjunction with other established tracking technologies) are expected to provide more precise 3-D fish behavioral tracking, which can be integrated with more precise and quicker quantitative forebay depictions of either velocity or acceleration flow fields to which fish are responding. These are a prerequisite for development of a satisfactory prototype test. Not using tools such as the CFD model is no longer satisfactory once we go beyond the question of *whether* a system performed to a satisfactory level. More precise and comprehensive assessment tools such as CFD modeling are necessary to enhance our chances of determining *why* a particular prototype configuration either did or did not work effectively.

Improving the ability to integrate precise localized forebay hydraulic conditions and individual fish behavior has been encouraged by NMFS (and the Corps) for the last few years, and is considered a much more powerful way in which to assess fish responses to prototype-induced forebay hydraulic conditions than with hydraulic general models alone.

Model Timing versus Project Schedule

We understand that the preparation of a CFD will take time. We propose to proceed with design of the 14 kcfs RSW prototype, with the preparation of the numerical model to be completed in time for integration with behavioral tracking results by spring 2002. Additional forebay modeling required to better define the forebay flow field for several representative operations can potentially be done prior to completion RSW design and installation with the existing 1:80 general model (or possibly even a rough flow field assessment using the 1:40 LGR general and numerical models). The rationale here is that we have a solid sense of the size of the LGR flow field during different operations with 6000 cfs; and we know that the John Day flow field with 14 kcfs will be much larger, especially upstream of skeleton bays 17-20 (where there is no active flow passage route or associated competing flow field). Pre-design RSW numerical modeling is, therefore, not as critical at John Day.

NWP Proposal to Consider Two Contrasting RSW Options at this Design Phase

We agree that two designs should continue to be developed at this early juncture. One, Alternative #2 presented by Ed Zapel at the May 9 meeting, is a "flow-efficient" design that extends the pier-nose upstream approximately 26' (which is the extent of upstream projection of the RSW temporary pier nose). In this option, the crest is approximately 6' downstream of the prototype RSW pier nose. With this 6 kcfs LGR RSW design, it was observed during modeling that this created an excessive flow separation downstream of the crest that led to additional assessment of pier nose shape. The final RSW option selected at LGR included a nearly-20' horizontal distance from the pier nose to the crest, for a total upstream projection of the prototype piers of approximately 45'. This allowed lower velocities at the pier nose and reduced the perceived hydraulic (and possibly fish injury) flow separation condition.

Additionally, the selected LGR-RSW was designed to induce gradually varied velocity conditions in the ramp zone downstream of the pier nose. Numerical modeling also showed that velocity isobars were slightly farther apart upstream of the pier nose, and extending a small distance into the forebay. One of the main themes being addressed at LGR (especially with the powerhouse prototype collector) is the need to collect a higher percentage of fish that approach the entrances. Too many are leaving after approaching the entrances, rather than entering/passing. Therefore, the selected LGR-RSW ramp limits/controls the increase of velocity per foot of horizontal displacement to 1 fps per linear foot, in contrast to the criterion of 0.1 fps per linear foot in NMFS Screen Criteria. This criterion was waived in the context of site-specific conditions associated with the LGR-RSW. The perceived benefit relates to the hypothesis that fish will be drawn closer to the crest and be trapped before the velocity isobars are close enough to incite an avoidance response from juvenile fish. (It is broadly accepted that fish avoid abrupt velocity and/or acceleration changes.) This ramp results in a longer upstream projection of the pier noses at LGR (approximately 45' total). Whether this additional length of projection at LGR is necessary to improve passage remains speculative, but there was a noticeable disparity (observed with the numerical model and 1:40 general forebay model) in the abruptness of velocity change upstream of and on the ramp, compared to the "flow-efficient" model RSW.

At John Day, with 14,000 cfs rather than 6000 cfs, the two logical options with which to proceed into modeling are the Alternative #2 "flow-efficient" (non-ramp) option with the 26' extension, and an option with the same pier extension (approximately 45') that will be tested at LGR. With larger RSW discharge at John Day, the ramp would have a steeper ramp slope. Further, this ramp would reduce the velocity at the pier nose and, possibly, the side separation magnitude. We recommend building both inserts for the sectional model, comparing/contrasting conditions early in the modeling process, then assessing whether to proceed with only the preferred option from that point.

Other Comments:

1. Current forebay range is limited to 262.5 - 264.0, with el 262.5 as the minimum irrigation pool (MIP). This was required by the 1995 Federal Columbia River Power System Biological Opinion, for juvenile fish migration periods.
2. We recommend that the design RSW flow of 14,000 cfs be adopted at forebay el 263.0. This is very close to the unit flow per crest length of the skeleton bay.
3. Prior to initiating prototype testing in 2002, a special tailrace spill schedule will have to be developed. We recommend this schedule only extend up to 100 kcfs spill.
4. We recommend that the low tailwater elevation for design of the spill bay 20 extended deflector be that associated with a project flow of 125 kcfs (approximately el 158-159).
5. We caution that a 48' width was adopted for final design at LGR. A 1' RSW structure width on both sides of the RSW was required for rigidity. Modeling of the expansion from 48-50' width (near the spill gate sill point and on both sides) was an important task with the sectional model.
6. Many of the tasks to be addressed and reconciled in RSW design development at John Day have already been reconciled at LGR. Included in this category is the issue of cavitation at the transition from RSW to existing ogee (just upstream of the tainter gate sill). We encourage more appreciable dialogue with NWW to address these issues at an early juncture, so that this design can proceed in the most efficient manner. Idaho Department of Fish and Game has requested a meeting in June to discuss the LGR-RSW in more detail - which may be a good time. Rock is the lead in reconciling an exact date.
7. If the decision is made to proceed with a partial forebay 3-D numerical model, we encourage depiction of the RSW flow field both in the context of velocities and accelerations. Fish may respond more strongly to one than the other. It is important to gain a better understanding of which has the greater influence.
8. We encourage inclusion of spill bay #1 deflector design and construction under this scope of work. This importance of this issue recently became evident in the context of preliminary late-April discussions of the John Day total dissolved gas near-field test results. Non-deflector flow from spill bay #1 gate settings were determined to have a disproportionately high influence on tailrace fixed monitoring station readings, and the magnitude of the 120% annual waiver spill cap. Early design and construction of the spill bay #1 deflector is now a high priority for installation and protection of listed salmon stocks, and it makes sense to add this work (pending SCT approval) to the RSW scope of work, since a spill bay #20 extended deflector is already included.

We anticipate sending this memo out with a shorter cover letter, while sending the NWP a draft in advance for their more immediate review.

Cc Blaine Ebberts, NWP
Rock Peters, NWP
Buchholz, NWP
Matt Hanson, NWP
Fredricks

M E M O R A N D U M

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Date: Friday, September 15, 2000

To: Blaine Ebberts, USACE

From: Chuck Tracy
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Subject: Trip Report for John Day Removable Spillway Weir

On August 6-8, 2000, I accompanied USACE personnel, Steve Rainey and Gary Fredricks (NMFS) and Tom Lorz (CRITFC) on a trip to observe the sectional model of the John Day removable spillway weir (RSW) at the Northwest Hydraulics laboratory in Vancouver, B.C. The purpose of the trip was to select a design for the RSW and conduct a review of the 30% report.

Two RSW designs were available for observation, one with an ogee, and one with a fillet to create a flat slope. Only the fillet design had acceptable hydraulic conditions on the deflector. The ogee design produced areas of insufficient depth on the deflector and intersecting jets in the reentry area beyond the deflector, which appeared to have unacceptable conditions. However, even the fillet design appeared to have a severe hydraulic jump in the reentry area at tailwater elevations of 161 or greater. At tailwater elevations of <161 and >158, good undulating or skimming conditions resulted. At 157, a significant plunge effect was apparent. Gas entrainment appears to be a possible concern at low tailwater elevations, but not at higher elevations. The bottom line is that there may be a fairly narrow operating range for the John Day RSW.

The RSW was intended to emulate Skeleton Bay conditions. However the Skeleton Bay has a shallower slope, a larger radius at the deflector, is divided into three separate chutes, each with a different deflector elevation, and operates at 6 KCFS in each chute. The RSW is a single chute that operates at 15 KCFS. It may be possible to improve conditions by decreasing the radius of the deflector, but I suspect that the improvement will be minimal. It may be that the sheer volume of water in the RSW model is responsible for the perception of excessive turbulence compared with the Skeleton Bay.

Two pier nose designs were available for observation, one that extended the same distance upstream as the weir, and one that extended farther. The shorter pier nose design worked reasonably well as far as creating attraction flows, but there was substantial drawdown along the upper margins that affected the shape of the jet on the deflector. The longer design reduced the drawdown effect and improved the shape of the jet, but was not as effective at creating an attraction flow. It is likely that some compromise design would be a possible solution.

The selection of the RSW design option should allow the contractor to proceed on schedule for the completion of the 60% report. In the meantime, ODFW would like to view a video of the general model to determine relative conditions of Skeleton Bay and RSW discharge conditions to see if they are similar over a range of conditions. If the resolution of the 1:80 general model is inadequate, it may be preferable to construct a 1:25 model of the Skeleton Bay at Northwest Hydraulics for a side-by-side comparison with the RSW.

cc: Boyce, Norman, Mallette, Nielson (WDFW), Yoshinaka (USFWS)

APPENDIX D

Model Alternative Report

John Day Surface Dypass
Removable Spillway Weir
Model Alternative Report

May 2000
Portland District
US Army Corps of Engineers

Montgomery Watson / Northwest Hydraulic Consultants / Glosten Associates / Civil Tech, Inc.

TADLE OF CONTENTS

1. Purpose and Scope

The removable spillway work is being performed as a result of the conclusions reached from a previous study regarding surface collection at John Day Dam; "Skeleton Day Surface Dypass Spillway Feature Design Memorandum No. 52 (Corps 1998)." Feature Design Memorandum No. 52 estimated Skeleton Day modifications for a surface bypass spillway to cost approximately $54 million for one bay and $85 million for 2 bays. The relatively high cost of the Skeleton Day modifications prompted the District to develop a means of testing the effectiveness of surface collection via limited spill before making such a large capital investment. The John RSW was conceived to test the effectiveness of a surface spillway outlet at much lower cost than modification of an existing skeleton bay. The ultimate goal of this work order is to determine the appropriate RSW geometry by which to simulate a skeleton bay surface bypass spillway. The RSW concept includes a removable ogee type crest and a 30 ft long flow deflector in Spillway Day 20 of the John Day spillway.

Walla Walla District is evaluating a similar concept at Lower Granite Dam with the intention of testing a prototype in April 2001. Plans and Specifications for the Lower Granite RSW are to be completed in July 2000. The Lower Granite RSW is being designed for 6,000 cfs, while the John Day design flow is about 14,000 cfs. The John Day flow simulates the unit discharge previously tested in the skeleton bay surface bypass spillway installed in the general John Day model at the Waterways Experiment Station (WES). The configuration of the RSW sections at the two projects are somewhat different, and the John Day skeleton bay discharge objective is greater than the RSW for Lower Granite.

The John Day RSW study is intended to develop a single recommended design as a "proof of concept" for the RSW. The RSW is a surface collection and bypass structure that can be deployed in the south spillway bay (Day 20) of John Day Dam. The RSW structure will be designed to float into place, secure to the spillway, and pass limited flow (from 14,000 up to 20,000 cfs, depending on configuration) over an uncontrolled crest. The objective of the RSW structure is to improve fish passage by creating an attraction flow at the surface of the reservoir, and then safely passing collected juvenile fish downstream. This Evaluation Report summarizes the results of the alternatives conceptual analysis and presents a recommended "proof of concept" RSW design for the John Day Dam Surface Collector Project.

A physical model study to verify hydraulic performance of the RSW concept follows this report and accompanies the development of the Design Documentation Report (DDR) for the RSW. If the "proof of concept" design is shown in the model study to be inadequate, or that the surface collector success could be improved with a different design approach, then an 'optimum RSW concept" will be investigated. The final selected RSW concept will be fully evaluated in the final DDR, which will be produced at the conclusion of this study.

2. Design Considerations

a. The initial hydraulic and biological criteria used in developing the RSW alternatives are listed below:

1) A submergence of 22.5 feet. (i. e. spillway crest to water surface is 22.5 feet)

2) To be located upstream of spillway gate (use existing spillway gate for flow control)
3) Flow acceleration to be 0.1 feet per second per foot, if possible
4) Flow would be around 14,000 cfs (similar flow rate per foot of width as the skeleton bay surface bypass spillway design)
5) Approximate the Skeleton Unit Surface Flow Dypass (SFD) zone of influence
6) Approximate skeleton bay hydraulic characteristics
7) RSW located near powerhouse to simulate skeleton bay geometry
8) Optimize pier nose shape to minimize flow separation

In general, the RSW is likely to be most effective when the proportion of total spillway flow passing over the RSW is high. It is expected that when spill volume through the RSW falls below a certain percentage of total spillway discharge, the RSW will no longer be effective, since the flow through adjacent spillway bays will reduce the relative attractiveness of the flow field entering the RSW. The consensus of the team members following the initial discussion during the first project site visit was to attempt to approximate the proposed unit discharge through the Skeleton Day SFD with the RSW because this discharge is what develops the attraction flow field in the reservoir.

The hydraulic discussion during the first site visit centered on two issues: the flow field created in the forebay and the hydraulic characteristics of the spillway, particularly at the junction of the RSW and the existing spillway. It was decided that the RSW design should pull proportionally more flow from surface, preferably from within the top 30 feet of reservoir. The RSW should emulate the Skeleton Day SFD as much as possible, in that it draws from near the surface, and is as near to the existing crest of the spillway as possible. Concepts that extended the RSW well out into the forebay were undesirable, as they do not appear to emulate the flow field of the Skeleton Day SFD.

Once preliminary evaluations are completed, the physical scale modeling work will be used to refine the RSW design. It is estimated that refinement of the upstream section of the RSW crest can optimize the amount of flow withdrawn from near the surface (within the top 30 feet of the water column).

During the first site visit, some participants noted that if the RSW piers are located too close to the RSW crest, there will be drawdown and turbulence near them. Ed Zapel stated that the pier shape will be determined following the selection of the RSW crest location. The pier shape is dependant upon the velocity of flow past them and their distance to the crest of the RSW. The pier shape will be calculated according to hydraulic criteria and evaluated in the model. At this time, the pier shape is a refinement that will come later as the design progresses.

b. Civil and Structural Criteria.

In addition to hydraulic analysis and evaluation of the RSW design, civil and structural analyses will also be conducted. The RSW structure will be deployed by floating into place, and partially flooding the internal chambers to effect settling, rotation, and placement of the structure against the existing spillway. This Interim Report discusses only the most basic of the structural and civil considerations and advantages and disadvantages of the six RSW conceptual designs. There are two basic considerations at this stage of design:

1) The draft of the RSW structure when it is in the floating position for transport, must be shallow enough to transit the locks at Donneville, The Dalles, and John Day Dams.
2) The RSW must be designed such that it can be removed or installed within less than 24 hours.

The relative ease of installing and removing the RSW from the spillway was estimated for the six alternatives. Maximum operating pool is 268.0. Normal operating pool during the fish passage season is elevation 264.0. Minimum operating pool is 257.0. The shape of the RSW is also important for flotation efficiency and effectiveness. The geometry will have to be evaluated carefully. The RSW must be designed such that it can be floated through the navigation locks readily. The draft restriction through the locks may control the geometry of the upstream face of the RSW. Changes to the geometry of the upstream face might be made during subsequent structural design in this study.

3. Alternatives Discussion

A total of six alternatives were developed to the conceptual level of hydraulic design. The six were presented to the National Marine Fisheries Service, Columbia River Inter-Tribal Fish Commission staff, and Corps personnel at an Alternatives Selection Meeting on 9 May, 2000 held at the John Day Project. The level of hydraulic design was limited to development of configurations based on previous Skeleton Day SFD collector work, Lower Granite Dam RSW work, and rough calculations of approximate velocities, discharge capacities, and water surface profiles in the vicinity of the RSW structure. As discussed above, the selected design(s) will be developed more fully during following phases of this Design Documentation Report (DDR) study. The performance of the selected design(s) will be documented in a sectional physical hydraulic model of the spillway and a larger general model of the John Day project. The six RSW geometries that were evaluated for model work are listed below:

Alternative 1	Skeleton Day Geometry, Piers 85' Upstream
Alternative 2	Vertical Face, Piers 46' Upstream
Alternative 3	Sloping Face, Piers 128' Upstream
Alternative 4	Vertical Face, Piers 41' Upstream, Step at Spillway Ogee Interface
Alternative 5	Semi-Permanent Lower Crest, Piers 46' Upstream
Alternative 6	Sloping Face, Piers 124' Upstream, Step at Spillway Ogee Interface

The following subsections provide brief discussions of the six RSW conceptual designs. Figures showing the configuration of each of the six designs are enclosed in this report. The recommended design, with additional discussion of potential improvements to be made to the design as a result of modeling performance evaluation, is discussed at the end of this report.

3.a. Alternative 1 - Skeleton Day Geometry, Piers 85’ Upstream

The design consists of the skeleton bay surface spillway geometry shown in the John Day Design Memorandum No. 52. It would be placed on the existing spillway crest upstream of the existing spillway gate (see Figure D1). The RSW structure extends about 85 feet upstream of existing spillway crest centerline (Construction Dase Line). Maximum submergence of the RSW crest of 22.5 ft is obtained at a maximum operating pool elevation of 268 ft msl. The RSW crest is about 20 feet long, with a short ogee transition to a 0.6V:1H sloped chute and 50 ft radius to existing crest at the downstream end. Pier sections would extend to the upstream limit of the RSW structure, with a 6 ft radius upstream nose.

- RSW crest length 20.0 ft.
- RSW crest elevation 245.5 ft msl.
- RSW structure extends 85 feet upstream of existing spillway crest centerline.
- RSW maximum discharge capacity is about 16,000 cfs at maximum pool 268 ft.
- RSW minimum discharge capacity is about 5,800 cfs at minimum pool 257 ft.
- RSW crest submergence 22.5 ft at maximum operating pool elevation 268 ft msl.
- RSW crest submergence 11.5 ft at minimum operating pool elevation 257 ft msl.
- Transition to existing crest is a 50-ft radius to tangent point.
- Alternative does not meet 0.1 fps/ft velocity flux limit criteria.
- Capture velocity 7 fps achieved just upstream of crest.
- The draft of this structure might be too deep for transit of locks.

3.b. Alternative 2 – Vertical Face, Piers 46' Upstream

This design consists of a configuration similar to that developed for the Lower Granite Dam RSW by Walla Walla District. The Lower Granite Dam Flow Efficient RSW configuration is designed for 15 feet of submergence at maximum operating pool elevation, while the John Day RSW configuration is designed for 22.5 feet of submergence at maximum operating pool elevation of 268 ft msl (see Figure D2). The RSW crest shape is designed for maximized flow capacity and efficiency, based on EM 1110-2-1603, COE design guidance for spillway. The RSW structure extends only 46 feet upstream of the existing spillway crest centerline (Construction Dase Line). Maximum submergence of the RSW crest is 22.5 feet at maximum operating pool elevation of 268 ft msl. The RSW crest is of standard shape designed for 22.5 ft head at the maximum operating pool elevation of 268 ft. It has a standard ogee shape that transitions to the existing crest through a 20 ft radius curve tangent at a point about 1 ft upstream of the existing spillway gate. Pier sections would extend to near the upstream vertical face of the dam with a similar nose shape as that for existing piers.

- RSW crest length (standard crest shape for 22.5 ft head, EM 1110-2-1603).
- RSW crest elevation 245.5 ft msl.
- RSW structure extends 46 feet upstream of existing spillway crest centerline.
- RSW maximum discharge capacity is about 21,000 cfs at maximum pool 268 ft.
- RSW minimum discharge capacity is about 7,700 cfs at minimum pool 257 ft.
- RSW crest submergence 22.5 ft at maximum operating pool elevation 268 ft msl.
- RSW crest submergence 11.5 ft at minimum operating pool elevation 257 ft msl.
- Transition to existing crest is a 20-ft radius to tangent point.
- Alternative does not meet 0.1 fps/ft velocity flux limit criteria.
- Capture velocity 7 fps achieved just upstream of crest.

- The draft of this structure might be too deep for transit of the locks; however, it can be modified to provide a shallower draft.

3.c. Alternative 3 – Sloping Face, Piers 128' Upstream

This alternative consists of an RSW design similar to that which was developed for Lower Granite Dam for fish collection efficiency, but adapted to meet the John Day RSW criteria (see Figure D3). The Lower Granite RSW was designed for 15 feet of submergence on the crest at a maximum normal operating pool elevation, while the John Day Dam RSW will be designed for 22.5 feet of submergence on the crest at the maximum normal operating pool elevation of 268.0 ft msl. The RSW crest shape is designed for maximized flow capacity and efficiency, based on EM 1110-2-1603 COE design guidance for spillways. The 'Fish Efficient' design requires that the approach flow velocity flux meet the criteria of 0.1 fps/ft wherever practical, up to the point at which velocity exceeds 7 fps ('capture' velocity). This design extends a sloping ramp upstream of the RSW crest, which, for the maximum design discharge, provides acceleration from the entrance section up to 7 fps velocity. Capture velocity occurs a short distance upstream of the RSW crest centerline for both the minimum flow at low normal operating pool elevation of 257 ft msl and for maximum operating pool elevation of 268.0 ft. The RSW extends 128 feet upstream of the existing crest centerline (Construction Dase Line). The standard ogee shape transitions to the existing crest through a 20 ft radius curve to a tangent point about 1 foot upstream of the existing spillway gate seal beam. Pier sections would extend to the upstream extent of the RSW structure, with a 6-foot radius nose shape.

- RSW crest length (standard crest shape for 22.5 ft head, EM 1110-2-1603).
- RSW crest elevation 245.5 ft msl.
- Entrance section 1.16V:1V slope up from elevation 153 to RSW crest elevation 245.5 ft.
- RSW structure extends 128 feet upstream of existing spillway crest centerline.
- RSW maximum discharge capacity is about 16,800 cfs at maximum pool 268 ft.
- RSW minimum discharge capacity is about 6,000 cfs at minimum pool 257 ft.
- RSW crest submergence 22.5 ft at maximum operating pool elevation 268 ft msl.
- RSW crest submergence 11.5 ft at minimum operating pool elevation 257 ft msl.
- Transition to existing crest is a 20-ft radius to tangent point.
- Alternative meets 0.1 fps/ft velocity flux limit criteria.
- Capture velocity 7 fps achieved well upstream of crest.
- Pier extensions cause large areas of 'dead' water on each side of entrance section.
- May be difficult for fish to find RSW spillway entrance in the prototype, since it extends so far upstream of spillway.
- The draft of this alternative would be too deep to move through the locks. It might have to be installed in two or more pieces.
- Quick installation and removal would be difficult due to the size of the RSW.

3.d. Alternative 4 – Vertical Face, Piers 41' Upstream, Step at Spillway Ogee Interface

The design nearly identical to Alternative 2 above, but with 2 ft high aeration step at downstream end of RSW structure (see Figure D4). The step is located at the extreme downstream end of the RSW structure, about 1 ft upstream of existing spillway gate seal beam. The design consists of a configuration similar to that developed for the Lower Granite Dam Flow Efficient RSW by the Walla Walla District. The RSW crest shape is designed for maximized flow capacity and efficiency, based on EM 1110-2-1603, COE design guidance for spillways. The RSW structure extends 41 feet upstream of the existing spillway crest centerline (Construction Dase Line). Maximum submergence of the RSW crest of 22.5 feet occurs at the maximum operating pool elevation of 268 ft msl. The RSW crest is of standard shape designed for 22.5 ft head at the maximum operating pool elevation 268 ft. The standard ogee shape transitions to the step through a 20 ft radius curve to an exit slope slightly steeper than the existing crest at the same location. The exit jet trajectory theoretically would separate at the step, joining the existing crest again at about elevation 153.0, at the upper end of the existing 3V:4H slope chute section. This point is also near the tangent point of the radius curve joining the existing crest to a flow deflector (not yet constructed) that will likely be at about elevation 153 ft msl. RSW flow will be highly aerated by the step, but the spillway chute surface should not be exposed to cavitation damaging by low pressures that may occur in the other alternatives. Pier sections would extend to near the upstream vertical face of the dam with a similar nose shape as that for existing piers.

- RSW crest length (standard crest shape for 22.5 ft head, EM 1110-2-1603).
- RSW crest elevation 245.5 ft msl.
- RSW structure extends 41 feet upstream of the existing spillway crest centerline.
- RSW maximum discharge capacity is about 21,000 cfs at maximum pool 268 ft.
- RSW minimum discharge capacity is about 7,700 cfs at minimum pool 257 ft.
- RSW crest submergence 22.5 ft at maximum operating pool elevation 268 ft msl.
- RSW crest submergence 11.5 ft at minimum operating pool elevation 257 ft msl.
- Transition to aeration step at end of 20 foot radius.
- Alternative does not meet 0.1 fps/ft velocity flux limit criteria.
- Capture velocity 7 fps achieved just upstream of crest.
- The design of the RSW can be modified to provide a suitable draft for transport.
- Compared to the other alternatives it would be relatively simple to install and remove.

3.e. Alternative 5 - Semi-Permanent Lower Crest, Piers 46’ Upstream

This design consists of the removable RSW crest placed atop a second, lower crest section, which extends both downstream and upstream of existing spillway gate seal beam (see Figure D5). The crest shape serves to permit RSW flow jet to nearly adhere to existing crest without separating or causing low pressures to occur on the face of the existing crest. The proposed shape, however, is not quite capable of entirely eliminating separation. The lower crest section would likely be constructed of steel, as would the RSW section above. If the maximum original spillway discharge capacity were ever required, this section could be separated from the existing crest with exploding connections or similar mechanisms and permitted to fall into the stilling basin. This flood event would be quite large, and would likely be accompanied by severe damage upstream and downstream of John Day Dam, so that the loss of the lower crest section would not be an important consideration.

The RSW section seats atop the lower semi-permanent crest section and extends upstream at a slope of 0.7338V:1H to the ogee curve. The ogee shape of the RSW and the lower semi-permanent crest section is based on maximizing the discharge capacity efficiency as shown in

EM 1110-2-1603, COE design guidance for spillways. The RSW structure extends about 46 feet upstream of the existing spillway crest centerline (Construction Dase Line). Maximum submergence of the RSW crest is 22.5 feet at a maximum operating pool elevation of 268 ft msl. The RSW crest is of standard shape designed for 22.5 ft head at the maximum operating pool elevation of 268 ft msl. The design head for the semi-permanent crest section is 40.5 ft at a crest elevation of 227.5 ft msl. The standard ogee shape of the RSW section transitions to a lower semi-permanent crest slope. The tangent point is about 1 ft upstream of the spillway gate seal beam location on a new semi-permanent lower crest section. The elevation of new gate seal beam would be about 215 ft msl. The semi-permanent crest section transitions to the existing spillway at elevation 195 through a flush tangent point. Pier sections would extend to near the upstream vertical face of the dam with a similar nose shape as that for the existing piers.

- RSW crest length (standard crest shape for 22.5 ft head, EM 1110-2-1603).
- Semi-permanent crest length (standard crest shape for 40.5 ft head, EM 1110-2-1603).
- RSW crest elevation 245.5 ft msl.
- Semi-permanent crest elevation 227.5 ft msl.
- RSW structure extends 46 feet upstream of the existing spillway crest centerline.
- Semi-permanent crest structure extends about 27.5 ft upstream of the existing spillway crest centerline.
- RSW maximum discharge capacity is about 21,000 cfs at maximum pool 268 ft.
- Semi-permanent crest maximum discharge capacity is about 50,000 cfs at maximum SDF pool elevation 276 ft msl (existing crest capacity is about 83,000 cfs at SDF pool 276).
- Existing spillway gate seal beam elevation raised to about elevation 215.4 on top of semi-permanent crest section.
- Semi-permanent crest section is detachable in case existing crest discharge capacity required.
- RSW minimum discharge capacity is about 7,700 cfs at minimum pool 257 ft.
- RSW crest submergence is 22.5 ft at maximum operating pool elevation 268 ft msl.
- Semi-permanent crest submergence 40.5 ft at maximum SDF pool elevation 276.
- RSW crest submergence 11.5 ft at minimum operating pool elevation 257 ft msl.
- Transition to existing crest through tangent point.
- Alternative does not meet 0.1 fps/ft velocity flux limit criteria.
- Capture velocity of 7 fps is achieved just upstream of crest.
- Removing the semi-permanent crest could be a problem and it might damage the stilling basin if removed during a flood.
- The geometry of the RSW makes it difficult to change ballast and floatation for installation.

3.f. Alternative 6 – Sloping Face, Piers 124' Upstream, Step at Spillway Ogee Interface

This design is identical to that for Alternative 4, except that upstream entrance is extended at 1.16V:1V slope to meet 0.1 fps/ft velocity flux criteria (see Figure D6). The RSW section extends 124 feet upstream of the existing crest centerline (Construction Dase Line). All other design configuration data are similar to Alternative 4 except for the pier extensions. The pier sections would extend well upstream of the vertical face of the dam with a 6.0 ft radius nose shape rather than the elliptical shape of the existing piers.

- RSW crest length (standard crest shape for 22.5 ft head, EM 1110-2-1603).

- RSW crest elevation 245.5 ft msl.
- RSW structure extends 124 feet upstream of the existing spillway crest centerline.
- RSW maximum discharge capacity is about 16,800 cfs at maximum pool 268 ft.
- RSW minimum discharge capacity is about 6,000 cfs at minimum pool 257 ft.
- RSW crest submergence 22.5 ft at maximum operating pool elevation 268 ft msl.
- RSW crest submergence 11.5 ft at minimum operating pool elevation 257 ft msl.
- Transition to aeration step is on a 20 ft radius.
- Alternative does meet 0.1 fps/ft velocity flux limit criteria.
- Capture velocity of 7 fps is achieved just upstream of crest.
- The draft of this alternative would be too deep to move through the locks. It might have to be installed in two pieces.
- Quick installation and removal would be difficult due to the size of the RSW.

4. Summary of Recommendations and Related Opinions

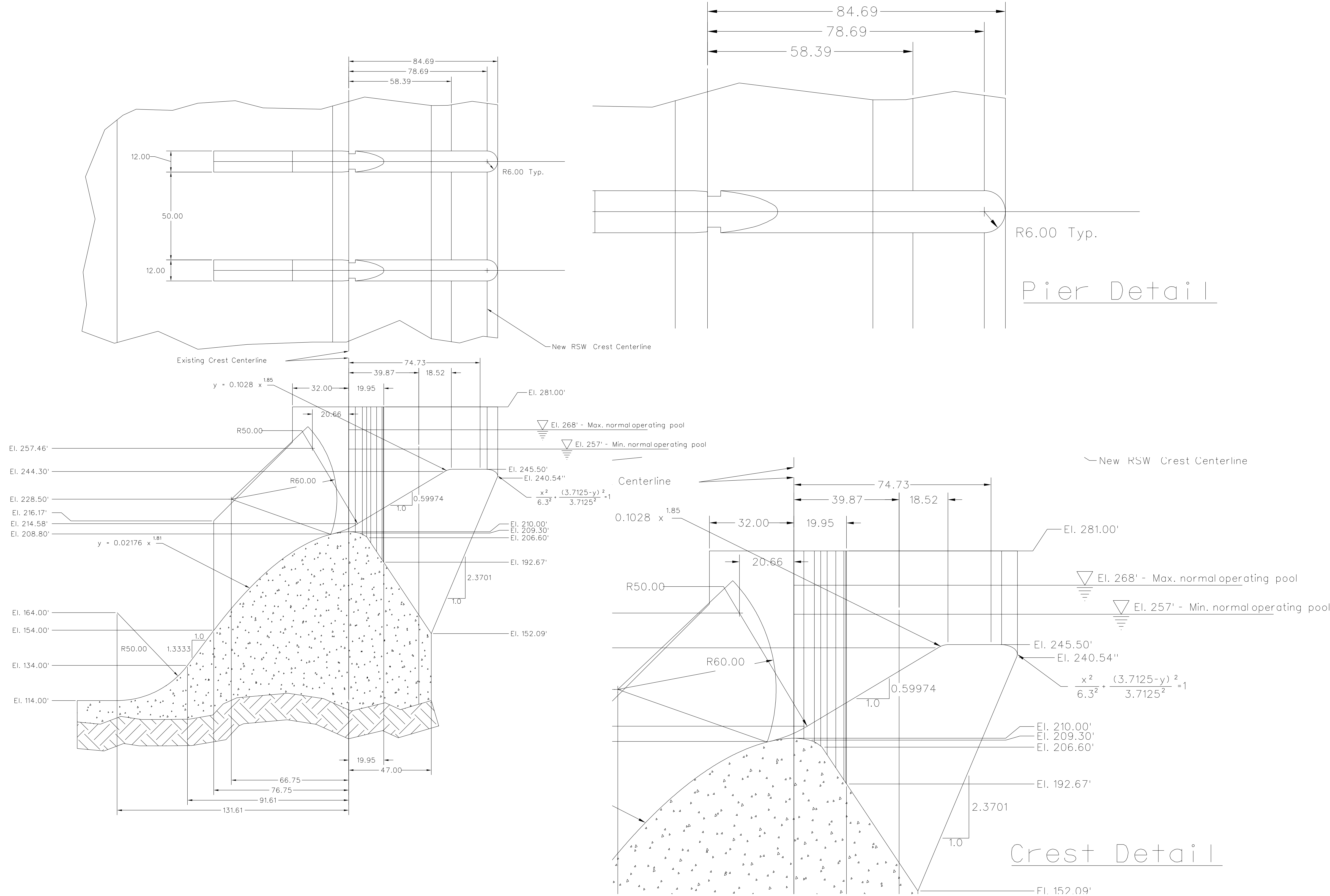
NMFS did not have a favorable opinion of Alternatives 4 and 6, both of which had an aeration step at the junction between the RSW and the existing spillway ogee. Their opinion was that it might lead to unsafe passage conditions for downstream migrants. However, Ed Zapel said that the flow at the end of the RSW is fast enough to produce cavitation and that providing air might be necessary to control cavitation conditions. At this time, Alternatives 4 and 6 will not be evaluated further unless the cavitation potential from model evaluation is high.

Through the process of elimination during the discussion, Options 1, 3, 4, 5 and 6 were dropped. Options 1, 3 and 6 were dropped because of the distance they extended into the forebay. Options 4 and 6 were dropped because of the unacceptable aeration step. Option 5 was dropped because it was felt that the semi-permanent decrease in spillway capacity as a result of the false crest section would be unacceptable. Alternative 2 was selected for further study.

Alternative 2 was selected because it was felt to be the most capable of approximating the Skeleton Day SFD flow characteristics without requiring such a large extension upstream of the existing spillway crest. Alternative 3 was selected to provide an evaluation of a gradual velocity profile entrance similar to what the Lower Granite extended RSW appears to provide. The physical modeling work will be comprised of a thorough evaluation of the Alternative 2 concept.

5. Recommended Alternative(s)

The selected design will be Alternative 2 above. This design will be tested in both the sectional and general models. A second design will be prepared for use in the sectional model that includes improvements in the pier shape and approach configuration. The selected design will be evaluated initially, and if significant improvements are required, a second design will be evaluated. A final configuration will be developed and tested in the physical models following initial model evaluations. Detailed design of the final configuration will be made following the model test work.



Designed by: E. ZAPEL	Date: 16 JUNE 2000	X	DD MMM YY	X	XXX
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Checked by:	Technical Manager:				
Submitted by:					

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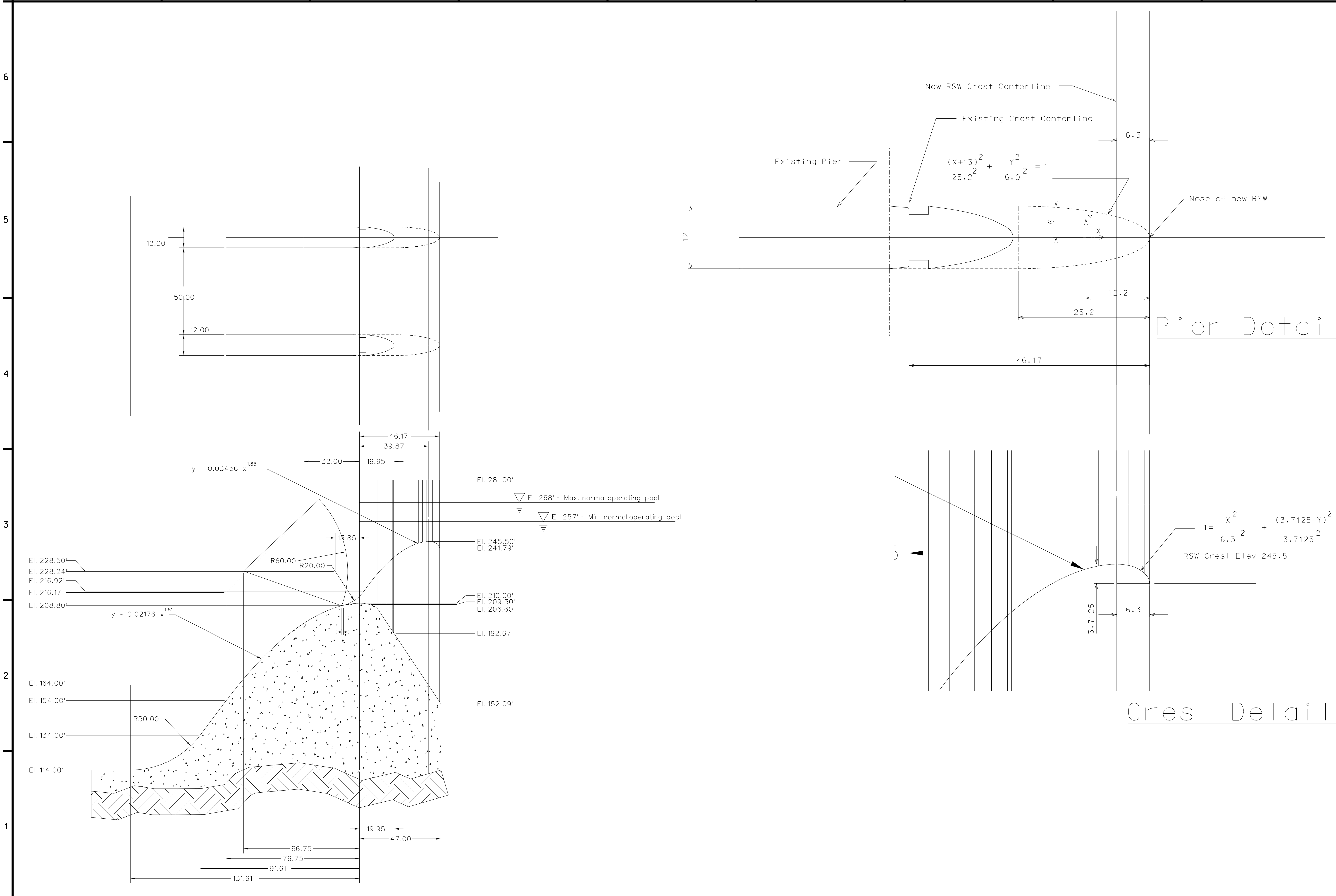
MONTGOMERY WATSON
JOINT VENTURE
NORTHWEST HYDRAULICS
CONSULTANTS

COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 1
SKELETON BAY GEOMERTY
PIERS 85' UPSTREAM

DRAWING STATUS:

DRAWING NO.

FIGURE D1



COLUMBIA RIVER — OREGON - WASHINGTON

JOHN DAY DAM

REMOVABLE SPILLWAY WEIR

ALTERNATIVE 2

VERTICAL FACE

PIERS 46' UPSTREAM

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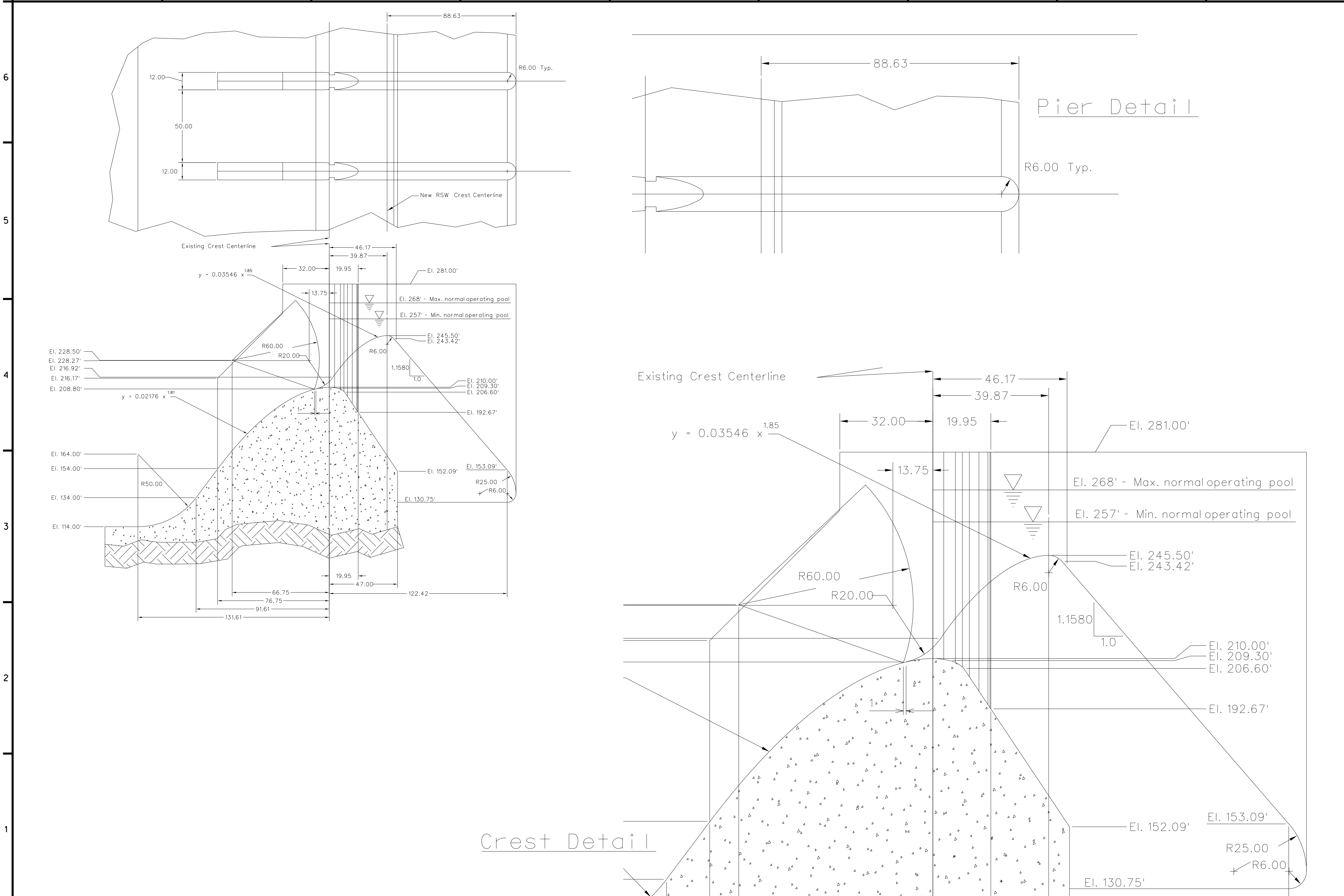
DRAWING STATUS:

DRAWING NO.

FIGURE D2



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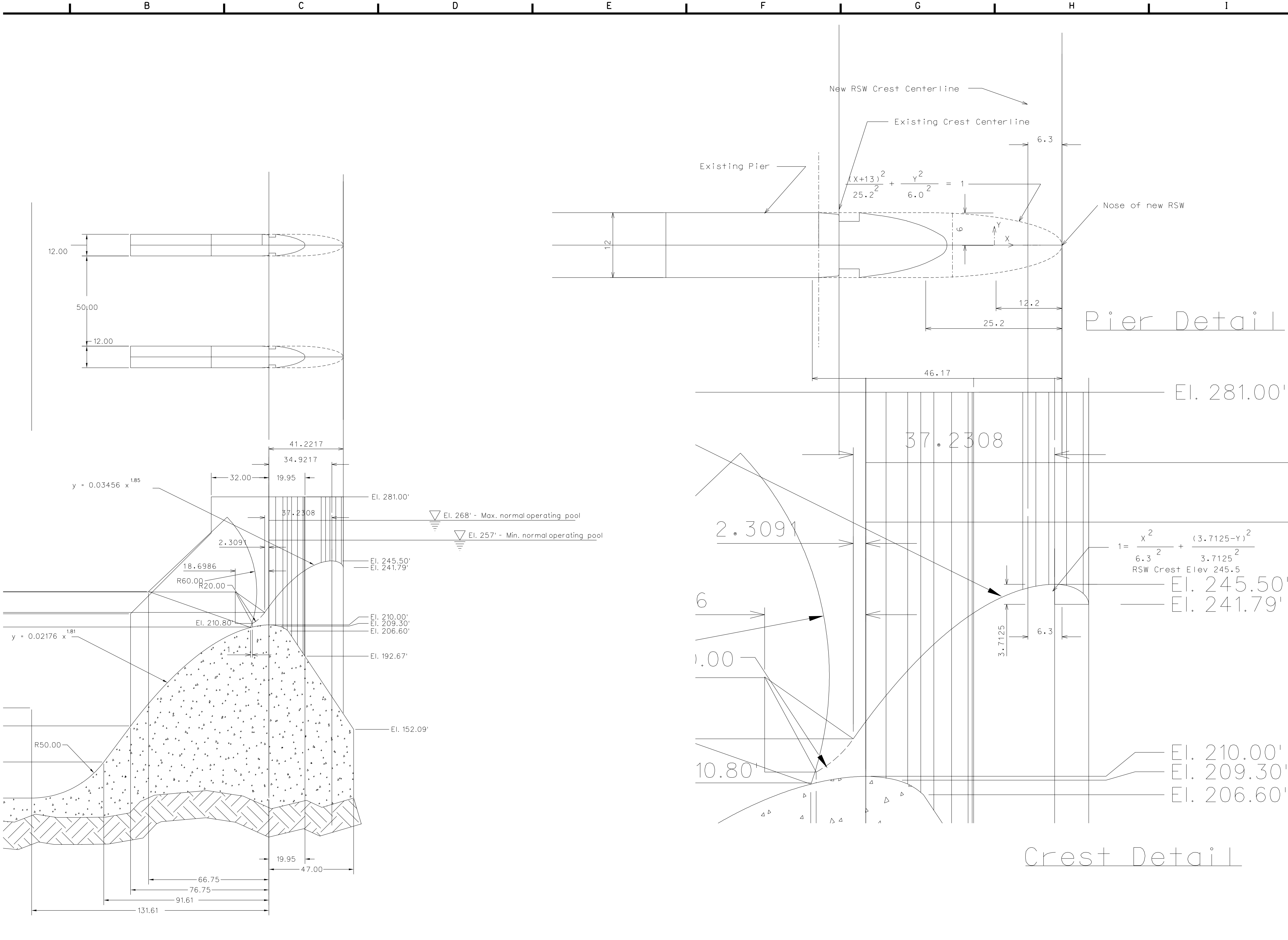
COLUMBIA RIVER — OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 3
SLOPING FACE
PIERS 128' UPSTREAM

DRAWING STATUS:

DRAWING NO.

FIGURE D3



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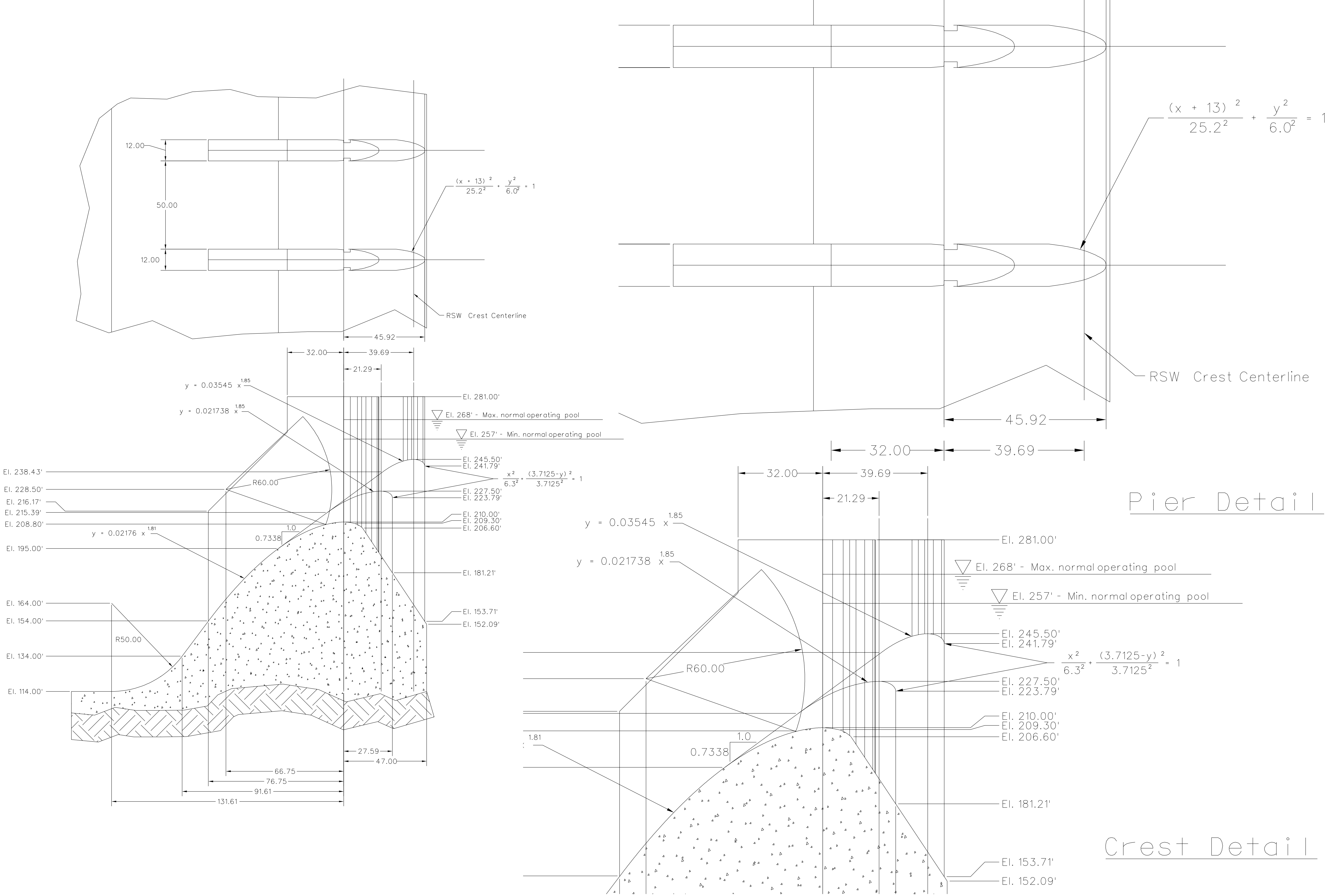
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 4
VERTICAL FACE
PIERS 41' UPSTREAM
STEP AT SPILLWAY OGEE INTERFACE

DRAWING STATUS:

DRAWING NO.

FIGURE D4



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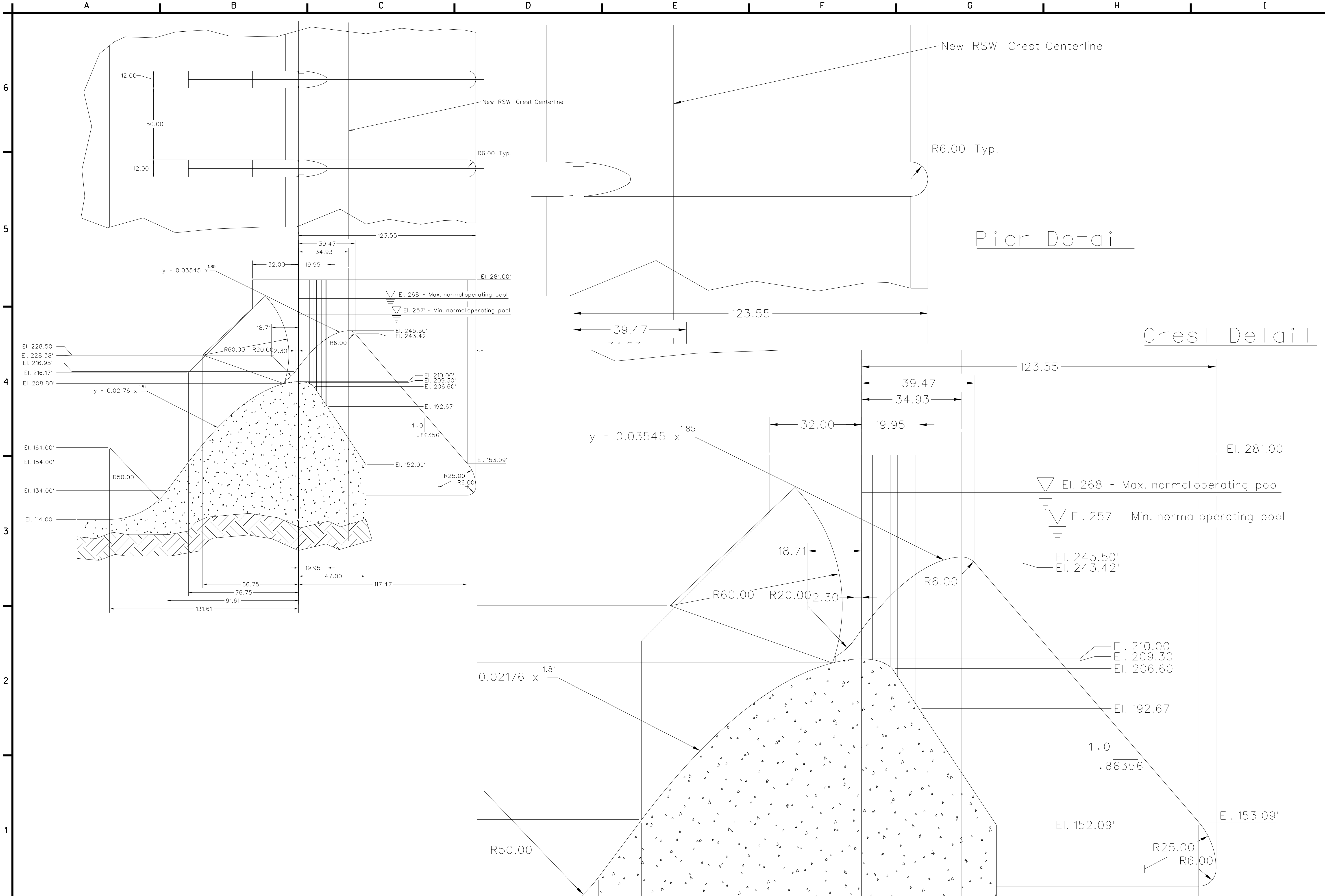
COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 5
SEMI-PERMANENT LOWER CREST
PIERS 46' UPSTREAM

DRAWING STATUS:

DRAWING NO.

FIGURE D5



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JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 6
SLOPING FACE
PIERS 124' UPSTREAM
STEP AT SPILLWAY OGEE INTERFACE

DRAWING STATUS:

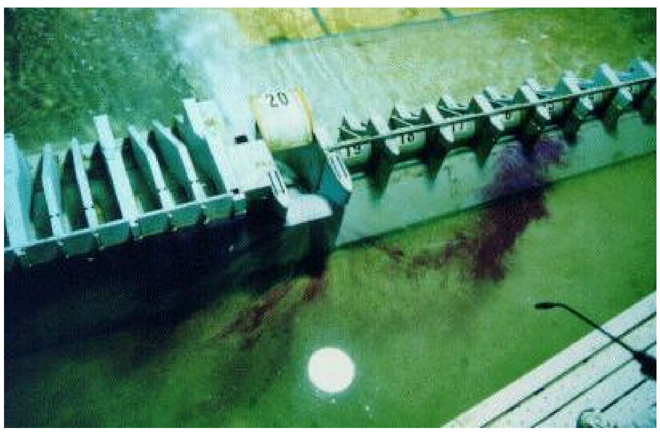
DRAWING NO.

FIGURE D6

APPENDIX E

Physical Model Alternatives Report

John Day Dam Surface Bypass
Removable Spillway Weir
Physical Model Alternatives Report



August 2000
Portland District
US Army Corps of Engineers

Montgomery Watson / Northwest Hydraulic Consultants / Glosten Associates / Civil Tech, Inc.

APPENDIX E

John Day Dam
Removable Spillway Weir

Physical Model Alternative Report

1. Purpose. The purpose of this appendix is to summarize the results of physical model studies of various Removable Spillway Weir (RSW) design geometry's for the John Day Dam project. The objective of the RSW is to improve fish passage by creating a surface flow outlet from the reservoir and safely pass collected juvenile fish downstream over the existing spillway crest. The RSW concept is being evaluated as an alternative to the powerhouse skeleton bay surface bypass concept. The RSW would be deployed in the south spillway bay (Bay 20) at John Day Dam. The RSW structure would be designed to be readily placed in, secured to and removed from the spillway bay and pass up to about 22,000 cfs over an uncontrolled crest.

2. General. Four different RSW design geometries were selected for study in physical models. A 1:25 scale sectional model was used to evaluate detailed flow conditions over the RSW crest and a 1:80 scale general model was utilized to evaluate the RSW approach flow conditions in the reservoir and the egress conditions in the tailrace. The sectional model simulated one full spillway bay and both adjacent ½ bays with portions of the reservoir forebay and stilling basin. The general model, Figure 1, includes the entire 20-bay spillway, the 16-unit powerhouse, 4 powerhouse skeleton bays, about 3 miles of the tailrace and downstream channel, and about 3.5 miles of the reservoir forebay. Two of the designs (a and d below) were tested both in the sectional model at the Northwest Hydraulic Consultants (NHC) laboratory and the general model at Waterways Experiment Station (WES). The other two designs (b and c below) were tested only in the sectional model at NHC. In addition to the RSW tests in the general model, tests were conducted to evaluate approach and egress conditions associated with flow through a skeleton powerhouse bay for comparison to the RSW. Specific details of the designs tested are as follows:

a. Alternative 2 - Vertical Face RSW w/Piers 46.2 ft Upstream. The RSW crest elevation would be 245.5 ft. The geometry downstream from the crest would be a standard shape with a design head of 22.5 ft. The RSW crest transitions via a 20-ft radius vertical curve to meet the existing spillway about 1 ft upstream from the spillway gate seat. The piers extend upstream 46.17 feet from the existing spillway crest centerline and the pier noses are elliptically shaped with a major axis of 25.2 ft and a minor axis of 6 ft. Details of the Alternative 2 RSW are shown on Figure 2.

b. Alternative 4 - Vertical Face RSW w/Piers 41 ft Upstream & Vertical Step at Spillway Ogee Interface. The RSW crest elevation would be 245.5 ft. The geometry downstream from the crest would be a standard shape with a design head of 22.5 ft. The RSW crest transitions via a 20-ft radius vertical curve to terminate in a vertical stepped offset about 2 ft above the surface of the existing spillway crest at a point about 1 ft upstream of the spillway gate seat. Air would be supplied beneath the step via conduits located in the piers. The piers extend upstream 41.22 feet from the existing spillway crest centerline and the pier noses are elliptically shaped with a major axis of 25.2 ft and a minor axis of 6 ft. Details of the Alternative 4 RSW are shown on Figure 3.

c. Alternative 5 – Vertical Face RSW w/Piers 45.9 ft Upstream & Crest Extended 37 ft Downstream From Gate Seat. The RSW crest would be constructed in two pieces with a crest elevation of 245.5 ft. The geometry for a distance of about 17-ft downstream from the crest centerline would be a standard ogee shape with a design head of 22.5 ft. From that point, the RSW crest would follow a constant slope to a point about 37 ft downstream from the existing spillway crest centerline where it would be tangent to, and meet, the existing spillway crest. The piers extend upstream 45.92 feet from the existing spillway crest centerline and the pier noses are elliptically shaped with a major axis of 25.2 ft and a minor axis of 6 ft. Figure 4 shows details of the Alternative 5 RSW.

d. Alternative 7 – Sloped Face RSW w/Piers 65.2 ft Upstream. The RSW crest elevation would be 245.5 ft. The geometry downstream from the crest would be a standard shape with a design head of 22.5 ft. The RSW crest transitions via a 20-ft radius vertical curve to meet the existing spillway about 1 ft upstream from the spillway gate seat. The piers extend upstream 65.17 feet from the existing spillway crest centerline and the pier noses are shaped to a 6-ft radius, simple circular curve. Details of the Alternative 7 RSW are shown on Figure 5.

3. Test Results. Following is a summary of information obtained from the physical modeling program. Trip reports describing observations made during visitations to the **nhc** and WES hydraulic laboratories are appended to the end of this summary report.

a. Alternative 2 Design. Testing in the sectional model was conducted for free flow (ungated) conditions with pool elevations in the range of 262 ft to 268 ft and tailwater elevations in the range of 154 ft to 168 ft. Spillway bay discharges associated with that range of pool elevations are about 13,000 cfs to 20,000 cfs. A preliminary spillway bay rating curve developed from cursory model measurements is shown on Figure 6. Initial tests revealed the existence of a rather large water surface drawdown around the spillway piers (photograph 1). The surface disturbances set off by this drawdown created up to five to eight rather stable standing surface waves that traveled over the RSW crest. These waves were amplified to an approximate height of 10 to 15 ft (prototype) as the flow moved through the reverse curve radius transition between the RSW crest and the existing spillway crest and set off numerous large roostertails as the flow contacted the spillway deflector (photographs 2 and 3). The roostertails were quite unstable and oscillated laterally across the width of the deflector and extended well downstream into the stilling basin. Flow across the deflector was very non-uniform with numerous areas having a very small flow depth. As these roostertail plumes impacted in the stilling basin, the highly aerated flow plunged deep into the basin.

Testing in the general model was conducted with a pool elevation of 264 ft, total project releases of 250,000 cfs and 350,000 cfs and with ratios of spillway flow to total flow of 0, 30 percent and 60 percent. The tailwater elevation varied from 161.7 ft to 164.1 ft with total discharge of 250,000 cfs and 350,000 cfs, respectively. Outflows of about 500 cfs were also simulated to account for fishladder and other miscellaneous project flows. The roostertails observed in the sectional model were also apparent in the general model, but at a much smaller scale due to the difference in model scale. Some roostertails were also apparent with the powerhouse skeleton bay surface flow collector but were not as large as those that existed in the RSW crest spillway bay. Photographs 4 through 6 illustrate approach hydraulic conditions. Photograph 7 shows comparable flow conditions approaching the skeleton bay collector concept. A description of reservoir approach and tailrace egress conditions is shown in Table 1.

b. Alternative 4 Design. Initial testing in the sectional model was conducted for free flow (ungated) conditions with pool elevations in the range of 262 ft to 268 ft. Spillway bay discharges associated with that range of pool elevations are about 13,000 cfs to 20,000 cfs. The crest geometry with this alternative is the same as Alternative 2, therefore the preliminary rating curve developed from Alternative 2 model measurements (Figure 6) is also representative of the Alternative 4 design. The intent of this design geometry was to develop a low pressure area beneath the jet that would draw air and subsequently cause the standing waves emanating from the separation around the pier noses to become fully distributed across the width of the spillway bay prior to hitting the deflector. However, tests revealed that higher than expected static pressure (20 ft of water measured in the model)) existed at the vertical face of the transition step and prevented air from being drawn to the underside of the jet exiting over the step. The hydraulic performance of the stepped design showed very little improvement over the Alternative 2 design (photographs 8 through 10).

c. Alternative 5 Design. This design geometry was generally approximated in the sectional model by modifying the Alternative 2 RSW crest section. The modification consisted of a straight-line fillet filling in the radius transition bucket between the Alternative 2 crest and the existing spillway crest (Figure 7). The fillet was extended up and downstream from the transition bucket sufficiently to approximately form tangent connections to the RSW and existing spillway crests. Testing in the sectional model was conducted for free flow (ungated) conditions with pool elevations in the range of 262 ft to 268 ft. Spillway bay discharges associated with that range of pool elevations are about 13,000 cfs to 20,000 cfs. The preliminary spillway rating curve developed from the Alternative 2 RSW model measurements (Figure 6) also represents the Alternative 5 spillway rating. Testing of the simplified Alternative 5 RSW geometry illustrated that elimination of the bucket transition prevented the amplification of the standing waves that had existed with the designs that included the bucket transition. Additionally, the standing waves that did exist over the face of the spillway were much more evenly distributed laterally across the spillway bay and did not form the large roostertails impacting on, and exiting off, the deflector (photographs 11 through 13).

d. Alternative 7 Design. This design geometry was approximately simulated in the sectional model by modifying the Alternative 2 model RSW crest. This modification consisted of adding an approximately 18.5-ft (prototype) length of readily available pier to the Alternative 2 model RSW and adding a removable inclined floor to simulate the sloping upstream face of the crest. The modification was constructed during the laboratory visit merely to see what a difference it might make to extend the piers and an approach ramp upstream. The Alternative #7 configuration was not selected by the District for accurate tests and observations, rather it was constructed hastily just to evaluate the potential for improvement n the chute flow characteristics. Observations in the sectional model were conducted for free flow (ungated) conditions with pool elevations in the range of 262 ft to 269 ft and tailwater elevations in the range of 154 ft to 168 ft. Spillway bay discharges associated with that range of pool elevations are about 13,000 cfs to 22,500 cfs. Preliminary spillway bay rating curves as developed from cursory model measurements with and without the upstream inclined floor are shown on Figures 8 and 9, respectively. As with Alternative 2, water surface drawdown around the spillway piers created numerous rather stable surface waves that traveled over the RSW crest. Because the pier nose is further upstream than with Alternative 2, the disturbance at the RSW crest appeared to be somewhat smaller than existed with Alternative 2. However, the waves were amplified when the flow moved through the reverse curve radius transition between the RSW crest and the existing spillway crest and set off numerous large roostertails as the flow hit the spillway deflector, similar to the conditions occurring with the Alternative 2 design (photographs 14 and 15). Insertion and removal of the inclined floor ramp did not appear to have any appreciable affect on overall hydraulic performance of the design. When the spillway gate was lowered into the flow to form a control, surface waves originated under the gate lip but were more stable than with the free overflow condition (photograph 16). However, a greater amount of energy appeared to exist at the deflector with the gate-controlled condition and large roostertails still reflected off the deflector and into the stilling basin.

Testing in the general model was conducted with a pool elevation of 264 ft, a total project release of 250,000 cfs and with a ratio of spillway flow to total flow of 30 percent. The tailwater elevation was 161.7 ft a total discharge of 250,000 cfs. Outflows of about 500 cfs were also simulated to account for fishladder and other miscellaneous project flows. The roostertails observed in the sectional model were also apparent in the general model, but at a much smaller scale due to the difference in model scale. Approach flow conditions in the reservoir were generally less favorable than with Alternative 2 because the RSW crest extended further upstream into the forebay and created more pronounced “dead flow zones” on each side of the RSW approach piers. Because the 1:25 scale model did not reveal any significant hydraulic improvement on the spillway chute with the Alt 7 RSW versus the Alt 2 RSW, and the approach conditions appeared somewhat better with Alt 2 in the 1:80 scale general model, testing of Alt 7 with larger project releases and spillway flows was not continued in the general model. Photograph 17 illustrates the RSW approach hydraulic conditions. A description of reservoir approach and tailrace egress conditions is shown in Table 1.

4. Summary and Conclusions. The initial tests performed in the 1:25-scale sectional model revealed the existence of unacceptable standing waves over the crest of RSW alternative designs Alternative 2 and Alternative 7. These standing waves were initiated by the large water surface drawdown around the pier nose and were amplified at the reverse curve transition radius connecting the RSW crest with the existing spillway crest and created unacceptable flow conditions down the existing spillway ogee chute and the spillway deflector. Large roostertails exited off the deflector and into the stilling basin. Alternative design number 4 included a 2-ft high stepped offset between the end of the RSW crest and the existing spillway crest. This design was intended to provide aeration and initiate boundary layer development at transition between the two crests that had amplified the pier nose waves with alternatives 2 and 7 and was anticipated to more evenly distribute the surface waves across the spillway face and subsequently eliminate the roostertails off the deflector. However, the high pressure in the toe of the RSW crest prevented air from being supplied to the jet as it exited from the step and did little to prevent the formation of unstable standing waves and roostertails. The step design also was thought to potentially create some objectionable biological characteristics. Tests in the 1:80 scale general model suggested that the Alt 2 RSW resulted in a somewhat better approach flow condition than did the Alt 7 RSW, which projected further upstream into the reservoir forebay.

The approximated Alternative 5 configuration eliminated the reverse curve sharp transition between the two crests that had amplified the pier nose waves with alternatives 2 and 7 by creating a constant sloping surface connecting the RSW crest and the existing spillway crest. The standing waves emanating from flow separation around the pier nose were not amplified in height as occurred in the designs that included the reverse curve bucket transition. The waves that did exist were much more evenly distributed across the spillway face and the roostertails that had existed with design alternatives 2 and 7 were essentially eliminated. The Alternative 5 RSW geometry appears to create the most acceptable hydraulic and biologic characteristics and is the recommended design to be carried into the next design phase.

Table 1. General Model Flow Conditions

RSW/SB	Q Total, kcfs	Q SW, kcfs	Q RSW/SB, kcfs	Observed Conditions
SB 20	250	56	18.9	Broad crest on SB permits primary & secondary wave reflection u/s of ogee. Some roostertails. Dye trace injected 2 bays away towards sw pulled thru SB
RSW 2	250	60.8	15.3	Increased zone of influence in immediate vicinity of RSW, but about same as SB @ distance. Roostertail similar to in sectional model, more pronounced than with SB collector. Flow depth greater in center of bay than along sides.
RSW 7	250	60.8	15.3	Similar to RSW 2. Roostertails about same as with RSW 2, no improvement. Flow depth greater along sides than in middle of bay.
SB 20	250	131.2	18.9	SB draws flow very well from both spillway and powerhouse sides of forebay. Zone of influences extends from center Of Bay 20 to PH unit 14.
RSW 2	250	142.4	15.3	RSW draws flow almost as well as SB. Influence zone extends into Bays 18 & 19 and well into PH unit 14.
SB20	250	0	18.9	Draws less strongly than w/spillway flow. Influence zone about mid-SB 18 to about SW bay 17/18. Tailrace egress poor, large eddy pulls SB flow to N. side of stilling basin.
RSW 2	250	0	15.3	Draws from about same general area as w/SB but concentrates more collection from directly u/s of the RSW bay.
SB 20	250	33.6	18.9	Draws from SB 18 to SW bay 16/15. Influence extends 100 ft or less u/s of PH.
RSW 2	250	33.6	15.3	Draws from SB 19 to SW bay 16. Draws more heavy concentration from center of bay than does SB collector. Influence extends about 150 ft u/s of SW.

Table 1 (continued)

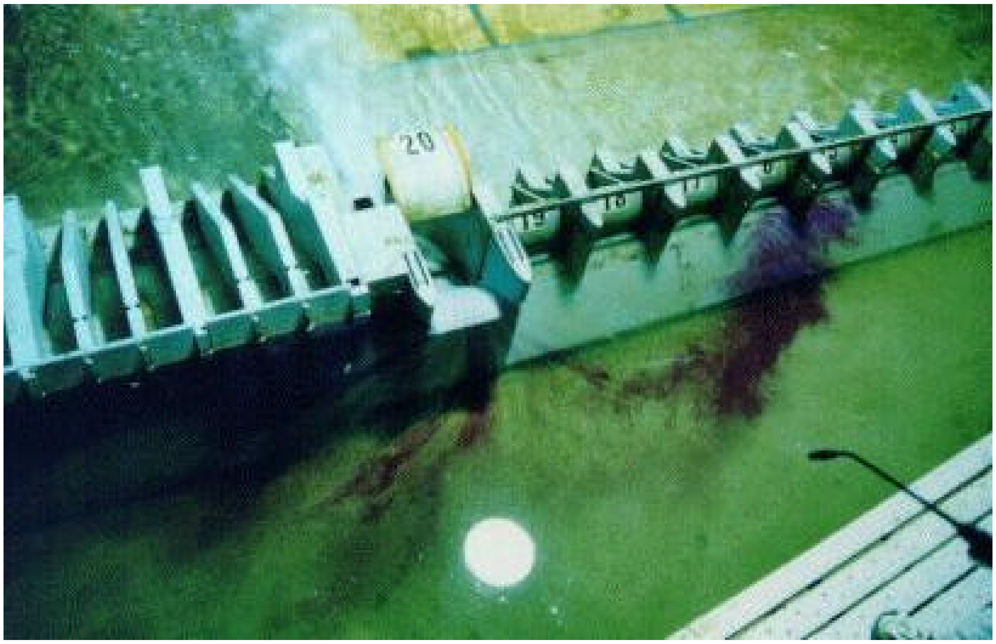
RSW/SB	Q Total, kcfs	Q SW, kcfs	Q RSW/SB, kcfs	Observed Conditions
SB 20	350	86.4	18.9	Draws from SB 18/19 joint to SW bay 19 to depth of 40 ft. Draws from up to 100 ft u/s of PH. Egress fair, clockwise eddy d/s of basin may draw in fish.
RSW 2	350	91.2	15.3	Draws from SB 19/20 joint to SW bay 18/17 to depth of 40 ft. High draw from up to 120 ft u/s of PH. Egress better than w/SB, very small eddy.
SB 20	350	192.0	18.9	Draws from PH unit 16/SB 17 joint to SW bay 20 to depth of 40 ft.. Heavy draw up to 70 ft u/s of PH. Egress good, but some counterclockwise eddy circulates to PH tailrace. SB influence stronger towards PH than SW.
RSW 2	350	204.8	15.3	Draws from PH unit 16/SB 17 joint to mid-bay SW bay 19. Not as much flow from 40 ft depth as w/ SB collector. Draws up to 100 ft u/s of crest, further u/s than does SB collector.
SB 20	350	30.4	18.9	Draws from SW bay 18 to mid-point SB 19 at surface. Stronger draw from 40 ft depth. Draw extends about 80 ft u/s of PH. Egress good, no circulation obs.
RSW 2	350	30.4	15.3	Draws from mid-PH unit 20 to SW bay 17/16 joint on surface and immediately in front of RSW 2 to SB 17/18 at 40 ft depth. Concentrated draw up to 100 ft u/s of crest. Egress conditions good, some clockwise circulation d/s of SW.
RSW 7	350	30.4	15.3	Not as strong lateral draw as w/ SB 20 or RSW 2. Concentrated draw up to 75 ft u/s of RSW face. Lateral draw limited to PH unit 20/RSW joint to SW bay 19/RSW joint. Draw from depth slightly better than RSW 2.

Photograph 1. RSW Alt 2, Water Surface Drawdown Around Piers

<u>Photograph 2</u>. RSW Alt 2, Standing Waves & Roostertails Down Spillway Face

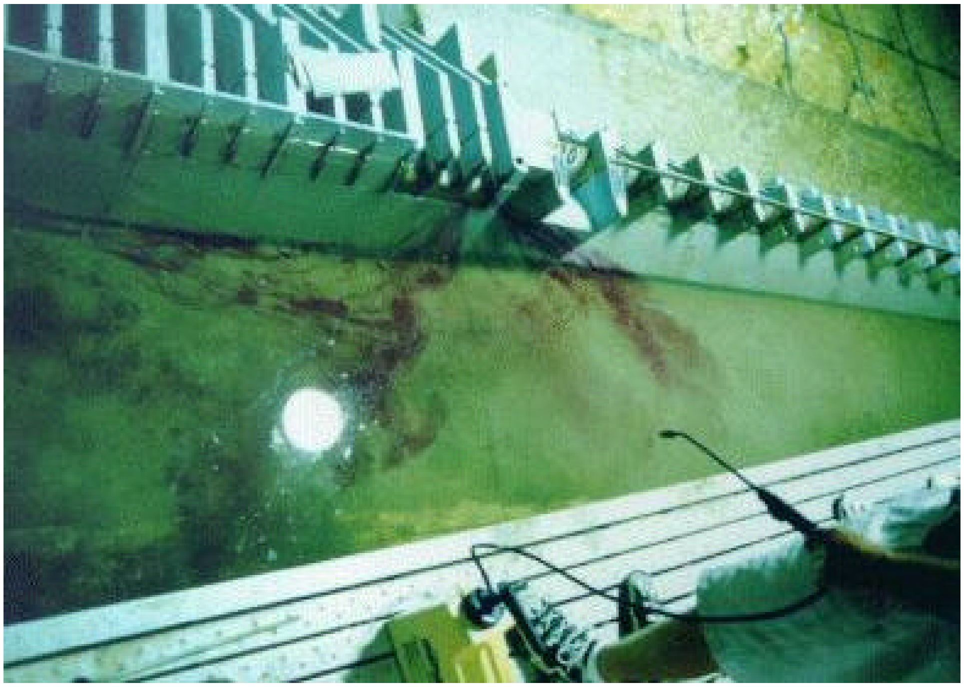
<u>Photograph 3</u>. RSW Alt 2, Roostertails Down Spillway Face

<u>Photograph 4</u>. RSW Alt 2, Approach Conditions, 15.3 kcfs, Total Q 250kcfs, 30% Total Flow Over Spillway (60.8 kcfs)

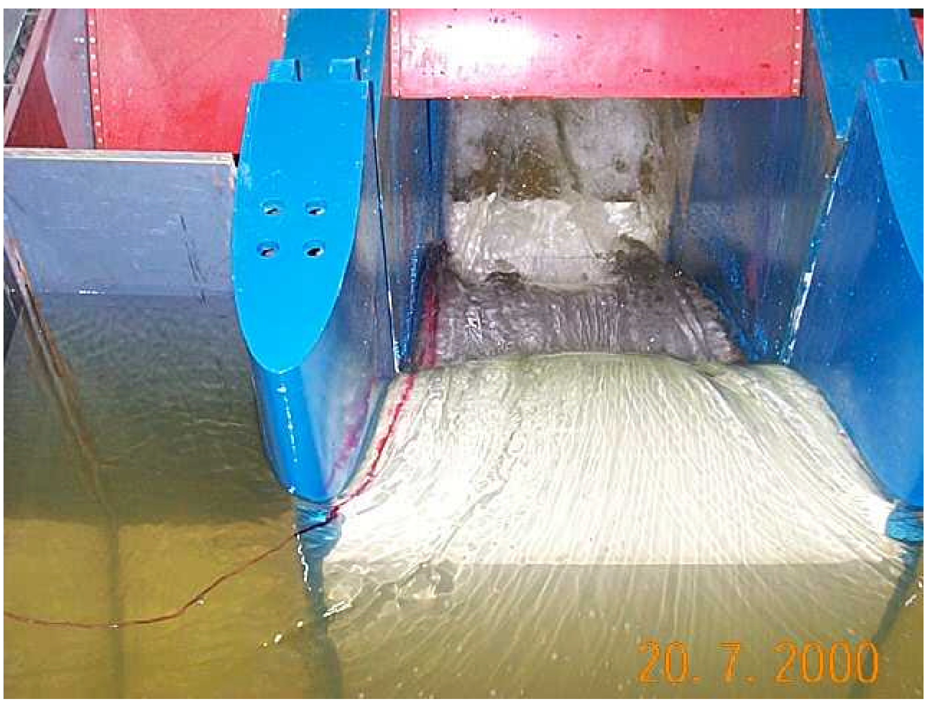


Photograph 5. **RSW Alt 2, Approach Conditions, 15.3 kcfs, Total Q 250kcfs, 60% Total Flow Over Spillway (131 kcfs)**

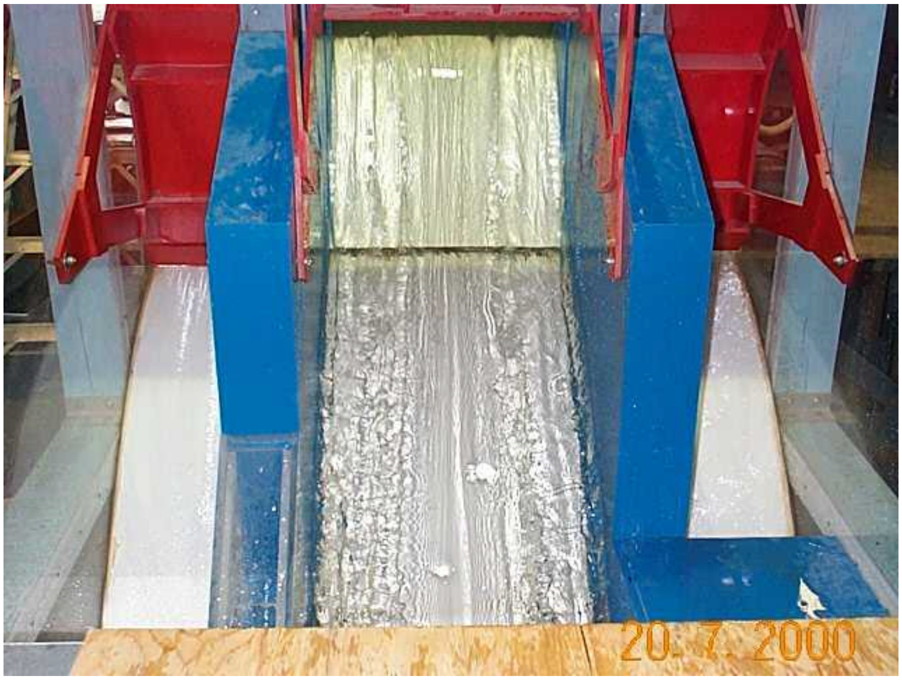
Photograph 6. **RSW Alt 2, Approach Conditions, 15.3 kcfs, Total Q 350kcfs, 30% Total Flow Over Spillway (91.2 kcfs)**



Photograph 7. **Skeleton Bay Collector, Approach Conditions, 18.9 kcfs, Total Q 250kcfs, 30% Total Flow Over Spillway (56 kcfs)**

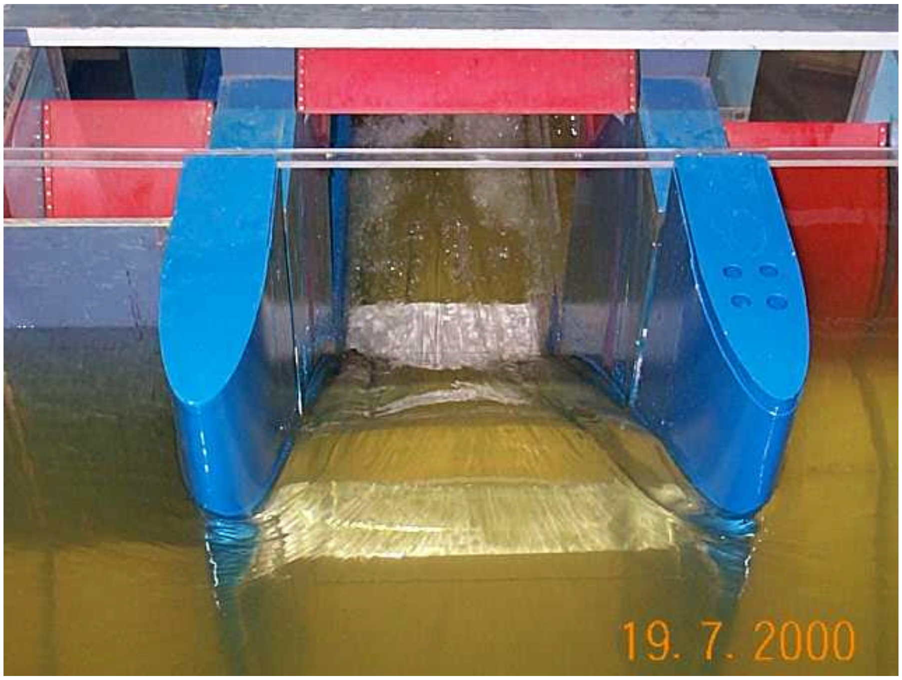


Photograph 8. **RSW Alt 4, Drawdown Around Piers and Flow Over Crest**



<u>Photograph 9</u>.   RSW Alt 4, Standing Waves Both Up and Downstream of Step

<u>Photograph 10</u>. RSW Alt 4, Standing Waves & Non-Uniform Flow Down SW Face



Photograph 11. RSW Alt 5, Drawdown Around Piers & Flow Over Crest

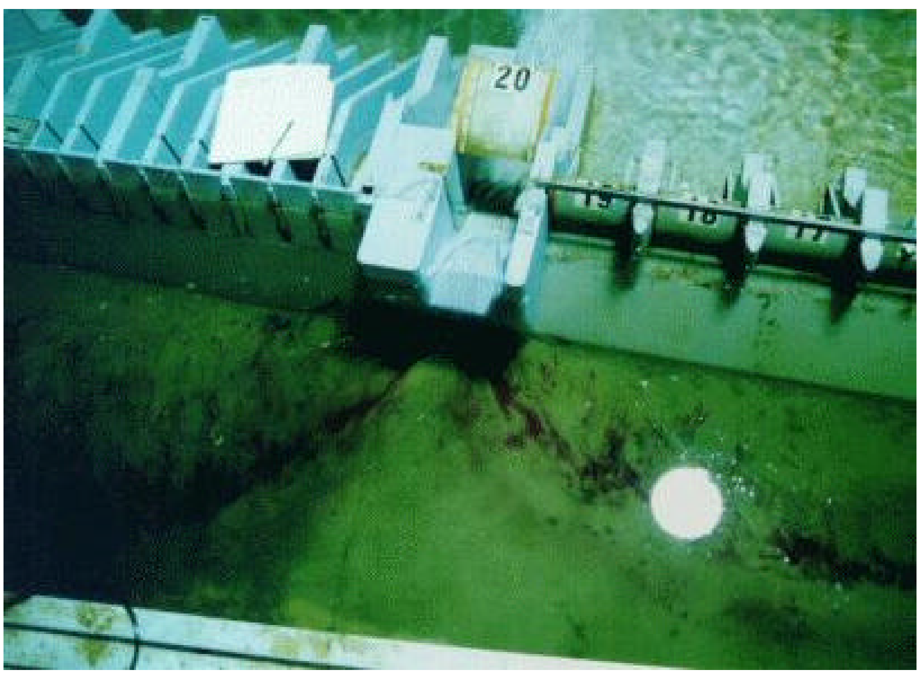
Photograph 12. RSW 5, Flow Down Spillway Face

<u>Photograph 13</u>. RSW Alt 5, Flow Down Spillway Face

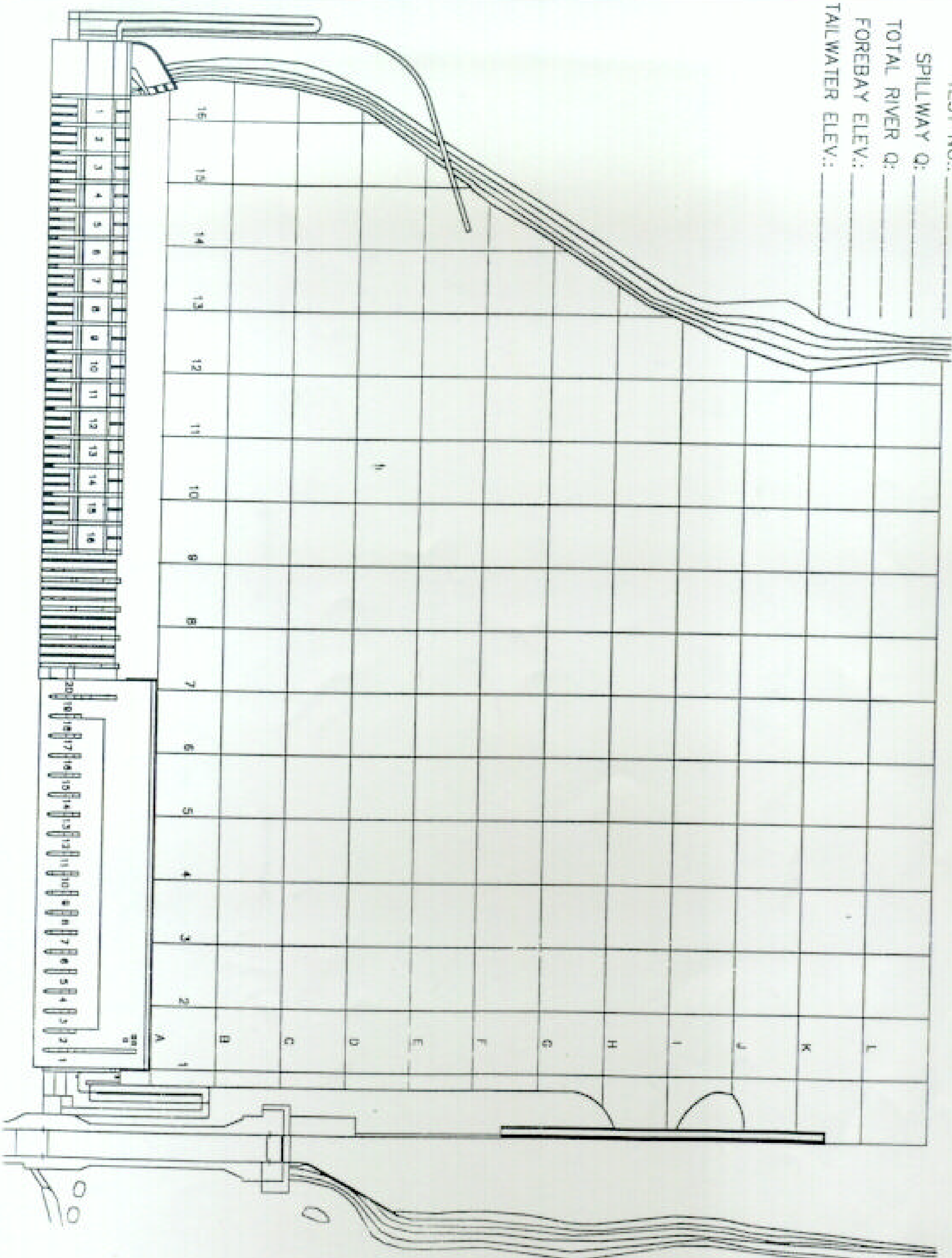
<u>Photograph 14</u>. RSW 7 (Simplified). Standing Waves on Spillway Face

<u>Photograph 15</u>. RSW Alt 7 (Simplified). Flow Conditions at Deflector

Photograph 16. **RSW 7 (Simplified). Flow Conditions With Spillway Gate Control**



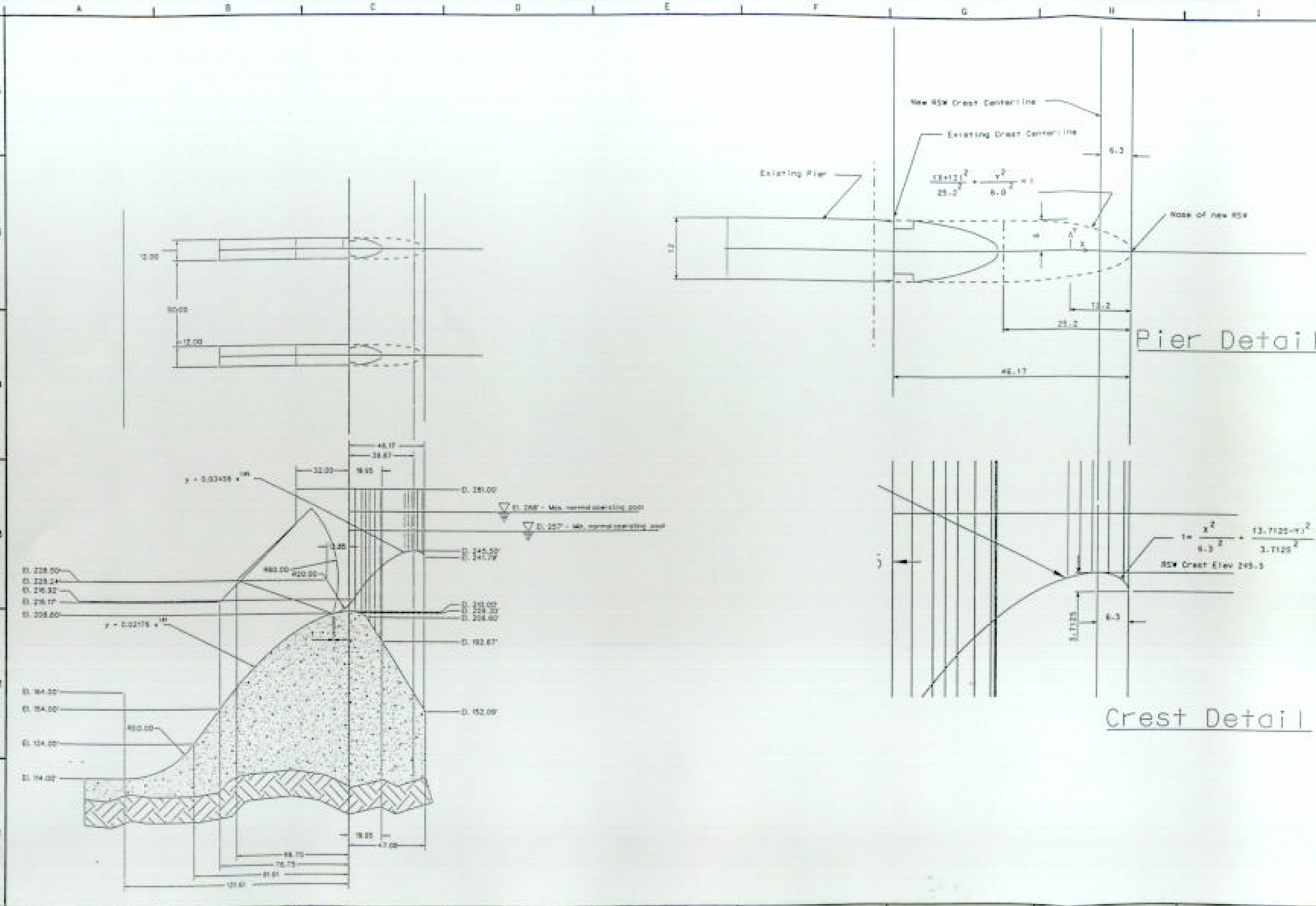
Photograph 17. **RSW 7. Approach Flow Conditions, 15.3 kcfs, Total Q 250 kcfs, 30% of Flow Over Spillway (60.8 kcfs)**



JOHN DAY DAM

GENERAL MODEL

FIGURE 1



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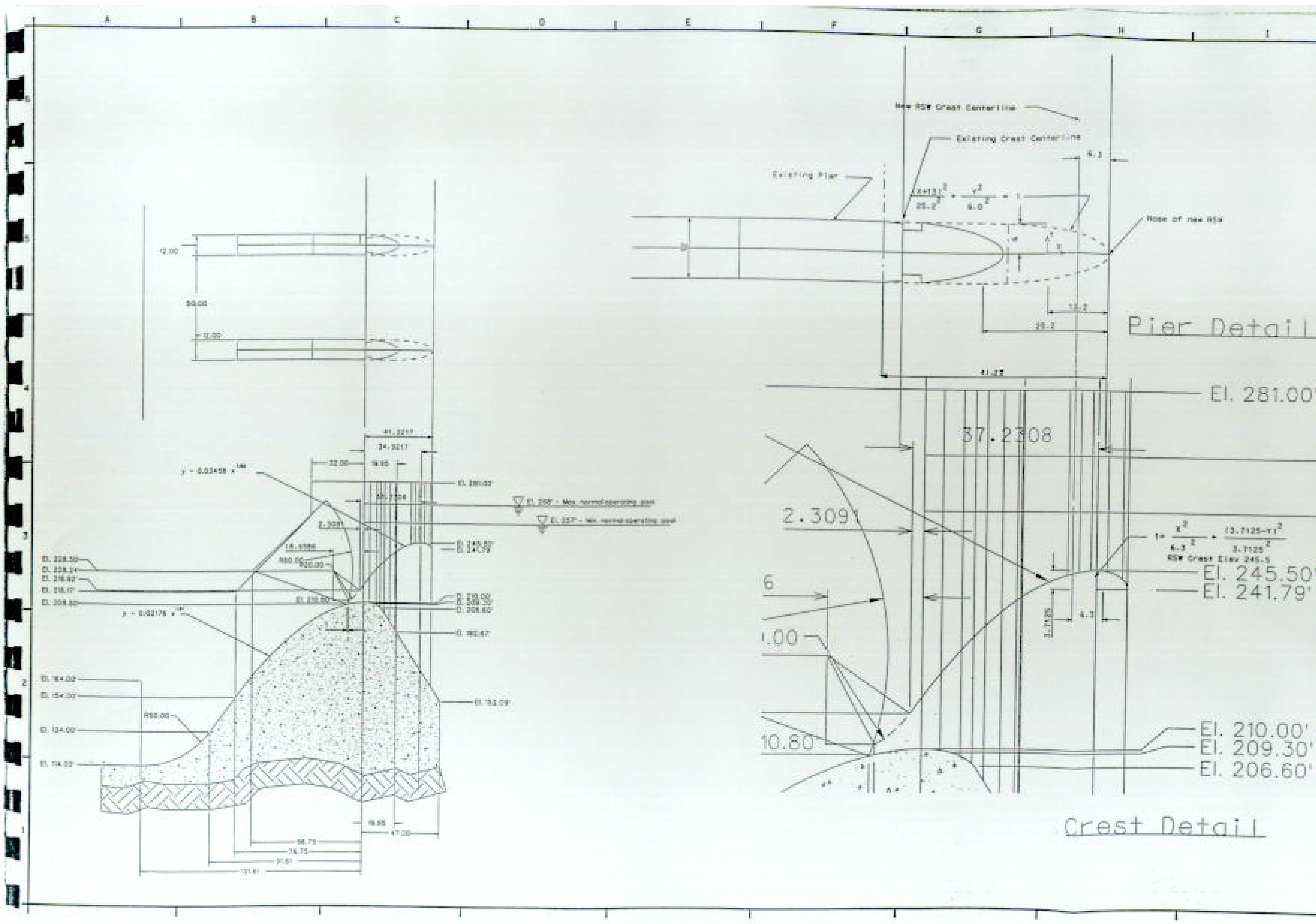
CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE
NORTHWEST HYDRAULICS CONSULTANTS

Designed by: E. ZAPEL
Date: 16 JUNE 2000

COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 2
VERTICAL FACE
PIERS 46' UPSTREAM

FIGURE 2



COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 4
VERTICAL FACE
PIERS 41' UPSTREAM
STEP AT SPILLWAY OGEE INTERFACE

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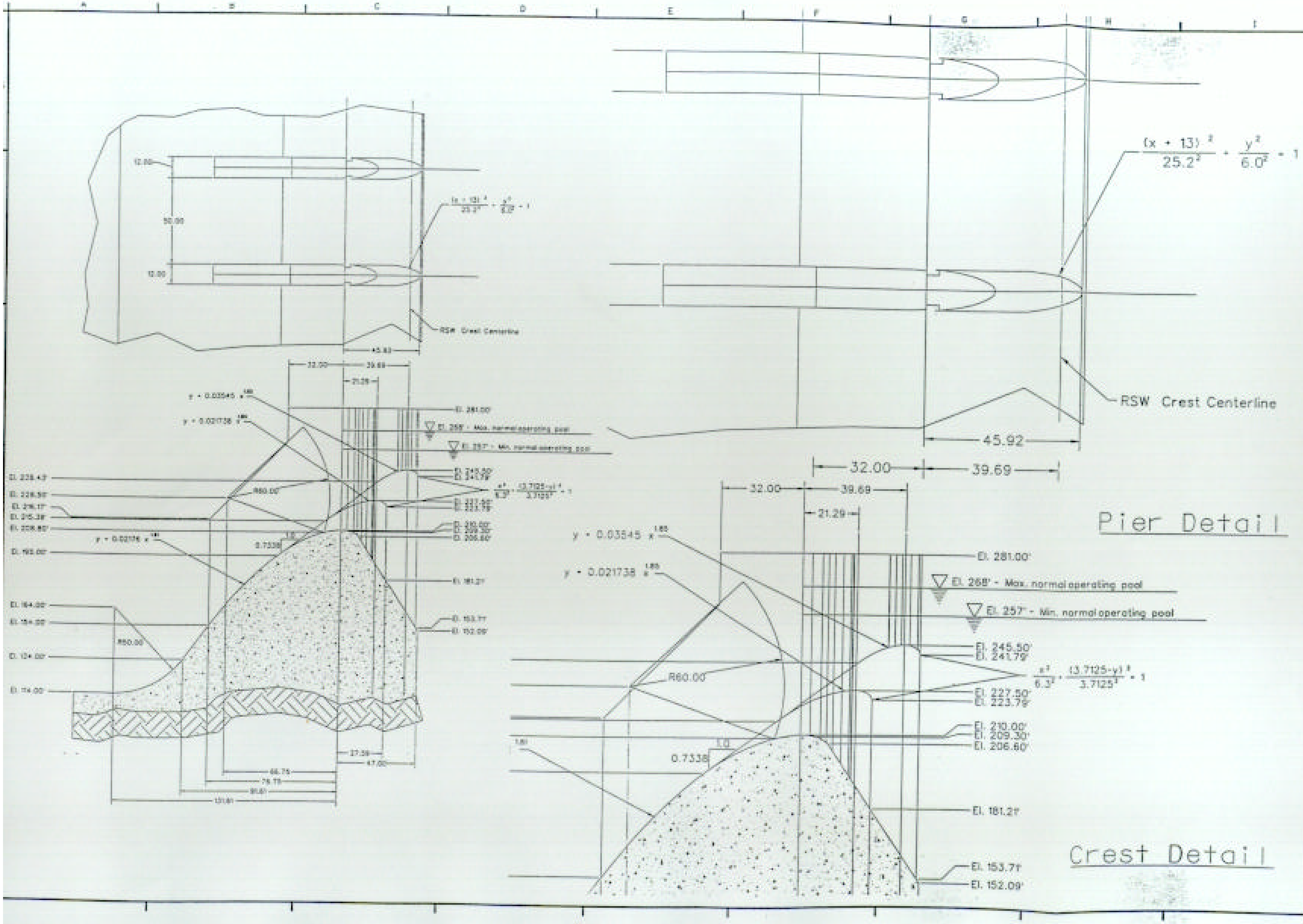
NORTHWEST HYDRAULICS
CONSULTANTS

Designed by: Date:
Drawn by: CADD File Name:
Checked by: Technical Manager:
Submitted by:

US Army Corps of Engineers Portland District

DRAWING STATUS:
DRAWING NO.

FIGURE 3.



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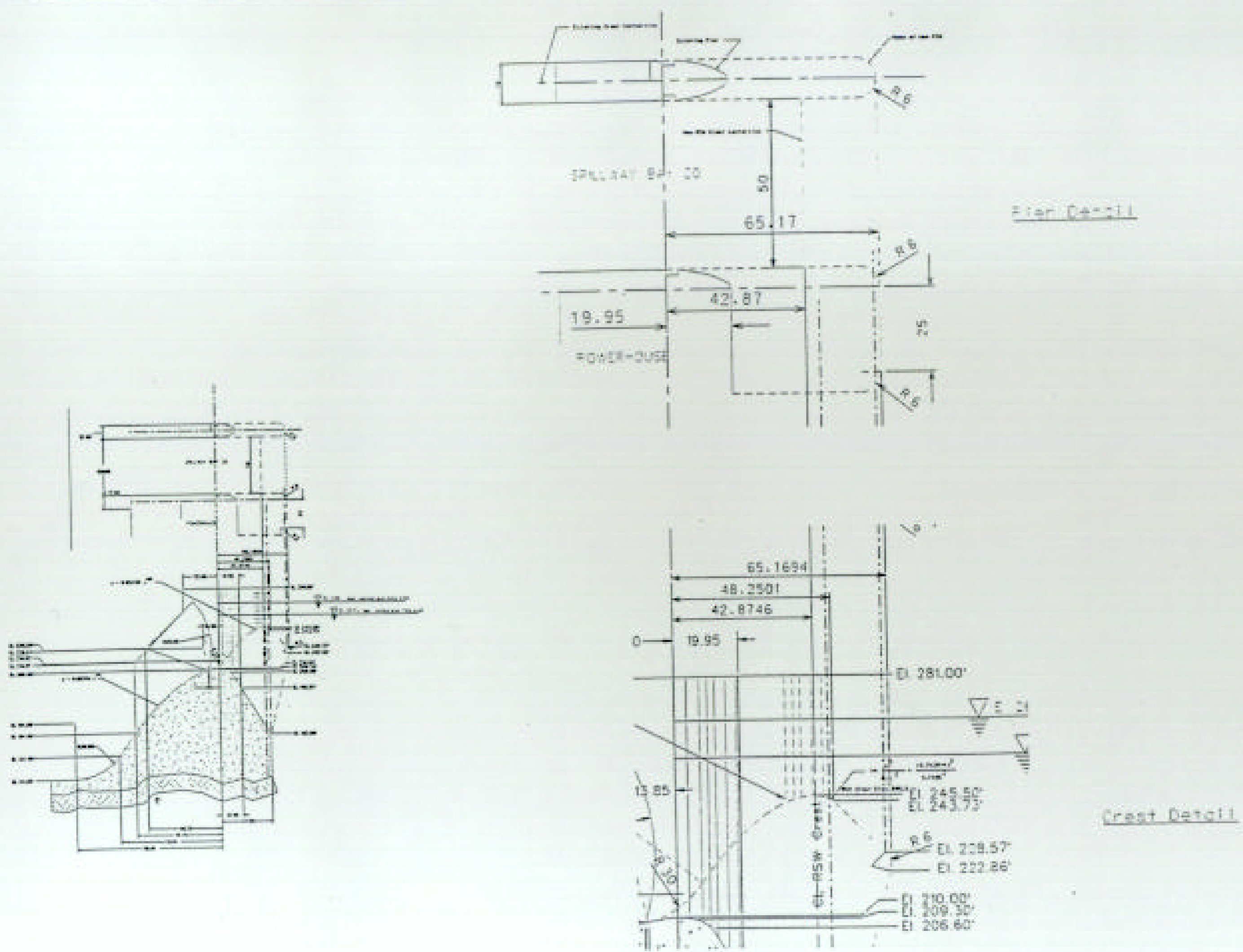
COLUMBIA RIVER — OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 5
SEMI-PERMANENT LOWER CREST
PIERS 46' UPSTREAM

Designed by: E. ZAPEL
Date: 15 JUNE 2000

FIGURE 4



ALTERNATIVE #7
Short Sloping Face, Piers 65 ft Upstream
Adapted to John Day RSW Criteria

US Army Corps of Engineers
Portland District

Designed by: E. ZAPEL	Date: 16 JUNE 2000
Drawn by:	CADD File Name:
Checked by:	Technical Manager
Submitted by:	

U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON

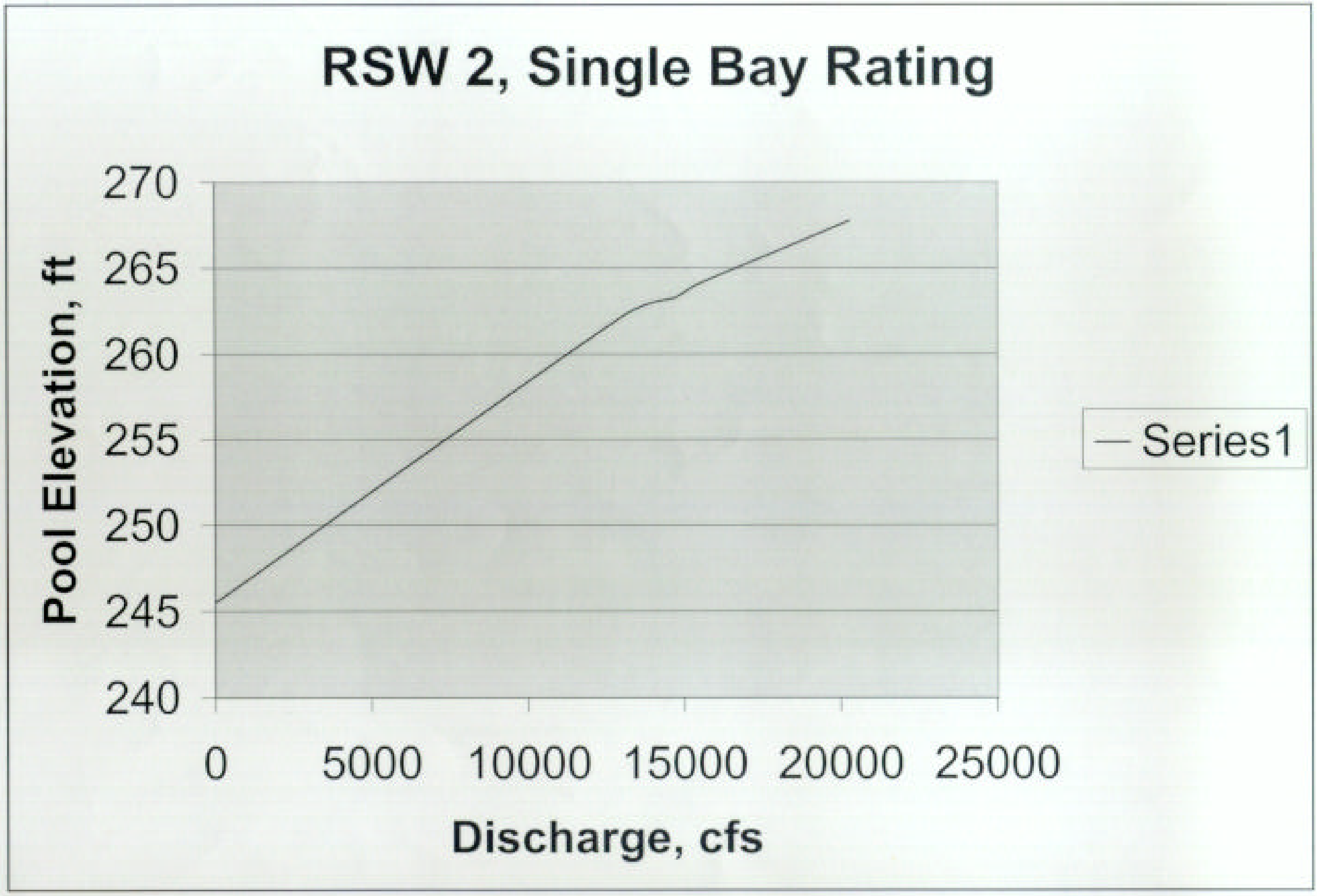
CH2M HILL
MONTGOMERY WATSON
JOINT VENTURE
NORTHWEST HYDRAULICS
CONSULTANTS

COLUMBIA RIVER — OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALTERNATIVE 7
SLOPING FACE
PIERS 65' UPSTREAM

DRAWING STATUS:

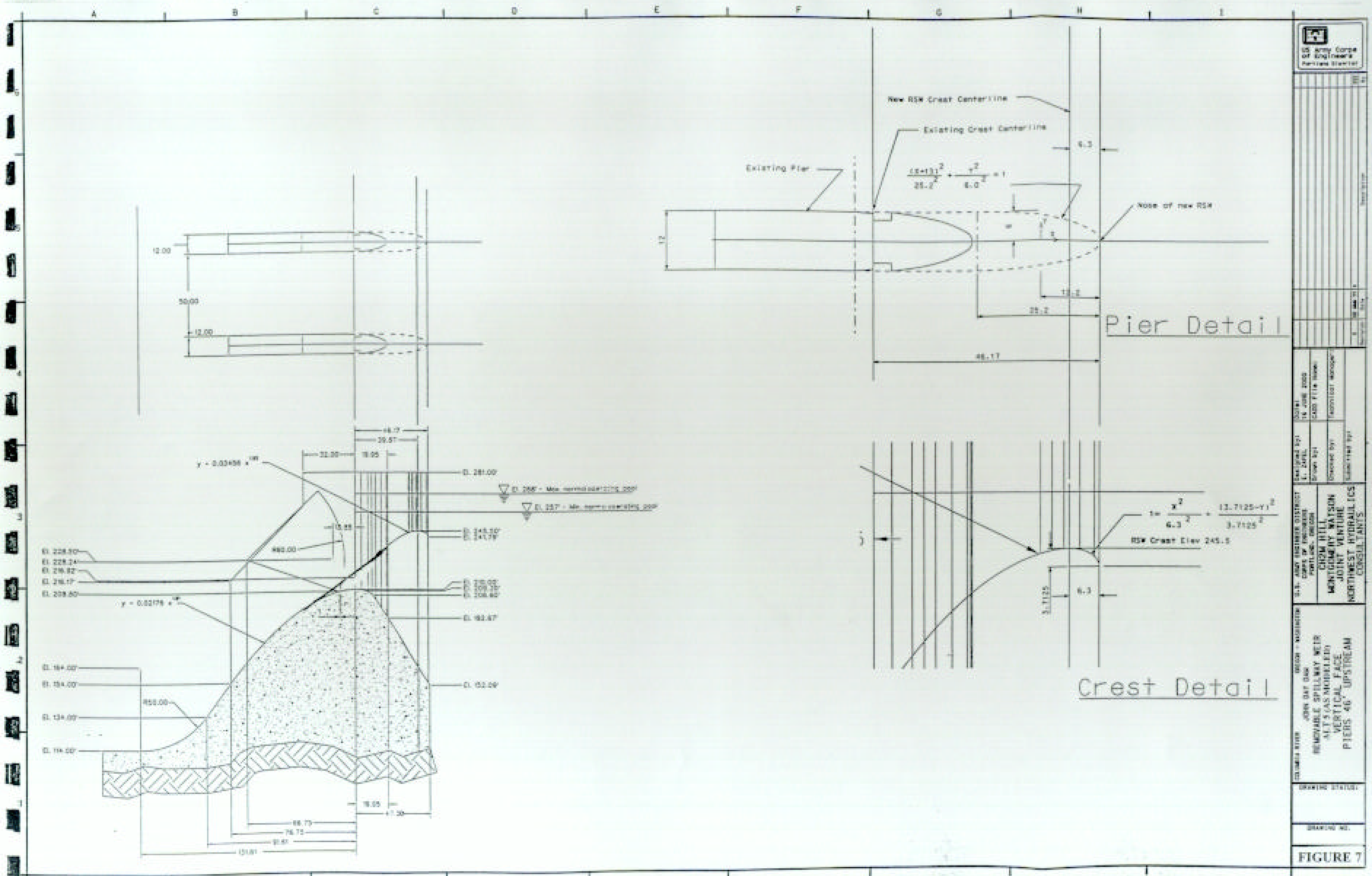
DRAWING NO.

FIGURE 5



RSW 2, Single Bay Rating
Pool Elevation, ft
270
265
260
255
250
245
240
0
5000
10000
15000
20000
25000
Discharge, cfs
Series1

FIGURE 6



Pier Detail
Crest Detail
New RSW Crest Centerline
Existing Crest Centerline
Existing Pier
Nose of new RSW
RSW Crest Elev 245.5
12.00
50.00
46.17
29.57
22.00
19.05
25.2
13.2
6.3
3.7125
y = 0.03456 x^1.85
y = 0.02176 x^1.85
R60.00
R50.00
EL 281.00'
EL 268' - Max normal/existing pool
EL 257' - Min normal/existing pool
EL 245.50'
EL 241.79'
EL 228.50'
EL 228.24'
EL 216.82'
EL 216.17'
EL 209.60'
EL 210.00'
EL 209.35'
EL 208.60'
EL 192.47'
EL 164.00'
EL 154.00'
EL 152.09'
EL 134.00'
EL 114.00'
68.73
76.73
91.61
131.61
US Army Corps of Engineers Portland District
Designed by: E. ZAPEL
Date: 16 JUNE 2000
Drawn by:
CADD File Name:
Checked by:
Technical Manager:
Submitted by:
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS PORTLAND, OREGON
CH2M HILL MONTGOMERY WATSON JOINT VENTURE
NORTHWEST HYDRAULICS CONSULTANTS
COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
ALT 5 (AS MODELED)
VERTICAL FACE
PIERS 46' UPSTREAM
DRAWING STATUS:
DRAWING NO.
FIGURE 7

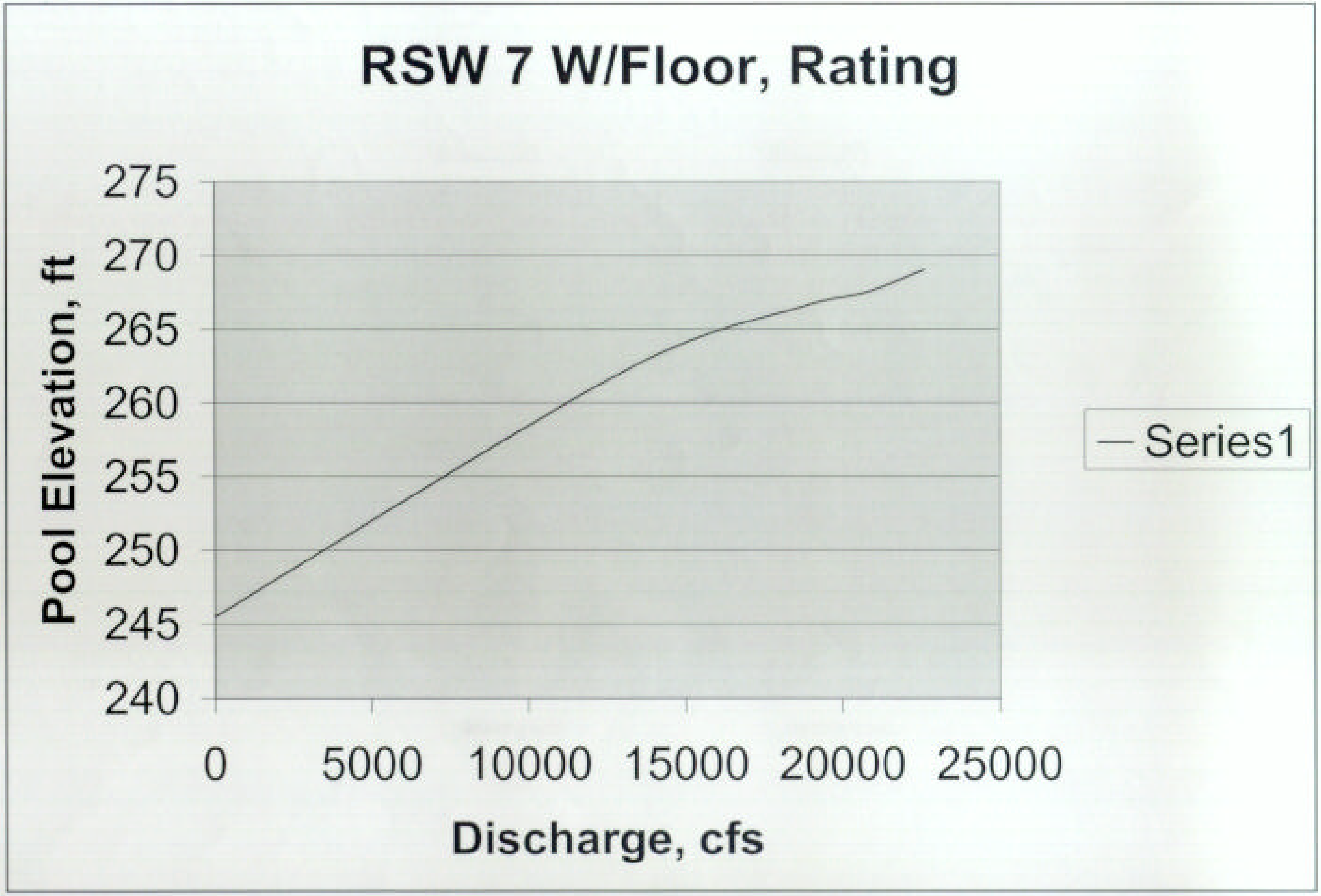

RSW 7 W/Floor, Rating
Pool Elevation, ft
275
270
265
260
255
250
245
240
0
5000
10000
15000
20000
25000
Discharge, cfs
Series1

FIGURE 8

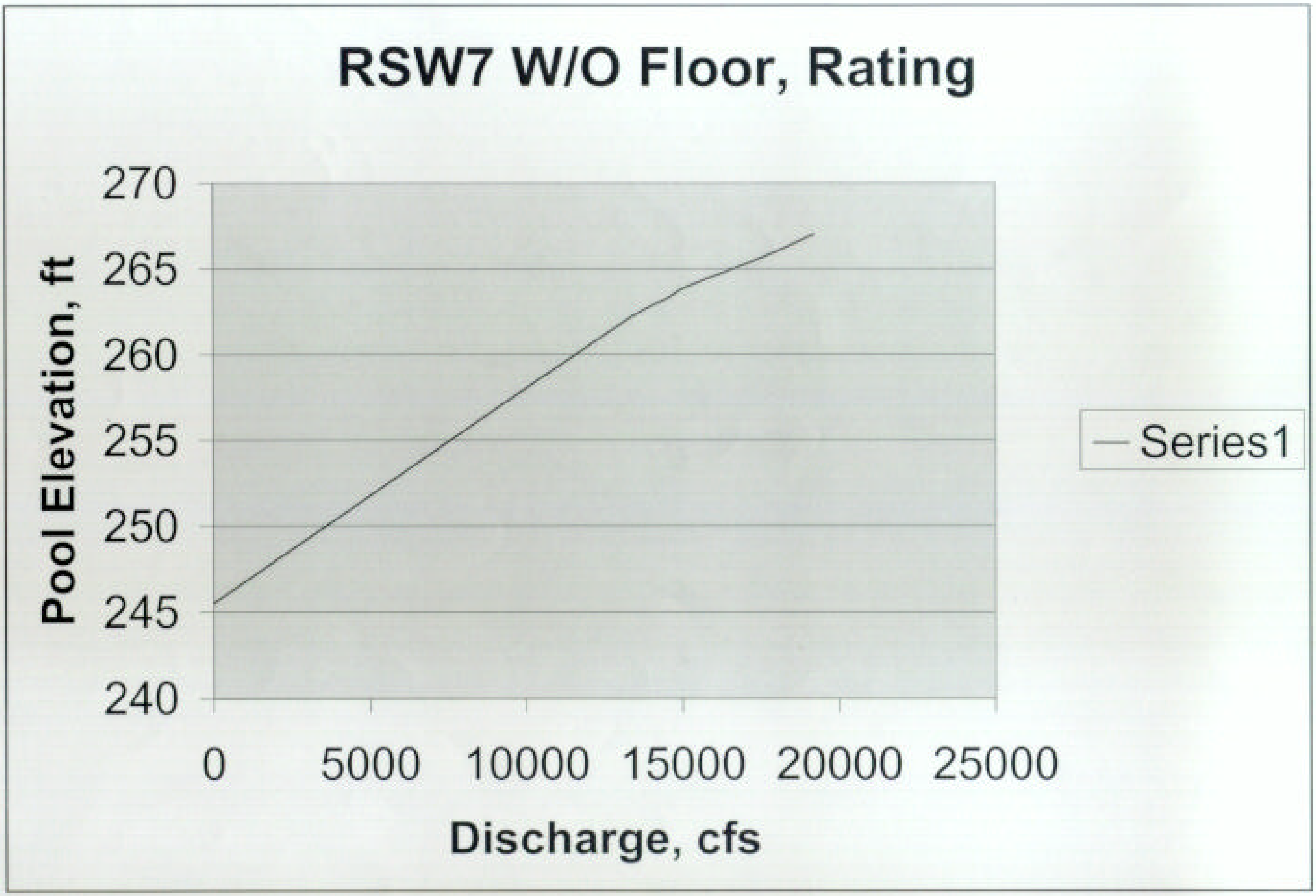

RSW7 W/O Floor, Rating
Pool Elevation, ft
270
265
260
255
250
245
240
0
5000
10000
15000
20000
25000
Discharge, cfs
Series1

FIGURE 9

APPENDIX F

Hydraulic Analysis of RSW Alternatives

APPENDIX F. HYDRAULIC ANALYSIS OF RSW ALTERNATIVES

1. RSW Design

A three phased design process was developed for the John Day Dam RSW. The initial phase of design consisted of conceptual hydraulic design of six alternatives that were considered to have some potential of emulating the hydraulic characteristics of the skeleton bay SBS. The next design phase consisted of preliminary physical model testing of some of the conceptual alternatives to serve as a "Proof of Concept" (POC) that the designs selected would in fact emulate the SBS hydraulic performance. As a further refinement in the design process, Optimum RSW alternatives were developed which were considered to not only emulate the SBS performance, but to actually have potential to improve upon the performance exhibited by the SBS. A more detailed discussion of this design process is included in the subsequent paragraphs.

2. Similar RSW Design for Lower Granite Dam

Walla Walla District is evaluating a similar concept at Lower Granite Dam, which will be prototype-tested in April 2001. Plans and Specifications for the Lower Granite RSW are to be completed sometime during 2000. The Lower Granite RSW is being designed for 6,000 cfs, while the John Day design flow is greater than 14,000 cfs. The John Day RSW is designed to simulate the flow conditions through the Skeleton Bay SBS. In addition, the configuration of the spillway RSW sections at the two projects are somewhat different, and the John Day skeleton bay discharge objective is greater than that for the Lower Granite RSW.

Four different Lower Granite RSW designs were developed, but the general crest geometry for all were similarly shaped and all were to be placed entirely upstream of the existing spillway radial gate. Three of the four concepts were designed for about 15 feet of submergence below the maximum normal operating pool elevation, and the fourth concept was designed for submergence of 21 feet below the maximum normal operating pool elevation. All four concepts include an ogee crest which transitions to the existing spillway face through a radius bucket, with the true tangent intersection occurring upstream of the existing spillway radial gate seal beam. The reader is referred to the 90% submittal document for the Lower Granite RSW design (JE Sverdrup March 2000) for a more detailed discussion of the Lower Granite RSW structure.

3. Skeleton Bay Surface Flow Bypass (SBS) Spillway Design

The design of the Skeleton Bay SBS was developed by Montgomery Watson to the Feature Design Memorandum (FDM) level (US Army Corps of Engineers, September 1998) in a previous study completed for the Portland District. Portland District, Corps of Engineers Hydraulic Design Section staff accomplished the hydraulic design with input from the National Marine Fisheries Service and other regional fisheries resource agencies.

The SBS design was intended to convert one or more of the existing powerhouse skeleton bays into a surface spill fish bypass by removing the top of the powerhouse and constructing three large open channel spillway chutes with deflectors through each powerhouse skeleton bay monolith. Each of the 3 chutes per skeleton bay would have a broad crest extending about 30 ft across the full breadth of the upstream upper powerhouse structure, then a chute extending over the existing turbine pits and down to a deflector below the existing downstream lower powerhouse deck. Two interior piers within each Skeleton Bay SBS would be 7 ft wide, and the exterior piers at the joint between skeleton bay units would be 13 ft wide. Total width of each skeleton bay is 90 ft and each of the three chutes would be 21 ft wide.

Model studies were conducted with the SBS geometry at the Corps Engineering Research Development Center (ERDC) to confirm hydraulic performance. The SBS was tested in both the 1:80 scale John Day general model, and in the 1:40 scale sectional model. Results were favorably reviewed by regional fisheries resource agencies. Data collection included velocity measurements in the forebay approaching the entrance to the SBS, on the chute, and in and around the discharge jet from the deflector into the tailwater. Qualitative observations made in the physical models included the upstream zone of influence of the SBS, approach velocity, and downstream egress characterization.

3.1 Surface Bypass Spillway 'Proof of Concept'

From 1995 to 1998 the Corps of Engineers, along with input from the regional fishery resource agencies, developed a Skeleton Bay Surface Bypass Spillway design for John Day Dam which utilized the four skeleton bay units in the powerhouse. However, following development of the FDM (US Army Corps of Engineers, September 1998), which found that the Skeleton Bay SBS was more expensive than originally anticipated, the Corps and fishery agencies decided to pursue a means of verifying the anticipated performance of such an SBS system. During the resulting exploration for alternatives, the concept of a Removable Spillway Weir (RSW) was conceived. This RSW would be installed temporarily in Spillway Bay 20 and would be used to determine the potential effectiveness of a large surface collector prior to funding the large capital costs of the Skeleton Bay SBS. The "Proof of Concept" (POC) RSW is intended to perform this function without compromising the existing spillway design flood discharge capacity.

The RSW concept was first developed for the Walla Walla District's Lower Granite Dam project, where similar fish passage issues exist. The RSW is generally described as a hollow steel structure that is filled with air for floating and towing into place. In the vicinity of the spillway bay, selective filling of the structure would occur to rotate the structure to vertical. Once vertical, the RSW would be moved into place and further submerged until it rests on support brackets permanently mounted on the spillway. The RSW would be designed to flow free of gate control during operational testing. The Lower Granite design, even though having different fish passage goals, flow criteria and design considerations, was considered to have application at John Day Dam and was considered a good candidate for testing the surface collection success at the John Day project. To serve as a POC for a Skeleton Bay SBS, the RSW should be located as close to the skeleton bays as possible in Spillway Bay 20, and should be designed to have similar flow attraction characteristics as the Skeleton Bay SBS. Being removable, an alternative means of passing the spillway design flood would not have to be considered. The RSW could be designed as either a POC for the SBS or could be designed to be a permanent bypass. The potential for a different geometry is possible if the RSW is not required to mimic the Skeleton Bay SBS.

4. Initial RSW Concepts and Model Alternatives Report

A total of six alternatives were developed to the conceptual level of hydraulic design. The six were presented to the National Marine Fisheries Service, Columbia River Inter-Tribal Fish Commission staff, and Corps personnel at an Alternatives Selection Meeting on 9 May, 2000 held at the John Day Project. The level of hydraulic design was limited to development of configurations based on previous Skeleton Bay SBS collector work, Lower Granite Dam RSW work, and rough calculations of approximate velocities, discharge capacities, and water surface profiles in the vicinity of the RSW structure. The selected design/s have been developed more fully in this DDR study, and the performance of the selected design/s have been documented in a 1:25 scale sectional physical hydraulic model of the spillway and a larger 1:80 scale general model of the John Day project located at ERDC.

Alternative 1 – Skeleton Bay Geometry w/ Piers 88' Upstream
Alternative 2 – Vertical Face RSW w/ Piers 46' Upstream
Alternative 3 – Sloping Face RSW w/ Piers 129' Upstream
Alternative 4 – Vertical Face RSW w/ Piers 41' Upstream, step at spillway ogee interface
Alternative 5 – Vertical Face RSW w/ Piers 46' Upstream w/ semi-permanent Lower Crest
Alternative 6 – Sloping Face RSW w/ Piers 124' Upstream, step at spillway ogee interface

The Model Alternative Report, presented to the District during the site visit/kickoff meeting on 9 May 2000, is furnished in Appendix D of this DDR.

5. 'Proof of Concept' RSW Alternative Concepts

Of these six initial RSW alternatives, five are presented below as the POC RSW geometries. The sixth alternative, Alternative 6, is presented as the Alternative A Optimum design in section 3.7 of this appendix. Figures F-1 through F-5 illustrate these five conceptual designs.

Alternative 1 (Figure F-1) – Skeleton Bay Geometry w/ Piers 88' Upstream
Alternative 2 (Figure F-2) – Vertical Face RSW w/ Piers 46' Upstream
Alternative 3 (Figure F-3) – Sloping Face RSW w/ Piers 129' Upstream
Alternative 4 (Figure F-4) – Vertical Face RSW w/ Piers 41' Upstream, step at spillway ogee interface
Alternative 5 (Figure F-5) – Vertical Face RSW w/ Piers 46' Upstream w/ semi-permanent Lower Crest
Alternative 6 is presented in this DDR as Optimum Alternative A.

The paragraphs below provide a more detailed discussion of the hydraulic characteristics of each of the remaining five POC RSW conceptual designs. The design selected for initial physical modeling was Alternative 2. Alternative 4 was later also tested in the physical model after

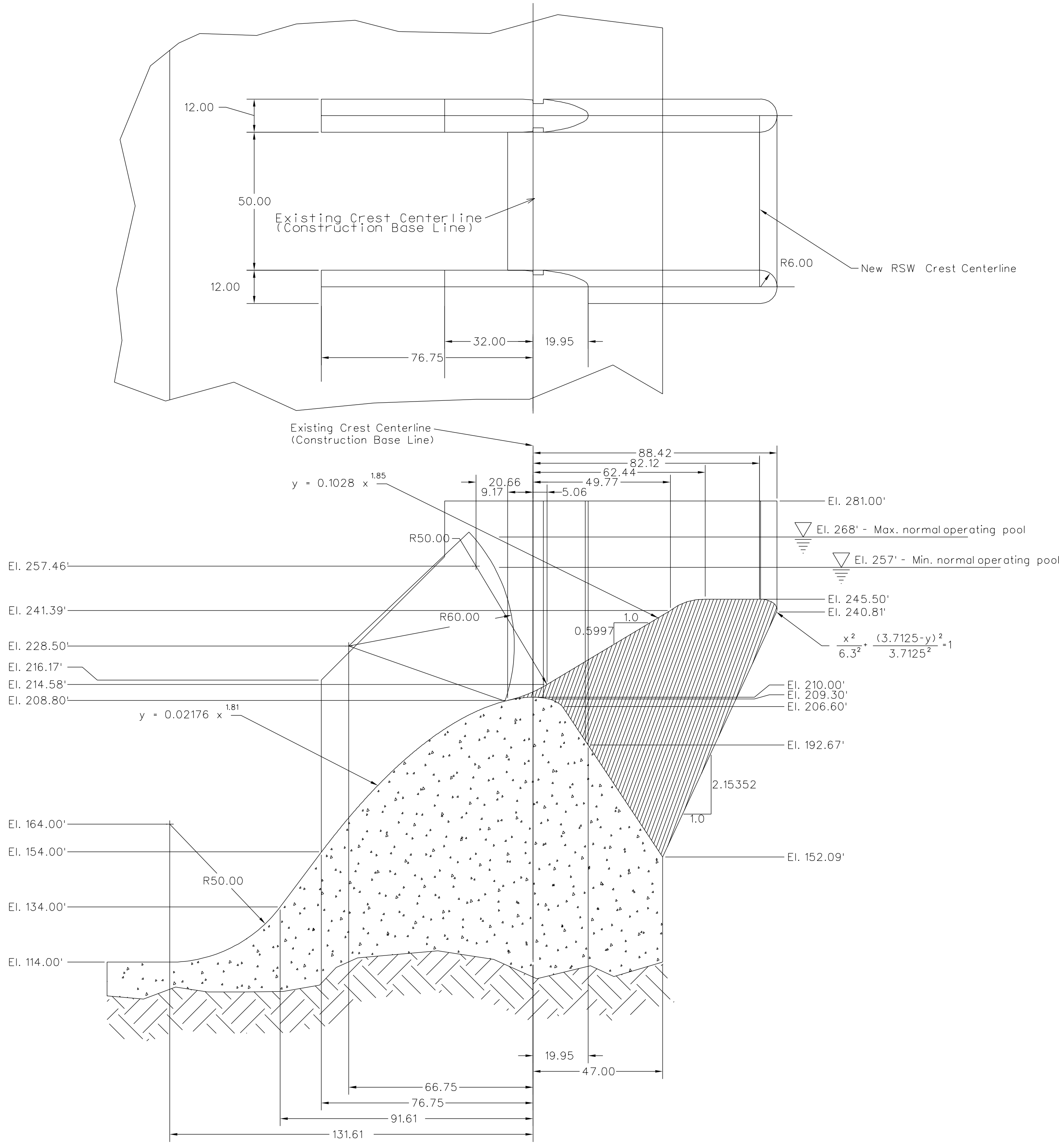


Figure 3-1
Proof of Concept
Alternative #1
Skeleton Bay Geometry with Piers 88' Upstream

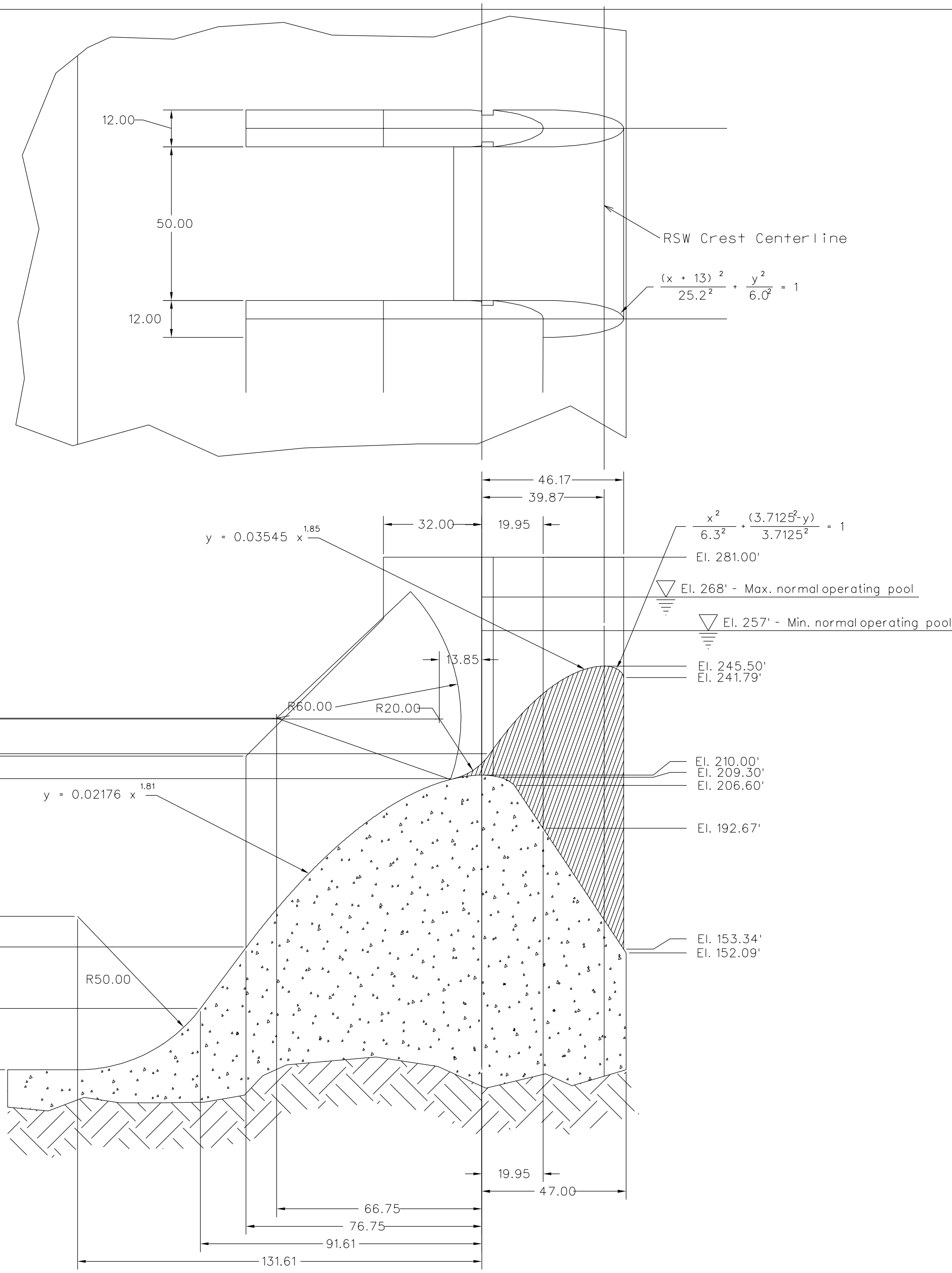


Figure 3-2
Proof of Concept
Alternative #2
Vertical Face RSW with Piers 46' Upstream

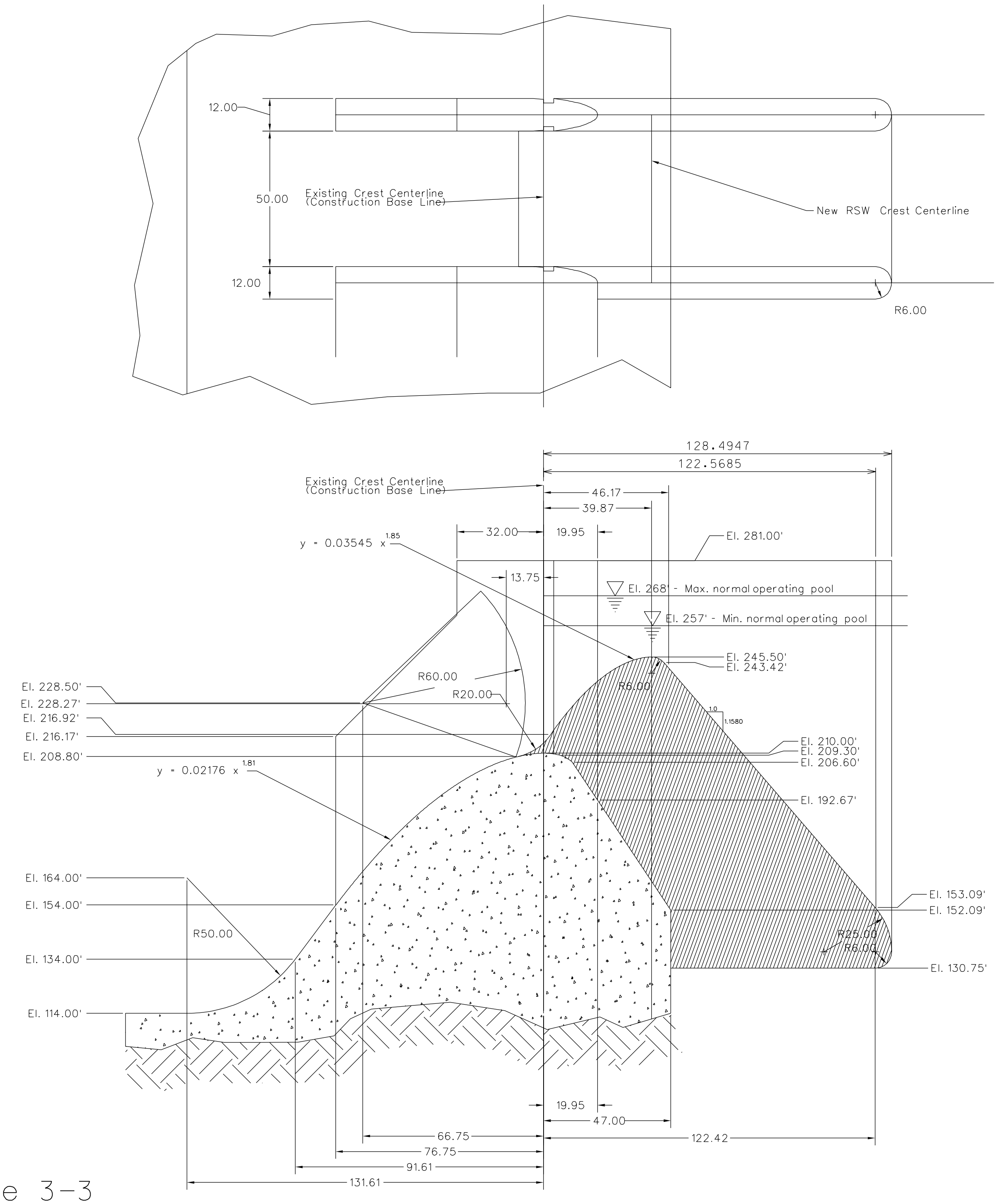


Figure 3-3
Proof of Concept
Alternative #3
Sloping Face RSW with Piers 129' Upstream

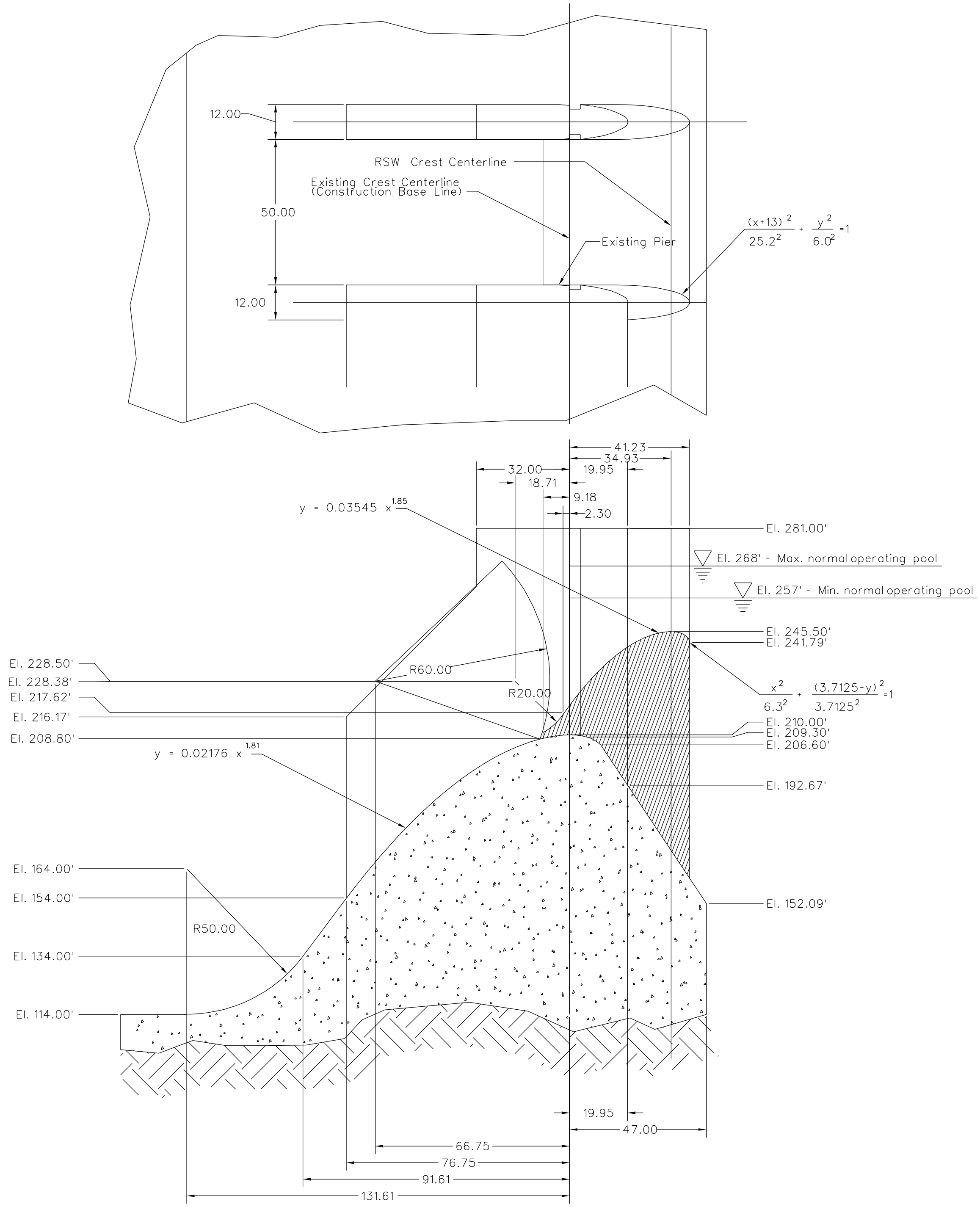


Figure 3-4
Proof of Concept
Alternative #4
Vertical Face RSW with Piers 41' Upstream,
Aeration Step at Spillway Ogee Interface

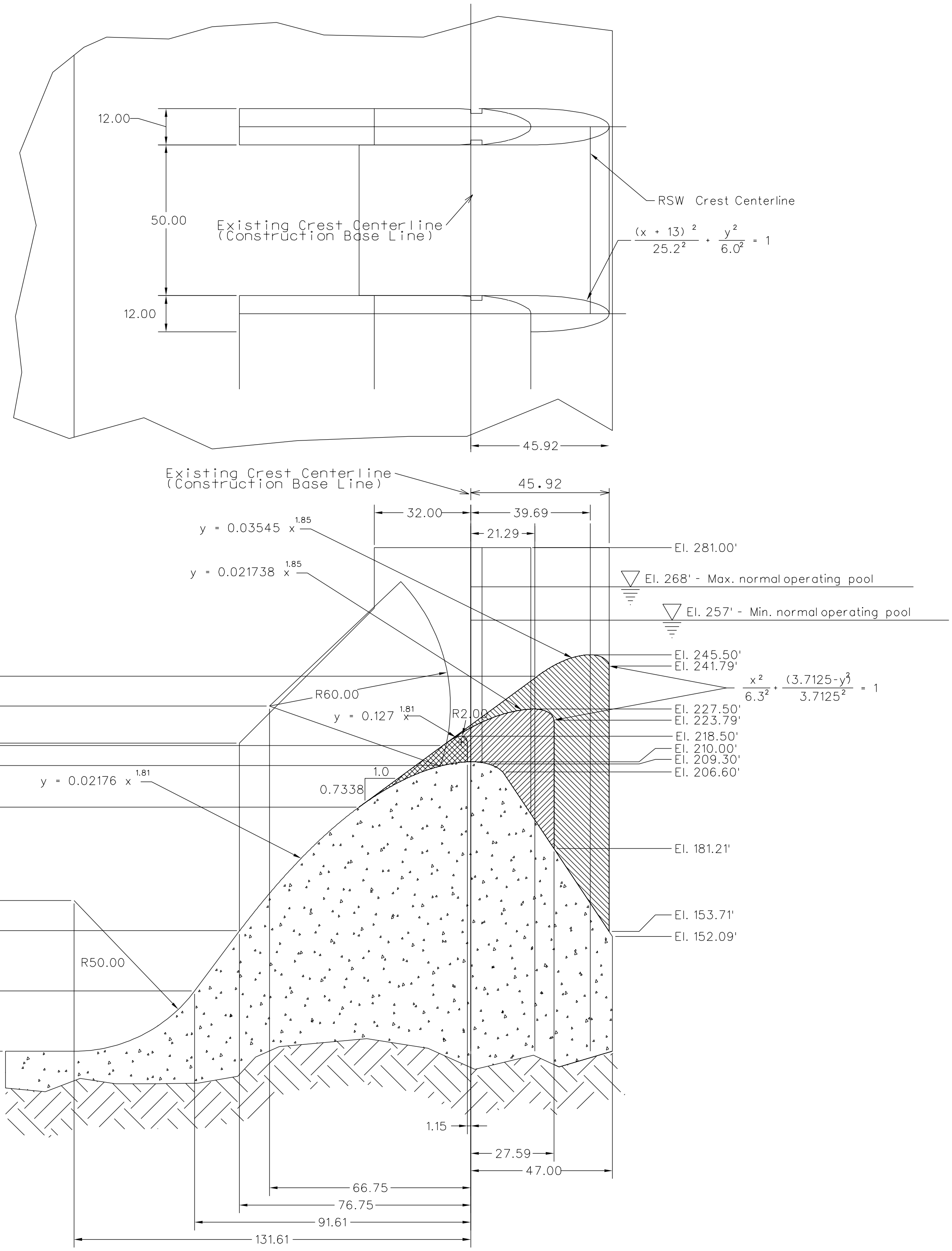


Figure 3-5
Proof of Concept
Alternative #5 (Initial Concept)
Vertical Face RSW with Piers 46' Upstream

undesirable hydraulic conditions were observed with the Alternative 2 configuration. Paragraph 8 of this appendix provides more detailed discussion of model results.

Alternative 1 was patterned after the Skeleton Bay SBS design, developed previously by the Portland District (US Army Corps of Engineers, September 1998). The RSW crest section is a broad crested weir with elevation 245.5 ft msl (NGVD), length of 50 ft, breadth of about 25 feet, with an upstream approach ellipse and downstream ogee ellipse, a constant slope chute, and a 20 ft radius bucket at the toe. The downstream toe terminates with a thickness of about 1 inch at a point just upstream of the true tangent intersection with the existing crest. The true tangent point is located about 1 foot upstream of the existing spillway radial gate seat. Capture velocity of 7 fps is reached upstream of the entrance section, and as a result, the maximum acceleration criteria of 0.1 fps/ft need not be met inside the RSW entrance.

Alternative 2 was generally patterned after the design of the 'Flow-Efficient' RSW for Lower Granite Dam (JE Sverdrup, March 2000). This design minimizes the distance the structure extends out into the forebay. The Alternative 2 RSW is designed for a maximum operating head (He) of 22.5 ft at normal high operating pool elevation 268 ft to emulate the head on the Skeleton Bay SBS. As such, the crest elevation is 245.5 ft and the crest length is 50 ft. The RSW crest shape was developed in accordance with guidance contained in EM 1110-2-1603. The downstream quadrant of the RSW has a design head (Hd) of 22.5 ft so that He/Hd equals 1.0 and the crest equation is:

$$Y=(0.03545)(X)^{1.85}$$

The upstream quadrant is the standard elliptical shape defined by the EM with a major axis of 6.3 ft and a minor axis of 3.7125 ft. The crest discharge coefficient is estimated to be about 4.0, therefore discharge capacity (considering pier losses) is expected to be about 17,000 cfs to 21,000 cfs with pool elevations 265 ft and 268 ft, respectively. The RSW crest and piers extend about 46 feet upstream of the existing spillway crest axis Construction Base Line (CBL). The RSW ogee shape transitions to the existing crest through a 20 ft radius curve bucket tangent at a point about 1 ft upstream of the existing spillway gate seat. Piers would extend to near the upstream vertical face of the dam with a similar nose shape as that for existing piers. Capture velocity of 7 fps is reached upstream of the entrance section. Consequently there is no need to meet the maximum acceleration criteria of 0.1 fps/ft within the RSW entrance.

Alternative 3 shares the same RSW ogee crest shape as Alternative 2, except that the entrance consists of a sloping ramp with adjacent piers that extend about 124 ft upstream of the existing CBL. This sloping ramp was designed to lower the velocity at the RSW entrance to 2 fps and meet the maximum acceleration criteria of 0.1 fps/ft. The pier shape for this alternative would be a simple 6 ft radius.

Alternative 4 shares a similar RSW crest and pier shape with Alternative 2, except that the downstream toe bucket terminates at a step 2 ft above the existing spillway crest, instead of a tangent interface. The RSW extends about 41 feet upstream of the existing crest centerline. The step would be vented to the atmosphere to provide for air entrainment into the bottom boundary layer of the spillway jet in order to raise potentially low pressures that might occur in the interface zone and to more uniformly distribute flow across the spillway face downstream from the RSW.

Alternative 5 also shares similar ogee and pier shape with Alternatives 2 and 4, except that the ogee crest transitions to a constant slope chute that joins the existing crest at a tangent point downstream of the existing spillway gate. The RSW would be constructed in two pieces, with the upper crest section detaching from the tailpiece section at a point upstream of the spillway gate seat. The purpose of this design was to eliminate the radius transition between the RSW and existing spillway, which would cause disruption to flow. Two different tailpiece alternative configurations are shown in Figure F-5. The large tailpiece would be a semi-permanent addition to the existing spillway crest. The large tailpiece would not be easily removable. It was considered a distant possibility only, and was designed to permit free-flow without gate control, whereas the small tailpiece would not be used without gate controlled discharge. The small tailpiece would be semi-permanently installed on the spillway crest, and would extend upstream to the downstream edge of the stoplog slots and downstream of the spillway gate seat. Removal of the small tailpiece section could be accomplished in the dry after the upper crest section was removed by placing bulkheads across the existing piers and in contact with the upstream face of the spillway structure. However, the tailpiece section would not be easily and rapidly removable.

6. Final 'Proof of Concept' RSW Design

The selected final POC RSW configuration is the Alternative 5 geometry identified above in the initial RSW configurations, except that the intermediate elevation crest has been deleted. Figure F-8 illustrates the design of the final POC RSW alternative. The design was also adopted as Alternative C of the 'Optimum RSW' configurations. Detailed model testing of the final selected RSW configuration was conducted during preparation of the DDR.

The Alternative 5 RSW geometry was selected following evaluation of Alternative 2 and Alternative 4 in the sectional physical model. Standing waves, unstable and oscillating hydraulic ridges and roostertails existed on both the Alternatives 2 and 4 RSW and spillway chute as a result of the reverse curvature bucket transition. However, when a continuous tangent sloping chute joined the RSW and existing spillway instead, the undesirable conditions were eliminated. The Alternative 5 geometry accomplished this objective by providing the desired smooth transition from the RSW crest section to the existing spillway chute.

Alternative 5 was considered acceptable only after determining that the RSW could be assembled in two individual sections, one upstream of the spillway gate and one under the gate and downstream of the stoplog slots. The shape of the downstream portion (tailpiece) of the crest was developed following evaluation of two alternative shaped sections in the physical model.

6.1 Crest Design

The final POC RSW configuration crest design is of standard ogee shape, designed for 22.5 ft of head at maximum normal operating pool elevation 268 ft msl (NGVD). The RSW will have a finished, in-place width of 50 feet, the same as the existing spillway crest. The RSW is comprised of the crest and piers on either side and will be floated into place. Bulb seals will provide positive head seal between the RSW and existing piers. Bulb type seals will also provide positive closure against reservoir head at the downstream toe of the crest section.

The RSW is designed as a two-piece installation, with a much smaller, lower, tail piece installed downstream of the existing stoplog slots and under the spillway radial gate. The larger RSW crest section (main structure) is installed upstream of the existing spillway radial gate and would fit with the tailpiece.

The large main section of the RSW is designed in accordance with guidance found in the COE design manual EM 1110-2-1603 Hydraulic Design of Spillways. The RSW is designed for 22.5 ft head, with an ogee crest shape as shown in Figure F-8 in Paragraph 7. The upstream face of the RSW crest is a standard elliptical approach (EM 1110-2-1603), with a longitudinal axis of 6.3 ft, and a vertical axis of 3.7 ft. The ogee crest transitions into a sloping chute, which joins the tail piece at a true tangent point just upstream of the radial gate seat on the RSW. The downstream toe or lip of the RSW will be connected to the tailpiece section with a true tangent intersection. The design of the connection between the downstream end of the upper crest section and the tailpiece section will be developed to eliminate any irregularities across the joint. The minimum computed cavitation index for this location is about 0.7, which requires a very smooth connection to prevent cavitation from damaging the structure.

The initial alternative tailpiece section design was a standard ogee shaped crest in accordance with the EM in an attempt to provide a design to allow satisfactory overflow conditions with only the tailpiece section in place. However, the ogee portion of the tailpiece section must fit in the space between the downstream edge of the stoplog slot and the spillway gate seat on the face of the spillway to accommodate placement and removal of the tailpiece. This space is limited to about 8 ft; therefore the maximum allowable design head for the downstream quadrant is about 5 ft. The downstream quadrant equation is:

$$Y = 0.127(X)^{1.85}$$

The downstream quadrant terminates at a distance of 7.01 ft downstream from the crest of the tailpiece ogee. This location is about 1 ft upstream from the gate seat. The remaining portion of the tailpiece extends downstream 30 ft on a constant slope of 0.7338 V: 1.0 H where it becomes tangent to the existing spillway face. The tailpiece crest elevation is 218.5 ft. Therefore, at normal high operating pool elevation 268 ft, the free flow head (He) is 49.5 ft and the ratio of He/Hd is 9.9. The minimum acceptable He/Hd ratio to prevent initiation of cavitation for a head of 49.5 ft is about 1.17 (Plate 3-13, EM 1110-2-1603), therefore free flow operation of the RSW with only the tailpiece section in place was not expected to be acceptable. If only the tailpiece itself were in place under free flow conditions, pressures as low as absolute zero could be expected on the face of the tailpiece section. Such conditions could lead to failure of the tailpiece section and significant damage to the concrete on the face of the existing spillway. Subsequent pressure measurements in the physical model revealed that significantly low pressures existed on the tailpiece section both with and without gate control. The pressures were low enough to reveal that any operation with only the tailpiece section in place could potentially compromise safe operation of the spillway bay. Therefor a simpler, triangular shaped tailpiece section was ultimately selected for the final design.

6.2 Interface at Existing Spillway Surface

The downstream end of the tailpiece section will terminate somewhat upstream of the true tangent point with the spillway to provide adequate plate thickness to meet structural requirements. The offset between the end of the steel tailpiece section and the concrete spillway will be finished with a smooth transition between the tailpiece and the spillway. Pressure measurements in the physical model revealed that a sloping transition no greater than 1 vertical to 6 longitudinal was necessary to minimize cavitation potential in this area.

6.3 Pier Design

The final POC RSW pier nose shape is slightly different from the design shape of the existing John Day Dam spillway piers. The existing pier nose was developed during extensive physical model testing prior to construction of the dam. The existing spillway pier nose was shaped in the form of an ellipse having a minor axis of about 6.3 ft, slightly larger than the 6-ft half-width of the spillway pier. With this geometry, the intersection of the downstream end of the pier nose and the face of the pier itself introduced a separation point that contributed to maximizing spillway discharge efficiency. However, as illustrated in the physical model tests accomplished as part of the POC design phase, this flow separation initiated standing waves and resulted in unstable hydraulic "ridges" which extended down the entire face of the spillway. These hydraulic conditions were considered to be unacceptable with respect to safe fish passage. In an attempt to eliminate, or minimize the formation of standing waves off the pier nose, the RSW pier nose ellipse has a minor axis of 6 ft, the same as the half width of the pier. This geometry forms a true tangency at the intersection of the downstream end of the pier nose and the pier face and is expected to improve flow characteristics as compared to those existing with the existing spillway pier nose geometry. Flow conditions with the final POC pier nose were evaluated in the physical model and found to be acceptable.

In order to provide sufficient structural integrity, an abrupt away-from-the flow offset of 3-inches will exist at the connection between the downstream end of the RSW pier and the face of the existing spillway pier. Velocities of about 45 fps exist at that location and pressures of 0.4 to 2.9 ft of water were measured in the physical model. The cavitation index is therefore about 1.06. Based on guidance in EM 1110-2-1603, a cavitation index of about 0.9 can be expected to initiate incipient cavitation with a 3-inch away-from-the-flow offset exposed to the velocity and pressure conditions which will exist with free-flow over the RSW. Therefore, cavitation should not occur at the downstream end of the RSW pier.

Physical model testing of the POC design also illustrated that a large (about 4-ft) drawdown existed around the pier nose with the pier length extending 46 ft upstream of the existing spillway crest. The initial thought was that elimination of this drawdown might reduce the standing wave phenomenon observed emanating from the pier nose and, subsequently, improve hydraulic characteristics down the face of the spillway. Model testing with the piers extended approximately 18 ft further upstream (about 64 ft upstream of the spillway crest) into the forebay where lower velocities existed illustrated that the drawdown around the piers could be decreased to about 1 ft and appeared to decrease the standing wave formation. However, contrary to initial thoughts, this decrease in drawdown had no improvement on the hydraulic conditions existing down the face of the spillway. Observation of approach flow conditions in the 1:80 scale general model indicated that piers extended into the forebay about 41 ft upstream of the existing spillway crest resulted in somewhat better approach conditions than existed with piers extending about 65 ft upstream. Therefore, extending of the piers upstream beyond 46 ft from the existing spillway crest was not considered beneficial.

7. 'OPTIMUM' RSW

7.1 'Optimum' RSW Alternative Concepts

The 'Optimum' RSW alternatives consist of a total of five different configurations. Several of the five share similar characteristics and designs. Alternative A was developed during the initial RSW alternatives investigation prior to the 10% DDR submittal. Its application as an Optimum RSW design was considered appropriate, since the configuration was developed as an early refinement of one of the initial five selected alternatives, in order that fish collection efficiency and structural design might be improved. Detailed hydraulic characteristics for the five Optimum RSW alternatives are provided in Paragraph 9, including water surface profiles, velocities, and rating curves. The five Optimum designs are listed below and shown in Figures F-6 through F-10:

- Alternative A (Figure F-6) – Sloping Face RSW w/ Piers 124' Upstream, step at spillway ogee interface
- Alternative B (Figure F-7) – Short Sloping Face RSW w/ Piers 65' Upstream
- Alternative C (Figure F-8) – Vertical Face RSW w/ Piers 46' Upstream, w/ semi-permanent tailpiece
- Alternative D (Figure F-9) – Skeleton Bay Geometry w/ Piers 68' Upstream with permanent downstream chute
- Alternative E (Figure F-10) – Vertical Face RSW w/ Piers 63' Upstream, w/ semi-permanent lower step and extended piers

Alternative A is the same as the initial POC RSW Alternative 6. The upstream approach entrance to the RSW is extended at 1.16V: 1H slope to meet 0.1 fps/ft flow acceleration criteria. The piers and approach ramp extend 124 feet upstream of the existing spillway crest CBL. Pier noses are 6 ft radius. The crest consists of the standard ogee shape defined in EM 1110-2-1603 for 22.5 ft head and crest elevation 245.5 ft msl (NGVD). Refer to Paragraph 9 for specific design water surface profile data, velocity data, and discharge rating curve for this alternative. The transition step at the interface with the existing spillway ogee was designed to permit air to be entrained into the bottom boundary layer, anticipating that the air will elevate potentially low pressures at this location and eliminate the potential risk of cavitation. The approach velocity at maximum operating pool elevation does not meet the acceleration criteria in some portions of the entrance,

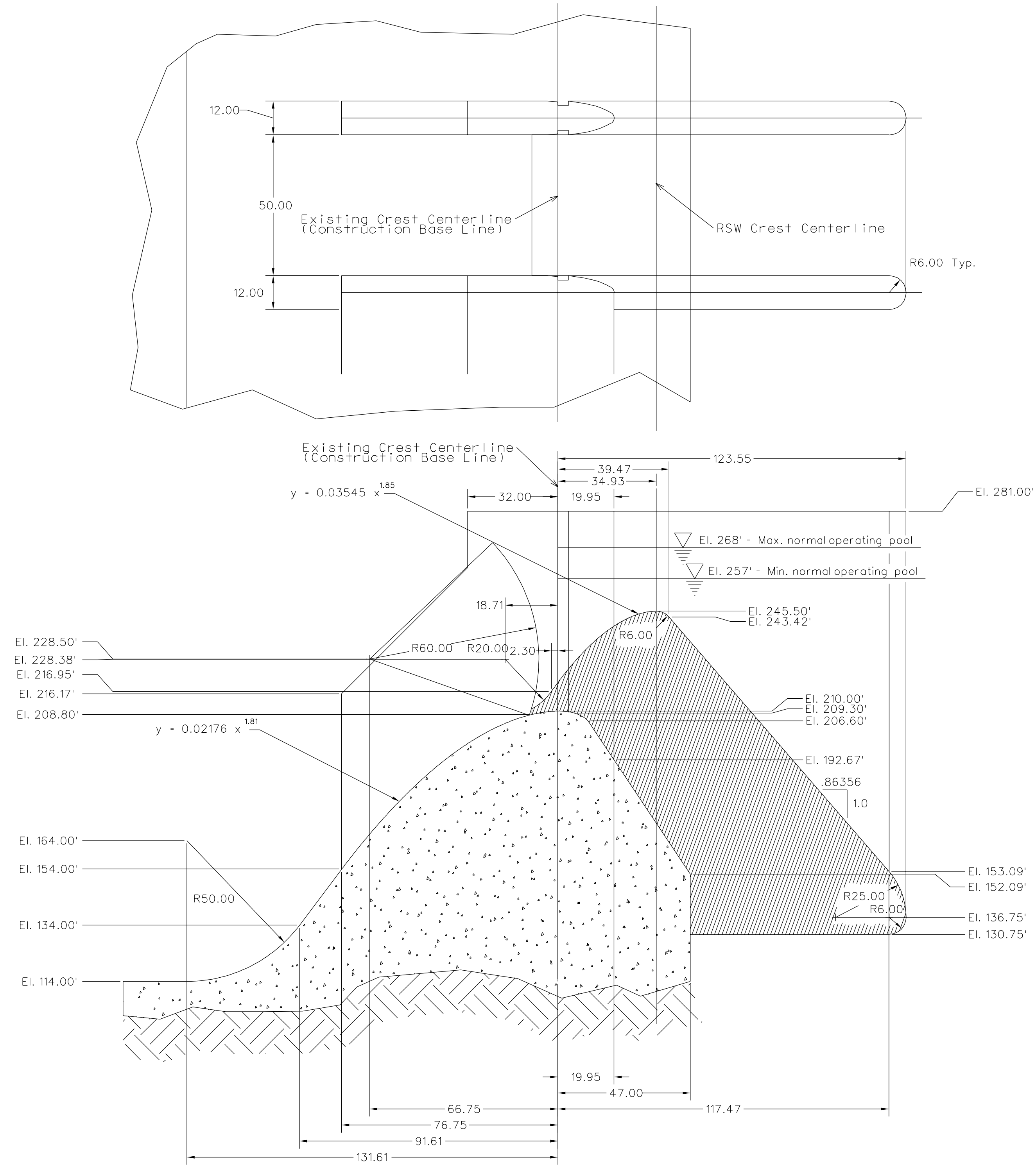


Figure 3-6
Optimum RSW Alternative A
Sloping Face RSW with Piers 124' Upstream,
Aeration Step at Spillway Ogee Interface

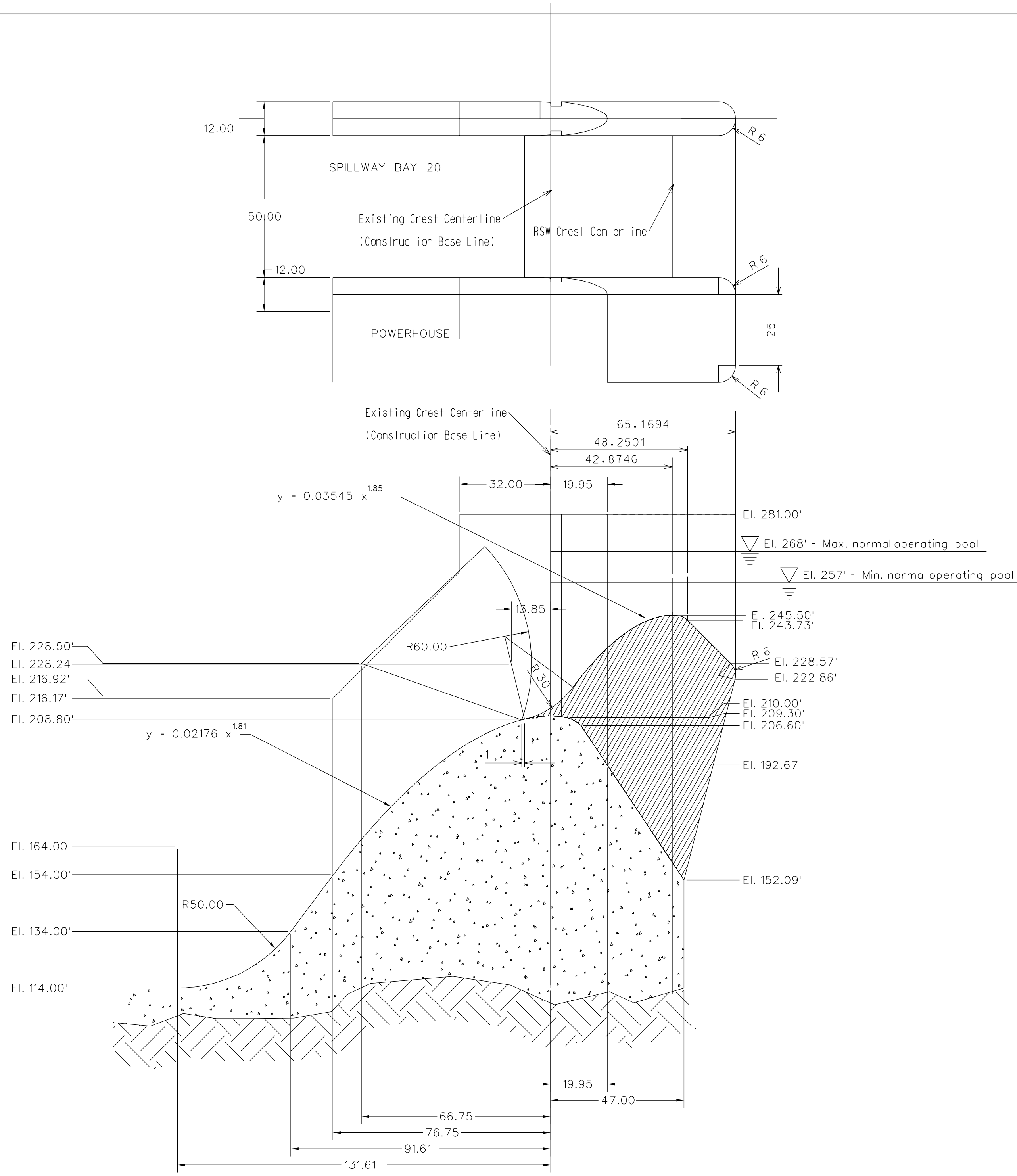


Figure 3-7
Optimum RSW Alternative B
Short Sloping Face RSW
with Piers 65 Upstream

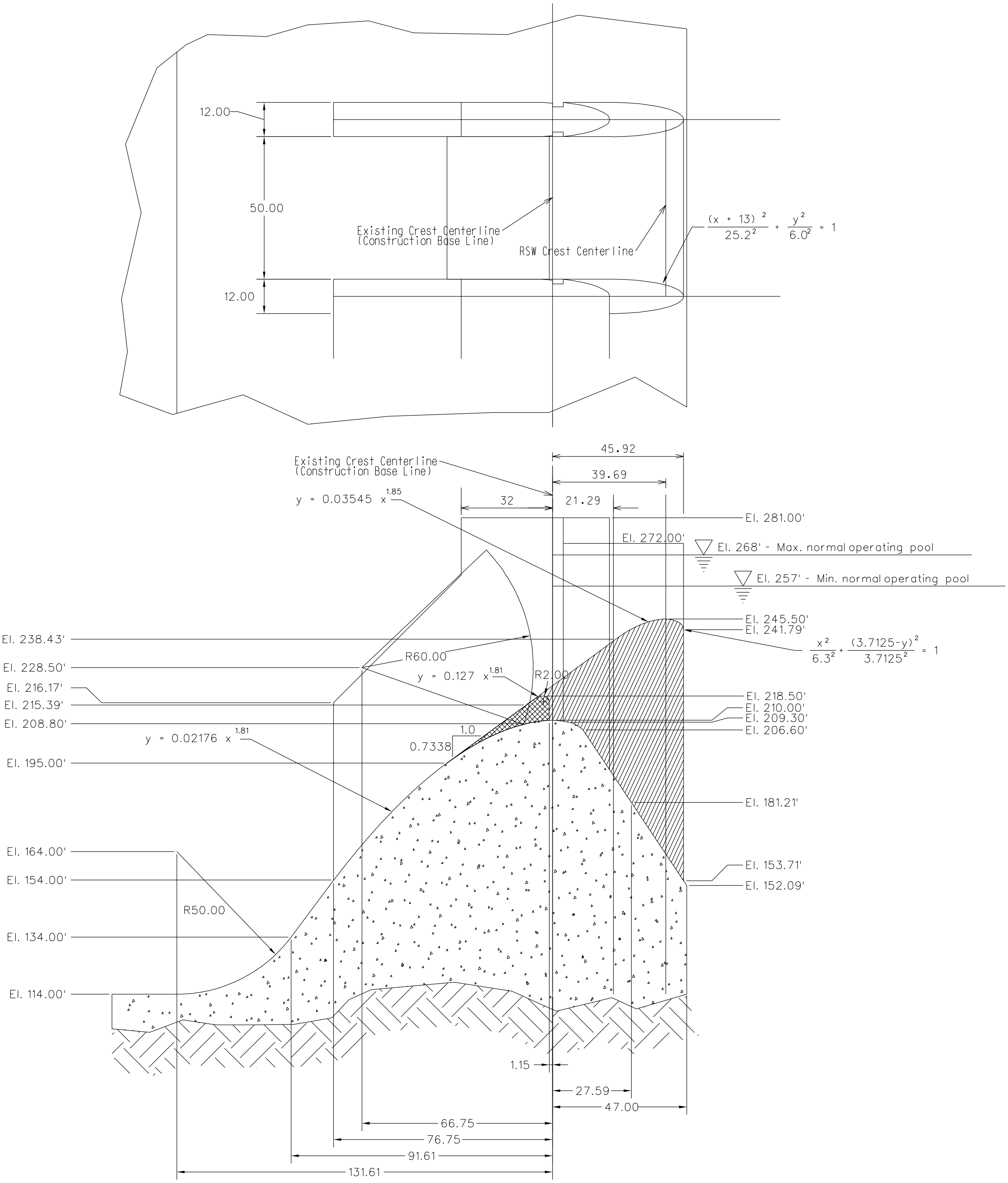


Figure 3-8
Optimum RSW Alternative C (Same as Final Proof Concept)
Vertical Face RSW with Piers 46' Upstream,
with Semi-Permanent Lower Step

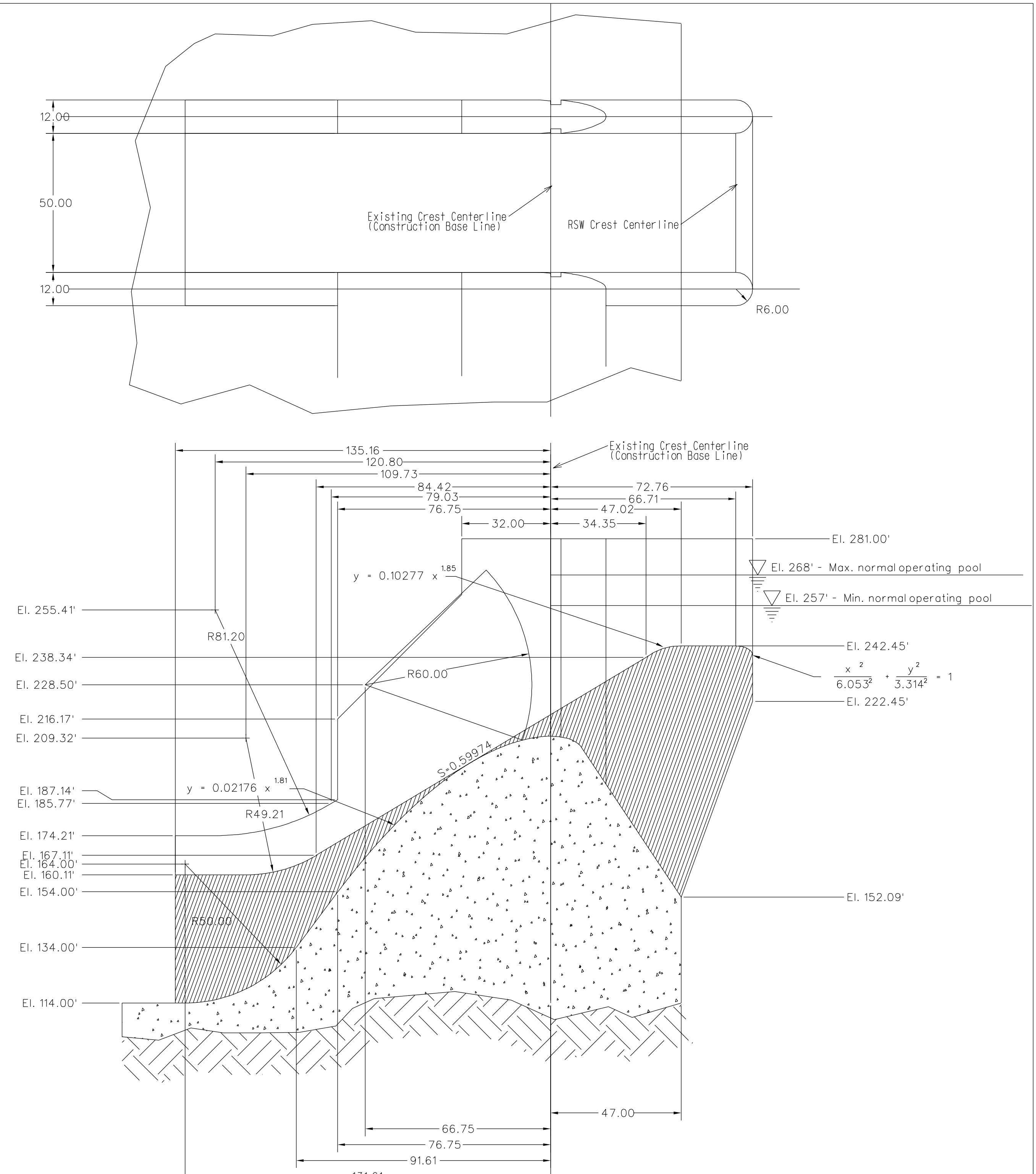


Figure 3-9
Optimum RSW Alternative D
Skeleton Bay Geometry with Piers 73' Upstream,
with Semi-Permanent Downstream Chute

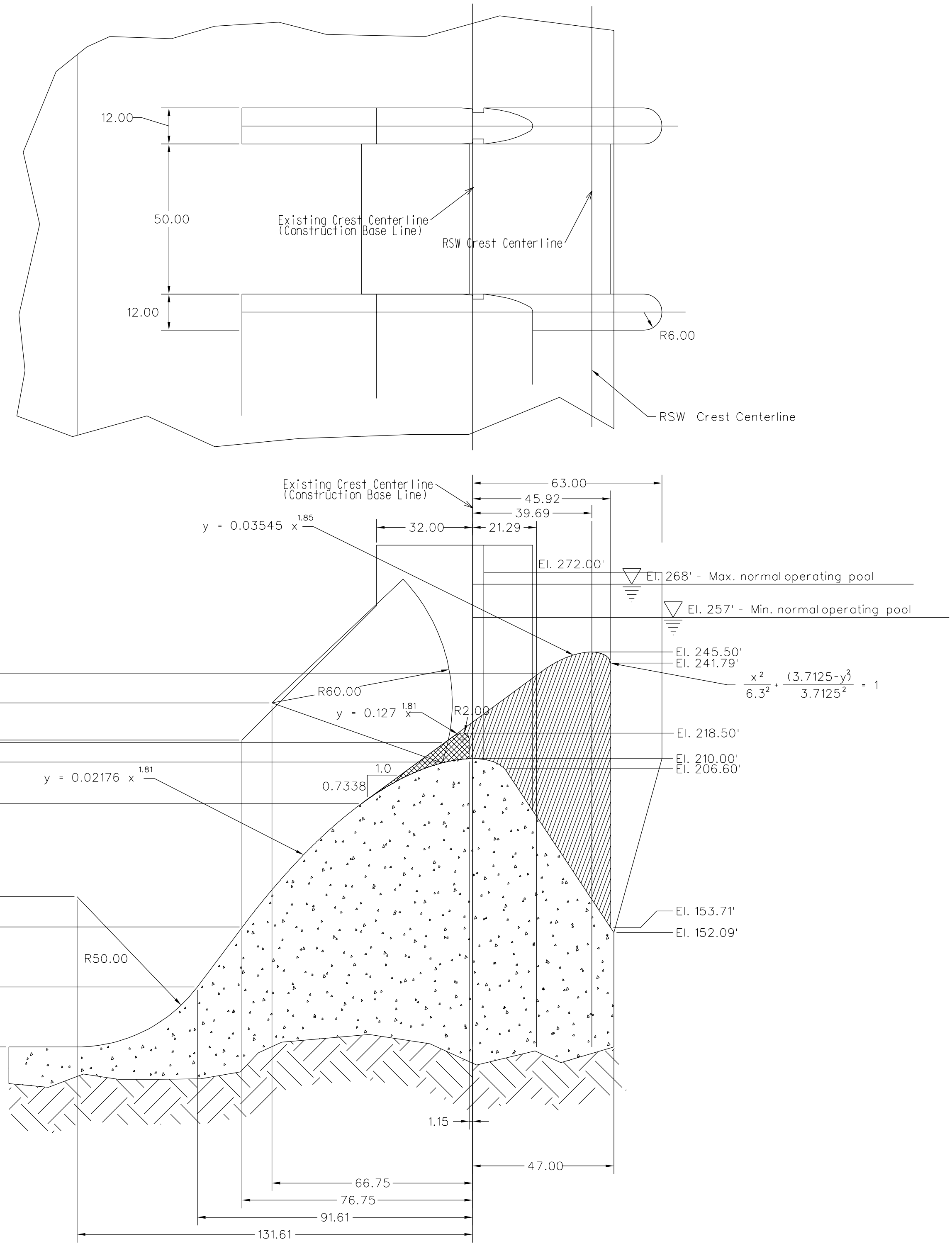


Figure 3-10
Optimum RSW Alternative E
Vertical Face RSW with Piers 63' Upstream,
with Semi-Permanent Lower Step and Extended Piers

but the capture velocity is reached a significant distance upstream of the RSW crest. Subsequent testing of POC Alternative 4 in the physical model illustrated that the step design did not improve hydraulic characteristics on the spillway.

Alternative B is similar to Alternative A, except that the entrance section is truncated and the transition step at the interface with the existing spillway ogee chute is not included. The piers and approach ramp extend 65 ft upstream of the existing crest axis. The upstream approach velocity exceeds capture velocity under maximum operating pool elevation of 268 ft msl (NGVD) prior to entering the entrance section. The RSW ogee crest geometry is identical to Alternative A, except for the toe. The downstream toe of the RSW will terminate upstream of the true tangent intersection with the existing spillway at a thickness of about 1 inch. The void area in the interface will be filled with a high strength material to provide a smooth transition between the RSW crest and the existing spillway crest. The computed cavitation index for this location ranges from about 0.7 to 0.9, so cavitation is not expected to occur provided that a smooth transition is accomplished.

Alternative C is similar to the POC RSW Alternative 5 and the final POC RSW configuration, with a two-section RSW crest. The difference is noted by the inclusion of only the small tailpiece and the entrance support section is angled back toward the spillway. For detailed discussion of this alternative, refer to Section 3 of the main report.

Alternative D is similar to the previously developed Skeleton Bay SBS. In this configuration, the Skeleton Bay SBS geometry is essentially superimposed upon the existing spillway crest, with the true tangent intersection between the existing spillway and the proposed RSW located about 25 feet downstream of the existing spillway radial gate seat. The crest elevation of the RSW would be 242.45, to correspond directly with the Skeleton Bay SBS geometry. Crest breadth will be about 20 feet, with an upstream approach ellipse, simple radius piers, and a downstream exit ellipse. The downstream exit ellipse will extend downstream to a true tangent to a constant slope chute. This constant slope chute will extend under the existing spillway radial gate and through the true tangent intersection with the existing spillway face to an extended deflector located well downstream of the existing spillway axis. The portion of this RSW shape downstream of the true tangent intersection with the existing spillway face will be permanent, and constructed of concrete. Each side of the proposed downstream chute will be framed by sidewalls identical in height to those illustrated in the Skeleton Bay SBS design. All portions of the RSW upstream of the true tangent intersection with the existing spillway will be removable, and will be constructed in two sections. The upper RSW crest section will extend upstream from the spillway radial gate seat, and the tailpiece will extend downstream of the existing stoplog slots to the true tangent intersection with the existing spillway, similar to Alternative C. Also as described for Alternative C, the connection between the downstream end of the tailpiece and the existing spillway will be filled with a smooth transition piece once the tailpiece is in place. The interface area between the upper crest section and the tailpiece will be a flush-mounted steel plate, and will be designed to minimize potential for cavitation.

Alternative E is identical to Alternative C, except the upstream piers are extended another 17 feet upstream into the forebay. The pier nose will be a simple radius instead of the same shape as the existing piers. Refer to Section 3 of the main report for more detailed discussion of crest design details for this alternative. The extended piers were expected to reduce the effects of pier drawdown noted at the crest of the other alternatives. Subsequent testing in the physical model illustrated that piers extended upstream 65 ft from the existing spillway crest did decrease the drawdown. However, even with decreased drawdown, hydraulic characteristics on the spillway face were not improved. Therefore, the extended pier length provided by this alternative is not considered to be beneficial.

8. Hydraulic Model Studies

Various RSW configurations have been tested in physical hydraulic models located at Northwest Hydraulic Consultants' Vancouver, British Columbia laboratory and at the COE's ERDC in Vicksburg, Mississippi. Both models were used to develop hydraulic data and to characterize performance of selected RSW configurations, and to help the District and the regional fisheries resource agencies select the most successful design.

8.1 Sectional Model Studies

Sectional model studies were accomplished at Northwest Hydraulic Consultants' Vancouver, B. C. laboratory facilities. The John Day Dam Spillway Bay 20, half of bay 19, and a portion of the Skeleton Bay 20 were constructed in a 1:25 scale physical hydraulic model. Selected RSW configurations were constructed to insert into the model for evaluation purposes. In addition to the final design, POC Alternatives 2, 4 and a simplified version of Alternative 5 as well as a simplified version of Optimum Alternative B were evaluated in the model facility. Standing waves were initiated by the drawdown around the pier nose with RSW POC Alternatives 2, 4 and the simplified Optimum Alternative B design. These waves were then amplified by the reverse curve transition between the end of the RSW and the existing spillway face. The amplified waves generated unacceptable flow "ridging" and large roostertails that traveled down the face of the spillway and impacted on and downstream from the spillway deflector. The extended piers with simplified Optimum Alternative B resulted in considerably less drawdown around the pier nose, but did not improve hydraulic conditions downstream from the transition bucket between the RSW and the existing spillway. The abrupt step feature at the downstream end of the RSW with POC Alternative 4 also had little significant improvement on hydraulic conditions downstream of the RSW. The only design concept providing acceptable hydraulic conditions downstream from the RSW was POC Alternative 5, which eliminated the reverse curve transition between the RSW and the existing spillway face. The POC Alternative 5, with some revisions to the Tailpiece configuration, became the final design. Discussion of the initial RSW configuration testing and observation is provided in the Physical Model Alternatives Report in Appendix E. The final draft Physical Model Study Report will be furnished following completion of the DDR.

8.2 General Model Studies

The general model consists of a large portion of the forebay and reservoir for the John Day Dam project, the dam (including the powerhouse, spillway, navigation lock, and abutments), and a large portion of the tailwater channel and downstream river. POC Alternative 2, Optimum RSW Alternative B and the final design RSW were evaluated in the model facility. The approach conditions to the POC Alternative 2 RSW were quite similar to those existing with the skeleton bay SBS. The Optimum Alternative B design, which has piers extending upstream further into the forebay than does Alternative 2, illustrated no improvement over the Alternative 2 configuration, and in fact had more areas of "dead" water behind the exterior faces of the approach piers. Observations in the general model indicated that the overall "zone of influence" or the attraction flow net to the RSW decreases as overall spillway flow increases.

9. Hydraulic Data

For each RSW configuration presented in this report, specific hydraulic characteristics were computed. These include predicted water surface profiles for several pool elevations, predicted flow velocities at various locations on the RSW, and predicted discharge rating curves. Some of these predicted data are to be verified in the physical hydraulic models for those configurations selected for testing.

9.1 Water Surface Profiles

Water surface profiles over the RSW crest were computed for each RSW configuration by methods suggested in the COE EM 1110-2-1603 for standard and elliptical crest spillways. Nappe profiles were computed with a numerical water surface profile model and with the aid of the information provided in the Hydraulic Design Criteria Sheets 111-11 to 111-14/1 for standard spillways. Table F-1 shows the computed water surface profiles for the five POC RSW alternatives in tabular format. Table F-2 below shows similar data for the Optimum RSW alternatives. Figures F-11 through F-15 illustrate the graphical equivalent of these computed profiles with pool elevations 262, 265 and 268 ft for the POC alternatives. Figures F-16 through F-20 illustrate similar data for the Optimum RSW alternatives.

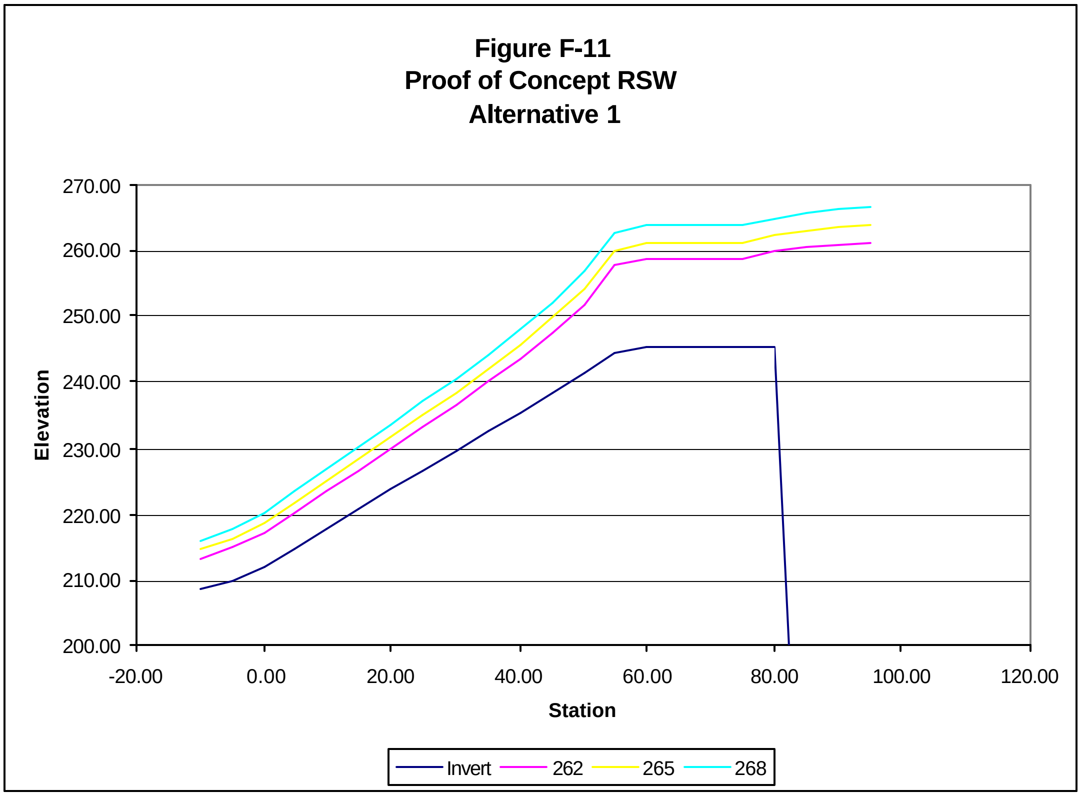


Figure F-11
Proof of Concept RSW
Alternative 1
Elevation
270.00
260.00
250.00
240.00
230.00
220.00
210.00
200.00
-20.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Station
Invert
262
265
268

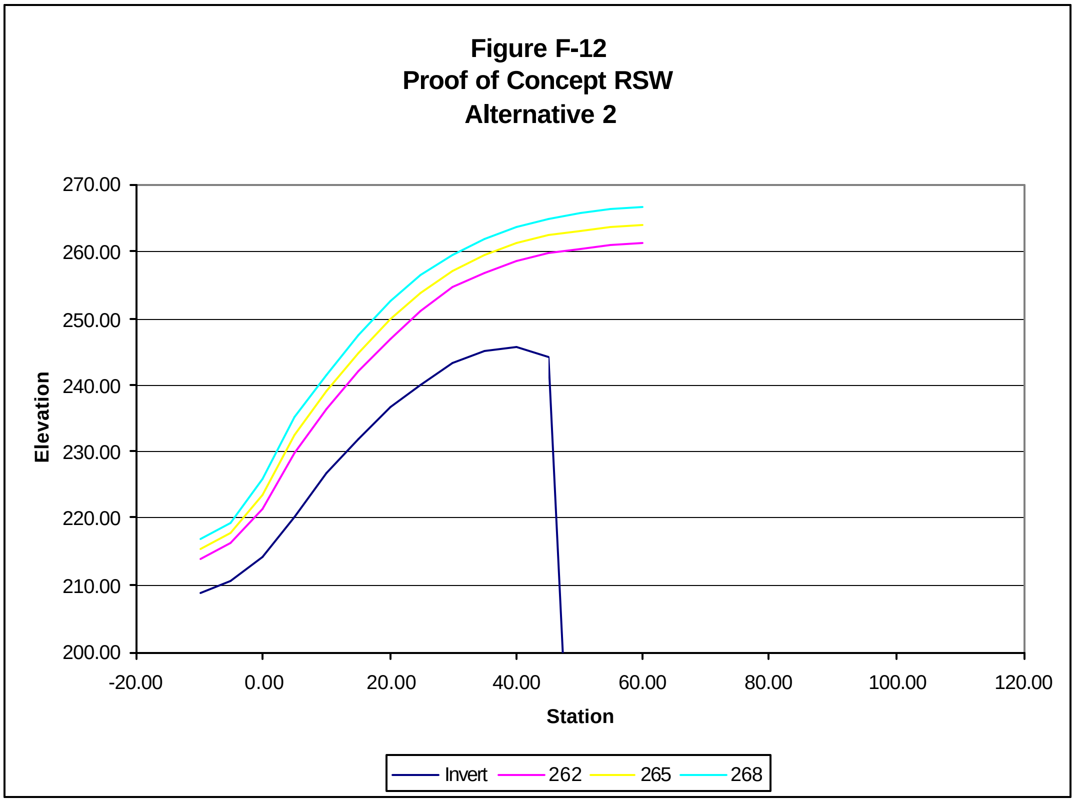


Figure F-12
Proof of Concept RSW
Alternative 2
Elevation
270.00
260.00
250.00
240.00
230.00
220.00
210.00
200.00
-20.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Station
Invert
262
265
268

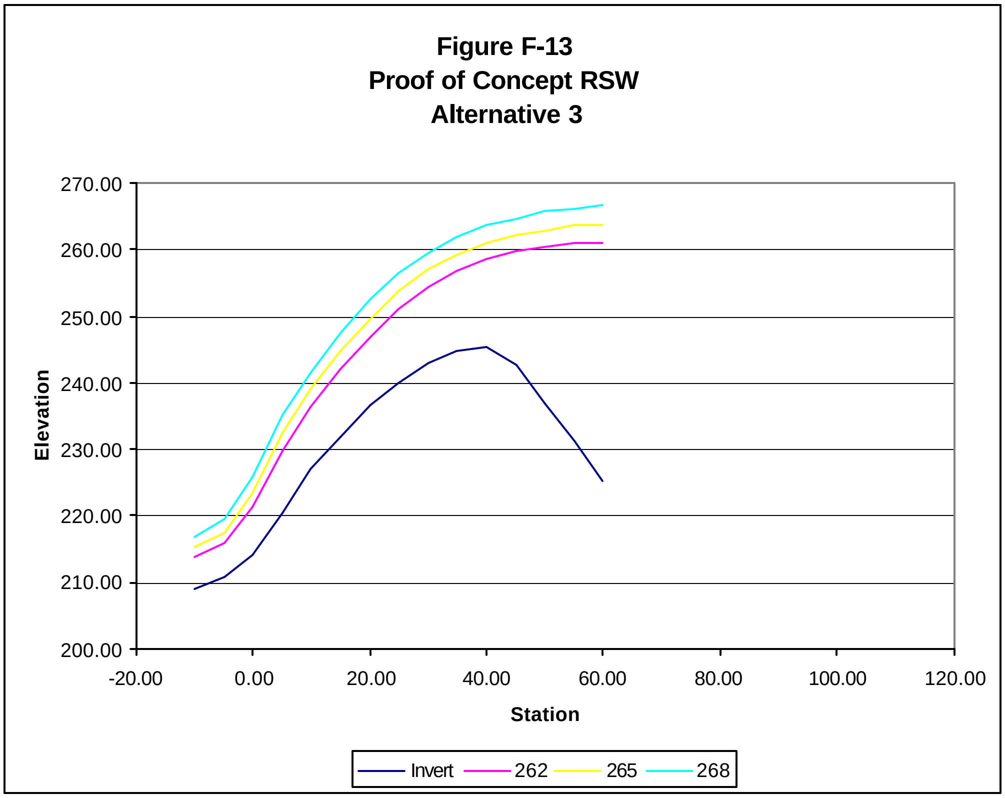


Figure F-13
Proof of Concept RSW
Alternative 3
Elevation
270.00
260.00
250.00
240.00
230.00
220.00
210.00
200.00
-20.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Station
Invert
262
265
268

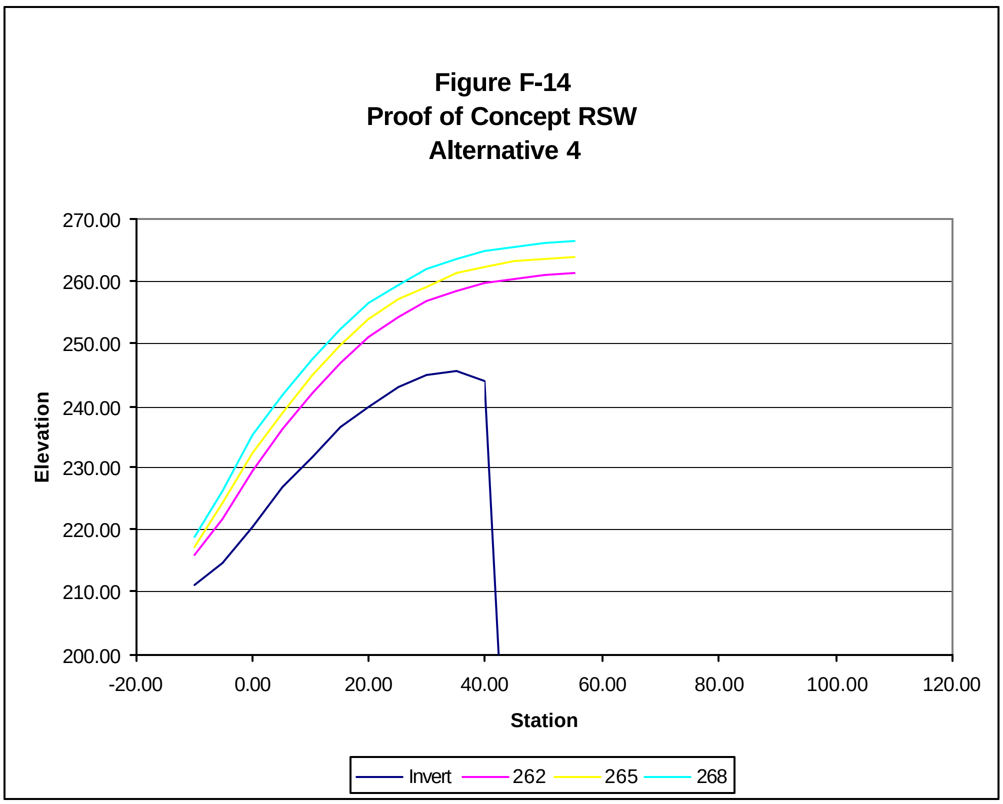


Figure F-14
Proof of Concept RSW
Alternative 4
Elevation
270.00
260.00
250.00
240.00
230.00
220.00
210.00
200.00
-20.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Station
Invert
262
265
268

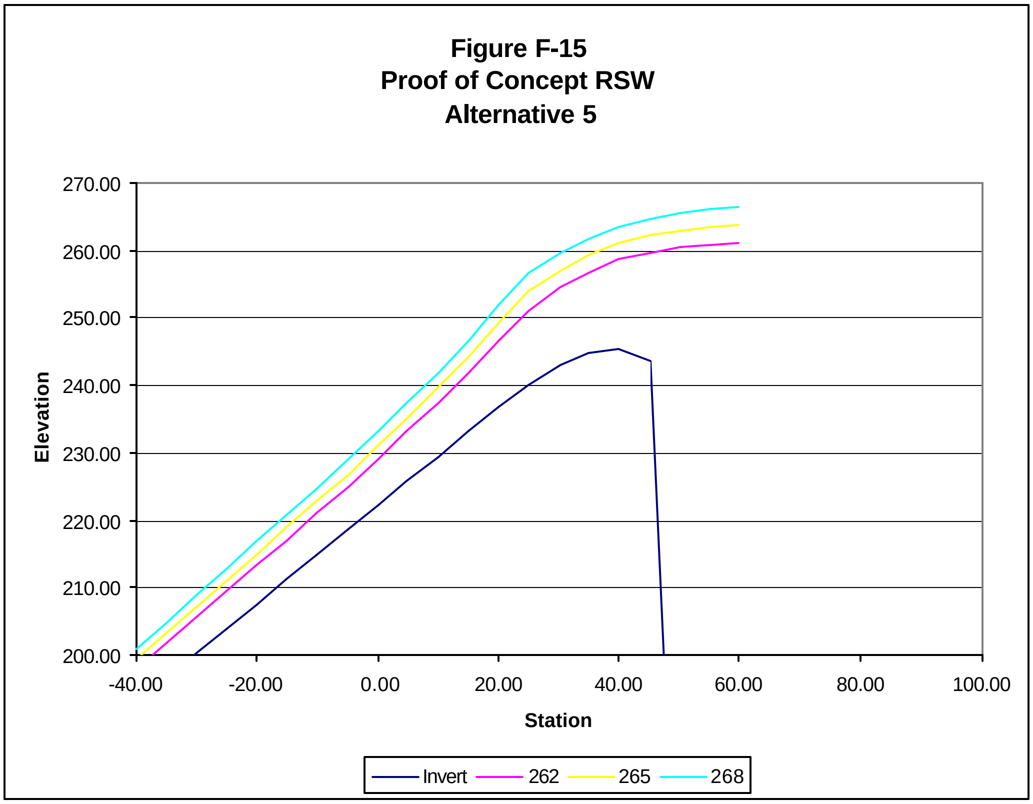


Figure F-15
Proof of Concept RSW
Alternative 5
Elevation
270.00
260.00
250.00
240.00
230.00
220.00
210.00
200.00
-40.00
-20.00
0.00
20.00
40.00
60.00
80.00
100.00
Station
Invert
262
265
268

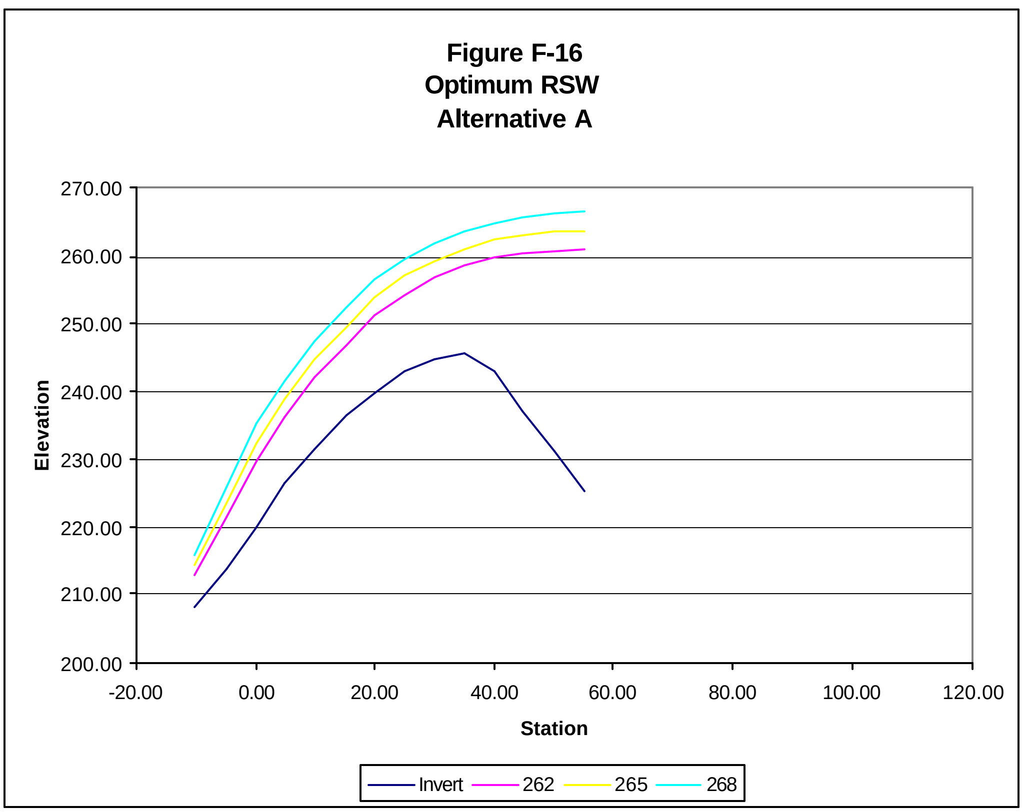


Figure F-16
Optimum RSW
Alternative A
Elevation
270.00
260.00
250.00
240.00
230.00
220.00
210.00
200.00
-20.00
0.00
20.00
40.00
60.00
80.00
100.00
120.00
Station
Invert
262
265
268

Figure F-17
Optimum RSW
Alternative B

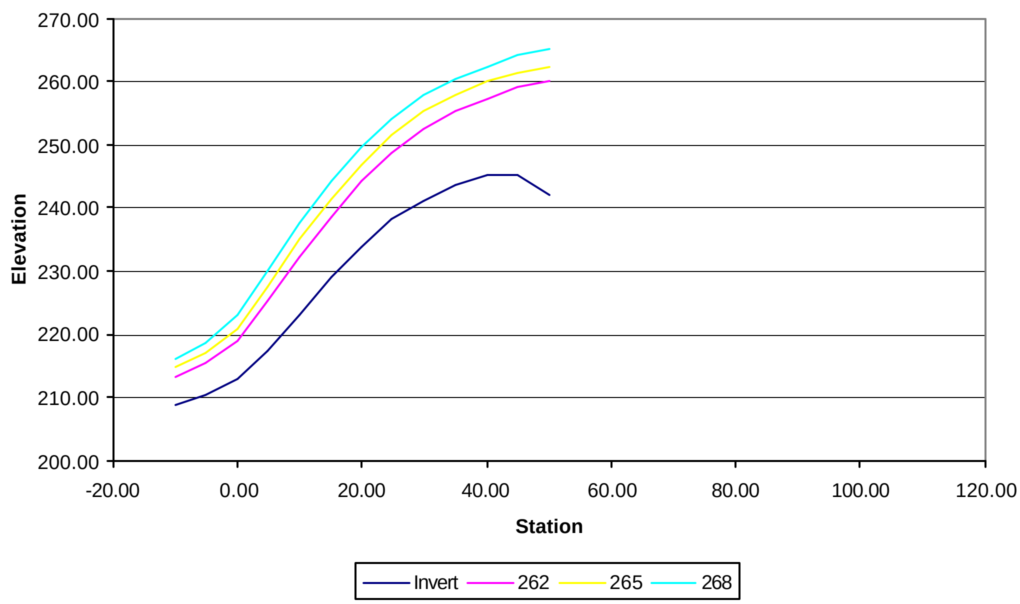
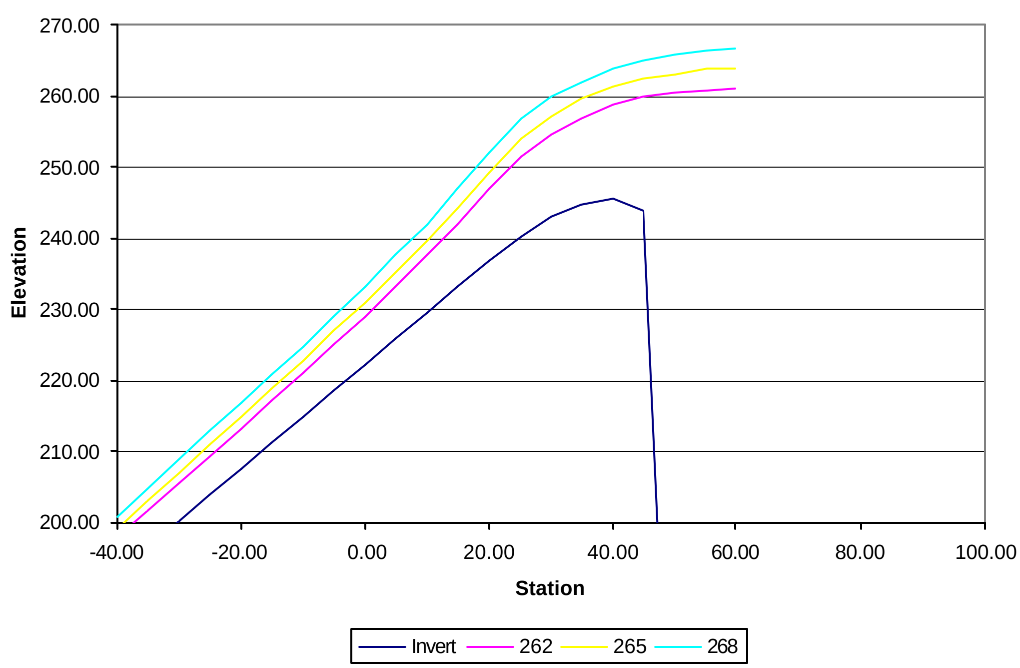


Figure F-18
Optimum RSW
Alternative C



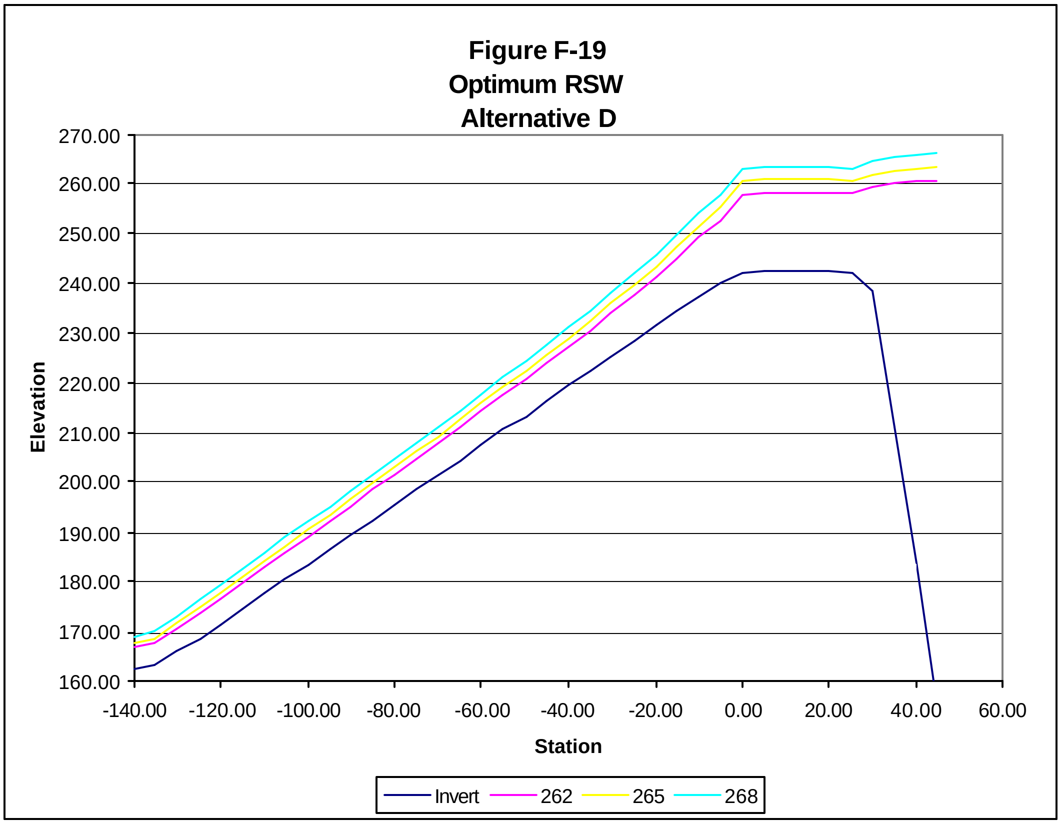


Figure F-19
Optimum RSW
Alternative D
270.00
260.00
250.00
240.00
230.00
220.00
210.00
200.00
190.00
180.00
170.00
160.00
Elevation
-140.00
-120.00
-100.00
-80.00
-60.00
-40.00
-20.00
0.00
20.00
40.00
60.00
Station
Invert
262
265
268

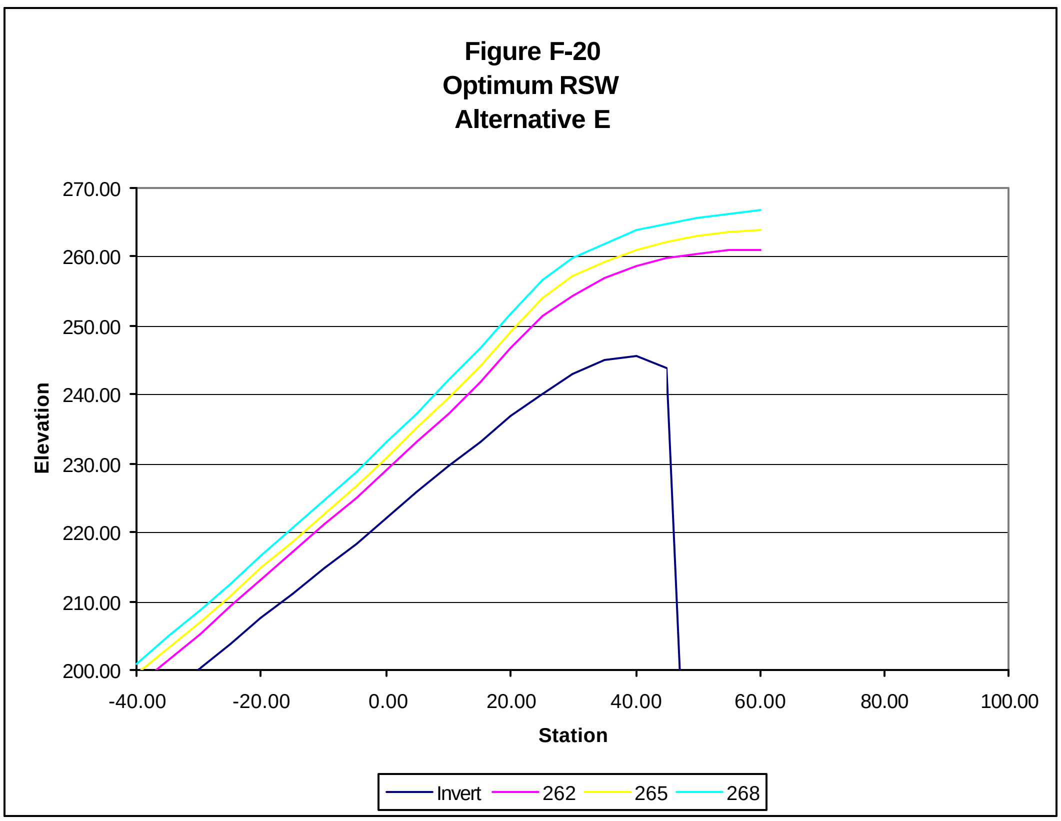


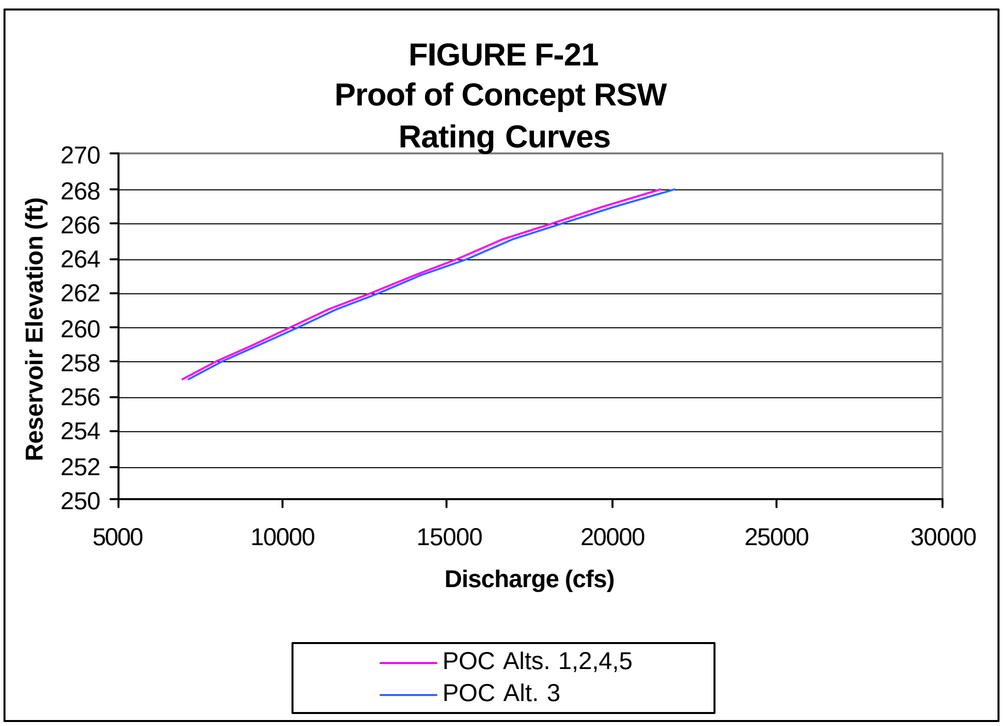
Figure F-20
Optimum RSW
Alternative E
270.00
260.00
250.00
240.00
230.00
220.00
210.00
200.00
Elevation
-40.00
-20.00
0.00
20.00
40.00
60.00
80.00
100.00
Station
Invert
262
265
268

9.2 Velocities

Flow velocities at various locations on each RSW configuration were computed with the numerical model discussed in Section 3.9.1. Table F-3 below shows the computed flow velocities for the five POC RSW alternatives in tabular format. Table F-4 below shows similar data for the Optimum RSW alternatives.

9.3 Rating Curves

Approximate discharge rating curves were developed for each of the POC and Optimum RSW alternative configurations, using guidance provided in Plates 3-3 and 3-4 of EM 1110-2-1603, and in Brater and King. These discharge-rating curves are very approximate, and should not be used for detailed analysis until verified in the physical model. Figures F-21 and F-22 below provide a graphical representation of the discharge ratings for the POC RSW alternatives and the Optimum RSW alternatives, respectively. POC Alternatives 1, 2, 4, 5 and Optimum Alternatives C & E all share a similar crest shape. The discharge coefficient for this shape was assumed to range from about 3.6 at low forebay elevations to about 4.0 at high forebay elevations. The discharge coefficient for POC Alternative 3 and Optimum Alternatives A & B is slightly less, due to the inefficiency of the approach ramp in maximizing crest discharge. The discharge coefficient for Optimum Alternative D approaches the value for a broadcrested weir at very low forebay elevations, but increases as forebay elevation, and the ratio of crest submergence to crest breadth increases. The assumed discharge coefficient ranges from about 3.6 at forebay elevation 257 to about 4.0 at forebay elevation 268.



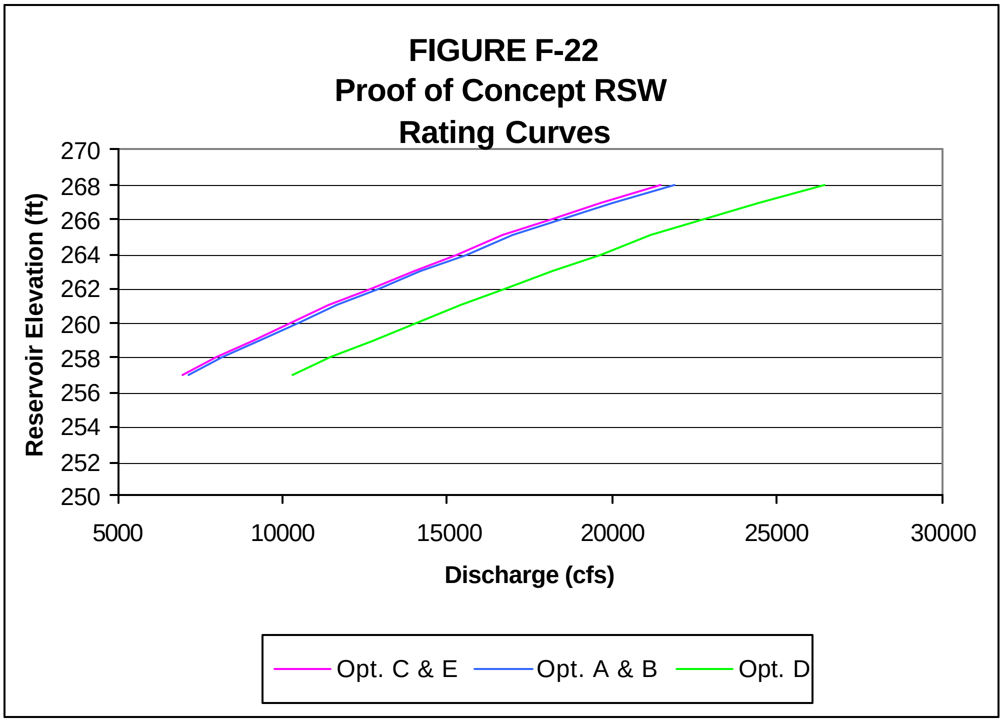


FIGURE F-22
Proof of Concept RSW
Rating Curves
Reservoir Elevation (ft)
270
268
266
264
262
260
258
256
254
252
250
5000
10000
15000
20000
25000
30000
Discharge (cfs)
Opt. C & E
Opt. A & B
Opt. D

APPENDIX G

MCASES Cost Summary Tables

**** TOTAL CONTRACT COST SUMMARY ****

JOHN DAY DAM SURFACE BYPASS REMOVABLE SPILLWAY WEIR

THIS ESTIMATE IS BASED ON THE SCOPE CONTAINED IN JOHN DAY DAM SURFACE BYPASS REMOVABLE SPILLWAY WEIR P&S - 90% - MAY, 2001.

PROJECT: JOHN DAY DAM SURFACE BYPASS RSW - SPILLWAY BAY 20 | DISTRICT: PORTLAND

LOCATION: JOHN DAY DAM SPILLWAY BAY 20 | P.O.C.: AL O'Connor Cost Estimating Branch

	CURRENT MCACES ESTIMATE PREPARED: MAY 21, 2001 EFFECTIVE PRICING LEVEL: 1 MAY 01					AUTHORIZ./BUDGET YEAR: 2003 EFFECT. PRICING LEVEL: 1 NOV 00				*** FULLY FUNDED ESTIMATE ***				
ACCOUNT NUMBER	FEATURE DESCRIPTION	COST ($K)	CNTG ($K)	CNTG (%)	TOTAL ($K)	OMB (%)	COST ($K)	CNTG ($K)	TOTAL ($K)	FEATURE MID PT	OMB (%)	COST ($K)	CNTG ($K)	FULL ($K)
6O10	FISH FACILITIES AT DAMS													
60144	FISH SURFACE BYPASS SPILLWAY	8,344	1,252	15%	9,596	0.0%	8,344	1,252	9,596	Jan-2003	5.7%	8,815	1,323	10,138
	TOTAL CONSTRUCTION COSTS ===>	8,344	1,252	15%	9,596	0.0%	8,344	1,252	9,596	Jan-2003	5.7%	8,815	1,323	10,138
01------	LANDS AND DAMAGES	0	0	0%	0	0.0%	0	0.0	0.0	Jan-2003	0.0%	0.0	0.0	0.0
30------	PLANNING, ENGINEERING & DESIGN	0	0	0%	0	0.0%	0	0	0	Jan-2003	0.0%	0	0	0
31------	CONSTRUCTION MANAGEMENT	0	0	0%	0	0.0%	0	0	0	Jan-2003	6.0%	608	0	608
	TOTAL PROJECT COSTS ========>	8,344	1,252	15%	9,596		8,344	1,252	9,596			9,423	1,323	10,746

TOTAL FEDERAL COSTS ==========>	10,746
TOTAL NON-FEDERAL COSTS =====>	0
THE MAXIMUM PROJECT COST IS ==>	$10,746

Note: 01, 30 and 31 account totals and mid-point dates to be provided by the Corps and project managers.

Table G -1

	QUANTY	UOM	CONTRACT	CONTINGN	ESCALATN	OWN FURN	SIOH	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES	1.00	EA	8,343,511	1,251,527	542,621	0	608,260	10,745,918	10745918
TOTAL John Day-Removable Spillway Weir	1.00	EA	8,343,511	1,251,527	542,621	0	608,260	10,745,918	10745918

	QUANTY	UOM	CONTRACT	CONTINGN	ESCALATN	OWN FURN	SIOH	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES									
6-01 Mobilization & Demob	1.00	EA	257,711	38,657	16,760	0	18,788	331,916	331916
6-02 Main Structure	1.00	JOB	6,793,760	1,019,064	441,833	0	495,279	8,749,936	8749936
6-03 Tailpiece	1.00	JOB	718,412	107,762	46,722	0	52,374	925,270	925270
6-05 RSW Main Structure Attachment	1.00	JOB	527,044	79,057	34,276	0	38,423	678,799	678799
6-20 Optional Item - Dive Crew	5.00	DAY	35,501	5,325	2,309	0	2,588	45,723	9144.66
6-22 Optional Item - Welder,Machinist	200.00	HR	11,082	1,662	721	0	808	14,273	71.37
TOTAL FISH & WILDLIFE FACILITIES	1.00	EA	8,343,511	1,251,527	542,621	0	608,260	10,745,918	10745918
TOTAL John Day-Removable Spillway Weir	1.00	EA	8,343,511	1,251,527	542,621	0	608,260	10,745,918	10745918

	QUANTY	UOM	CONTRACT	CONTINGN	ESCALATN	OWN FURN	SIOH	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES									
6-01 Mobilization & Demob	1.00	EA	257,711	38,657	16,760	0	18,788	331,916	331916
TOTAL Mobilization & Demob	1.00	EA	257,711	38,657	16,760	0	18,788	331,916	331916
6-02 Main Structure									
6-0202 Fab RSW	625.30	TON	6,163,328	924,499	400,832	0	449,320	7,937,979	12695
6-0203 Dive Crew for surveying	5.00	DAY	48,258	7,239	3,138	0	3,518	62,153	12431
6-0207 Transport RSW	1.00	JOB	33,134	4,970	2,155	0	2,416	42,675	42675
6-0208 Install RSW	1.00	JOB	323,459	48,519	21,036	0	23,581	416,595	416595
6-0209 Grating and ladders	1.00	JOB	8,849	1,327	576	0	645	11,397	11397
6-0211 Charter Tugboat for Installation	1.00	JOB	195,330	29,300	12,703	0	14,240	251,573	251573
6-0212 Mooring Points at Dam	1.00	JOB	21,401	3,210	1,392	0	1,560	27,564	27564
TOTAL Main Structure	1.00	JOB	6,793,760	1,019,064	441,833	0	495,279	8,749,936	8749936
6-03 Tailpiece									
6-0302 Fabricate Tailpiece	50.40	TON	496,766	74,515	32,307	0	36,215	639,803	12695
6-0303 Barge tailpiece to Dam	1.00	JOB	9,467	1,420	616	0	690	12,193	12193
6-0304 Install stoplogs	1.00	JOB	1,302	195	85	0	95	1,677	1676.81
6-0306 Anchor Plates	44.00	EA	125,188	18,778	8,142	0	9,126	161,234	3664.41
6-0308 Install Tailpiece	1.00	JOB	85,690	12,853	5,573	0	6,247	110,363	110363
TOTAL Tailpiece	1.00	JOB	718,412	107,762	46,722	0	52,374	925,270	925270
6-05 RSW Main Structure Attachment									
6-0502 Fab Main Structure Attachment	20.50	TON	197,563	29,635	12,849	0	14,403	254,449	12412
6-0503 Barge Main Support Structure to	1.00	JOB	9,467	1,420	616	0	690	12,193	12193
6-0506 Anchor Bolts/Install Steel	1.00	JOB	117,452	17,618	7,638	0	8,562	151,271	151271
6-0508 Crane crew to lower frame	23.00	DAY	69,066	10,360	4,492	0	5,035	88,953	3867.51
6-0509 Dive crew to attach frame	13.00	DAY	92,925	13,939	6,043	0	6,774	119,681	9206.27
6-0510 Seal Assembly Detail	1.00	JOB	40,570	6,086	2,638	0	2,958	52,252	52252
TOTAL RSW Main Structure Attachment	1.00	JOB	527,044	79,057	34,276	0	38,423	678,799	678799
6-20 Optional Item - Dive Crew	5.00	DAY	35,501	5,325	2,309	0	2,588	45,723	9144.66
6-22 Optional Item - Welder,Machinist	200.00	HR	11,082	1,662	721	0	808	14,273	71.37
TOTAL FISH & WILDLIFE FACILITIES	1.00	EA	8,343,511	1,251,527	542,621	0	608,260	10,745,918	10745918
TOTAL John Day-Removable Spillway Weir	1.00	EA	8,343,511	1,251,527	542,621	0	608,260	10,745,918	10745918

	QUANTY UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES	1.00 EA	7,050,621	253,586	292,168	676,077	71,059	8,343,511	8343511
TOTAL John Day-Removable Spillway Weir	1.00 EA	7,050,621	253,586	292,168	676,077	71,059	8,343,511	8343511
Contingency - 15%							1,251,527	
SUBTOTAL							9,595,038	
Escalation -							542,621	
SUBTOTAL							10,137,659	
SIOH - 6%							608,260	
TOTAL INCL OWNER COSTS							10,745,918	

	QUANTY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES									
6-01 Mobilization & Demob	1.00	EA	217,777	7,833	9,024	20,882	2,195	257,711	257711
6-02 Main Structure	1.00	JOB	5,741,015	206,484	237,900	550,501	57,860	6,793,760	6793760
6-03 Tailpiece	1.00	JOB	607,089	21,835	25,157	58,213	6,118	718,412	718412
6-05 RSW Main Structure Attachment	1.00	JOB	445,374	16,019	18,456	42,707	4,489	527,044	527044
6-20 Optional Item - Dive Crew	5.00	DAY	30,000	1,079	1,243	2,877	302	35,501	7100.24
6-22 Optional Item - Welder,Machinist	200.00	HR	9,365	337	388	898	94	11,082	55.41
TOTAL FISH & WILDLIFE FACILITIES	1.00	EA	7,050,621	253,586	292,168	676,077	71,059	8,343,511	8343511
TOTAL John Day-Removable Spillway Weir	1.00	EA	7,050,621	253,586	292,168	676,077	71,059	8,343,511	8343511
Contingency - 15%								1,251,527	
SUBTOTAL								9,595,038	
Escalation -								542,621	
SUBTOTAL								10,137,659	
SIOH - 6%								608,260	
TOTAL INCL OWNER COSTS								10,745,918	

	QUANTY	UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES									
6-01 Mobilization & Demob	1.00	EA	217,777	7,833	9,024	20,882	2,195	257,711	257711
TOTAL Mobilization & Demob	1.00	EA	217,777	7,833	9,024	20,882	2,195	257,711	257711
6-02 Main Structure									
6-0202 Fab RSW	625.30	TON	5,208,273	187,323	215,824	499,416	52,491	6,163,328	9856.59
6-0203 Dive Crew for surveying	5.00	DAY	40,780	1,467	1,690	3,910	411	48,258	9651.59
6-0207 Transport RSW	1.00	JOB	28,000	1,007	1,160	2,685	282	33,134	33134
6-0208 Install RSW	1.00	JOB	273,337	9,831	11,327	26,210	2,755	323,459	323459
6-0209 Grating and ladders	1.00	JOB	7,478	269	310	717	75	8,849	8849.22
6-0211 Charter Tugboat for Installation	1.00	JOB	165,062	5,937	6,840	15,828	1,664	195,330	195330
6-0212 Mooring Points at Dam	1.00	JOB	18,085	650	749	1,734	182	21,401	21401
TOTAL Main Structure	1.00	JOB	5,741,015	206,484	237,900	550,501	57,860	6,793,760	6793760
6-03 Tailpiece									
6-0302 Fabricate Tailpiece	50.40	TON	419,788	15,098	17,395	40,253	4,231	496,766	9856.47
6-0303 Barge tailpiece to Dam	1.00	JOB	8,000	288	332	767	81	9,467	9466.98
6-0304 Install stoplogs	1.00	JOB	1,100	40	46	105	11	1,302	1301.94
6-0306 Anchor Plates	44.00	EA	105,789	3,805	4,384	10,144	1,066	125,188	2845.17
6-0308 Install Tailpiece	1.00	JOB	72,411	2,604	3,001	6,943	730	85,690	85690
TOTAL Tailpiece	1.00	JOB	607,089	21,835	25,157	58,213	6,118	718,412	718412
6-05 RSW Main Structure Attachment									
6-0502 Fab Main Structure Attachment	20.50	TON	166,950	6,005	6,918	16,009	1,683	197,563	9637.24
6-0503 Barge Main Support Structure to	1.00	JOB	8,000	288	332	767	81	9,467	9466.98
6-0506 Anchor Bolts/Install Steel	1.00	JOB	99,252	3,570	4,113	9,517	1,000	117,452	117452
6-0508 Crane crew to lower frame	23.00	DAY	58,364	2,099	2,419	5,596	588	69,066	3002.87
6-0509 Dive crew to attach frame	13.00	DAY	78,526	2,824	3,254	7,530	791	92,925	7148.07
6-0510 Seal Assembly Detail	1.00	JOB	34,284	1,233	1,421	3,287	346	40,570	40570
TOTAL RSW Main Structure Attachment	1.00	JOB	445,374	16,019	18,456	42,707	4,489	527,044	527044
6-20 Optional Item - Dive Crew	5.00	DAY	30,000	1,079	1,243	2,877	302	35,501	7100.24
6-22 Optional Item - Welder,Machinist	200.00	HR	9,365	337	388	898	94	11,082	55.41
TOTAL FISH & WILDLIFE FACILITIES	1.00	EA	7,050,621	253,586	292,168	676,077	71,059	8,343,511	8343511
TOTAL John Day-Removable Spillway Weir	1.00	EA	7,050,621	253,586	292,168	676,077	71,059	8,343,511	8343511
Contingency - 15%								1,251,527	

	QUANTY UOM	DIRECT	FIELD OH	HOME OFC	PROFIT	BOND	TOTAL COST	UNIT
SUBTOTAL							9,595,038	
Escalation -							542,621	
SUBTOTAL							10,137,659	
SIOH - 6%							608,260	
TOTAL INCL OWNER COSTS							10,745,918	

	QUANTY	UOM	MANHRS	LABOR	EQUIPMNT	MATERIAL	SUBCONTR	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES	1.00	EA	41,694	2,295,479	223,489	4,233,793	297,860	7,050,621	7050621
TOTAL John Day-Removable Spillway Weir	1.00	EA	41,694	2,295,479	223,489	4,233,793	297,860	7,050,621	7050621
Field Overhead - Calculated								253,586	
SUBTOTAL								7,304,207	
Prime's Home Office Expense								292,168	
SUBTOTAL								7,596,375	
Prime Contractor's Profit								676,077	
SUBTOTAL								8,272,452	
Prime Contractor's Bond								71,059	
TOTAL INCL INDIRECTS								8,343,511	
Contingency - 15%								1,251,527	
SUBTOTAL								9,595,038	
Escalation -								542,621	
SUBTOTAL								10,137,659	
SIOH - 6%								608,260	
TOTAL INCL OWNER COSTS								10,745,918	

	QUANTY	UOM	MANHRS	LABOR	EQUIPMNT	MATERIAL	SUBCONTR	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES									
6-01 Mobilization & Demob	1.00	EA	62	72,522	122,000	17,895	5,360	217,777	217777
6-02 Main Structure	1.00	JOB	34,805	1,856,481	17,370	3,789,165	78,000	5,741,015	5741015
6-03 Tailpiece	1.00	JOB	4,614	238,452	46,946	304,191	17,500	607,089	607089
6-05 RSW Main Structure Attachment	1.00	JOB	2,012	118,659	37,173	122,543	167,000	445,374	445374
6-20 Optional Item - Dive Crew	5.00	DAY	0	0	0	0	30,000	30,000	6000.00
6-22 Optional Item - Welder,Machinist	200.00	HR	200	9,365	0	0	0	9,365	46.83
TOTAL FISH & WILDLIFE FACILITIES	1.00	EA	41,694	2,295,479	223,489	4,233,793	297,860	7,050,621	7050621
TOTAL John Day-Removable Spillway Weir	1.00	EA	41,694	2,295,479	223,489	4,233,793	297,860	7,050,621	7050621
Field Overhead - Calculated								253,586	
SUBTOTAL								7,304,207	
Prime's Home Office Expense								292,168	
SUBTOTAL								7,596,375	
Prime Contractor's Profit								676,077	
SUBTOTAL								8,272,452	
Prime Contractor's Bond								71,059	
TOTAL INCL INDIRECTS								8,343,511	
Contingency - 15%								1,251,527	
SUBTOTAL								9,595,038	
Escalation -								542,621	
SUBTOTAL								10,137,659	
SIOH - 6%								608,260	
TOTAL INCL OWNER COSTS								10,745,918	

	QUANTY	UOM	MANHRS	LABOR	EQUIPMNT	MATERIAL	SUBCONTR	TOTAL COST	UNIT
6 FISH & WILDLIFE FACILITIES									
6-01 Mobilization & Demob	1.00	EA	62	72,522	122,000	17,895	5,360	217,777	217777
TOTAL Mobilization & Demob	1.00	EA	62	72,522	122,000	17,895	5,360	217,777	217777
6-02 Main Structure									
6-0202 Fab RSW	625.30	TON	34,064	1,798,015	800	3,409,458	0	5,208,273	8329.24
6-0203 Dive Crew for surveying	5.00	DAY	0	0	0	10,780	30,000	40,780	8156.00
6-0207 Transport RSW	1.00	JOB	0	14,000	14,000	0	0	28,000	28000
6-0208 Install RSW	1.00	JOB	697	32,353	0	192,984	48,000	273,337	273337
6-0209 Grating and ladders	1.00	JOB	44	2,113	70	5,295	0	7,478	7477.97
6-0211 Charter Tugboat for Installation	1.00	JOB	0	2,500	0	162,562	0	165,062	165062
6-0212 Mooring Points at Dam	1.00	JOB	0	7,500	2,500	8,085	0	18,085	18085
TOTAL Main Structure	1.00	JOB	34,805	1,856,481	17,370	3,789,165	78,000	5,741,015	5741015
6-03 Tailpiece									
6-0302 Fabricate Tailpiece	50.40	TON	2,739	141,440	0	262,348	16,000	419,788	8329.13
6-0303 Barge tailpiece to Dam	1.00	JOB	0	4,000	4,000	0	0	8,000	8000.00
6-0304 Install stoplogs	1.00	JOB	24	1,100	0	0	0	1,100	1100.19
6-0306 Anchor Plates	44.00	EA	953	48,979	14,259	41,051	1,500	105,789	2404.29
6-0308 Install Tailpiece	1.00	JOB	898	42,933	28,687	792	0	72,411	72411
TOTAL Tailpiece	1.00	JOB	4,614	238,452	46,946	304,191	17,500	607,089	607089
6-05 RSW Main Structure Attachment									
6-0502 Fab Main Structure Attachment	20.50	TON	1,126	59,163	0	107,787	0	166,950	8143.88
6-0503 Barge Main Support Structure to	1.00	JOB	0	4,000	4,000	0	0	8,000	8000.00
6-0506 Anchor Bolts/Install Steel	1.00	JOB	42	1,784	0	10,968	86,500	99,252	99252
6-0508 Crane crew to lower frame	23.00	DAY	552	26,891	31,473	0	0	58,364	2537.56
6-0509 Dive crew to attach frame	13.00	DAY	9	432	0	94	78,000	78,526	6040.42
6-0510 Seal Assembly Detail	1.00	JOB	283	26,389	1,700	3,694	2,500	34,284	34284
TOTAL RSW Main Structure Attachment	1.00	JOB	2,012	118,659	37,173	122,543	167,000	445,374	445374
6-20 Optional Item - Dive Crew	5.00	DAY	0	0	0	0	30,000	30,000	6000.00
6-22 Optional Item - Welder,Machinist	200.00	HR	200	9,365	0	0	0	9,365	46.83
TOTAL FISH & WILDLIFE FACILITIES	1.00	EA	41,694	2,295,479	223,489	4,233,793	297,860	7,050,621	7050621
TOTAL John Day-Removable Spillway Weir	1.00	EA	41,694	2,295,479	223,489	4,233,793	297,860	7,050,621	7050621
Field Overhead - Calculated								253,586	

	QUANTY	UOM	MANHRS	LABOR	EQUIPMNT	MATERIAL	SUBCONTR	TOTAL COST	UNIT
SUBTOTAL								7,304,207	
Prime's Home Office Expense								292,168	
SUBTOTAL								7,596,375	
Prime Contractor's Profit								676,077	
SUBTOTAL								8,272,452	
Prime Contractor's Bond								71,059	
TOTAL INCL INDIRECTS								8,343,511	
Contingency - 15%								1,251,527	
SUBTOTAL								9,595,038	
Escalation -								542,621	
SUBTOTAL								10,137,659	
SIOH - 6%								608,260	
TOTAL INCL OWNER COSTS								10,745,918	

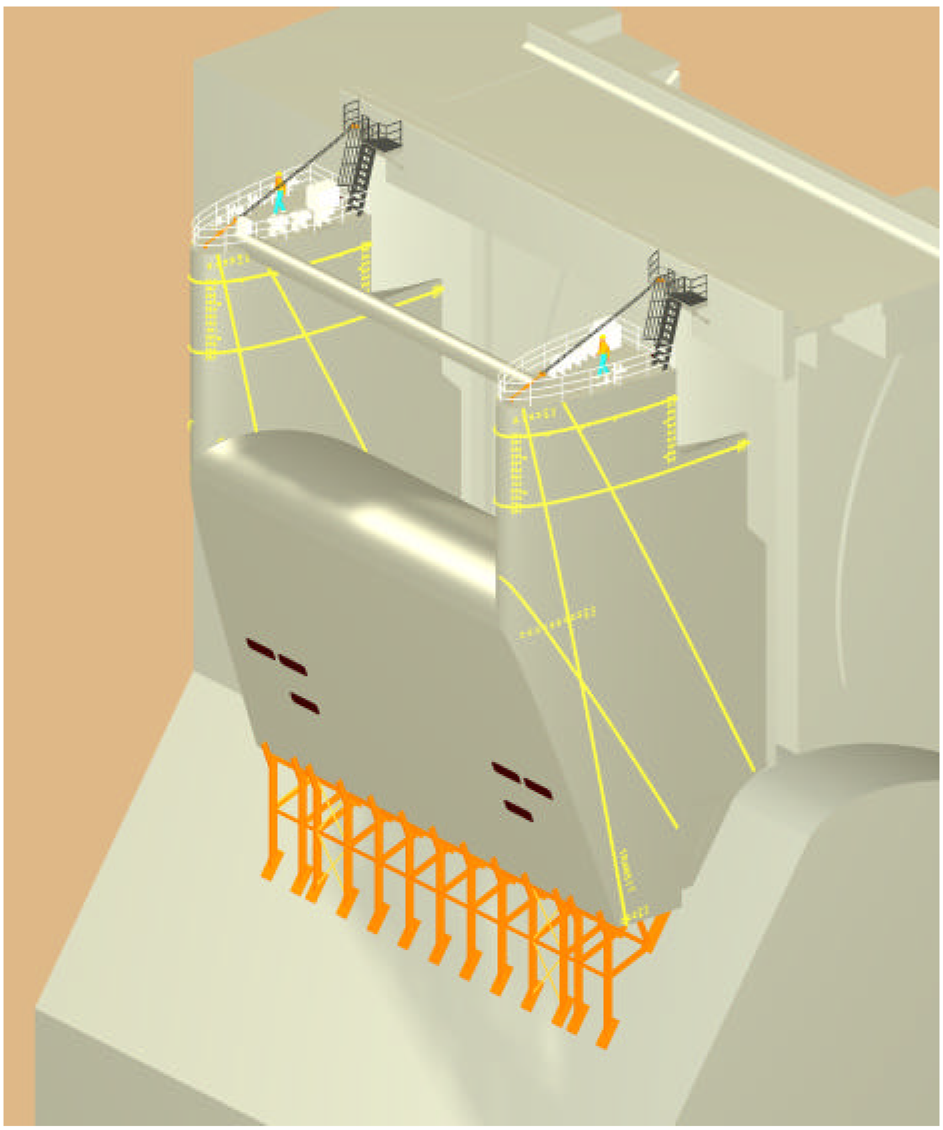
APPENDIX H

Operations and Maintenance Manual

US Army Corps of Engineers ®
Portland District

Operations and Maintenance Manual

John Day Dam
Removable Spillway Weir



September 2001

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LIST OF FIGURES

John Day Dam

Removable Spillway Weir

Operations Manual

1. INTRODUCTION

This is the first draft of the operations manual for the Removable Spillway Weir (RSW) to be installed on Spillway Bay 20 at the John Day Dam. It is based on the final plans and specifications submitted for bid. This manual will be updated during construction and installation of the RSW project. In the update, text will be rewritten and the figures containing the 3-D renderings from the CAD design model might be replaced with photos of the constructed project. In addition, manufacturers' data will be placed in notebooks, which will become the appendices to this manual. The construction drawings are a necessary reference to this manual. A set of drawings should be kept with each copy of the maintenance manual. Construction drawings are referenced throughout this manual.

a. Organization of This Manual

As background information for operations personnel this manual provides a discussion of the purpose of the RSW Project at John Day. This is followed by a basic description of the RSW project features and components in Section 2. In Section 3 there is a description of the construction and installation of the RSW components. Then Section 4 provides instructions for normal operation, and Section 5 describes general procedures for emergency operations. Section 6 describes maintenance of the RSW. Since the RSW is meant to be a prototype test bed for a surface bypass spillway, its design life is meant to be short and very little maintenance is required.

b. Purpose of Project

As required by the Biological Opinion for endangered salmon on the Columbia River each project on the river is meant to achieve a minimum 95% survival rate and 80% fish passage efficiency for downstream migrating juveniles. Beginning in 1995 the Corps of Engineers began to study surface flow bypass options for passing juvenile salmon past main stem projects. In 1998 the Corps completed Feature Design Memorandum No. 52 outlining the use of the skeleton bays as surface bypass spillways at the John Day Project. After review the System Configuration Team (SCT) decided that the cost was too high given the untested nature of the surface bypass flow concept. The SCT requested that the Corps investigate the idea of using the existing spillway as a lower cost approach to providing surface bypass flow. A permanent modification to the spillway was impractical since the flood passing capacity of the project would be reduced by a permanent surface flow bypass installation on the spillway. The concept of a removable spillway weir (RSW) was developed so that the RSW could be removed in case of a flood, restoring the full spillway capacity. At the John Day Dam the RSW could also be used as a prototype of the skeleton bay surface bypass spillway. Therefore, the John Day RSW is designed to be a prototype spillway to evaluate the surface bypass spillway with the possibility of converting the RSW to a more permanent operation.

2. PROJECT DESCRIPTION

a. Design Capacity

The John Day RSW is designed to operate during the fish passage season, from approximately April through mid-September. During the rest of the year there would be no flow over the RSW. The RSW is designed to be operated with no flow control. That is, the radial gate will be out of the water during RSW operation, and fully closed at other times. The flow over the spillway will range from 14,000 cfs to 21,000 cfs. This corresponds to a forebay water surface range from 262.0 to 268.0 feet msl or a range of 16.5 to 22.5 feet of head on the weir.

b. RSW Project Features

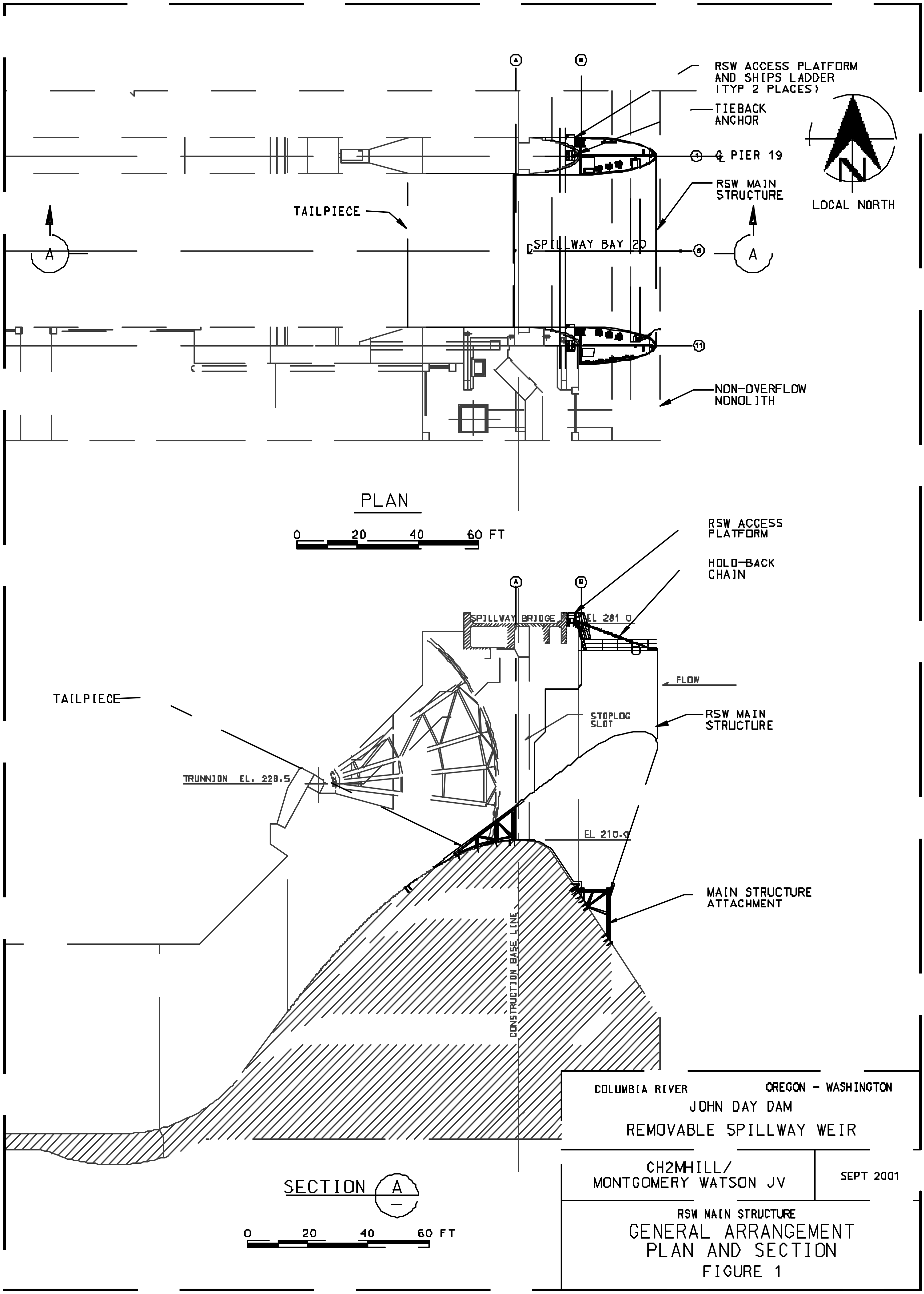
The RSW consists of three main components, Main Structure Attachment, Tailpiece, and Main Structure. See Figure 1. These components are designed to be installed on Spillway Bay 20, which is adjacent to the non-overflow section and near the powerhouse. A description of the physical features of each component is provided below.

1) Main Structure Attachment

The Main Structure Attachment is a series of frames connected to the dam from approximately elevation 194 to elevation 177 feet msl. See Figures 1 and 2. This is from 70 to 87 feet below the water surface at normal pool level. The Main Structure Attachment consists of 13 structural frames bolted to the spillway monolith across Spillway Bay 20. Structural members span between the frames to provide lateral support. The tops of the frames provide a platform, on which the Main Structure is to rest. The location of the frames corresponds to the location of the structural frames within the Main Structure. Shims are placed on the frames to set the Main Structure at the proper elevation.

2) Tailpiece

The tailpiece is located on the spillway surface just behind or downstream of the Main Structure. It forms the downstream portion of the RSW spillway. See Figure 1. The Tailpiece consists of a series of frames bolted to the surface of the existing spillway. These frames are covered with a skin plate on top, which forms part of the RSW spillway surface. On the upstream face the skin plate forms a surface to resist the hydrostatic force when the tainter gate is shut. The Tailpiece has a watertight seal along the bottom


RSW ACCESS PLATFORM AND SHIPS LADDER (TYP 2 PLACES)
TIEBACK ANCHOR
℄ PIER 19
RSW MAIN STRUCTURE
LOCAL NORTH
TAILPIECE
℄ SPILLWAY BAY 20
NON-OVERFLOW NONOLITH
PLAN
0 20 40 60 FT
RSW ACCESS PLATFORM
HOLD-BACK CHAIN
SPILLWAY BRIDGE
EL 281.0
FLOW
TAILPIECE
STOPLOG SLOT
RSW MAIN STRUCTURE
TRUNNION EL. 228.5
EL 210.0
MAIN STRUCTURE ATTACHMENT
CONSTRUCTION BASE LINE
SECTION A
0 20 40 60 FT
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JOHN DAY DAM
REMOVABLE SPILLWAY WEIR
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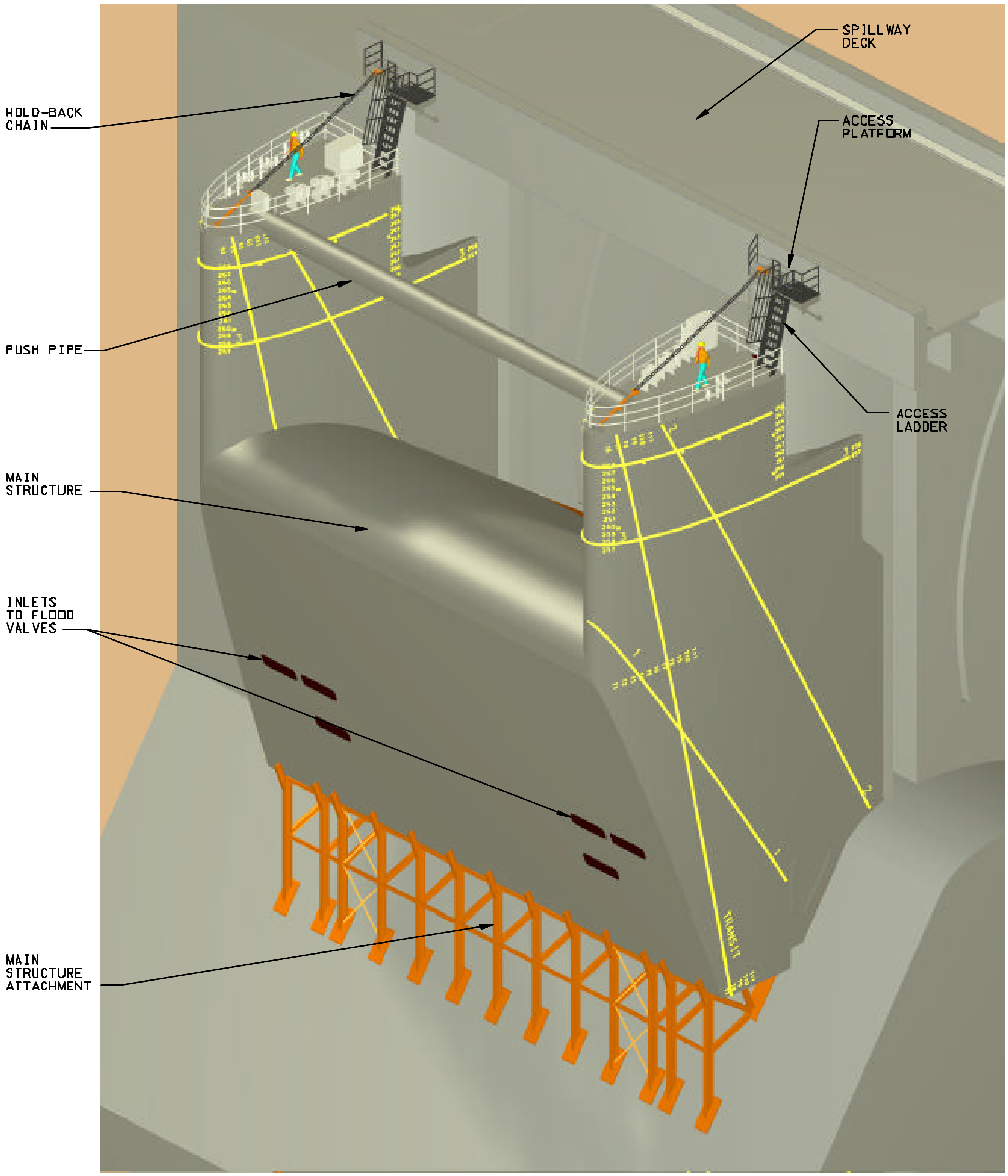
RSW MAIN STRUCTURE
GENERAL ARRANGEMENT
PLAN AND SECTION
FIGURE 1



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RSW MAIN STRUCTURE
INSTALLED RSW
VIEW FROM FOREBAY
FIGURE 2

and the sides to a point just downstream of the tainter gate. The Tailpiece is designed to operate under three main conditions:

1. With the Main Structure in place and the tainter gate open. This provides flow over the RSW and down the spillway and is the operating condition.
2. With the Main Structure in place and the tainter gate closed. This would be the case when the RSW is not in operation, generally October through March.
3. With the Main Structure removed and the tainter gate closed. This would only happen during installation or removal of the Main Structure or when the Main Structure is being inspected.

Since the tailpiece is sealed at its upstream face and the Main Structure is sealed, the head on the Tailpiece in condition 1 above is just above the top of the Tailpiece. Water leaking past the Main Structure is ponded upstream of the Tailpiece. In conditions 2 and 3 above, the full forebay hydrostatic head acts against the skin of the Tailpiece upstream of the tainter gate.

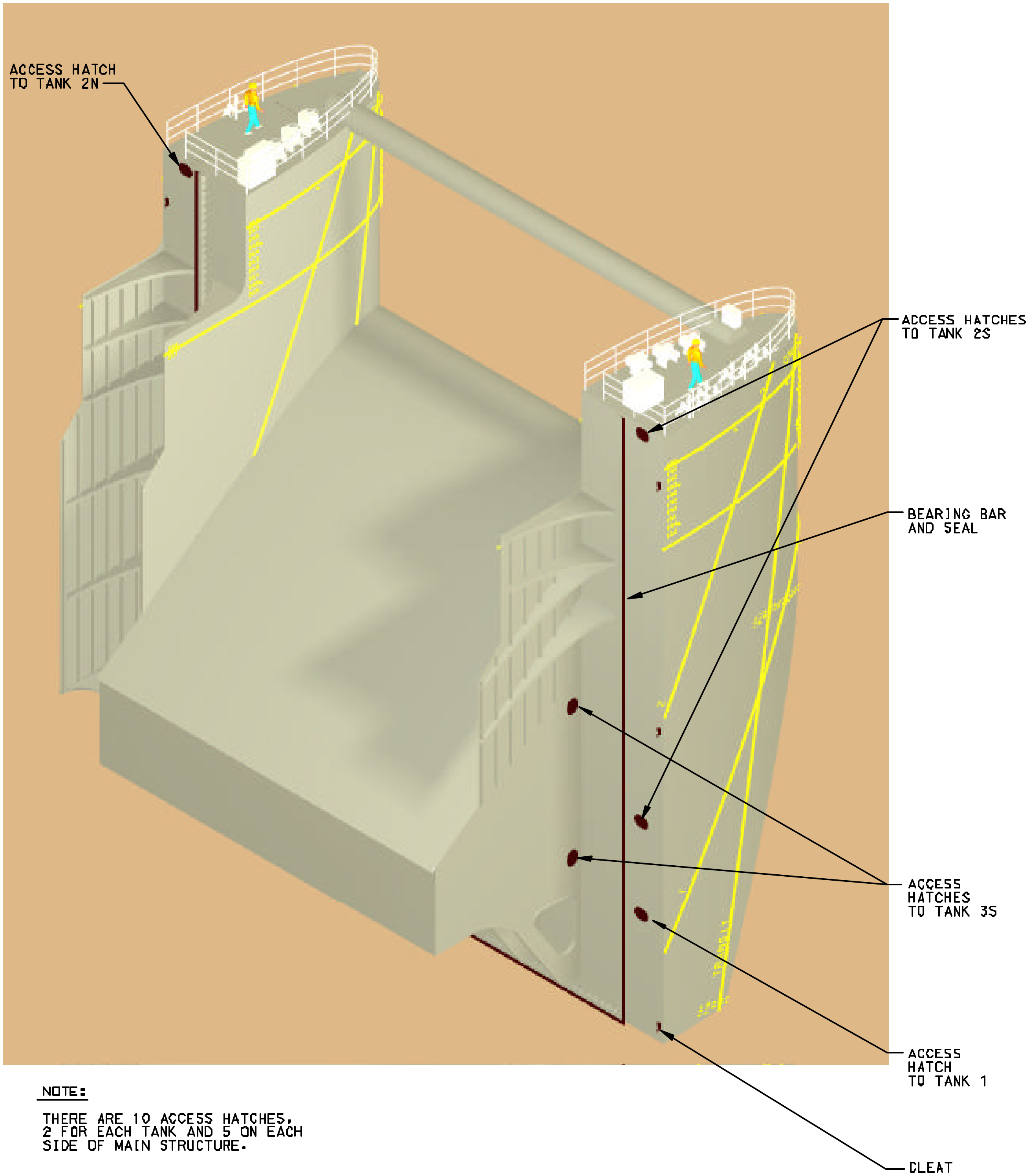
The Tailpiece is meant to be removed if the RSW is to be deactivated or in case of an impending large flood. See Section 4 Emergency Operation.

3) Main Structure

The Main Structure is the largest component of the RSW. See Figures 2 and 3. It is constructed of steel and weighs about 500 tons empty. It contains five tanks, which can be flooded with water to changes its orientation. It is meant to be transported to John Day Dam by a tugboat pushing on the 36-inch diameter push pipe, which spans between the tops of the piers. See Figure 4. Once at the dam the tanks of the Main Structure would be flooded to rotate it into a vertical position. The tugboat would push it into position on top of the spillway, and it would be filled with water to sink it into place on the Main Structure Attachment. See section 3 for a description of the installation procedure for all three of the RSW components.

<u>Structural Layout.</u> The Main Structure is constructed of steel. It is built somewhat like a ship in that the skin plating and watertight bulkheads act as structural beams. Inside the Main Structure there are 10 web frames and one watertight bulkhead oriented vertically when the Main Structure is installed on the dam. These features extend the full height and width of the structure. There are four flats, which are like decks extending horizontally within the spillway section of the RSW. One of these flats is watertight. Five horizontal flats including the deck on top of the pier are located in each pier of the Main Structure. A transverse water tight bulkhead runs parallel to the axis of the dam. Throughout the structure stiffeners that support the skin plating are connected to the web frames. All members in the structure are welded. See the structural construction drawings S1 through S24.

The skin of the structure and the interior water tight vertical bulkheads form five tanks within the Main Structure. See Figure 5. Tank 1 extends the full length of the Main



NOTE:

THERE ARE 10 ACCESS HATCHES, 2 FOR EACH TANK AND 5 ON EACH SIDE OF MAIN STRUCTURE.

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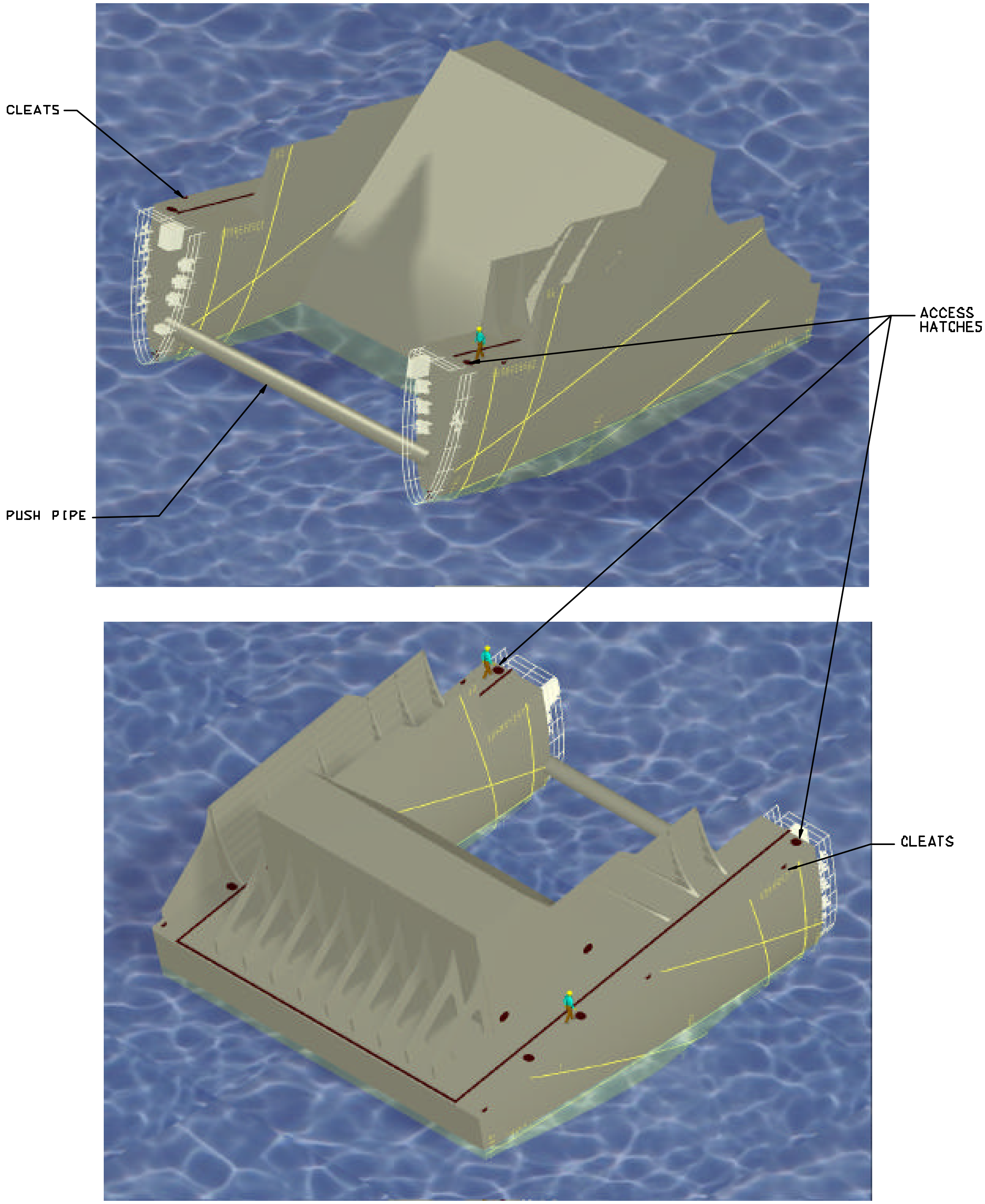
RSW MAIN STRUCTURE REAR VIEW FIGURE 3	



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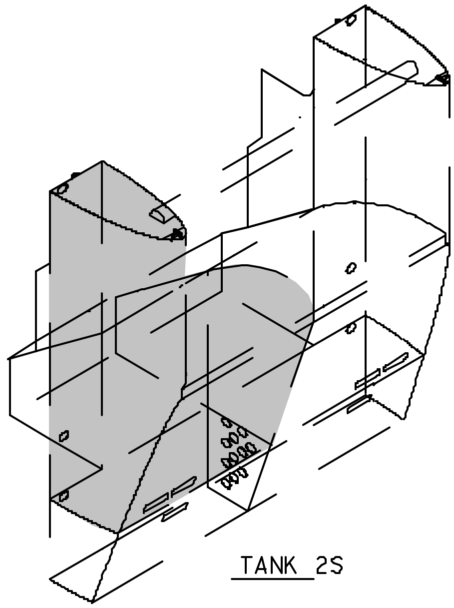
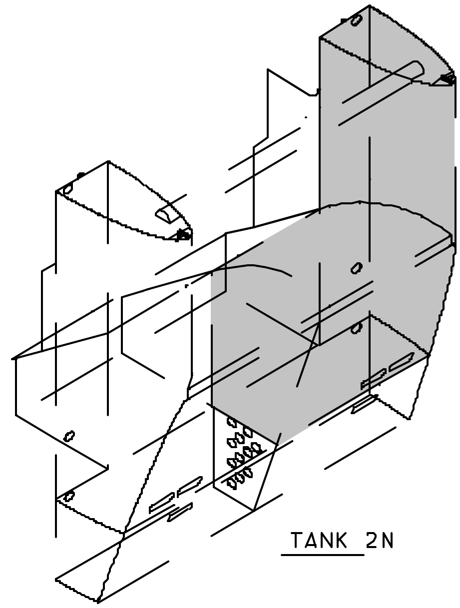
RSW MAIN STRUCTURE

TRANSPORT POSITION

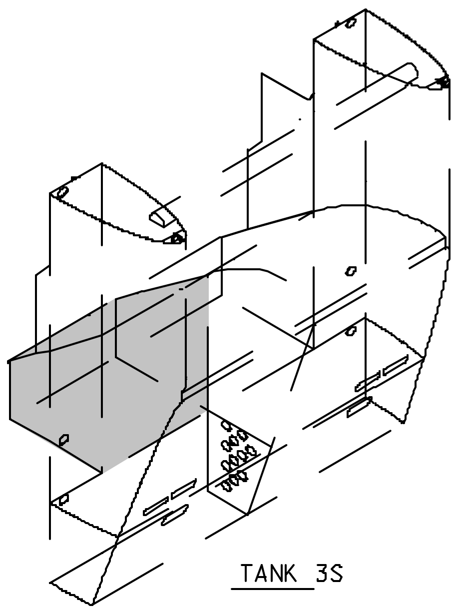
FIGURE 4



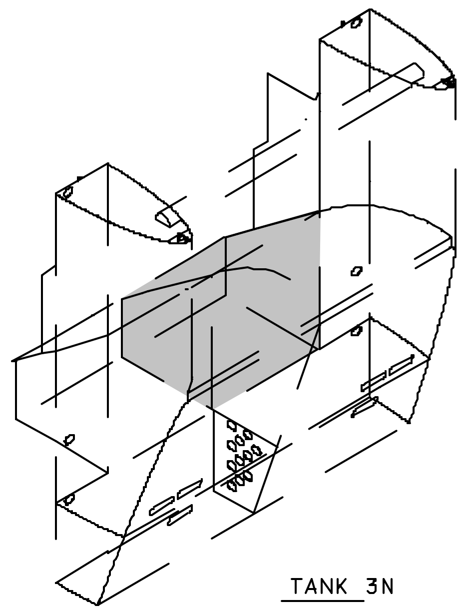
TANK 2S



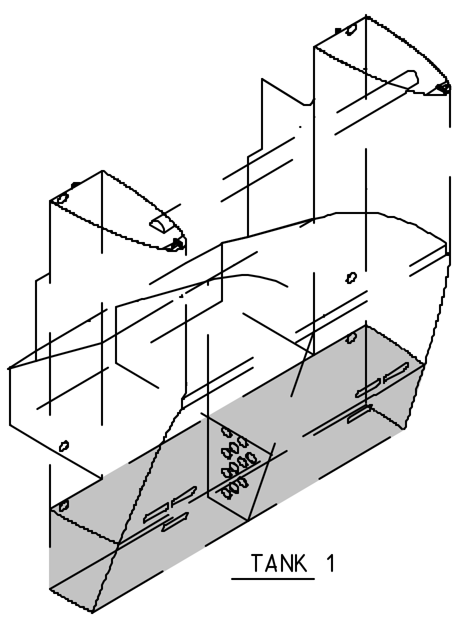
TANK 2N



TANK 3S



TANK 3N



TANK 1

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RSW MAIN STRUCTURE

BALLAST TANKS

FIGURE 5

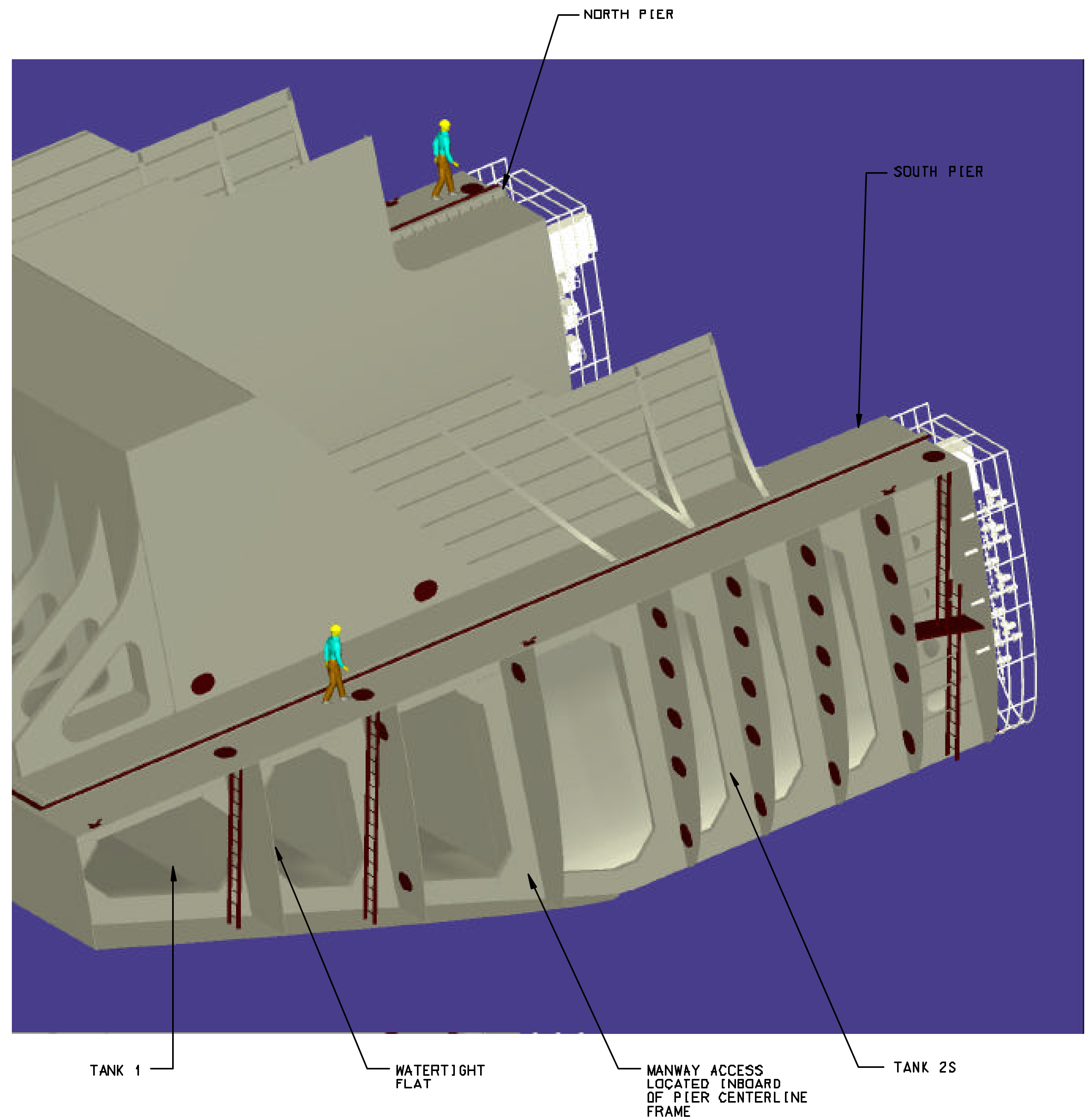
Structure and is located on the bottom. Tanks 2S and 2N are located above Tank 1 and extend up in the piers. Tanks 3S and 3N are located in the downstream portion of the Main Structure.

Access Provisions. Access to the top of each pier of the Main Structure is by means of an access platform and ladder. There is one platform for each pier. The top of the ladder is bolted to the access platform and the bottom of the ladder rests on the deck on top of the RSW pier. A portable ladder would be used to get over the parapet wall at the edge of the spillway deck down to the access platform. The platform also provides access to the top of the existing spillway pier for inspection and installation of the hold-back chain.

Access to the interior of the Main Structure can be gained through ten watertight hatches that are bolted shut. See Figure 3. The hatches for the tanks on the upstream or forebay side of the Main Structure, tanks 1, 2N, and 2S, are located on the flat back sides of the piers. The two hatches for the downstream tank, 3N, are located at the north end of the tank, and the two hatches for tank 3S are located on the south end of tank 3S. Access to the inside is only possible when the Main Structure is in the transport position. The ten openings provide access to the five watertight tanks. There are two hatches for access to each tank to allow for proper ventilation for personnel in the tanks. Ladders extend from the hatches to the bottom of tanks 2S, 2N, and 1. The hatches for tanks 3S and 3N provide access directly to the bottom of the tank when in the transport position. Openings are provided in the flats for access to all parts of each tank. See Figure 6.

Mechanical Systems. There are six flood valves, one for each tank except Tank 1 which has two. They are located on the front of the Main Structure as shown on Figure 2. The valves are recessed inside the structure to protect them during transport and installation of the Main Structure. The valves are 8-inch butterfly valves. Strainers are located at the ends of the pipe/valve assemblies to prevent material from entering the valves during filling and emptying of the tanks. See construction drawings S23, M6, M8, and M9. The strainers on the outside of the structure are cone strainers that can be removed and serviced by divers. A pipe cap retained by a victaulic type coupling is provided over the end of the pipe surrounding the strainer. The cap is to be installed for transport, installation, and removal of the Main Structure. The valves are actuated by electric operators located on the top deck of each pier. Three operators are located on each pier. The actuators also have manual wheel operators for backup operation. Reach rods extend from the operators down to the valves. Stuffing boxes are provided at the penetrations of the watertight flats and 90-degree gear boxes are provided where required. See the mechanical construction drawings, M8 and M9, for locations and further details.

Five vent and relief valve assemblies are installed on the Main Structure, one assembly for each tank. Each assembly consists of 3-inch vent pipes extending from the deck at the top of each pier into each tank. The vent and compressed air valves and other equipment extend above the deck level. The assemblies for Tanks 1, 2S, 3S, and 3N are located on the south pier, and the assembly for Tank 2N is on the north pier. See Figures 7 and 8. Each assembly above the deck has the following equipment as shown on Figure 9:



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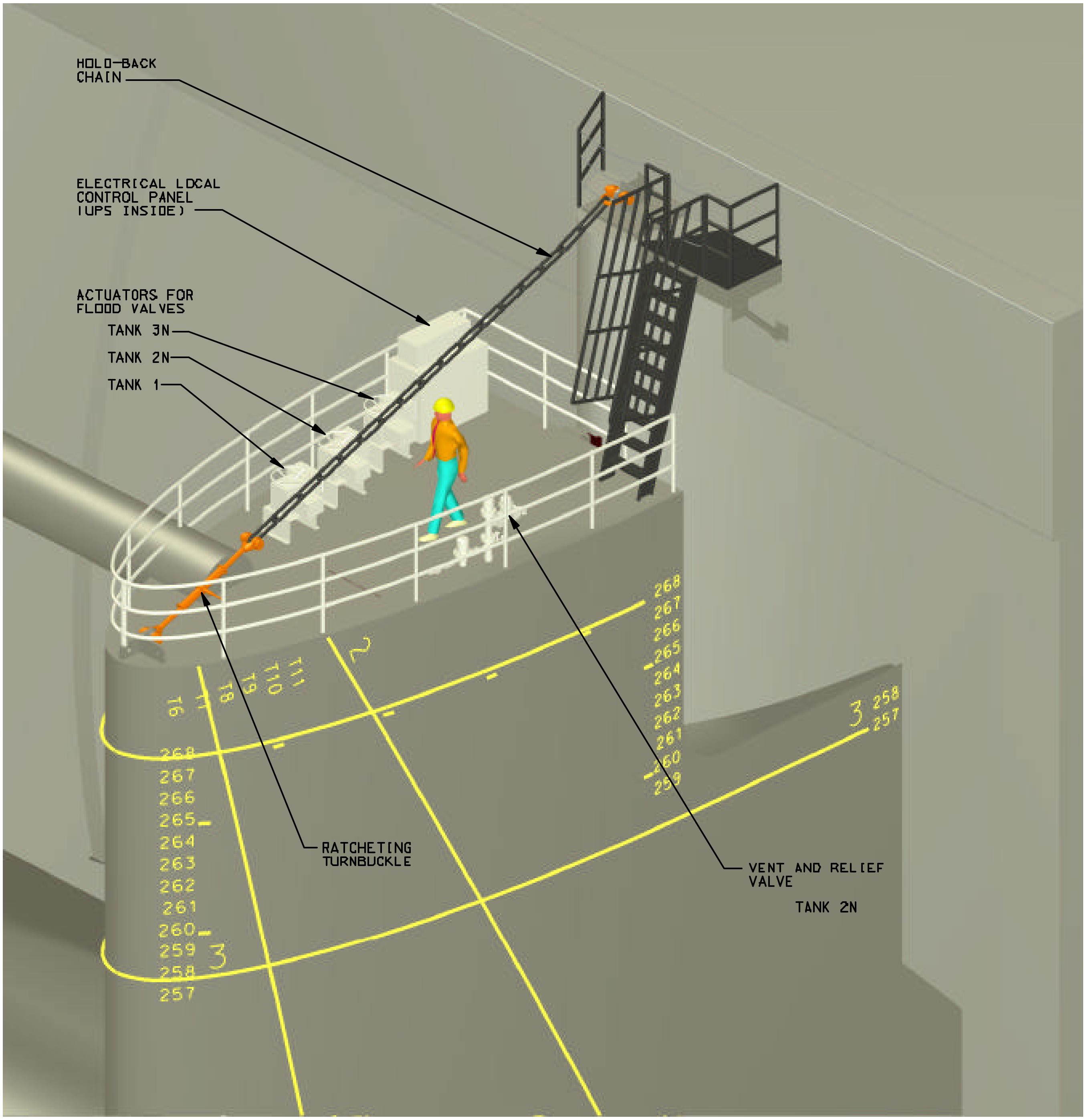
RSW MAIN STRUCTURE

ACCESS LADDERS

FIGURE 6



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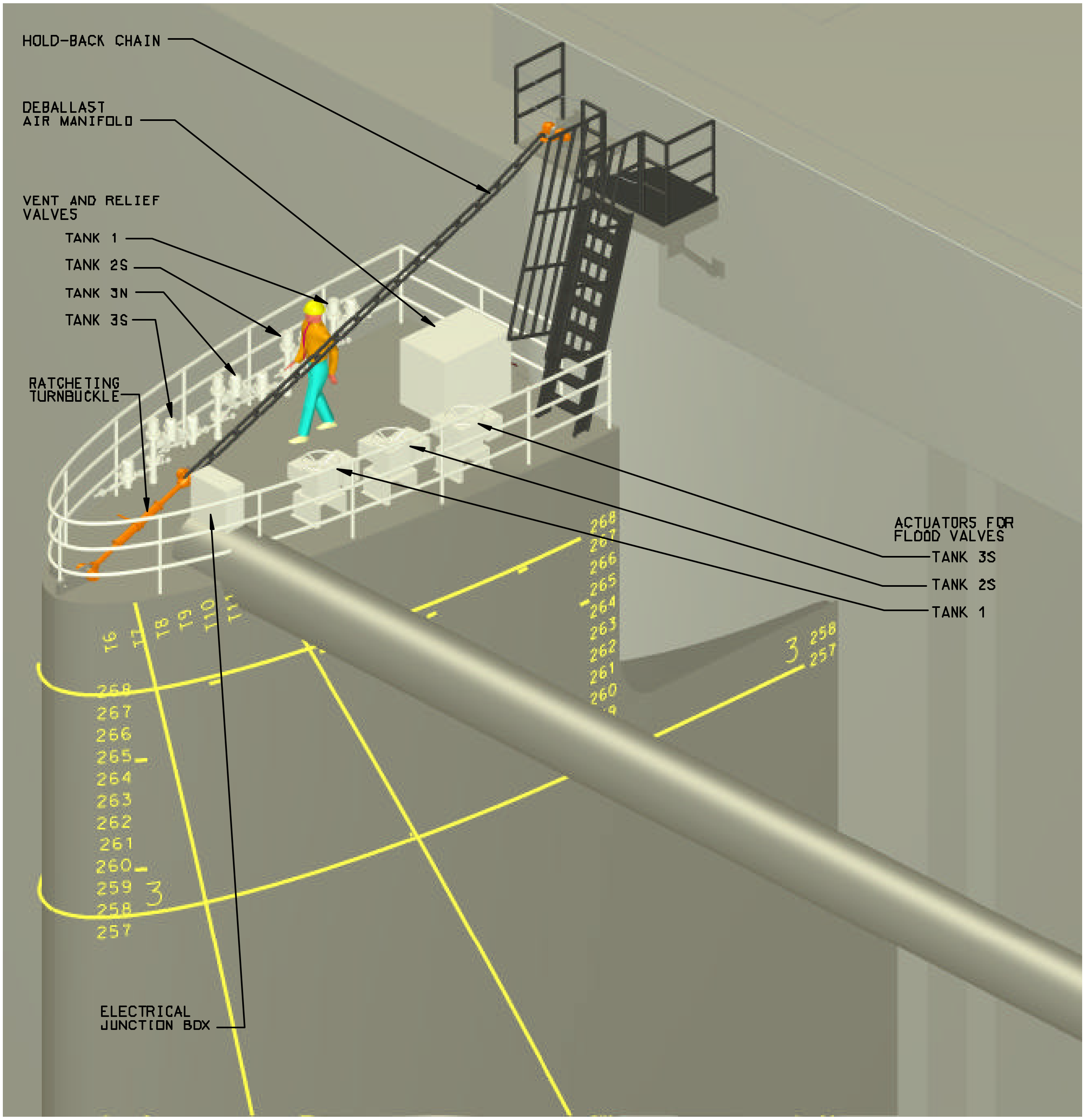
RSW MAIN STRUCTURE

NORTH PIER

FIGURE 7



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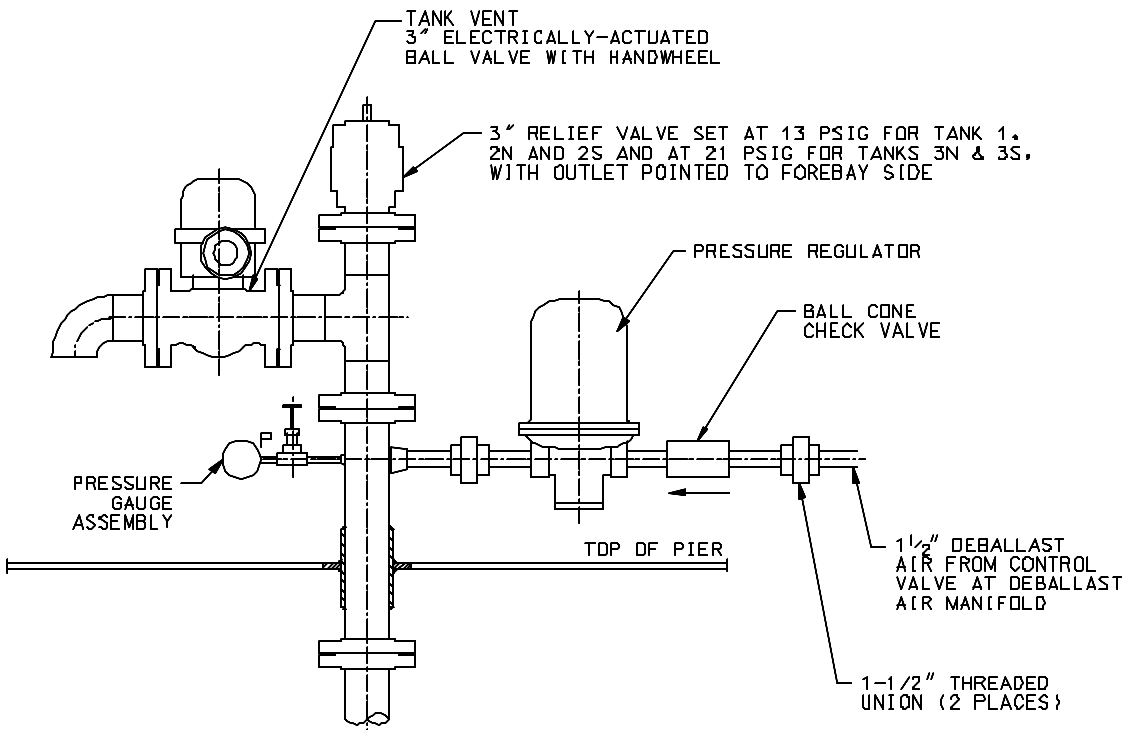
RSW MAIN STRUCTURE

SOUTH PIER

FIGURE 8



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RSW MAIN STRUCTURE

TYPICAL VENT & RELIEF
VALVE ASSEMBLY

FIGURE 9

1. Vent valve – a 3-inch electrically actuated ball valve. The actuator is equipped with a hand wheel for manual operation.
2. A 3-inch relief valve. The valve is to be set for 13 psig for tanks 1, 2N, and 2S and 21 psig for tanks 3N and 3S.
3. A pressure gauge assembly.
4. A pressure regulator on the 1½-inch compressed air supply pipe.
5. A ball cone check valve on the 1½-inch compressed air supply pipe.

Note the Main Structure design includes limber holes to avoid trapping air in pockets created by the internal stiffeners and girders. Adequate limbering is necessary for proper tank ventilation and successful installation of the Main Structure.

The compressed air system on the Main Structure is used to provide air to empty the tanks by forcing water out of the flood valves. Compressed air will be supplied by a unit mounted on the tugboat. The unit must be capable of providing 350 SCFM at 100 psig. The air hose from the tugboat will be connected to a quick disconnect breakaway coupling at the compressed air manifold on the south pier. Piping will then carry the compressed air through a pressure regulator at the manifold, which limits the pressure to 125 psig. The compressed air then flows to the air fittings on the vent and relief valve assemblies, where the air passes through a check valve and another pressure regulator to reduce the pressure to 13 or 21 psig depending on the tank. One pipe is run through the push pipe to the north pier to supply compressed air to the vent and relief valve assembly for Tank 2N. See the schematic on construction drawing M5.

Electrical Systems. The electrical systems for the Main Structure are all located on the decks at the top of both piers. See Figures 7 and 8. The electrical components and their locations are described below:

1. Electrical Junction Box – located on top of the push pipe on the Main Structure south pier. The electrical power and actuator signal wires are plugged into receptacles in this box.
2. Electrical Local Control Panel (LCP) - located on the downstream end of the Main Structure north pier.
3. Uninterruptible Power Supply (UPS) – located inside the electrical local control panel on the north pier. This unit is a standard commercially available type providing battery backed power for automatically closing the valve actuators in case external power is lost or the cables to the tugboat are broken. Once the Main Structure is installed the UPS can be removed from the electrical local control panel and stored elsewhere for future use. The UPS should be recharged once a month to keep it ready for immediate use.
4. Vibration Sensor Cable Connections – located in the electrical local control panel. A portable laptop computer can be connected to the cables to collect vibration data, which would be analyzed later for frequency and amplitude.

The power and signal cables run from the tugboat to the electrical junction box on the south pier. The power and signal cables run to the actuators for the fill, vent, and compressed air valves. The cables that cross to the other pier run through the push pipe. From the electrical local control panel cables run to the flood and vent valve actuators on both piers to actuate the valves when powered by the UPS.

Electrical power is required only during installation or removal of the Main Structure. The tugboat must supply 120VAC 1 phase 60 Hertz power through a standard receptacle/outlet. The feeder circuit breaker should be rated for at least 20 amps, though up to 55 amps would be acceptable. The power is used to actuate the flood, vent, and deballast air valves. Under normal ballasting and deballasting operations the signals for opening and closing the flood and vent valves come from a portable local control panel which will be set on the tugboat for the installation and removal process. This panel has manual Open/Off/Close switches for each flood, vent, and deballast air valve.

In case the tugboat power is lost or cables break during installation or removal, the flood and vent valves must be closed to prevent uncontrolled flooding and possible sinking of the Main Structure. Therefore, the UPS is required to supply power to close these valves in those cases. The UPS does not have the power to close all 8-inch flood valves at once. In case of a loss of power or broken cables, time delay relays located in the electrical local control panel will sequentially close vent and flood valves using UPS power to stop flooding of the tanks. The valve closing sequence is as follows:

1. All vent valves will close simultaneously. Since no air can be expelled, flooding is limited.
2. Close flood valve for tank 2S.
3. Close flood valve for tank 2N.
4. Close flood valve for tank 3S.
5. Close flood valve for tank 3N.
6. Close flood valve for tank 1.

3. CONSTRUCTION AND INSTALLATION

An explanation of the construction and installation of the RSW is given here to provide operations personnel with some background on these issues. This might help in performing the operations and maintenance tasks associated with the RSW.

a. Construction

A first step in the construction is to survey the existing spillway and piers. Most of the survey will take place under water. Therefore, a template would probably have to be constructed and attached at surveyed locations on the dam. This information would be used in fabrication of the RSW components.

The construction of the RSW components is envisioned to take place at a fabrication site some distance from John Day Dam. The frames and other structural members of the Tailpiece and Main Structure Support would be fabricated and transported to the site, probably by barge, since access to the top of the dam is limited. The Main Structure would be fabricated in a yard or drydock. After construction and inspection and testing it would be launched and transported to the dam. A tugboat would push the Main Structure up river, or the Main Structure could be placed on a barge. The Main Structure Attachment would have to be ready first followed by the Tailpiece to accommodate the installation procedure described below.

b. RSW Installation

The construction contractor would perform the installation of the RSW components. The contractor is also responsible for installing the Main Structure according to the procedure detailed in the construction drawings. The construction contract also contains optional tasks for the contractor to remove the RSW in an emergency situation and at the end of the RSW testing period.

The sequence of installation of the RSW components is as follows:

1. The Main Structure Attachment would be installed first. The frames would be bolted to the dam and the cross members attached.
2. Dewatering stoplogs would be installed upstream of the tainter gate.
3. The tainter gate would be raised to a fully open position and pinned in place.
4. <u>Tailpiece Installation.</u> The outer frames of the tailpiece would be installed and the tainter gate closed on the frames. The height of the frames would be adjusted so that the tainter gate would rest equally on each frame. Then the inner frames would be installed to the same height as the outer ones. This will provide an even seating surface for the tainter gate to rest on the Tailpiece and would minimize leakage. Note: The material for the Tailpiece could be brought to the spillway in any of three ways: 1) lowered by crane through the stoplog opening in the spillway deck; 2) lowered through the opening between the tainter gate and spillway bridge; 3) by crane on a barge in the stilling basin. Schemes 1) and 2) would require that protection, such as wood, be attached to the tainter gate or top of stoplogs to protect them.
5. The skin plates would then be welded onto the frames to complete the Tailpiece installation. The top of the Tailpiece would then be surveyed.
6. The tainter gate would be lowered onto the Tailpiece and the stoplogs removed.
7. The top of the Main Structure Attachment frames would be surveyed to ascertain their elevation. Shims of the correct size would then be installed on each frame. The shims would be at the correct elevation so that the Main Structure would fit properly at the Tailpiece.
8. The Main Structure would then be installed to rest on the shims on the Main Structure Attachment. A general explanation of the Main Structure installation is given in the next section. See the construction drawings, G4 through G6, for detailed installation procedures.

9. Divers would then measure the fit between the Main Structure and the Tailpiece at the RSW spillway surface to see if the fit is within tolerance.
10. If the fit does not meet tolerance the Main Structure would be raised one or two feet by forcing compressed air into the Main Structure. Divers would then adjust the height of the shims. Tanks 2N, 2S, 3N, and 3S would be flooded to lower the Main Structure into position.
11. Repeat steps 9 and 10 until the fit between the Main Structure and Tailpiece is within the required tolerance.
12. If the fit between the Main Structure and Tailpiece at the RSW Spillway surface meets tolerance, attach the hold-back chains and disconnect the electrical and control cables and air hoses from the Main Structure. Remove the uninterruptible power supply (UPS) and store it per manufacturer's recommendations. This completes the installation procedure.

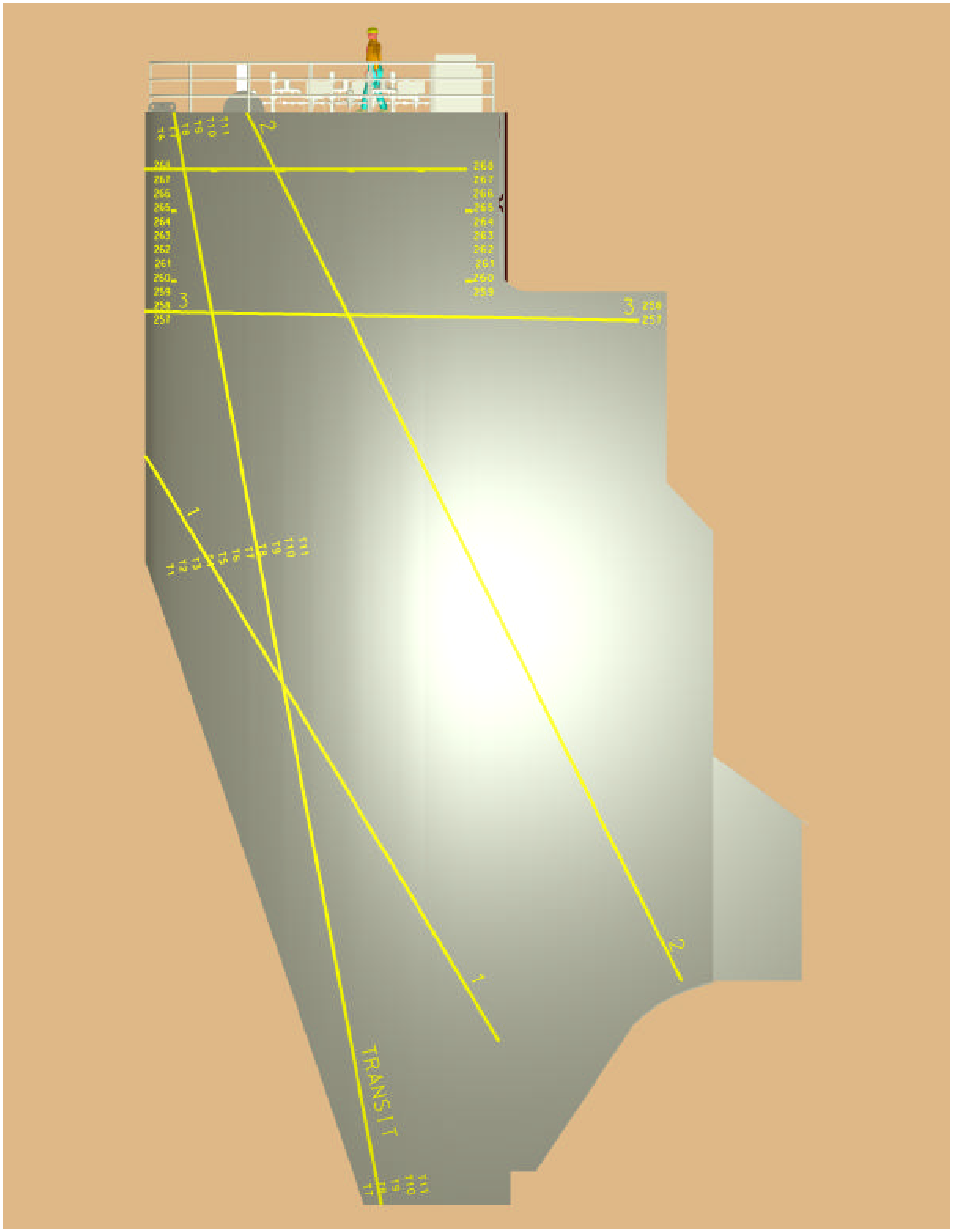
c. Main Structure Transport, Installation, and Removal

Transport. The Main Structure is to be transported to the dam on a barge or by a tug pushing the Main Structure. The orientation of the Main Structure for transport is shown on Figure 4. The water surface would be at the TRANSIT line on the Main Structure as shown on Figure 10. The tug would be held to the Main Structure by lines and would push on the push pipe to move it. The connection points for the lines on the Main Structure would be designed by the Contractor.

Installation. The Main Structure is designed to rest atop the Main Structure Attachment when in the operating position. See Figure 2. The bottom downstream portion of the Main Structure is not designed to rest on the existing spillway crest. It is to be ½ inches above the existing crest. For installation, the tanks within the Main Structure would be flooded to rotate it and lower it onto the Main Structure Attachment. This would be done at two times, during initial installation and replacement after inspection. The vent valves would be opened and high-pressure air blown into the tanks to float the Main Structure and remove it.

Detailed instructions for installing and removing the Main Structure are provided in the construction drawings on drawings G5 and G6. The general steps for installation of the Main Structure are provided below.

1. Preparation: Attach the power and control cables and compressed air hoses to the Main Structure from the tug. The power and compressed air will be supplied by the tug. The power required is 120 volts AC, and the compressed air supply is to be 350 CFM at 100 psig. The Tug Local Control Panel for opening and closing all valves on the Main Structure will be located on the tug. Construction drawings E2 and E4 show the Tug Local Control Panel. Slack the lines holding the Main Structure to the tug. This will allow distance from the tug for the Main Structure to rotate. Install the UPS in the Local Control Panel atop the north pier of the Main Structure. Divers remove the cover plates over the ends of the flood pipes. Test the operation of each valve.



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RSW MAIN STRUCTURE

HULL MARKINGS FOR BALLASTING

FIGURE 10

Note: In the valve operation below the flood valves can only be opened one at a time due to power constraints. Two or more flood valves cannot be activated at the same time or a circuit breaker may pop.

2. Step 1: Open the flood valves and then the vent valves in Tank 1. See Figure 5 for the location of the tanks. Allow the tank to completely fill and Main Structure to rotate so that the water surface is at Line 1. See Figure 10. This will take about 2 hours. Close the vent and flood valves for Tank 1.
3. Step 2: Open the flood valves and then the vent valves in Tanks 2N and 2S. Allow the tank to completely fill until it rotates so that the water surface is at Line 2 as shown on Figure 10. This will take about 2 hours and Tanks 2N and 2S will be about 73% filled. Close the vent and flood valves for Tanks 2N and 2S.
4. Step 3: Open the flood valves and then the vent valves in Tanks 3N and 3S. Allow the tank to fill until it rotates so that the water surface is at Line 3 as shown on Figure 10. This will take about 24 minutes. During this time the Main Structure is expected to rotate slowly to an angle of about 45 degrees from vertical. The Main Structure will remain near 45 degrees for a time and then move rapidly to an angle of about 30 to 35 degrees from vertical. The rotation will then continue slowly until the Main Structure is vertical. Close the vent and flood valves for Tank 3N and 3S.
5. The tug will push the Main Structure into position against the spillway piers. The forebay water level must be at elevation 259.3 feet or above in order to install the Main Structure and get it over the guide beams at the upstream side of the Main Structure Attachment. Attach lines and blocks and tackle to the dam as required to guide the tug in placement of the Main Structure up to the face of the dam.
6. Open flood valves to Tanks 2N, 2S, 3N, and 3S and then open vent valves for Tanks 2N, 2S, 3N, and 3S. This will lower the Main Structure onto the Main Structure Attachment. This completes the Main Structure installation except for fine adjustments to its elevation as described in steps 8, 9, and 10 in the previous section.

Removal. The Main Structure would be removed by following the installation steps in reverse order. The flood valves and compressed air valves would be opened to admit pressurized air into the tanks to force water out through the flood valves. Tanks 3, 2, and 1 would be evacuated in order. This procedure would bring the Main Structure back to the transport position. See construction drawings G7 and G8 for detailed instructions on removal of the Main Structure. The general Main Structure removal process is outlined below.

1. Preparation: Attach the power and control cables and compressed air hoses to the Main Structure from the tug. The power and compressed air will be supplied by the tug. The power required is 120 volts AC, and the compressed air supply is to be 350 CFM at 100 psig. The Tug Local Control Panel for opening and closing all valves on the Main Structure will be located on the tug. Construction drawings E2 and E4 show the Tug Local Control Panel. Attach the lines holding the Main Structure to the tug. Install the UPS in the Local Control Panel atop the north pier of the Main Structure. Attach lines to the dam as required to guide the tug during removal of the Main Structure. Remove the Hold-back Chains from the top of the Main Structure piers. Test the operation of each valve and insure that all vent valves are closed.

Note: In the valve operation below the flood valves can only be opened one at a time due to power constraints. Two or more flood valves cannot be activated at the same time or a circuit breaker may pop.

2. Step 1: Open the flood valves and then the deballast air valves in Tanks 2N, 2S, 3N, and 3S. See Figure 5 for the location of the tanks. Allow the Main Structure to float up to Line 3. See Figure 10. This will take about 30 minutes. Close the deballast air and flood valves for Tanks 2N, 2S, 3N, and 3S. The tug will then back away from the dam to begin the rotation of the Main Structure to the transport position.
3. Step 2: Slack the lines connecting the tug and Main Structure before starting the rotation process. Open the flood valves and then the deballast air valves in Tanks 3N and 3S. Allow the tank to empty until it rotates so that the water surface is at Line 2 as shown on Figure 10. This will take about one hour. During this time the Main Structure is expected to rotate slowly to an angle of about 30 to 35 degrees from vertical. The Main Structure will then move rapidly to an angle of about 45 degrees from vertical, before resuming a slow rotation to the orientation of Line 2 as shown in Figure 10. Air bubbles should be visible at the water surface and Tanks 3N and 3S will be empty as Line 2 is reached. Close the flood and deballast air valves.
4. Step 3: Open the flood valves and then the deballast air valves in Tanks 2N and 2S. Allow the tank to completely empty until it rotates so that the water surface is at Line 1 as shown on Figure 10. This will take about 3 hours. Close the vent and deballast air valves.
5. Step 4: Open the flood and deballast air valves for Tank 1. When large air bubbles can be seen exiting the tank, close the flood and deballast air valves. The Main Structure should be floating near the TRANSIT line. Air bubbles will be seen on the water surface when the tank is empty. This should take about one hour.
6. At this point the manholes on the Main Structure can be opened and submersible pumps used to pump out the water remaining in the tanks to reach the TRANSIT line. Remove the power and control cables and compressed air hoses. Move the tug into position on the Push Pipe and tighten the lines holding the tug to the Main Structure.

4. NORMAL OPERATIONS

a. Raising and Lowering the Tainter Gate

The operation of the RSW is for uncontrolled spill only. The tainter gate must be fully open out of the water. The RSW is not designed for flow control using the tainter gate. **Do not operate the RSW using the tainter gate for flow control.**

In model studies of the RSW it was found that during raising and lowering the tainter gate a sloshing was set up immediately in front of the tainter gate and above the RSW Main Structure spillway. The sloshing waves are estimated to be six to eight feet high. Therefore, when opening or closing the tainter gate, once the gate is started, do not stop until it is fully open out of the water or fully closed.

In the model it was demonstrated that opening the tainter gate without the Main Structure installed on the dam caused a major flow separation over the top of the Tailpiece. This caused severe uplift pressures on the Tailpiece, for which it was not designed. **Do not open the tainter gate with the Main Structure removed.**

b. Vibration Monitoring

During design there was a concern about cyclical loading on the Main Structure. Cyclical loading could cause fatigue and possible failure in some structural members or connections. Model studies indicated that the cyclical loading would not be strong enough to cause structural problems. However, as a precaution four vibration sensors are installed inside the Main Structure. These sensors are strain gages that can measure strain variations and temperature (necessary for temperature compensation). The strain gauges are of the vibrating tensioned wire type typically used inside dams, and other large permanent structures. The gauges are mounted to be waterproof and retain accuracy over many years. The gauges require no external power, the cables plug directly into the portable readout.

Output cables from the sensors terminate in the electrical local control panel on the north pier. Follow the manufacturers' instruction as to how to connect the portable readout to the sensor output. The readout can download the data onto a personal computer running on Microsoft Windows ® operating system. Software for analyzing the data is supplied with the sensor units.

The vibration sensors produce a very large amount of data. So, the data will be gathered for short lengths of time at intervals during operation of the RSW. The data shall be collected according to the schedule below.

- At initial startup
- At the beginning of continuous operation
- Once per day for 14 days commencing one day after start of continuous operation
- Once per week while the RSW is in operation starting after completion of the 14 days of daily sampling

Note the elevation of the forebay water surface for each sampling session.

The data shall be analyzed for frequency and amplitude of vibration. These values shall be noted for each sensor for each sampling session. A change in frequency or amplitude over time in any of the sensors could indicate that a structural problem is developing.

c. Inspections

Visual inspections of the RSW shall be carried out daily. The inspector is to look for any change in the amount of vibration, any debris on the RSW piers, the amount of flutter in the pier wings, which lie along the side of the existing pier, and anything unusual on the deck of the tops of the RSW piers. The tension in the hold-back chain should be checked periodically. If it begins to loosen, the ratchet turnbuckle can be tightened.

5. EMERGENCY OPERATION

Emergencies and their nature are difficult to anticipate. Conditions, which could constitute an emergency, would be structural failure or anticipated structural failure of any RSW component. In case of structural problems close the tainter gate immediately. Once the gate is closed the hydrostatic force is borne by the tainter gate and Tailpiece. Closing the tainter gate unloads the Main Structure.

a. Emergency RSW Removal

The presence of the RSW restricts the capacity of Spillway Bay 20 and reduces the flood passing capability of the John Day Dam. If a flood approaching the size of the spillway design flood is expected, the Main Structure and Tailpiece will have to be removed. For the expected testing period an optional task for removal of the Main Structure is in the construction contract. The contractor must start removal within 72 hours notice from the Government. After removal of the Main Structure, the dewatering stoplogs would be installed and the tainter gate raised. Crews would then be lowered down to the Tailpiece and it would be cut up and removed as quickly as possible. Once the tailpiece has been removed the stoplogs would be taken out, restoring the full spillway capacity.

6. MAINTENANCE

a. Inspections

An inspection of the entire RSW is scheduled for immediately after the end of the first season of operation. The construction contractor will remove the Main Structure to a safe anchorage area near the dam. Personnel would inspect the interior and exterior of the structure. **<u>Safety precautions for work in confined spaces must be followed.</u>**

Since the expected life of the RSW is three years, no maintenance is envisioned.

APPENDIX I

Model Study Report*

APPENDIX I
MODEL STUDY
REPORT

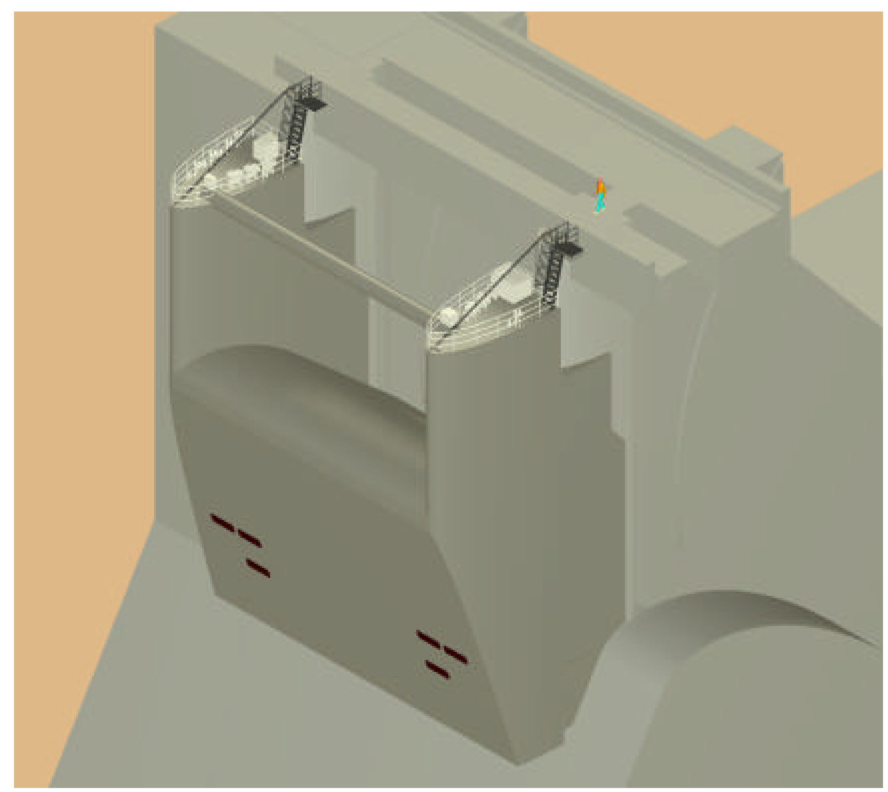
John Day Lock and Dam
Removable Spillway Weir
Design Documentation Report No. 53

NOVEMBER 2001

US Army Corps
of Engineers ®
Portland District

Design Documentation Report
No. 53

John Day Lock and Dam
Removable Spillway Weir
And Spillway Bay 20 Deflector
Appendix I
Hydraulic Model Study



Prepared by
Northwest Hydraulic Consultants

November 2001

Contract DACW57-97-D-0004 T.O. No. 14

JOHN DAY LOCK AND DAM
REMOVABLE SPILLWAY WEIR
And SPILLWAY BAY 20 DEFLECTOR

Design Documentation Report No. 53

APPENDIX I

HYDRAULIC MODEL STUDY
FINAL REPORT

Prepared for:

U.S. Army Corps of Engineers
Portland, Oregon

Prepared by:

northwest hydraulic consultants
Tukwila, Washington

November 2001

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1.0 INTRODUCTION

1.1 General

The U. S. Army Corps of Engineers (USACE) developed a feature design memorandum for design of a high flow surface collector modification to existing powerhouse skeleton bays at John Day Dam on the lower Columbia River in 1998 ("John Day Lock and Dam Surface Bypass Spillway, Feature Design Memorandum No. 52, USACE Portland District, September 1998"). The purpose of the modification, referred to as the Skeleton Bay Surface Bypass (SBSB), is to assist in passing downstream migrants at the project. Because of the relatively high capital cost of the SBSB, the USACE is considering construction of a removable spillway weir (RSW) to provide a full-scale prototype test of the efficiency of the high flow surface bypass concept. The RSW, which would emulate the hydraulic forebay withdrawal and tailwater egress characteristics of the SBSB, at a lower cost, would be installed within an existing spillway bay. The RSW was designed so that it could be removed in a timely manner to provide capacity to meet spillway design discharge requirements during large flood events. The design of the RSW is documented in "John Day Lock and Dam Removable Spillway Weir Design Documentation Report No. 53", dated October 2001. A 1:25 scale section model of the spillway and the RSW, and a companion 1:25-scale model of the SBSB, were used to: (1) evaluate the hydraulic performance characteristics of various RSW design concepts, (2) evaluate the hydraulic performance of a spillway deflector downstream from the RSW on Spillway Bay 20, which is adjacent to the powerhouse skeleton bays, and (3) enable a same scale comparison of the hydraulic characteristics of the RSW and the SBSB. This model report constitutes Appendix I to Design Documentation Report No. 53. The study was conducted in accordance with Contract No. DACW57-97-0004, T.O. No. 14, between the USACE and CH2M Hill/Montgomery Watson Joint Venture. Northwest Hydraulic Consultants (NHC), a sub-contractor to the Joint Venture, conducted the model study under the overall direction of Montgomery Watson. The Statement of Work (SOW) for the study is reproduced herein as Appendix A.

Notice to Proceed on the model study was received April 24, 2000. Edwin T. Zapel and James L. Lencioni of NHC's Seattle office had overall direction of the model study, and maintained liaison with Diana Modini, Chris Goodell and Kyle McCune of the USACE and Dennis Dorratcague of

Montgomery Watson. Dr. Alan F. Babb supervised the model study. Kenneth Christison was project engineer for the study. Dr. Albert Mercer provided independent technical review of the model study.

Construction of the model was completed June 16, 2000. Preliminary tests of four different RSW alternative designs were conducted between June and August. These designs were demonstrated to representatives from the USACE and resource agencies during laboratory visits on June 20 and 21, 2000; July 19 and 20, 2000; and August 7 and 8, 2000. Participants for all model demonstration visits are listed in Table 1.1. At a meeting immediately following the August demonstration, the representatives from the USACE, the agencies, and NHC jointly selected the Alternative 5 design concept as the final design to be used for detailed documentation.

The performance of various deflector geometries on Spillway Bay 20 with the Alternative 5 RSW was evaluated during a model demonstration for USACE personnel February 26 to March 2, 2001. This demonstration resulted in selection of the preferred deflector design geometry for final documentation. The Alternative 5 RSW with the selected deflector design and the SBSB were demonstrated to NMFS personnel from April 2 to 6, 2001.

1.2 Existing Project

John Day Dam (Figures 1-1 and 1-2) is located on the Columbia River about 25 miles upstream from The Dalles Dam, and approximately 216 miles upstream from the mouth of the river. The project consists of a navigation lock on the right (north) shore, a right bank fish-passage facility adjacent to the lock, a 20-bay radial-gated ogee-crest spillway constructed across the right portion of the river, a powerhouse with four skeleton bays and 16 operational bays adjacent to the left (south) shore, and an adult fish-passage facility between the powerhouse and the left shore. The overall reservoir operating range is 257 ft to 268 ft, however the reservoir normally operates in the range of 262 ft to 265 ft.

The spillway is designed to pass a discharge of 2,250,000 cfs (112,500 cfs per bay) at pool elevation 276 ft. Each spillway bay is 50 ft wide and separated by 12 ft wide piers. The spillway crest is at elevation 210 ft, and the downstream stilling basin has a floor elevation of 114 ft. The end sill of the stilling basin is approximately 290 ft downstream from the spillway crest and has a top elevation of 127 ft. The RSW is being considered for installation in Spillway Bay 20 that is adjacent to the four powerhouse skeleton bays that separate the spillway and powerhouse.

Spillway Bays 1 and 20 differ from the remaining bays in that they each have long training walls or piers extending downstream into the stilling basin. The spillway bays are numbered from 1 to 20 beginning at the north end of the spillway.

Deflectors having a length of 12.5 ft and set at elevation 148 ft with a 15 ft radius transition between the deflector and spillway, as shown in Figure 1-3, were added to Spillway Bays 2 through 19 in 1997 and 1998. These deflectors direct flows along the surface of the water in the stilling basin to assist in controlling dissolved gas levels. Deflectors are currently being considered for installation in both end bays (Spillway Bay 1 and 20). A separate model study ("John Day Dam Spillway Flow Deflectors Performance Curves, Existing Interior-Bay and Proposed End-Bay Deflectors", NHC, August 1999) was used to study a deflector addition to Bay 1, adjacent to the north-fish ladder.

1.3 Proposed Removable Spillway Weir Structure

The proposed RSW structure includes a crest with piers that would be installed in Spillway Bay 20 of the existing spillway structure. The entire RSW structure is constructed to be removable from the spillway bay. Various RSW alternative configurations were tested but all had a standard shaped ogee crest with a width of 50 ft and a crest elevation of 245.5 ft. The ogee crest shape is designed for a head (H_d) of 22.5 ft with a downstream quadrant equation of $Y = 0.03545\ X^{1.85}$. The RSW crest's upstream quadrant is an ellipse having a minor axis of 3.7125 ft and a major axis of 6.3 ft. The RSW is designed to pass up to about 22,000 cfs over an uncontrolled crest at normal maximum operating pool elevation 268 ft; therefore, the ratio of the maximum operation head (H_e) to H_d is 1.0. The RSW is designed to operate with free flow over the RSW (no gate control). The RSW piers are integral with the RSW crest and the piers extend upstream of the existing spillway pier nose.

Six RSW alternatives having a standard ogee crest at elevation 245.5 ft (35.5 ft above the existing spillway crest) and a design head of 22.5 ft were initially designed on a conceptual basis for the Design Documentation Report (DDR) study; however, only four of the alternative designs were tested in the model. A seventh design alternative was developed during the model study. The four alternatives tested are shown in Figures 1-4 to 1-7 and main features of each are summarized in Table 1.2. Three of the alternatives (Nos. 2, 4, and 7) tested terminated about 1 ft upstream from the spillway gate seat with a concave upward curvature that allowed a tangent intersection with the existing spillway upstream from the radial gate. This geometry simplifies the construction and removal of the RSW, as the entire RSW structure is located upstream of the radial gate. Alternatives 2 and 4 had vertical upstream faces, whereas Alternative 7 had a sloping upstream face with piers that extended further upstream than the other alternatives. The main design feature of Alternative 4 that was different from the other alternatives was that the downstream end terminated with a 2 ft vertical step offset above the intersection of the RSW with the existing spillway, with air being supplied to the step through conduits installed in the piers.

Alternative 5 differed from the other three alternatives in that the terminal concave-upward curvature connection of the RSW and the existing spillway was replaced with a constant-sloped section that connected tangent to the existing spillway about 27 ft downstream from the spillway gate seat. This required construction of the RSW in two sections, with the downstream smaller section (referred to as the tailpiece section) installed under the radial gate, extending from just downstream of the bulkhead slot to approximately 27-ft downstream from the spillway gate seat as shown in Figure 1-6. The location of the tailpiece section makes it's removal relatively difficult and time consuming. There was some concern that, because of the time required to remove the tailpiece, some conditions could exist requiring flow to be passed over the tailpiece with the main RSW section removed. Therefore, two alternative designs of the tailpiece section (Figure 1-6) operating with the main RSW section removed were evaluated in the model.

The model tests demonstrated that the hydraulic performance of Alternative 5 was superior to the performance of the other alternatives. Alternative 5 was, therefore, recommended as the final design.

1.4 Design Objectives

1.4.1 Skeleton Bay Surface Bypass

The objective of the SBSB is to improve downstream juvenile fish passage by creating a high discharge surface flow outlet from the reservoir to safely pass juvenile fish downstream. The SBSB, shown in Figure 1-8, is a new spillway structure that would be constructed above two of the four existing skeleton powerhouse bays located between Spillway Bay 20 and the existing powerhouse units. Each skeleton bay is 90 ft wide and a 13-ft wide pier separates the individual skeleton bays. Each SBSB structure consists of three 21-ft wide channels separated by 6.9-ft wide piers. The crest of the SBSB is a broad-crested weir at elevation 242.45-ft that transitions to a 1 horizontal to 0.6 vertical sloping chute. A 49.2-ft radius vertical curve at the downstream end of the chute connects the chute to a horizontal deflector at elevation 157.2-ft. The deflector directs the flow horizontally into the tailrace. The SBSB design was previously tested in a 1:40 scale model at the USACE Engineering Research Development Center (ERDC), formerly known as the Waterways Experiment Station, at Vicksburg, MS.

1.4.2 Removable Spillway Weir

The objective of the RSW is to develop an alternative concept to the SBSB, combining a reduction of the high costs associated with the SBSB with bypass capabilities similar to those associated with the SBSB. The RSW concept is being evaluated by the USACE to serve as a "proof of concept" for, or as a permanent alternative to, the more expensive powerhouse SBSB. The RSW would therefore need to emulate, or improve upon, the hydraulic flow characteristics exhibited with the SBSB.

1.4.3 Spillway Bay 20 Deflectors

The objective of the spillway deflector is to provide ability to pass relatively large flow rates over a spillway without increasing total dissolved gas concentrations in the river downstream from the spillway. Water passing over spillways becomes highly entrained with air. When this air entrained water plunges deeply into a stilling basin, the receiving water becomes highly saturated with dissolved gases that are detrimental to fish. Deflectors installed on the spillway below normal tailwater elevations direct the spillway flow along the water surface in the stilling basin, thereby reducing the concentration of air near the stilling basin invert. Therefore, the overall level of dissolved gas in the stilling basin discharge is not increased to the levels that would occur without the deflector. Because the RSW would pass large rates of flow over the spillway, a deflector would be installed on the spillway downstream of the RSW to aid in reducing gas supersaturation associated with flow over the RSW.

2.0 MODEL STUDY OBJECTIVES

The initial objectives of the model study were to:

- Modify an existing two-dimensional 1:25 scale section model of the John Day spillway to evaluate various RSW design geometries for establishing a "proof of concept" that the structure will perform at least as well as the SBSB to effectively transport downstream migrants by the dam.
- Evaluate the tested RSW designs using fish-passage criteria that consists of attraction effectiveness. Acceptable fish-passage attraction effectiveness is demonstrated by approach velocity distribution, spillway passage characteristics that minimize potential fish injury (particularly at the intersection of the RSW and the existing spillway), and the performance of a spillway deflector downstream from the RSW to establish desirable flow conditions in the stilling basin.
- Assure that any RSW geometry developed to satisfy fish passage also results in hydraulic performance that is compatible with basic hydraulic design criteria. These criteria include considerations such as flow separation from the boundaries, pressure, cavitation potential, velocity and standing wave patterns that cause excessive splash and turbulence.
- Develop discharge-rating curves for the final design RSW.
- Assess qualitatively the operational characteristics of the tainter gate during gate opening and closing with the RSW in place and operating.
- Host model visitations designed to assist USACE and resource agency personnel in conjunction with the consultants to arrive at mutually developed geometries that satisfy the stated performance criteria. These visits complement separate working trips to test identical geometries on a three-dimensional 1:80 scale comprehensive model of John Day Dam at the USACE ERDC.

Additional objectives outlined in Modification Case 3 (December 21, 2000) were to:

- Modify the section model to simulate flows from the powerhouse that are entrained in the flow over the RSW exiting from the spillway deflector. Measurements of powerhouse flow that was entrained into the jet exiting from the spillway deflector were made in the 1:80 comprehensive model at ERDC to confirm the necessity of simulating those conditions in the sectional model and to determine the amount of entrained flow to be included in the section model.

- Fully document the performance of RSW Alternative 5, including discharge capacity, pressures, and water surface elevation at various locations over and downstream from the RSW.
- Conduct preliminary testing of various deflectors on the RSW Spillway Bay 20 to determine the optimum deflector geometry (length, transition radius, elevation) considering the typical operating tailwater range.
- Develop deflector performance curves for the selected final deflector geometry.
- Qualitatively document the stilling basin energy dissipation characteristics of the RSW operating with no deflector.
- Participate in the testing of the selected final deflector geometry on the 1:80 scale comprehensive model at ERDC.
- Construct and test a two-dimensional 1:25 scale sectional model of the SBSB in a separate flume with qualitative documentation of tailrace hydraulic conditions and water surface elevations at various locations on the SBSB.
- Document pressures on, and develop discharge rating data for, the tailpiece section of the final design (Alternative 5) RSW.

Modification Case No. 0006 (May 13, 2001) added:

- Documentation of the energy dissipation characteristics in the stilling basin with the deflector installed at spillway discharges up to the spillway design flood of 112,500 cfs/bay. Deflector lengths from 30 ft up to 105 ft were to be considered for evaluation.

3.0 MODELS

3.1 Scales

Four separate 1:25 scale models, installed in flumes from previous studies, were used. The model used for development of the RSW and Spillway Bay 20 deflector tests used an existing flume which contained a model of one full central spillway bay plus two half bays of the John Day Dam spillway from previous studies. The second flume, formerly used for spillway deflector tests on a section model of McNary Dam, had a higher flow capacity than the John Day flume, and was used to test the tailpiece section of the Alternative 5 RSW, the energy dissipation characteristics of the final design deflector with high flows, and the tailrace hydraulic conditions with the SBSB. The 1:25 scale results in the following scaling ratios applicable to all four models:

Parameter	Scaling Relationship	Value
Length	L_r	1:25
Velocity	$L_r^{1/2}$	1:5
Discharge	$L_r^{5/2}$	1:3,125
Unit Discharge	$L_r^{3/2}$	1:125
Pressure	$L_r^{1/2}$	1:5

3.2 Model Controls and Instrumentation

The basic model, Model #1, shown in Figures 3-1 and 3-2 and Photo Plate 3-1, was constructed in a 5 ft wide, 6 ft high, 50 ft long flume that simulated 1,250 ft of river length. The flume walls in the vicinity of the spillway were fabricated from transparent acrylic plastic, allowing visualization of the flow patterns in the stilling basin. The spillway was constructed from plastic and the piers from marine plywood. Lateral inflow, simulating powerhouse discharges entrained into the jet exiting from the RSW Spillway Bay 20 deflector, was provided along the length of the stilling basin. The amount of powerhouse entrainment flow was 4,000 cfs as determined from measurements made in the 1:80 scale comprehensive model at ERDC on 5-8 December 2000. Detailed results from these tests are described in the memorandum "General Trip Report" dated December 11, 2000 included in Appendix B.

Centrifugal pumps circulated the flow to the two model head boxes, one for upstream approach flow, the other for downstream powerhouse lateral flow. The flow to both head boxes was controlled with valves, and measured with orifice meters located at the downstream ends of long straight sections of pipe. The pressure head differentials across the orifice meters were measured with air-water manometers for the lower flows, and mercury-water manometers for the higher flows. Adjustable vanes were installed at the exit of the lateral flow head box to control the direction of lateral flow into the tailrace.

Forebay and stilling basin water levels were measured using stilling wells connected to flush mounted pressure taps installed in the walls of the flume. Point gages were used to measure the water levels in the stilling wells.

Pressures were measured at various locations on the models using manometer tubes connected to flush mounted pressure taps for time-averaged pressure conditions and electronic pressure transducers for time-variable pressures.

The SBSB and high flow deflector models, shown in Photo Plates 3-2 and 3-3, were constructed in a 4.8 ft wide, 8 ft high, 60 ft long flume (Figure 3-3) that simulated 1,500 ft of river length. Flow was circulated through the model using three low-head propeller pumps fitted with variable speed controllers. The flow rate was measured with Venturi meters located at the downstream ends of long straight rectangular sections receiving flow from the pumps. The pressure head differentials across the Venturi meters were measured with manometers.

3.3 Model Accuracy

Hydrodynamic forces are accurately simulated in the 1:25 scale models. Friction forces are small compared with the dominant gravity forces. Therefore, velocities over and on the spillway and entering the model stilling basin are scaled closely in accordance with the Froude criterion.

Air that is typically entrained on a prototype spillway face is not accurately reproduced in the model due to air scaling affects and the inability to construct to scale the prototype surface roughness in the model. Air that is entrained by the jet as it enters the stilling basin is reproduced in the model. However, air bubble sizes are larger in the model than the scale relationship requires. The concentration and distribution of entrained air near the upstream end of the stilling basin probably scales well for most flow regimes, since the initial entrainment of air is governed mainly by large-scale turbulence, which scales reasonably well, and dominates over the buoyancy effects. The downstream release of the air, however, is determined to a large extent by the rise velocity of the air bubbles, which depends on the bubble size. Because of the relatively large size of the air bubbles in the model, the air will be released relatively more quickly in the model than in the prototype, so that the model air concentration is underpredicted in the downstream reaches of the stilling basin.

Orifices used to measure flow rates in the model were installed in accordance with ASME Power Test Codes. ASME Power Test codes specify that installation of orifices in accordance with the codes will provide discharge measurements to an accuracy within 2%. This level of accuracy is considered sufficient to measure spillway discharge rating data and to classify deflector performance.

4.0 TEST PROGRAM

The test program, outlined in the Statement of Work and Modifications 3 and 6, consisted of preliminary, developmental, and final testing of the RSW and Spillway Bay 20 deflector. The Statement of Work and Modifications have been reproduced in Appendix A. Preliminary and developmental tests were conducted with:

- RSW Alternatives 2, 4, 5, and 7 during three visits by USACE and resource agency personnel in June, July, and August, 2000, to develop the final RSW design geometry to be documented during the final testing program, and
- the adopted final RSW design (Alternative 5) to develop the spillway deflector geometry.

The testing for documentation of the final design of the RSW and Spillway Bay 20 deflector was completed in October 2001, and addressed the objectives in the previous section. Final design documentation tests included comparison of the RSW and SBSB tailrace hydraulic characteristics and evaluation of the energy dissipation characteristics of the final design deflector at very high spillway discharges. A comparison of the energy dissipation characteristics in the stilling basin and downstream tailwater channel were made for the spillway design flood conditions both with and without the 50-ft long deflector installed on the spillway.

5.0 PRELIMINARY TESTS - REMOVABLE SPILLWAY WEIR AND SPILLWAY BAY 20 DEFLECTOR SELECTIONS

5.1 Removable Spillway Weir Selection

The initial RSW DDR No. 53 design study originally developed six alternative RSW designs to the conceptual level. A seventh design was developed during the model study. The Alternative 1 design extended into the reservoir about 80 ft upstream of the axis of the existing spillway and had a mild sloping downstream face similar to that of the SBSB design. The Alternative 3 design had a long, sloping upstream face that extended into the reservoir about 125 ft upstream from the spillway axis. The sloping upstream face was intended to control the rate of increase in velocity approaching the crest. The downstream end of the RSW terminated on, and tangent to, the existing spillway about 1 ft upstream from the spillway gate seat. The Alternative 6 design had a shape similar to that of Alternative 3 but terminated at the downstream end with an offset step about 2-ft above the existing spillway face. The DDR study team concluded that none of the above alternatives warranted sufficient merit to advance to model study.

Preliminary model tests on the remaining four of the seven various RSW alternative designs were considered warranted and were conducted during three visitations to the NHC 1:25 scale sectional model (June, July and August 2000) and one visit (June 2000) to the comprehensive model at ERDC. Trip reports describing observations made during these visitations are reproduced in Appendix B. All tests were conducted with free flow (un-gated) conditions for pool elevations ranging from 262.5 ft to 268 ft, producing discharges ranging from about 14,000 cfs to 21,000 cfs. The overall reservoir operating range is 257 ft to 268 ft, however the normal operating range is 262 ft to 265 ft. Model performance revealed that hydraulic flow characteristics improved with lower pool elevations (i.e., below about 260 ft). Therefore, model investigations were limited to pool elevations at the higher end of the overall operating range. Tailwater elevations ranged from 154 ft to 168 ft. Model results of the alternatives evaluated in the model during the preliminary tests are summarized in the following paragraphs.

Alternative 2 (Figure 1-4) – The Alternative 2 RSW had piers that extended 26.2 ft upstream of the existing spillway pier nose. At the downstream end, it terminated tangent to, and on, the existing spillway about 1 ft upstream from the spillway gates seat. Tests with Alternative 2 revealed the presence of a large (approximately 4 ft) water surface drawdown around the spillway piers (Photo Plate 5-1) that generated a series of five to eight stable longitudinal standing surface waves that propagated over the RSW crest. These waves amplified to an approximate height of 10 to 15 ft as the flow passed through the reverse curve radius transition between the RSW crest and the existing spillway crest, producing large elevated aerated jets (roostertails) as the flow contacted the spillway deflector (Photo Plate 5-2). The roostertails were unstable, oscillating laterally across the width of the deflector and trajecting well downstream, where the highly aerated flow plunged deeply into the basin. Depth across the deflector was very non-uniform with much of the area being very shallow (approximately 6 to 12 inches).

Testing of Alternative 2 in the general model at ERDC (see ERDC Laboratory June 28-30, 2000 Trip Report in Appendix B) also identified the presence of the upstream water surface drawdown at the piers, standing waves, and roostertails as were observed in the sectional model. General hydraulic conditions in the reservoir approach to the RSW and the stilling basin and tailrace were evaluated in the general model with RSW Alternatives 2 and 7 as well as the SBSB. From these tests, a comparison was made of the reservoir approach and tailrace hydraulic conditions with the RSW concepts and the SBSB to evaluate the ability of the RSW to emulate the SBSB conditions.

Alternative 4 (Figure 1-5) - The Alternative 4 geometry is the same as for Alternative 2 except that it's piers extend 21.2 ft upstream of the existing spillway pier nose and it terminates with the 2 ft high offset step above the existing spillway. The intent of the offset step was to develop low pressure beneath the jet that would aerate the flow, and hopefully stabilize and attenuate the standing waves observed with Alternative 2. The tests revealed that the high pressures produced by the jet through the concave shaped curve at the end of the RSW extended to the offset step location and prevented natural aeration at the offset. Therefore, the hydraulic characteristics downstream from the RSW (Photo Plate 5-3) were essentially unchanged from those with the Alternative 2 design.

Alternative 5 (Figure 1-6) - For preliminary testing, the true Alternative 5 RSW design was approximated in the model by modifying the Alternative 2 RSW crest section to produce a straight-line fillet that filled in the reverse curve transition bucket between the Alternative 2 crest and the existing spillway crest. The fillet extended up and downstream from the reverse curve transition bucket sufficiently to approximately form tangent connections with the existing spillway. The RSW piers extend 25.2 ft upstream of the nose of the existing spillway piers. The RSW pier nose shape is a true ellipse having a major axis of 25.2 ft and a minor axis of 6.0 ft. The downstream end of the RSW pier is located 40.5 ft downstream from the pier nose (just upstream from the existing spillway pier bulkhead slot) and is offset 3-inches away from the existing spillway pier for structural purposes. The downstream end of the RSW becomes tangent to the face of the existing spillway about 27 ft downstream from the spillway gate seat at a slope of 1 horizontal to 0.7338 vertical and an elevation of 195 ft. Testing of the simplified Alternative 5 RSW geometry illustrated that elimination of the bucket transition prevented the amplification of the standing waves that had existed with the designs that terminated with the bucket transition. In addition, the attenuated standing waves were much more evenly distributed laterally across the spillway bay and did not form the large roostertails impacting on the deflector (Photo Plates 5-4 and 5-5) even with the large drawdown around the piers (Photo Plate 5-6) that generated the standing waves at the RSW crest. Depths on the deflector were much more stable than with the other designs and varied from 3 to 3.5 ft.

Alternative 7 (Figure 1-7) - The Alternative 7 geometry consisted of an approximate 18.5-ft upstream extension of the pier added to the Alternative 2 model RSW and a sloping upstream approach ramp to the Alternative 7 crest. For simplification in the model, a removable inclined floor was used to simulate the sloping upstream approach to the crest. The purpose of the sloping upstream approach to the crest was to create a controlled increase in velocity from the reservoir to the RSW crest to meet fish passage criteria. This modification was constructed and tested during the laboratory visits of June 20 and 21, 2000, to evaluate the potential for chute flow improvement.

Water surface drawdown around the spillway piers was significantly reduced (to about 1 ft) from the drawdown that occurred with Alternatives 2, 4, and 5. However, numerous stable standing waves still propagated over the RSW crest. Because the pier nose is further upstream than with Alternative 2, the crest disturbance appeared smaller than observed for Alternative 2. However, the waves were substantially amplified as the flow moved through the reverse curve radius transition between the RSW crest and the existing spillway crest, resulting in large roostertails at the spillway deflector similar to those observed with the Alternative 2 design (Photo Plate 5-7). Model performance with and without the inclined floor ramp installed did not have any appreciable affect on overall hydraulic performance.

When the spillway gate was lowered into the flow, surface waves originating under the gate lip were more stable than with the free overflow condition. However, large roostertails still reflected off the deflector and plunged into the stilling basin.

Roostertail formation similar to that observed in the sectional model was also observed in the general model at ERDC. Approach flow conditions in the reservoir were generally less favorable than with Alternative 2 because the RSW crest extended further upstream into the forebay and created more stagnant flow zones on each side of the approach piers.

Since neither the 1:25 section nor 1:80 scale general model of Alternative 7 RSW design revealed any hydraulic improvement of either spillway chute or approach conditions existing with the Alternative 2 RSW design, further detailed testing of Alternative 7 with larger project releases and spillway flows was not continued in the general model. A description of reservoir approach and tailrace hydraulic conditions is described in the ERDC Laboratory June 28-30, 2000 Trip Report in Appendix B.

Selection of RSW Alternative 5 for further Model Testing - The preliminary test program revealed unacceptable standing waves with RSW Alternatives 2, 4, and 7 that initiated at the pier nose and were amplified by the reverse curvature transition at the intersection of the RSW with the existing spillway. These waves were essentially eliminated with the replacement of the reverse curvature with the straight-line transition between the RSW and the existing spillway face provided by Alternative 5. The low-amplitude waves that did exist were much more evenly distributed across the spillway face and the large roostertails produced by Alternatives 2, 4, and 7 were eliminated. For these reasons, the Alternative 5 geometry was adopted as the final design for further developmental testing and performance documentation. The RSW geometry used in the developmental testing phase of the model study was the true geometry as opposed to the simplified, approximate shape used for the preliminary tests.

5.2 Spillway Bay 20 Deflector Development

The purpose of the spillway deflector is to direct the spillway jet along the upper water surface in the stilling basin in order to prevent deep plunging into the basin and to subsequently minimize total gas supersaturation. The size of the deflector, the deflector submergence below the tailwater elevation, and the spillway discharge are the primary factors that determine the hydraulic flow regimes in the stilling basin. Acceptable deflector performance relative to reducing gas supersaturation is characterized by the ability to produce skimming and/or undular flow over a broad range of tailwater elevation and spillway bay discharge conditions. A classification system, based on similar studies conducted at ERDC for the Ice Harbor Dam Spillway, ("Data Report, Ice Harbor Section Study", US Army Engineer Waterways Experiment Station, 1996) was used to develop the deflector performance curves.

In the previous studies of deflectors at ERDC, hydraulic performance was classified into several categories depending upon the action in the stilling basin. Similar categories with only slight modification, used to describe the performance of the "modified Bonneville deflector" ("Data Report, Modified Bonneville Deflector, Bonneville Section Model", US Army Engineer Waterways Experiment Station, 1999) and adopted for the this study, are described below:

a. *Plunging flow* (Figure 5-1) - includes *aerated plunging flow*, which occurs when the underside of the surface jet is vented at the downstream end of the deflector; *unstable aerated plunging flow,* which occurs when the underside venting of the surface is intermittent, and *non-aerated plunging flow,* which occurs when the underside aeration ceases, but there is sufficient downward momentum to still cause flow to plunge off the deflector. This category also includes *oscillating or surging flow,* which is an unstable condition with the flow alternately attempting to ride the surface of the tailwater, but then plunging to the stilling basin floor with tailwater surging over the plunging flow.

b. *Skimming flow or surface jet* (Figure 5-1) occurs when the spillway jet remains along the surface of the tailwater with a relatively flat water surface with no plunging action and little downwelling.

c. *Undulating flow or an undulating surface jet* (Figure 5-2) occurs when the spillway jet exiting the deflector "ramps up" on the downstream water surface forming an undulating surface with standing waves under some conditions.

d. *Hydraulic jump (either surface or submerged)* (Figure 5-2) occurs when a hydraulic roller either forms on the deflector, or with higher tailwater, the spillway jet is inundated on the deflector, resulting in a submerged hydraulic jump that is elevated off the stilling basin floor. This includes an *unstable surface jump*, which occurs when the sloping upstream face of the surface jet attempts to break over into a "surface jump," then retreats and reforms again.

The skimming zone had previously been preferred as the flow regime that minimizes dissolved gas in a stilling basin. The undular and hydraulic jump classifications occurring with the higher submergences are considered superior to the plunging flow regime occurring with low submergence. However, near-field tests at John Day Dam in February 2000 by ERDC ("Total Dissolved Gas Exchange During Spillway Releases at John Day Dam, February 12-19, 2000", dated 18 December 2000) indicated that there is not a distinct difference in the total dissolved gas downstream from John Day between the skimming and undular flow regimes. Therefore, the Spillway Bay 20 final deflector geometry was designed to result in performance in the lower range of the undular flow regime with typical tailwater elevation and spillway discharge conditions occurring during the fish passage period.

Prior to conducting testing of the spillway deflectors, the model was modified to simulate flow entrainment from the adjacent powerhouse tailrace into the jet exiting from the deflector. The amount of powerhouse entrainment flow was 4,000 cfs as determined from measurements made in the 1:80 scale comprehensive model at ERDC on 5-8 December 2000. Detailed results from these tests are described in the memorandum "General Trip Report" dated December 11, 2000 included in Appendix B. Limited sensitivity tests of deflector hydraulic performance with powerhouse entrainment flows increased from 4,000 to 9,000 cfs had little or no affect on the deflector hydraulic performance characteristics.

Deflector design development tests were accomplished cooperatively with the USACE during the model visit of February 26 to March 1, 2001. Inter-changeable combinations of deflector radii of 6.1, 20, 35, and 50 ft and lengths of 25, 30, and 35 ft were tested at pool elevation 264 ft over a range of tailwater elevations varying from about 156 to 176 ft. The deflector submergence (tailwater elevation minus deflector elevation) was identified for which the flow regime transitioned from plunging to skimming, skimming to undular, and undular to hydraulic jump. General flow characteristics on the deflector were also observed with the aid of dye movement to assist in defining the overall dispersal of flow downstream from the deflector. The undular and hydraulic jump regimes appeared to pass the dye downstream more readily as compared to the longer dye retention time associated with the plunging and skimming flow regimes.

The results of preliminary tests conducted during the visit of late February with six deflector lengths and transition radii combinations, all at elevation 150 ft, are listed in Table 5.1. The tested combinations consisted of lengths of: 25 ft with radii of 20 and 35 ft, 30 ft with radii of 20 and 50 ft, 35 ft with radius 35 ft, and 50 ft with radius 50 ft. Based on these tests, the 50 ft radius design provided the best overall characteristics with respect to both fish passage and general hydraulics both on the deflector and in the stilling basin (i.e., meeting the desired flow regime, providing stable flow on the deflector, minimizing wave rideup on deflector pier faces, and minimizing flow impact on the powerhouse deck) and was therefore selected for more detailed evaluation.

Following the late February laboratory visit, three additional deflector geometries, all with an elevation of 148 ft, were tested. At that time, the John Day project operating period being used for design of the deflector was considered to be March through November. The geometries tested were a 30 ft long deflector with radii of 20 (Test P-1) and 50 ft (Test P-2) to compare the effect of the transition radius on the 30 ft deflector length; and a 50 ft long deflector with a 50 ft radius (Test P-3) to compare the 50 and 30 ft lengths with the 50 ft transition radius. The tailwater range used in these tests varied from about 156 to 176 ft. Summary results of the deflector performance tests for the three additional tests are listed in Table 5.1 with detailed results in Tables 5.2 to 5.4 and plotted in Figures 5-3 to 5-5. The 50-ft long deflector with a 50-ft transition radius performed more in the preferred hydraulic regime during the design period than did the other design geometries. The threshold submergence at which the various flow regimes occurred with the 148-ft elevation deflector appeared in general to be somewhat higher than those observed with the 150-ft elevation, as shown in Table 5.1. However, the tailwater elevation for Tests P-1 through P-3 was measured further downstream from the location used during the visit of late February, resulting in higher indicated water levels, and thus higher submergence on the deflector. Therefore, the difference in threshold submergence between the two designs can not necessarily be entirely attributed to the deflector elevation difference.

In general, the deflector performance improved as both the radius and the length increased. The 50 ft radius exhibited much more stable and smoother flow on the deflector than the smaller radii, as shown in Figure 5-6 (Tests P-2 and P-3 versus Test P-1), which plots the water surface cross-section at the downstream end of the deflector. The 20-ft radius produced waves of 5 to 7 ft along the deflector on both pier faces. These waves created exiting roostertail trajectories that impacted in the stilling basin. With the 35-ft radius, the wave amplitudes along the piers at the deflector decreased to about 3 to 4 ft, and with the 50-ft radius, the wave amplitudes decreased further to about 1 to 2 ft. The 30-ft long deflector with a 50-ft radius had a somewhat more stable cross section water surface at the end of the deflector than did the 50-ft long deflector (Figure 5-6, Test P-2 versus Test P-3). However, the 50-ft long deflector had a larger range of submergence producing skimming flow when compared with the 30-ft long deflector (compare Figure 5-5 with Figures 5-3 and 5-4) and performed more in the preferred flow regime than did the other geometries. In addition, the peak elevation of the high amplitude surface fluctuations in the jet exiting from the deflector when operating in the undular regime occurred along the powerhouse deck with the 30-ft long deflector, while the undular wave peak elevation occurred downstream from the powerhouse with the 50-ft long deflector. This difference in hydraulic performance substantially reduced the flow impact on the powerhouse deck.

Deflector lengths greater than 50-ft were not tested because a comparison of construction costs for longer deflectors and the incremental improvement in overall performance between the 30- and 50-ft long deflector did not justify a longer length deflector.

Following selection of the 50-ft long deflector with a transition radius of 50-ft as the preferred configuration to be carried further into detailed design documentation, the USACE revised the fish passage design period to be April through June. The revision of the design period resulted in a 2-ft higher design tailwater range than that previously being considered. In order to maintain the same deflector hydraulic performance flow regime that occurred with the previously tested elevation 148-ft deflector, the deflector would need to be raised by 2 ft. Therefore, the final selected deflector design is 50 ft long with a 50-ft transition radius and is located at elevation 150 ft as shown on Figure 5-7.

5.3 Removable Spillway Weir Tailpiece Section

The initial design philosophy considered that, because the tailpiece section would be time consuming to remove from the spillway, extremely large flood conditions might exist that required the tailpiece section to pass flow without the main section RSW in place. Therefore, tests were accomplished to determine the pressure conditions that would exist on, and downstream from, the tailpiece section in place by itself (i.e., the main RSW section removed) with gated and ungated flow over the tailpiece section at the spillway design flood pool elevation of 276 ft.

Two tailpiece designs, as shown in Figure 1-6, were tested in the model. The initial design was an ogee crest shaped for a head of about 5-ft to meet space limitations. The crest design equation was $Y= 0.127\ X^{1.85}$. The crest elevation was 218.5 ft, therefore the crest was extremely under-designed, having an operating head to design head ratio of almost 10. The second design, initially tested during the model demonstration of February 26 – March 2, 2001 (see Appendix B for test description), consisted of a simplified triangular shape with a crest elevation of 221.3 ft that produced a well-defined upstream separation point.

The preliminary tests conducted during the model demonstration revealed that complete flow separation with subsequent extremely low pressures occurred both with the under-designed ogee shape and the simplified triangular shape. This indicated that the ogee-shaped crest probably would not exhibit any more acceptable hydraulic performance than would a simpler shape that was preferred for structural considerations. Pressure measurements made with the simplified triangular shaped tailpiece section indicated the presence of somewhat higher pressures and a smaller zone of separation with the simplified shape, when compared with the ogee design.

Detailed time-averaged pressures measured on the two tailpiece designs at the locations shown on Figure 5-8 are listed in Table 5.5. Average pressures of minus 17.6 ft were measured for a gate opening of 9.7 ft with the ogee shape. This negative pressure (below atmospheric) exceeds the cavitation condition threshold pressure of minus 15 ft. For un-gated flow, pressures measured in the model were less than minus 34 ft, indicating that prototype pressures would be absolute zero. With the simplified shaped tailpiece section, minimum pressures of minus 16.1 ft and minus 29.6 ft were measured for gated and un-gated conditions, respectively. Because the average pressures on either tailpiece section were so low when operating without the main RSW section in place even with spillway gated control, the design team concluded that operation of the tailpiece section without the main RSW section in place would constitute an unsafe condition.

Following more detailed consideration of the hydrologic response of the Columbia River system during extremely large floods, the design team concluded that more than ample response time would be available to remove the tailpiece section in event of a large flood on the system. The team concluded that the only reasonable event that could possibly create a condition where the entire John Day Dam spillway capacity could be required with very little response time would be an upstream dam break event. The potential for operating the tailpiece section by itself was concluded to be too remote to be considered in the design, and the USACE decided that such operation would not be considered in future operation of the project. Therefore time variable pressures were not measured on the tailpiece in operation by itself.

Cavitation-inducing pressures, on the order of minus 15 ft and lower, existed with the simplified triangular shape with gate openings greater than about 9 ft. However, in general, pressures with the simplified shape were slightly higher (smaller negative pressures) than those existing with the ogee shape, with the boundary pressures above absolute zero, even for the un-gated flow condition. The potential for having to operate the tailpiece section without the main RSW section in place is extremely remote, and even if such operation would be necessary, the pressures existing with the simplified shaped section were somewhat greater than those with the ogee-shaped section. Additionally, the simplified triangular-shaped section has significant advantages over the ogee-shaped section in design of the connection between the main RSW section and the tailpiece section. Therefore, the simplified triangular shape was selected for the final design.

5.4 Skeleton Bay Surface Bypass Performance

The SBSB, as described in Section1.4.1, had previously been model tested by the USACE at a 1:40 scale. Based on those model tests, the USACE had concluded that the approach and tailwater hydraulic conditions with the SBSB had resulted in acceptable fish passage conditions. To be considered an acceptable structure to test the surface bypass concept in the prototype, the RSW would need to emulate the SBSB hydraulic performance. In order to provide a same scale comparison of the hydraulic performance characteristics of the SBSB and the RSW, a 1:25 scale model of the SBSB was constructed and tested concurrently with the 1:25 scale RSW model at the NHC laboratory. The primary point of interest in this comparison was the hydraulic conditions in the tailrace and/or stilling basin area.

Deflector Performance – Table 5.6 lists the SBSB deflector hydraulic performance results from tests conducted with forebay water surface elevations of 257.0, 262.5, and 264.0 ft for a range of tailwater elevations. Curves plotted in Figure 5-9 from the data in Table 5.6 show the extents of the zones defining the hydraulic performance for the SBSB deflector. The skimming flow submergence range varies from 1.5 ft to 6.8 ft for a discharge of 10,500 cfs and from 3 ft to 9.5 ft for a discharge of 18,900 cfs. The range of typical operating conditions, also plotted on Figure 5-9, would fit entirely into the skimming range if the submergence were increased 1 ft by lowering the deflector to elevation 156.2-ft. A lowering of the deflector elevation by about 6-ft would be necessary for the SBSB to operate in the lower range of the undular regime similar to that for which the RSW deflector is designed. The skimming flow range of 5.2 ft was larger for the SBSB than for the RSW (Figure 5-5), which varied from a submergence range of 3 ft at 7,500 cfs to 4.5 ft at 16,000 cfs.

Chute Hydraulic Characteristics – A relatively uniform surface pattern existing on the SBSB chute at all tested flows. Water surface elevations and profiles measured along the center and left wall of the center chute at a pool elevation of 264 ft and a discharge of 18,900 cfs are plotted in Figure 5-10, and shown on Table 5.7, whereas transverse water surface elevation sections are plotted in Figure 5-11. Wave formation was modest with small concentrations of flow delivered to the tailrace (see Section F in Figure 5-11). Velocities were measured at Section B (on the crest as shown in Figure 5-11) to provide an independent check of the SBSB discharge rating relationship determined from the 1:40 scale ERDC model. The measured velocities are plotted in Figure 5-12 for the three tested discharges. The velocities are uniformly distributed across the section as shown on the figures.

6.0 FINAL DESIGN PERFORMANCE

The Alternative 5 RSW design with the simplified triangular-shaped tailpiece section, as shown on Figure 1-6, was selected as the final design. The final design selected for the Spillway Bay 20 deflector as shown on Figure 5-7 is 50 ft long with a 50 ft radius transition to the spillway face and at an elevation of 150 ft.

6.1 Removable Spillway Weir

General Flow Characteristics – Flow conditions over the selected final RSW design geometry are described in Section 5.1. The discharge rating curve for the final design RSW is shown on Figure 6-1.

Water Surface Profiles – Water surface profiles over the final design RSW were measured with pool elevations 257, 262.5, 264 and 268 ft, with corresponding discharges of 7,000, 14,000, 15,500 and 21,000 cfs. The measured water surface profile for the RSW design pool elevation of 268 ft (21,000 cfs) is shown on Figure 6-2 and water surface elevations for pool elevations 257, 264 and 268 ft are shown in Tables 6.1 through 6.3. The 3-inch offset between the downstream end of the RSW pier and the existing pier did not affect hydraulic characteristics on the RSW or existing spillway.

Velocities Over RSW - Mid-depth velocities were measured across the width of the RSW at the intersection of the RSW and the existing spillway downstream from the spillway gate at forebay elevations 264 and 268-ft (discharge of 15,500 and 21,000 cfs). The measured and theoretical velocities at that location are shown in Table 6.4.

Forebay Velocities – Forebay velocities immediately upstream from the un-gated RSW crest were measured for forebay elevations of 257, 262.5 and 264 ft, with corresponding discharges of 7,000, 14,000, and 15,500 cfs. The velocities are plotted in both plan and section views in Figures 6-3 to 6-8. The velocities in general illustrate the expected acceleration toward the crest and increased velocities for higher forebay elevations.

Average Pressures – Time averaged pressures were measured with manometers connecting to two rows of pressure taps on the downstream face of the RSW and existing spillway. The first row was located along the centerline of Spillway Bay 20 and the second row was located at 1 ft offset from the pier separating bays 19 and 20. Data collected and location of the pressure taps are presented in Table 6.5. Pressures near, or below, atmospheric were observed at Taps 12, 13, and 14, on the centerline of the existing spillway immediately downstream from its intersection with the RSW. The lowest pressure of -2.1 ft was measured at Tap 14. In addition, a low pressure of -2.5 ft was measured near the pier at Tap 2. The lowest measured pressure downstream from the pier offset was 0.4 ft. A discontinuity of about 1-inch (prototype) was installed in the model at the intersection of the RSW and existing spillway to assist in developing construction tolerances. With the 1-inch discontinuity, average pressure could be as low as minus 5.3 ft immediately downstream from the discontinuity.

Time Variable Pressures - Pressure transducers were installed to measure the instantaneous time variable pressure regime at the following three positions, as shown in Table 6.6: (1) DT1 near the point of tangency (PT) of the RSW ogee and straight sections, (2) DT2 midway between the PT and the tailpiece section, and (3) DT3 at the connection plate between the RSW and the tailpiece section. Time variable pressures were measured at reservoir elevations 264 ft and 268 ft. The results are presented in Table 6.6. These measurements indicated that the local, instantaneous pressures could be up to about 5-ft lower than the average pressure. The fluctuations were small, with a maximum standard deviation of 1.4 ft measured at Tap DT3. The dominant frequency observed was below 0.1 Hz (prototype) for all tests.

Time Variable Pressures During Radial Gate Closure – Time variable pressures measured immediately upstream from the gate at Tap DT3 during gate closure and opening are plotted in Figure 6-9 for reservoir elevations 264 and 268 ft. The maximum amplitude of 11 ft was measured after the gate had fully closed. This was probably caused by wave action in the relatively confined model head tank, and probably would not occur in the prototype. Maximum pressure fluctuations having amplitudes of approximately 3 ft were measured during gate closure.

Average Pressures on the Tailpiece Section Operating Without the Main RSW Section – The hydraulic performance of the tailpiece section is discussed in paragraph 5.3. Average pressures measured on the tailpiece section are listed in Table 5.5. Because the average pressures on the tailpiece section when operating without the main RSW section in place were so low even with spillway gated control, the design team concluded that operation of the tailpiece section without the main RSW section in place would constitute an unsafe condition. The USACE decided that such operation would not be considered in future operation of the project, therefore time variable pressures were not measured on the tailpiece in operation by itself.

6.2 Spillway Bay 20 Deflector

6.2.1 Design Flow Performance

The final design deflector geometry (Figure 5-7) is 50 ft long and set at elevation 150 ft. A 50-ft radius curved transition connects the deflector to the spillway. Hydraulic conditions on the final design deflector geometry are described in Section 5.2. The deflector hydraulic performance curves developed for the final design geometry, illustrated in Table 6.7 and Figure 6-10, are almost identical to those existing with the same length and radius deflector set at elevation 148 ft (Figure 5-5). The typical range of tailwater and spillway discharge operating conditions during the fish passage season, as shown on Figure 6-10, almost fits entirely in the lower half of the undular flow regime range. There is a slight difference in the zone/line separating plunging and skimming flow regimes between the elevation 148 and 150 ft deflectors. However, the difference can be attributed to the significantly more data collected for the final deflector geometry (elevation 150-ft), especially under lower flow conditions. The flow conditions in the stilling basin and tailrace with the design deflector with a discharge of 14,000 cfs are illustrated on Photo Plates 6-1 and 6-2.

When evaluating performance of the final design deflector, an estimate was made of the relative degree of air entrainment versus depth in the stilling basin. As discussed in Section 3.3, air entrainment can not be accurately simulated in the model. Therefore, air entrainment at various depths in the water column in the stilling basin was estimated based on the amount of air bubbles observed in the model to develop a relative comparison of air concentration in the basin for various spillway discharge and deflector submergence conditions. Air entrainment conditions, plotted in Figures 6-11 and 6-12, were developed from the observed air bubble conditions in the model. Figure 6-11 defines the air entrainment zones for a range of spillway discharge and deflector submergence conditions and illustrates that under typical operating conditions entrained air penetrates to the invert of the stilling basin, El. 114 ft. However, air concentration is not continuously high at depth. Figure 6-12 shows contour depths of air penetration bubbles in the model.

Average pressures were measured on the centerline of the deflector at the following locations: (1) three places on the top surface of the deflector, (2) on the downstream vertical face immediately below the top surface, and (3) under the cantilevered overhang. The maximum measured average pressure on the top surface of the deflector was 70 ft and occurred with a discharge of 112,500 cfs. The measured pressures and measurement location are shown in Table 6.8.

6.2.2 High Flow Performance

The effect of the long deflector on stilling basin energy dissipation was evaluated with large spillway discharge conditions. A secondary objective of the high flow performance tests was to identify the maximum deflector length that could be constructed in spillway bay 20 while still containing the hydraulic jump in the stilling basin. The RSW is designed as a temporary structure that would be removed prior to the high spillway discharges, therefore the RSW was not included on the existing spillway crest for the high flow tests.

Discharges tested included: 33,000, 40,000, 53,000, 88,300, and the spillway design discharge of 112,500 cfs/bay. The tailwater elevations used for these tests were adopted from the original design 1:41.14 scale sectional model study conducted at the USACE North Pacific Division (NPD) Hydraulic Laboratory, Bonneville, Oregon ("Spillway and Stilling Basin, John Day Dam, Columbia River, Oregon and Washington, Technical Report No. 97-1, November 1974"). The tailwater elevations used for each flow represented: 1) the minimum tailwater required to hold the toe of the hydraulic jump at the point of tangency of the existing bucket; 2) the normal tailwater; and 3) 10 ft above normal tailwater. Deflector lengths of 30, 50, 75, and 105 ft and for the existing John Day spillway crest geometry (no deflectors) were tested with each flow and tailwater condition. The existing prototype topography downstream from the stilling basin at spillway bay 20 varies from about elevation 80 to 100 ft in the initial 100 ft downstream from the stilling basin and slopes to elevation 145 ft at a distance of about 2,000 ft downstream from the basin. The floor elevation in the existing flume immediately downstream of the end sill was at elevation 89.0 ft. Therefore, the actual existing bed topography downstream from the end sill was not incorporated into the model.

Initially, qualitative comparisons between the USACE's original design 1:41.14 scale model and the NHC 1:25 scale sectional model were considered to be used as the basis for evaluating stilling basin energy dissipation with the 50-ft long deflector. However, such a comparison was determined to be not reasonable because the USACE NPD model tests were conducted with high flows passing through the interior bays, while the NHC test program focused on spill through the end Spillway Bay 20. Flow characteristics vary significantly between interior and outside bays. There are two primary reasons for the difference in flow patterns. First, training walls and the powerhouse contain the flow in Spillway Bay 20 for a distance of approximately 50 ft downstream of the spillway toe. The boundaries do not allow the jet to interact with the surrounding spill as exists for the interior bays. Second, the tailwater estimated for large discharges approaching the spillway design flood is about 206 ft, which is about 20 ft above the powerhouse tailrace deck. The model indicated that flow depths on the order of 5-10 ft passed over the powerhouse deck into the stilling basin with the spillway design flood condition. A significant amount of the flow passing over the powerhouse tailrace is entrained into the stilling basin jet and has a distinct effect on energy dissipation in the basin. Additionally, the tailrace invert downstream from the stilling basin in the USACE NPD model sloped from elevation 127 ft at the stilling basin end sill to elevation 145 ft about 100 ft downstream from the end sill. The downstream bed elevation in the USACE NPD model was, therefore, significantly higher than that in either the NHC model or the prototype. The difference in topography downstream from the stilling basin is believed to have a significant influence on energy dissipation characteristics downstream from the stilling basin. Therefore, the process used to evaluate the effect of the long deflector on energy dissipation was revised to qualitatively characterize, and compare, the amount of energy dissipation occurring in the stilling basin and downstream tailrace with no deflector and with the 50-ft long deflector in the 1:25 scale NHC sectional model.

Flow patterns in the stilling basin under gated flow conditions of 33,000, 40,000, and 53,000 cfs/bay, remained relatively unaffected by the length of the deflector. The plunging and hydraulic jump flow regimes (Figures 5-1 and 5-2) did not develop, as the hydraulic performance varied within skimming and undular flow regimes with discharges up to 53,000 cfs per bay. For these lower discharges, the skimming and undular flow remained in the upper section of the water column while a relatively slow moving return flow defined the lower portion of the water column.

With gated flow conditions the jet exiting from the deflector remained in the upper section of the water column, and did not interact directly with the stilling basin floor. Photo Plates 6-3 and 6-4 illustrate flow conditions in the stilling basin and tailrace with 30 and 75-ft long deflectors at a discharge of 40,000 cfs and various tailwater elevations.

Free flow (ungated) high flow test were also conducted with the spillway design flood discharge of 112,500 cfs/bay both with the existing (no deflector) and with a 30 ft long and a 50 ft long deflector installed configurations. General flow characteristics for the existing spillway geometry (no deflector) matched well with the previous USACE model study. A tailwater elevation between 200 and 205 ft was required to hold the toe of the hydraulic jump at the point of tangency between the spillway crest and stilling basin. The introduction of the extended deflectors significantly changed the hydraulic characteristics in and downstream from the stilling basin. Photo Plates 6-5 and 6-6 illustrate the spillway design flood flow conditions in the stilling basin and tailrace with the existing (no deflector) and with both a 30 ft and 50 ft long extended deflector in place. Under the existing (no deflector) condition, flow plunged to the floor of the basin and a large counter-clockwise roller existed in the basin. A large upward flow component exited from the end sill into the downstream channel. This high-energy upward flow created large surface waves and turbulence in the channel well downstream from the end of the stilling basin. With the 50-ft long deflector installed, the high velocity flow tended to remain in the upper depths of the stilling basin and subsequently created a very rough surface condition in the basin. A clockwise roller, not as strong as the roller that occurred with the no deflector configuration, existed in the stilling basin. However, as the flow exited over the basin end sill, the surface conditions downstream in the channel were significantly less turbulent than existed without the deflector. Surface waves in the channel downstream from the stilling basin had amplitudes of about 20-ft without the deflector, but were reduced to less than about 10-ft with the 50-ft long deflector in place.

Maximum and mean velocities were measured at the spillway design flood discharge (112,500 cfs per bay) at various depths at points located 100, 200 and 300-ft downstream from the end sill both with and without the deflector installed. Due to the highly turbulent flow conditions in the model, velocity measurements could not be obtained in the immediate 100-ft reach downstream from the end sill. The measured velocities are shown in Table 6.9 and indicate that velocities in the channel downstream from the stilling basin throughout the water column are quite similar both with and without the 50-ft long deflector in place.

Movement of sediment on the apron and in the scoured region downstream of the end sill was also observed during the high flow tests. Gravel, approximately 1-ft diameter prototype, was placed onto the stilling basin and into the scour hole prior to testing. Sediment introduced to the stilling basin remained on the apron, swirling in counter-clockwise direction approximately 25 ft downstream of the deflector. Significant gravel movement was not observed in the scour hole area downstream of the end sill. The sediment movement was relatively consistent for all flow conditions and deflector lengths. High flows over the existing spillway geometry (no deflectors) wash all sediment out of the stilling basin and into the tailrace. Sediment deposited in the tailrace exhibited only slight movement.

Testing in the sectional model indicates that stilling basin energy dissipation should not be significantly affected by addition of the 50-ft long deflector in Spillway Bay 20. However, being a two-dimensional model, the sectional model is unable to simulate the three-dimensional flow conditions that will exist in the prototype. Therefore, the three-dimensional general model at ERDC should be used to further evaluate overall energy dissipation characteristics of the 50-ft long deflector under extreme spillway discharge conditions.

7.0 SUMMARY

7.1 Preliminary Tests with Removable Spillway Weir Alternatives

Preliminary tests on four alternative RSW designs were conducted for pool elevations ranging from 262.5 to 268 ft, producing discharges from 14,000 to 21,000 cfs. Tailwater elevations ranged from 154 to 168 ft.

- The Alternative 2 design resulted in longitudinal standing waves generated by the large surface drawdown at the pier nose. These longitudinal standing waves were amplified to approximate heights of 10 to 15 ft as flow passed through the reverse curvature transition between the downstream end of the RSW and the existing spillway face and produced large unstable roostertails when the flow impinged on the horizontal deflector. Depth across the deflector was very non-uniform.
- The Alternative 4 design, with the addition of a 2-ft offset step at the intersection of the RSW and the existing spillway to aerate the flow, was ineffective in attenuating the longitudinal standing waves. This was due to the high pressure at the offset that resulted from the reverse curvature transition that prevented air from entering the flow at the offset.
- The Alternative 5 design, which replaced the reverse curvature transition at the downstream end of the Alternative 2 and 4 designs with a straight-line tangency between the RSW and the existing spillway, prevented the amplification of the longitudinal standing waves. Subsequently, the large roostertails that occurred with Alternatives 2 and 4 were eliminated with the Alternative 5 design. The standing waves that did remain were much more evenly distributed across the spillway for Alternative 5 than with the other geometries and subsequently resulted in a much more stable and uniform water surface on the deflector.
- The Alternative 7 design, consisting of an upstream pier extension added to Alternative 2 in an attempt to control the original source of the standing waves, reduced the drawdown at the pier nose, but did not eliminate the standing wave attenuation and roostertails caused by the reverse curvature transition at the downstream end of the RSW.

- The amplification of the longitudinal standing waves initiating at the pier noses generated by the reverse curvature transition between the RSW and the spillway and the resulting roostertails that had occurred with the Alternative 2, 4, and 7 designs were considered unacceptable. Therefore, the straight-line tangent design of Alternative 5 was adopted for the final design and documentation.

7.2 Spillway Bay 20 Deflector Development

Various spillway deflector designs on Spillway Bay 20 with the final design RSW in place were tested to optimize the hydraulic performance relative to downstream gas supersaturation. These tests included flow entrainment from the powerhouse into the jet exiting into the stilling basin from the deflector. An entrainment flow of 4,000 cfs was used based on estimates from earlier tests with the 1:80 scale model at ERDC. Interchangeable combinations of deflector lengths and deflector/spillway face transition radii varying from 20 to 50 ft were evaluated. The major results from these tests are:

- Flow characteristics on the deflector improved with increasing transition radius. For example, increasing the radius from 20 ft to 35 ft reduced the wave heights on the deflector surface along the pier faces from a range of 5-7 ft to a range of 3-4 ft, respectively. A further increase in radius to 50 ft reduced the wave heights along the pier faces even further to between about 1-2 ft.
- The longer deflector length of 50 ft improved hydraulic performance when compared with the shorter lengths by both expanding the range of skimming flow, and moving the onset of undulating flow further downstream so that the undular flow jet did not impinge on the powerhouse deck.

7.3 Removable Spillway Weir Tailpiece Section

The RSW tailpiece section is a separate portion of the RSW required with the final design (Alternative 5) to accomplish a straight-line tangent transition between the RSW and the spillway. The tailpiece section extends under and downstream of the existing radial gate. Two tailpiece shapes were tested under both gated and un-gated conditions with the main RSW crest removed.

- The initially tested geometry, an extremely under-designed ogee shape, produced complete flow separation of flow over the tailpiece and extremely low pressures downstream from the ogee crest. Average pressures with the ogee shape were as low as minus (below atmospheric) 17.6 ft for a gate opening of 9.3 ft at pool elevations approaching the spillway design flood condition of 276 ft. For un-gated flows at those pool elevations, measured pressures in the model indicated that prototype pressures would be at absolute zero.
- The second shape, a simplified triangular geometry preferred for structural considerations, had somewhat higher pressures and a smaller zone of separation when compared with the ogee design. With the triangular shape, pressures lower than about minus 15 ft were measured at gate openings greater than about 9 ft with pool elevation of approximately 276 ft. The minimum measured pressure with an un-gated condition and those pool elevations was minus 29.6 ft.
- Pressures with both the simplified and ogee shaped tailpiece sections were extremely low. Operation without the main RSW and either tailpiece section in place would result in cavitation damage to the tailpiece as well as the existing spillway. The simplified shape had a significant structural benefit in that it reduced the length of the connection between the main RSW and the tailpiece section and, subsequently, reduced the hydraulic loading considerations required for design of the connection. Therefore, the simplified triangular shape was selected for the final design.

7.4 Skeleton Bay Surface Bypass Performance

Tests were conducted with the SBSB concept to compare its hydraulic performance with that of the RSW.

- Water surface patterns on the chute were relatively uniform.
- Skimming flows ranged over a submergence depth range of 5.5 to 6.5 ft. That was greater than observed for the RSW (Figure 6-10), which had a skimming zone ranging over a depth of 4 to 5 ft.
- The elevation of the deflector at the downstream end of the SBSB would need to be lowered significantly from it's present design elevation (157.2 ft) to operate in the same flow regime as the RSW for the design tailwater elevation range.

7.5 Final Design Removable Spillway Weir and Spillway Bay 20 Deflector Tests

After selection of the Alternative 5 RSW design, with the 50 ft long, 50-ft radius deflector at elevation 150 ft, as the final design geometry, final documentation tests involving RSW approach velocities and pressures and deflector hydraulic performance at high flows were conducted.

- Approach velocities measured in the forebay immediately upstream from the crest of the RSW illustrated the expected acceleration as flow approached the crest, and increased velocities for higher forebay elevations.
- Average pressures on the RSW and existing spillway (including downstream from the pier offset) at design pool elevation 268-ft were above, or only slightly below, atmospheric (lowest pressure of -2.5 ft).
- Time-variable pressures measured at selected locations on the RSW indicated the existence of small fluctuations, with local, instantaneous pressure as much as 5 ft lower than the average pressure and with a maximum standard deviation of 1.4 ft at various locations.
- Time-variable pressure fluctuations measured on the RSW during gate closure had amplitudes of approximately 3 ft.
- A 50-ft long deflector placed in Spillway Bay 20 at elevation 150 ft with a 50-ft radius transition between the deflector and the spillway face was satisfactory in producing the desired flow conditions in the stilling basin with the typical fish passage season operating conditions at John Day Dam.
- The 50-ft long deflector did not allow the formation of a positive hydraulic jump in the stilling basin at high flows resulting in a high velocity surface jet traveling along the surface of the stilling basin and a strong back roller under the jet. This condition is similar to that of a negative step hydraulic jump. However, the 50-ft long deflector has very little affect on the velocities throughout the water column in the channel beginning about 100-ft downstream from the stilling basin as compared to the existing (no deflector) condition. Therefore, the energy dissipation in the stilling basin and the 100-ft length of tailrace channel immediately downstream of the stilling basin is considered to be essentially the same with the 50-ft long deflector as without the deflector.

- Neither decreasing the length of the deflector from 50 to 30 ft, nor increasing the length beyond 50 ft, significantly improves or affects flow patterns or energy dissipation in the stilling basin. Previous model studies indicated that flow deflectors 12 ft long act as roughness elements under high flow conditions and have little affect on the energy dissipation characteristics in the stilling basin. This study found that deflector lengths 30-ft and longer will effectively turn spillway flow, creating a high-energy jet which extends out into the tailrace downstream from the stilling basin.
- Sediment remains in the stilling basin during high flow conditions with the long deflectors increasing the potential for apron damage due to scour. Under existing (no deflector) conditions sediment present on the apron prior to high flow events would be washed downstream into the tailrace.

TABLE 1.1
John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

SUMMARY of MODEL VISITS

Date	Participants: United States Army Corps of Engineers										NMFS		Montgomery Watson	CRITFC	ODFW	nhc Seattle			nhc Vancouver	
	D. Modini	B. Bird	B. Ebberts	M. Hanson	C. Goodell	K. McCune	D. Maggio	D. Wilson	M. Schneider	B. Buchholz	G. Fredericks	S. Rainey	D. Dorratcague	T. Lorz	C. Tracy	E. Zapel	R. Regan	J. Lencioni	A. Babb	K. Christison
June 20-21, 2000	✔	✔	✔	✔									✔			✔	✔	✔	✔	✔
June 28-30, 2000[1]	✔		✔					✔								✔				
July 19-20, 2000	✔	✔	✔													✔	✔		✔	✔
August 7-8, 2000	✔		✔	✔							✔	✔	✔	✔	✔			✔	✔	✔
December 5-8, 2000[1]	✔		✔		✔		✔	✔								✔		✔		✔
February 26 - March 2, 2001			✔		✔				✔				✔				✔	✔	✔	✔
April 2-6, 2001																				
May 14-18, 2000[1]			✔		✔	✔	✔	✔	✔		✔	✔		✔						✔
June 13, 2001													✔				✔	✔	✔	✔
June 19, 2001						✔												✔	✔	✔
October 12, 2001						✔				✔							✔	✔	✔	✔

Note: 1) Trips to USACE facilities in Vicksburg, Mississippi

TABLE 1.2

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

SUMMARY of TESTED RSW ALTERNATIVES

Alternative	U/S Face	U/S Pier Extension[1]	Vertical Step[2]	Crest Elevation	Tangency Location[3]	Model	
		ft	ft	ft	ft	1:25 Section	1:80 Comprehensive @ EDRC
2	Vertical	46.2	0.0	245.5	-1.0	✔	✔
4	Vertical	41.0	2.0	245.5	-1.0	✔	
5	Vertical	45.9	0.0	245.5	26.4	✔	
7	Sloped (1:2.784)	65.2	0.0	245.5	-1.0	✔	✔

Notes: 1) Distance upstream of spillway crest.
2) Vertical step located at joint between RSW and existing spillway crest.
3) Location of the RSW/existing spillway joint relative to the exisiting gate seat. (- upstream)

TABLE 5.1

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

SUMMARY of DEFLECTOR PERFORMANCE TESTING

Test	Flow Deflector Geometry Deflector Elevation ft	 Deflector Length ft	 Transition Radius ft	Submergence ft	Flow Condition
Initial Deflector Geometries Tested				8.0	Plunging/Skimming
	150	30	50	10.0	Skimming/Undular
				28.0	Undular/Hydraulic Jump
				9.0	Plunging/Skimming
	150	30	20	12.0	Skimming/Undular
				26.0	Undular/Hydraulic Jump
				9.0	Plunging/Skimming
	150	25	20	12.0	Skimming/Undular
				27.0	Undular/Hydraulic Jump
				9.0	Plunging/Skimming
	150	25	35	12.0	Skimming/Undular
				28.0	Undular/Hydraulic Jump
				9.0	Plunging/Skimming
	150	35	35	12.0	Skimming/Undular
				27.0	Undular/Hydraulic Jump
				8.0	Plunging/Skimming
	150	30	50	13.0	Skimming/Undular
				28.0	Undular/Hydraulic Jump
				8.0	Plunging/Skimming
	150	50	50	12.0	Skimming/Undular
				27.0	Undular/Hydraulic Jump
				9.5	Plunging/Skimming
P-1	148	30	20	13.5	Skimming/Undular
				27.5	Undular/Hydraulic Jump
				10.0	Plunging/Skimming
P-2	148	30	50	12.5	Skimming/Undular
				28.3	Undular/Hydraulic Jump
				8.0	Plunging/Skimming
P-3	148	50	50	12.5	Skimming/Undular
				26.0	Undular/Hydraulic Jump
				8.3	Plunging/Skimming
Final Geometry	150	50	50	12.8	Skimming/Undular
				26.0	Undular/Hydraulic Jump
				3.0	Plunging/Skimming
SBSB	157.2	34.1	49.2	9.6	Skimming/Undular
				17.3	Undular/Hydraulic Jump

Note: Forebay water surface elevation remained constant at 264 ft

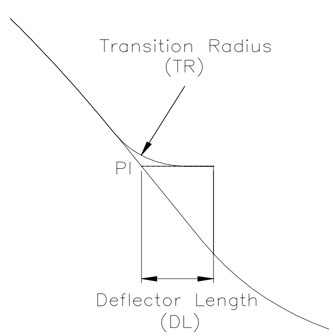


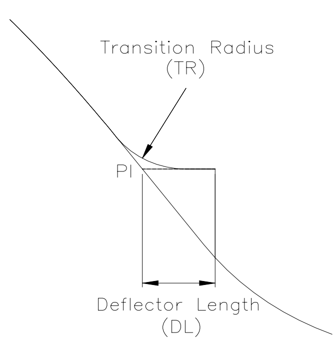
TABLE 5.2

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

DEFLECTOR PERFORMANCE for TEST P-1
Deflector Length 30.0 ft, Transition Radius 20 ft
Deflector Elevation 148.0 ft

Forebay WSE	Discharge	Tailwater	Submergence[2]	Performance Classification[3]
ft	cfs/bay	ft	ft	Bay 20 (full bay)
257.0	7,000	153.5	5.5	Skimming
		154.5	6.5	Skimming
		155.0	7.0	Undular/Skimming
		156.0	8.0	Undular
		160.3	12.3	Undular
		165.5	17.5	Undular/Hydraulic Jump
		166.5	18.5	Hydraulic Jump/Undular
		170.0	22.0	Hydraulic Jump
262.5	14,000	155.5	7.5	Plunging
		156.5	8.5	Plunging/Skimming
		157.5	9.5	Skimming
		160.0	12.0	Skimming/Undular
		167.0	19.0	Undular
		171.5	23.5	Undular
		173.0	25.0	Undular/Hydraulic Jump
		177.0	29.0	Hydraulic Jump
264.0	15,500	157.0	9.0	Plunging
		157.5	9.5	Skimming/Plunging
		159.0	11.0	Skimming
		160.0	12.0	Skimming
		161.0	13.0	Skimming
		162.0	14.0	Undular
		168.0	20.0	Undular
		175.5	27.5	Hydraulic Jump/Undular
		184.5	36.5	Hydraulic Jump
268.0	21,000	159.0	11.0	Plunging
		161.0	13.0	Plunging
		162.0	14.0	Plunging/Skimming
		162.5	14.5	Skimming/Plunging
		164.0	16.0	Skimming
		165.0	17.0	Undular
		172.5	24.5	Undular
		178.5	30.5	Undular/Hydraulic Jump
		180.5	32.5	Hydraulic Jump
		184.0	36.0	Hydraulic Jump

Notes:
1) Tailwater measured 650 ft downstream of deflector
2) Submergence = Tailwater Elev. - Deflector Elev. (148 ft)
3) Flow characteristics considered as either plunging, skimming, undular, or hydraulic jump



Tables 5-1 thru 5-4, 5-6, 6-7 1004 -015.xls
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TABLE 5.3

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

DEFLECTOR PERFORMANCE for TEST P-2
Deflector Length 30.0 ft, Transition Radius 50 ft
Deflector Elevation 148.0 ft

Forebay WSE	Discharge	Tailwater	Submergence[2]	Performance Classification[3]
ft	cfs/bay	ft	ft	Bay 20 (full bay)
257.0	7,000	153.5	5.5	Skimming
		154.5	6.5	Skimming
		155.3	7.3	Undular/Skimming
		158.0	10.0	Undular
		163.0	15.0	Undular
		165.5	17.5	Undular/Hydraulic Jump
		167.0	19.0	Hydraulic Jump
		170.5	22.5	Hydraulic Jump
262.5	14,000	156.0	8.0	Plunging
		157.0	9.0	Skimming
		159.0	11.0	Skimming
		160.5	12.5	Undular
		167.0	19.0	Undular
		173.5	25.5	Undular/Hydraulic Jump
		174.0	26.0	Hydraulic Jump/Undular
		175.0	27.0	Hydraulic Jump
264.0	15,500	156.5	8.5	Plunging
		158.0	10.0	Skimming/Plunging
		159.0	11.0	Skimming
		161.0	13.0	Undular/Skimming
		161.5	13.5	Undular
		168.5	20.5	Undular
		175.0	27.0	Undular
		176.0	28.0	Undular
		176.5	28.5	Hydraulic Jump
		179.5	31.5	Hydraulic Jump

Notes: 1) Tailwater measured 650 ft downstream of deflector
2) Submergence = Tailwater Elev. - Deflector Elev. (148 ft)
3) Flow characteristics considered as either plunging, skimming, undular, or hydraulic jump

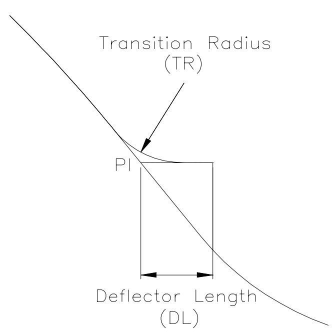


TABLE 5.4

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

DEFLECTOR PERFORMANCE for TEST P-3
Deflector Length 50.0 ft, Transition Radius 50 ft
Deflector Elevation 148.0 ft

Forebay WSE	Discharge	Tailwater	Submergence[2]	Performance Classification[3]
ft	cfs/bay	ft	ft	Bay 20 (full bay)
257.0	7,000	153.0	5.0	Skimming
		154.5	6.5	Skimming/Undular
		155.0	7.0	Undular
		159.0	11.0	Undular
		164.5	16.5	Undular/Hydraulic Jump
		165.3	17.3	Hydraulic Jump/Undular
		166.0	18.0	Hydraulic Jump
262.5	14,000	154.5	6.5	Plunging
		155.0	7.0	Skimming
		157.0	9.0	Skimming
		158.5	10.5	Skimming
		159.5	11.5	Undular
		164.0	16.0	Undular
		172.0	24.0	Undular/Hydraulic Jump
		173.0	25.0	Hydraulic Jump
264.0	15,500	155.5	7.5	Plunging
		156.3	8.3	Skimming/Plunging
		157.5	9.5	Skimming
		160.0	12.0	Skimming
		161.0	13.0	Undular
		167.0	19.0	Undular
		174.0	26.0	Undular/Hydraulic Jump
		175.0	27.0	Hydraulic Jump
		179.0	31.0	Hydraulic Jump

Notes: 1) Tailwater measured 650 ft downstream of deflector
2) Submergence = Tailwater Elev. - Deflector Elev. (148 ft)
3) Flow characteristics considered as either plunging, skimming, undular, or hydraulic jump

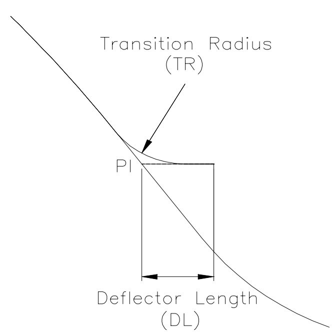


TABLE 5.5
John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

TAILPIECE TIME-AVERAGED PRESSURE SUMMARY

Tap Elevation Ogee Shaped Crest		Pressure Head (ft)													
		Gate Opening (Go) =6.1		Gate Opening (Go) = 7.7		Gate Opening (Go) = 8.9		Gate Opening (Go) = 9.3		Gate Opening (Go) = 9.7		Gate Opening (Go) = 10.8		Ungated Flow	
		Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier
Position 1	210.0	54.8		56.9		56.1		56.5		56.5		56.1		60.2	
Position 2	214.8	51.4		51.6		50.8		50.8		50.8		49.2		52.9	
Position 3	218.4	8.3	6.6	-4.4	-5.7	-11.8	-12.3	-13.4	-14.7	-17.1	-18.0	-26.1	-27.0	below -75	below -75
Position 4	218.1	1.2	0.5	-8.2	-9.0	-8.2	-9.0	-14.8	-15.5	-16.8	-17.6	-23.4	-24.1	-57.9	-61.1
Position 5	216.8	-1.6	-0.9	-4.0	-3.8	-4.9	-5.4	-5.3	-5.9	-6.5	-7.9	-8.6	-8.3	-59.8	-59.6
Position 6	214.8	0.1	1.7	-0.4	1.3	-0.2	1.5	0.1	1.7	-0.2	1.5	-0.8	0.9	-29.1	-27.4
Position 7	211.7	0.6	2.3	1.4	3.1	1.9	3.5	1.9	3.5	2.3	3.9	2.5	4.1	-11.3	-9.6
Position 8	204.4	5.5	7.4	5.9	7.8	6.7	8.6	6.7	8.6	7.1	9.0	7.5	9.4	6.7	8.6
Position 9	197.4	3.3		5.5		5.5		6.4		6.0		6.4		17.9	
Position 10	188.3	4.4		5.2		5.2		6.0		5.6		6.0		26.5	
Position 11	178.0	4.4		5.2		5.6		6.4		6.9		6.9		not recorded	

Tap Elevation Sharp Crested Tailpiece		Pressure Head (ft)													
		Gate Opening (Go) = 5.6		Gate Opening (Go) = 8.0		Gate Opening (Go) = 9.6		Gate Opening (Go) = 10.9		Gate Opening (Go) = 11.6		Gate Opening (Go) = 12.7		Ungated Flow	
		Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier	Bay Centerline	Near the Pier
Position 1	210.0	58.1		57.7		57.3		56.9		56.9		56.9		62.6	
Position 2	214.8	53.1		52.9		52.9		52.5		52.1		52.1		57.0	
Position 3	219.8	5.3	4.0	-11.1	-10.7	-15.2	-16.1	-14.4	-14.8	-14.8	-13.6	-15.2	-15.6	-25.5	-29.6
Position 4	218.4	21.8		-11.0		-15.1		-14.3		-14.7		-15.1		-24.5	
Position 5															
Position 6	214.8	11.5	11.5	12.3	13.1	10.7	13.1	-4.5	-2.4	-9.8	-7.4	-12.1	-14.7	-20.5	-18.8
Position 7	211.7	-1.9	-3.5	3.1	0.6	10.4	7.2	12.5	3.9	7.6	2.2	6.7	8.0	-18.7	-16.6
Position 8	204.4	3.2	2.4	4.6	4.0	6.3	4.9	9.5	6.5	12.0	12.2	12.4	15.5	-13.4	-13.2
Position 9	197.4	-1.9		-0.6		0.2		1.4		2.2		3.1		1.0	
Position 10	188.3	0.3		1.1		1.5		1.9		2.3		2.7		24.9	
Position 11	178.0	0.3		1.1		2.8		3.2		3.6		4.0		33.9	

Notes: 1) Pressures presented are relative to elevation of the pressure tap
2) Pressures below -33 ft indicate sub-zero absolute prototype pressure. In reality, severe cavitation would occur and the flow regime would be different than tested in the model.
3) Forebay pool elevations remain constant at 268.0 ft during gated tests and at 276.0 ft during ungated tests.
4) Gate Opening (Go) is the minimum gate opening. Go is defined by the distance from the gate sill to a point perpendicular to the tailpiece crest.
5) Measurement locations presented in Figure 5-8.
6) Pressures less than -15 ft are indicative of cavitation conditions.

TABLE 5.6

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

DEFLECTOR PERFORMANCE for the SBSB
Deflector Length 34.1 ft, Transition Radius 49.2 ft
Deflector Elevation 157.2 ft

Forebay WSE	Discharge	Tailwater	Submergence[2]	Performance Classification[3]
ft	cfs/bay	ft	ft	Middle Bay
257.0	10,500	158.00	0.8	Plunging
		158.50	1.3	Plunging
		159.00	1.8	Skimming/Plunging
		159.50	2.3	Skimming
		163.50	6.3	Skimming
		164.00	6.8	Skimming/Undular
		168.75	11.6	Undular
		169.25	12.1	Undular/Hydraulic Jump
		170.25	13.1	Hydraulic Jump
262.5	17,000	159.50	2.3	Plunging/Skimming
		160.50	3.3	Skimming
		164.00	6.8	Skimming
		165.75	8.6	Skimming/Undular
		166.25	9.1	Undular/Skimming
		171.00	13.8	Undular
		173.00	15.8	Undular
		174.00	16.8	Undular/Hydraulic Jump
		174.50	17.3	Hydraulic Jump
		175.00	17.8	Hydraulic Jump
264.0	18,900	160.00	2.8	Plunging
		160.50	3.3	Skimming
		161.00	3.8	Skimming
		165.50	8.3	Skimming
		166.25	9.1	Skimming/Undular
		167.25	10.1	Undular/Skimming
		168.00	10.8	Undular
		173.00	15.8	Undular
		174.25	17.1	Undular/Hydraulic Jump
		174.75	17.6	Hydraulic Jump/Undular
		175.50	18.3	Hydraulic Jump
		176.50	19.3	Hydraulic Jump

Notes: 1) Tailwater measured 650 ft downstream of deflector
2) Submergence = Tailwater Elev. - Deflector Elev. (157.2 ft)
3) Flow characteristics considered as either plunging, skimming, undular, or hydraulic jump
4) Crest Elevation 242.5 ft

TABLE 5.7

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

WATER SURFACE PROFILE over the SBSB
Forebay Water Surface Elevation = 264.0 ft

Location	Distance Downstream from D/S Face of Gate Slot	Crest Elevation	Water Surface Elevation		Depth of Flow	
			Chute Centerline	Left Pier	Chute Centerline	Left Pier
	ft	ft	ft	ft	ft	ft
1	-1.26 (Upstream)	242.5	-	263.7	-	21.2
2	-0.821 (Upstream)	242.5	260.0	257.8	17.5	15.3
3	0.00	242.5	256.4	256.4	13.9	13.9
4	12.48	242.1	252.8	253.8	10.7	11.7
5	24.96	236.4	246.3	248.1	7.3	8.6
6	37.56	228.9	236.8	238.1	5.8	6.7
7	47.52	222.8	230.9	230.7	6.0	5.8
8	58.32	216.3	224.5	224.0	6.0	5.6
9	68.16	210.1	217.5	216.7	5.5	4.8
10	78.96	203.9	210.5	210.3	4.9	4.7
11	89.64	197.4	204.4	203.8	5.1	4.7
12	100.20	191.3	196.8	197.1	4.1	4.3
13	111.12	185.0	190.1	191.0	3.8	4.4
14	121.80	178.7	183.4	184.5	3.5	4.3
15	132.36	172.3	177.0	177.8	3.4	4.1

Notes: 1) Discharge = 18,900 cfs
2) Data and measurement locations given in Figure 5-10

TABLE 6.1

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

FINAL DESIGN RSW WATER SURFACE PROFILE
Forebay Water Surface Elevation = 257.0 ft

Location	Distance Downstream of Pier Nose	Crest Elevation	Water Surface Elevation		Depth of Flow	
			Chute Centerline	Left Pier	Chute Centerline	Left Pier
	ft	ft	ft	ft	ft	ft
1	-	-	255.3	255.5	-	-
-	0.8	243.6	-	254.4	-	10.8
-	1.6	244.3	-	253.2	-	8.9
-	2.5	244.7	-	253.1	-	8.4
-	3.3	245.0	-	252.9	-	7.8
2	4.1	245.3	254.4	252.9	9.1	7.6
3	8.2	245.4	253.0	252.3	7.7	6.9
4	12.3	244.5	251.7	251.5	7.1	7.0
5	16.4	242.9	249.1	248.5	6.2	5.6
6	20.5	240.7	246.6	246.3	5.9	5.6
7	24.6	237.8	243.1	242.7	5.3	4.9
8	28.7	234.8	240.1	239.6	5.2	4.8
9	32.8	231.8	236.2	236.1	4.4	4.2
10	36.9	228.8	232.8	233.1	4.0	4.3
11	41.0	225.8	229.4	230.3	3.6	4.5
12	50.0	219.2	223.9	223.4	4.7	4.2
13	57.4	213.8	218.5	217.0	4.7	3.3
14	65.6	207.8	212.2	209.8	4.4	2.1
15	82.0	195.7	200.2	200.2	4.5	4.5

Notes: 1) Discharge = 7,000 cfs

TABLE 6.2

John Day Lock and Dam Removable Spillway Weir

Hydraulic Model Study

FINAL DESIGN RSW WATER SURFACE PROFILE

Forebay Water Surface Elevation = 264.0 ft

Location	Distance Downstream of Pier Nose	Crest Elevation	Water Surface Elevation		Depth of Flow	
			Chute Centerline	Left Pier	Chute Centerline	Left Pier
	ft	ft	ft	ft	ft	ft
1	-	-	261.1	262.3	-	-
-	0.8	243.6	-	259.5	-	15.9
-	1.6	244.3	-	258.1	-	13.8
-	2.5	244.7	-	257.3	-	12.6
-	3.3	245.0	-	256.7	-	11.7
2	4.1	245.3	260.1	256.5	14.8	11.2
3	8.2	245.4	259.0	255.0	13.6	9.6
4	12.3	244.5	257.2	253.2	12.7	8.7
5	16.4	242.9	255.0	252.5	12.1	9.5
6	20.5	240.7	252.5	249.4	11.8	8.7
7	24.6	237.8	249.4	246.4	11.6	8.6
8	28.7	234.8	246.2	243.5	11.4	8.7
9	32.8	231.8	242.5	240.1	10.7	8.3
10	36.9	228.8	239.1	236.8	10.3	8.0
11	41.0	225.8	236.7	233.4	10.9	7.6
12	50.0	219.2	229.1	227.0	9.9	7.8
13	57.4	213.8	222.8	221.0	9.0	7.2
14	65.6	207.8	214.7	214.7	7.0	7.0
15	82.0	195.7	201.8	201.5	6.1	5.7

Notes: 1) Discharge = 15,500 cfs

TABLE 6.3

John Day Lock and Dam Removable Spillway Weir

Hydraulic Model Study

FINAL DESIGN RSW WATER SURFACE PROFILE

Forebay Water Surface Elevation = 268.0 ft

Location	Distance Downstream of Pier Nose	Crest Elevation	Water Surface Elevation		Depth of Flow	
			Chute Centerline	Left Pier	Chute Centerline	Left Pier
	ft	ft	ft	ft	ft	ft
1	-	-	264.9	266.3	-	-
-	0.8	243.6	-	263.1	-	19.5
-	1.6	244.3	-	262.1	-	17.8
-	2.5	244.7	-	261.5	-	16.8
-	3.3	245.0	-	260.9	-	15.9
2	4.1	245.3	264.4	259.8	19.2	14.5
3	8.2	245.4	262.6	257.1	17.3	11.7
4	12.3	244.5	261.0	254.9	16.5	10.3
5	16.4	242.9	259.1	252.8	16.2	9.9
6	20.5	240.7	256.5	250.3	15.8	9.6
7	24.6	237.8	253.8	247.0	15.9	9.1
8	28.7	234.8	250.0	243.4	15.2	8.6
9	32.8	231.8	246.4	241.6	14.6	9.8
10	36.9	228.8	243.3	237.9	14.5	9.1
11	41.0	225.8	239.6	235.2	13.8	9.4
12	50.0	219.2	233.0	230.1	13.8	10.9
13	57.4	213.8	225.0	224.6	11.2	10.8
14	65.6	207.8	217.1	218.7	9.4	10.9
15	82.0	195.7	204.1	205.7	8.4	10.0

Notes: 1) Discharge = 21,000 cfs

TABLE 6.4

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

FINAL DESIGN RSW
VELOCITY AT TRANSITION WITH EXISTING SPILLWAY

Forebay	Discharge		Velocity (ft/s)	
ft	cfs/bay	Calculated[1]	Calculated[2]	Measured[3]
264.0	15,500	63.4	63.3	61.5
268.0	21,000	66.1	62.7	63.8

Notes: 1) Assumes minimal energy loss over the RSW crest. $V=(2gh)^{1/2}$
where h = forebay WSE - WSE at the end of the tailpiece section
(data adpoted from Figure 6-2)
2) From continuity; V = Discharge / Flow area
where depth of flow is measured at the end of the tailpiece section
(data adpoted from Figure 6-2)
3) Mean velocity across the width of the spillway.

TABLE 6.5

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

FINAL DESIGN RSW
TIME-AVERAGED PRESSURE SUMMARY

Tap Elevation		Pressure Head			
		Forebay WSE = 264 ft		Forebay WSE = 268 ft	
		Bay Centerline	Near the Pier	Bay Centerline	Near the Pier
	(ft)	(ft)	(ft)	(ft)	(ft)
Position 1	244.3	9.0	12.3	8.6	12.7
Position 2	245.7	4.5	0.8	4.1	-2.5
Position 3	245.1	3.3	10.0	3.3	11.9
Position 4	239.5	3.7	no readings	4.5	no readings
Position 5	232.4	7.0	4.5	8.6	6.6
Position 6	218.9	6.2	4.9	7.8	2.5
Position 7	217.6	4.9	no tap	7.4	no tap
Position 8	215.8	6.6	7.4	6.6	9.8
Position 9	212.9	7.8	4.9	7.8	7.4
Position 10	206.9	5.3	4.1	7.0	4.1
Position 11	198.6	6.2	2.1	6.2	2.9
Position 12	189.8	-1.2	no tap	-1.2	no tap
Position 13	179.9	1.2	no tap	0.4	no tap
Position 14	169.1	-1.6	no tap	-2.1	no tap
Position 11[4]	198.6	-5.3	no tap	-4.5	no tap
Position 11[5]	198.6	0.4	no tap	-0.4	no tap
Bottom[2]	227.2		6.2		10.7
Top[3]	232.2		0.4		2.9

Note : 1) Pressures presented are relative to elevation of the pressure tap
2) Pressure tap located downstream of the pier offset 1 ft above the crest
3) Pressure tap located downstream of the pier offset 6 ft above the crest
4) 1 inch (prototype) discontinuity 0.4 ft (prototype) upstream of Pressure Tap 11
5) 1 inch (prototype) discontinuity 0.8 ft (prototype) upstream of Pressure Tap 11

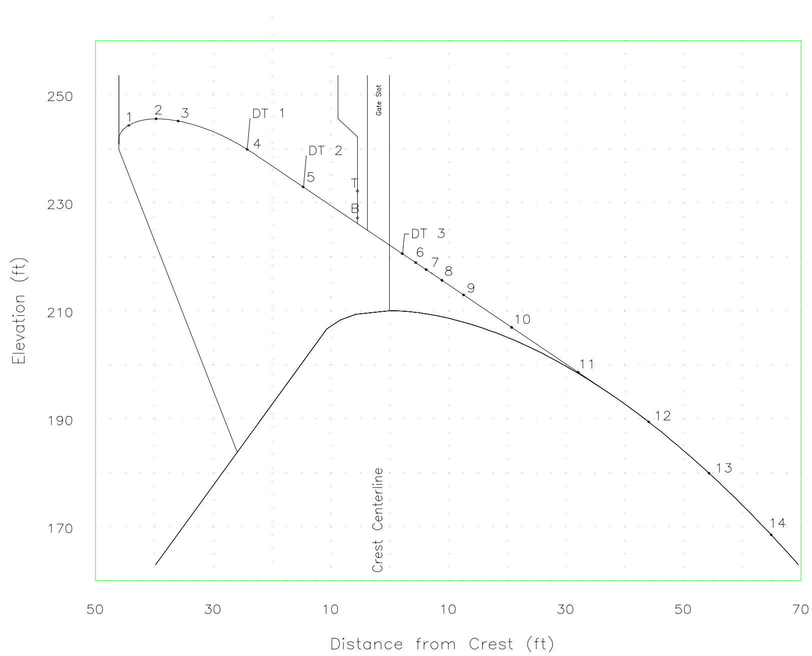


TABLE 6.6

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

FINAL DESIGN RSW
TIME-VARIABLE PRESSURE SUMMARY

Transducer Elev. (ft)		Pressures (ft)							
		Forebay WSE = 264 ft				Forebay WSE = 268 ft			
		HGL	Maximum	Minimum	St. Dev.	HGL	Maximum	Minimum	St. Dev.
DT 1	239.9	3.7	6.7	-0.8	1.2	4.5	6.1	2.9	0.7
DT 2	232.5	7.0	9	4.7	0.7	8.6	13.1	7.4	1.1
DT 3	220.9	6.2	9.6	3.2	1.4	7.8	10.8	7.4	1.2

Note : 1) Pressures presented are relative to elevation of the pressure transducer
2) Pressure transducer located downstream of the pier offset 1 ft above the crest
3) Dominant frequency less than 0.1 Hz for all test conditions

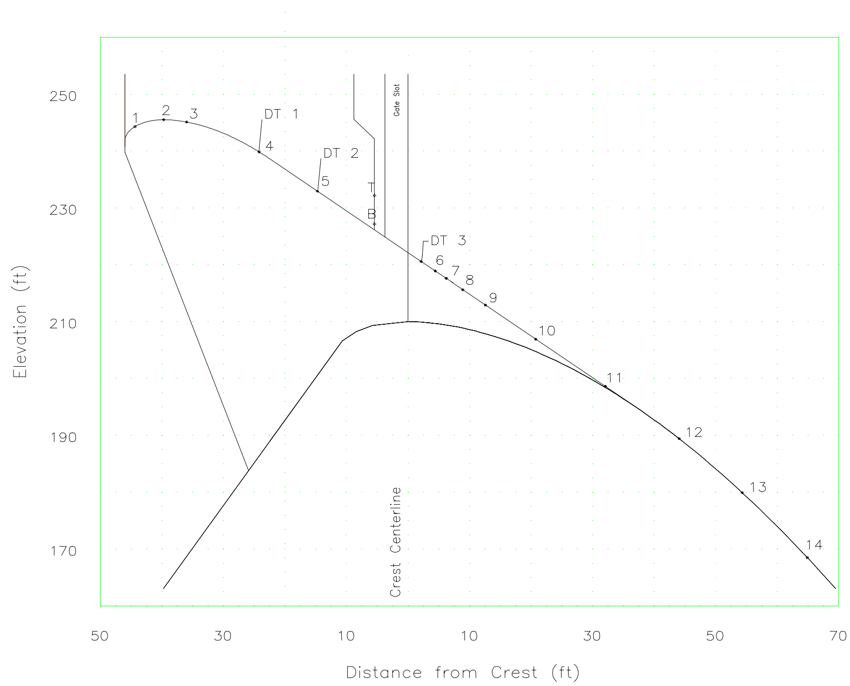


TABLE 6.7

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

FINAL DESIGN DEFLECTOR HYDRAULIC PERFORMANCE
Deflector Length 50.0 ft, Transition Radius 50 ft
Deflector Elevation 150.0 ft

Forebay WSE	Discharge	Tailwater	Submergence[2]	Performance Classification[3]
ft	cfs/bay	ft	ft	Bay 20 (full bay)
252.0	3,000	150.0	0.0	Skimming/Plunging
		151.8	1.8	Skimming
		152.0	2.0	Undular
		155.0	5.0	Undular
		157.3	7.3	Undular
		157.5	7.5	Hydraulic Jump
		158.0	8.0	Hydraulic Jump
257.0	7,000	152.3	2.3	Plunging
		153.0	3.0	Skimming/Plunging
		156.0	6.0	Skimming/Undular
		157.0	7.0	Undular
		164.5	14.5	Undular
		166.8	16.8	Undular/Hydraulic Jump
		167.5	17.5	Hydraulic Jump
		168.0	18.0	Hydraulic Jump
262.5	14,000	155.5	5.5	Plunging
		156.0	6.0	Plunging/Skimming
		157.0	7.0	Skimming
		160.8	10.8	Skimming
		161.5	11.5	Undular/Skimming
		162.5	12.5	Undular
		167.5	17.5	Undular
		174.5	24.5	Undular
		175.3	25.3	Hydraulic Jump/Undular
		176.3	26.3	Hydraulic Jump
264.0	15,500	157.0	7.0	Plunging
		158.0	8.0	Plunging/Skimming
		159.0	9.0	Skimming
		162.0	12.0	Skimming
		163.0	13.0	Undular/Skimming
		169.0	19.0	Undular
		175.3	25.3	Undular
		176.0	26.0	Undular/Hydraulic Jump
		176.3	26.3	Hydraulic Jump/Undular
		177.3	27.3	Hydraulic Jump

Notes: 1) Tailwater measured 650 ft downstream of deflector
2) Submergence = Tailwater Elev. - Deflector Elev. (150 ft)
3) Flow characteristics considered as either plunging, skimming, undular, or hydraulic jump

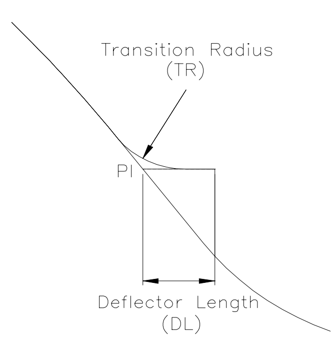


TABLE 6.8

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

PRESSURES ON FINAL DESIGN DEFLECTOR

Tap Elevation		Pressure Head (ft)				
		Discharge (cfs)				
	(ft)	112,500	88,300	53,000	40,000	25,000
Position 1	168.1	27	20	11	4	4
Position 2	154.8	45	38	27	22	20
Position 3	150.0	70	53	39	33	30
Position 4	147.0	28	22	14	8	3
Position 5	139.0	51	36	25	19	16

Note : 1) Pressures are relative to elevation of the pressure tap

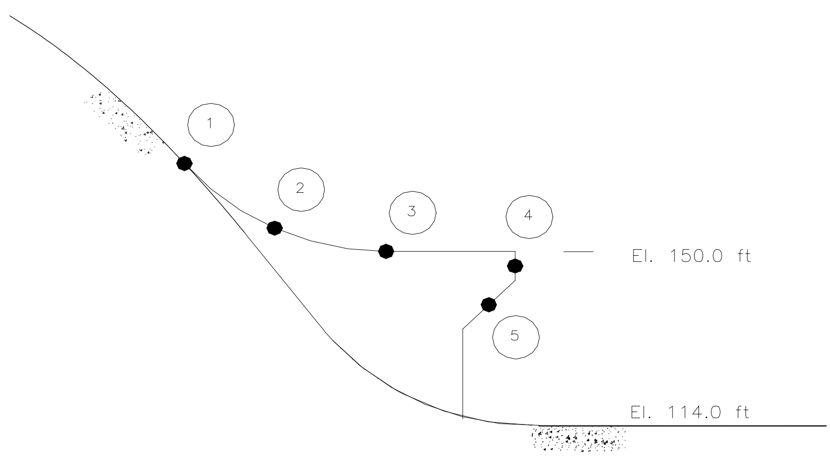


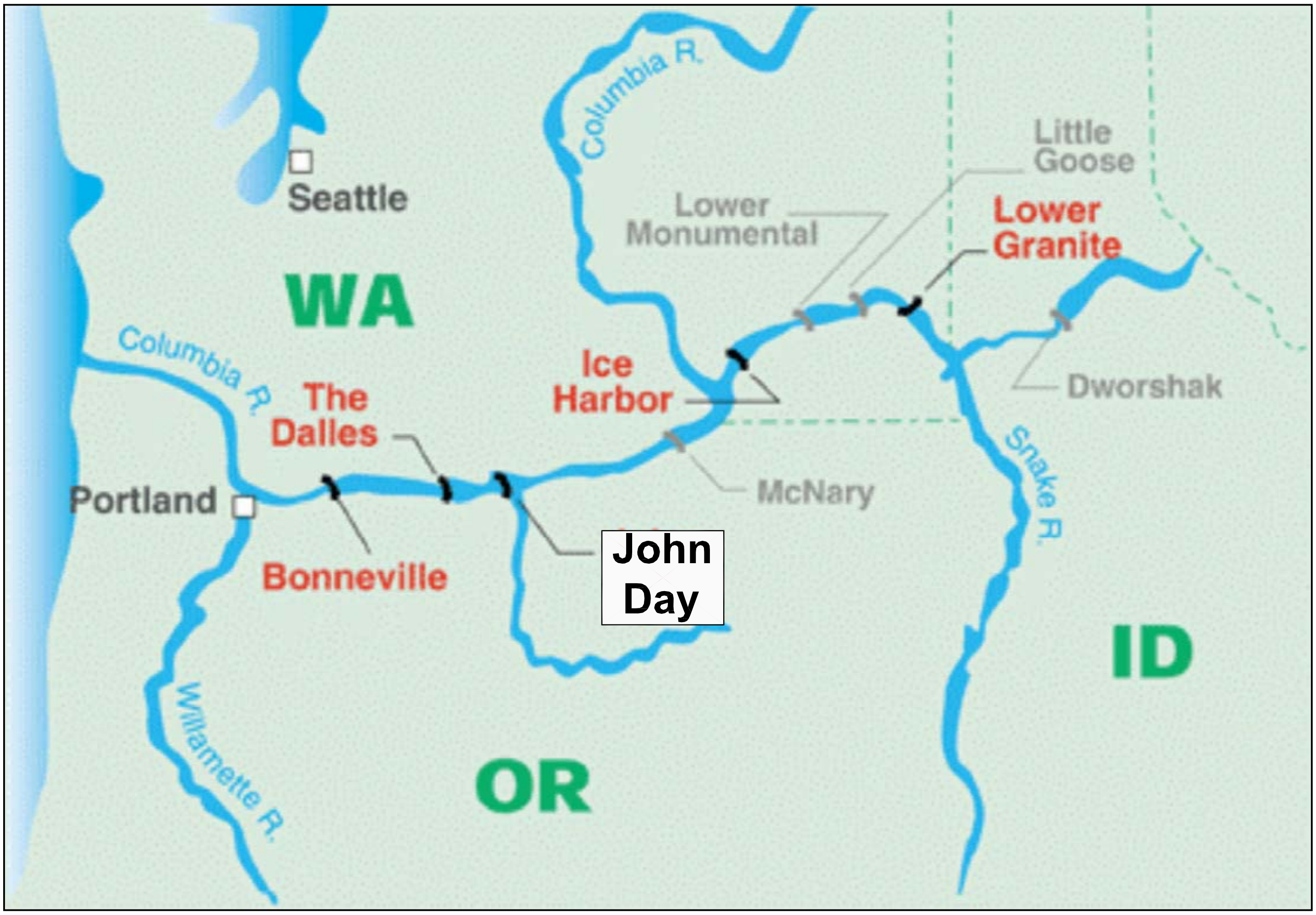
TABLE 6.9

John Day Lock and Dam Removable Spillway Weir
Hydraulic Model Study

TAILRACE VELOCITY WITH AND WITHOUT DEFLECTOR - SPILLWAY DESIGN FLOOD

Location[1]	Elevation	Velocity (ft/s)[2,3]			
		Existing Geometry (no Deflector)		Final (50-ft Long) Deflector	
	ft	Mean	Maximum	Mean	Maximum
100	Surface	35	49	35	42
	175	35	49	35	42
	160	35	49	35	42
	145	28	42	28	42
	130	14	42	14	42
	115	-14	-28	-14	-28
	100	-7	-14	-7	-14
200	Surface	35	49	35	42
	175	35	49	35	42
	160	35	49	35	42
	145	28	35	35	42
	130	21	28	14	28
	115	-14	-28	-14	-28
	100	-7	-14	-7	-14
300	Surface	28	42	21	35
	175	28	42	21	35
	160	28	42	21	35
	145	21	35	21	35
	130	14	28	21	28
	115	14	28	14	28
	100	-7	-14	-7	-14

Notes: 1) Distance in feet downstream relative to the end sill.
2) Velocities were measured through the center of the jet.
3) Negative velocities refer to flow moving upstream toward the spillway.



JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

John Day Dam
Location Plan

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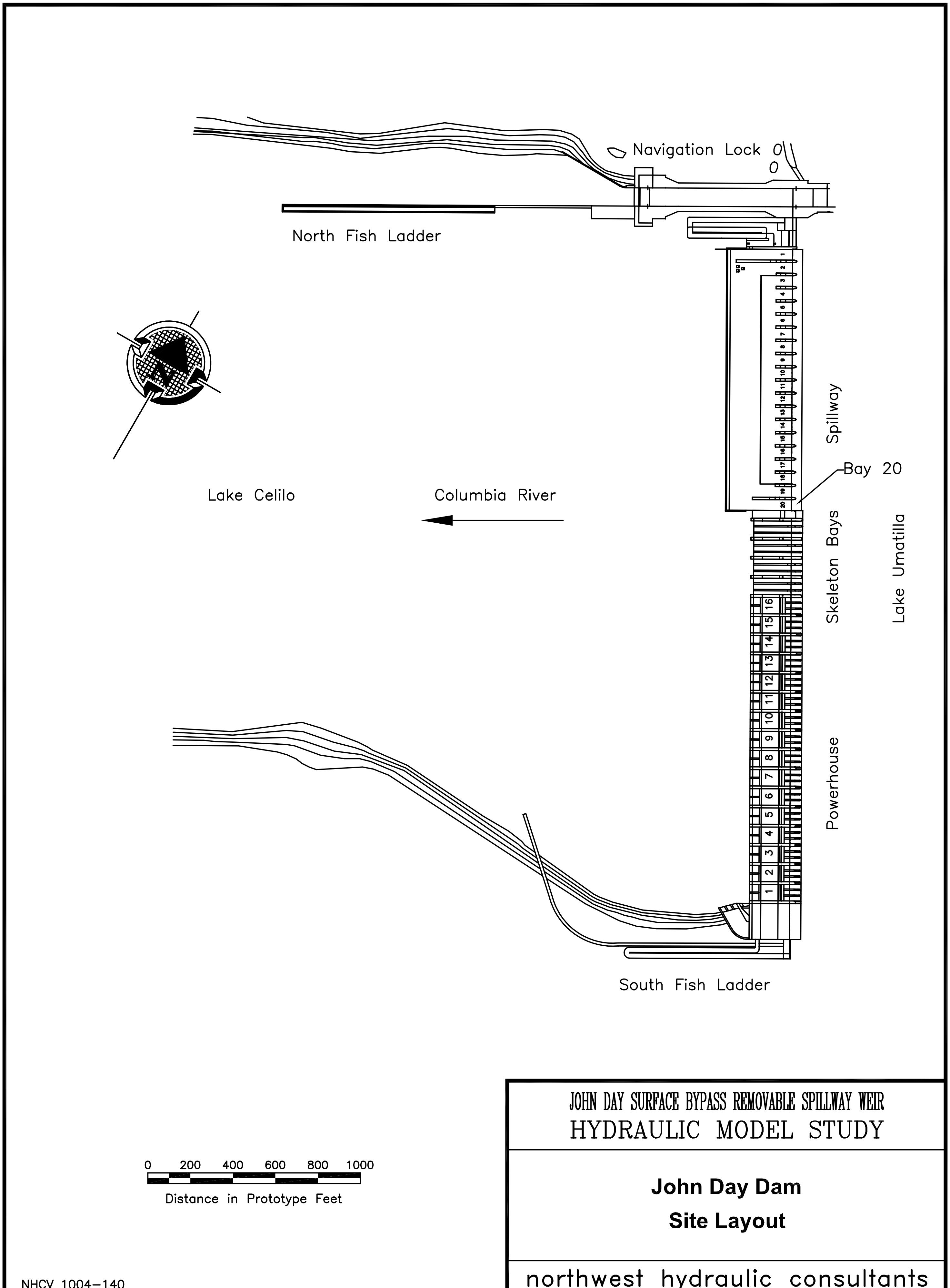
NHCV 1004–139

FIGURE 1-1



JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

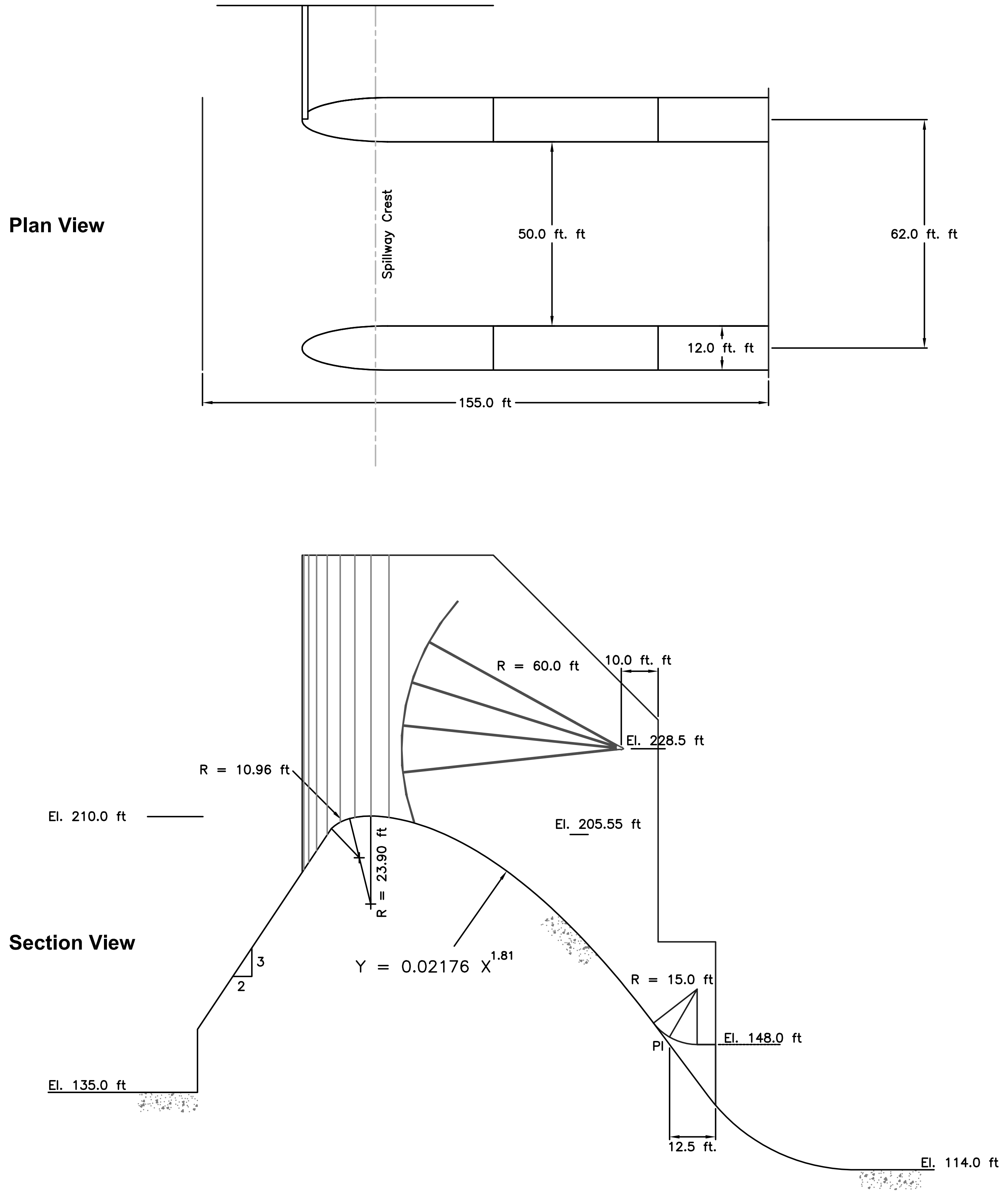
HYDRAULIC MODEL STUDY

John Day Dam

Site Layout

northwest hydraulic consultants

FIGURE 1-2



0 10 20 30 40 50

Distance in Prototype Feet

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Existing 12.5 ft Interior-Bay Deflectors at Elevation 148 ft

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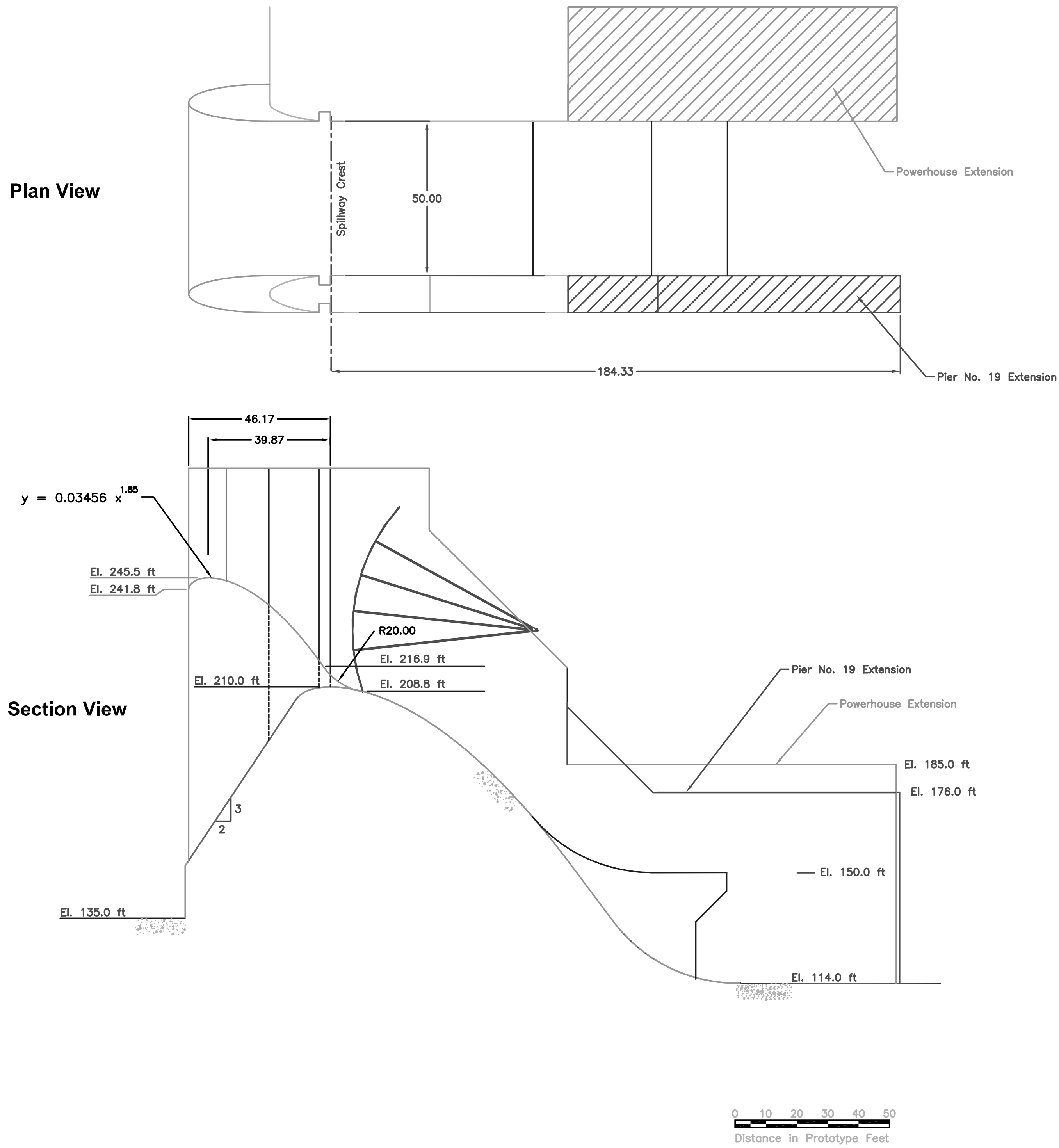
NHCV 1004–101

FIGURE 1-3



JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Removable Spillway Weir

Alternative 2

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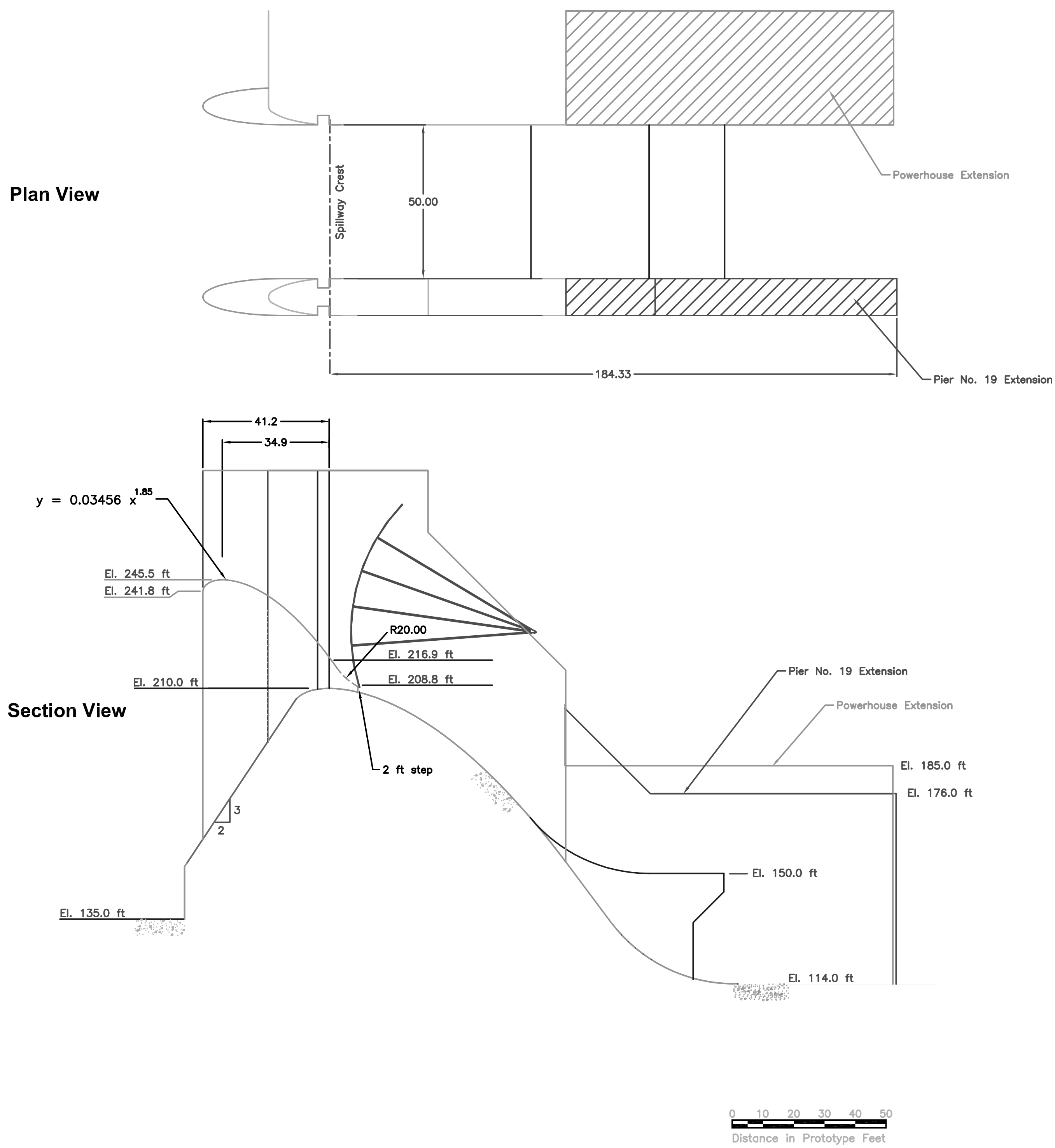
NHCV 1004–150

FIGURE 1-4



JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Removable Spillway Weir

Alternative 4

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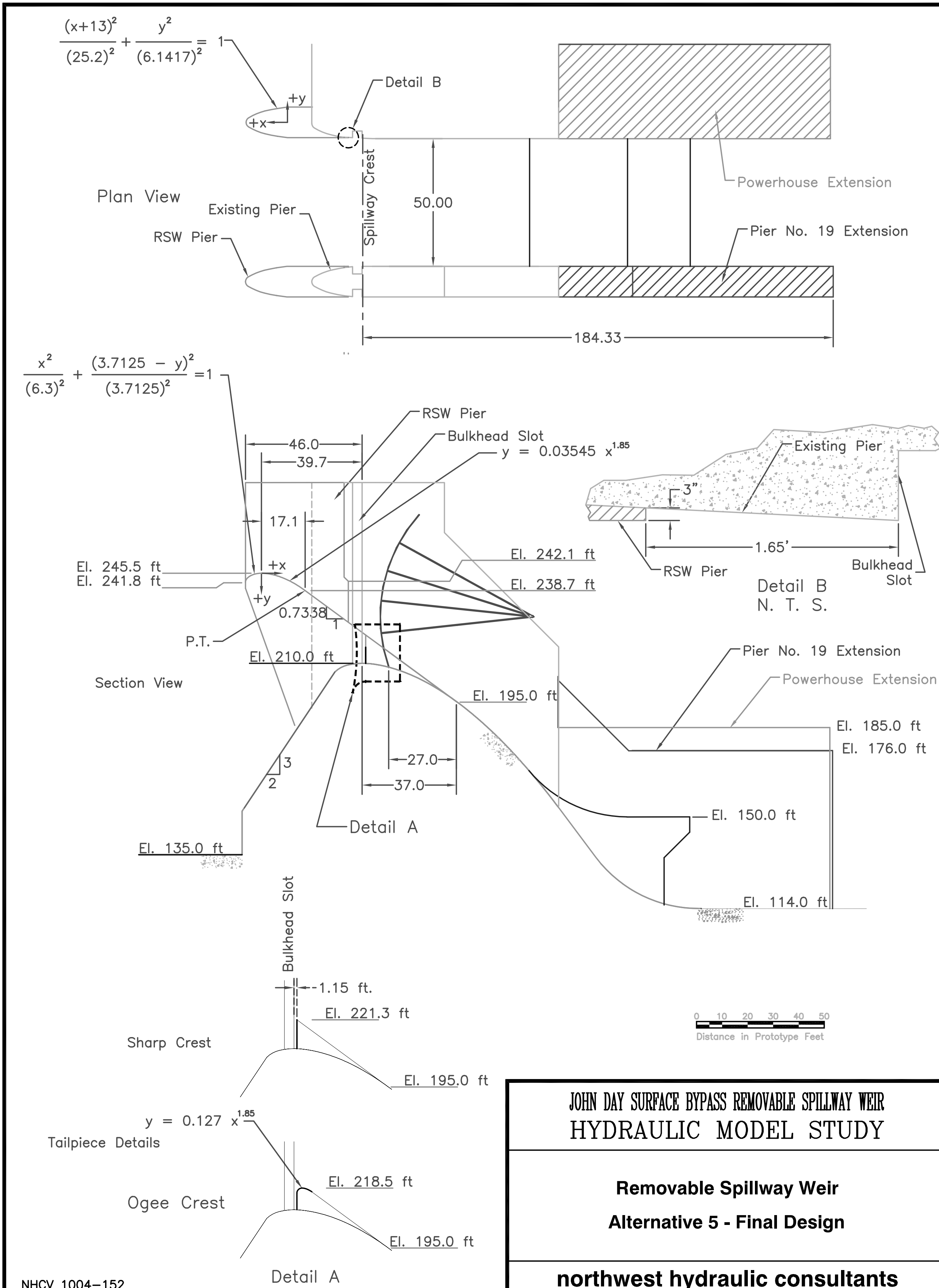
NHCV 1004–151

FIGURE 1-5



JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Removable Spillway Weir

Alternative 5 - Final Design

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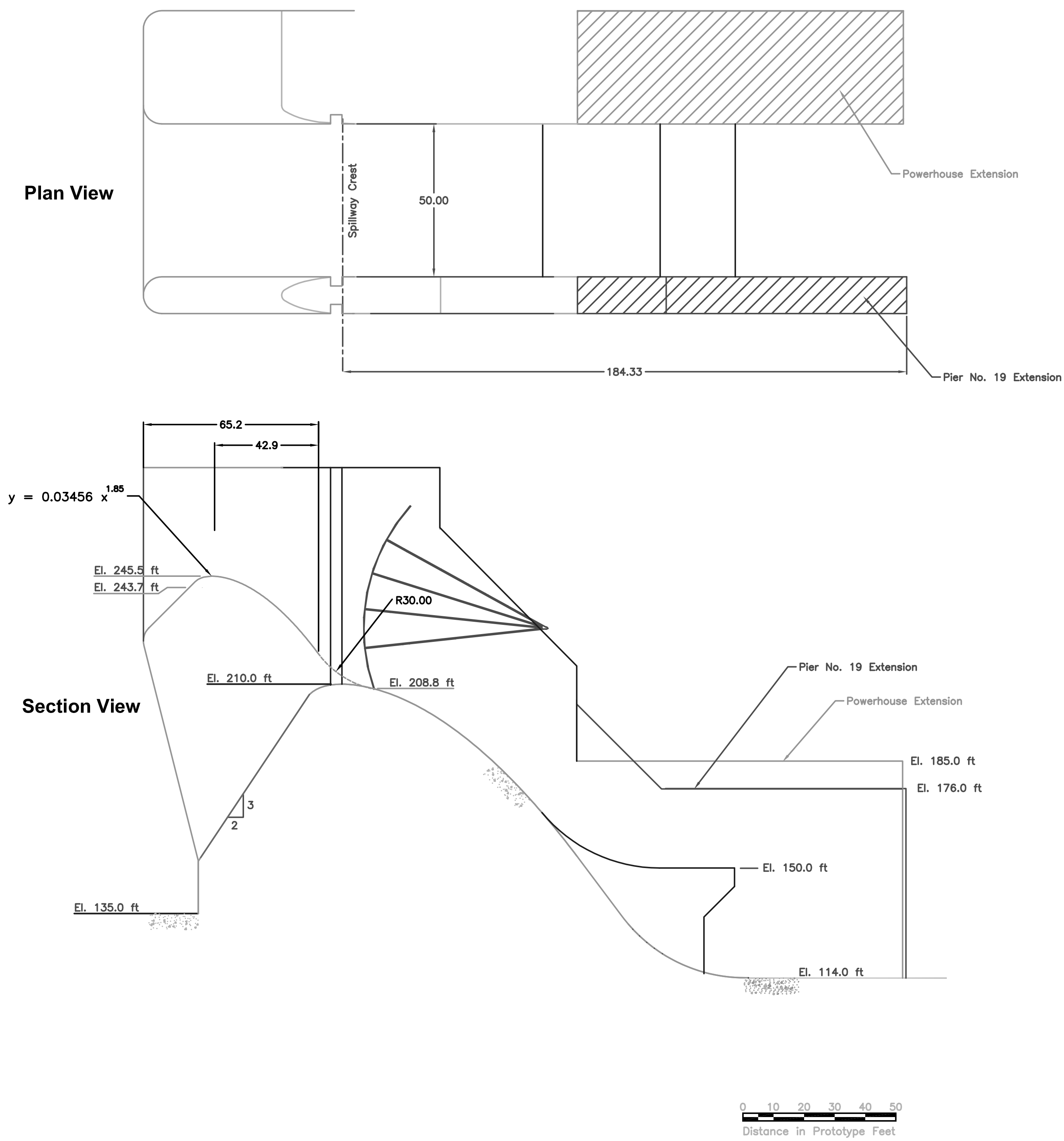
FIGURE 1-6



JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Removable Spillway Weir

Alternative 7

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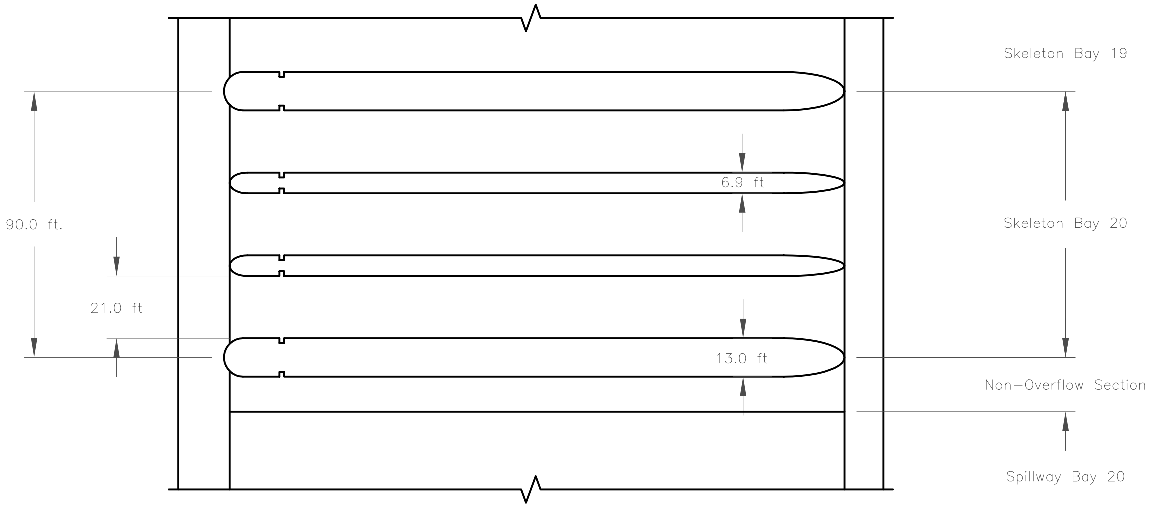
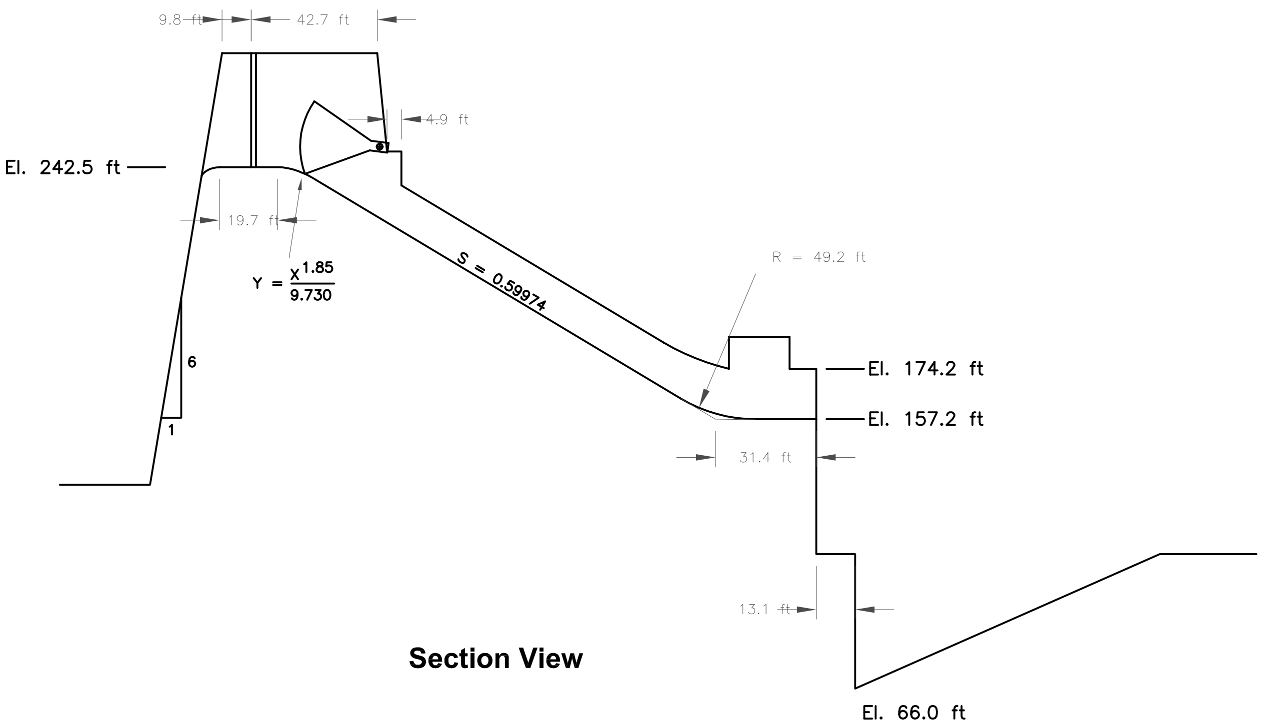
NHCV 1004–153

FIGURE 1-7



Plan View



Section View

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Skeleton Bay Surface Bypass Design

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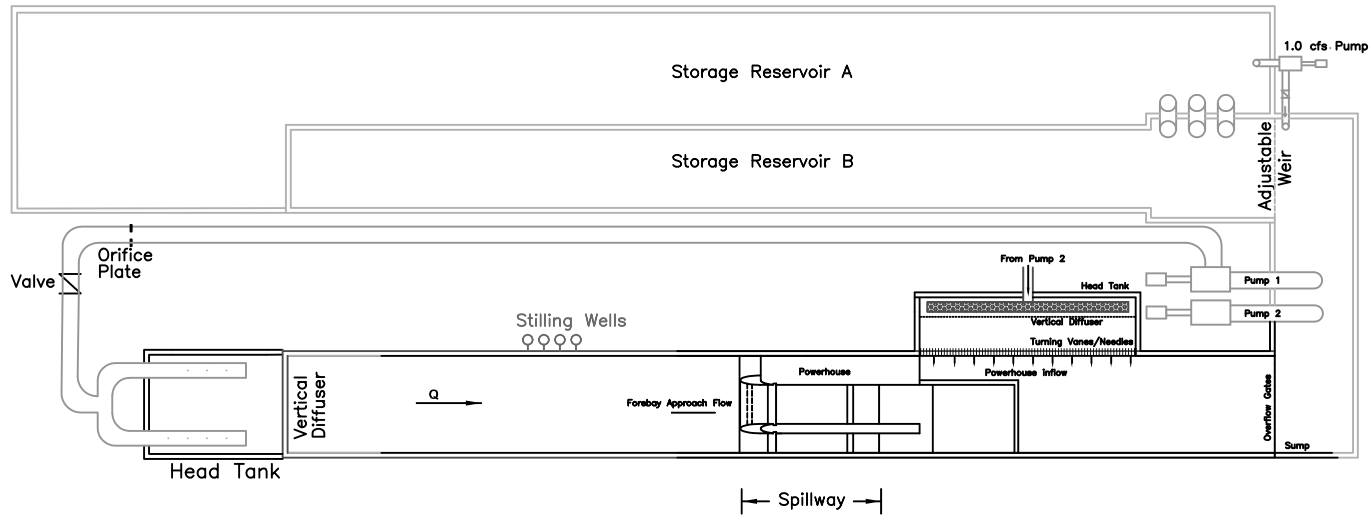
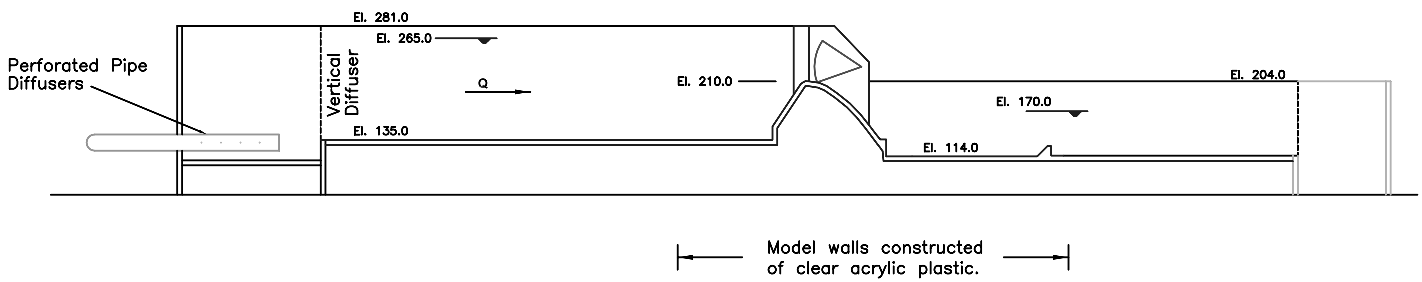
NHCV 1004–119

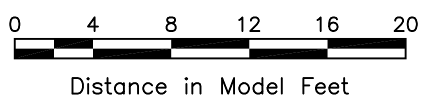
FIGURE 1-8



PLAN



SECTION



JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY

HYDRAULIC MODEL STUDY

Model #1 Layout

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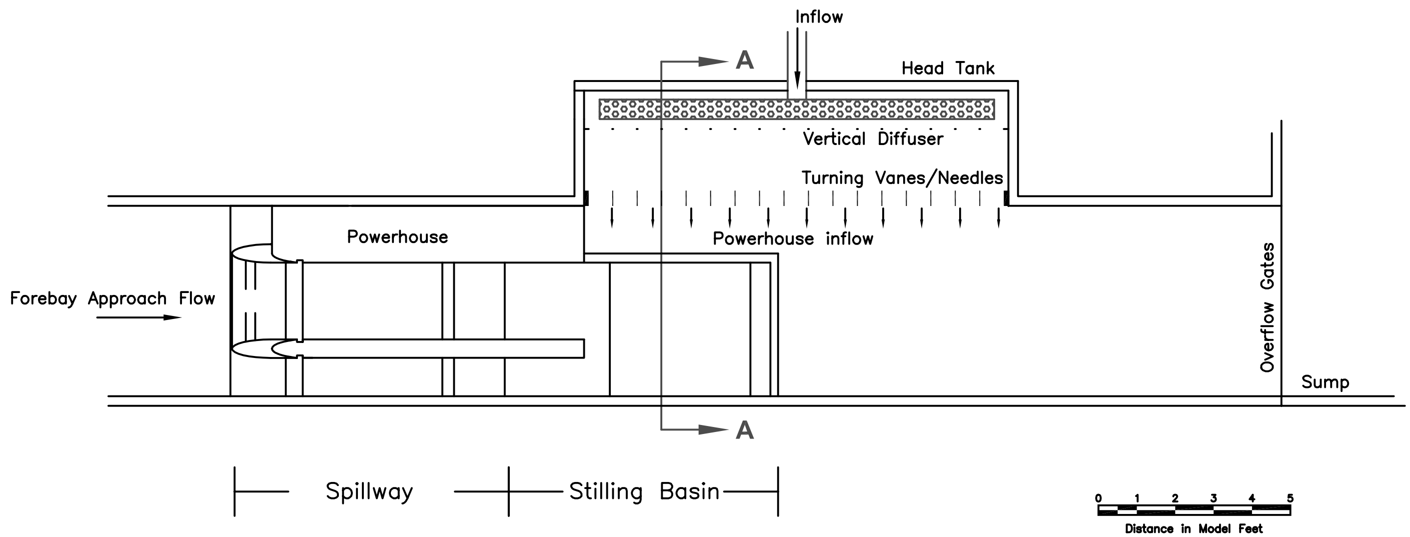
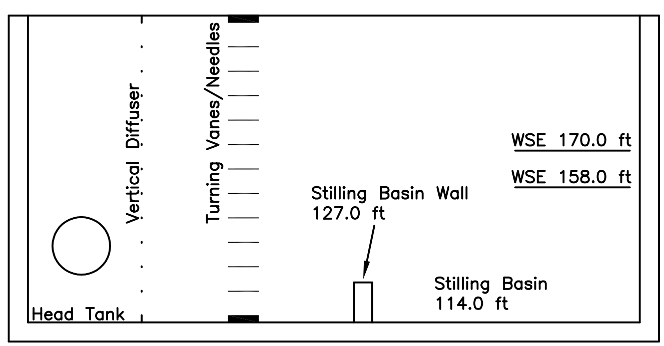
NHCV 1004–142

FIGURE 3-1





SECTION A-A

JOHN DAY SURFACE BYPASS REMOVALBLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Model #1 Details

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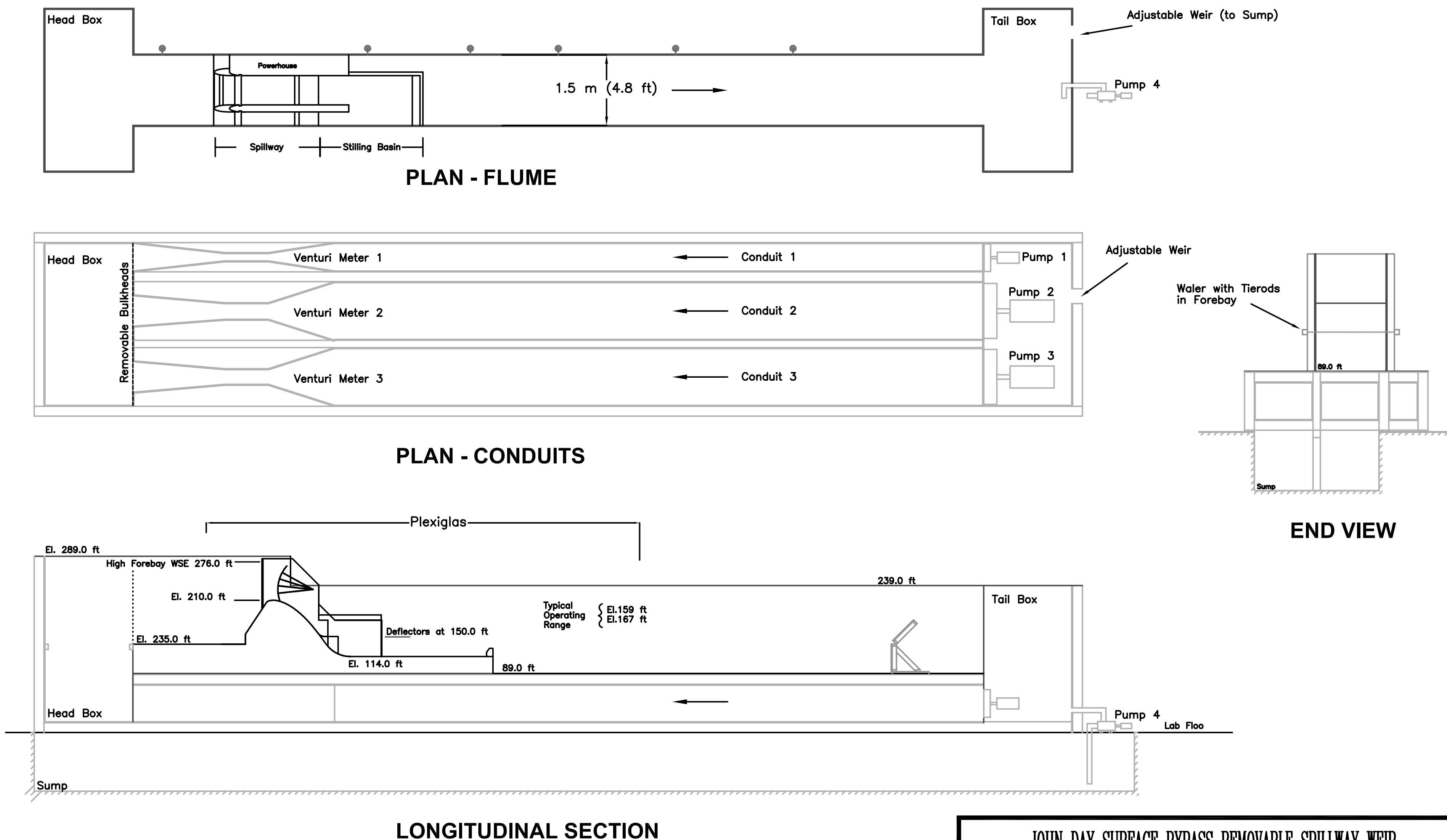
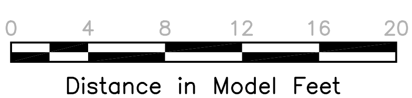
NHCV 1004–106

FIGURE 3-2



Notes: 1) Pumps 1, 2 and 3 are low–head propeller pumps with variable speed controllers.
2) • Pressure Tap Locations
3) Pump 4 (small centrifugal pump) continuously circulates to sump



NHCV 1004–143

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

Hydraulic Model Study

Model #2 Layout

High Flow Model

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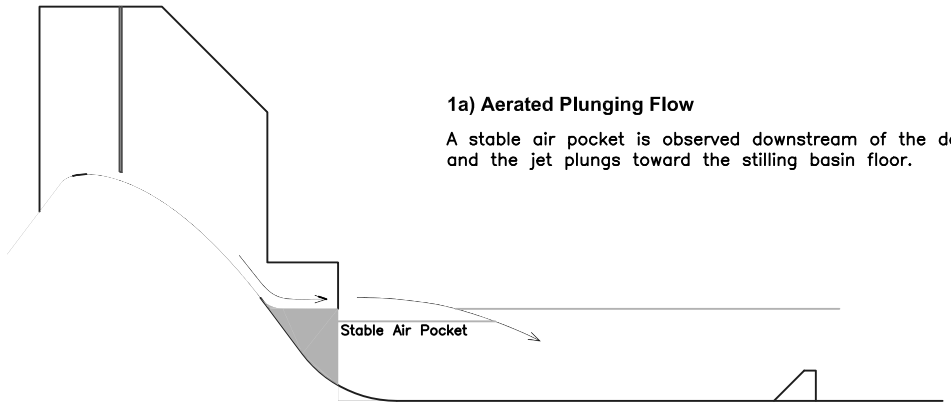
FIGURE 3-3

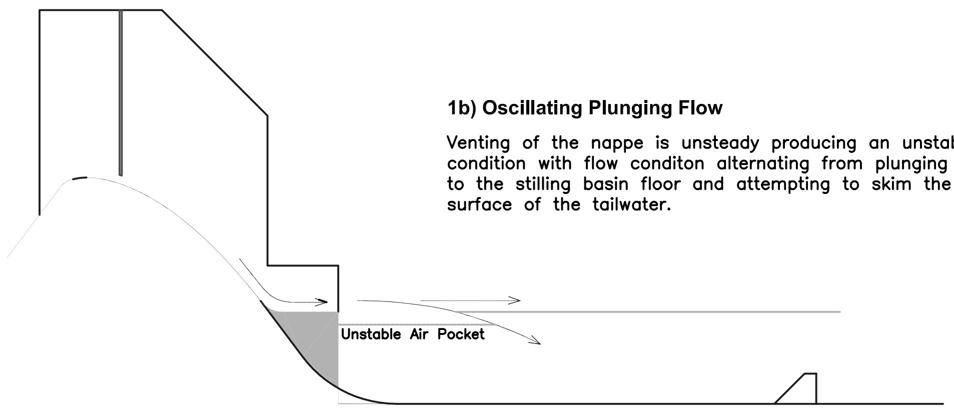
1a) Aerated Plunging Flow

A stable air pocket is observed downstream of the deflector and the jet plungs toward the stilling basin floor.



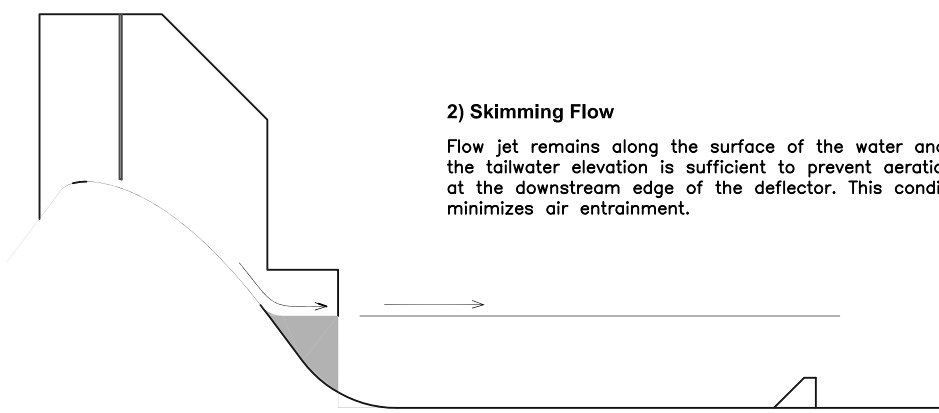
1b) Oscillating Plunging Flow

Venting of the nappe is unsteady producing an unstable condition with flow conditon alternating from plunging to the stilling basin floor and attempting to skim the surface of the tailwater.



2) Skimming Flow

Flow jet remains along the surface of the water and the tailwater elevation is sufficient to prevent aeration at the downstream edge of the deflector. This condition minimizes air entrainment.



Note

Classifications are adopted from WES Data Report, Bonnevillle Spillway Section Model, Columbia River, OR, 31 August 1997.

John Day Surface Bypass Removable Spillway Weir
Hydraulic Model Study

Deflector Performance Classification
Plunging and Skimming Flow Conditions

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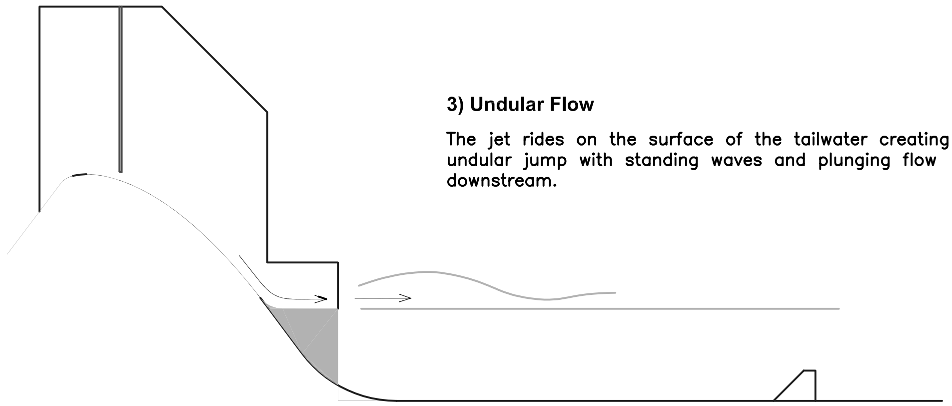
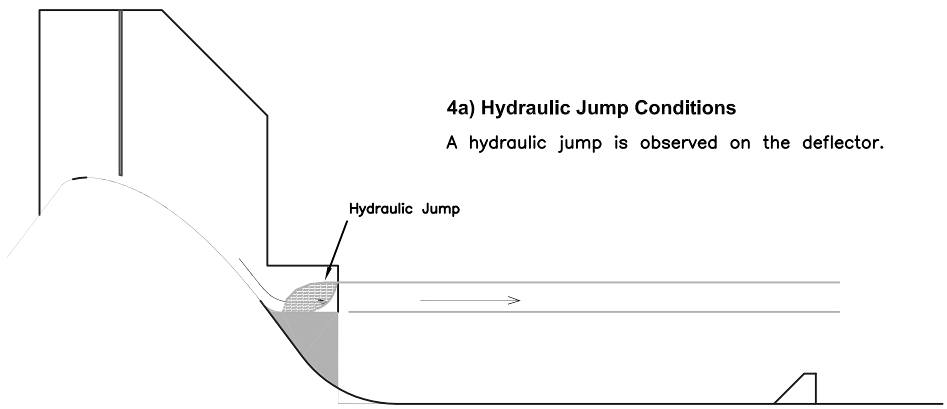
NHCV 1004-135

FIGURE 5-1



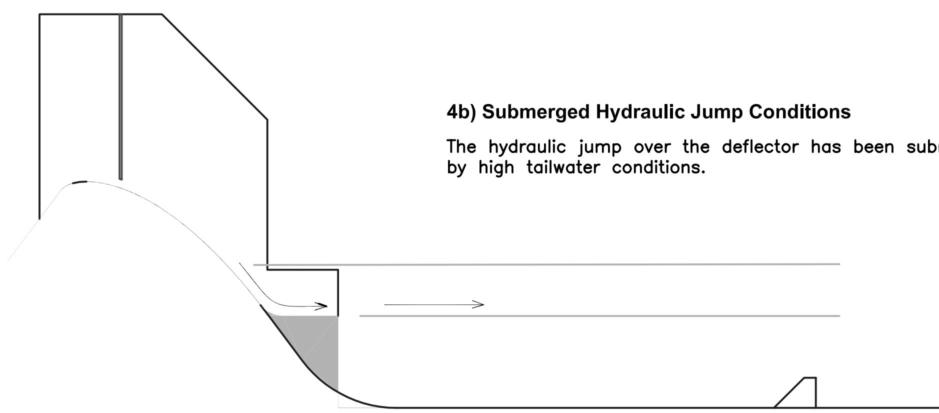
3) Undular Flow

The jet rides on the surface of the tailwater creating an undular jump with standing waves and plunging flow downstream.



4a) Hydraulic Jump Conditions

A hydraulic jump is observed on the deflector.



4b) Submerged Hydraulic Jump Conditions

The hydraulic jump over the deflector has been submerged by high tailwater conditions.

Note

Classifications are adopted from WES Data Report, Bonnevillle Spillway Section Model, Columbia River, OR, 31 August 1997.

John Day Surface Bypass Removable Spillway Weir
Hydraulic Model Study

Deflector Performance Classification
Undular and Hydraulic Jump Conditions

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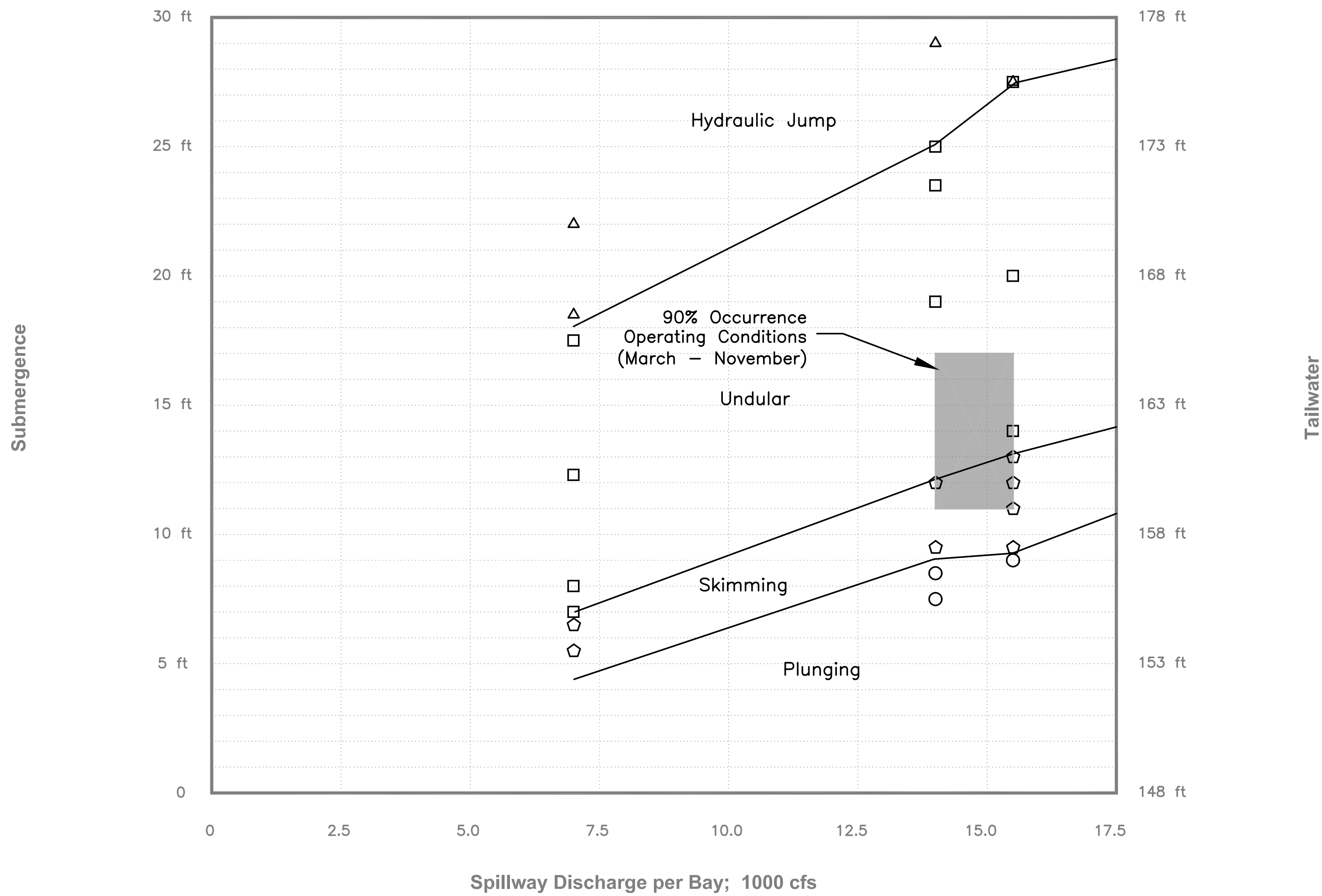
NHCV 1004–136

FIGURE 5-2



Notes
1) Performance for full center bay in model flume
2) Deflector Length (DL) 30.0 ft with a 20 ft Transition Radius (TR)
3) Deflector Elevation (DE); 148.0 ft

Legend (Flow Conditions)
△ Hydraulic Jump
□ Undular
⬠ Skimming
○ Plunging

See Figure 5–1 and 5–2

TEST P–1
NHCV 1004–111

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Bay 20 Deflector Performance (Test P-1)
DL = 30.0 ft; TR = 20 ft
DE = 148.0 ft

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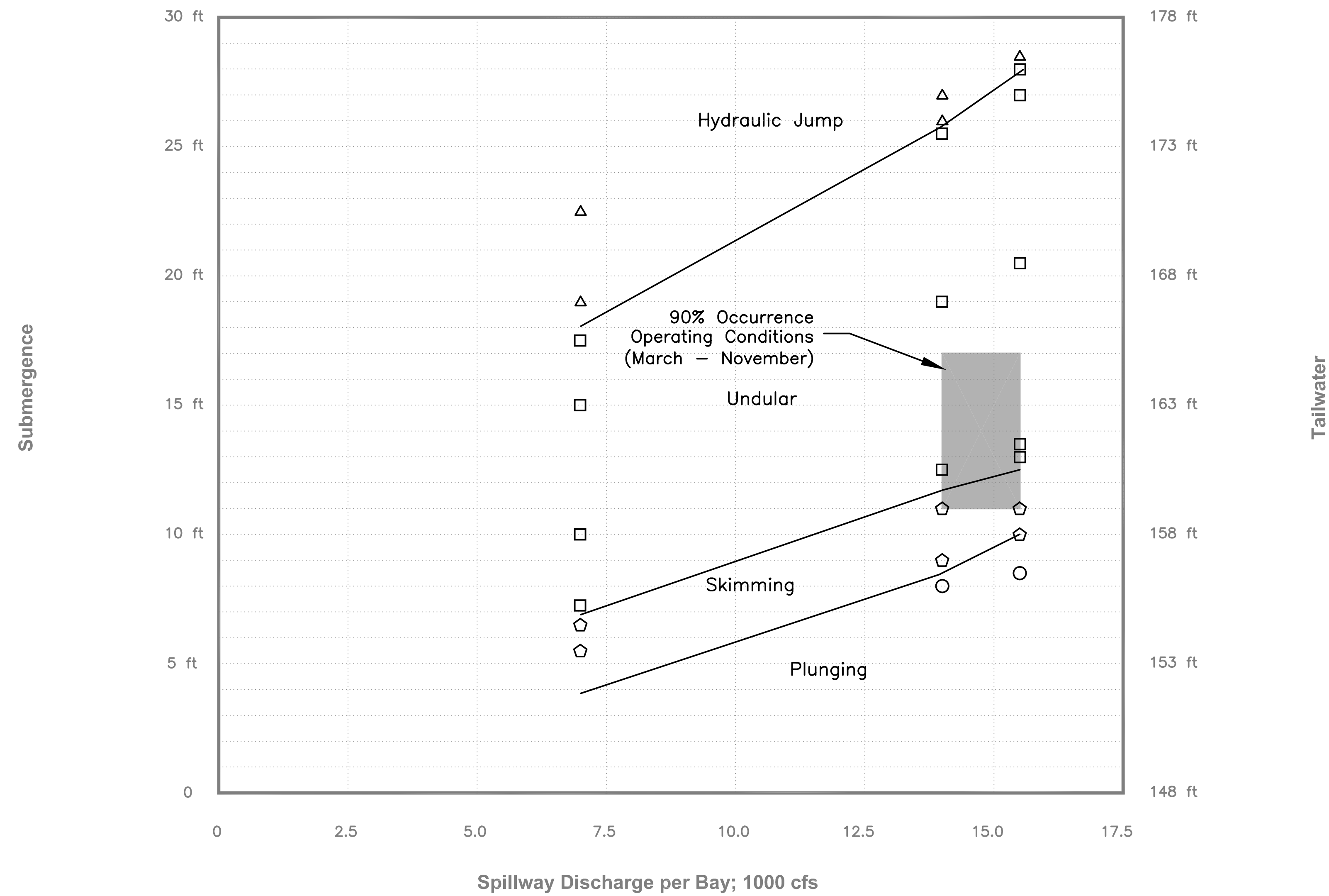
FIGURE 5-3



Notes 1) Performance for full center bay in model flume
2) Deflector Length (DL) 30.0 ft with a 50 ft Transition Radius (TR)
3) Deflector Elevation (DE); 148.0 ft

Legend (Flow Conditions)
- △ Hydraulic Jump
- □ Undular
- ⬠ Skimming
- ○ Plunging

See Figure 5–1 and 5–2

TEST P–2
NHCV 1004–113

JPHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Bay 20 Deflector Performance (Test P-2)
DL = 30 ft; TR = 50 ft
DE = 148.0 ft

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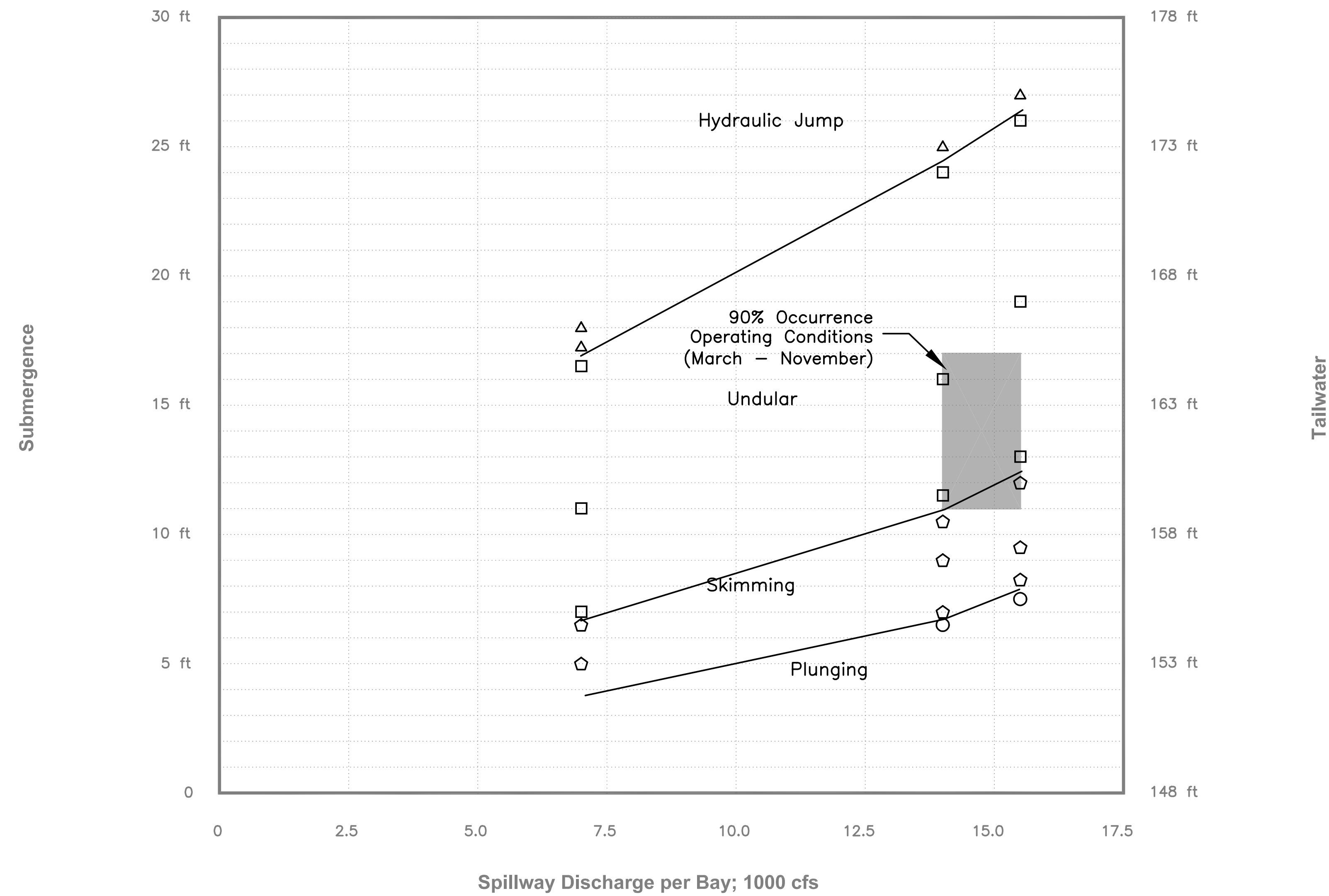
FIGURE 5-4



Notes 1) Performance for full center bay in model flume
2) Deflector Length (DL) 30.0 ft with a 50 ft Transition Radius (TR)
3) Deflector Elevation (DE); 148.0 ft

Legend (Flow Conditions)
- △ Hydraulic Jump
- □ Undular
- ⬠ Skimming
- ○ Plunging

See Figure 5–1 and 5–2

TEST P–3
NHCV 1004–114

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Bay 20 Deflector Performance (Test P-3)
DL = 50.0 ft; TR = 50 ft
DE = 148.0 ft

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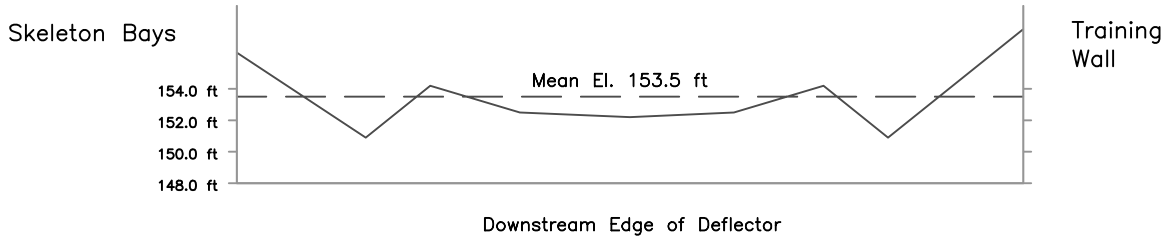
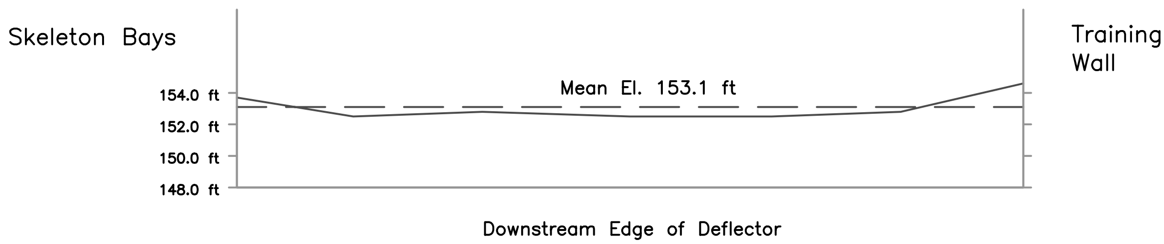
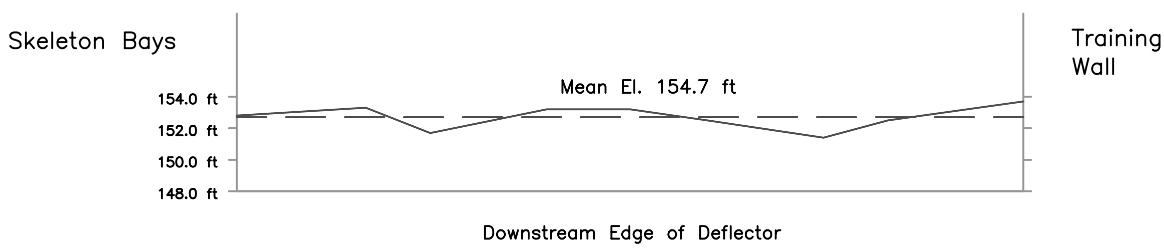
FIGURE 5-5



Test P-1 Deflector Elevation (DE) = 148.0 ft
Deflector Length (DL) = 30 ft
Transition Radius (TR) = 20 ft



Test P-2 Deflector Elevation (DE) = 148.0 ft
Deflector Length (DL) = 30 ft
Transition Radius (TR) = 50 ft



Test P-3 Deflector Elevation = (DE) 148.0 ft
Deflector Length (DL) = 50 ft
Transition Radius (TR) = 50 ft

Notes: 1) Forebay WSE = 264.0 ft
2) Discharge over Bay 20 RSW = 15,500 cfs

NHCV 1004–115

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Cross-Section of Jet Over the Downstream Edge of the Deflector Forebay WSE = 264.0 ft

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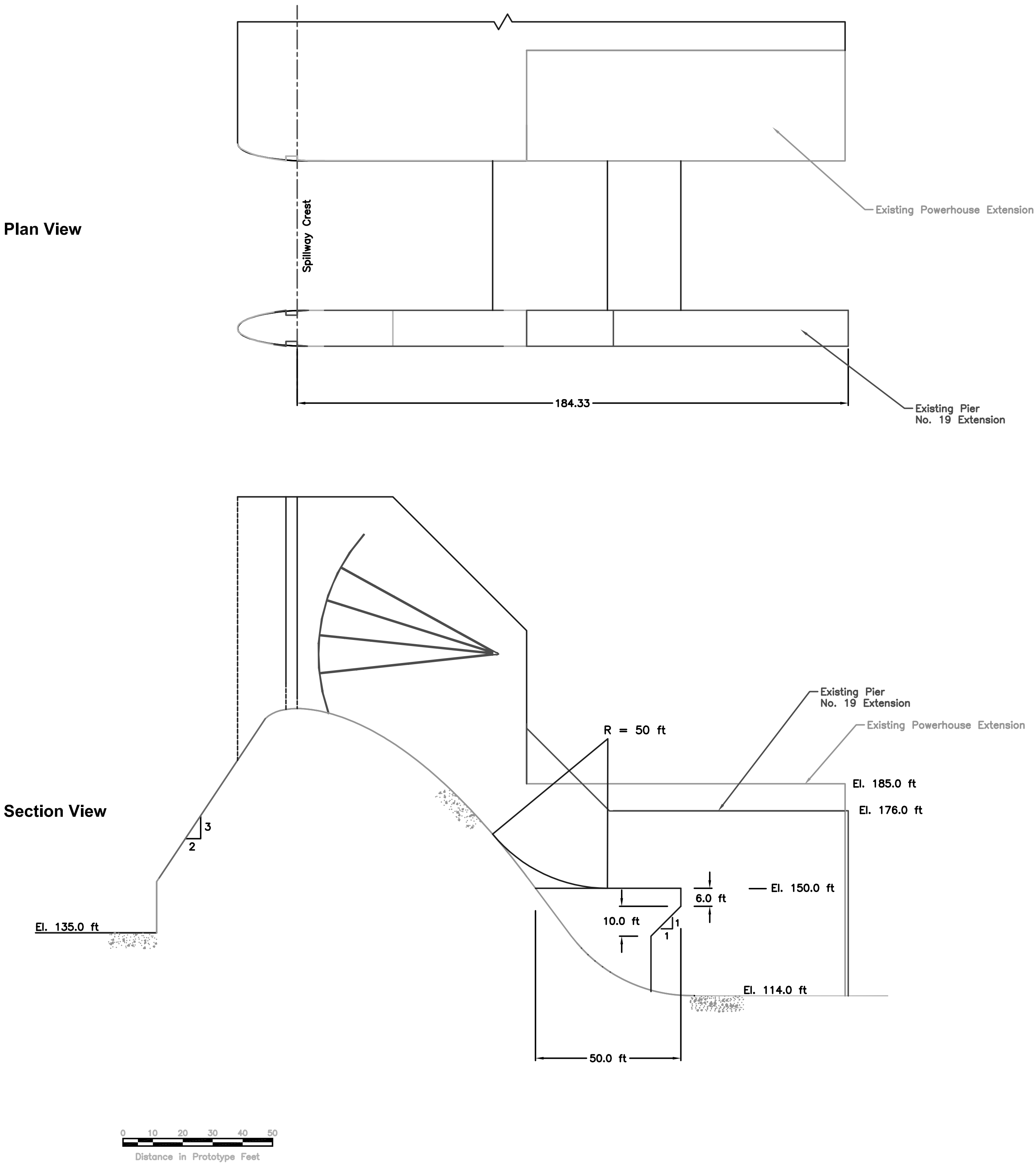
FIGURE 5-6



JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Spillway Bay 20 Deflector

Final Design Geometry

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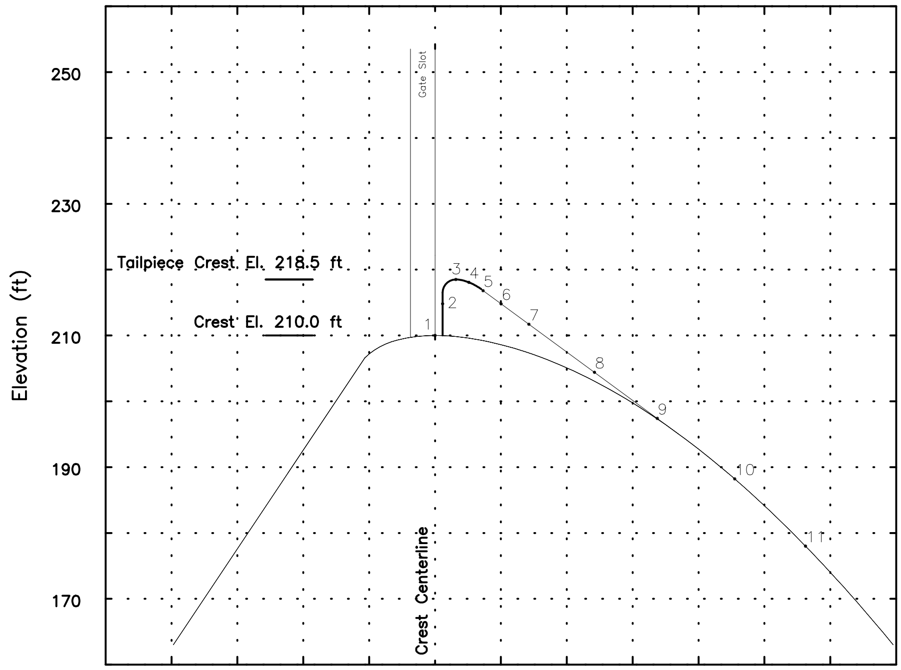
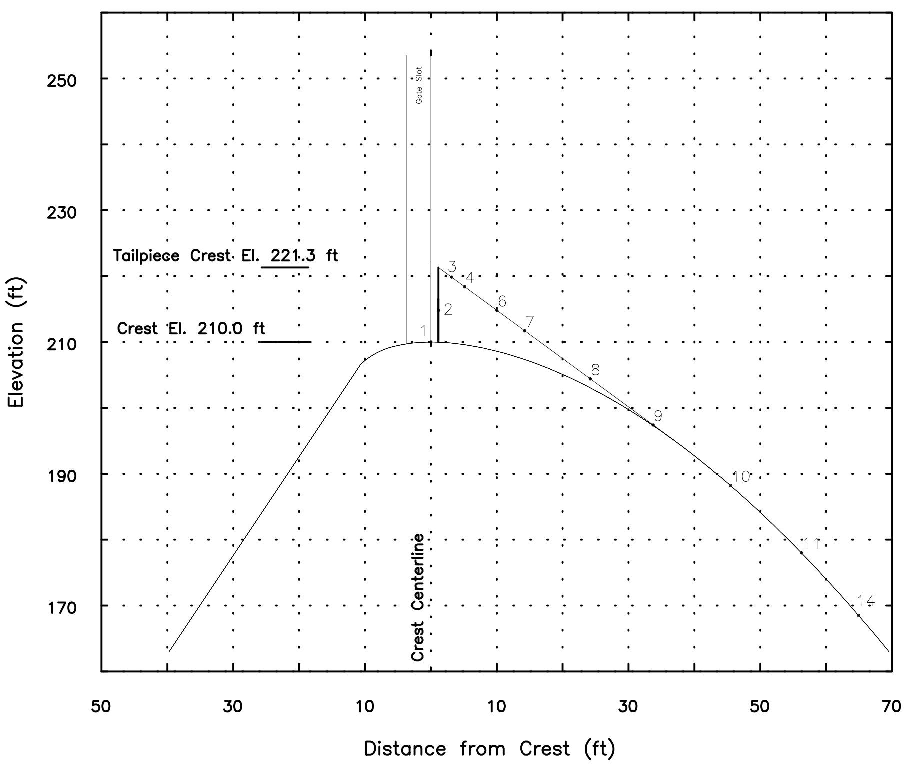
NHCV 1004–137

FIGURE 5-7





Note: Numbers on tailpiece section designate location of piezometers

NHCV 1004–138

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

RSW Tailpiece Section Geometries

Pressure Measurement Locations

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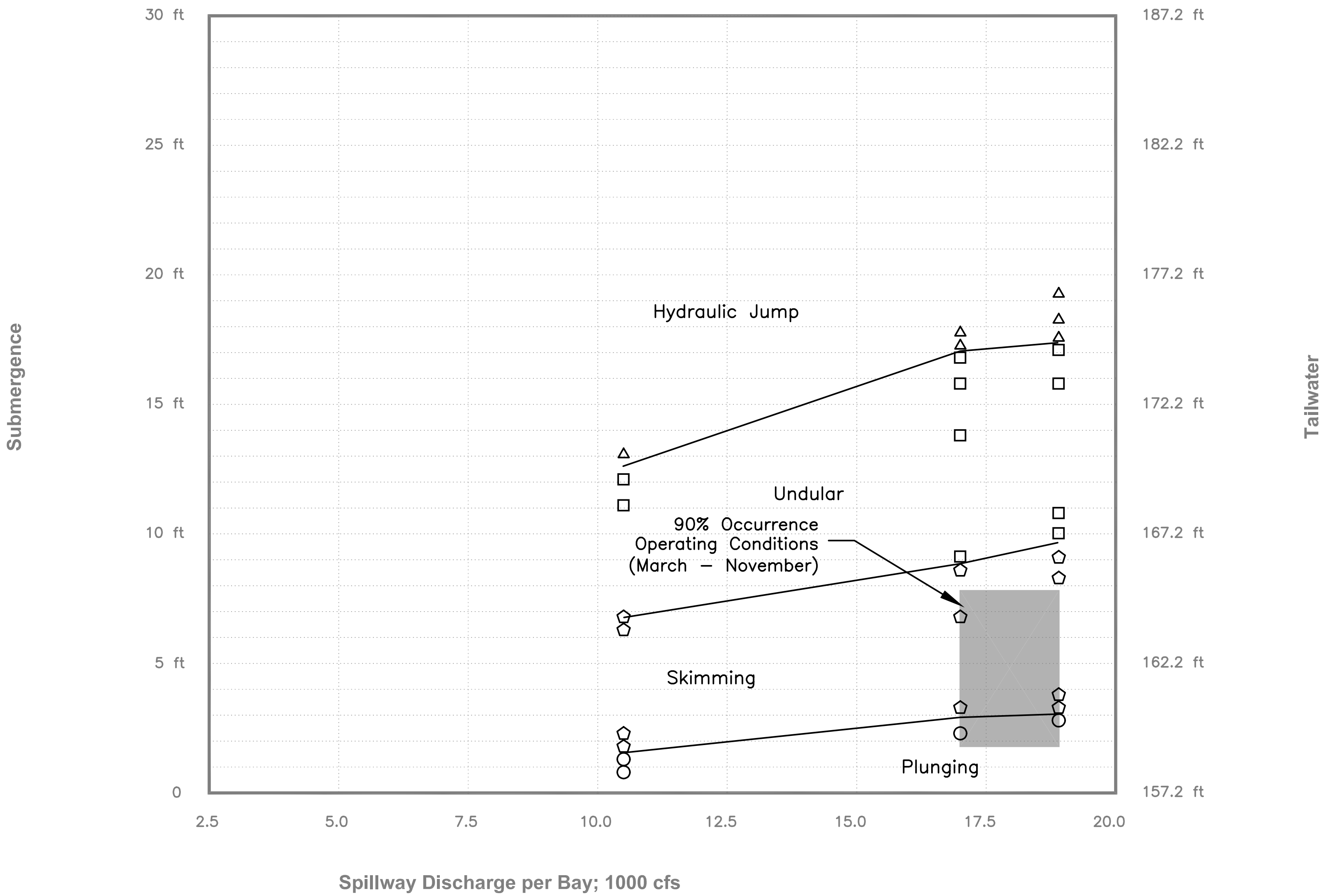
FIGURE 5-8



Notes

1) Performance for center bay in model flume
2) Deflector Length (DL) 34.1 ft with a 49.2 ft Transition Radius (TR)
3) Deflector Elevation (DE); 157.2 ft

Legend (Flow Conditions)

△ Hydraulic Jump
□ Undular
⬠ Skimming
○ Plunging

See Figure 5–1 and 5–2

TEST SBSB
NHCV 1004–118

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Skeleton Bay Surface Bypass Deflector Performance

DL = 34.1 ft; TR = 49.2 ft; DE = 157.2 ft

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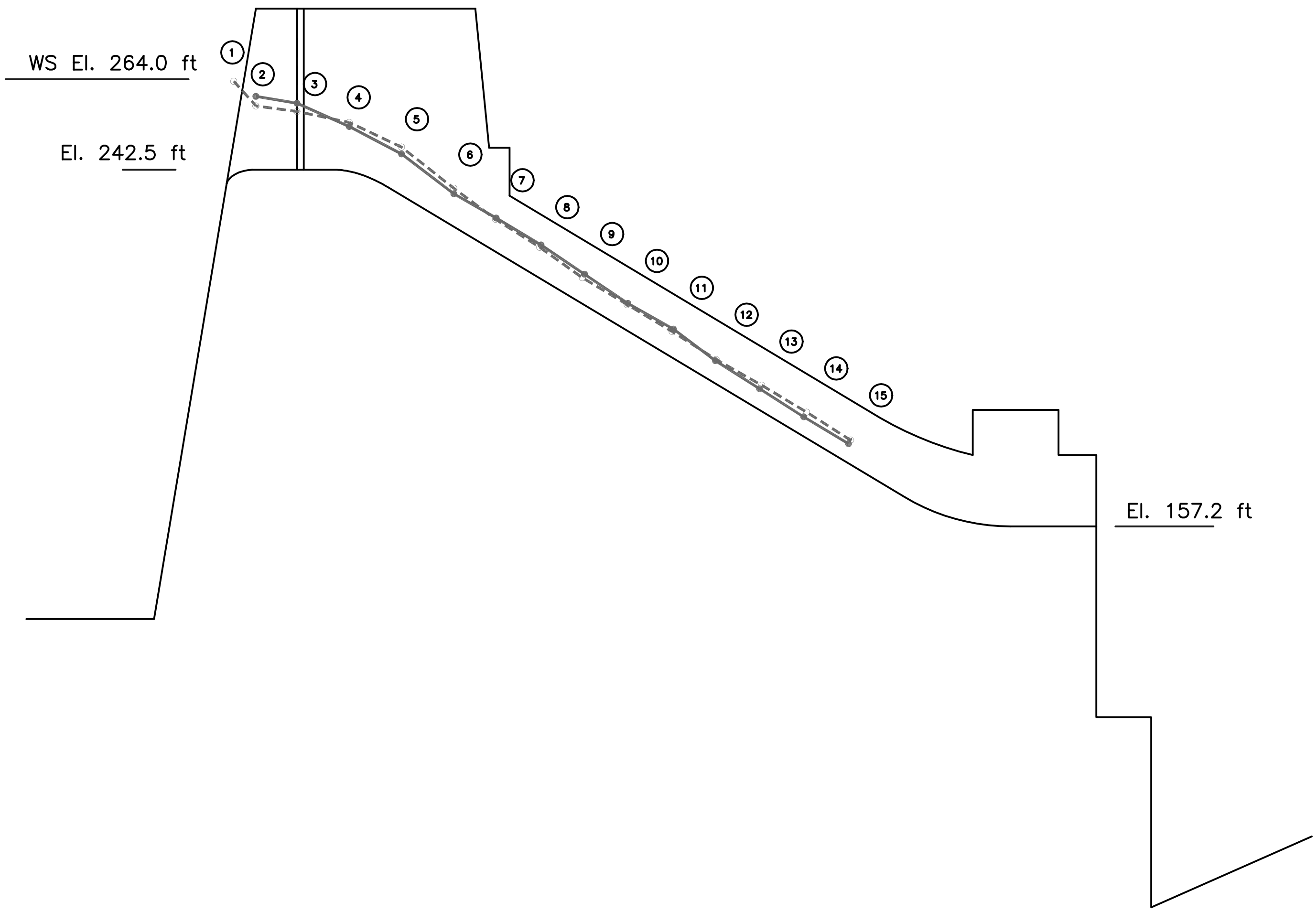
FIGURE 5-9



Notes: 1) Forebay WSE = 264.0 ft
2) Discharge = 18,900 cfs
3) Water surface elevation along the centerline of the bay
4) Water surface elevation along the left pier
5) Data presented in Table 5.7
6) ① Measurement point

NHCV 1004–120

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Skeleton Bay Surface Bypass
Water Surface Profile Down the Chute

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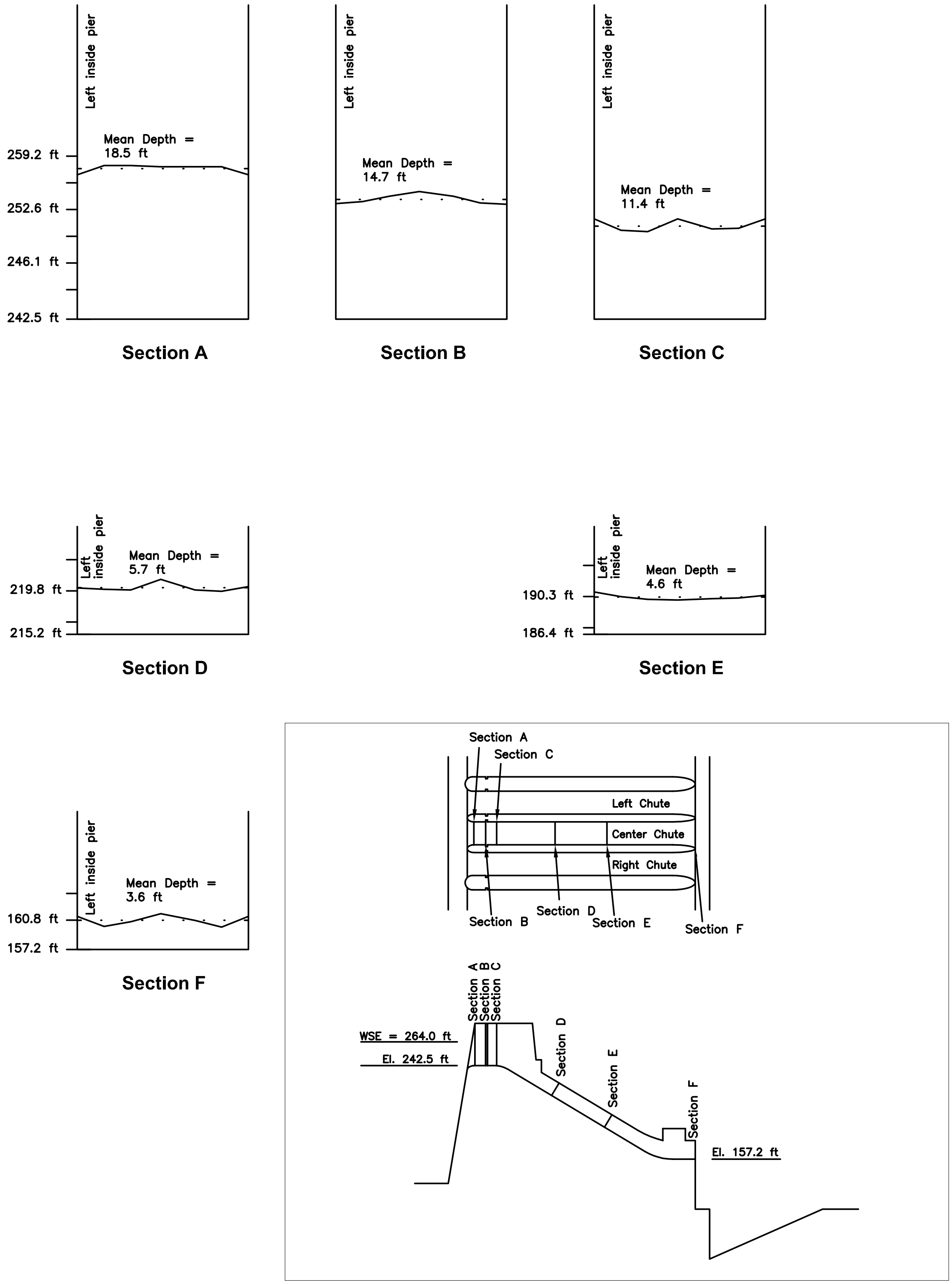
FIGURE 5-10



Notes: 1) Forebay WSE = 264.0 ft
2) Discharge = 18,900 cfs

NHCV 1004–120

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Skeleton Bay Surface Bypass
Water Surface Cross-Sections

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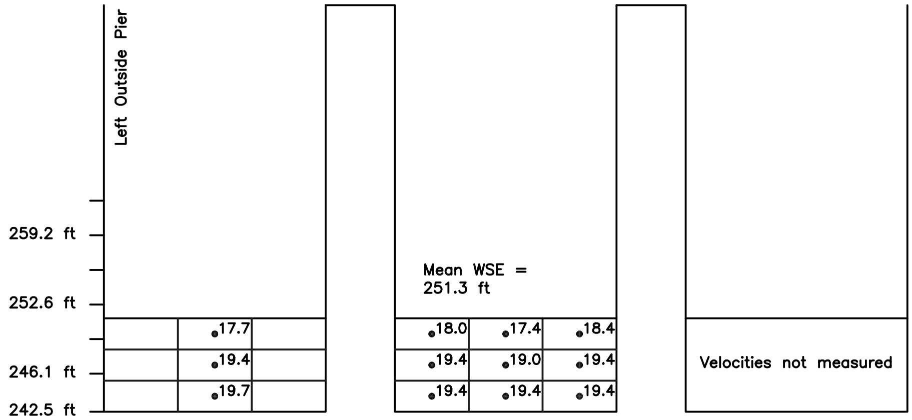
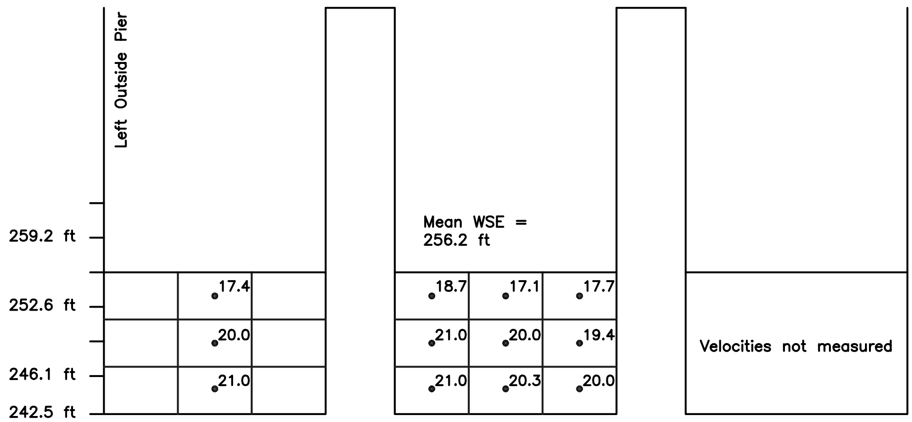
FIGURE 5-11

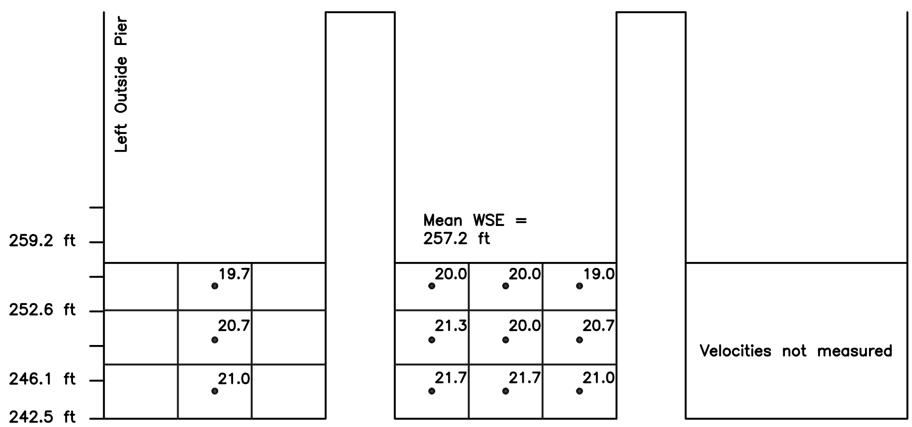
Reservoir WS El. = 257.0 ft
Discharge = 10,500 cfs
Mean Velocity = Q/A = 18.7 ft/s



Reservoir WS El. = 262.5 ft
Discharge = 17,000 cfs
Mean Velocity = Q/A = 19.6 ft/s



Reservoir WS El. = 264.0 ft
Discharge = 18,900 cfs
Mean Velocity = Q/A = 20.3 ft/s



Notes: 1) Velocities presented in ft/s
2) Velocities measured along the upstream edge of the bulkhead slot (Section B; see Figure 5–11)

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Skeleton Bay Surface Bypass
Velocity Cross-Sections

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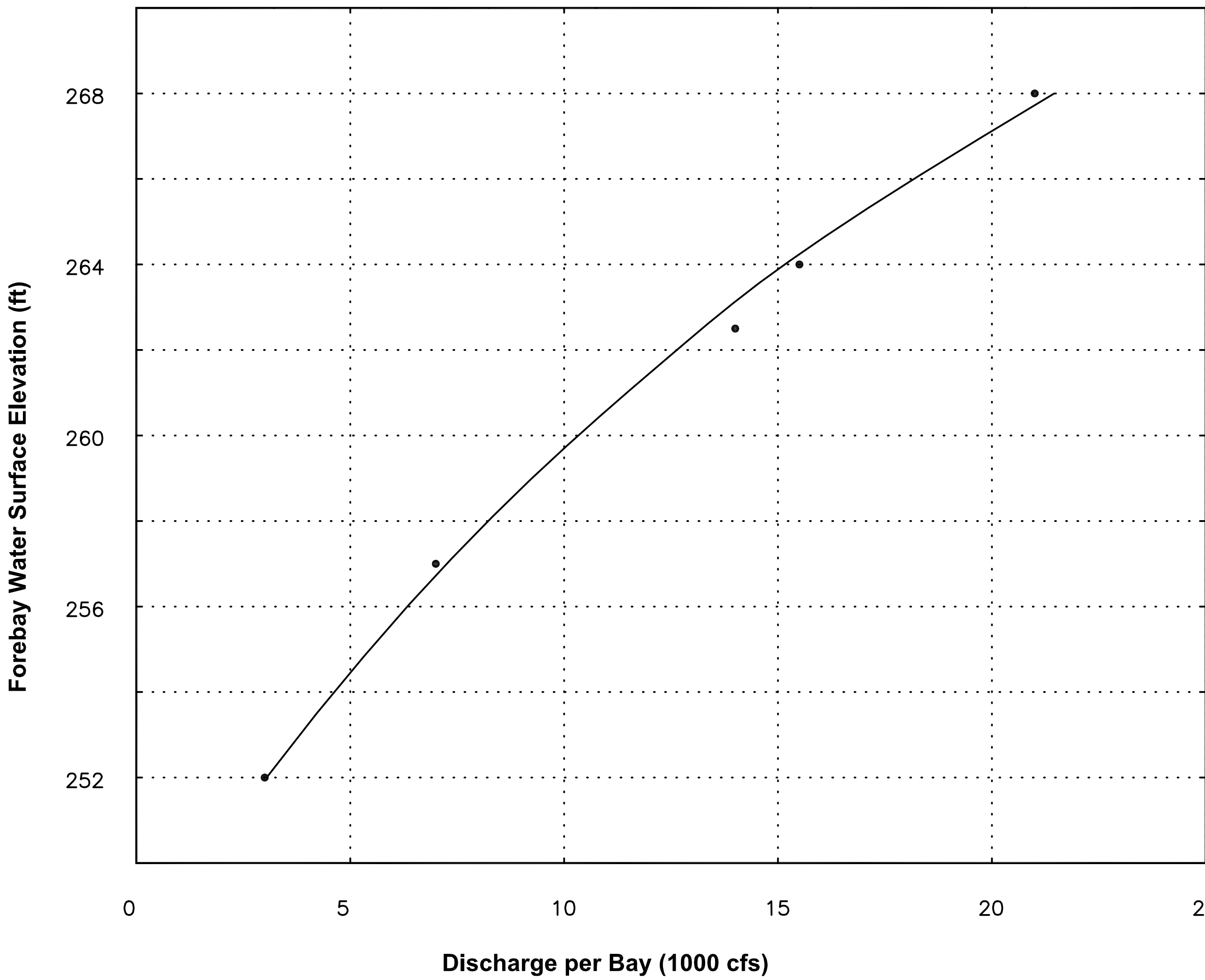
NHCV 1004–120

FIGURE 5-12



Notes: Model Data

NHCV 1004-144

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Removable Spillway Weir

Alt.5 - Final Design Geometry

Discharge Rating Curve

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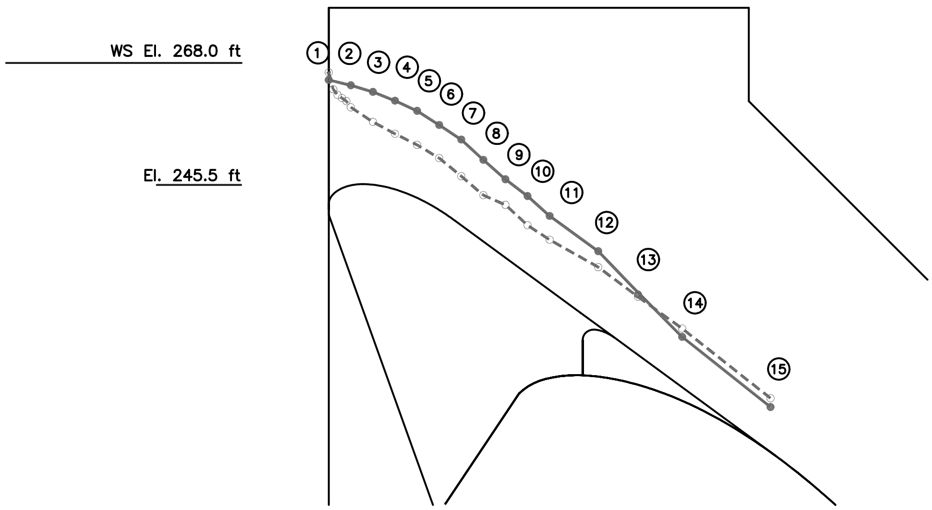
FIGURE 6-1



Discharge = 21,000 cfs

Notes: 1) Water surface elevation along the centerline of the bay
2) Water surface elevation along the left pier
3) Data presented in Table 6.3
4) ① Measurement point

NHCV 1004–133

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Final Design RSW
Water Surface Profile
Forebay WSE = 268.0 ft

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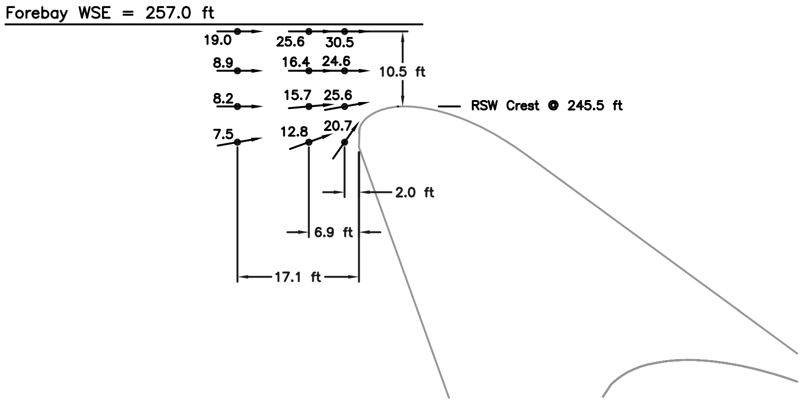
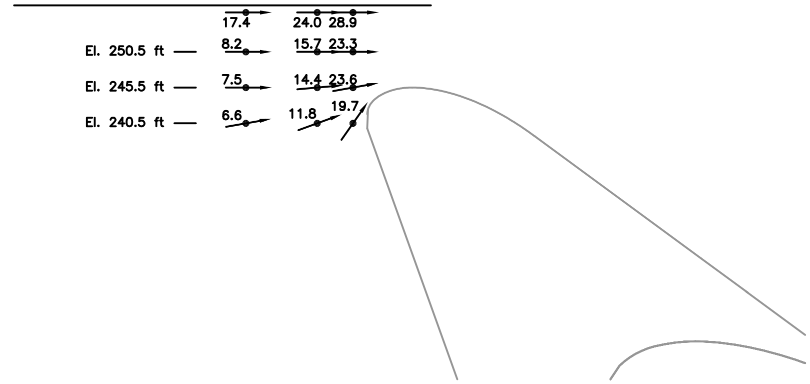
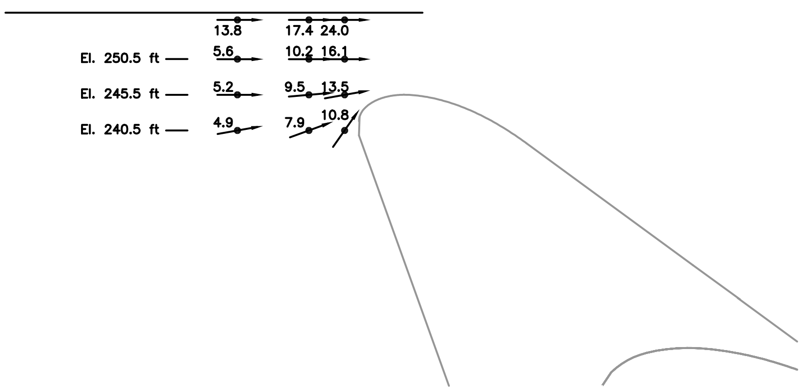
FIGURE 6-2



RSW Centerline



16.7 ft from RSW Centerline



Pier Nose

FLOW CONDITIONS

Pool Elevation: 257.0 ft
Spillway Discharge: 7,000 cfs

NOTES

1) All velocities in prototype ft/s
2) Center of arrow illustrating flow direction represents measurement location
3) See Figure 6–4 for Plan View

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Final Design RSW
Forebay Velocities
Forebay WSE = 257.0 ft - Section View

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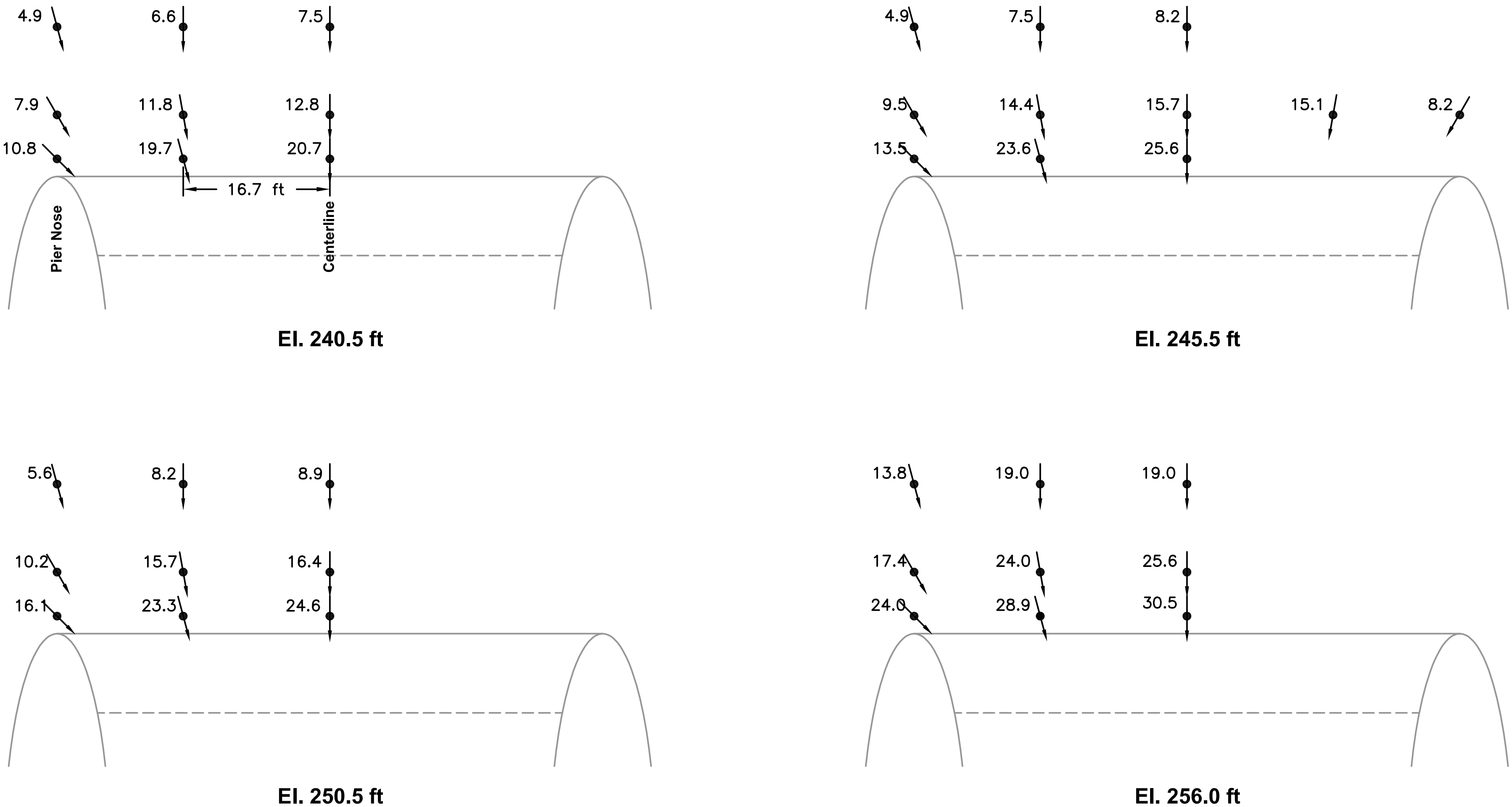
NHCV 1004–127

FIGURE 6-3



FLOW CONDITIONS

Pool Elevation: 257.0 ft

Spillway Discharge: 7,000 cfs

NOTES

1) All velocities in prototype ft/s
2) Center of arrow illustrating flow direction represents measurement location
3) Approach velocity symmetry reflected in data at elevation 245.5 ft., therfore data not measured for other elevations

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Final Design RSW

Forebay Velocities

Forebay WSE = 257.0 ft - Plan View

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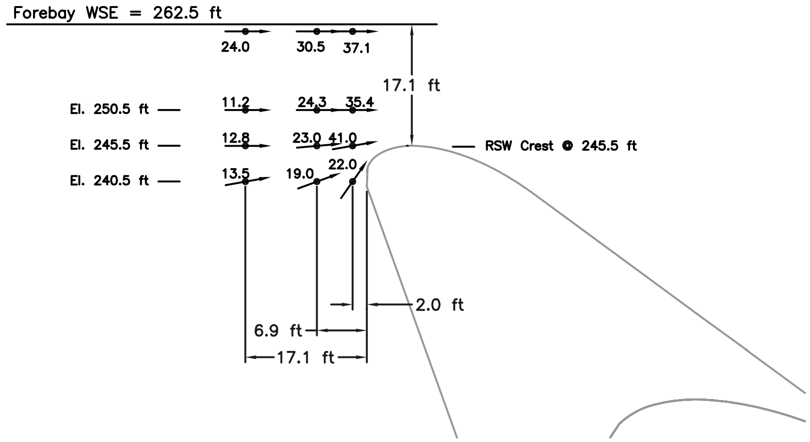
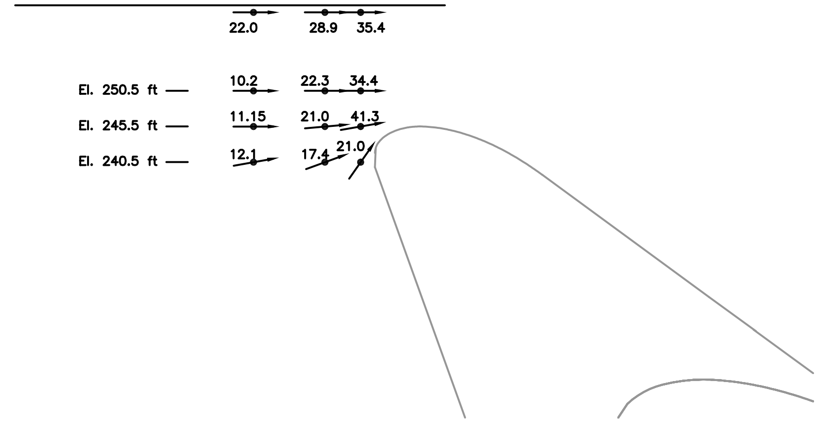
NHCV 1004–128

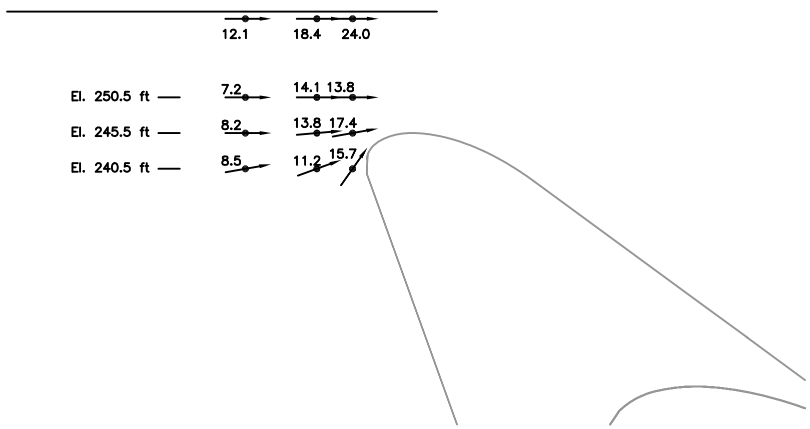
FIGURE 6-4



RSW Centerline



16.7 ft from RSW Centerline



Pier Nose

FLOW CONDITIONS

Pool Elevation: 262.5 ft
Spillway Discharge: 14,000 cfs

NOTES

1) All velocities in prototype ft/s
2) Center of arrow illustrating flow direction represents measurement location
3) See Figure 6–6 for Plan View

NHCV 1004–129

Final Design RSW
HYDRAULIC MODEL STUDY

Final Design RSW
Forebay Velocities
Forebay WSE = 262.5.0 ft - Section View

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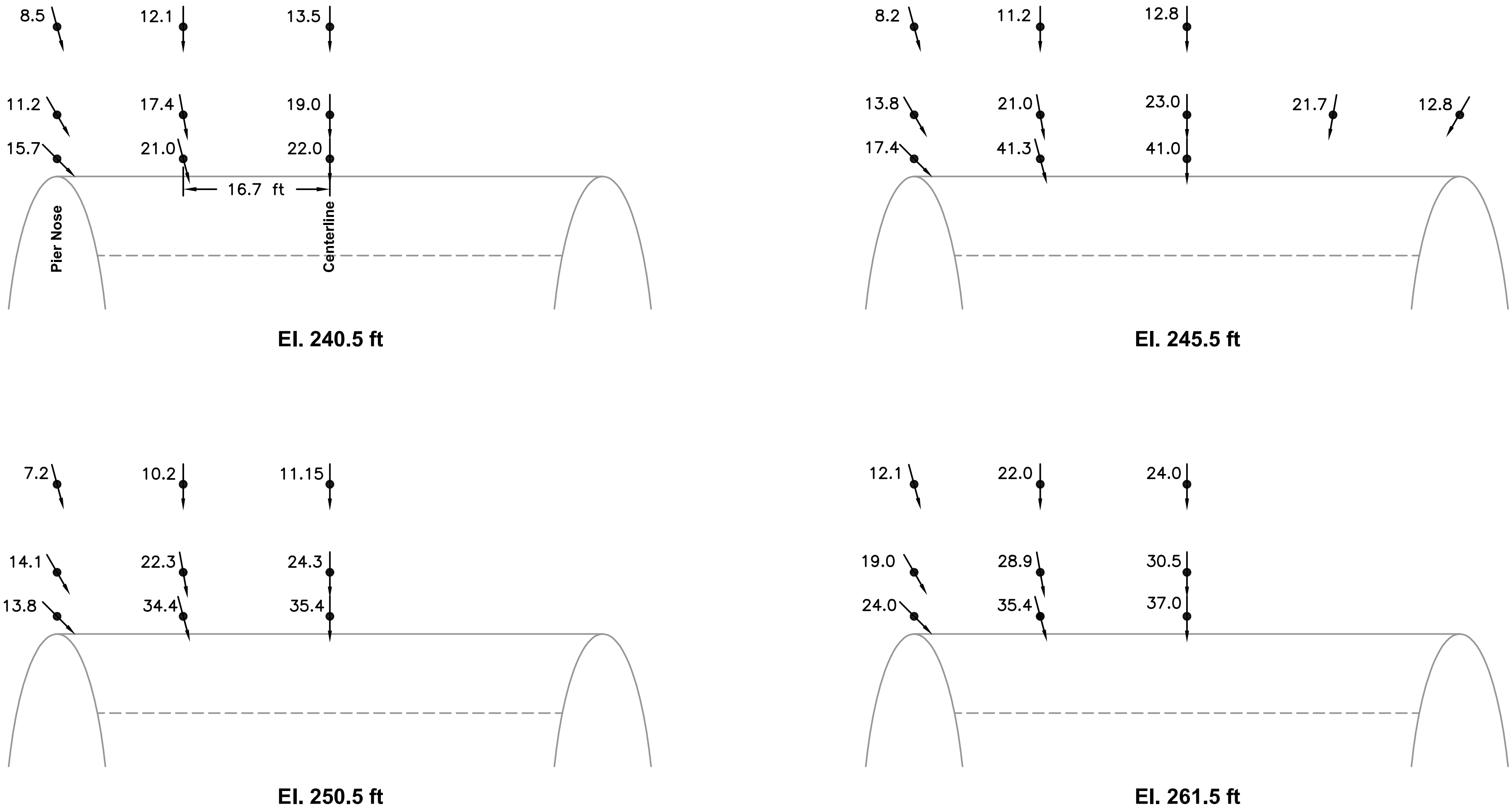
FIGURE 6-5



FLOW CONDITIONS

Pool Elevation: 262.5 ft

Spillway Discharge: 14,000 cfs

NOTES

1) All velocities in prototype ft/s
2) Center of arrow illustrating flow direction represents measurement location
3) Approach velocity symmetry reflected in data at elevation 245.5 ft., therfore data not measured for other elevations

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Final Design RSW

Forebay Velocities

Forebay WSE = 262.5 ft - Plan View

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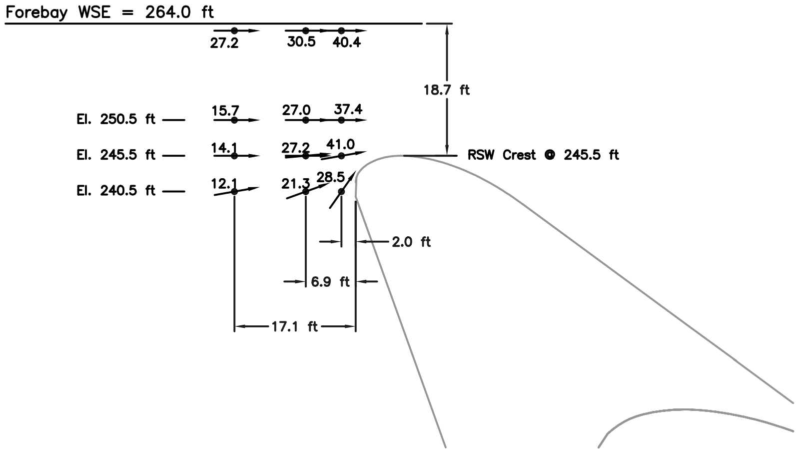
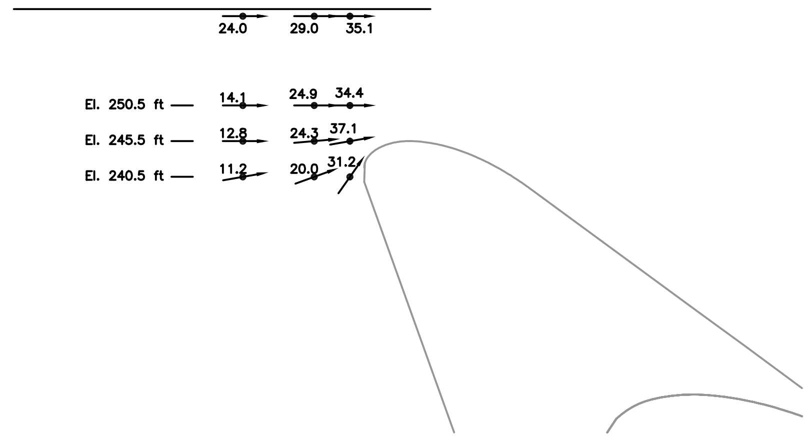
NHCV 1004–130

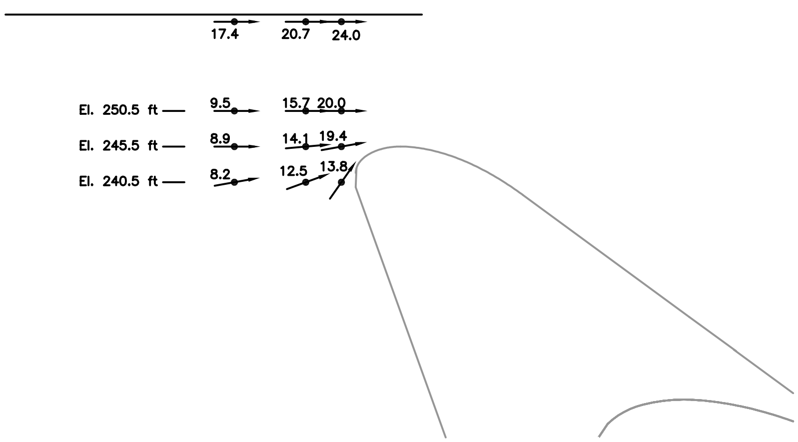
FIGURE 6-6



RSW Centerline



16.7 ft from RSW Centerline



Pier Nose

FLOW CONDITIONS

Pool Elevation: 264.0 ft
Spillway Discharge: 15,500 cfs

NOTES

1) All velocities in prototype ft/s
2) Center of arrow illustrating flow direction represents measurement location
3) See Figure 6–8 for Plan View

NHCV 1004–131

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Final Design RSW
Forebay Velocities
Forebay WSE = 264.0 ft - Section View

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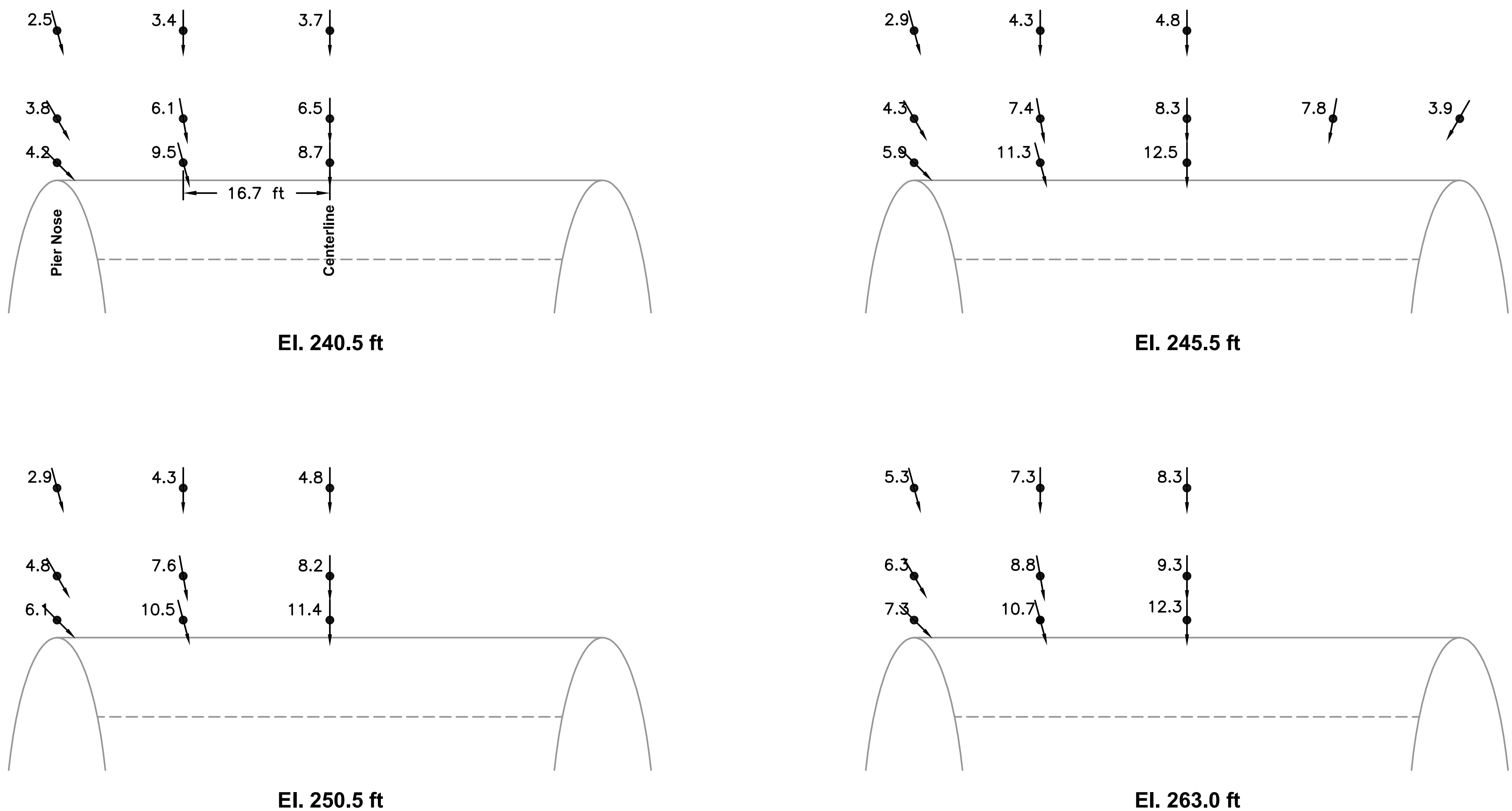
FIGURE 6-7



FLOW CONDITIONS

Pool Elevation: 264.0 ft

Spillway Discharge: 15,500 cfs

NOTES

1) All velocities in prototype ft/s
2) Center of arrow illustrating flow direction represents measurement location
3) Approach velocity symmetry reflected in data at elevation 245.5 ft., therfore data not measured for other elevations

NHCV 1004–132

JOHN DAY DAM SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Final Design RSW

Forebay Velocities

Forebay WSE = 264.0 ft - Plan View

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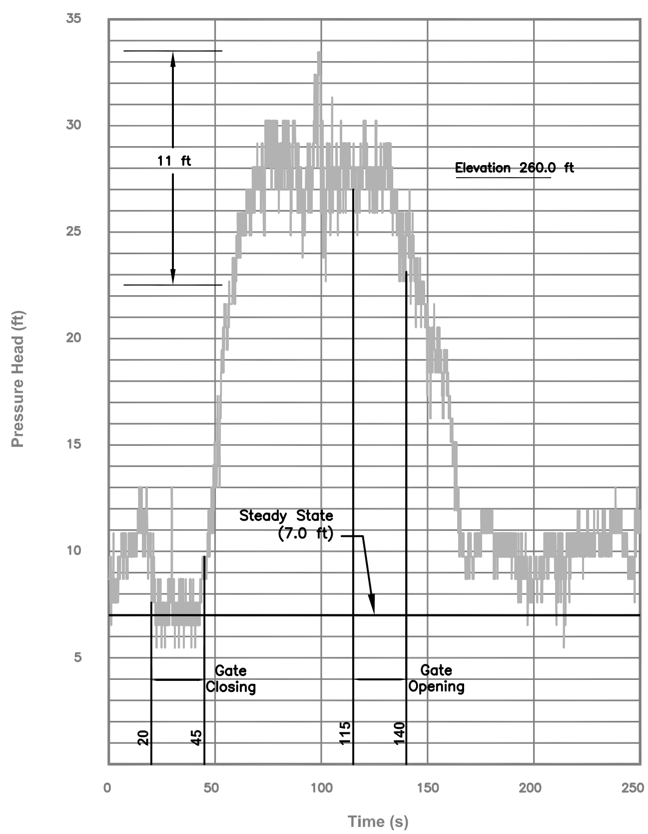
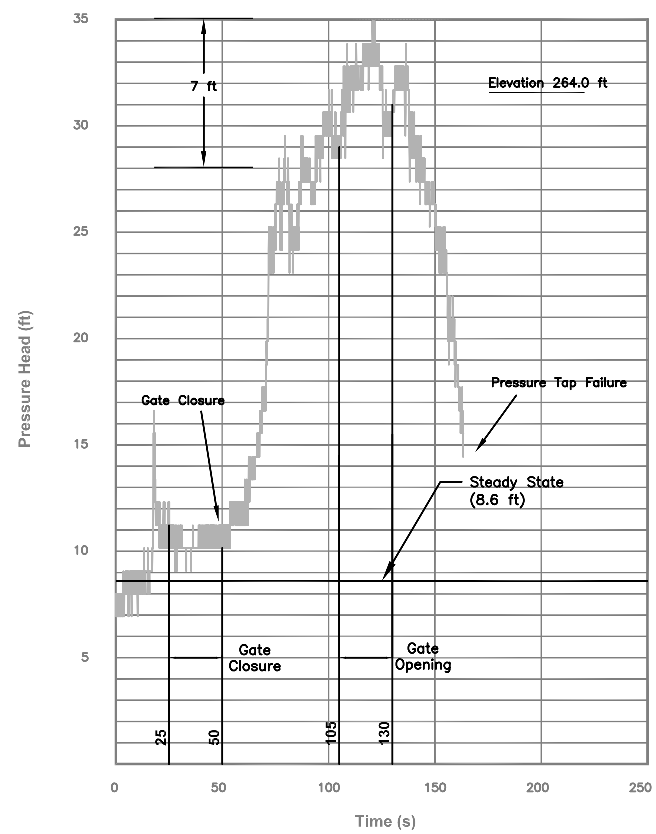
FIGURE 6-8

Forebay WSE = 264.0 ft



Forebay WSE = 268.0 ft



Notes

1) Time in prototye seconds
2) Pressures measured on RSW approximately 20 ft upstream of RSW gate seat on centerline of spillway bay

NHCV 1004–134

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR

HYDRAULIC MODEL STUDY

Time-Variable Pressure on Removable Spillway Weir During Gate Closure and Opening

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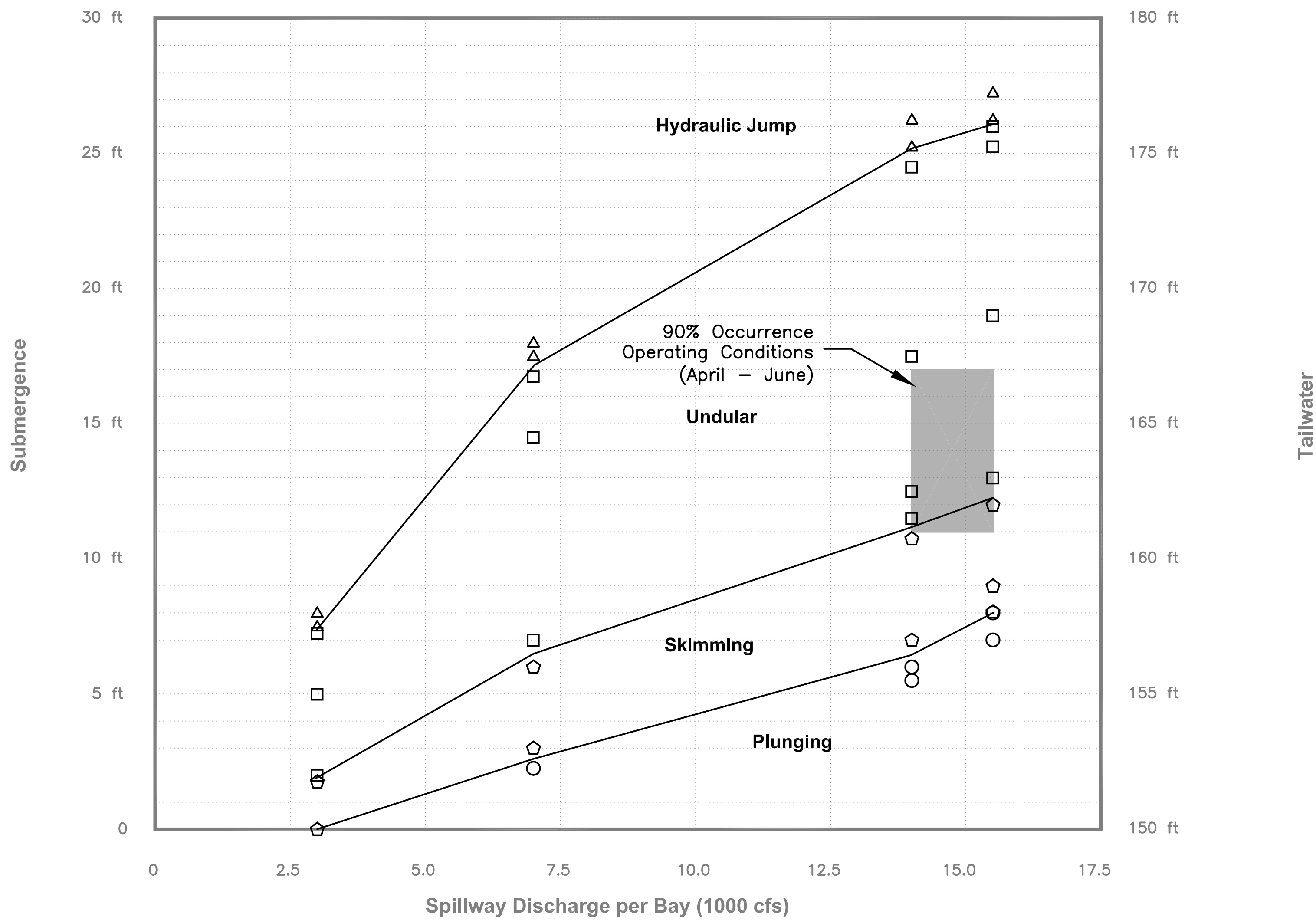
FIGURE 6-9



Notes
1) Performance for full center bay in model flume
2) Deflector Length (DL) of 50.0 ft with a 50 ft Transition Radius (TR)
3) Deflector Elevation (DE) 150.0 ft

Legend (Flow Conditions)
- △ Hydraulic Jump
- □ Undular
- ⬠ Skimming
- ○ Plunging

See Figure 5–1 and 5–2

NHCV 1004–121a

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Spillway Bay 20 Deflector Performance
Final Design Geometry

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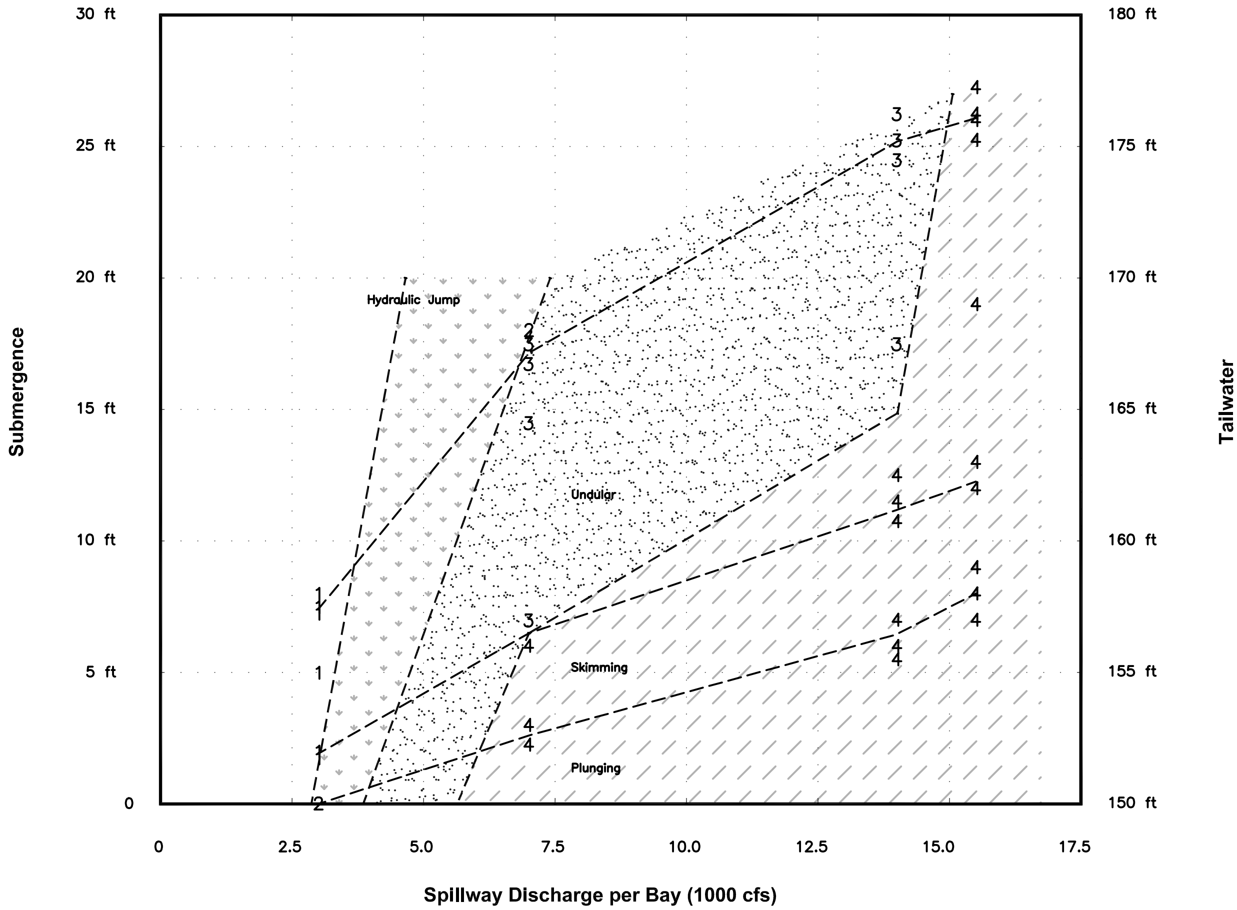
FIGURE 6-10



Levels of Entrained Air

1 No entrained air below the level of the deflector (El. 150 ft)
2 Entrained air penetrates at least 5 ft below the deflector (El. 145 ft), but no significant amount penetrate 15 ft below the deflector (El. 135 ft)
3 Entrained air penetrates at least 15 ft below the deflector (El. 135 ft), but no significant amount penetrate 25 ft below the deflector (El. 125 ft)
4 Entrained air penetrates to the invert of the stilling basin El. 114 ft, however the air concentration is not continuously high at depth
5 Air concentration remains continuously high throughout the entire stilling basin

Note

-- Deflector Performance Curves from Figure 6-10

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
Hydraulic Model Study

Air Entrainment Characteristics

Spillway Bay 20 Final Deflector Geometry

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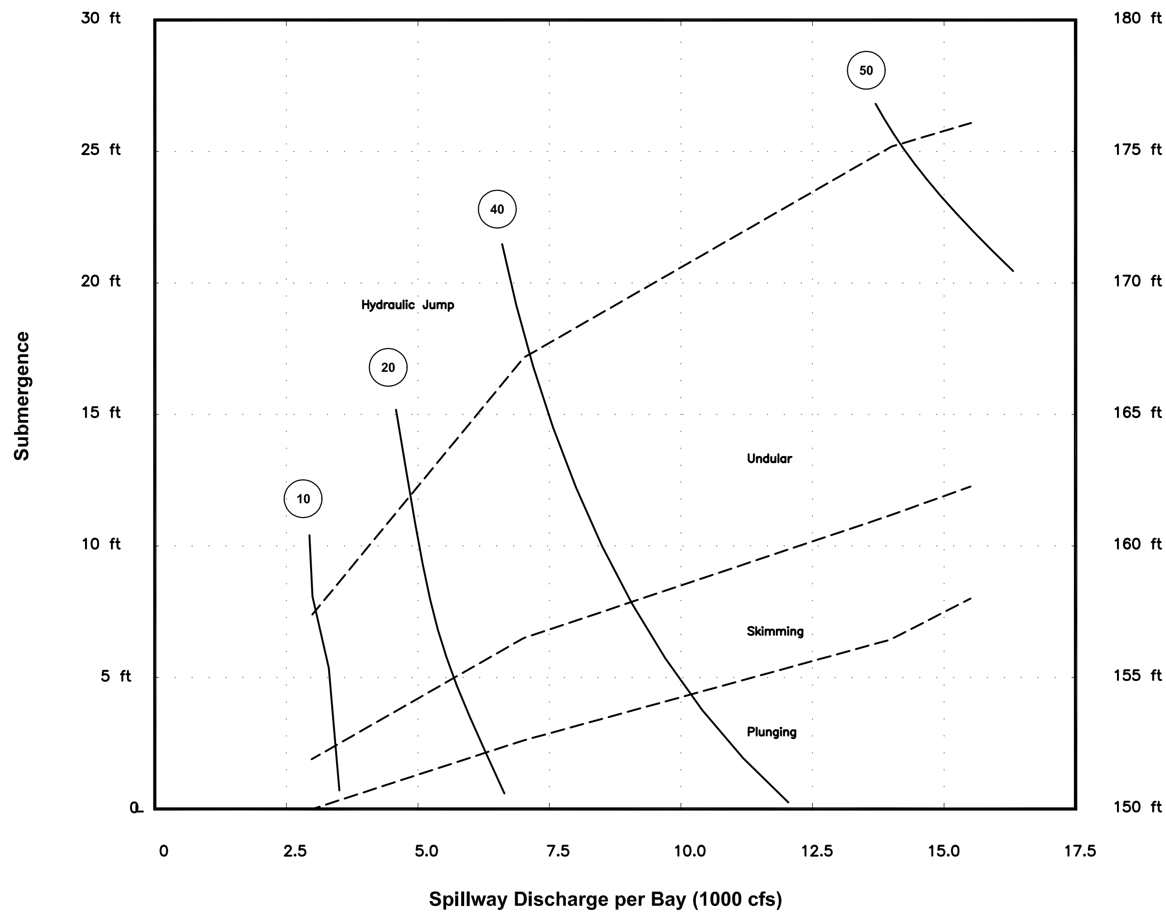
NHCV 1004-125

FIGURE 6-11



Notes

-- Deflector Performance Curves, Figure 6–10

20 — Curve of Air Penetration depth below tailwater water surface (ft)

JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR
HYDRAULIC MODEL STUDY

Depth of Air Penetration

Spillway Bay 20 Final Deflector Geometry

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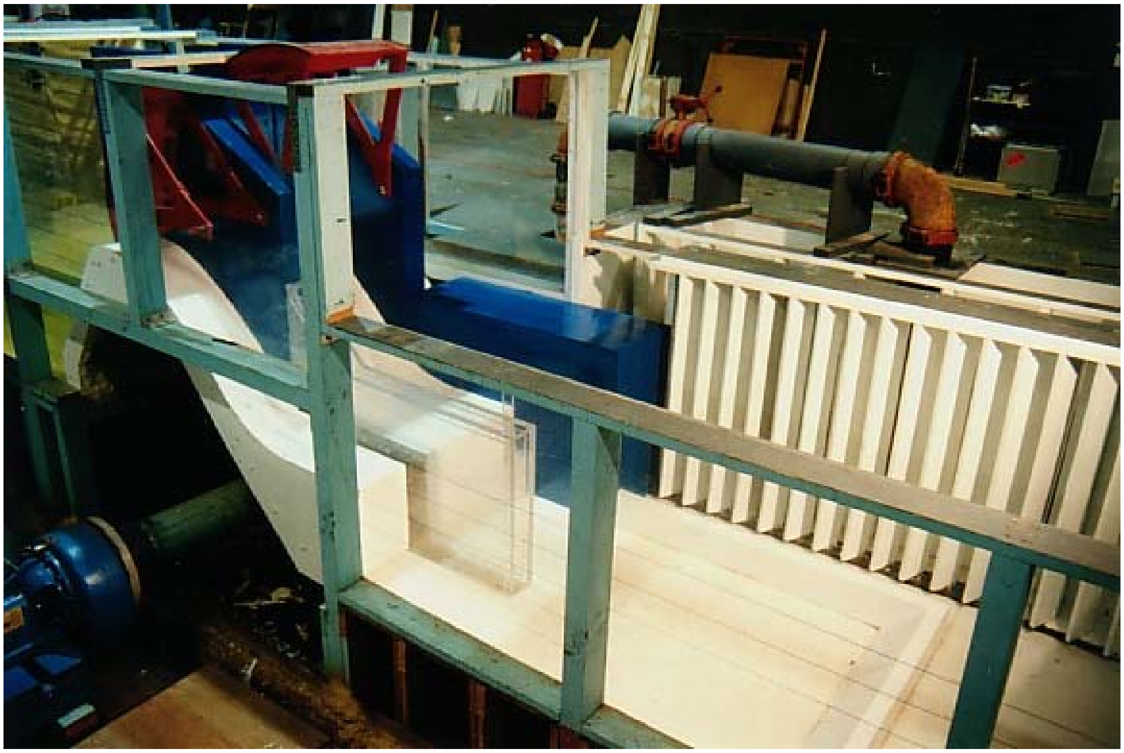
NHCV 1004–126

FIGURE 6-12



1) Model layout showing the John Day section model with the powerhouse entrainment flow modifications. (R19-11)

2) Model layout looking upstream toward the Raised Spillway Weir. (R19-07)

Model #1 – RSW Model

PHOTO PLATE 3-1

1) Looking upstream from the stilling basin to the Skeleton Bay Surface Bypass spillway crest and piers. (R15-15)

2) View from above the forebay looking downstream. (R15-07)

Model #2– Skeleton Bay Surface Bypass

PHOTO PLATE 3-2

1) View of the John Day Bay 20 50 ft deflector geometry with the existing spillway crest geometry. (R22-04)

Model #2 –High Flow Tests

PHOTO PLATE 3-3

1) Alternative 2: View looking downstream at the water surface drawdown around the pier additions.

RSW Alternative 2
Water Surface Drawdown Around Piers

PHOTO PLATE 5-1

1) Alternative 2: View of the water surface patterns looking from upstream of the RSW and existing spillway crest.

RSW Alternative 2
Hydraulic Conditions Over Spillway

PHOTO PLATE 5-2

1) Alternative 4: View of the water surface patterns looking from downstream over the stilling basin. Note the 2 ft step at the transition between the RSW and the existing spillway crest.

RSW Alternative 4
Hydraulic Conditions Over Spillway

PHOTO PLATE 5-3

1) Alternative 5: View of the water surface patterns looking from downstream over the stilling basin. Note the relatively smooth water surface when compared to the flow patterns over Alternatives 2 and 4.

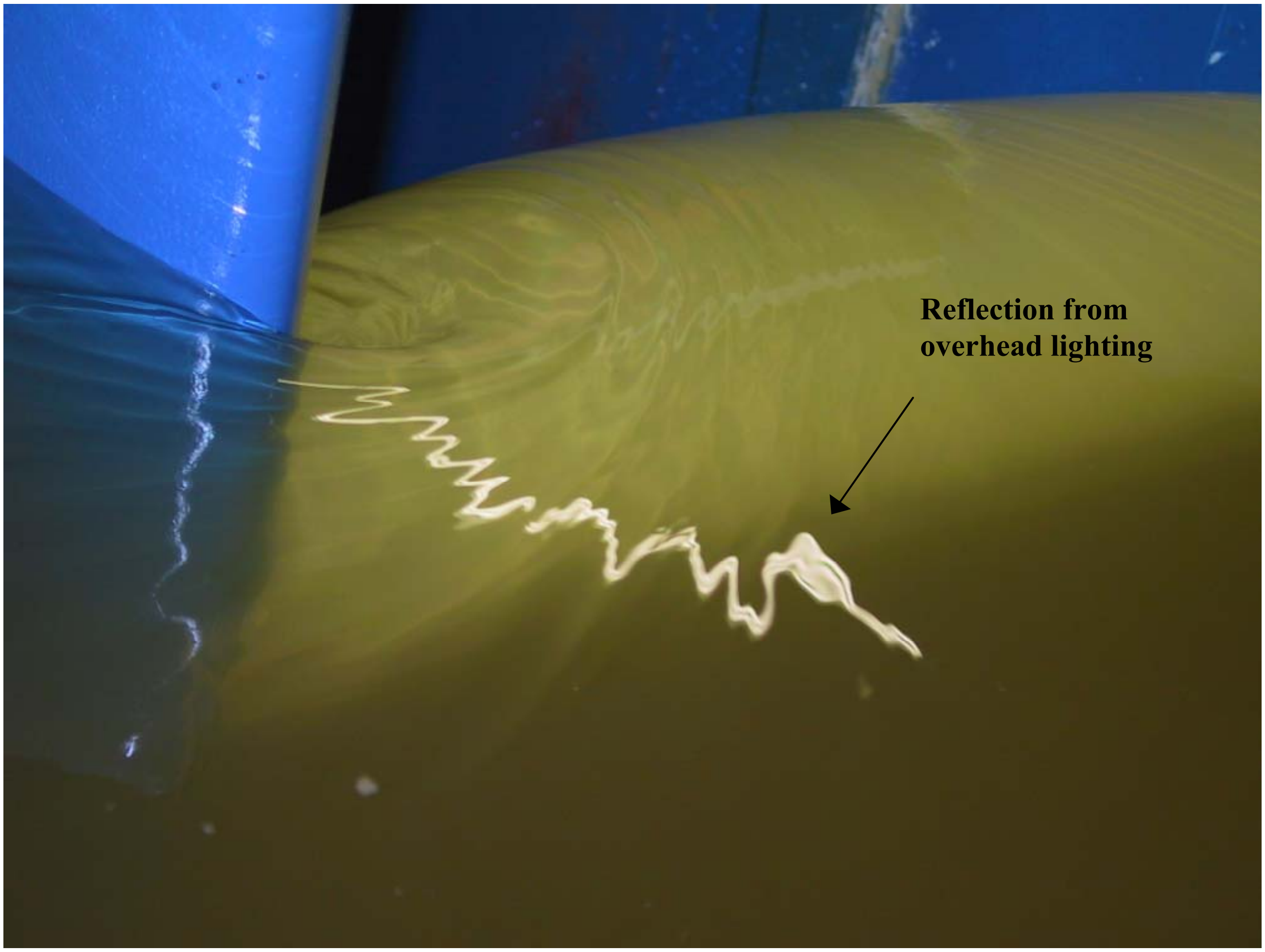
RSW Alternative 5
Hydraulic Conditions Over Spillway

PHOTO PLATE 5-4

1) Alternative 5: View of the water surface patterns looking from upstream of the RSW and existing spillway crest.

RSW Alternative 5
Hydraulic Conditions Over Spillway

PHOTO PLATE 5-5



1) Alternative 5: View looking downstream at the water surface drawdown around the pier additions.

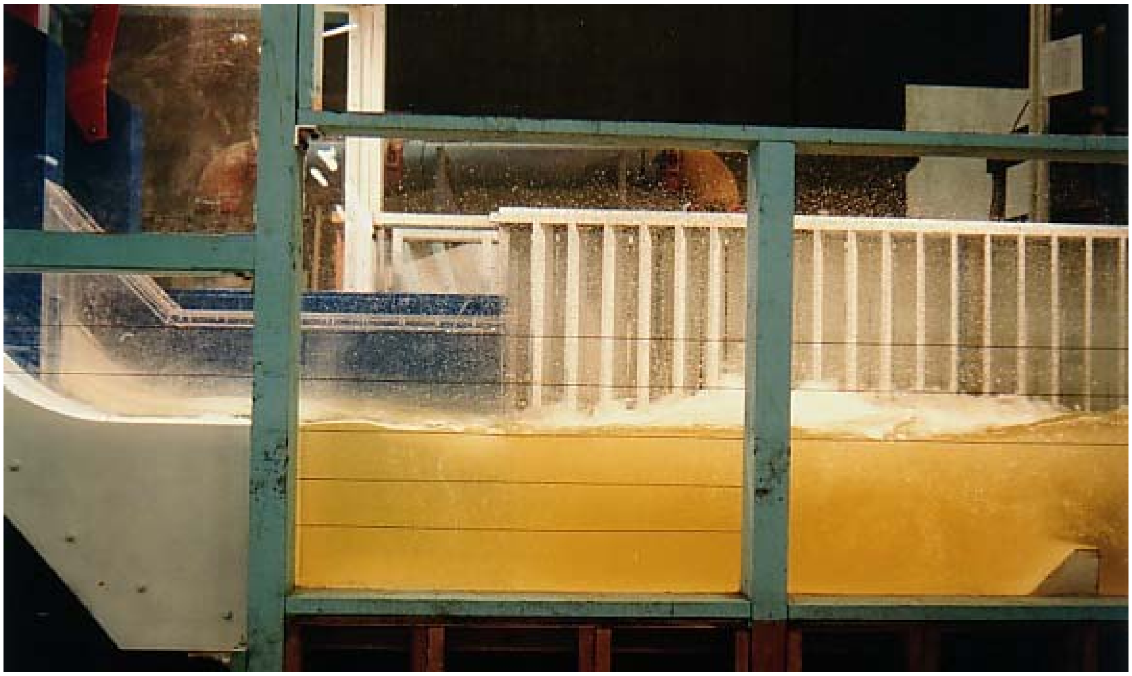
RSW Alternative 5
Water Surface Drawdown Around Piers

PHOTO PLATE 5-6

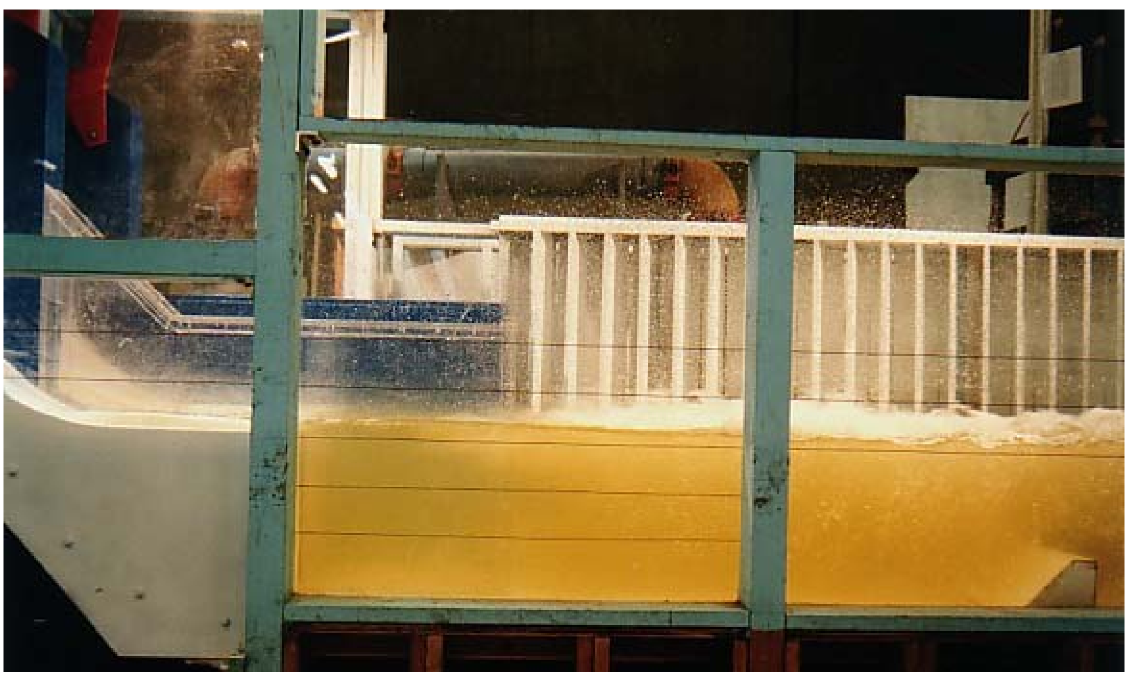
1) Alternative 7: View of the water surface patterns looking from upstream of the RSW and existing spillway crest.

RSW Alternative 7
Hydraulic Conditions over Spillway

PHOTO PLATE 5-7



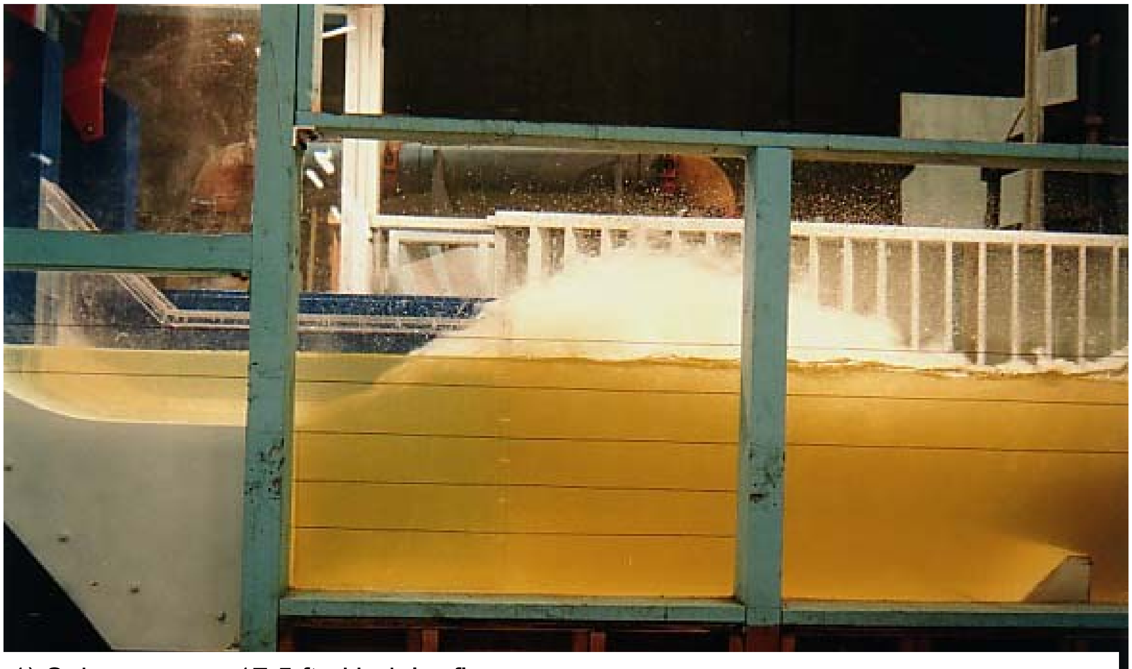
1) Submergence 5.5 ft: Plunging flow. (R18-24)



2) Submergence 7.0 ft: Skimming flow. (R18-26)

Final Design Spillway Bay 20 Deflector Performance
Plunging and Skimming Conditions
Forebay WSE = 262.5 ft; Q = 14,000 cfs

PHOTO PLATE 6-1



1) Submergence 17.5 ft: Undular flow. (R18-28)

2) Submergence 26.3 ft: Hydraulic jump over the deflector. (R18-31)

Final Design Spillway Bay 20 Deflector Performance
Undular and Hydraulic Jump Conditions
Forebay WSE = 262.5 ft; Q = 14,000 cfs

PHOTO PLATE 6-2

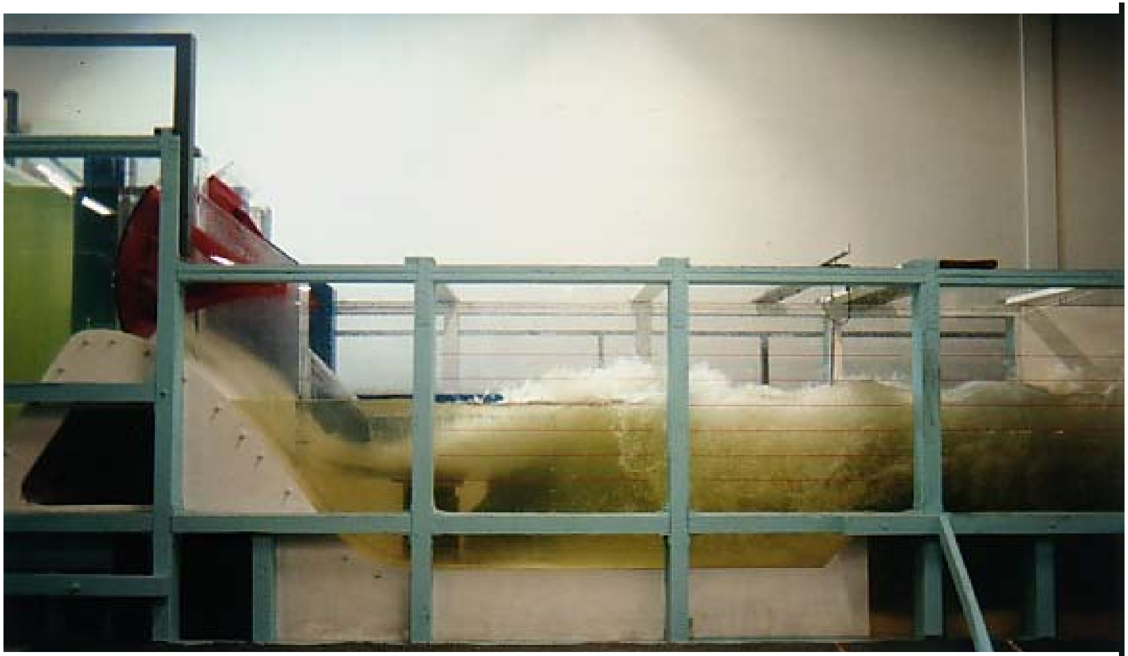
1) 30 ft Deflector
Discharge 40,000 cfs/bay (800,000 cfs project discharge);
Forebay Water Surface Elevation 268.0 ft
Tailwater Elevation 173.0 ft. (R2A-23)

2) 75 ft Deflector
Discharge 40,000 cfs/bay (800,000 cfs project discharge);
Forebay Water Surface Elevation 268.0 ft
Tailwater Elevation 173.0 ft. (R2A-23)

Spillway Bay 20 Deflector Performance
30 and 75 ft Long Deflectors
Gated Flow- Skimming Flow Conditions
40,000 cfs/bay
PHOTO PLATE 6-3

1) 30 ft Deflector

Discharge 40,000 cfs/bay (800,000 cfs project discharge)
Forebay Water Surface Elevation 268.0 ft
Tailwater Elevation 188.5 ft. (R2A-26)



2) 75 ft Deflector

Discharge 40,000 cfs/bay (800,000 cfs project discharge)
Forebay Water Surface Elevation 268.0 ft
Tailwater Elevation 186.5 ft. (R2A-11)

Spillway Bay 20 Deflector Performance
30 and 75 ft Long Deflectors
Gated Flow – Undular Flow Conditions
40,000 cfs/bay

PHOTO PLATE 6-4

1) 30 ft Deflector

Discharge 112,500 cfs/bay (2,250,000 cfs project discharge)
Forebay Water Surface Elevation 276.0 ft
Tailwater Elevation 206.0 ft. (R22-02)

1) 50 ft Deflector

Discharge 112,500 cfs/bay (2,250,000 cfs project discharge)
Forebay Water Surface Elevation 276.0 ft
Tailwater Elevation 210.0 ft. (R22-14)

Spillway Bay 20 Deflector Performance
30 and 50 ft Long Deflectors
Spillway Design Flood
PHOTO PLATE 6-5

1) Existing Conditions (no deflector)
Discharge 112,500 cfs/bay (2,250,000 cfs project discharge)
Forebay Water Surface Elevation 276.0 ft
Tailwater Elevation 206.0 ft. (R30-02)

2) Final Design 50 ft Deflector
Discharge 112,500 cfs/bay (2,250,000 cfs project discharge)
Forebay Water Surface Elevation 276.0 ft
Tailwater Elevation 206.0 ft. (R30-14)

Spillway Bay 20 Performance
No Deflector and Final Design 50 ft Long Deflector
Spillway Design Flood
PHOTO PLATE 6-6

DETAILED STATEMENT OF WORK FOR
CONTRACT NO. DACW57-97-D-0004, CH2M HILL/MONTGOMERY WATSON (JV)

TASK ORDER CASE NO. 14: JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR DESIGN DOCUMENTATION REPORT

1. PROJECT INFORMATION. The John Day Dam Project, operated and maintained by Portland District, U.S. Army Corps of Engineers (CENWP), is located approximately 95 miles east of Portland, Oregon, at River Mile 215.6 on the Columbia River. The John Day Powerhouse began operation in 1968. The powerhouse has been modified in recent years to enhance the downstream migration of juvenile salmonids. Studies for enhancements to the existing Juvenile Bypass System (JBS) in addition to surface collection will be concurrent with this work. When the John Day Project was constructed, the Powerhouse was constructed with 16 units for power generation, 20 spillway bays for flood control and four empty units for future power generation. The four units for future power generation, Units 17 through 20, were identified as Skeleton Units. These units are open pits with a minimum structure to allow for future modifications for the insertion of generating units.

2. SURFACE COLLECTION AND BYPASS SYSTEM PROGRAM.

a. Juvenile salmonid bypass system development on the lower Columbia River has evolved considerably over the last several decades. Primary regional direction to improve juvenile salmonid survival past hydroelectric facilities within the Columbia River basin consisted of construction of mechanical screened bypass systems. These systems were designed to guide downstream migrants within the turbine intakes away from turbine passage and into a bypass system. As screened bypass systems were evaluated, the regional goals of non-turbine passage were not always met. To enhance the screened bypass systems, the region has also initiated spill programs to further enhance non-turbine passage and presumably survival of juvenile salmonids. There are presently three possible passage routes for downstream migrant juvenile salmonids past the John Day project. They can either be guided by the existing mechanical screened bypass system, be passed through voluntary (or non-voluntary) spill, or pass through the turbines. Several new juvenile salmonid passage programs are either being implemented or are in the planning phase for possible implementation at John Day. Although all of the existing or planned programs are intended to improve juvenile salmonid survival, several issues have yet to be verified. The main goal is passage with an acceptable level of survival; however, not all existing passage routes have been evaluated for survival. Also, with voluntary spill to pass downstream migrants, issues such as increased total dissolved gas and lost power production/reliability are still being evaluated.

b. Beginning in 1995 the Corps of Engineers along with input from regional fishery managers began a Surface Flow Bypass (SFB) program (Corps 1995, Harza 1995). This program was intended to look at possible ways to bypass downstream migrant salmonids with surface oriented flows. As different SFB concepts were evaluated, the primary focus for the John Day project was the possible use one or more skeleton bays located between the operational powerhouse turbines and spillway. In 1998 the Corps completed a Feature Design Memorandum (Corps 1998) outlining the use of the skeleton bays as a possible Surface Bypass Spillway (SBS) for juvenile salmonids. After review of the SBS FDM, the regional System Configuration Team (SCT) decided the cost of constructing the skeleton bay SBS were too high given the uncertainty of "proof of concept". In 1999 the SCT requested the Corps evaluate two possible directions for

SBS at John Day. One was to evaluate the use of four skeleton bays as a possible SBS; the second was to evaluate the use of a spillway bay to see if a less costly test of the SBS concept was possible. As the Corps evaluated the use of a spillbay for a possible prototype SBS, an issue of flood spill capability and loss thereof also was raised. This possible loss in spillway capacity made the permanent modification of a spillway bay unpractical (capacity would be required elsewhere if it is removed from the spillway, and costs for additional capacity are high).

c. A Removable Spillway Weir (RSW) was conceptually designed as an option in the Surface Bypass Collection System Combinations Report, Lower Snake River (completed in 1998). A RSW is a hollow steel structure that is filled with air for floating and towing into place. In the vicinity of the spillway bay, selective filling of the structure would occur to rotate the structure to vertical. Once vertical, the RSW can be moved into place, further submerged until it rests on support brackets permanently mounted on the spillway. The existing spillway tainter gate would still accomplish flow control. This design has applications for use at John Day Dam, and could serve extremely well as a prototype test of the efficiency of a high flow surface bypass spillway. To serve as a "proof of concept" for a Skeleton Bay Surface Bypass Spillway, the RSW should be located as close to the skeleton bays as possible in spillway bay 20, and should be designed to have similar flow attraction characteristics as the skeleton bay surface bypass spillway. Being removable, an alternative means of passing the spillway design flood would not have to be considered.

d. The RSW could be designed as either a "proof of concept" for the SBS or could be designed to be a permanent bypass. The potential for a different geometry is possible if the RSW is not required to mimic the skeleton Bay SBS.

e. Construction of flow deflectors in bays 2 through 19 was completed in February 1998. Flow deflectors in bays 1 and 20 were not completed due to the uncertainty in the deflector elevation, potential Total Dissolved Gas (TDG) reduction, and potential negative effects to the fish ladder entrance conditions on the north and south side of the spillways. These deflectors are currently being evaluated for installation in a future construction contract, but a single deflector in bay 20 will be designed to test the extended deflector as designed in the Skeleton Bay Surface Bypass Spillway as outlined in FDM No. 52.

f. To simulate the skeleton bay surface bypass spillway and to test the skeleton bay surface bypass spillway outlet, an extended spillway deflector will be designed as part of the RSW design. The extended flow deflector will be designed for dual purpose; to operate with or without a RSW. The geometry of the flow deflector will be similar to the option evaluated in NHC's "John Day Dam Sluice Model, Hydraulic Model Study Report, May 1999". The length of the extended deflector shall be 30 feet with an outlet elevation that will give the optimum TDG benefits during operation of the spillway bay with and without the RSW.

g. The RSW should be designed so that there is minimal work on the spillway to attach the weir to the existing spillway bay concrete. The RSW should be totally removed from the spillway bay in a relatively short period of time such that the entire spillway flood design capacity is not affected (it is expected that this process will take less than a day to perform). During the non-spill portion of the year, the RSW could be removed for normal spillway operation or maintenance to the RSW.

h. The NWW RSW shown in Appendix 1 has an overflow depth of approximately 10 feet at maximum pool. The proposed NWP overflow depth for the "proof of concept" RSW is 22.5-foot.

i. Due to the fact that the existing tainter gate will be used for flow control, a "double" ogee flow profile is unavoidable. The RSW will be constructed with an ogee shape intersecting the existing spillway bay crest. From there, the existing spillway bay will lead flow down to the flow deflector. The point where the RSW ogee intersects the existing spillway bay crest will create a condition similar to the radius at the existing flow deflectors, which will have to be addressed for it's potential for physical injury.

3. MANDATORY WORK TASKS. The Contractor shall complete the work as described herein. The following tasks shall be accomplished:

a. Quality Control Plan (QCP). Within 30 days of notice to proceed, the Contractor shall submit a QCP. The QCP shall be in accordance with ER 1110-1-12, "Quality Management".

b. Project Management. The Contractor shall provide overall project management and coordination for work tasks during development of the Design Documentation Report.

c. The Contractor shall prepare a hydraulic model test on the existing 1:25 John Day sectional model at Northwest Hydraulic Consultants' Vancouver, B.C. laboratory to evaluate the geometry of the RSW and the downstream deflector. All physical model documentation and design calculations conducted to produce the final model report shall be included as an appendix to the Design Documentation Report (DDR, which replaces the acronym FDM). Only the pertinent information on the final design shall be included in the main text of the DDR. General model work at Waterways Experiment Station (WES) shall be overseen by CENWP-EC-HD and performed concurrent with the sectional model work. One trip to WES for two Contractor design personnel and one trip for one Contractor design person will be included in this scope. The two trips will be used to evaluate geometry options for the upstream Zone of Influence (with respect to the RSW) and the tailrace egress (for the flow deflector issues). The initial trip to WES will be to evaluate two chosen sectional model alternatives and the second will be to finalize the geometry and information for the DDR. CENWP personnel will visit the Contractor's lab facility at least two times during this task order. The Contractor shall investigate the following items in their sectional model:

(1) RSW design for "proof of concept" of the skeleton bay Surface Bypass Spillway (SBS). The RSW geometry shall be designed to mimic the hydraulic conditions and fish attraction potential of the three bays of turbine unit 20 skeleton bay surface bypass (SBS) concept, as set forth in the John Day Feature Design Memorandum No. 52. To mimic the SBS in spillbay 20, the contractor shall identify up to five alternative RSW crest geometries with respect to the following design criteria: flow depth, flow volume, flow velocity, flow acceleration, and zone of influence. The contractor is free to suggest and utilize additional evaluation criteria with concurrence of NWP POC. An interim letter report submittal shall be provided to NWP at the

Model Alternative Meeting (see paragraph g.3.a outlining the pros and cons of the evaluated RSW geometries related to the above criteria and recommending two spillway crest geometries to be built and tested in the Contractor's existing sectional model). With the concurrence of CENWP, the Contractor shall build and test one of the two designs in the sectional model and document the results in accordance with the documentation outlined in paragraph 3c (3). CENWP may direct the Contractor to construct and conduct preliminary tests (photos, video, general observations for three flows) with the second selected geometry prior to the first model visit by CENWP personnel. This option is further described in Section 4 (Optional Work Tasks). Only preliminary tests are to be conducted before the first visit. If two designs are evaluated during the first visit, one of the designs will be chosen for the more complete documentation described in paragraph 3c (3). It is assumed that the selected design(s) will consist of a finite step at the intersection of the RSW and existing spillways, allowing the model RSW to be placed above the existing spillway. Thus, no modifications to the existing model spillway surface are required. The Contractor shall participate in a WES trip in which the alternative(s) shall be evaluated in the John Day 1:80 General Model. Following the initial WES trip, the selected alternative will be confirmed and design completed and presented in the DDR. All five alternatives - geometry, design information, reasoning of selection, and resulting decisions, etc shall be included in the model study report which will be included as an appendix to the DDR.

(2) Optimum RSW design. As a separate study activity, the Contractor shall develop up to five RSW geometries that optimize the surface bypass and fish attraction capabilities of the raised crest. Selection criteria shall be the same but not limited to those of the "proof of concept" design in item 1 above. The resulting design may or may not mimic the zone of influence of the SBS. The alternative's geometry, design information, reasoning of selection, and resulting decisions, etc shall be documented in a letter report as shown in section 4, Submittals. See optional work tasks for numerical modeling of these five alternatives. Upon concurrence of CENWP, up to two optimum RSW design alternatives may be constructed and evaluated in the sectional model under optional work tasks. If the optimum RSW alternatives are modeled, the Contractor shall participate in a WES trip in which the two alternatives shall be evaluated in the John Day 1:80 General Model (see optional work tasks). Following the initial WES trip, one alternative will be selected and design completed and presented in a separate letter report. In order to assist in evaluation of the design criteria, the Contractor may propose to utilize numerical models to screen the alternatives (see optional work tasks related to numerical modeling).

(3) RSW Modeling Documentation. Once the selected design for "proof of concept" is determined, prepare a minimum of three water surface profiles with the water surface at three forebay elevations (during fish season) at minimum pool (El. 262), normal pool (El. 265), and maximum pool (El. 268). The final RSW geometry is to be optimized to minimize the potential for fish injury at the intersection of the RSW and the existing spillway. The flow criteria for assessing performance optimization will be finalized during the first model visit with the input of both CENWP and fisheries agency personnel. Optimization of the design may require both mandatory and selected optional work tasks. The flow conditions at the intersection of the RSW with

the existing spillway crest shall be documented including: flow separations, pressures, velocities, and cavitation indices shall be computed. The zone of influence of the RSW in the forebay shall be documented including information from both the sectional and WES general model. In this section model, velocities will be measured for three flows in three vertical planes parallel to the main flow direction. One plane will coincide with the centerline of the bay, the other two planes will coincide with the quarter points of the bay. Approximately 12 velocities will be measured in each plane. For purposes of comparison with the existing spillway, qualitative observations will be made with flow under a partially open radial gate that produces the same flows used for the RSW. Qualitative observations will be made during operation of the radial gate during opening and closure to assess operational characteristics (see paragraph 3c (5) and (6). Discharge rating curves shall be developed over the full range of pool elevations for all alternatives modeled.

(4) Flow Deflector Elevation. The elevation of a 30 ft. deflection will be selected. The selection will be based on previous deflector tests conducted by Northwest Hydraulic Consultants on the 2:25 scale John Day section model and on the tailwater elevation vs flow rate operating curves to be provided by CENWP. The RSW is to be installed in Bay 20, adjacent to the powerhouse. Because the lateral inflow from the powerhouse may influence deflector performance, and is not simulated in the section model, detailed deflector performance tests will not be conducted. A qualitative evaluation will be made of the effect of the RSW on deflector performance with selected comparisons made with the earlier tests with the 30 ft. deflector (at elevation 153 ft.). Detailed performance curves and optimization of the deflector elevation are not included in this study. Future work may include modifying the model to simulate powerhouse flow.

(5) Assess the operational characteristics on the final “proof of concept” RSW geometry that will result during the operation of the tainter gate downstream of the RSW. Of particular concern is the potential for instability or vibration of the tainter gate (or RSW) when the tainter gate is lifted out of the flow or dropped into the flow.

(6) Qualitatively assess the hydrodynamic loading on the RSW and tainter gate for use in structural design of the RSW.

d. Design Documentation Report (DDR). The following information shall be included in DDR (DDR is a term which replaces a FDM). The DDR is a design document and serves as the technical basis for the plans and specifications and serves as a summary of the design decisions with respect to the design of the Removable Spillway Weir. The content and format for the DDR is discussed in Appendix D of ER 1110-2-1150. The DDR provides the layout, concept, most design loadings and design criteria. The Contractor is responsible for design and minimum detailing sufficient to determine feasibility of the design.

(1) Hydraulic Design. The Contractor shall include all information determined from the sectional model study, and shall perform calculations as necessary to define the hydraulic conditions on the RSW, the spillway ogee, the deflector and the tailrace.

This information shall be presented in a concise manner in the DDR. Include a rating curve from minimum operating pool elevation 257 to maximum probable flood elevation 276, three water surface and velocity profiles for pool elevations 262, 265, and 268.

(2) Structural Design. The Contractor shall provide structural engineering, design, design drawings, and itemized quantity takeoffs on the RSW and it's attachments to the spillway. The design of conceptual design of a concrete extended spillway deflector will also be developed. The Contractor shall perform structural design of the RSW to withstand all hydraulic loads. The RSW shall be designed to be floated and sunk into place. The connections to the spillway bay shall be such that the RSW can be removed within 24 hours. Any temporary fixtures that are attached to the spillway bay shall not decrease the capacity of the spillway bay when the RSW is removed. A cursory evaluation of the stability of the spillway monolith with the RSW shall be included in the structural calculations.

(3) Naval Architecture. Considerations for sinking, floating, and towing shall be included in the DDR. The Contractor shall perform sufficient design necessary to determine the approximate locations of the flotation chambers of the RSW and shall evaluate the floating, towing and sinking, characteristics of the RSW.

(4) Mechanical Engineering. Determination of the concept design of the chamber filling and chamber draining piping for floatation and sinking of the RSW.

(5) Construction Schedule and Methods. The Contractor shall determine fabrication timeline, onsite preparation work and other necessary items for construction and installation of the RSW at the John Day spillway. The Contractor shall prepare a construction schedule from Construction Notice to Proceed to completion of RSW installation.

(6) MCACES Cost Estimate. The Contractor is responsible for providing itemized quantity takeoff and backup material to CENWP in a format that is preapproved by the Government cost estimator. Cost estimates shall be prepared for the 60% and 90% and final submittals. It is anticipated that the Contractor will only have to *update* the quantity calculation sheets at each successive submittal, rather than generate all new quantities.

(7) Plates. Plates shall be prepared as necessary to describe the RSW design, and shall be included in the DDR. Also an installation and removal sequence shall be included as one of the DDR plates.

e. Drawing List. The Contractor shall prepare a draft list of contract drawings for construction of the RSW.

f. Independent Technical Review (ITR). The Contractor shall provide for an ITR, which shall be outlined in the QCP. To ensure an independent review is accomplished, the Contractor shall provide a team composed of members that are not directly involved in the design of the products in this scope of work. This team shall have a designated project manager or technical leader that will provide overall review team coordination and interface directly with the Product Review Team Leader (David Illias; Assistant Chief Design Branch; 503-808-4901) at CENWP. An ITR shall be performed at 60 percent Review. The ITR confirms the proper selection and application of established criteria, regulations (EM, TM, ER, ETL, etc.) laws, codes principles, and professional procedures to ensure a quality product. ITR also confirms the constructability and effectiveness of the product and the utilization of clearly justified and valid assumptions that are in accordance with policy. The ITR should also confirm that the assumptions and technical criteria are well documented in the Design Documentation. The IRT shall be in accordance with the following guidance:

(1) EC 1165-2-203: Technical and Policy Compliance Review (Appendix E).

(2) CECW-EP/CECW-EC Memorandum, dated 18 July 1997, titled Accountability and Responsibility for Technical Products (Appendix F).

g. Coordination. Coordination of design activities is critical to the successful completion of this contract. Every effort shall be made to resolve critical issues in a timely manner before they become problems. Communication by telephone, email (preferred), and FAX is strongly encouraged. Regularly scheduled meetings are described in paragraph 6, Schedule. Other meetings, including additional site visits and telephone conference calls, required to exchange information may be scheduled by mutual agreement. The Contractor shall provide written records of all significant conversations and submit copies by FAX or email within three (3) working days to each party involved as well as the Project Manager, regardless of who initiates the call.

(1) Project Manager. The Contractor shall assign a project manager (PM) to act as a primary point-of-contact. The PM shall be responsible for the coordination of the work developed under the task order. The Point-of-Contact (POC) for CENWP is Matt Hanson (503) 808-4934, FAX 503-808-4934; Hydraulic Design POC for modeling concerns is Diana Modini (503) 808-4896, FAX: (503) 808-4875, email addresses can be provided as necessary.

(2) Review Comment Responses and Incorporation. The Contractor shall respond to review comments and incorporate all necessary comments into the model report or DDR for all disciplines at the 30 percent submittal, 60 percent submittal, 90 percent submittal, and Final submittal. This includes comments from both the project design review team and the independent technical review team. The Contractor shall provide comment responses within 10 working days from the end of each review period.

h. Meetings. The Contractor shall attend the following mandatory meetings. All meetings shall be held at CENWP offices in Portland, Oregon, unless agreed to otherwise. The Contractor shall fax or email draft meeting notes no later than five (5) working days following any meeting. CENWP comments to the draft notes will be faxed to the Contractor within three (3) working days of receipt for incorporation into the final notes. The Contractor shall prepare and submit final meeting notes not later than five (5) calendar days following receipt of comments, but not later than 10 calendar days after the meeting.

(1) Model Alternative Meeting. The Contractor shall send lead engineers and a project manager to the John Day project 20 days after Notice to Proceed (NTP) to present the letter report containing the RSW "proof of concept" alternatives and recommendations on options to test in the physical hydraulic models. The Contractor shall provide pertinent design information for each alternative (hydraulic information including flow volume and upstream conditions, etc.) such that a decision can be made to proceed with model testing of the two alternatives. Following the alternatives discussion, the Contractor shall discuss installation and operational concerns with the John Day Project operators.

(2) Progress Review Meetings (PRMs). Three (3) PRMs will be held at CENWP during the DDR development at the 30 percent, 60 percent, and 90 percent completion levels. The project manager, lead structural engineer, and lead hydraulic engineer shall attend these meetings. See paragraph 6, Schedule, for anticipated dates for meetings.

(3) Quantity Takeoff Procedure Meeting. One (1) teleconference meeting to discuss procedures to generate itemized quantity takeoffs. This meeting will occur before the 60 percent quantities are due.

i. Review of Ongoing work at Walla Walla District Corps of Engineers. The Contractor will be expected to include pertinent information from the Walla Walla RSW modeling and design effort as it applies to the CENWP design. CENWP will provide information to the Contractor, as it becomes available.

4. OPTIONAL WORK TASKS. The optional work tasks will not be funded at the time of contract award. The Contractor shall not proceed with the use of any optional services described in this Statement of Work without formal modification and written authorization by the Contracting Officer.

(1) CFD modeling of RSW geometry alternatives proposal. A numerical model can be a useful tool to evaluate the hydraulic effectiveness of a RSW, especially the "optimal design". A CFD will allow comparisons between alternatives, quantify the difference between various designs and fine tune the design before it could be modeled in the physical model. It is anticipated that a 3-dimensional or computation fluid dynamics (CFD) model shall be required to fully evaluate the upstream zone of influence, velocities, and accelerations in the forebay and to compare the various configurations. The Architect/Engineer (A/E) is invited to present a proposal, cost estimate, and schedule to build, calibrate, validate, and run a CFD model to evaluate the optimum RSW concept. The Contractor's cost estimate shall be contained within a separately sealed envelope from the rest of the proposal and will only be evaluated after the technical proposal has been evaluated for completeness. The proposal shall address the following items but not be limited to: The CFD software to be utilized and a brief discussion of how the Contractor proposes to utilize a CFD for this work, and the extent of the grid (width and length) and cell size shall be determined based on the area of concern.

(i) The proposal shall be prepared with the following base criteria:

- The model is expected to extend from the crest and upstream into the forebay far enough that the upstream boundary condition does not affect the flow conditions at the RSW
- The numeric modeling grid shall utilize the same horizontal and vertical control as in the Government furnished hydrosurvey (detail survey conducted in September 1999).
- The model shall as a minimum simulate at least three adjacent spillbays, the skeleton bay area between the powerhouse and spillbay 20 (as a non-overflow section) and two turbine intake units to allow quantification of project operations on RSW performance. The remainder of the boundary condition can be identified from the general model
- Model Calibration, Verification, and Documentation. The Contractor shall calibrate and validate the model using ADCP data and physcial model data provided by the Government. The A/E shall document the model calibration and verification results (plots and tables), boundary conditions, how the grid was generated, and the model limitations in a memorandum submitted to the Government. The Contractor shall not proceed with Baseline Simulations until the Government gives approval.
- Simulations. The baseline simulations will provide the measure against which the alternatives will be evaluated. The baseline simulations shall be run at particular operation scenarios as agreed by the Government. Submit operation scenarios and reasoning of those selected to the Government prior to running the simulations. Additional simulations to evaluate alternatives shall be proposed by the Contractor. Assume that the baseline and each alternative shall require 24 simulations, three pool elevations and eight project operations at each pool elevation.
- Numerical Model Letter Report. The Contractor shall document the development and operation of the numerical model in a draft and final letter report to be submitted to the Government for review and comment. The Contractor shall respond and incorporate the comments into the report and supply the final report in electronic form as well as one hard copy. The report shall include a summary and presentation of all simulations, identification of simulation trends, and recommendations based on the simulations. Appendices shall include limited user documentation for the CFD models, including example runs and listing of external programs used for manipulation and processing of raw input/output data from each of the model runs. The reports shall also provide an application process flow chart with an example, i.e., input data files needed, data flow to and from model, etc. The reports shall also provide complete instructions on how to run the model for a simple test case. Due to the large amount of data that will be generated, it is important that all simulation output be archived so that retrieval of selected data is as easy and flexible as possible. Some examples of the types of summaries envisioned are: Plan-view "snapshots" of the velocity field in the form of vector and/or contour plots at a given time; Snapshots and animations of streamlines in the forebay for a given simulation; Snapshots and animations

of particle traces from specified release points within the forebay for a given simulation; Comparisons of velocities and streamlines at numerous locations for different operating conditions. The proposal is to include model grid to be delivered to the government in a format that is compatible with other CFD software packages. The model will be provided electronically as well as in hardcopy. The input and output from the CFD analysis will be provided to the Government in electronic form.

(2) Additional Sectional Modeling. Two categories of additional modeling are anticipated: The first category involves the construction and detailed testing of up to two options (designated Options 1 and 2) for three flows in accordance with paragraph 3c (3). The cost of one option shall be the cost to model one additional RSW geometry. Establish two units of Additional Sectional modeling.

(i) The second category involves the construction of an optional RSW for demonstration during the first visit to the sectional model in combination with demonstration of the first configuration to be constructed and tested under the mandatory tasks. Preliminary testing only is required for this option (see paragraph 3c (1) for description of preliminary test.

(3) Contractor Services. If unforeseen work is required then the Contractor shall provide resources and expertise required to complete the work.

(i) Engineering Services. The Contractor shall establish four (4) optional engineering units as follows:

Project Manager	2 hours
Senior Structural Engineer	16 hours
Staff Engineer	16 hours
Secretary	2 hours

(ii) CADD Services. The Contractor shall provide the CADD resources and expertise required to develop or revise drawings or sketches. Establish two (2) optional CADD units as follows:

Project Manager	1 hour
Staff Engineer	4 hours
Senior CADD Operator	20 hours

(iii) Site Trips. The Contractor shall attend additional site trips, including travel to CENWP or project site for a full day meeting and/or investigation. Establish three (3) optional site trips as follows:

Project Manager	12 hours
Senior Engineer	12 hours

(iv) Additional WES Trips. The Contractor shall attend WES site trips to evaluate options in the General Model at WES including travel and Trip report. Establish three (3) optional site trips as follows:

Staff Engineer	50 hours
Senior Hydraulic Engineer	40 hours

5. SUBMITTALS. Products for review submittals shall include as necessary; DDR Plates, design calculations, quantity computations and summaries, construction schedule, comment responses, and meeting documentation. All reports and submittals shall be provided in electronic (*.pdf (Adobe) and *.doc (Microsoft Word 97) or *xls (Microsoft Excell 97)) formats and one reproducible hard copy. Products shall be submitted in accordance with the following requirements:

a. <u>Model alternative evaluation submittal.</u> The Contractor shall prepare separate letter reports for the five RSW geometries for the SBS "proof of concept" and the optimum RSW designs, then submit documentation for selecting the best two alternatives to further evaluate in the sectional model. The Contractor shall make sufficient copies to fully present the information to all meeting attendees.

b. <u>CFD Modeling Submittal</u>. If this option is exercised then submit the following prior to running the baseline simulations: the recommended grid configuration, documentation of model verification results, documentation of Baseline Simulations in draft and final Numeric Model Letter Reports as outlined in section 1d. The Contractor shall document the model verification results (plots and tables), boundary conditions, how the grid was generated, and the model limitations. Once this memorandum is received, the Contractor shall not proceed with Baseline Simulations until the Government gives approval.

c. <u>30 Percent DDR Submittal</u>. At this stage, the Contractor will present the two alternatives for final design of the SBS "proof of concept" RSW design. The Contractor shall report on the progress and decisions made to this point. Fifty copies of the submittal shall be submitted for review. This submittal format will not necessarily conform to the DDR requirements because the DDR is in formulation at this time. An outline for the DDR shall be submitted for review that includes all sections to be discussed. The 30 percent submittal will also include a presentation of at least three attachment alternatives with a reconnaissance level cost estimate for each to assist in decision making for the final design. From this point on, only one attachment alternative will be considered. The Contractor shall also prepare the five RSW geometry's for the optimum RSW design and submit information on these alternatives at this submittal.

d. <u>60 Percent DDR Submittal.</u> At this stage, it is expected that a majority of the hydraulic modeling is complete and the model report should be at a 90% level. The 60% DDR shall have design criteria finalized and some of the controlling dimensions determined. The design shall be developed such that only details will be added to the 90% submittal. The Contractor shall submit 50 copies of the DDR in addition to a scanned version in electronic "pdf" format (readable by Adobe Acrobat). The Contractor shall also submit one electronic copy of all CADD drawing files. The 60 percent submittal shall incorporate responses to all meetings and reviews to date.

e. <u>90 Percent DDR Submittal.</u> The 90% submittal shall be considered completed except for minor revisions. The Contractor shall submit 50 copies of the DDR for review including appendices. In addition, a scanned "pdf" version, the text document and all CADD drawing files shall be submitted electronically. The 90 percent submittal shall incorporate responses to all meetings and reviews to date. A drawing list for a RSW construction contract shall also be provided at this submittal.

f. Final Submittal. The Contractor shall submit 60 copies of the DDR for distribution and one reproducible copy of the DDR, and all appendices. The Contractor shall also submit electronic copies of all CADD drawing files, text files, and figures in two files formats: The first file format shall be from the software that created the file, the second shall be in Adobe Acrobat "pdf" file format. All electronic files shall be submitted on a Compact Disk. Additional files and information to be submitted shall include, but not be limited to, two paper copies of supporting calculations for design, quantities, related correspondence, and meeting minutes.

6. PREPARATION AND FORMAT OF SUBMITTALS. The Contractor shall prepare a complete DDR in sufficient detail to prepare contract drawings. CADD drawings shall be prepared in Microstation format and shall conform to CADD drafting requirements as listed in the CENWP-EC-D CADD Standards Manual available through the Corps of Engineers Portland District Internet Homepage (http://cadd.nwp.usace.army.mil/). All drawings shall adhere to these CADD standards. Preliminary and final submittals shall also adhere to the following:

a. Drawing/Plate Scale/Size. The contract sheet size shall be 14- x 20-inches. The review sheets to be reprinted in the DDR shall be reduced to fit 11- x 17-inches.

b. Drawing File Format. All drawings shall be created using Intergraph Microstation and submitted to CENWP on high-density floppy disks (3.5-inch) or CD-ROM.

c. Border File and Cell Library. The Government shall furnish the Contractor with the border, including title block, on 3.5" high-density floppy disks (IBM Compatible). The use of cell library PDX.CEL shall be required. The Contractor shall add an appropriate block which shall show designer, drafter, checker, and reviewer.

d. Software Requirements. All documents shall be prepared utilizing the latest version of MS Word for Windows unless otherwise approved by the Contracting Officer.

e. Calculations. The Contractor shall submit calculations for all design work. Calculations shall support the design and document criteria, loadings, design decisions, material selection, and geometry. Calculations shall be performed in accordance with referenced publications. In cases where sound and prudent engineering judgement conflicts with the above guidance, the Contractor shall use the guidance that is more conservative and consult with CENWP prior to using the guidance. The calculations shall be presented in clear and legible form, incorporating tables to show all code references, design criteria, loads and load combinations. Assumptions and conclusions shall be clearly explained. Calculation sheets shall bear the names of the designer and checker with corresponding dates. No portion of a calculation shall be designed and checked by the same individual. Calculation sheets shall be 8 ½- x 11-inches or 11- x 17-inch foldouts (including all computer input/output). Sheets shall be numbered, bound in loose-leaf, 3-ring binders and include a table of contents. Dividers shall be provided to separate individual design features.

f. Documentation. The Contractor shall document the necessary conversations, catalog cuts, etc., which were used to determine the features of the design.

7. SCHEDULE. The following schedule shall be used in completing the work under this contract:

EVENT	CALENDAR DAYS AFTER NTP
Model Alternative and Site Meeting	20
30% DDR Submittal	62
30% Progress Review Meeting (PRM)	76
60% DDR Submittal	111
60% PRM	125
(Includes Quantity Takeoff Mtg) begin optional items, optimum RSW.	
90% DDR Submittal	167
90% PRM	181
100% DDR	194
Submittal of all electronic files, and computations	200

8. GOVERNMENT-FURNISHED MATERIALS. The Government will furnish the following:

a. Existing construction drawings will be made available to the Contractor. The Contractor may select up to 30 drawings for the Government to reprint to give to the Contractor. If the Contractor requires additional drawings, they shall be made at a cost of $2.00 per copy. Drawings will only be supplied in ½ size, and may also be provided in a scanned format for the Contractor to print.

b. Reference Engineering Manuals for design.

c. Electronic CADD Drawing Border File.

d. Electronic Cell Libraries.

e. If the CFD modeling option is exercised, the Government will furnish the following:

(1) Geo-referenced bathymetric hydrosurvey for John Day (Sept 99).

(2) John Day ADCP data (time, location, and x,y,z velocity components) in a geo-referenced form suitable for importing into ARC/INFO GIS software.

(3) John Day project operations (turbine unit and spillway bay discharges, forebay elevations) for ADCP data, Baseline Conditions, and Alternatives.

(4) McNary project operations (total spill and powerhouse flows) for ADCP data, Baseline Conditions, and Alternatives

(5) John Day forebay physical model data.

(6) John Day structural drawings for skeleton bays and turbine units.

9. EXPENDITURE REPORT. The Contractor shall provide a schedule showing projected monthly costs, cumulative monthly costs, funding balance for the current month and the following eleven months, or the remainder of the contract period, whichever is less. A sample format is provided in Appendix E. This 12 Month Expenditure Schedule shall be updated monthly to reflect actual expenditures/revised projections. Three copies shall be submitted monthly with the monthly Progress Report.

10. SAFETY. The Contractor shall conform to all safety standards of the U.S. Army Corps of Engineers Safety and Health Requirements Manual EM-385-1-1, Dated September 1996, and all subsequent additions and amendments. An accident prevention plan and hazard analysis shall be prepared as required in Section 1 of EM-385-1-1 by the Contractor prior to initialization of the work tasks for this Task Order. In conjunction with the accident prevention plan, the Contractor shall report the total man-hours expended in field operations monthly by all employees, including Contractor and Sub-Contractor, supervisor and labor. The reporting period shall end at midnight on the last day of each month. The report shall be made to Cheryl Frank by the 5^{th} of the following month. Ms. Frank's telephone number is (503) 808-4822 and her FAX number is (503) 808-4805.

11. ADDITIONAL WORK. The Contractor shall not perform any services under this task order which are considered by the Contractor to be a change in the scope of work or services required without the written consent of the Contracting Officer.

12. CORRECTION OF UNSATISFACTORY WORK. The Contracting Officer's Representative maintains the right to reject any work that is found to be in error, incomplete, illegible, or in any way not conforming to this workorder. The Contractor shall be liable for all costs in connection with correcting such errors. Corrective work may be performed by Government forces or by Contractor forces at the discretion of the Contracting Officer.

13. USE OF INFORMATION. The information developed, gathered, assembled, and reproduced by the Contractor or his Contractors, Subcontractors, or their associates in fulfillment of the task order requirements as defined in or related to the Statement of Work, will become the complete property of the Government. Therefore, the Contractor will not use the information for any purpose at any time without the written consent of the Contracting Officer.

14. REVIEW OF DELIVERED WORK. Review and acceptance of delivered work shall be the basis for final payment. The Contractor shall be responsible for the professional quality and technical accuracy of all services furnished under this contract. The Contractor shall, without additional compensation, correct or revise any errors or deficiencies in the services, and shall resubmit the work within seven (7) calendar days after request for such services is made by the Contracting Officer.

15. RELEASE OF INFORMATION. Drawings, sketches, reports, and other documents/information generated under this contract become the property of the Government and distribution by the Contractor to any source, for any purpose at any time, without the written consent of the Contracting Officer is prohibited.

16. PARTNERING. This task order is expected to be mutually managed in compliance with the mission statement and objectives defined by the A/E Contract Partnering Agreement.

17. PAYMENTS. The Contractor shall submit monthly invoices indicating actual work and services performed to date for approval by the Government. Payments shall be made in the amount of 90 percent of the value and services shown on the monthly invoice upon approval of the Contracting Officer and in accordance with the Payments Clause of the contract.

18. CONTRACTOR'S RELEASE OF CLAIMS. The Contractor shall submit a written "Release of Claims", signed by the firm's president with the final invoice for services rendered under the terms of this Task Order.

REVISED DETAILED STATEMENT OF WORK
FOR CONTRACT DACW57-97-D-0004, CH2M HILL/MONTGOMERY WATSON (JV),
TASK ORDER NO. 21

Modification Case 3: Extended Deflector Testing, Lower Crest RSW Testing, and Construction of the Skeleton Bay Surface Bypass

1. BACKGROUND. The original task order included construction and testing of a 1:25 scale Removable Spillway Weir (RSW) surface flow collector in spillway bay 20 and verification of the hydraulic performance of a proposed 30-ft long spillway deflector at elevation 148 ft downstream from the RSW. The RSW is being evaluated as a "proof of concept" for, or a permanent alternative to, the powerhouse Surface Bypass Skeleton Bay (SBSB) collector. However, performance of the SBSB collector has not been investigated in a 1:25 scale model comparable to the RSW model tested under the original task order. Preliminary testing of various RSW concepts has been completed leading to selection of RSW Alternative 5 as the recommended RSW geometry. The selected RSW Alternative 5 consists of a low portion of crest, which may remain in place during spillway operation. An extended spillway flow deflector is presently being considered in John Day Dam's spillway bay 20 to reduce dissolved gas concentrations of the spillway flow. Spillway bay 20 is located immediately adjacent to four skeleton units which is next to the powerhouse therefore, spillway bay 20 stilling basin and tailrace experiences a significant contribution of powerhouse flow entrainment. At other dams, it has been observed that powerhouse flow can be entrained in the spillway flow increasing total dissolved gas production from the project.

The purpose of this modification is to test and document the performance of the low crest portion of RSW Alternative 5, to determine the optimum geometry and elevation of the extended deflector for spillway bay 20 with and without the RSW, to test and document the impact of powerhouse flows on spillway bay 20 extended deflector performance (if it can be represented in the sectional model), and to document the SBSB collector for comparison to the RSW Alternative 5.

Construction of the two-piece (low and high crest) RSW is being accomplished under the original task order, while testing of the low crest RSW shall be accomplished under this modification.

2. MODELING OBJECTIVES.

- Test and document the discharge capacity and average pressures on the low crest portion of the Alternative 5 RSW. An optional work task is included in this modification to construct and test an intermediate crest.

- If the Optional Work Task (a) is exercised the Contractor shall modify and calibrate the existing John Day sectional model flume to simulate the powerhouse flow contribution for evaluation of the extended flow deflector and development of performance curves.

- Test and develop performance curves for the optimum extended deflector geometry in spillway bay 20 with and without the RSW in place.

- When at WES for the RSW and deflector modeling, qualitatively document the influence of the spillway bay 20 extended deflector on the North Collection System Entrances and the North Fish Ladder Entrances.

- Qualitatively document and compare, by photographs and visual observation, the energy dissipation performance in the stilling basin with and without the extended deflector in place.

- Prepare a report, which compiles and documents results of all previous John Day Dam flow deflector sectional model studies.

- Construct and qualitatively document the performance of the SBSB.

3. MANDATORY TASKS. The Contractor shall provide all labor, equipment, materials, and items incidental thereto except as noted in the following tasks. All dimensions and discharges specified in this SOW are at the prototype scale.

Task 1: John Day Model Modification. The Contractor shall modify the John Day 1:25 sectional spillway model and submit the proposed modification plan to Portland District, Corps of Engineers (CENWP) for approval. The John Day sectional model has been designed to readily install and remove both pieces of the RSW Alternative 5 (low and high crest) from the existing spillway. The cost for this is included in the original task order. Specific guidelines for model design and construction include, but are not limited to, the following:

a) The Contractor shall develop a test plan for the John Day 1:80 scale general model and submit to CENWP for approval one week prior to trip. The results of the WES trip shall determine whether the powerhouse entrainment flow should be represented in the 1:25 NHC sectional model. If the flow should be represented, then determine the quantity of flow, and how that flow contribution should be incorporated in the sectional model. During the WES trip the Contractor shall also collect necessary calibration data for the powerhouse entrainment. See Optional Work Task (a). WES Trips are included in the original task order.

b) The John Day spillway sectional model shall be designed and modified to readily adjust the extended deflector length, transition radius, and elevation to facilitate positioning at any elevation between 143 ft and 153 ft.

Task 2. Prepare a Summary Document for John Day Flow Deflectors. The Contractor shall review and prepare a brief summary of all previous sectional model flow deflector studies at John Day Dam, including the 1970's study accomplished at the Bonneville laboratory (TR No. 104-1 September 1984). The document shall summarize all findings in the form of an executive summary. CENWP will furnish all sectional model reports and report quality photographs, drawings, figures, etc., for use in the summary document. The Contractor shall provide one reproducible copy of the brief summary and the flow deflector studies to CENWP.

Task 3. Extended Flow Deflector Tests. Model testing shall be performed in the John Day sectional model flume to determine an optimum extended deflector length, transition radius, and elevation for spillway bay 20 with and without the RSW, based on past model studies and hydraulic performance curves with discharges up to the maximum RSW flow at normal maximum operating pool elevation 268.

a) Preliminary Testing. Preliminary testing for the extended deflector geometry with and without the RSW shall be accomplished to determine the optimum geometry to test and document in the final design documentation tests. The preliminary phase of testing shall consist of three deflector geometries, each applied at one elevation. These geometries shall be selected by the Contractor and CENWP and shall be selected following completion of Task 2. The Contractor shall assume that the preliminary tests will be accomplished at three discharges and four submergences. These three geometry combinations shall be tested to develop the final deflector geometry that best optimizes the range of spillway discharge and deflector submergence producing a skimming flow regime. The Contractor shall conduct preliminary testing of these combinations of conditions and deflector geometries to narrow and refine the number of tests to be accomplished in the final design selection process. CENWP personnel will participate in the latter part of this phase of testing after the Contractor has completed preliminary testing of the numerous combinations. Note that during Task 2, if it is concluded that the number of geometries for this Task 3 are excessive, the scope of testing to be accomplished under Task 3 shall be reviewed and adjusted.

Documentation for the preliminary extended deflector work is qualitative, consisting of photographs and video. However, preliminary hydraulic performance curves shall be developed as needed during the preliminary testing to assist in the selection process of the optimum deflector geometry to test for the final design documentation.

Optional Work Tasks (c) and (d) are provided in this modification for additional deflector combinations and testing the deflector with the powerhouse contribution.

b) Final Design Testing. The final design deflector geometry and elevation that is selected from the results of the preliminary tests, shall be tested with the RSW at four discharges and four tailwater elevations and without the RSW at two discharges and two tailwater elevations (to be jointly selected by the Contractor and CENWP).

Performance curves, air entrainment characteristics, and depth of air penetration curves shall be prepared similar to Figs 3-3, 3-5, and 3-6 respectively, presented in NHC's model study "John Day Dam Spillway Flow Deflectors Performance Curves, Existing Interior-Bay and Proposed End-Bay Deflectors" August 1999. See Optional Work Tasks (c) for deflector testing with the powerhouse contribution. In addition, documentation of the final extended deflector design shall include photographs of flow conditions in the stilling basin for all conditions tested, noting discharge, submergence and tailwater elevation.

c) Stilling Basin. As a base condition, qualitatively document by photographs and visual observations, the energy dissipation characteristics in the stilling basin of the existing spillway crest without the extended deflector for two of the gated discharges and two of the tailwater elevations (to be jointly selected by the Contractor and the Government) tested in Task 3b. Compare this with the results of Task 3b (high RSW crest with deflector).

d) WES Modeling. The Contractor shall develop a test plan for the John Day 1:80 scale general model to be submitted to CENWP for approval two weeks prior to trip, for the final extended deflector geometry and elevation and spill pattern development. The Contractor shall participate in testing the final extended deflector design and gathering information to document the influence of the spillway bay 20 extended deflector on the flow conditions near the North Collection System Entrances (south of spillway bay 20).

Task 4. Surface Bypass Skeleton Bay (SBSB) Model Construction and Documentation. The Contractor shall construct the SBSB in NHC's previously occupied McNary 1:25 scale sectional model flume. This will allow for simultaneous operations and comparisons of Alternative 5 RSW (in the John Day flume) and the SBSB. The SBSB model shall be constructed such that it can be easily modified for potential future testing; such as the changing the SBSB crest, SBSB deflector elevation, and allowing for different deflector elevations in the adjacent bays. Drawings of the SBSB are included in the UASCE "John Day Lock and Dam Surface Bypass Spillway Feature Design Memorandum No. 52", September 1998.

The hydraulic performance documentation shall consist of qualitative observations (photographs) at pool elevations 262, 265, and 268. Qualitative observations shall consist of documenting the water surface nape profile along one pier and the centerline of one SBSB bay at up to seven locations selected by CENWP and the Contractor. This allows for an observation of the drawdown around the piers, which was not very visible in the general model.

Task 5. Model Tests on the Low Crest Portion of the Alternative 5 RSW: The testing of the low crest portion shall be accomplished in NHC's 1:25 McNary sectional model flume. The Contractor shall submit a proposed test plan and the locations and types of the pressure measuring instruments for approval one week prior to the low crest portion of the RSW work.

a) Rating Curves. Gated and un-gated (free flow) discharge rating curves shall be developed for the low crest portion of the RSW. The gated rating curve shall be developed at 1,2,3,5,7,9,12, and 15 ft gate openings for four pool elevations between RSW low crest elevation 218.5 and normal maximum pool elevation 268. Un-gated discharge shall be measured at pool elevations 268 and 276.3 (PMF).

b) Reduction in Spillway Capacity. The discharge rating curves for the low crest portion of the RSW (Task 2a) shall be compared to NHC's "John Day Spillway Rating Curve" September 1998. However the existing spillway discharge rating curve does not extend to the PMF. Therefore the existing spillway rating curve shall be developed at pool elevations 268 and 276.3.

c) Pressures. The Contractor shall measure the average pressures on the low crest RSW and adjacent downstream portion of the existing spillway at normal maximum pool elevation 268 and PMF 276.3 (un-gated). Pressures shall be measured using piezometers and pressure cells at up to 20 locations along the existing spillway face downstream from the RSW. Pressure measurements for the high crest RSW are covered in the original contract

Task 6. Meetings. Formal model demonstrations and meetings shall be held at the Contractor's model laboratory for:

(1) Task 3a, Preliminary Deflector Model Testing with CENWP and agencies, for four days,

(2) Near the end of Task 3b, Final Design Deflector Testing and SBSB (beginning of Task 4) with CENWP, for two days,

(3) During Task 4, Performance Testing of the SBSB and final deflector for CENWP and agencies, for three days.

(4) During Task 5, Low Crest Portion of the RSW Discharge and Pressure Tests with CENWP and agencies, for two days, and

The Contractor shall provide agendas and present briefings on the status, test results, and interpretation of test results with respect to meeting the projects design objectives. The Contractor shall also prepare meeting minutes to be included in the report.

Task 7. WES Trips. Two trips to WES are required in this modification. The first trip (5 days) shall be to determine if the powerhouse entrainment flows should be represented in the sectional model, obtain powerhouse flow requirements for the sectional model, and collect powerhouse influence data for calibrating the sectional model. The second trip (5days) shall be used for the final optimum extended deflector design. The original contract provided for two mandatory and three optional trips. To date one trip has been used, therefore, no additional estimates are required for trips to WES associated with this modification. CENWP will coordinate this effort with WES.

Task 8. NHC Lab Trips. Four trips to the NHC laboratory are required in this modification. The original task order provided for two mandatory and three optional trips to the NHC laboratory. To date, three trips have been used. Provide an estimate for four additional trips to the NHC laboratory. Note the remaining two optional trips in the original task order shall be exercised, if required, in the future.

Task 9. Interim Reports. The Contractor shall prepare and submit to the Government Point of Contact (POC) brief reports summarizing the testing results accomplished under Tasks 3a, 4, and 5 immediately upon completion of those tasks. The interim reports shall be in the form of a letter report and need not be bound. The summary reports shall include an introduction, test results, conclusions, photographs, tables, figures and any pertinent information from that phase of testing. The interim reports shall be prepared in a manner that facilitates direct incorporation into the final model study report.

Task 10. Draft Final Report.

a) The Contractor shall prepare a comprehensive report summarizing the results of the model study. Each test shall be in a separate appendix to the main report.

b) The model facilities shall be shown in a schematic drawing with the measuring instruments clearly located. A description of the model specifications, construction techniques, as built drawings, and colored photographs of the modified model and the model in operation shall be included in the report.

c) If the powerhouse entrainment is built into the sectional model, then the procedure followed for calibrations the model and the data collected during this process shall be discussed in the report. The report shall include a discussion of the instrumentation used to measure the data and any limitations associated with use of the data.

d) The report shall present all the data measured and determined from preliminary and final deflector testing and low crest RSW testing in a clear, concise, and complete format using tables, graphs, drawings, and photographs as appropriate. Tables shall be included to document measured data and flow characteristics such as pressures, discharge ratings, etc. In addition to the measured test results, the Contractor shall note all conditions deemed significant to the overall performance of the deflector design. The Contractor shall provide conclusions drawn from the modeling effort, with explanations of the methods in which the data were used to develop those conclusions. Discussion of those designs not selected for final deflector design shall be limited only to the extent of illustrating why that design was not selected. The report shall also include a complete description, discussion, and data results for the final deflector design. CENWP will review and provide written comments. For the evaluation of the deflector design, a sufficient number of photographs shall be presented in the report to clearly illustrate significant areas of the evaluation. See Report Submittals for the types and number of reports to be submitted to CENWP.

Task 11. Final Report. Following CENWP's review of the draft report, comments shall be provided to the Contractor and the Contractor shall respond in writing to the comments and incorporate revisions required into the report and produce a final report. All raw data such as calculations, assumptions, data plots, photographs, etc., obtained during the model tests shall be placed in the appendix of the final report. These data shall be clearly described as to origin. Provide miscellaneous portions of this study in an appendix**.** See Report Submittals for the types and number of reports to be submitted to CENWP.

4. OPTIONAL TASKS.

a) Optional Task a. John Day Flume Modification for Powerhouse Entrainment. During the initial WES trip, the Contractor and CENWP personnel shall determine whether the powerhouse entrainment flow should be represented in the sectional model. If it should, then this optional item shall be exercised. The John Day sectional model flume shall be modified to simulate the powerhouse hydraulic flow regime with the quantity of flow that is determined during the initial WES trip. The Contractor shall assume that a powerhouse entrainment discharge of up to 10,000 cfs shall be simulated in the sectional model. If tests with the 1:80 scale general model at WES indicate that powerhouse entrainment flows greater than 10,000 cfs need to be simulated in the sectional model, this contract modification shall be subject to modification. The powerhouse flow shall have the capability of simulating the flow direction in the tailrace as determined from the 1:80 scale model.

b) Optional Task b. John Day Model Calibration for Powerhouse Entrainment. If the sectional model flume is modified, then the model shall be calibrated to simulate the tailwater hydraulic characteristics downstream from spillway bay 20, due to the influence of the powerhouse contribution. During the WES trip the data collection program shall consist of point velocities, directions, and magnitudes with

a grid pattern necessary to accomplish the calibration data. Model calibration data and results shall be included in the report.

c) Optional Task c. Deflector Testing with Powerhouse Entrainment. If the sectional model is modified for the powerhouse entrainment, then testing and documenting of the preliminary and final flow deflector with the powerhouse entrainment shall be included as stated in Task 3a and b.

d) Optional Task d. Additional Deflector Combinations. The preliminary phase of testing consisted of three deflector geometry combinations (three geometries and one elevation). This option provides for six additional combinations as determined by the Contractor and CENWP. See Task 3a. Establish six optional units of additional deflector combinations:

Principal	.5 hours
Project Manager	4.7 hours
Sr. Hydraulic Engr.	8.9 hours
Jr. Hyrdaulic Engr.	13.8 hours
Sr. Cadd Tech	2 hours
Sr. Technician	8.7 hours
Jr. Technician	2.7 hours
Secretary	2 hours

e) Optional Task e. Intermediate Crest. If exercised, this task includes construction, testing, and documenting of an additional section of the RSW Alternative 5. This crest would be intermediate in elevation between the low crest RSW and the high crest RSW. Testing and documentation shall be identical to the requirements of Task 5.

f) Optional Task f. SBSB Deflector Velocity Measurements. If exercised, this option shall require measured mid-depth point velocities for three pool elevations in three vertical planes parallel to the flow at six locations between the downstream face of the SBSB and the deflector. The vertical planes and longitudinal measurement locations shall be located to best capture the range of conditions existing in the flow over the SBSB. Approximately 18 velocity measurements will be made for each pool elevation. Depth of flow shall be measured at the same locations where the velocity is measured.

5. GOVERNMENT FURNISHED INFORMATION. CENWP will furnish the following information upon request:

a. Drawings of the spillway bay 20 stilling basin and the area of the powerhouse tailrace adjacent to bay 20.

b. Tailwater rating curves and typical operating range of tailwater during the spill season.

c. Previous Corps of Engineers deflector model study report(s) for the project.

d. Specific testing scenarios (i.e., pool elevation, discharge, gate settings, etc.).

e. River channel bathymetry downstream from spillway bay 20, SBSB, and powerhouse tailrace

f. Powerhouse details at north end (last 4-5 units)

g. Drawings of the Skeleton Bay Surface Bypass

6. COORDINATION. During work progress, the Contractor shall maintain bi-weekly (or as needed) coordination with the POC to assure the orderly progression and completion of the work. In addition, the Contractor shall record meeting minutes held with CENWP and submit draft copies of the meeting memorandum to the POC within seven days following each meeting. CENWP will review the draft memorandums and provide written comments to the Contractor within seven days of receipt of the drafts for incorporation into the final meeting report memorandum.

7. REPORT SUBMITTALS. The reports described in Tasks 2, 8, 9, and 10 shall be prepared in accordance with the following requirements:

a. Photographs. The photographs shall be provided in the hard copy of the report as scanned images and electronic copy of the report as digital images.

b. Format. The reports shall be written in easily understandable language and presented in clear, concise and logical format that describes in detail the technical analysis of the data collected for this contract. Interim, draft, and final reports shall be single-spaced and laser printed on 8-1/2 by 11-inch paper. Provide one electronic copy (CD) of the draft and final reports. The electronic copies shall be compatible with Microsoft Word 97, Microsoft Excel 97, and PDF formats.

c. Interim Reports. Provide two stapled copies with color photos of each interim report to the POC for CENWP review and approval. See Task 8.

d. Draft Reports. Provide ten bound copies with color photos and one unbound copy of the draft final model study report and one reproducible copy of the draft John Day Flow Deflector Sectional Model Summary Report (Task 2) to the POC for CENWP review and approval. See Task 9.

e. Final Report. Provide ten bound copies with color photos and one unbound copy of the final model study report and the final John Day Flow Deflector Sectional Model Summary Report (Task 2) to the POC for CENWP review and approval. See Task 10.

f. Contractor Submittals. The Contractor submittals described in Tasks 1, Task 2, and Task 3 shall be submitted to the POC for CENWP review and approval.

8. SCHEDULE. The schedule does not include allowance for Optional Work Tasks. If any of the Optional Work Tasks are exercised, the schedule will be adjusted. Note that the general model is being verified and WES trips cannot occur until it is completed. The Contractor shall complete all work in accordance with the following schedule:

Event	Calendar Days After NTP
Begin model(s) design and preparation of condensed John Day Deflector Report	1
Submittal #1. Test plan for powerhouse contribution for WES trip	3
Submittal #2. Deflector and SBSB model design and layout	7
WES trip #1. Powerhouse contribution	14
Submit Prelim. JD Condensed Sectional Def. Model Rpt	28
Submit Final JD Condensed Sectional Def Model Rpt	42
Complete Deflector Model construction (doesn't incl flume mod if req'd)	49
NHC trip #1. Demonstration of preliminary deflector model with NWP and agencies (Task 3a)	63
Complete Prelim. Deflector Model Tests	70
Submit Preliminary Deflector Interim Report	84
NHC trip #2. Demonstration of final deflector modeling and SBSB with NWP (Task 3b)	91
Complete Final Deflector Design Documentation Tests	105
Submittal #3. Test plan for spill patterns / RSW and final deflector modeling for WES trip	105
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9. POINT OF CONTACT(S). The Contractor shall appoint a Point of Contact (POC) for all work under this task order. The Government POC's are Diana Modini, telephone number (503) 808-4868 and Chris Goodell, telephone number (503) 808-4896 Ms. Modini's email address is diana.l.modini@usace.army.mil and Mr. Goodell's email address is christopher.r.goodell@usace.army.mil.

10. SAFETY. The Contractor shall conform to all safety standards of the U.S. Army Corps of Engineers Safety and Health Requirements Manual EM-385-1-1 dated September 1996, and all subsequent additions and amendments. If applicable, the Contractor shall report the total man-hours expended in field operations monthly by all employees, including Contractor and Subcontractor, supervisor and labor. The reporting period shall end on midnight on the last day of the month. The report shall be made by telephone or FAX to Cheryl Frank by the 5th of the following month. Ms. Frank's telephone number is (503) 808-4822 and the FAX number is (503) 808-4805.

11. ADDITIONAL SERVICES. The Contractor shall not perform any services under this task order which are considered by the Contractor to be a change in the work or services required by this agreement without the written approval of the Contracting Officer.

12. CORRECTION OF UNSATISFACTORY WORK. The Contracting Officer's Representative maintains the right to reject any work that is found to be in error, incomplete, illegible, or in any way not conforming to the specifications outlined in this task order. The Contractor shall be liable for all costs in connection with correcting such errors. Corrective work may be performed by Government forces or by Contractor forces at the discretion of the Contracting Officer.

13. USE OF INFORMATION. The information developed, gathered, assembled, and reproduced by the Contractor or his Subcontractors, or their associates in fulfillment of the task order requirements as defined in or related to this Statement of Work, will become the complete property of the Government. Therefore, the Contractor will not use the information for any purpose at any time without the written consent of the Contracting Officer.

14. RELEASE OF INFORMATION. Reports and information generated under this contract become the property of the Government and distribution by the Contractor to any source, for any purpose, at any time, without the written consent of the Contracting Officer is prohibited.

15. PAYMENTS. The Contractor shall submit monthly invoices indicating the actual work on services performed to-date for approval by the Government. Payment will be made in the amount of 90 percent of the value of services shown on the monthly invoice, upon approval of the Contracting Officer.

16. CONTRACTOR RELEASE. The Contractor shall submit a written 'Release of Claims" signed by the Contractor's company president, with the final invoice for services rendered under the terms of this contract.

DETAILED STATEMENT OF WORK FOR
CONTRACT NO. DACW57-97-D-0004: CH2M HILL/MONTG. WATSON
TASK ORDER NO. 0021: JOHN DAY SURFACE BYPASS REMOVABLE SPILLWAY WEIR DESIGN

MODIFICATION NO. CASE NO. 0006: DOCUMENT PERFORMANCE OF DEFLECTOR AT EXISTING SPILLWAY

1. GENERAL. This detailed statement of work (dsw) supplements the dsw contained in the base task order. All requirements in the base task order dws remain in effect except as specifically cited in this document.

2. WORK TO BE PERFORMED UNDER THIS MODIFICATION. During a recent visit to the modeling lab, the Government determined it will be necessary to document the performance of the deflector with an existing spillway at a spillway design discharge. During an event like this, it is critical that the stilling basin still contains the hydraulic jump and dissipates a sufficient amount of energy. The memorandum to be developed under this modification shall establish the tasks that are required to be performed in order to determine the longest deflector that can be constructed in spillway bay 20 without compromising the structural integrity of the stilling basin apron and the dam itself. The tasks to be completed are as follows:

a. Reconstruct Spillway Bay. Reconstruct John Day spillway bay number 20 with a 30-foot long, 50-foot radius deflector at elevation 150 in the high-discharge flume (formally McNary model). To simulate total spillway uniform discharge, false walls shall be constructed such that the interior half of the piers project into the flow. The walls shall extend upstream and downstream to the extent necessary to obtain accurate conditions.

b. Review NPDEN-TE-L Report. Review NPDEN-TE-L Final Report, “John Day Dam Spillway and Stilling Basin” (to be provided by CENWP-EC-HD), to determine what “sufficient dissipation of energy” is.

c. Construct Deflector Extensions. Construct extensions to the deflector so that different lengths of deflectors can be tested. The longest deflector shall extend to the end of the training wall. For a training wall length of 75 feet, this will require lengths of 30-feet, 50-feet, 75-feet, and 105-feet (105-feet extends to the end of the training wall). Provide for testing of one additional deflector length, in the event that the “critical” length occurs between two of the above lengths (example, may find it necessary to test a 40-foot deflector). With a base case, this will require six (6) separate tests.

d. Test Procedure Requirements. For each deflector length and for a base case with no deflector:

(1) Operate a spillway design discharge (112,500 cfs for one gate) with the appropriate tailwater elevation of 205.3 feet, msl.

(2) Determine position of the toe of the hydraulic jump relative to the PT of deflector radius.

(3) Take digital photographs of the stilling basin.

(4) Observe and document movement of appropriately sized sediment within the stilling basin and just downstream of the end sill.

(5) Take velocity measurements in the centerline of the bay above the end sill at a point near the end still, at half-depth and near the surface.

e. <u>Document and Provide Results and Recommendations</u>. Document the test procedure, compile data, and provide results and recommendations in the final report submitted under contract DACW57-97-D-0004, Task Order No. 0021, Modification No. 0003.

f. <u>Trip to Water Ways Experiment Station</u>. One trip to WES by one NHC representative will be added under this modification. The purpose of this trip will be for the representative to provide consultation and expertise efforts toward developing spill patterns and John Day Spillway Bay No. 1 flow deflector design.

3. SCHEDULE. This work will change the completion date to 03 August 2001.

20-21 JUNE 2000 VISIT TO
NORTHWEST HYDRAULIC CONSULTANTS LABORATORY

30 June 2000

MEMORANDUM FOR: Record

SUBJECT: Trip Report – NHC Lab visit 20 – 21 June to observe John Day Removable Spillway Weir

1. The proposed John Day Dam Removable Spillway Weir (RSW) physical model was demonstrated for Portland District Corps of Engineers (NWP) staff at the Northwest Hydraulic Consultants' (NHC) laboratory facilities on 20 and 21 June 2000. The John Day RSW is under investigation in a physical hydraulic model of 1:25 scale in the NHC laboratory. The primary purpose of the visit was to observe and evaluate a demonstration of the Alternative #2 design. Participants in the laboratory visit included:

Diana Modini – NWP Hydraulic Design Section
Brad Bird – NWP Hydraulic Design Section
Blaine Ebberts – NWP Environmental Resources Section
Matt Hanson – NWP Structural Design Section and Project Technical Manager
Al Babb – NHC, Physical Model Principal Investigator
Ken Christison – NHC, Physical Model Project Engineer
Dennis Dorratcague – Montgomery Watson, Project Engineer
Ed Zapel – NHC, Project Engineer and Principal Designer
Dick Regan – NHC, ITR Reviewer
Jim Lencioni – NHC, Senior Project Engineer

2. The model demonstration was limited to observation of the RSW design Alt # 2, which was the single design selected for model evaluation by NWP following the alternatives Review Meeting, held at the John Day Dam project on 9 May 2000. RSW Alternative No. 2 was selected from 6 different conceptual designs presented during the Alternatives Review Meeting. On June 1st, NWP requested that NHC also build and test a seventh RSW alternative (Alt #7) that combined several features of those presented at the Alternatives Review Meeting.

3. Alt #7 consisted of a RSW crest and chute design similar to that of Alt #2, but with entrance piers extended about 25 feet further into the forebay, and with slightly less efficient pier shapes (i.e., simple radius) and an inclined ramp leading into the RSW up to the crest. The purpose of the upstream pier extension was to move any affects of flow separation around the piers further away from the crest while the purpose of the floor ramp was to achieve the velocity flux criteria of 0.1 fps/ft. Subsequent to their request to build and test Alt #7, the NWP reversed its decision and chose not to complete construction of that alternative RSW design for use in model evaluation and demonstration.

4. The visit opened with a short briefing of District staff on model progress and presentation of the proposed schedule for observation of the RSW in operation in the 1:25 scale model. The proposed test schedule included tests of the RSW Alternative 2 at forebay elevations 262 ft, 264 ft, 266 ft, and 268 ft (the normal maximum operating pool

elevation). At each forebay elevation, the model would be demonstrated at various tailwater elevations between 158 ft and 170 ft. Of primary interest in the model was flow contraction/separation around the pier noses and hydraulic characteristics in vicinity of the tangency point between the RSW crest and the existing spillway crest. Testing of similar raised crests at lower Granite Dam by the Walla Walla District illustrated the occurrence of objectionable rooster tails beginning near the crest and extending down the chute of the spillway. The Alt #2 geometry has a crest elevation of 245.5 ft, ogee design shape head of 22.5 ft and piers extending 46 ft upstream from the existing spillway crest centerline. The RSW crest is connected to the existing spillway crest via a 20-ft radius simple curve tangent to the existing crest at a point about 1 ft upstream of the existing spillway gate seat.

5. At forebay elevation below about 257 ft, hydraulic conditions down the RSW crest and downstream along the existing spillway crest appeared quite acceptable. However, as the forebay elevation increased to the normal operating range of 262 ft – 268 ft, contraction around the pier nose created unstable lateral wave disturbances to move downstream. The wave disturbances were amplified when the flow passed through the tangency point between the RSW crest and the existing spillway crest to the degree that very large rooster tails occurred down the crest and off of the spillway deflector into the tailrace. These rooster tails oscillated laterally across the spillway and were so pronounced that areas having no flow depth actually existed on the deflector. Some of the flow disturbances appeared to be originating at the bulkhead side slots as the high velocity flow moved past the slot. However, closing off of the bulkhead side slots did not substantially improve flow conditions downstream. The conditions were considered to be so unacceptable that future testing of Alt #2 to obtain performance data in accordance with the SOW was considered to be unwarranted. A decision was made to modify the RSW over night to roughly simulate the Alt #7 design geometry and test that design the next day.

6. The model RSW crest geometry used to simulate the Alt #7 design extended the piers upstream about 70.5 ft from the centerline of the existing spillway crest in lieu of 71 ft as with the actual Alt #7 design. A removable inclined floor was constructed to simulate the floor ramp intended to achieve the velocity flux criteria approaching the crest of the RSW. Due to the simplistic nature of the model configuration for this alternative, a firm joint between the side faces of the extended pier and the existing pier could not be obtained. This imperfection resulted in flow irregularities that created some flow disturbances that would actually not exist in a perfectly constructed model or prototype installation. However, it was the best that could be achieved in the short time available.

7. The emulated Alt #7 design was tested essentially through the same forebay and tailwater conditions as with Alt #2 the previous day. The emulated Alt #7 design did appear to improve (decrease) the flow disturbances originating from contraction around the pier nose somewhat. However, the wave disturbances hitting the tangency point between the RSW crest and the existing spillway crest still amplified enough that, although maybe being somewhat smaller, the large rooster tails similar to those observed

with Alt #2 remained. Insertion and removal of the inclined floor ramp did not appear to have any appreciable affect of the overall hydraulic performance of the design. Cursory review revealed that capture velocity (i.e., 7 fps) was exceeded at the entrance to the emulated Alt #7 RSW, it was not necessary to meet the velocity flux criteria downstream of the pier noses. The model was also run with the spillway gate lowered into the flow sufficiently to create a flow control to simulate hydraulic conditions down the spillway face under existing conditions. With the gate control, flow disturbances originated as water passed under the gate and created somewhat similar rooster tails at the deflector as existed with free flow over the RSW. However, the rooster tails appeared to be more uniformly spread across the width of the deflector and did not oscillate laterally across the deflector. The noise level in the model was greater with gate control, which at least gave the impression of larger energy dissipation at the deflector than existed with the RSW free flow condition. In any case, the gate-controlled conditions do not appear to be appreciably more hydraulically acceptable than does free flow over the RSW.

8. Following demonstration of the emulated Alt #7, NWP and NHC staff convened to discuss a future course of action with the model. In general, the group did not feel that either Alt #2 or Alt #7 would achieve a hydraulically acceptable design. The group thought that acceptable flow conditions could be obtained only by eliminating the rather sharp tangent connection between the RSW and the existing spillway crest or the reverse radius bucket connecting the two ogee shapes. This could be achieved by either constructing a longer RSW crest which would extend downstream of the existing spillway seat or terminating the RSW crest upstream of the existing gate seat but at an elevation higher than the existing crest (i.e., a vertical stepped offset). Either of these alternatives have some design issues to be considered. The NWP design staff agreed to furnish NHC direction by Thursday, June 22, on whether to construct and test Alt #7, the longer crest design or the stepped design (subsequently, NWP decided to construct and test the stepped design and to test a fillet added to the radius bucket of Alt. #2 to approximate the Alt. #5 geometry).

Edwin T. Zapel, P.E.
Senior Hydraulic Engineer

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28-30 JUNE 2000 VISIT TO USACE ERDC LABORATORY

30 June 2000

MEMORANDUM FOR: Record

SUBJECT: WES Lab visit 28 – 30 June to observe John Day RSW designs in general model of John Day project.

1. The proposed John Day Dam Removable Spillway Weir (RSW) physical model was demonstrated for Portland District Corps of Engineers (NWP) staff at the Waterways Experiment Station (CEWES) laboratory facilities on 28 – 30 June 2000. The John Day RSW is being investigated in a physical hydraulic model of 1:80 scale in the WES laboratory, and also in a separate 1:25 scale sectional model in Northwest Hydraulic Consultant's laboratory. The primary purpose of this visit was to observe and evaluate a demonstration of the Alternative #2 RSW design and the Alternative #7 RSW design, and compare the overall forebay attraction and tailrace egress conditions established with the RSW to those achieved by the Skeleton Bay Surface Collector. Participants in the laboratory visit included:

Diana Modini – NWP Hydraulic Design Section
Blaine Ebberts – NWP Environmental Resources Section
Don Wilson – WES, Physical Model Principal Investigator
Ed Zapel – NHC, Project Engineer and Principal Designer

2. The general model is a 1:80 scale representation of the John Day Dam, navigation lock, powerhouse, and spillway, as well as a significant portion of the reservoir and tailrace. The existing powerhouse skeleton bays have been modified in the model to represent the configuration of the skeleton bay surface collector, which was carried to the feature design level by the District during 1997 through 1999. The RSW concept was developed as an inexpensive way to evaluate the surface collection success of the Skeleton Bay Collector concept in the prototype at much less cost than the full cost of the Skeleton Bay Collector. This physical model evaluation of the RSW concept is intended to determine whether the RSW can approximate the surface collection and tailrace egress performance of the Skeleton Bay concept sufficiently well to validate the Skeleton Bay design.

3. The model demonstration included the following discharge conditions:

Total River Flow 250 kcfs,	0% spillway flow (except RSW/Skeleton Bay)
	20% spillway flow (inc. RSW/Skeleton Bay)
	30% spillway flow (including RSW/Skeleton Bay)
	60% spillway flow (inc. RSW/Skeleton Bay)
Total River Flow 350 kcfs,	0% spillway flow (except RSW/Skeleton Bay)
	20% spillway flow (inc. RSW/Skeleton Bay)
	30% spillway flow (inc. RSW/Skeleton Bay)
	60% spillway flow (inc. RSW/Skeleton Bay)

Each of these discharge conditions was evaluated with the three different surface collector configurations: Skeleton Bay Collector, RSW Alternative #2, RSW Alternative

#7. The spillway gate spill patterns as defined in the Fish Passage Plan were used to set each spill gate opening. In the Fish Passage Plan, the spill bays nearest the powerhouse are not opened as much as the middle and north bays until spillway flows reach high levels. Of particular interest during the demonstrations was the apparent distribution of flow approaching the RSW or skeleton bay collector. The 'flow net' approaching the RSW should approximately cover the same general area as that of the skeleton bay, it should develop similar strength as that of the skeleton bay in the immediate vicinity of the entrance, and it should draw from as near the existing spillway crest as possible.

4. The first series of observations, at total river flow of 250 kcfs and 30% spillway flow (i.e., 30 % of 250 kcfs is passed over spillway + RSW/skeleton bay), indicated that the skeleton bay tended to draw strongly from as far as 2 powerhouse units away and 2 spillway bays away. Total width of the apparent immediate vicinity flow net was about 180 ft (prototype) on the powerhouse side and 120 ft on the spillway side. The RSW #2 seemed to draw from approximately the same limits as the skeleton bay, although centered about the spillway bay 20 instead of about skeleton bay 20. The RSW #7 geometry displayed roughly similar attraction flow characteristics as the RSW #2, but located generally a little further out into the forebay. The RSW #7 configuration crest is located at the same distance from the existing spillway crest as RSW #2, but the approach ramp and extended piers push the actual entrance further upstream into the forebay. Small 'dead' flow zones developed on both exterior sides of the longer approach piers in RSW #7. No such dead zones were noted with either the RSW #2 or skeleton bay configurations. Since no improvement in approach flow characteristics over RSW #2 were noted with the RSW #7 configuration, and the sectional model observations made the previous week concluded that both RSW #2 and #7 had similar hydraulic conditions on the spillway chute, we concluded that there was no apparent benefit to further demonstration of the RSW #7 configuration.

5. The next set of observations, at total river flow of 250 kcfs and 60% spillway flow (i.e., 60% of 250 kcfs is passed over spillway + RSW/skeleton bay), showed that the RSW #2 did not draw quite as well from the spillway side as did the skeleton bay collector. This could be partly because the RSW #2 is located closer to the much larger withdrawal flow of the other spillway bays than is the skeleton bay. In general, the immediate zone of influence of the skeleton bay collector extended about 180 ft (prototype) on the powerhouse side and about 40 to 60 ft on the spillway side. The RSW #2 zone of influence extended about the same distance on the powerhouse side (180 ft), and nearly the same distance on the spillway side (35 to 50 ft). It also appeared as if the RSW #2 drew flow more strongly from directly upstream of the entrance than did the skeleton bay, possibly because the RSW #2 unit discharge capacity is somewhat greater than the skeleton bay. Although the skeleton bay draws more total flow than the RSW (about 25% more), the higher unit discharge of the RSW established nearly the same general approach flow patterns as the skeleton bay. Generally, with the higher spillway flow, the zone of influence for both the RSW and the skeleton bay was similar, but it extended laterally less distance on the spillway side than for the lower spillway flow, as expected.

6. The next series of observations was conducted with total river flow of 250 kcfs, and no spillway flow (except the RSW/skeleton bay). A cursory observation of low velocity flow patterns well upstream in the reservoir (300 to 1000 feet) showed that neither the RSW #2 or the skeleton bay appeared to collect as much of the surface flow as when the spillway was passing some flow. The greatly increased powerhouse flow tended to draw more surface flow away from the skeleton bay collector and the RSW collector. The immediate zone of influence for both the RSW and the skeleton bay extended a greater distance across the spillway under these conditions, as expected. However, the zone of influence on the powerhouse side did not appear to be diminished as much as expected. The limits of the immediate zone of influence for the skeleton bay extended about 120 ft on the powerhouse side and about 120 ft on the spillway side. The limit of the immediate zone of influence was actually slightly greater for the RSW #2 on the powerhouse side (about 140 to 150 ft), and about the same on the spillway side. This was thought to be the result of the lack of flow withdrawal through the four skeleton bays. By virtue of its being located 70 feet closer to the spillway, and about 80 feet further away from the powerhouse units that were actually drawing flow, the RSW appeared to be more successful at drawing surface flow from the powerhouse side than the skeleton bay.

7. At 20% spillway flow (spillway + RSW/skeleton bay) and total river flow of 250 kcfs, the performance of the RSW and the skeleton bay was similar to that with 30% spillway flow. In general, the zone of influence of the RSW #2 was similar in size and characteristics to that for the skeleton bay. The only real difference seemed to be that the RSW drew more heavily from directly upstream of the entrance than the skeleton bay. As discussed above, we theorized that the slightly greater unit discharge capacity of the RSW resulted in this characteristic observation. The limits of the zone of influence for both the RSW and the skeleton bay were nearly the same as with the 30% spill, but less extensive on the spillway side than with the 60% spill, as expected.

8. At the higher river flow of 350 kcfs and 20% spillway flow (spillway + RSW/skeleton bay), the zone of influence of both the RSW and the skeleton bay appeared to be reduced in strength and size compared to the lower river flows. The additional flow withdrawn from the spillway and powerhouse reduced the ‘flow net’ approaching both types of collectors, but generally didn’t change the distribution of flow from one side or the other of the collector entrance. The limits of this zone of influence followed after similar patterns as for the lower total river flow, but consistently extended only about 80 to 100 ft on the powerhouse side and 60 ft or so on the spillway side with 30% spill. The limits with 60% spill extended about 150 to 160 ft on the powerhouse side and only about 30 ft on the spillway side for both the RSW and the skeleton bay collector. Again, the higher unit discharge capacity of the RSW permitted it to establish a stronger approach flow net that extended consistently 20 ft or so further into the forebay than the skeleton bay, even though the skeleton bay total discharge capacity was higher than the RSW.

9. Tailrace egress conditions were consistently poor with no or low spillway discharge percentage for both the RSW and the skeleton bay collector. A large eddy formed in the spillway stilling basin and downstream that tended to circulate the dye

plume exiting the RSW or skeleton bay in a wide arc as large as 1000 feet into the zone below the spillway. For the 30% spill condition at both low and high total river flow, the skeleton bay egress dye plume tended to rotate clockwise into the stilling basin, while the RSW egress dye plume tended to move more directly downstream, with only some limited counterclockwise rotation into the powerhouse tailrace. At 60% spillway flow, both the RSW and the skeleton bay tailrace egress dye plume circulated counterclockwise into the powerhouse tailrace.

10. Overall, the RSW #2 configuration appeared to approximate the performance of the skeleton bay in terms of approach flow conditions. Some differences were evident in tailrace egress between the two, as discussed in paragraph 9 above, but generally the disposition of the exiting jet was similar. The RSW #7 configuration generally showed no improvement over the RSW #2 configuration, and in fact showed more areas of 'dead' water behind the exterior faces of the approach piers. That characteristic, considered with the lackluster results of the sectional model, led the group to terminate further investigation of the RSW #7 geometry. Now perceived benefits were apparent with the larger RSW #7 configuration over the RSW #2 or skeleton bay collector. More investigation of other methods of reducing or eliminating the undesirable chute flow conditions will be conducted in the sectional model. The larger scale of the sectional model will permit more detailed observations and measurements of hydraulic characteristics on the spillway chute. The next laboratory demonstration will be the 1:25 scale model at NHC's Vancouver facility on 19 and 20 July 2000, in which other RSW configurations will be evaluated with an interest in reducing the undesirable chute flow characteristics. The nest WES laboratory demonstration is tentatively scheduled for 29 September 2000, during which the improved RSW configuration will be evaluated. In the interim, the John Day general model will be modified to show more accurately the downstream bathymetry, as depicted in more recent survey data.

Edwin T. Zapel, P.E.
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19-20 JULY 2000 VISIT TO
NORTHWEST HYDRAULIC CONSULTANTS LABORATORY

24 July 2000

MEMORANDUM FOR: Record

SUBJECT: Trip Report – NHC Lab visit 19 to 20 July to observe John Day Removable Spillway Weir

1. The proposed John Day Dam Removable Spillway Weir (RSW) physical model was demonstrated a second time for Portland District Corps of Engineers (NWP) staff at the Northwest Hydraulic Consultants' (NHC) laboratory facilities on 19 and 20 July 2000. The John Day RSW is being investigated in a physical hydraulic model of 1:25 scale in the NHC laboratory. The primary purpose of the visit was to observe and evaluate a demonstration of the Alternative #4 design and the Alternative #2 design with a fillet to approximate the Alternative #5 design. Participants in the laboratory visit included:

Diana Modini – NWP Hydraulic Design Section
Brad Bird – NWP Hydraulic Design Section
Blaine Ebberts – NWP Environmental Resources Section
Al Babb – NHC, Physical Model Principal Investigator
Ken Christison – NHC, Physical Model Project Engineer
Ed Zapel – NHC, Project Engineer and Principal Designer
Dick Regan – NHC, ITR Reviewer

2. The model demonstration was limited to observation of the RSW design Alt #2 with the fillet on the first day, then Alt #4 on the second day. Alt #2 with the fillet was the design selected by NWP to approximate the performance of the continuous ogee geometry of Alternative #5, without requiring construction of an entirely new RSW crest and piers. We recognized that the performance of this 'simulated' Alt #5 geometry might not accurately represent the actual performance of Alt #5, but we were reasonably confident it could show the effects of a continuous ogee on the flow profile. The fillet designed for this demonstration was selected following the conclusions drawn from the previous visit to NHC's laboratory, where Alternative #2 was demonstrated.

3. Alternative #4 incorporated an aeration step at the transition between the existing spillway face and the proposed RSW, just upstream of the existing spillway gate seat. The aeration step was designed to permit air to pass into the flow at the transition point, which would elevate potentially damaging low pressures in the transition zone to pressures which would prevent cavitation damage from occurring, and encourage full development of the boundary layer in the chute flow. A fully developed boundary layer with sufficient air content would generally help to better distribute the total energy of flow across the spillway chute surface. More uniform distribution of flow energy across the chute width would theoretically decrease the 'rooster tail' phenomenon observed in the Alternative #2 geometry without changing overall RSW attraction flow from the forebay.

4. The model demonstration results for the 'filleted' Alternative #2 geometry showed a remarkable decrease in the size and distribution of the 'rooster tail' phenomenon noted without the fillet. During the demonstration, we theorized that the radius transition between the Alternative #2 RSW and the existing spillway ogee in some way permitted, or even increased, the development of uneven distribution of flow across the width of the spillway chute. The water surface draw down observed along the entrance piers was the same for both the Alt #2 and Alt #2 with-fillet geometries. Flow characteristics for both geometries was similar up to the beginning of the fillet tangent point. Downstream of this point, the water surface characteristics differed significantly between the two configurations.

5. In the Alternative #2 geometry observed during the previous visit on 20 June, the pier draw down-induced uneven water surface caused uneven pressure head to become evident in the development of ripples or standing waves on the RSW ogee downstream of the critical depth control section at the crest. The reverse curvature in the radius transition appeared to cause the higher water surface threads to collapse at the base of the reverse curve, forcing the adjacent water surface to rise higher in response to the lateral locally increased energy head. As a result, when flow exited the reverse curvature radius section, increased pressure head was observed in the form of exaggerated standing waves or flow threads in locations immediately downstream of those sections adjacent to the upstream higher water surface. These flow threads of higher pressure head continued nearly unchanged to the flow deflector surface at the base of the existing spillway chute. Upon reaching the deflector, the higher threads would again collapse at points further downstream on the deflector than adjacent threads of lower pressure head, causing exaggerated uneven water surface on the deflector. Upon entrance of the deflected jet into the stilling basin showed that the uneven energy distribution on the deflector surface resulted in uneven energy dissipation in the stilling basin. Of greater concern, though, is the more direct effects of collapse of these 'rooster tails' of higher pressure head flow threads on the deflector surface. Some appeared to strike the deflector at a fairly acute angle, with significant force, implying some risk of fish injury for those fish either in or adjacent to these 'rooster tails'.

6. The Alternative #2 geometry with the fillet showed a significant decrease in the strength of, and consequences thereof, of the 'rooster tails' noted without the fillet. The same draw down occurred along each pier, but the water surface became more even and uniform as it passed down the smooth surface of the fillet (with no reverse curvature). The exiting flow from the fillet tangent point with the existing spillway chute appeared to be much more uniform than that observed with the Alt #2 reverse curve transition. We theorized that the effects of the reverse curvature of Alt #2 without the fillet did not permit, and in fact may exacerbate, the uneven pressure head distribution across the chute width to form large 'rooster tails'. The flow striking the deflector surface appeared to be much more uniform, without the exaggerated acute angle of impact of large flow threads noted with the Alt #2 geometry.

7. The Alt #2 with fillet RSW was removed and the Alt #4 RSW configuration was attached to the spillway on the evening of the first day of demonstrations. Initial

observations at relatively low head (pool elevation below about 255 ft msl), indicated that the aeration step drew some air into the flow. However, as the head increased with rise in forebay to the normal operation level of 264, less and less air was observed drawn into the flow. The flow depth on the chute at lower forebay elevations was much less than that at full design head, specifically at the location of the step transition. In addition, the aeration step did not appear to successfully trigger a fully developed boundary layer at full design head. We theorized that the flow depth on the chute at the step transition was too great at full design head for the boundary layer development to be initiated by the aeration step. In addition, no air was observed being drawn into the flow at full design head. Preliminary investigation of static and dynamic head in the vicinity of the step transition showed that there was about 20 feet of total head just under the aeration step. This was higher than expected, and indicated that flow separation at the step transition location is not as great as expected. Total head as measured in the main flow at the same location was nearly up to the forebay level, indicating that very little energy loss was occurring over the crest and down the chute to the step location.

8. The group decided to simulate a slightly broader RSW crest configuration following demonstration of the Alt #4 geometry with a quickly constructed modification to the Alt #4 crest. The broad crest was about 25 feet (prototype) long, and piers were rounded instead of elliptical. The modification was roughly constructed of a horizontal plywood sheet with no upstream transition radius affixed to the Alt #4 crest along a fairly uneven line near the centerline of the Alt #4 crest. Significant leakage of flow at the tangent locations of the rounded piers to the RSW #4 elliptical piers made it difficult to observe a document the disposition of the draw down in water surface beginning at the rounded piers. However, it seemed to appear that the drawdown effects and generation of uneven pressure head at the crest control section was not entirely mitigated by the extended crest. We theorized that the broad crest would have to extend much further upstream of the RSW crest for the uneven water surface at the crest to be eliminated.

9. Following demonstration of the Alt #4 configuration, the group retired to the conference room to discuss the model observations and develop a course of action for the design of the RSW. Summarizing the performance of the three different types of RSW configurations, the group decided that the Alt #2 configuration with the fillet eliminating the reverse curvature appeared to provide the most satisfactory results. The Alt #4 configuration did not successfully draw enough air at full design head to eliminate the uneven pressure head distribution across the width of the spillway chute. The Alt #2 configuration alone produced unacceptable uneven pressure head distribution across the spillway width, resulting in unacceptable development of ‘rooster tails’ and subsequent collapse on the deflector surface. The group discussed the relative success of the fillet in the Alt #2 geometry in reducing the ‘rooster tail’ phenomenon and the resulting unacceptable hydraulic conditions on the deflector surface. We also discussed the applicability of the Alt #2 with fillet performance to that expected to the Alt #5 configuration. The two configurations (Alt #2 with fillet and Alt #5) would be expected to provide similar results, but not identical. Elimination of the reverse curvature appeared to effect a more uniform energy distribution across the width of the chute and provide more acceptable conditions on the deflector surface.

10. The meeting closed with a discussion about schedule for the 30% Progress Review Meeting (PRM) and remaining design issues. We tentatively established the 30% PRM date at 7 and 8 August 2000, to be held at NHC's laboratory facilities to facilitate demonstration of the Alt #2 and Alt #2 with fillet geometry for agency staff. Demonstration of the Alt #2 geometry with fillet will be on 7 August, and then the fillet will be removed and the Alt #2 without fillet will be observed by agency staff. The 30% PRM will be held the following morning, on 8 August. We concluded that the step aeration transition concept (Alt #4) was not successful for the full design head of the RSW. We also concluded that the Alt #2 with fillet geometry was adequate for testing the approximate performance of the continuous spillway chute concept, but not adequate for confirming the final, or "optimum" RSW configuration. More work was needed to define the methods for removal of the continuous spillway chute concept within the 24 hour period as specified in the scope. However, Blaine Ebberts did mention that perhaps the 24 hour criteria might not be a solid as once thought, and thus perhaps some slight extension would be available. Also, the configuration of the RSW in two sections to facilitate removal of the upstream RSW crest section separate from the downstream transition section was discussed. More work is needed to determine the feasibility of such a two-section RSW configuration.

Edwin T. Zapel, P.E.
Senior Hydraulic Engineer

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Dick Regan, nhc
Jim Lencioni, nhc
Dennis Dorratcague, Montgomery Watson

7-8 AUGUST 2000 VISIT TO
NORTHWEST HYDRAULIC CONSULTANTS LABORATORY

9 Aug 2000

MEMORANDUM FOR: Record

SUBJECT: John Day Removable Spillway Weir (RSW), 7-8 Aug Model Demo & 30% Progress Review Mtg (PRM)

1. The 1:25 scale model for the proposed RSW was demonstrated for Portland District Corps of Engineers (NWP) staff and resource agency staff at the Northwest Hydraulic Consultants' (NHC) laboratory facilities on 7 and 8 Aug 2000. Following viewing of the model, the 30% PRM for the Design Documentation Report was held. Participants in the laboratory visit included:

Diana Modini – NWP Hydraulic Design Section
Matt Hanson – NWP PM
Blaine Ebberts – NWP Environmental Resources Section
Gary Fredricks – NMFS
Steve Rainey – NMFS
Tom Lorz – CRITFC
Chuck Tracy - ODFW
Dennis Dorratcague – Montgomery Watson
Al Babb – NHC, Physical Model Principal Investigator
Ken Christison – NHC, Physical Model Project Engineer
Jim Lencioni – NHC, ITR Reviewer

The PRM participants included, in addition to the above, John Springer of Glosten Associates and Lee Miesbauer of Civil Tech Corporation who are also members of the RSW design team.

2. Prior to demonstration of the physical model, Mr. Lencioni furnished the group copies of NHC's August 2000 document "Physical Model Alternatives Report". This report summarized results of the John Day RSW physical modeling program to date which recommended carrying the Alternative #5 concept into the next phase of the design process. The model demonstration was limited to observation of the RSW design Alt #2 and a modified Alternative #2 design (subsequently referred to as Alt 5B in this document) which approximated the Alternative #5 as described in the June 2000 "John Day Surface Bypass Removable Spillway Weir Design Documentation Report". The Alternative #2 crest terminates upstream of the existing spillway gate seat and includes a 20 ft radius bucket transition between the crests of the RSW and the existing spillway ogee. The Alternative 5 crest described in the June 2000 report extends about 28 ft downstream from the existing gate seat and connects with the existing spillway ogee via a uniform slope of 0.7338 which is tangent to both crests, subsequently eliminating the bucket transition existing with Alt #2. The Alt 5B crest shape demonstrated in the model has a slightly different ogee equation downstream from the crest apex than did Alt 5 and connected to the existing spillway crest about 27 ft downstream from the gate seat via a 0.7244 uniform slope tangent to both crests. Therefore, the hydraulic characteristics over

the crest and down the spillway face of the previously designed Alternative 5 RSW shape and the modeled Alternative 5B RSW shape would be very similar.

3. As previously observed in the model and documented in the Physical Model Alternatives Report, drawdown around the nose of the piers creates standing wave and/or surface disturbances which travel downstream. With a pool elevation of about 264 ft, the drawdown around the pier nose was measured to be about 2 inches in the model or about 4 ft in the prototype with both the Alternative 2 and Alternative 5B designs. With the Alternative 2 geometry, the waves were greatly amplified as they passed through the 20-ft radius bucket transition between the RSW crest and the existing spillway crest and created large roostertails roughly approximated to be as high as 6 inches in the model (about 12 ft prototype). These roostertails impacted on the deflector and then deflected and plunged deep into the stilling basin. Flow depths on some portions of the deflector were measured to be as little as ¼ to ½ inch in the model (6 to 12 inches prototype). All participants concluded the conditions to be unacceptable.

4. With the Alternative 5B design, the standing waves generated off the pier drawdown were not amplified (and may in fact have been somewhat attenuated) in height as they traveled down the spillway face. Although standing waves and/or surface disturbances did extend down the face of the spillway, the large roostertails existing with the Alt 2 design were not produced. Minimum flow depths of 1.5 inches were measured (about 3 ft prototype) on the deflector. A 4 inch (8 ft prototype) wave rideup existed along the downstream pier extension wall above the deflector. The maximum flow depth in the wave disturbances on the deflector proper was about 2 ¼ inches (about 4.5 ft prototype) with the predominant depth being about 2 inches (4 ft).

5. Although the participants considered the Alternative 5B hydraulic characteristics to be acceptable, some questions remained relative to the effect that the pier drawdown had on the wave disturbances. Although the drawdown itself was not considered to present any problems, there remained a thought that reducing the drawdown might improve flow characteristics down the spillway and on the deflector. The Alternative 5B model design was quickly modified to extend the piers about 18 ft (prototype) into the forebay. With this modification, the drawdown around the piers was reduced to about ½ inch (1 ft prototype) with a pool elevation of 264 ft. However, the hydraulic characteristics on the spillway face and deflector were not significantly changed from those with the original piers. The maximum rideup along the wall and at various locations on the deflector was about 3 1/8 to 3 3/8 inches (6.5 to 7 ft prototype). The minimum flow depth was measured to be about 1 5/8 inches (3.5 ft) and the predominant depth was 2 inches (4 ft prototype). Based on these observations, the participants concluded that further efforts to reduce the pier drawdown were not warranted.

6. The remaining portion of the model demonstration was devoted to observing the deflector hydraulic performance at various tailwater elevations. Gary Frede ricks expressed some concern regarding the hydraulic characteristics in the stilling basin with the deflector and questioned whether the downstream characteristics with the RSW really emulated those of the Skeleton Bay (SB) surface collector. Although he was satisfied with the Alternative 5B RSW approach conditions and overall performance, he stated that

he could not concur with any RSW construction until adequate documentation was developed to show that the RSW performed at least as well as the SB design. The participants concluded that such documentation could not be developed unless the SB performance were evaluated in a 1:25 scale model to be comparable to the RSW model. NWP will consider modification of the existing RSW contract to construct and test a 1:25 scale SB collector. Matt Hanson said the earliest funding could be obtained for this effort would be Oct 2000. Al Babb said that NHC would have the resources and room available to construct the model, probably in the existing McNary spillway flume. The advantage to using that flume is that both the RSW model and SB model could be run and demonstrated concurrently for comparison.

7. Following demonstration of the models, the group retired to the conference room to participate in the 30% PRM. Dennis Dorratcague will prepare minutes of the PRM. At the beginning of the PRM, the participants agreed that the RSW concept that eliminates the transition radius between the RSW and existing spillway crests would be the alternative to be carried forward into the 60% design effort. Subsequently, Mr. Lencioni and Mr. Zapel of NHC, concluded that the previously designed Alternative 5 RSW crest shape would be used for the 60% design in lieu of re-designing an entirely new crest shape strictly to the details of the modified Alternative 2 crest demonstrated in the model. To summarize the PRM, the hydraulic performance and structural considerations of the Alternative 5 concept were discussed. The majority of the discussion regarded the various methods that were being considered for placement and removal of the Alternative 5 concept. NWP stated that the previously furnished RSW placement and removal criteria of 24 hours could be relaxed to at least 3 days. NWP requested that information regarding the reduction in overall spillway capacity be developed assuming that the portion of the RSW downstream from the stoplog slot remained in place. NHC will develop this information after Montgomery Watson furnishes the final geometry. Mr. Lencioni suggested that the hydraulic characteristics of flow over the short piece of deflector downstream from the slot would be more of a problem than would the reduction in discharge capacity it presented. Dennis stated that the 60% design submittal is presently scheduled for about Sept 11. A date of Sept 26 at 9:30 in the NWP office was scheduled for the 60% PRM. Assuming no complications, Dennis said that the 90% design should be completed by about the end of the year.

8. Following the PRM, Dennis, Al and I briefly discussed follow-on NHC work. NHC needs to complete the final documentation of model performance per the SOW. Al will develop and furnish to the Seattle office and NWP a proposal regarding the type and locations of data to be collected in accordance with paragraph 3c(3) of the SOW. Dennis stated that the RSW crest would be offset some nominal distance (say on the order of 1-2 inches) from the spillway concrete face at the downstream end and that NHC would need to provide him information regarding the hydraulic information and requirements for design details. NHC will review and consider how best to construct the model to best simulate the design offset in order to obtain the most reasonable hydraulic data from the model. This may require construction of an entirely new RSW crest in lieu of utilizing the present modified Alternative 2 crest. If a new Alternative 5 crest is constructed, NHC will also consider the impacts of constructing the crest in a manner to readily facilitate

modeling with only the portion of the crest d/s from the stoplog in place to obtain discharge rating and other hydraulic data for that condition.

James L. Lencioni, P.E.

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Dick Regan, nhc
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5-8 DECEMBER 2000 VISIT TO USACE EDRC LABORATORY

December 11, 2000

MEMORANDUM FOR: Record

SUBJECT: General Trip Report – WES Lab visit to observe flow patterns downstream of the John Day Removable Spillway Weir model, Alternative C.

1. Objectives

The objectives of this part of the December 7th and 8th, 2000 WES trip included:

a) taking velocity measurements of the lateral entrainment flow entering the Bay 20 discharge with a 30 foot deflector,
b) observing the interaction between powerhouse, spillway, and RSW flow downstream of the John Day Dam under various spill patterns and tailwater elevations, and
c) determining the feasibility of modifying the section model at NHC to account for the laterally entrained flow downstream of Bay 20.

2. Test Program

2a. Test Conditions

The following table lists the test conditions observed over the two days using the 1:80 scale general model at WES.

WES Comprehensive Model Testing
John Day Dam RSW Hydraulic Model Study
December 7-8, 2000

		Flow Conditions							
Date	Test	Total River Flow	Spillway Flow		Powerhouse Flow	Misc. Flow	RSW / Skeleton Bay Flow[1,2]	Forebay Elevation	Tailwater Elevation
		(cfs)	(%)	(cfs)	(cfs)	(cfs)	(cfs)	(ft)	(ft)
December 7th	1	150,000	0%	0	135,500	500	14,000	264	159.5
	2	150,000	20%	30,400	105,100	500	14,000	264	159.5
	3	150,000	30%	44,800	90,700	500	14,000	264	159.5
	4	159,000	60%	98,600	45,900	500	14,000	264	159.5
	5	350,000	15%	51,200	284,300	500	14,000	264	164.1
	6	350,000	30%	105,600	229,900	500	14,000	264	164.1
	7	350,000	60%	209,600	125,900	500	14,000	264	164.1
December 8th	8	250,000	0%	0	235,500	500	14,000	264	161.7
	9	250,000	20%	49,600	185,900	500	14,000	264	161.7
	10	250,000	30%	75,200	160,300	500	14,000	264	161.9
	11	250,000	30%	75,200	160,300	500	14,000	264	159.8
	12	250,000	30%	75,200	160,300	500	14,000	264	163.3
	13	350,000	30%	105,600	229,900	500	14,000	264	165.4
	14	350,000	30%	105,600	229,900	500	14,000	264	162.9

Notes: 1) Forebay WSE remains constant at 264 ft
2) Data includes velocities (current meter), video, and general photographs.

2b. Identifying Flow Patterns

Injecting a dye tracer throughout the water column within the study area was used to observe flow patterns associated with each test condition. The area of interest extended 300 feet downstream from powerhouse. Laterally, flow patterns were observed approximately 75 feet both north and south of Bay 20 centerline. Photographs and video were taken to supplement this data.

2c. Quantifying Entrained Flow

Velocity measurements were taken to determine the magnitude and orientation of entrained powerhouse and spillway flow downstream of Bay 20 RSW. Measurements were taken at 50 foot increments beginning 200 feet downstream of the spillway crest and extending to 200 feet downstream of the end sill. Laterally, velocities were recorded approximately 40 feet to the north and south of the Bay 20 RSW centerline. Vertically, three velocity measurements were taken at each location to determine surface, mid depth and full-depth velocities.

At each measurement location dye was injected into the water column to determine the general flow direction. A Nixon Current Meter was then orientated with the flow. Velocity magnitude and direction were recorded over approximately 20 seconds. Velocity measurements were subject to large instabilities as was expected downstream of the spilled flow. Data tended to fluctuate between +/- 50% of the mean.

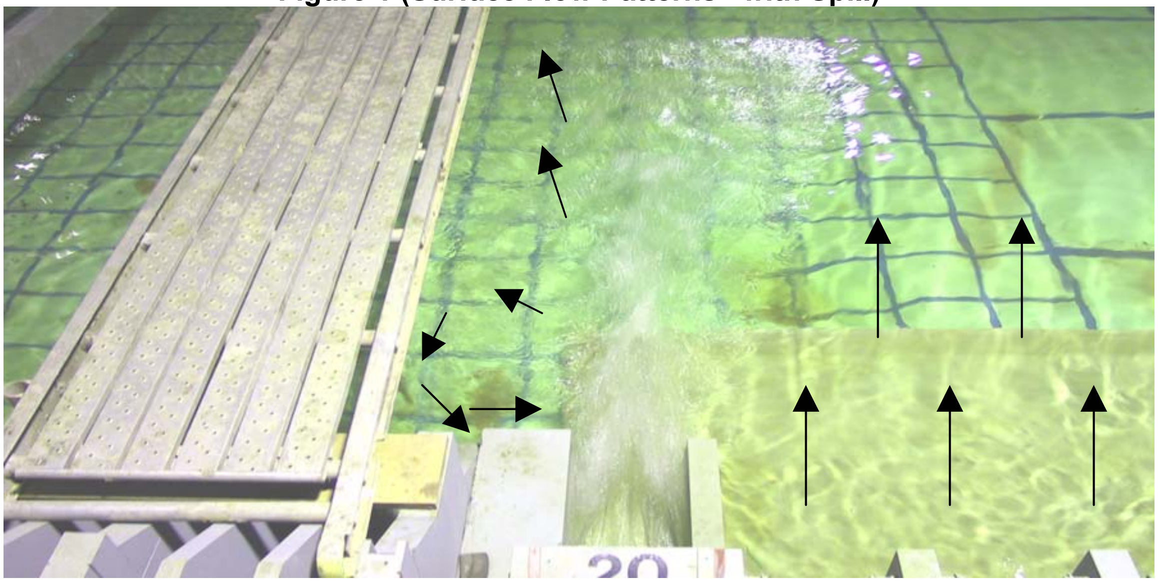
To calculate the total entrained flow the recorded velocities were multiplied by the cosine of their approach angles and then integrated over vertical planes located approximately 40 feet to the north and south of Bay 20 centerline. The approach angle, discussed previously in this document, tended to approximate 45 degrees in the horizontal plane.

3. Observations

3a. Surface Flow Patterns

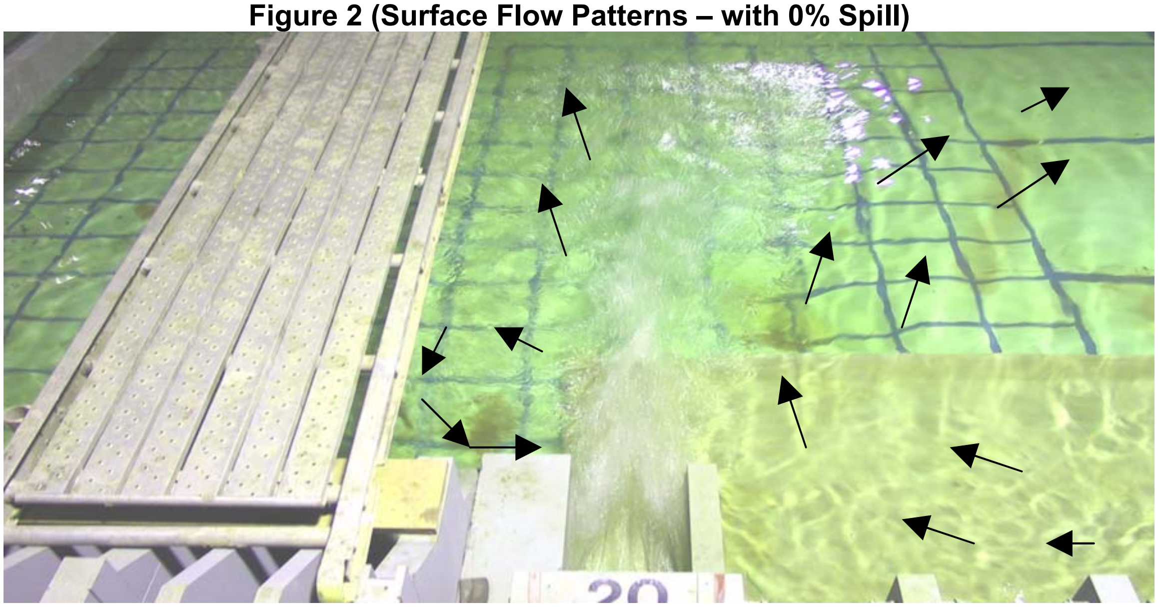
There were two general flow conditions observed during the laboratory tests. The most common condition was observed when the total river flow was split between the powerhouse and the spillway. Surface flow to the left (powerhouse) side of the Bay 20 RSW jet formed a slow re-circulating pattern. The right (spillway) side was directed downstream by the spilled discharge. Figure 1, on the following page, illustrates general flow vectors for combined powerhouse and spillway flow. The photograph does not actually represent this condition and is used to illustrate flow patterns with superimposed flow vectors.

Figure 1 (Surface Flow Patterns– with Spill)



The second general flow condition was observed when the entire river channel flow was passed through the powerhouse (0% spill condition). Surface flow to the left (powerhouse) side of the jet was similar to the first case. However, to the right (spillway) side of the jet a large re-circulating eddy formed, extending beyond the downstream limits of the stilling basin to the north (right) shore of the Columbia River (Figure 2 illustrates general flow vectors).

Figure 2 (Surface Flow Patterns – with 0% Spill)

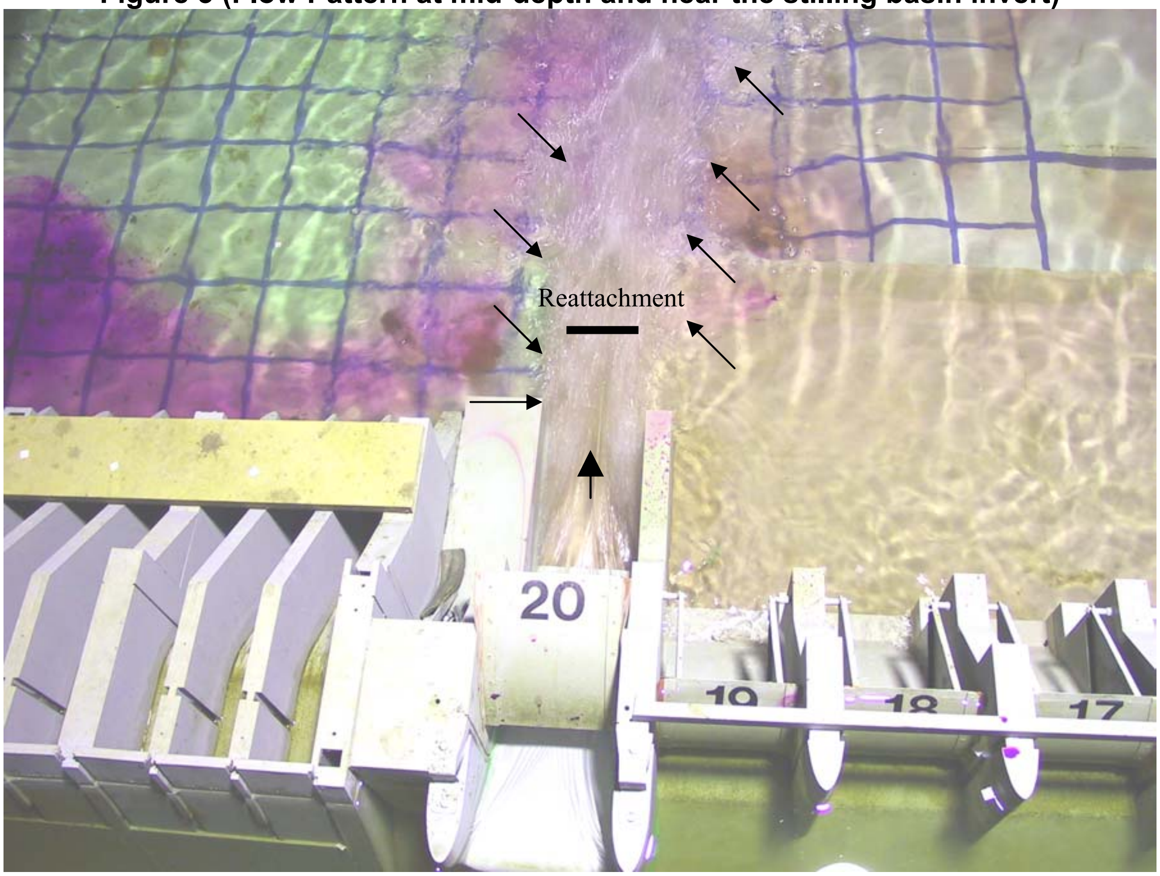


3b. Flow Patterns at Mid-depth and near the Stilling Basin Invert

Flow was entrained into the jet from both the powerhouse and the spillway side of the Bay 20 RSW. From the powerhouse side, flow at mid and near full depth was generally drawn upstream (relative to the direction of the jet) toward the deflector at an angle of approximately 45 degrees (Figure 3).

From the spillway side, flow at mid and full depth was generally drawn downstream (relative to the direction of the jet) away from the deflector at an angle of approximately 45 degrees (Figure 3).

Figure 3 (Flow Pattern at mid-depth and near the stilling basin invert)



3c. Point of Reattachment

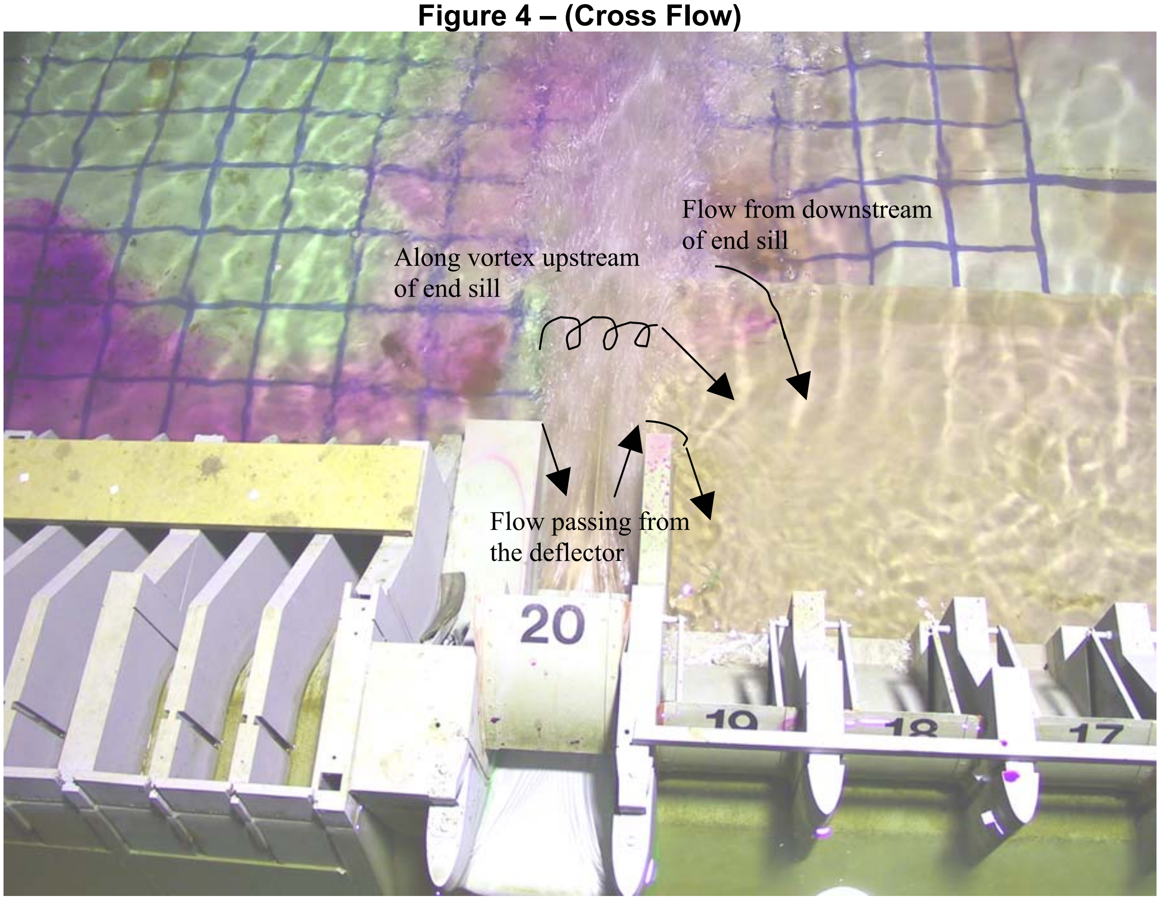
Point of reattachment to the stilling basin invert is located approximately 260 ft downstream of the spillway crest. Entrained flow upstream of this point is drawn under the jet and toward the deflector. Downstream of this point entrained flow is drawn into the jet and then out into tailrace with the jet. The point of reattachment was determined from the left (powerhouse) side of the jet only. (above Figure 3)

3d. Presence of Cross-flow

Prior to the WES trip we were concerned with the potential for flow passing from the powerhouse, under the Bay 20 RSW jet and into the stilling basin. This condition would have been difficult to reproduce in the section model and would require drawing flow out of the model from the right (north) side. This attraction flow was observed by injecting a dye tracer into the water column to the left of Bay 20 and observing its path. Dye passed into Bay 19 at three locations. Some of the dye making its' way to Bay 19 seemed to pass along a vortex generated immediately upstream of the end sill. A smaller percentage of the cross flow followed a path from downstream of the deflector, around the training wall and into Bay 19. A third path began directly downstream of the end sill, in a relatively deep section of the tailrace downstream of Bays 19 and 20, over the end sill and into Bay 19. Figure 4 identifies the approximate path of the cross-flow.

The most cross-flow was observed during Test 7. Other than Test 7, only a trace amount of cross flow was observed in Tests 6, 10, 11, 12, 13 and 14. It is important to note that the amount of flow passing through the Bay 20 RSW was minimal, even during Test 7. When redeveloping the 1:25 section model to incorporate entrain powerhouse flow it is not necessary to account for these cross-flow conditions.

Figure 4 – (Cross Flow)



3e. Effect of Topography Downstream of End Sill

Topography downstream of the end sill is lower in the 1:80 comprehensive model relative to the 1:25 section model. The effects of this depression on the deflector performance are considered minimal compared to entrained powerhouse flow. Also, previous end bay model studies on McNary Dam deflectors showed only minor differences in the hydraulic performance upstream of high and low bathymetric elevations.

3f. Entrained Flow

Flow between 3500 and 6100 cfs (average 4700 cfs) was entrained into the 14,000 cfs RSW jet. The following Table summarizes the test results. Figures 5 to 18 illustrate flow patterns associated with the tests.

Date	Test	Total River Flow	Spillway Flow		Powerhouse Flow	Misc. Flow	RSW / Skeleton Bay Flow[1]	Forebay Elevation	Tailwater Elevation	Entrainment (cfs)	% of RSW Flow (cfs)
		(cfs)	(%)	(cfs)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	Left	Left
December 7th	1	150,000	0%	0	135,500	500	14,000	264	159.5	4400	31%
	2	150,000	20%	30,400	105,100	500	14,000	264	159.5	3500	25%
	3	150,000	30%	44,800	90,700	500	14,000	264	159.5	4300	31%
	4	159,000	60%	98,600	45,900	500	14,000	264	159.5	5100	36%
	5	350,000	15%	51,200	284,300	500	14,000	264	164.1	4800	34%
	6	350,000	30%	105,600	229,900	500	14,000	264	164.1	4700	34%
	7	350,000	60%	209,600	125,900	500	14,000	264	164.1	6100	44%
December 8th	8	250,000	0%	0	235,500	500	14,000	264	161.7	3300	24%
	9	250,000	20%	49,600	185,900	500	14,000	264	161.7	4900	35%
	10	250,000	30%	75,200	160,300	500	14,000	264	161.9	4700	34%
	11	250,000	30%	75,200	160,300	500	14,000	264	159.8	4800	34%
	12	250,000	30%	75,200	160,300	500	14,000	264	163.3	4300	31%
	13	350,000	30%	105,600	229,900	500	14,000	264	165.4	6000	43%
	14	350,000	30%	105,600	229,900	500	14,000	264	162.9	4900	35%

Notes:1) Forebay WSE remains constant at 264 ft

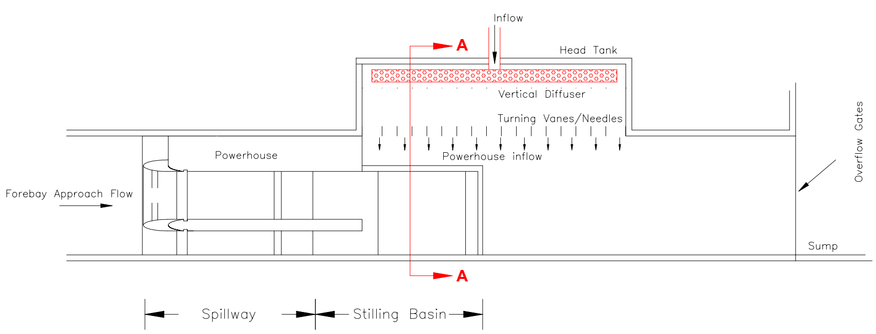
4. Discussion of Results

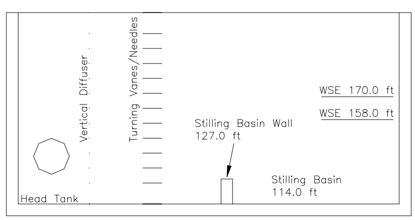
Previous end bay hydraulic model studies, conducted on McNary and John Day Dams, indicate that the inclusion of entrained flow can effect the hydraulic performance of deflectors. Tests on both the McNary and John Day Dams were performed with and without the end bay training walls. Training walls stopped entrained flow from entering the jet from the side immediately downstream of the deflector. Tests conducted at WES confirm that the amount of flow entrained into the jet downstream of Bay 20 RSW represents a significant percentage of the total spilled flow.

5. Recommended Model Modifications

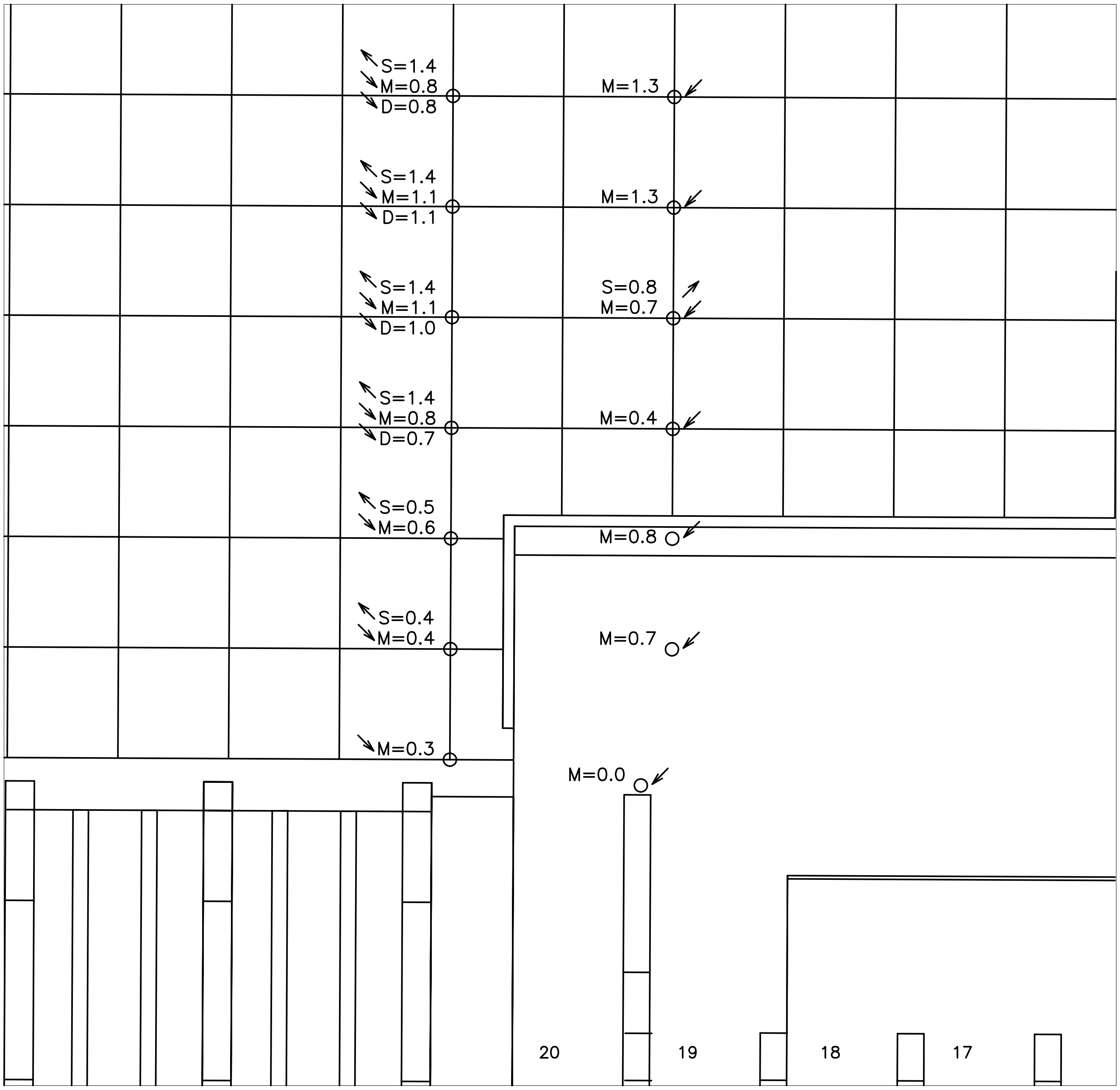
Based on the observations made during this December 2000 WES trip a significant volume of water is entrained into the Bay 20 RSW jet and this entrained flow should be incorporated in the sectional model (Task 1a of Contract DACW57-97-D-004, Task Order 21).

The recommended model design is presented below. It includes a head tank located left of Bay 20. The head tank will extend from face of the powerhouse to 150 feet downstream of the end sill. Diffused flow will enter the model from the left. Flow will be drawn into the jet through turnings vanes and needles. Calibration tests will be required to determine the orientation of the turning vanes and needle spacing. The needles allow for the entrainment flow to be effectively shut off in order to conduct sensitivity tests, if required.





Ken Christison, P.Eng.
Hydraulic Engineer



TEST 1 River Discharge = 4250 m³/s (150,000 cfs)
Spillway Discharge = 0 m³/s (0 cfs) – 0% Spill
Powerhouse Flow = 135,500 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 48.62 m (159.5 ft)

NOTES:

Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

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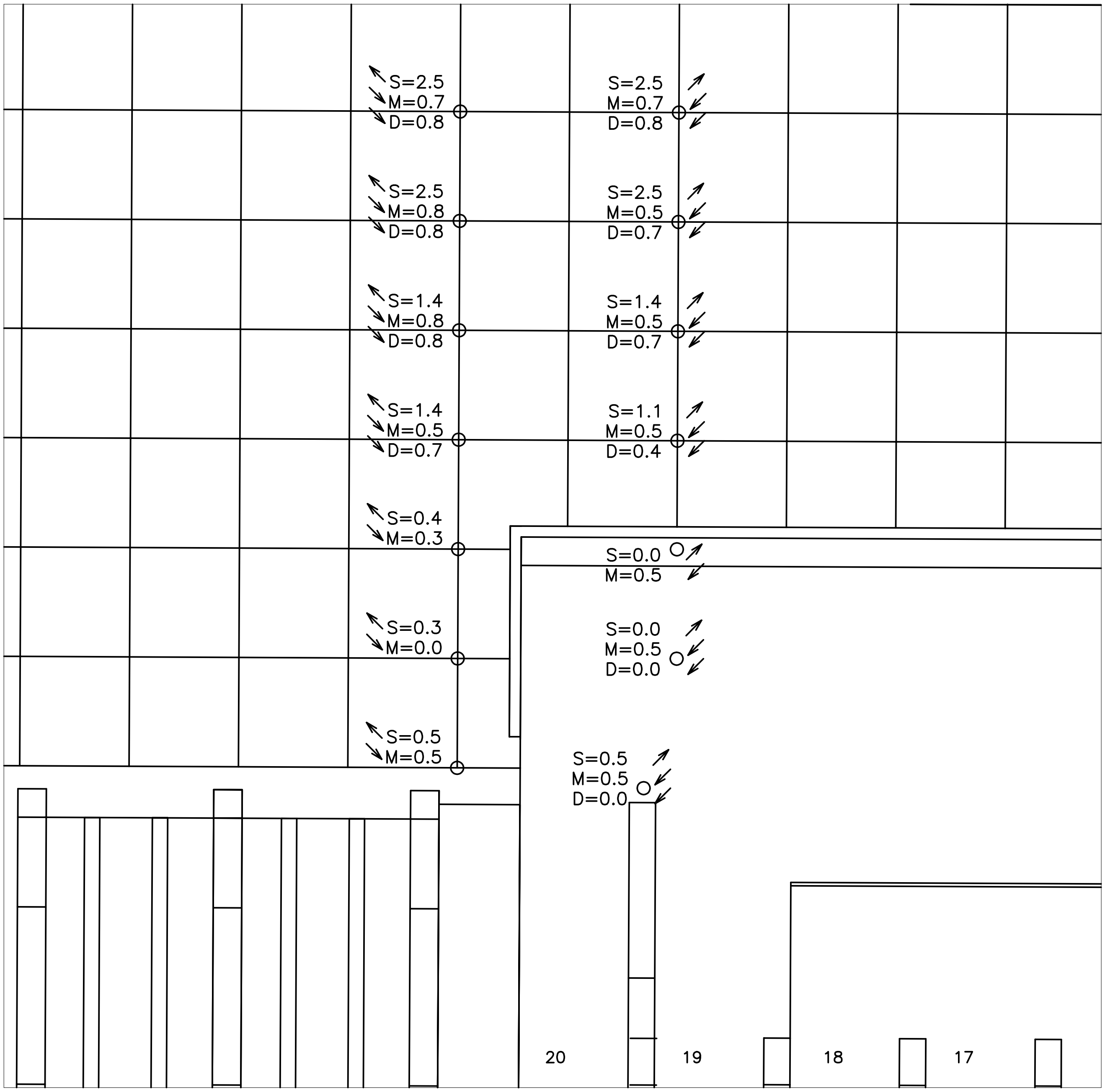
JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 1

northwest hydraulic consultants

FIGURE 5



TEST 2 River Discharge = 4250 m³/s (150,000 cfs)
Spillway Discharge = 9266 m³/s (30,400 cfs) – 20% Spill
Powerhouse Flow = 105,100 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 48.62 m (159.5 ft)

NOTES:
Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

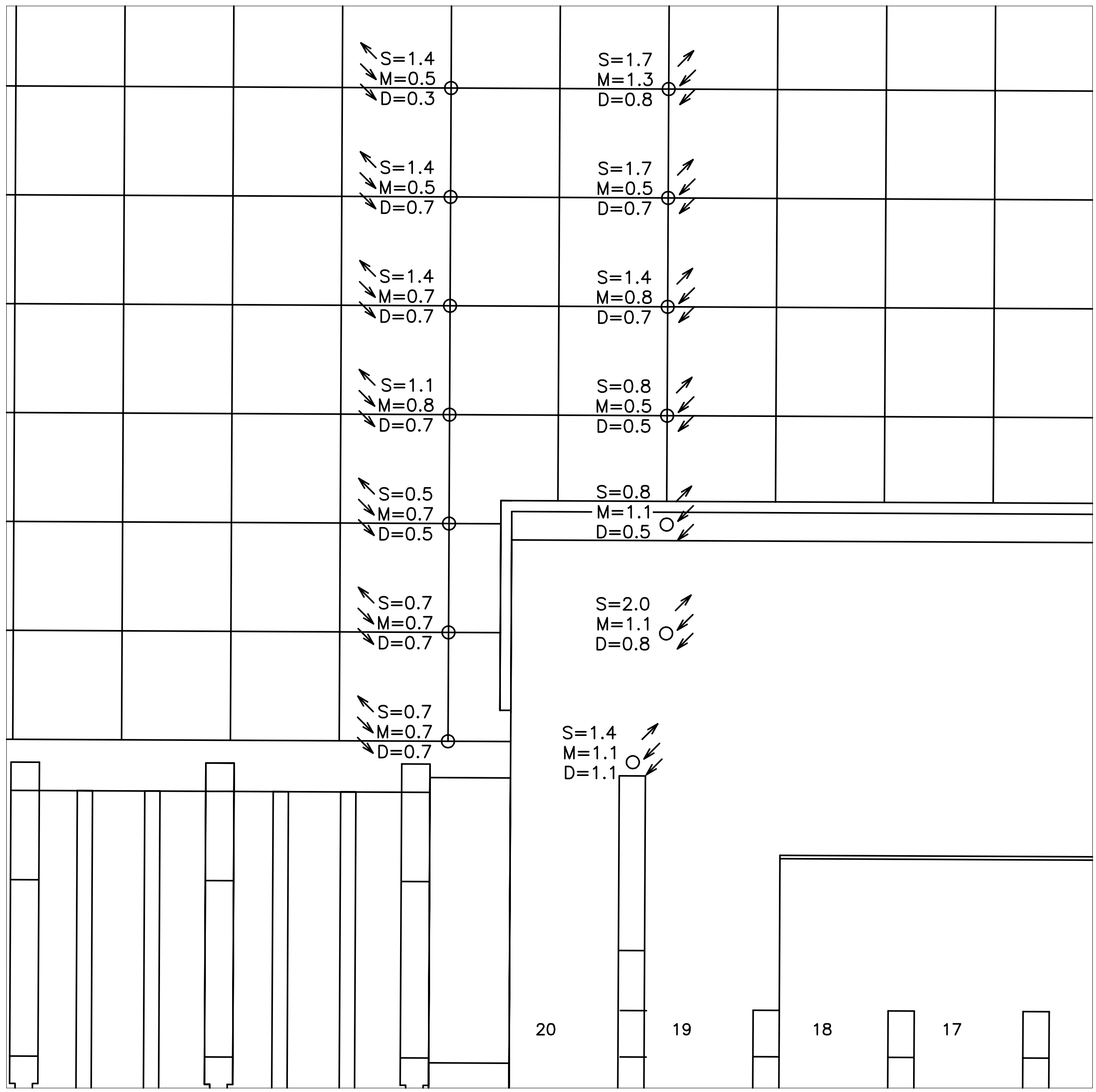
NHCV 0000–000

JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 2

northwest hydraulic consultants

FIGURE 6



TEST 3 River Discharge = 4250 m³/s (150,000 cfs)
Spillway Discharge = 13,655 m³/s (44,800 cfs) – 30% Spill
Powerhouse Flow = 90,700 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 48.62 m (159.5 ft)

NOTES:

Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

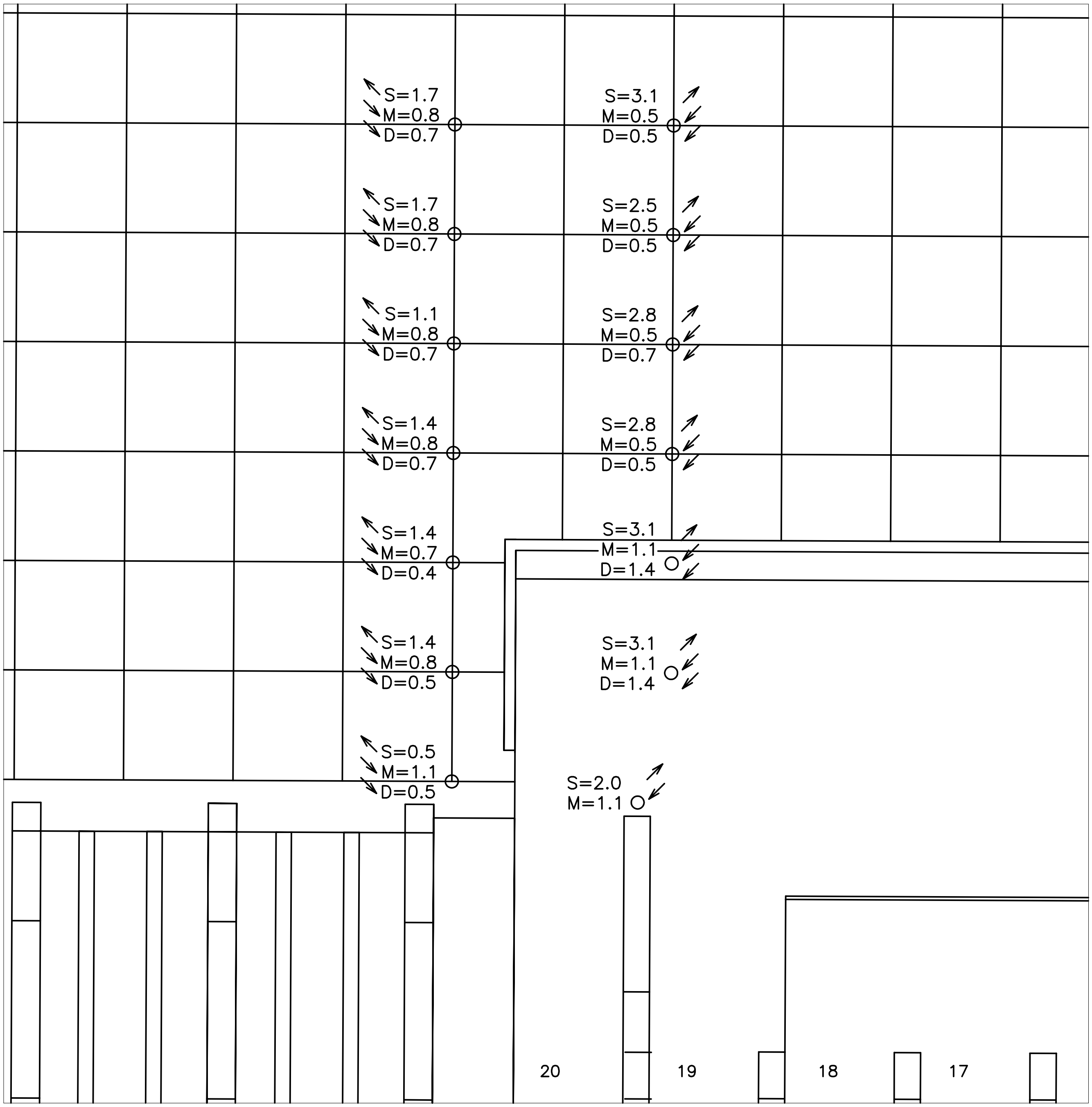
NHCV 0000–000

JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for
Powerhouse Entrainment
Test 3

northwest hydraulic consultants

FIGURE 7



TEST 4 River Discharge = 4504 m^3/s (159,000 cfs)
Spillway Discharge = 30,053 m^3/s (98,600 cfs) – 60% Spill
Powerhouse Flow = 45,900 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 48.62 m (159.5 ft)

NOTES:

Velocities shown in m^3/s (1 m^3/s = 3.3 ft^3/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

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HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 4

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FIGURE 8

TEST 5 River Discharge = 9918 m^3/s (350,000 cfs)
Spillway Discharge = 15,605 m^3/s (51,200 cfs) – 15% Spill
Powerhouse Flow = 284,300 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 50.02 m (164.1 ft)

NOTES:

Velocities shown in m^3/s (1 m^3/s = 3.3 ft^3/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid-depth
D – Velocities measured 2 m above channel invert

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HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 5

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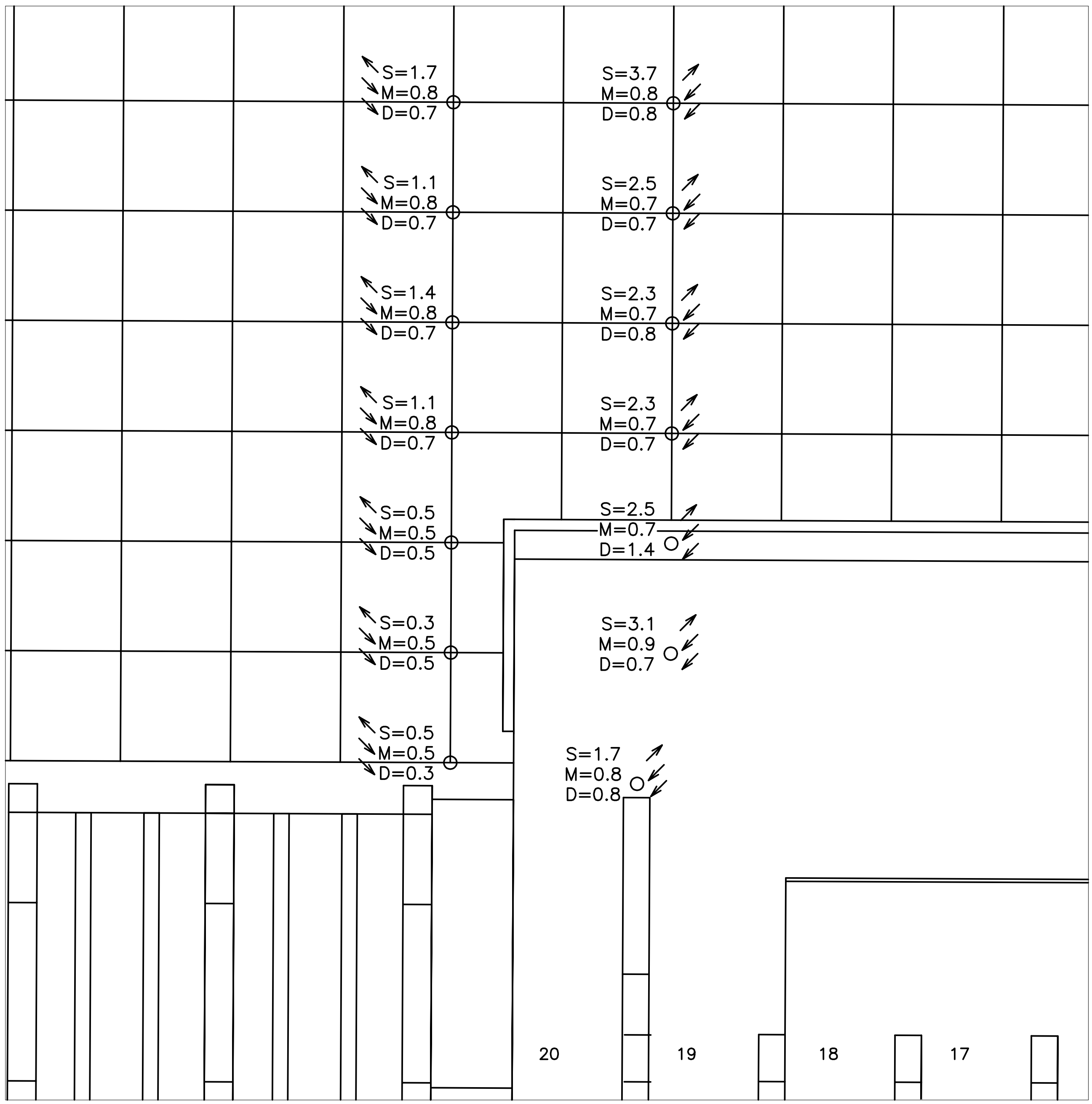
NHCV 0000-000

FIGURE 9



TEST 6 River Discharge = 9918 m^3/s (350,000 cfs)
Spillway Discharge = 32,186 m^3/s (105,600 cfs) – 30% Spill
Powerhouse Flow = 229,900 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 50.02 m (164.1 ft)

NOTES:

Velocities shown in m^3/s (1 m^3/s = 3.3 ft^3/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

NHCV 0000–000

JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 6

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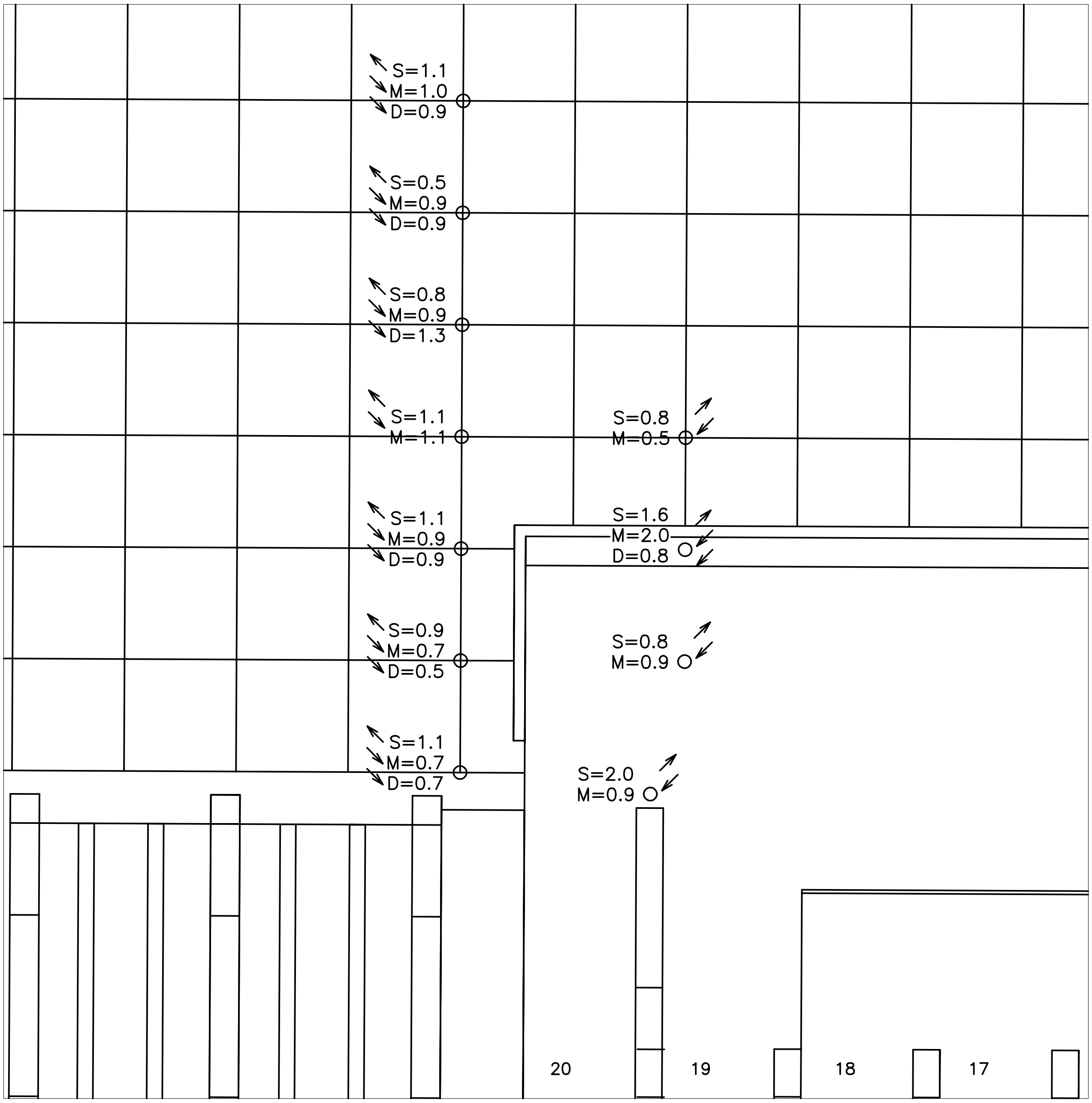
FIGURE 10



TEST 7 River Discharge = 9918 m³/s (350,000 cfs)
Spillway Discharge = 63,886 m³/s (209,600 cfs) – 60% Spill
Powerhouse Flow = 125,900 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 50.02 m (164.1 ft)

NOTES:

Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

NHCV 0000–000

JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 7

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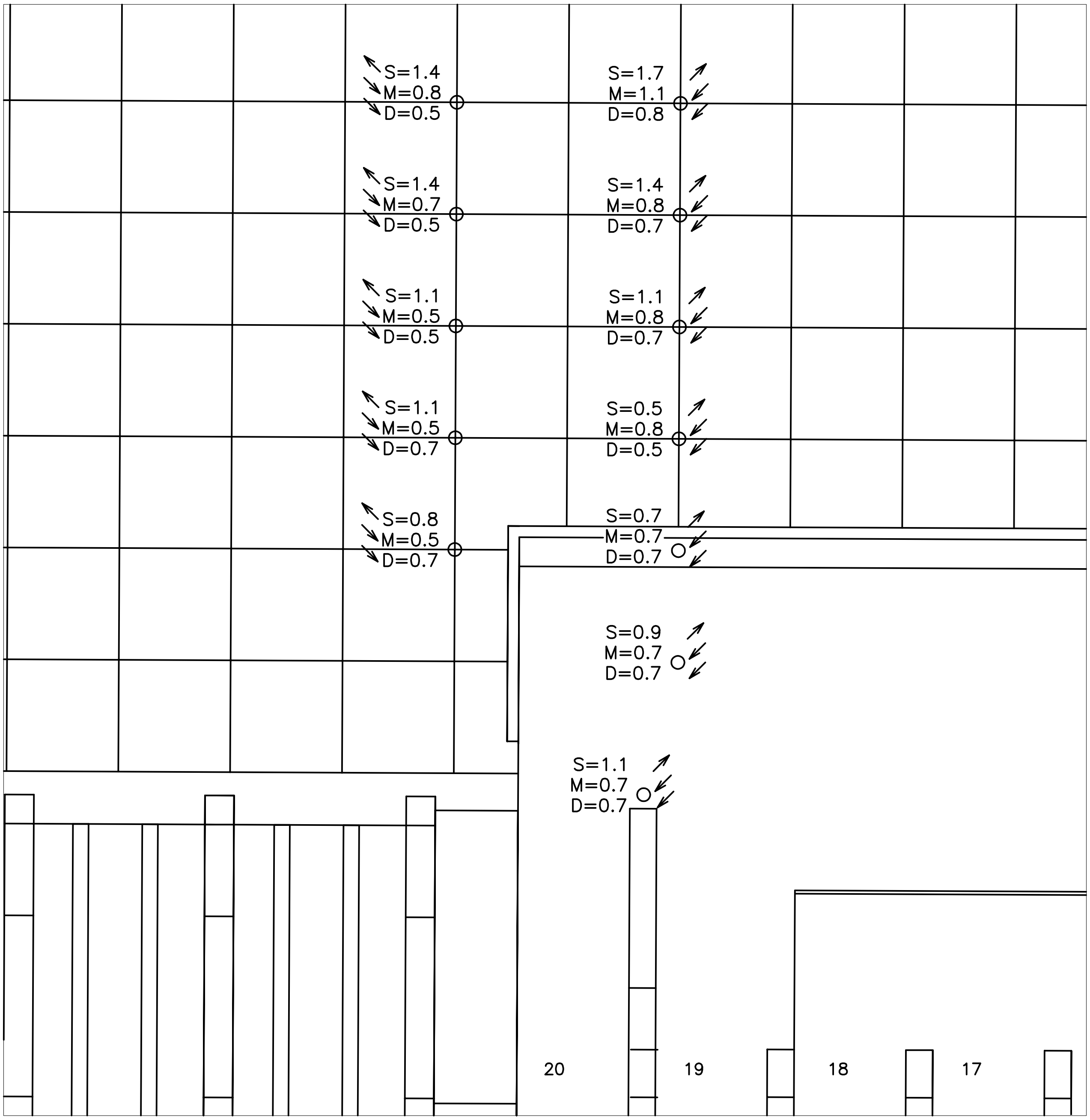
FIGURE 11



TEST 8 River Discharge = 7742 m^3/s (250,000 cfs)
Spillway Discharge = 0 m^3/s (0 cfs) – 0% Spill
Powerhouse Flow = 235,500 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 49.28 m (161.7 ft)

NOTES:

Velocities shown in m^3/s (1 m^3/s = 3.3 ft^3/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

NHCV 0000–000

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HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 8

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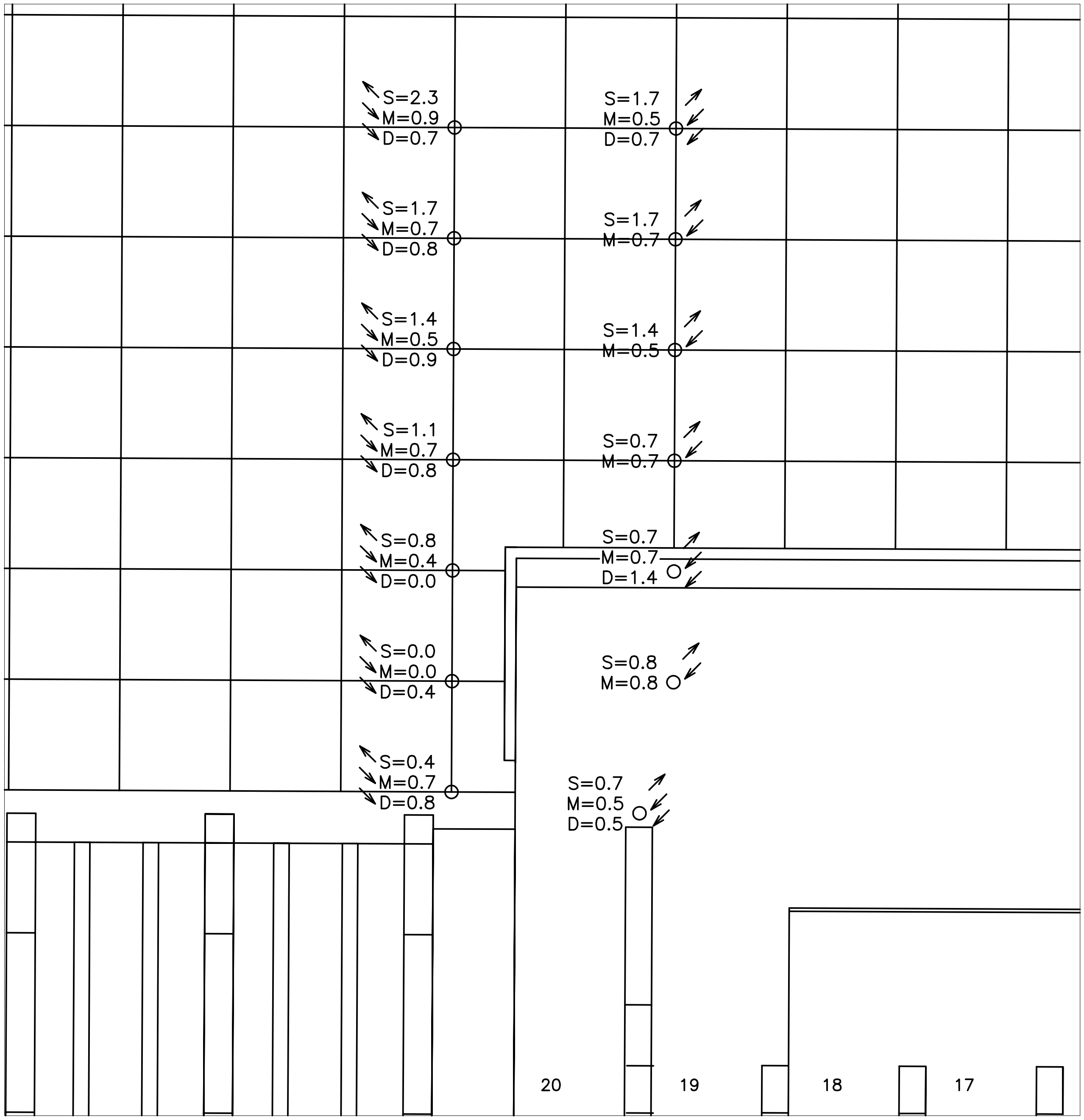
FIGURE 12



TEST 9 River Discharge = 7084 m³/s (250,000 cfs)
Spillway Discharge = 15,118 m³/s (49,600 cfs) – 20% Spill
Powerhouse Flow = 185,900 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 49.29 m (161.7 ft)

NOTES:

Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid-depth
D – Velocities measured 2 m above channel invert

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HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 9

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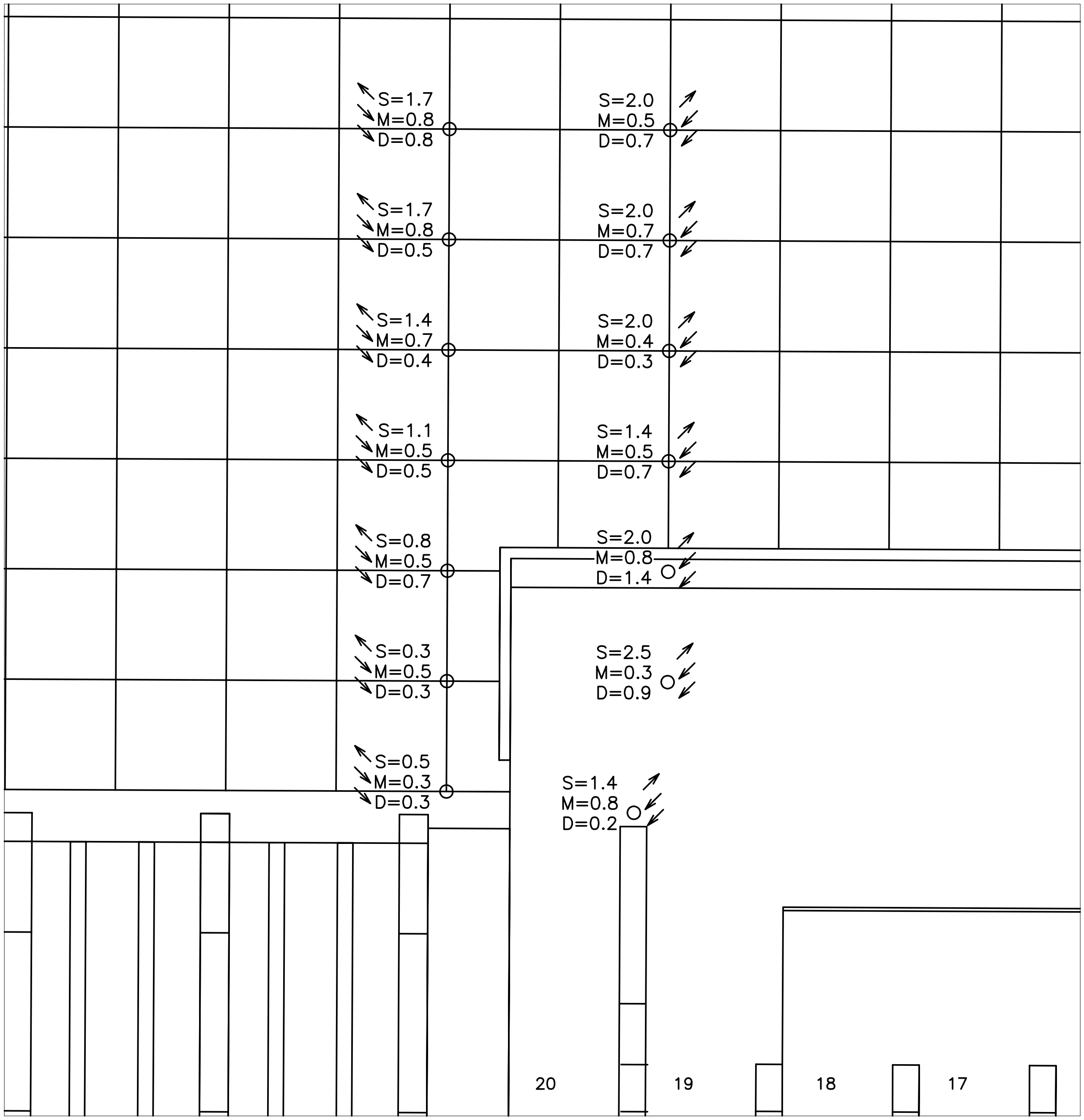
FIGURE 13



TEST 10 River Discharge = 7084 m³/s (250,000 cfs)
Spillway Discharge = 22,920 m³/s (75,200 cfs) – 30% Spill
Powerhouse Flow = 160,300 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 49.35 m (161.9 ft)

NOTES:

Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

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JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 10

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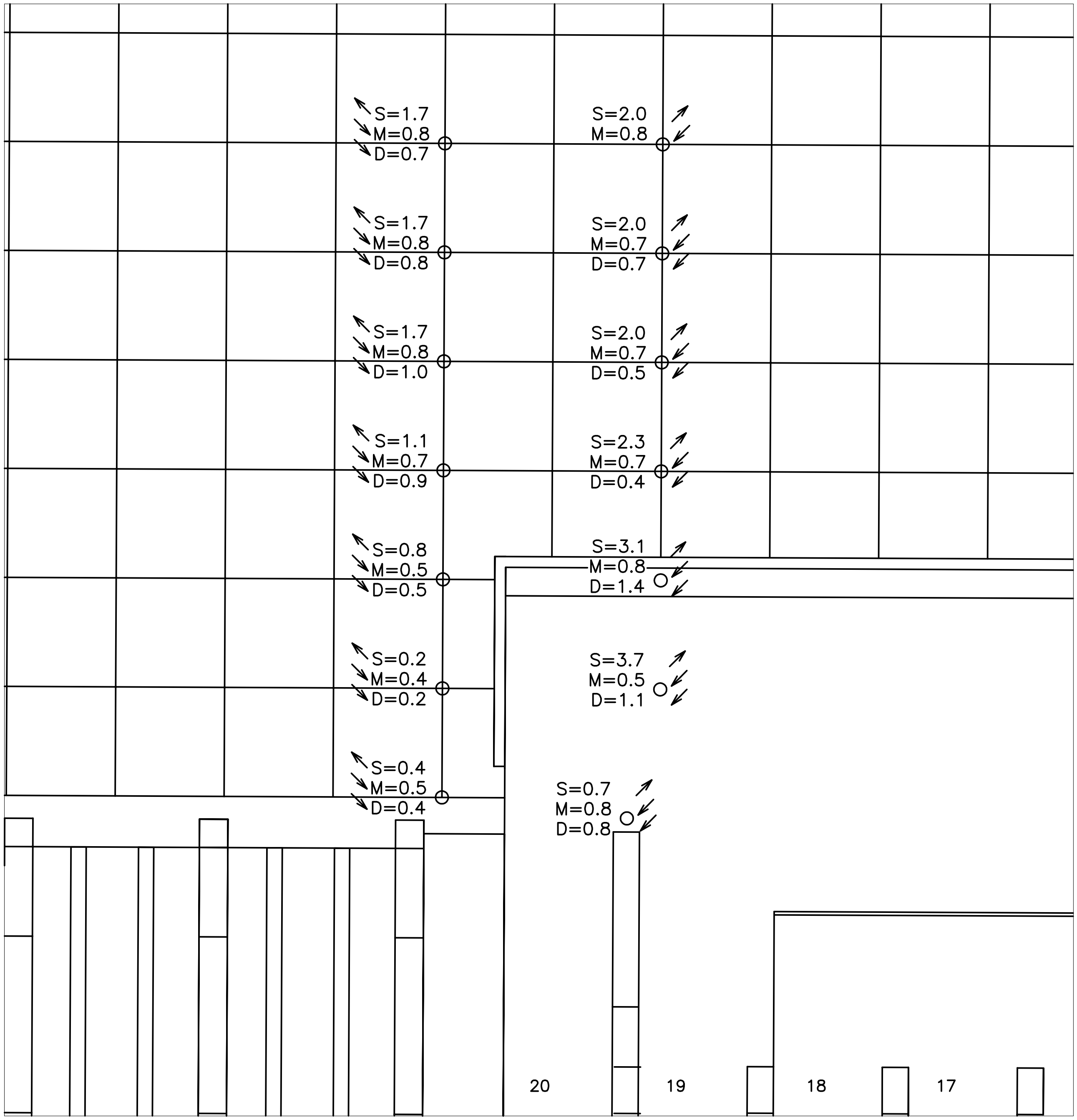
FIGURE 14



TEST 11 River Discharge = 7084 m³/s (250,000 cfs)
Spillway Discharge = 22,920 m³/s (75,200 cfs) – 30% Spill
Powerhouse Flow = 160,300 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 48.70 m (159.8 ft)

NOTES:

Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

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HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 11

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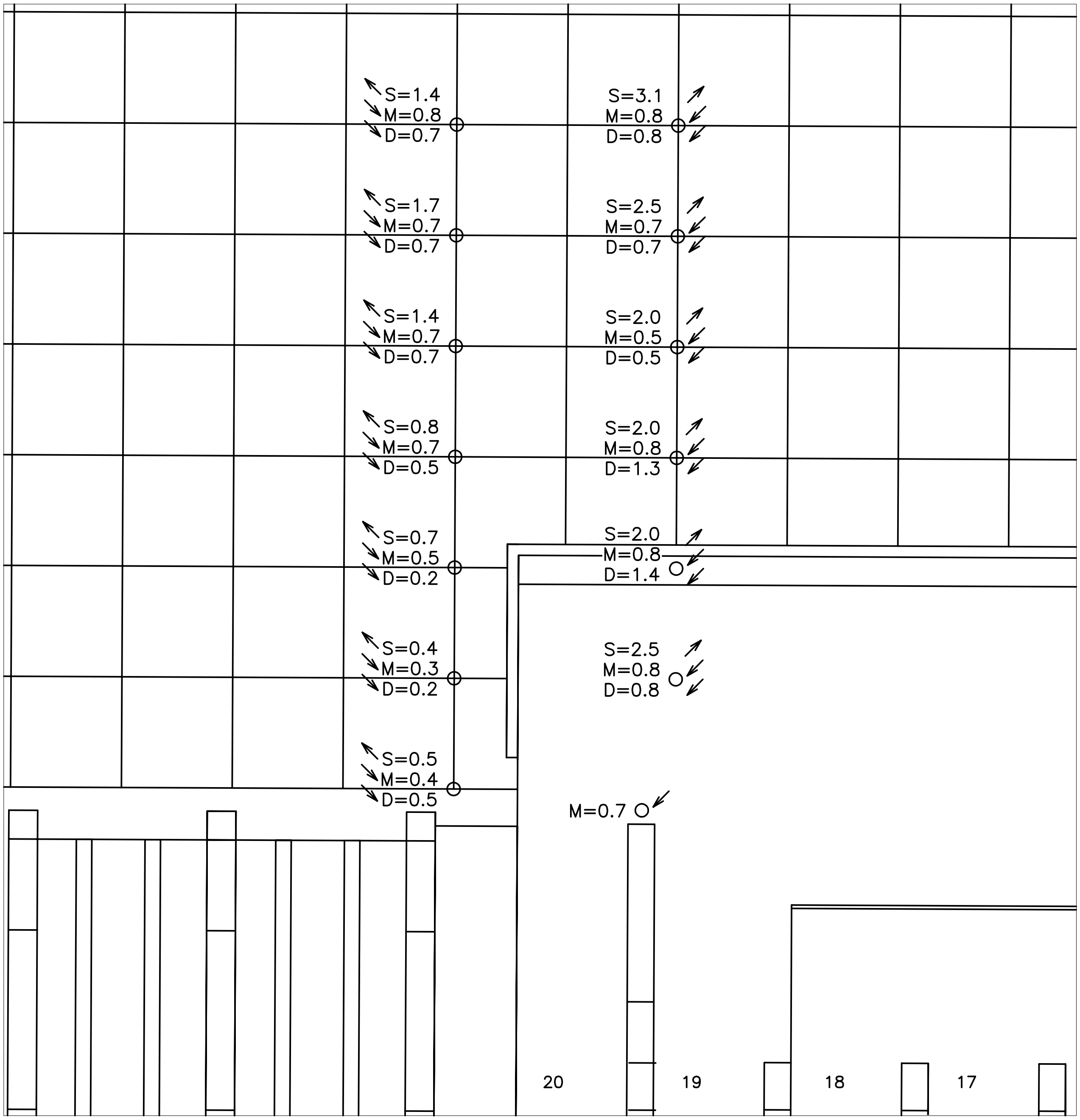
FIGURE 15



TEST 12 River Discharge = 7084 m^3/s (250,000 cfs)
Spillway Discharge = 22,920 m^3/s (75,200 cfs) – 30% Spill
Powerhouse Flow = 160,300 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 49.77 m (163.3 ft)

NOTES:

Velocities shown in m^3/s (1 m^3/s = 3.3 ft^3/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

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JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 12

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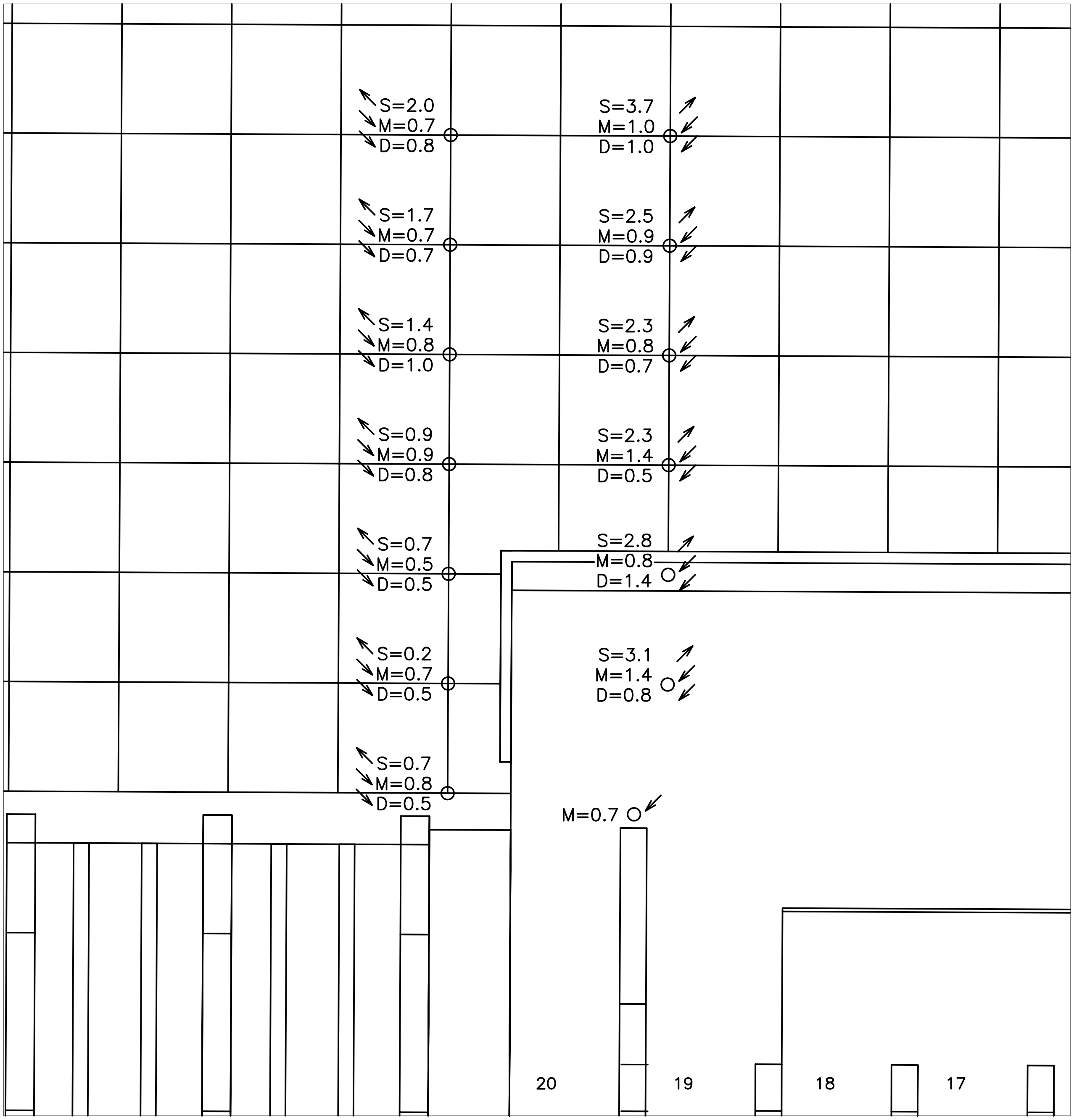
FIGURE 16



TEST 13 River Discharge = 9918 m³/s (350,000 cfs)
Spillway Discharge = 32,186 m³/s (105,600 cfs) – 30% Spill
Powerhouse Flow = 229,900 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 50.41 m (165.4 ft)

NOTES:

Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

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JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for Powerhouse Entrainment Test 13

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FIGURE 17

TEST 14 River Discharge = 9918 m³/s (350,000 cfs)
Spillway Discharge = 32,186 m³/s (105,600 cfs) – 30% Spill
Powerhouse Flow = 229,900 cfs
Bay 20 (RSW) Flow = 14,000 ft
Misc. Flow = 500 cfs
Forebay WSE = 80 m (264 ft)
Tailwater WSE = 49.65 m (162.9 ft)

NOTES:

Velocities shown in m³/s (1 m³/s = 3.3 ft³/s).
O – Location of velocity measurements
S – Velocities measured 2 m below WSO
M – Velocities measured at mid–depth
D – Velocities measured 2 m above channel invert

NHCV 0000–000

JOHN DAY DAM RSW
HYDRAULIC MODEL STUDY

Model Calibration Data for
Powerhouse Entrainment
Test 14

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FIGURE 18

Dec 15, 2000

MEMORANDUM FOR: RECORD

SUBJECT: John Day RSW Model Demonstration at WES, 5-7 Dec 2000

1. Performance of the John Day Removable Spillway Weir (RSW) Proof of Concept Alternative 5 (Optimum Alternative C) was demonstrated in the 1:80 scale general model at WES during the week of 5-7 December 2000. The purpose of the model demonstration was to compare the approach condition and egress condition performance of the RSW with the Skeleton Bay Surface Collector (SBSC). Participants in the demonstration were Mr. Chris Goodell and Ms Diana Modini of the COE's Portland District Hydraulic Design Section, Messrs. Dave Maggio and Don Wilson (part time) of WES and Mr. Jim Lencioni of NHC's Seattle office. On 7 and 8 Dec, Mr. Ken Christison of NHC's Vancouver office measured velocity data adjacent to the RSW exit jet for use in NHC's 1:25 scale model bay 20 deflector test program.

2. The Alternative 5 RSW / SBSC comparison testing was accomplished at total project release discharges of 250,000 cfs and 350,000 cfs. For the 250 Kcfs condition, spillway discharges of zero, ~75 Kcfs, and ~100 Kcfs representing spillway discharges of 0, 30 and 40 percent of total release were demonstrated. For the 350 Kcfs condition, spillway discharges of zero, ~51 Kcfs, ~ 105 Kcfs and ~210 Kcfs representing spillway discharges of 15, 30 and 60 percent of total release were demonstrated. The spillway operating pattern was based on 1996 spill conditions which prioritized spill from the north side of the spillway. All tests were accomplished with a reservoir elevation of 264 ft. The tailwater elevation for the 250 Kcfs simulations varied between about 161.7 and 162 ft. For the 350 Kcfs condition, the tailwater elevation varied between about 164.1 and 164.2 ft. The tailwater elevations were set approximately to the median value of the upper and lower range of possible tailwater elevations existing at the project for those discharges. The discharge of the RSW is about 14 Kcfs and the discharge of the SBSC is about 18.9 Kcfs. The majority of the remaining portion of the total project release was passed through the powerhouse with a small (~500 cfs) release through the fish ladders.

3. Following are general qualitative observations made based on cursory observation of dye traces and confetti movement. More detailed quantitative measurements are necessary to provide definitive comparative data.

a. <u>250 Kcfs project release, no spillway discharge</u>.

(1) <u>Approach Conditions</u>. With the RSW operating, a strong zone of influence extends about 100 ft upstream and to the sides of the RSW. A noticeable zone of lateral approach influence extends to about spillway bay 16/17 pier and about powerhouse skeleton bay 17/18 joint. Dye deposited below the upstream approach ramp to the spillway crest was not drawn up into the RSW flow. Confetti movement on the surface drew from about 700 ft (prototype) upstream,

but the velocity was too low to record with a Nixon velocity meter at a location about 150 ft (prototype) upstream. Hydraulic characteristics over the RSW and onto the existing spillway face were similar to those exhibited in the 1:25 scale section model except that the surface waves were smaller in the 1:80 scale model because of the smaller scale.

With the SBSC operating, approach flow conditions from upstream and the spillway side of the dam was quite similar to those with the RSW operating. However, the lateral zone of influence on the powerhouse side of the SBSC seemed to extend only to about the middle of powerhouse skeleton bay 18, not as far as with the RSW operating. Hydraulic characteristics over the broad crested weir were rougher (more surface disturbances) than exhibited on the RSW. However, flow characteristics down the steep downstream face of the SBSC were not much different from that observed on the RSW.

(2) <u>Egress Conditions</u>. The RSW flow travels about 1000 ft downstream where it begins to be pushed towards the north bank by the powerhouse flows. A strong eddy is formed by the large powerhouse flow with no spillway flow and draws about 50% of the RSW release back towards the spillway where it terminates in a rather dead zone. The spillway deflector in the model is set at elevation 148 ft and has a submergence of about 13-ft at this flow condition. At that elevation, the flow exiting the deflector appears to be about in the transition range between undular flow and a hydraulic jump.

SBSC flow is very similar to that with the RSW operating except that the eddy downstream of the spillway appears to draw a bit less flow than exists with the RSW operating. I estimate that on the order of 35-40% of the flow is drawn into the eddy. Flow exiting from the SBSC deflectors is smoother and more in a skimming regime than exists with the RSW. The reason for this difference is that the SBSC deflectors are set at elevations of 157 and 160 ft, therefore the submergence on the deflectors is significantly less than on the RSW deflector constructed in the model. The optimum deflector elevation for total dissolved gas "control" is being developed in the 1:25 scale section model at NHC.

b. <u>250 Kcfs project release, 75 Kcfs spillway discharge</u>.

(1) <u>Approach Conditions.</u> With the RSW operating, a relatively strong lateral draw existed near the spillway bay 14/15 pier with some draw from as far away as the bay 12/13 pier. From the south side of the reservoir, flow was drawn from near the middle of powerhouse skeleton bay 19. Dye deposited on the floor of the reservoir near the RSW indicated that the flow through spillway bay 19 adjacent to the RSW was influential in drawing flow from near the reservoir floor into the RSW.

With the SBSC operating, noticeable draw existed from near the middle of spillway bay 12 to the north and skeleton bay 17/18 joint to the south.

(2) Egress Conditions. RSW releases moved predominantly downstream with the majority of the flow travelling to the north side of the channel with a small amount down the center. The spillway discharge was sufficient to move the RSW release downstream and no significant backflow extended to the stilling basin.

Flow from the SBSC moved downstream in a very similar manner as occurred with the RSW except that a more equal distribution of the SBSC release moved to the south bank as towards the north bank. In terms of overall downstream flow distribution, the SBSC operation may be somewhat better than the RSW operation.

c. 250 Kcfs project release, 100 Kcfs spillway discharge.

(1) Approach Conditions. With the RSW operating, the lateral zone of influence begins to be noticeable near the spillway bay 13/14 pier and the joint between skeleton bays 17/18. With the SBSC operating, the zone of influence is stronger from the spillway side of the project than from the powerhouse side. With either the RSW or SBSC in operation, the large amount of spillway flow tends to decrease the strength of approach flow drawn towards the RSW or SBSC as compared to lesser spillway flow conditions.

(2) Egress Conditions. RSW releases moved predominantly downstream with about 50% in the center of the channel and 25% towards each the north and south shores. SBSC releases tended to move towards the south shore and some backflow was evident towards the powerhouse tailrace. Overall egress conditions appeared better with the RSW operating than with the SBSC operating.

d. 350 Kcfs project release, 51 Kcfs spillway discharge.

(1) Approach Conditions. RSW operation attracted flow from about the spillway bay 13/14 pier on the north side and about the skeleton bay 18/19 joint on the south. Overall approach attraction from upstream was generally quite a bit stronger from the north (spillway) side of the reservoir than from the south side based on dye movement. The SBSC operation attracted flow from about the spillway bay 11/12 pier to the north and about the middle of skeleton bay 18 to the south. Confetti travel time from upstream was somewhat longer with the SBSC than with the RSW suggesting a somewhat overall attraction flow velocity with the RSW.

(2) Egress Conditions. Release flow from the RSW tracked generally directly downstream for a distance of about 2000 ft at which point about 20% of the flow tended to split towards the north shore. Confetti indicated a clockwise eddy

downstream of skeleton bays 17 and 20. Releases from the SBSC tended to travel somewhat more towards the north shore than did the RSW release. Dye traces indicated that a small portion of flow migrated back upstream towards the spillway with SBSC operation. Hydraulic performance of the SBSC deflector remained in a skimming flow regime even with the higher tailwater elevation existing with the larger project release.

e. <u>350 Kcfs project release, 105 Kcfs spillway discharge</u>.

(1) <u>Approach Conditions</u>. The RSW draws from the north to about the spillway bay 11/12 pier and from the south to about the middle of skeleton bay 19. The SBSC draws generally from the same area to the north while the zone of influence extends to the south to about the skeleton bays 18/19 intersection. The larger spillway flow at this condition effectively reduces the overall approach flow effectiveness to either the RSW or SBSC.

(2) <u>Egress Conditions</u>. Downstream flow tracked essentially the same with both the RSW and the SBSC operating. Overall egress conditions were good with about a 50-50 split between the north and south shores.

f. <u>350 Kcfs project release, 210 Kcfs spillway discharge</u>.

(1) <u>Approach Conditions</u>. Lateral draw with the RSW extended to about spillway bay 9 to the north and the middle of skeleton bay 18 to the south. With the high spillway flow, approach flow from upstream was greatly directed towards the spillway making rendering both the RSW and SBSC relatively inefficient in capturing flow from upstream.

(3) <u>Egress Conditions</u>. RSW releases generally travel directly downstream except that a small portion migrates to the north shore immediately downstream from the end of the stilling basin. As the flow extends downstream further, the strong spillway flow moves the majority of flow back towards the south bank. With SBSC operation, releases from the SBSC travel approximately in a 45-degree direction towards the south shore. A large counter-clockwise eddy develops and brings a significant amount of flow back towards the powerhouse.

4. On 7 Dec, observations were made of RSW operation at a total project release of 150,000 cfs at the same time that measurements were being made to quantify powerhouse flow entrainment into the jet exiting from the RSW. These tests were made at spillway releases of zero, 30 Kcfs, 45 Kcfs and 90 Kcfs with a tailwater elevation of 159.5 ft. With no spillway flow, the large counterclockwise eddy that formed downstream from the spillway with the larger total project releases again occurred. At 30 and 45 Kcfs spillway flow, a very uniform downstream flow pattern across the entire width of the downstream channel existed. However, with 90Kcfs spillway flow, the RSW flow tracked entirely towards the south shore and some backflow existed upstream into the powerhouse tailrace area. The jet exiting from the deflector was much smoother and

exhibited more of a skimming flow regime that existed at the higher project releases due to the decreased tailwater elevation at 150,000 cfs.

5. Summary. In general, the RSW emulated the SBSC very well both in approach and egress flow conditions. In both cases, the egress conditions with no or small spillway releases resulted in a large clockwise eddy extending up into the stilling basin. This is not considered to be an acceptable condition with regards to satisfactory fish passage. With the spillway passing more than about 10 percent of the total project release, relatively uniform downstream flow movement existed. The flow exiting the SBSC deflector was smoother and of more of a skimming regime than existed with the RSW. The reason for this difference is due to the difference in elevation of the deflector, which creates a significantly greater submergence on the RSW than on the SBSC. The elevation, and length, of the RSW deflector is being studied in detail in the 1:25 scale section model at the NHC laboratory. Overall approach velocity from upstream appeared to be somewhat faster with the RSW operating than with the SBSC operating based on cursory confetti travel time measurements. Both surface collectors generally tended to draw flow laterally from as far away as near spillway bays 11/12 and from near skeleton bay 17. A strong attraction existed within about 100 ft (prototype) of the RSW and SBSC from all directions. Increased spillway flow rates tended to diminish the overall effectiveness of attraction flow from upstream to either the RSW or SBSC.

JAMES L. LENCIONI, P.E.
Sr. Hydraulic Engineer

Cc:

Chris Goodell / Diana Modini, CE-NWP
Al Babb / Ken Christison, NHC-Vancouver
Ed Zapel, NHC-Seattle
Dick Regan, NHC-Seattle

26 FEBRUARY-2 MARCH 2001 VISIT TO
NORTHWEST HYDRAULIC CONSULTANTS LABORATORY

16 Mar 2001

MEMORANDUM FOR: Record

SUBJECT: John Day SW Bay 20 Deflector, 26 Feb-2 Mar 2001 Model Demo

1. The 1:25 scale model for the proposed John Day spillway bay 20 deflector was demonstrated for Portland District Corps of Engineers (NWP) staff at the Northwest Hydraulic Consultants' (NHC) laboratory facilities during the period 26 Feb through 2 March 2001 in accordance with Task 3a (Preliminary Deflector testing) of the SOW. The RSW tailpiece section was also demonstrated as part of Task 5. Participants at the demonstrations were:

Chris Goodell – NWP Hydraulic Design Section
Blaine Ebberts – NWP Environmental Resources Section
Dennis Dorratcague – Montgomery Watson
Al Babb – NHC, Physical Model Principal Investigator
Ken Christison – NHC, Physical Model Project Engineer
Jim Lencioni – NHC Project Manager
Dick Regan (part time) – ITR Reviewer
Mike Schneider (part time) - WES

2. Prior to demonstration of the physical model, Ken Christison presented a summary of a proposed model demonstration agenda. The deflector testing was to be accomplished in the section model installed in the John Day flume. That flume had been modified to incorporate about 2,000 cfs entrainment into the flume downstream from the deflector from the powerhouse side of spillway bay 20 based on results of measurements made in the 1:80 scale general model at WES in December. Deflector radii of 20 ft, 35 ft and 50 ft ; and lengths of 25 ft, 30 ft and 35 ft had been pre-constructed. The radii and lengths were inter-changeable so that any combination could be demonstrated. Removal and installation of the separate deflector designs required about 2 hours. The proposed demonstration was to operate each selected deflector geometry at a pool elevation of 264 ft and various tailwater elevations. The intent was to roughly identify the tailwater at which the hydraulic flow regime transitioned from plunging to skimming, skimming to undular, and undular to hydraulic jump. General flow characteristics on the deflector were also to be reviewed. The design and intended use of the RSW tailpiece section was discussed. The proposed ogee shape tailpiece had been installed in the old McNary flume and was scheduled to be demonstrated on the morning of February 27. The ogee-shaped design was developed for the DDR as an attempt to achieve optimum hydraulic performance should the tailpiece section need to be operated alone either for biologic or flood control reasons. However, in order to fit into the available space on the spillway crest between the bulkhead slot and the spillway gate, the tailpiece ogee was so under-designed (He : Hd ratio of about 9) that safe operation of the tailpiece by itself under high flow conditions was extremely doubtful. Further, the connecting plate between the main RSW and the tailpiece obviously created potential structural integrity concerns and was

creating design issues. COE staff stated that operation of the tailpiece section, either in a gated or free-flow condition, by itself would not be required except possibly in a remote, upstream dam failure type scenario. Given that, Dick Regan suggested that a simplified tailpiece shape would probably present no more hydraulic issues (i.e., low pressure) than the proposed ogee shaped design and would present less of a structural stability problem than did the ogee shape. The group decided that the ogee shape design in the DDR should be revised to a more simplified shape. NHC agreed to expediently install and accomplish some preliminary testing of a simplified shape later in the week if possible to compare hydraulic performance of the ogee and simplified shape.

3. Six deflector designs, all at an elevation of 150 ft, were demonstrated. They were (1) R=50', L=30'; (2) R=20', L=30'; (3)R=20', L=25'; (4)R=35', L=25'; (5) R=35', L=35'; and R=50', L=50'. The 50' radius with a length of 30' was installed and demonstrated on two separate occasions. A powerhouse entrainment flow of 2,000 cfs was supplied to the model. In addition to identifying the flow regime transition tailwater elevations, dye movement was observed to attempt to better define the spatial (depth, lateral and longitudinal) dispersal of flow downstream from the deflector. Table 1 is a summary of the tailwater elevations at which the flow regime transitioned. In general, the hydraulic characteristics on the deflector improved as the radius increased. The 50-ft radius exhibited a pronouncedly more stable and smooth flow than did the smaller radii. With the 20 ft radius, wave rideup of 5-7 ft existed along the deflector on both pier faces. This rideup created roostertails which exited from the deflector in a trajectory and impacted in the stilling basin. With the 35 ft radius the wave rideup decreased to about 3-4 ft and with the 50 ft radius, the wave rideup decreased further to about 1-2 ft. The undular and hydraulic jump regimes appeared to entrain less dye at depth than did the skimming regime. Limited operation of the model with powerhouse entrainment flows increased from 2,000 cfs to about 4,000 cfs did not appear to change the deflector hydraulic perfromance characteristics.

4. Operation of the ogee-shaped tailpiece section showed, as expected, that extreme flow separation occurred over the crest with even small gate openings (6 ft and above) at pool elevation 268 ft and with a pool elevation of about 228 (9 ft of head on the crest) under free flow conditions. Static pressures as low as minus 10 ft existed with a gate opening of 8 ft and decreased to as low as minus 25 ft at a gate opening of 10 ft. Table 2 illustrates some preliminary pressure data observed on the crest. On the morning of March 2, a simplified triangular shaped crest was installed over the ogee shaped section and demonstrated at various gated flow conditions. Flow separation with this design actually appeared to be less than with the ogee shaped design. Dye indicated some separation at small gate openings but separation could not visually be observed without dye until the gate opening was about 15 ft (as compared with about 6 ft for the ogee shape). Under free flow conditions, extreme flow separation occurred over the crest at a pool elevation of about 260 ft. With a pool of 274 ft, pressures in the range of minus 20 to minus 22 ft existed about 5 ft downstream from the crest apex. Only limited pressure data could be obtained with the simplified shape because some of the pressure taps on the ogee shape were covered over by the simplified section. However, where comparative measurements could be made, the simplified tailpiece design appeared to actually have

higher pressures than did the ogee shape. Table 3 illustrates the preliminary pressure data obtained on the simplified triangular shaped tailpiece section during the demonstration.

5. At the conclusion of the model demonstration, the group convened to discuss follow up actions and schedules. The consensus reached was that the 50 ft transition radius design afforded the best hydraulic performance and was therefore selected for more detailed performance testing. Mike Schneider commented that (1) some of the observations made may be the result of the dimensional effects in the section model that may not actually exist in the prototype, (2) the large hole in the prototype tailrace d/s from bay 20 was not captured in the model and (3) the lateral powerhouse entrainment flow in the section model did not seem as strong as exists in the 1:80 scale general model. In response to the latter two comments, NHC staff stated that deflector test observations made in the McNary model indicated that deeper tailrace depths did not significantly affect deflector hydraulic performance results in that model. Regarding the powerhouse entrainment flow, the powerhouse entrainment flow of 2,000 cfs being supplied in the section model was based upon the measurements made on the general model in December. The model does have the capability of adding up to 10,000 cfs powerhouse entrainment flow. The group consensus was that a powerhouse entrainment flow of 2,000 cfs should continue to be used in the section model. The following decisions and schedule were developed:

a. Priority 1: NHC will document pressures on the main RSW crest at pool elevations of 264 ft and 268 ft. Pressure measurements will include four electronic transducers to capture the dynamic pressure regime (amplitude and frequency). NHC lab staff will develop the dynamic amplitude and frequencies. These transducers will be located as near the RSW crest as possible without encroaching onto the curved section of the crest, on the main RSW section downstream from the PT, near the connecting plate between the main RSW and Tailpiece section and on the Tailpiece section. <u>The documentation data will be furnished to Dennis Dorratcague by March 9.</u>

b. Priority 2: NHC will document Tailpiece section static pressures and discharges for both the ogee shaped design and a simplified triangular shaped design. The group decided that as long as the District policy would be that the tailpiece section would not be operated alone, measurement of dynamic pressures on the tailpiece section would not be necessary. The static pressure and discharge data shall be obtained for both tailpiece shapes at four (4) gated conditions at pool elevation 268 ft and one free flow (ungated) condition at pool elevation 276 ft. The gated openings to be tested shall be selected by NHC to essentially cover the gate opening range up to openings resulting in minimum static pressures somewhat lower than about minus 15 ft of water. Static pressures lower than about minus 15 ft (COE EM 1110-2-1603) are considered unacceptable with respect to cavitation. The free flow discharge capacity of the existing spillway will be measured at, or near, pool elevation 276 ft. The SOW called for constructing, installing and documenting only one tailpiece shape as follows:

- Develop a gate-controlled rating based on 32 combinations of gate opening and pool elevation.
- Measure pressures at up to 20 locations on the tailpiece.
- Measure free flow discharge at two pool elevations
- Measure free flow discharges for the existing spillway at two pool elevations to document reduction in maximum bay discharge with the tailpiece in place.

NHC concluded that, because of the decreased testing program (10 combinations of gate opening vs 32 combinations), construction, installation and testing of the two shapes could be accomplished with no increase to the contract cost. <u>The documentation data will be furnished to Dennis Dorratcague by March 16.</u>

c. Priority 3: NHC will document hydraulic performance of three deflector designs. Documentation data for each design shall consist of hydraulic performance curves at pool elevation 257, 262.5, and 264 ft and at a sufficient range of tailwater elevations to define the transition between plunging/skimming regime, skimming/undular regime and undular/hydraulic jump regime. Side and overhead video with a short description of flow observations for each condition will also be furnished. The three designs to be documented are: (1) 50 radius with 30 ft length, (2) 50 ft radius with 50 ft length and (3) 20 ft radius with 30 ft length. All three designs will be at elevation 148 ft. A cross sectional water surface near the downstream end of the deflector will be measured (8-10 locations to capture the major undulations in the water surface) for each deflector design. <u>Documentation data will be furnished to Chris Goodell by March 26.</u>

6. The next formal model visitation is scheduled for the week of 2-6 April. The purpose of this visit will be to demonstrate the RSW with the final design deflector to the COE staff and the agencies. The skeleton bay surface collector model will also be operational in the McNary flume for a side-by-side comparison of the RSW and the skeleton bay at comparable model scales.

James L. Lencioni, P.E.

Cc:
Chris Goodell, CENWP
Blaine Ebberts, CENWP
Al Babb, nhc
Ken Christison, nhc
Dick Regan, nhc
Ed Zapel, nhc
Dennis Dorratcague, Montgomery Watson

Table 1. John Day RSW Deflector, Summary of Hydraulic Performance Testing, Feb 26 through March 2 2001 Lab Visit

Flow Deflector Geometry Transition Radius		Deflector Length		Tailwater		Flow Condition
(m)	(ft)	(m)	(ft)	(m)	(ft)	
				48.2	158	Plunging/Skimming
15.2	50	9.1	30	48.8	160	Skimming/Undular
				54.3	178	Undular/Hydraulic Jump
				48.5	159	Plunging/Skimming
6.1	20	9.1	30	49.4	162	Skimming/Undular
				53.6	176	Undular/Hydraulic Jump
				48.5	159	Plunging/Skimming
6.1	20	7.6	25	49.4	162	Skimming/Undular
				53.9	177	Undular/Hydraulic Jump
				48.5	159	Plunging/Skimming
10.7	35	7.6	25	49.4	162	Skimming/Undular
				54.3	178	Undular/Hydraulic Jump
				48.5	159	Plunging/Skimming
10.7	35	10.7	35	49.4	162	Skimming/Undular
				53.9	177	Undular/Hydraulic Jump
				48.2	158	Plunging/Skimming
15.2	50	9.1	30	49.7	163	Skimming/Undular
				54.3	178	Undular/Hydraulic Jump
				48.2	158	Plunging/Skimming
15.2	50	15.2	50	49.4	162	Skimming/Undular
				53.9	177	Undular/Hydraulic Jump

Note: Forebay water surface elevation remain constant at 80.5 m (264 ft)

Table 2. RSW Tailpiece Section, Preliminary Pressure Data, Feb 27, 2001

Piez. No.	Piez. El.	Pressure (ft) G.O. 5'	G.O. 6'	G.O. 8'	G.O. 10'	G.O. 15'
		10,300 cfs	12,300 cfs	16,200 cfs	20,100 cfs	29,200 cfs
1C						
2C	218.4	+18.1	+8.1	-3.9	-23.9	-50.0
3C	218.5	+10.0	-1.0	-10.0	-25.0	-43.0
4C	218.2	-5.7	-5.7	-5.7	-7.7	-15.7
5C	216.7	-4.2	-3.2	-3.2	-4.2	-5.2
7C	210.6		-1.1	-0.1	-0.1	
8C	202.8		+11.7	+6.7	+11.7	+7.7
1P	218.4	+19.1	+9.9	-2.9	-3.9	-13.9
2P	218.5	+12.0	+10.0	-4.0	-11.0	-26.0
5P	218.2	-2.7	-2.7		-2.7	-3.7

Notes:

1) 'C' located on crest c.l.
2) 'P' located 1' from rt pier
3) '1C' in u/s face
4) '3C' & '2P' at crest apex
5) Pressures less than –32' will not exist in prototype
6) Static pressure < - 15' not acceptable
7) Discharge based on 'C' = 0.75

Piez No.	Distance (ft) from Crest	
2	-1	(U/S)
3	0	On Apex
4	1.5	(D/S)
5	4.2	(D/S)
7	12.5	(D/S)
8	23.2	(D/S)

Table 3. Preliminary Pressures, Triangular Tailpiece Section

Piez. No.	Piez. El.	Pressure (ft) G.O. 2.2' 4,500 cfs	G.O. 4.3' 8,700 cfs	G.O. 6.6' 13,100 cfs	G.O. 9' 17,600 cfs	G.O. 15' 28,300 cfs
4C	218.2	+2.8	+1.3	+1.8	+1.8	-9.2
5C	216.7	-3.7	-2.7	-2.7	-1.7	-0.7
7C	210.6			-2.6		+0.4
8C	202.8			+5.2		+6.8

2-6 APRIL 2001 VISIT TO
NORTHWEST HYDRAULIC CONSULTANTS LABORATORY

April 10, 2001

MEMORANDUM FOR: RECORD

SUBJECT: John Day Dam Deflector Model Study, 2-6 Apr 01 Trip Report

1. The 1:25 scale John Day spillway deflector and Skeleton Bay (SB) surface collector models were demonstrated for staff from the Corps of Engineers (COE) and resource agencies during the week of April 2-6. The resource agency staff were present only on April 4 and 5. Participants included:

Chris Goodell (Portland District Hydraulic Engineer)
Blaine Ebberts (Portland District Biologist)
Mike Schneider (WES Hydraulic Engineer)
Don Wilson (WES Hydraulic Engineer)
Dave Maggio (WES Hydraulic Technician)
Matt Hanson-part time (Portland District Structural Engineer & PM
Steve Rainey (NMFS Hydraulic Engineer)
Gary Fredricks (NMFS Biologist)
Tom Lorz (Col River Inter-Tribal Fisheries Council Hydraulic Engineer)
Dennis Dorratcague (Montgomery Watson)
Dick Regan part-time (NHC Hydraulic Engineer & Internal Technical Reviewer)
Ken Christison (NHC Hydraulic Engineer & Model PI)
Jim Lencioni (NHC Hydraulic Engineer and Project Manager)

2. Group discussions were held on the morning of April 2 and April 4 to brief the COE and agency staffs on the status of the testing to date. Hydraulic performance curves for three deflector designs had been completed by NHC and furnished to COE staff by letter dated March 28, 2001. The agency representatives were advised that a decision on the deflector design needed to be made by April 6 so that the deflector P&S could be completed in time to meet the end of February 2002 deflector installation date at John Day. Dick Regan cautioned that the longer deflectors (30 and 50 ft) could create potential problems with inadequate energy dissipation in the stilling basin during extremely large spillway releases and that issue needed to be addressed if a long deflector were selected.

3. The spillway deflector designs demonstrated were the 30 ft and 50 ft long deflectors at elevation 148 ft with a 50-ft transition radius between the spillway face and the deflector. Performance was demonstrated at pool elevations 262.5 ft and 264 ft (spillbay discharges of approximately 13,800 cfs and 15,500 cfs, respectively) and tailwater elevations 157.5 ft, 159 ft, 162 ft and 165 ft. The upper and lower tailwater elevations demonstrated reflect the range of tailwater elevations occurring 90% of the time during the fish migration time period of 1 March through 30 November. In addition to verification of the deflector hydraulic performance curves previously developed by NHC, dye retention time in the stilling basin was documented during the visit (Tables 1 and 2). General hydraulic characteristics in the stilling basin and tailrace downstream from the basin were

observed for all conditions. The apex of the jet trajectory with the 30 ft deflector was located near the end of the pier and powerhouse deck and resulted in significant amounts of water impacting on the top of deck and pier at the higher range of tailwater elevations. A primary advantage with the 50 ft length deflector was that the apex of the jet trajectory exiting from the deflector moved downstream from the end of the training wall and powerhouse deck and significantly reduced the water impact on those structures. Small amounts of gravel placed in the stilling basin migrated upstream towards the deflector face where the gravel remained and continually moved in a counter-clockwise direction. This indicates that stilling basin floor erosion may be a concern if material is drawn into the basin from downstream. On April 6, Mike Schnieder video taped hydraulic performance of the 50 ft long deflector at pool elevations of 251.5 ft, 256.5 ft and 260 ft (spillbay discharges of approximately 4,500 cfs, 6,400 cfs and 8,500 cfs respectively).

4. The SB surface collector with a downstream chute invert elevation of 157 ft was demonstrated with a pool elevation of 264 ft and with the same tailwater elevations as used in the deflector demonstration. A tailwater elevation of 174 ft was also demonstrated to simulate the same submergence (17 ft) as exists on the elevation 148 ft deflector with tailwater elevation 165 ft. The SB exhibited more of a skimming flow regime than does the RSW with spillway deflector at comparable tailwater elevations. With a 17 ft submergence on the SB, a hydraulic jump forms whereas about 26 ft of submergence is required to form a hydraulic jump with the elevation 148 ft deflector. The difference in downstream hydraulic performance of the SB and the RSW/deflector is likely the result of (1) greater energy loss on the SB chute than the RSW/spillway, (2) the SB exit invert is 9 ft higher than the RSW deflector, (3) exit jet expansion is restricted by the training wall with RSW but not with the SB and (4) the flow distribution entering the tailwater with the three-chute SB is more uniform than from the single bay RSW.

5. On April 5, the group met to discuss and decide upon the deflector geometry to be carried into the final design for P&S. The group agreed that the final design geometry for the P&S would be the 50-ft transition radius, 50-ft long deflector at elevation 148 ft. However, the agency representatives stated that the late April through mid-July time period was the most critical time period to be considered in design of the deflector and Steve Rainey still wants to observe performance of the design in the 1:80 scale general model at WES. Therefore, the length and elevation are contingent upon the results of testing of the deflector at PMF flow conditions (to be conducted at NHC in the 1:25 scale model), review of the 90% occurrence tailwater elevations during the April through June time period and observations in the 1:80 scale model at WES. Matt Hanson suggested a potential design modification at the downstream end of the deflector (cantilevering the end of the deflector) to reduce the volume of concrete in the deflector. Pressures on various locations of the cantilevered section will be measured during the high flow testing program to determine design loads and identify whether any potential low pressure areas exist with such a design.

6. The COE will prepare a contract modification to evaluate the long deflector(s) under PMF conditions in the 1:25 scale model at NHC. This model work will be conducted in the old McNary flume to accommodate the discharge required for PMF conditions (112,500 cfs per bay). Chris Goodell will furnish NHC with a SOW for this effort by April 13. I suggested that one option to accomplishing this work would be under an existing option in the present deflector contract if costs were comparable. The following prioritization of outstanding work under the present contract was made:

a. Priority 1: Complete documentation of the SB work per Task 4 of the contract. Additional documentation not included in the SOW will include development of hydraulic performance curves similar to those accomplished for the deflector, development of chute cross sectional water surfaces at various appropriate locations in the chute(s) to capture flow anomalies and measurement of the vertical velocity distribution at the entrance to the SB center chute bay centerline.

b. Priority 2: Complete testing of the 50-ft long deflector at PMF flows to evaluate energy dissipation performance. The scope of this effort will be developed by the COE. Preliminary thoughts are to evaluate various (say up to 5) lengths between 30 and 100 ft using visual observation, velocity measurements near the end sill and gravel movement downstream of the end sill.

c. Priority 3: Complete documentation of the final design deflector agreed to on April 5 (50-ft radius, 50-ft length, elevation 148 ft) per the contract Task 3b.

d. Priority 4: Complete documentation of the RSW per the original contract.

Chris Goodell will also advise NHC whether our (NHC) participation at the upcoming WES trip (early to mid-May) is desired. NHC has not yet been funded for this activity, therefore a contract option will need to be exercised if our participation is desired.

James L. Lencioni, P.E.
NHC
Sr. Hydraulic Engineer

cc:
Chris Goodell
Dick Regan

Table 1. Dye Retention Time in Stilling Basin, 30-ft Long Deflector

	Dye Injection Location	TW 157.5	TW 159	TW 162	TW 165
PE 262.5	Surface of Jet	1.6 seconds	1.8 seconds	1.5 seconds	1.0 seconds
PE 262.5	Bottom of Jet	4.0 seconds	2.5 seconds	3.2 seconds	3.2 seconds
PE 264	Surface of Jet	3.4 seconds	2.0 seconds	1.2 seconds	1.7 seconds
PE 264	Bottom of Jet	6.1 seconds	4.1 seconds	2.4 seconds	2.3 seconds

Table 2. Dye Retention Time in Stilling Basin, 50-ft Long Deflector

	Dye Injection Location	TW 157.5	TW 159	TW 162	TW 165
PE 262.5	Surface of Jet	1.2 seconds	1.1 seconds	1.4 seconds	1.2 seconds
PE 262.5	Bottom of Jet	1.6 seconds	1.6 seconds	1.9 seconds	1.9 seconds
PE 264	Surface of Jet	1.2 seconds	1.2 seconds	1.2 seconds	1.0 seconds
PE 264	Bottom of Jet	1.9 seconds	1.8 seconds	1.8 seconds	1.5 seconds

14-18 MAY 2001 VISIT TO
USACE ERDC LABORATORY

MEMORANDUM FOR RECORD

FROM: Kyle McCune

SUBJECT: John Day RSW - WES Trip Report for 14-18 May 2001

DATE: 21 May 2001

Attendees: Kyle McCune, NWP
Chris Goodell, NWP
Blaine Ebberts, NWP
Dave Maggio, ERDC
Don Wilson, ERDC
Gary Fredericks, NMFS
Steve Rainey, NMFS
Miroslaw Zyndol, NWP
Mike Schneider, WES
Tom Lorz, CRITFC
Ken Christinson, NHC

Trip Objectives:

1. Verify as necessary the John Day Dam 2001 interim spill pattern.
2. Model three deflector (standard type) elevations (el. 145, el. 148, and el. 152.5) for Bay 1 and compare the differences in performances, effects at the adult fish entrance, and in egress. Use the 1998 spill pattern (no restriction on Bay 1 discharge). Determine whether Bay 1 should be included in the spill pattern for and interim or final spill pattern.
3. Model RSW and extended deflector in Bay 20. Examine deflector (50 ft radius, 50 ft length) elevations of 150 and 153. Check spill patterns and required training spill (i.e., bay spill used to help direct RSW flow and improve egress conditions).

Observations and Discussions:

The primary points of discussion during this trip revolved around; the effectiveness of Bay 1 spill, the proposed Bay 1 deflector elevations, and juvenile egress conditions with the current interim spill pattern, the Removable Spillway Weir (RSW) and the Skeleton Bay Surface Bypass (SBSB). Observations and discussion of each of these issues are summarized below.

In prior model work Chris Goodell and Blaine Ebberts noted long dye retention times in Bay 1 when it was allowed to pass spill. The dye is being held up in an eddy similar to those created at the other spill bays. Retention times in Bay 1 are longer because lateral flow and entrainment are restricted because it is bounded by the spillway retaining wall to the south and the adult fish ladder to the north. This condition is undesirable for fish passage and the agencies agree that Bay 1 shouldn't be operated except for forced spill.

The near field performance of the Bay 1 deflector at el. 148 ft and el. 152.5 ft was hard to determine in the 1:80 scale model. The eddy condition in Bay 1, created by the training wall to the South and the adult ladder to the North, also showed little dependence on the elevation of the deflector used. The higher deflector elevation may have contributed to a slightly quicker evacuation of dye but not enough to readily quantify. On the other hand, the deeper deflector is expected to have greater effect in reducing total dissolved gas.

As with the Bay 1 deflector, the near field performance of the 50 ft Bay 20 deflector at 150 ft elevation versus 153 ft elevation was hard to differentiate. Without a visible difference in performance between the two alternatives it was determined that the 150 ft elevation deflector would provide better dissolved gas benefits and should therefore be used.

A majority of discussion during this trip was over the ability to generate good juvenile egress conditions at John Day. Initial concern over conditions created by the interim spill pattern was raised during the first day of testing. The agencies voiced concern over the tendency of the bulk of the spill to pass along shallow areas along the north shore. The current interim spill pattern is bulked on the north shore intentionally in order to create a faster flow past the north shore and move juveniles past that area quicker. The agencies propose that the interim spill pattern be re-evaluated before the 2002 juvenile passage season. Concern was also raised over juvenile egress with the RSW and the SBSB. The only spill patterns that showed satisfactory egress for the agencies was when 30% of the spill was passed through the spillway as training flow. This does not meet the project objective of passing more fish with less spill and is therefore unacceptable to the agencies. A number of trials were run with 20% spillway flow as training flow for the RSW or SBSB but all trials created some local conditions that were undesirable for juvenile egress either in the RSW or SBSB or the training spill. Specifically if the training flow was bulked to the south adjacent to the RSW/ SBSB the RSW/ SBSB egress was good but a high percent of the training flow (also assumed to contain fish) would eddy back toward the dam. If a uniform flow pattern was used, the RSW/ SBSB egress was good but flows from the middle spill bays would clear the stilling basin and then slow down significantly (nearly stalling) before continuing down the river. The current position of the agencies is that the RSW/ SBSB will not be able to provide acceptable egress conditions and therefore the value of the project should be re-evaluated. Members of the agencies will be returning to WES the week of June 11 and would like to revisit the subject at that time before making a final decision.

Recommendations and Action Items:

In light of conditions observed during this trip there are a number of suggested actions to be taken. Primary of these is to determine whether more acceptable juvenile egress conditions can be generated at John Day for either a general spill pattern or spill patterns that include the RSW or SBSB. The current interim spill pattern is acceptable for 2001 except that Bay 1 should not spill except during forced flow. A new general spill pattern should be created before the 2002 juvenile passage season. It would be preferable to create a "best case" general spill pattern (intended 2002 pattern) for John Day as soon as possible so that spill patterns that include RSW or SBSB spill can be created and evaluated against this base case. Time is a limiting factor in this process and there may not be enough to fully come up with best case conditions and RSW/ SBSB conditions before the proposed June 11 trip.

Regardless of operating conditions at the project, attendees concluded that deflectors should be placed in Bays 1 and 20. The Bay 1 deflector should be 12.5 ft long and should be placed at el. 148 ft. Depending on whether we move forward with the RSW or SBSB project the deflector for Bay 20 should be 50 ft long and placed at el. 150 ft (with the RSW) or the same design as that proposed for Bay 1 (without the RSW).

13 JUNE 2001 VISIT TO
NORTHWEST HYDRAULIC CONSULTANTS LABORATORY

MEMORANDUM FOR RECORD 18 June 2001

SUBJECT: John Day Bay 20 Deflector, Model Lab Visit

1. Operation of the 1:25 scale extended 50-ft long spillway bay 20 deflector in the existing McNary spillway flume with an approximate PMF flow condition was observed at NHC's lab on 13 June by Messrs. Jim Lencioni and Dick Regan of NHC and Mr. Dennis Dorratcague of Montgomery Watson. Messrs. Al Babb and Ken Christison of the lab staff were also in attendance.

2. The high flow testing of the extended deflector is being accomplished in the existing flume that was originally constructed for the McNary Dam spillway deflector tests to take advantage of its existing large capacity pumping system. The McNary flume is being used in lieu of a costly modification that would be required to add sufficient pumping capacity to the John Day flume to simulate high flows. Due to model limitations resulting from retrofitting the John Day spillway section into the flume that was originally designed for the McNary spillway, the John Day PMF hydraulic parameters and tailwater channel geometry can not be exactly simulated.

- The simultaneous highest pool and lowest tailwater condition that can be achieved in the model is about 273 ft and 210 ft, respectively (as opposed to the actual PMF design condition of a 276-ft pool with a 205-ft tailwater). This difference is not considered to significantly affect the conclusions of the model effort.

- The floor of the model flume downstream from the stilling basin simulates an elevation of about 90 ft. The tailrace invert downstream from bay 20 in the prototype is about elevation 100-110 ft for a distance approximately 300 ft downstream from the basin end sill. The tailrace invert modeled in the original 1:41.14-scale sectional model used in design of the spillway and stilling basin at the Corp's Bonneville laboratory (Technical Report 97-1, November 1974) began at the end sill (elevation 127-ft) and then sloped up to elevation 145-ft on a 1:6 slope. Therefore, the hydraulic conditions downstream from the stilling basin in the NHC model can not be directly compared to the hydraulic conditions observed in the original Bonneville laboratory 1:41.14-scale model study and report. However, the NHC model tailrace more closely simulates the prototype tailrace topography condition than did the Bonneville laboratory model.

3. The model was initially operated with "artificial" transparent walls along the powerhouse and stilling basin. The purpose of these walls was to improve viewing of flow conditions in the basin. However, these walls eliminated the energy dissipation that would be achieved as a result of flow entrainment that would exist from the powerhouse tailwater (south of bay 20) and the remainder of the spillway tailwater (north of bay 20). With this model setup, the jet exiting from the deflector impacted downstream of the end sill. We believe that elimination of the entrainment flow resulting from the transparent walls causes an unrealistic flow condition, therefore the transparent walls were removed from the model.

4. With the walls removed, visual observation of the jet impact in the stilling basin was made extremely difficult due to the turbulence and bubbles that existed. However, the overall energy dissipation in the stilling basin was significantly improved over that which occurred when the walls were in the model. Because of the difference in the invert elevation of the tailwater channel in this NHC model and the original Bonneville laboratory model, a direct comparison of flow conditions with photographs from the Bonneville laboratory model report could not be made. The criteria for satisfactory stilling basin performance stated in paragraph 3(d) of the Bonneville laboratory model report was that the stilling basin needs to provide the "minimum energy dissipation consistent with safety of the dam for the design flood of 2,250,000 cfs through 20 bays". The initial impression is that the energy dissipation in the stilling basin and the downstream channel with the 50-ft long deflector probably meets that criterion. However, the degree of entrainment flow into the deflector bay 20 from the spillway to the north of bay 20 (i.e., the 3-dimensional aspects not simulated in the 2-D sectional model) needs to be better simulated in the model. The potential affects of the NHC models' lower than actual tailwater channel invert on flow conditions downstream from the end sill also needs some further consideration. Energy dissipation at lower discharges will also need to be reviewed in the model. Therefore, prior to drawing a final conclusion, the following actions will be accomplished in the NHC 1:25 scale sectional model:

- The 50-ft deflector will be removed from the model and the base condition (i.e., no deflector) at the PMF and 3-4 lower flows will be documented in the 1:25 scale model both within, and downstream from, the stilling basin to serve as a direct comparison with deflector added conditions.
- The profile of the hydraulic jump with the base conditions will be used to approximate the area where the stilling basin tailwater north of bay 20 overtops the training wall into bay 20. The model will be modified to approximate this overtopping condition for subsequent deflector installed condition tests.
- The 50-ft deflector will be re-installed in the model and its performance will be documented and compared to the base condition per the items of documentation requested by the SOW (location of toe of jump, gravel movement downstream from the end sill, and velocity measurements near the endsill).
- The remaining deflector lengths as stated in the SOW will be tested, documented per the SOW and compared to the base condition.

In addition to these actions in the sectional model, we strongly recommend that the energy dissipation performance of the extended 50-ft long deflector under high flow conditions be reviewed in the 3-dimensional 1:80 scale general model at ERDC.

5. Digital photographs were taken of the model in operation with the 50-ft deflector installed. These photographs will be electronically transferred to the Kyle McCune in the Portland District.

MEMORANDUM FOR RECORD

FROM: Kyle McCune

SUBJECT: John Day Deflector Study – Northwest Hydraulic Consultants (NHC) Trip Report for 13 June 2001

DATE: 25 June 2001

Attendees: Kyle McCune, NWP
Jim Lencioni, NHC
Al Babb, NHC
Ken Christison, NHC

Trip Objectives:

View the effectiveness of stilling basin under a Probable Maximum Flood (PMF) event given the use of a 50 ft deflector in Bay 20.

Observations and Discussions:

The Purpose of the high flow test is to determine whether, under the an extreme event, an extended deflector (50 ft) would cause the jet from the spill bay to overshoot the stilling basin and therefore create the potential for unacceptable erosion downstream of the project. The high flow testing required a higher capacity pumping system so the original John Day Deflector model was moved into a flume originally used to model the McNary spillway. Because the flume used to test the PMF was not designed for John Day originally the PMF hydraulic parameters and tailwater channel geometry could not be exactly simulated. The limitations of the McNary flume on John Day modeling were documented by Mr. Jim Lencioni in his 18 June 2001 trip report for the model visit and are as follows:

- The simultaneous highest pool and lowest tailwater condition that can be achieved in the model is about 273 ft and 210 ft, respectively (as opposed to the actual PMF design condition of a 276-ft pool with a 205-ft tailwater). This difference is not considered to significantly affect the conclusions of the model effort.

- The floor of the model flume downstream from the stilling basin simulates an elevation of about 90 ft. The tailrace invert downstream from bay 20 in the prototype is about elevation 100-110 ft for a distance approximately 300 ft downstream from the basin end sill. The tailrace invert modeled in the original 1:41.14-scale sectional model used in design of the spillway and stilling basin at the Corp's Bonneville laboratory (Technical Report 97-1, November 1974) began at the end sill (elevation 127-ft) and then sloped up to elevation 145-ft on a 1:6 slope. Therefore, the hydraulic conditions downstream from the stilling basin in the NHC model can not be directly compared to the hydraulic conditions observed in the original Bonneville laboratory 1:41.14-scale model study and report. However, the NHC model tailrace more closely simulates the prototype tailrace topography condition than did the Bonneville laboratory model.

In addition to these limitations, it should be noted that during this simulation the adjacent half bay (representing Bay 19) was not operated. Due to the high tailwater elevation during the PMF, in the model water backed up in Bay 19 and spilled over the retaining wall into the Bay 20 jet. The spillage from Bay 19 in the model likely acted to reduce the momentum of the Bay 20 jet. This condition would not exist in the prototype since, in the case of the PMF, Bay 19 would be required to operate at the same capacity as Bay 20. Instead of reducing the momentum of the Bay 20 jet as in the model Bay 19 would more realistically add to the momentum in the system and create a worse condition than witnessed in the model.

Understanding the potential limitations of the model, the model was initially operated to simulate, as closely as possible, the conditions of the PMF. This includes uncontrolled flow of 112,000 cubic feet per second (cfs) (prototype) through Bay 20. In the model the jet off the deflector did impinge on the end sill of the stilling basin and was consequently directed up toward the surface. There was still quite a bit of energy apparent in the jet as evidenced by significant turbulence continuing downstream of the stilling basin. The turbulence didn't appear to penetrate to depth though. The only point of concern would be the return eddy that is created downstream of the stilling basin. It was enough to scatter gravel material placed just downstream of the model stilling basin, which would suggest active scour at the prototype under similar conditions.

After viewing the PMF a number of additional flow conditions were viewed in the model. These conditions consisted progressively less flow and lower tailwaters and created eddies similar, though not as strong, as that of the PMF. The other flow conditions viewed on this model trip include 83,000 cfs and 53,000 cfs (prototype).

Recommendations and Action Items:

Though the subject requires further investigation, primarily to more accurately simulate the project conditions, it appears that the end sill of the stilling basin still acts to direct the flow momentum up toward the surface when using the 50 ft deflector. In addition, though there is a significant amount of energy being dissipated outside of the stilling basin, it does not appear to penetrate to depth. This suggests that operation of a 50 ft deflector would be acceptable at John Day. As stated above, further investigation is required to determine whether lower flow conditions and subsequent low tailwater elevations may prove a larger concern than the PMF. If this is the case the next step will be to devise an operating plan that excludes Bay 20 with the 50 ft deflector under those conditions.

NHC is still testing various deflectors to determine at what length a deflector would cause the spillway jet to overshoot the stilling basin all together. The final results of this study will be documented in a report to be released in draft form by October 1, 2001 and again final by December 1, 2001.

JAMES L. LENCIONI, P.E.
Sr. Hydraulic Engineer

cc:
Dick Regan
Al Babb / Ken Christison
Dennis Dorratcague

11-12 OCTOBER 2001 VISIT TO
NORTHWEST HYDRAULIC CONSULTANTS LABORATORY

MEMORANDUM FOR RECORD

FROM: Kyle McCune

SUBJECT: John Day RSW- Trip report for 11- 12 October 2001

DATE: 23 October 2001

Attendees: Kyle McCune, NWP
Bob Buchholz, NWP
Dick Regan, NHC
Jim Lencioni, NHC
Al Babb, NHC
Ken Christison, NHC

Trip Objectives:

1. View 50 ft deflector performance under probable maximum flood (PMF) conditions.
2. View John Day RSW model prior to its removal.

Observations and Discussions:

In viewing the PMF conditions it was apparent from upwelling over the end sill that a significant amount of energy dissipation is occurring in the stilling basin. It is also apparent that energy was carried outside the stilling basin to be dissipated beyond the end sill. This suggests that the introduction of the 50-ft deflector may have caused the stilling basin to lose some of its effectiveness at the PMF flow. Though the effectiveness of the stilling basin is compromised by the addition of the deflector there appears to be no adverse affects transferred to the riverbed downstream of the stilling basin because the energy dissipation appears to be limited to the upper portions of the water column and does not penetrate to a depth that would cause concern about erosion of bed material. At bed level outside the stilling basin a minimum amount of disturbance was apparent by the movement of gravel placed adjacent to the stilling basin. The gravel was swirled but was not moved out of the area by currents returning through the bottom portion of the water column as flow was entrained in the jet above. The lack of penetration to depth of the turbulent energy dissipation is probably due to the relatively high tailwater present during the PMF condition that allows for a buffer between the jet and the bed downstream of the stilling basin.

For further comparison the PMF passage through Bay 20 was also viewed without the 50-ft deflector with similar results. The primary difference between the two scenarios was an increased upwelling of the end sill when the deflector wasn't present. This upwelling appeared to be about 20 ft prototype (peak to trough) and is a result of the jet being forced upwards after plunging over the undeflected spillway. Without the deflector the jet still failed to be dissipated within the stilling basin but once again turbulence remained at the top of the water column.

One limitation noted in the sectional model was that it failed to simulate lateral flow conditions at the dam. Specifically, due to the limited pump capacity only Bay 20 could be run and the adjacent partial Bay 19 was left closed. This in combination with the high tailwater caused a return flow in front of bay 19 that spilled over into the Bay 20 jet and most likely helped dissipate the jets energy sooner. This most likely is artificially decreasing the energy dissipation outside of the stilling basin. In the actual case of a PMF the additional flow from Bay 19 could possibly increase the momentum of the Bay 20 jet over that which we saw in the sectional model. Whether including Bay 19 flow would create a significant increase in turbulent energy dissipation penetrating to depth outside the stilling basin is questionable.

Conclusions and Recommendations:

As stated above, though the 50-ft deflector does appear to reduce the effectiveness of the stilling basin during PMF conditions, the higher tailwater elevations that would likely be present during those same conditions allows for a buffer between the turbulent energy dissipation beyond the end sill and the river bed adjacent to the stilling basin. Therefore, qualitatively, the extended deflector does not appear to be a cause for concern at John Day at the flows witnessed during this model visit.

To further evaluate the 50 ft conditions attendees suggested two things be done and included in the final report. The first would be to collect velocity measurements starting a distance outside the stilling basin and progressing back toward the stilling basin until air makes measurement impractical. These measurements would be conducted during the PMF with and without the deflector and the results compared so that a more quantitative determination of the deflectors affect could be made. The second recommendation was to simulate, as closely as possible, the PMF at the 1:80 scale general model at WES so that the affect of additional spill through adjacent bays can be at least qualitatively determined.

ITR REVIEW COMMENTS

PROJECT: John Day Dam RSW and SB 20 Deflector – Physical Model LOCATION: nhc

☐ CENPP-PE-DS ☐ Air Force ☐ Army ☐	Date: 26 Oct 01 Reviewer: **A. G. Mercer** Phone: (604) 980 6011	Draft Final Report ☐ D.Memo ☐ Concept ☐ P & S ☐ Prelim. ☐ ☐ Final	☐ Arch./LA ☐ Civ./San. ☐ Mech./Elec. ☐ Struct.	Action taken on comment by: REVIEW CONFERENCE: A - Comment Accepted; W - Comment Withdrawn (If neither, explain) DESIGN OFFICE: C - Correction made (If not, explain) Back Check by: (INITIALS)

Item No.	Drawing Sht. Spec. Para.	COMMENTS	REVIEW CONFERENCE	DESIGN OFFICE	Back Check by: (INITIALS)
		Comments on Draft Final Report *John Day Dam Removable Spillway and Spillway Bay 20 Deflector Hydraulic Model Study, Oct 2001*			
1	Section 1.3	This paragraph is unclear as to whether this is the originally proposed RSW or the final proposal. Perhaps it refers to elements common to all alternatives tested. In any case a drawing is needed. Especially unclear is that the piers need to be modified as well as the crest. Are the extended piers also removable? It is recommended that Section 1.3 and Section 1.5 be merged into a larger Section 1.3.	A	The paragraph has been re-written to clarify. Section 1.5 has been incorporated into Section 1.3.	AM
2	Section 1.3	There needs to be more explanation of the 3" offset. It is not shown in any drawings. Does it affect the flow?	A	Detail has been added to Figure 1-6. Does not affect flow characteristics down spillway, added sentence to Section 6.1.	AM
3	Section 1.4.1	Reference is made to a 13' wide center pier but Fig 1-4 seems to show 13' end piers. What is beyond or outside of these piers?	A	Section 1.4.1 and Figure 1-8 revised to clarify.	AM
4	Section 1.5	It should be mentioned in this section that the piers need to be extended upstream to accommodate the RSW's and that they, presumably, would not be removable.	A	Test revised to clarify required pier extension and that piers are removable.	AM
5	Section 3.3 para 2	Is there actually air entrained on the prototype face as stated and, if so, why is it not reproduced in the model?	A	Air is typically entrained on face of spillways. Can not accurately reproduce in model due to air scaling limitations and inability to simulate prototype surface roughness in model. Wording added in text.	AM
6	Section 5.1 Alt 2, para 1	The standing waves would be more understandable if called 'longitudinal' standing waves.	A	Revised as suggested.	AM
7	Appendix C referred to in Section 5.1	The first page of text in Appendix C is labeled Appendix E. This confusion should be clarified.	A	Appendix C in draft report will be removed from final report.	AM
8	Table 1 of Appendix C	This table is completely confusing and there is no text that leads the reader through it, either in the Appendix or in Section 5.1. For	A	Appendix C in draft report will be removed from final	AM

	referred to in Section 5.1	instance, there is no real definition of 'egress'. What comprises 'egress'? The important concepts and observations from Table 1 for RSW 2 and 7 should be included in separate paragraphs of Section 5.1 and perhaps Appendix C could be excluded from the report. The figures and photos of Appendix C have been eliminated already.		report. The ERDC Laboratory Trip Report in Appendix B will be referenced.	
9	Section 5.2 Para 12, bottom of page 15	To avoid confusion, the three additional tests should be identified as P-1, P-and P-3 in the text and it should be noted that summary results are presented in Table 5.1 with detailed results in Tables 5.2, etc.	A	Text revised as suggested.	AM
10	Section 5.2 Para 13 and Fig 5-7	The tailwater level should be given for these tests and it should be stated whether the results are affected by tailwater level. Also, since the longest design tested is best it should be stated why longer designs were not tested.	A	Text revised to give tailwater elevations for tests and explain why deflectors lengths greater than 50 ft were not tested. Comparison of increased construction cost and incremental benefit based on 30 and 50 ft long deflector performance indicated longer deflector not justified.	AM
11	Section 5.3 Para 1	This paragraph should make it clear that the tests were without the main RSW in place, only the tailpiece section.	A	Clarification added to text.	AM
12	Section 5.3 Para 1	This paragraph is the first one that contains the word 'static' used in the sense of 'time averaged'. Static normally is used where there is no motion such as in 'hydrostatic'. Unless the present usage of 'static' can be found in the literature elsewhere it should not be used here with this meaning. The phrase ' time averaged' should be used instead. In the same way in later sections 'dynamic' is used in the sense of 'time variable' or 'fluctuating' where dynamic actually refers to conditions of motion. The term 'dynamic' should be replaced with a term like 'fluctuating' wherever it has this meaning. Considering the static and dynamic pressure taps on a pitot tube adds to the confusion in using these terms here.	A	Text revised to use suggested terminology.	AM
13	Section 5.3 Para 2 and Table 5.5	Constructing the tailpiece to withstand the required large loadings and still be removable will be a formidable feat. Fluctuating pressures should be taken with only the tailpiece section in place such as given for DT-3 in Table 6.5 because it is expected the fluctuations will be quite large and difficult to design for.	A	Agree. However, decision was made that conditions requiring operation of tailpiece section without the RSW main section in place was too remote to design for. Therefore, measurement of time variable pressures with only tailpiece in place was deleted from SOW.	AM

14	Section 6.2.1 Para 2 and Figure 6-12	Figure 6-11 was missing in the report copy reviewed. Figure 6-12, similar to Figure 6-11, requires considerable thought to understand and the vertical curves do not seem to fit the rest of the data. Section 6.2.1 is very brief and should be expanded to clearly explain these figures.	A	Curve plots and data points in Figure 6-12 have been revised to be consistent. Additional text added to Section 6.2.1 to add some clarification to figures.	AM
15	Section 6.2.1 Para 3	This paragraph refers to Table 6-8 where it should refer to Table 6-7.	Reference to Table 6.8 is correct.		AM
16	Table 6.8	Measured velocities in Table 6-8 do not seem to fit any expected pattern with respect to deflector length and should be examined for errors and/or explained in Section 6.2.2, Para 5.	A	Table 6.8 in reviewed draft has been replaced with Table 6.9 which compares tailrace velocities with the 50-ft long deflector to the existing (no deflector) condition.	AM

File: ITR-JDAYRWS

REVIEW COMMENTS

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO; CONCEPT; FINAL; PLANS & SPECS; PRELIMINARY; %	John Day Dam Removable Spill Way Weir and Spill Bay 20 Deflector	John Day Dam	11/15/01

REVIEWER			
CENWP-EC-HD; AIR FORCE; X ARMY	NAME Kyle McCune; PHONE NUMBER (503)808-4897	ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY	MECHANICAL; ELECTRICAL; STRUCTURAL; X HYDRAULIC

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
1.	Page 1 Paragraph 1 Line 3	Reference missing.	A	Reference added.	
2.	Page 4 Paragraph 4 Line 11	Reference missing.	A	Deleted title of reference as the documentation could not be located.	
3.	Page 8 Section 3.2	More explanation of the how the lateral flow representing entrained powerhouse flow was determined and controlled. Was it varied in the model when different flow conditions were set up (i.e., river Qs /operation set ups)? How was the lateral flow determined and reproduced? In general, is more detail possible.	A	Detailed discussion contained in the Trip Report dated Dec 11, 2000 that is in Appendix B. Model report text references the Trip Report on page 16, paragraph 2.	
4.	Page 11 Section 5.1 Paragraph 2 Line 6	The report states tests were conducted for various pool elevations ranging from 262.5 to 268. Why did we stop at 262.5ft when the minimum operating pool is 257ft?	A	Model performance showed that hydraulic characteristics improved significantly at pool elevations below about 260 ft. Therefore, detailed investigations were limited to the higher range of pool elevations. Clarification added to report.	
5.	Page 14 Section 5.2 Paragraph 1	The report states that "good performance is characterized by the ability to produce skimming flow". Yet we are designing to produce undular flow (for the RSW). Maybe we should make that statement past tense and explain why we decided to design for undular flow off the deflector. In general I think we need to present the thought process that brought us to designing for undular flow (maybe on page 15 after subsection d)	A	Discussion regarding design of deflector for the undular flow regime is included on page 15 in the 7th paragraph of Section 5.2.	
6.	Page 16 Paragraph 3 Last Sentence	Did we ever describe the "best overall hydraulic conditions"? This may be a good time to do that if we haven't.	A	Description added in 4th paragraph on page 16.	
7.	Page 16 Paragraph 4 Sentence 4	Why make the statement about the threshold submergence difference between the 148-ft and 150-ft deflector and then reference Table 5.1? I didn't see that Table 5.1 supports that statement and then you go on to say that it is not	A	Text in paragraph 2 on page 17 revised to clarify.	

REVIEW COMMENTS

DESIGN DOCUMENT TYPE	PROJECT	LOCATION	DATE
X DESIGN MEMO; CONCEPT; FINAL; PLANS & SPECS; PRELIMINARY; %	John Day Dam Removable Spill Way Weir and Spill Bay 20 Deflector	John Day Dam	11/15/01

REVIEWER			
CENWP-EC-HD; AIR FORCE; X ARMY	NAME Kyle McCune; PHONE NUMBER (503)808-4897	ARCHITECT; LAND ARCHITECT; CIVIL; SANITARY	MECHANICAL; ELECTRICAL; STRUCTURAL; X HYDRAULIC

ITEM NO.	DRAWING SHEET SPEC PARA	COMMENTS	REVIEW CONFERENCE (A = Comment accepted) (If not accepted explain)	DESIGN OFFICE (C = Correction made. List drawing or paragraph number where correction made) (If not corrected, explain)	BACK CHECK BY (Initials)
		necessarily a function of elevation difference in the next sentence.			
8.	Page 19 "Deflector Performance"	Again we are discussing skimming flow when I thought we are actually designing for an undular flow.	A	The text referred to is for the SBSB, not the RSW deflector. Added text on page 20, paragraph 2 of Section 5.4.	
9.	Page 21 Section 6.1	I think the table references get sort of mixed up in this section. For example under "Velocities Over RSW" you reference Table 6.4 (which is actually pressures). Consequently, Tables 6.5 and 6.6. referenced later in the same section might be incorrect too. I see no table with the RSW velocities that we were going to take.	A	Table numbering has been corrected. Table 6.4 does show measured velocities on RSW crest.	