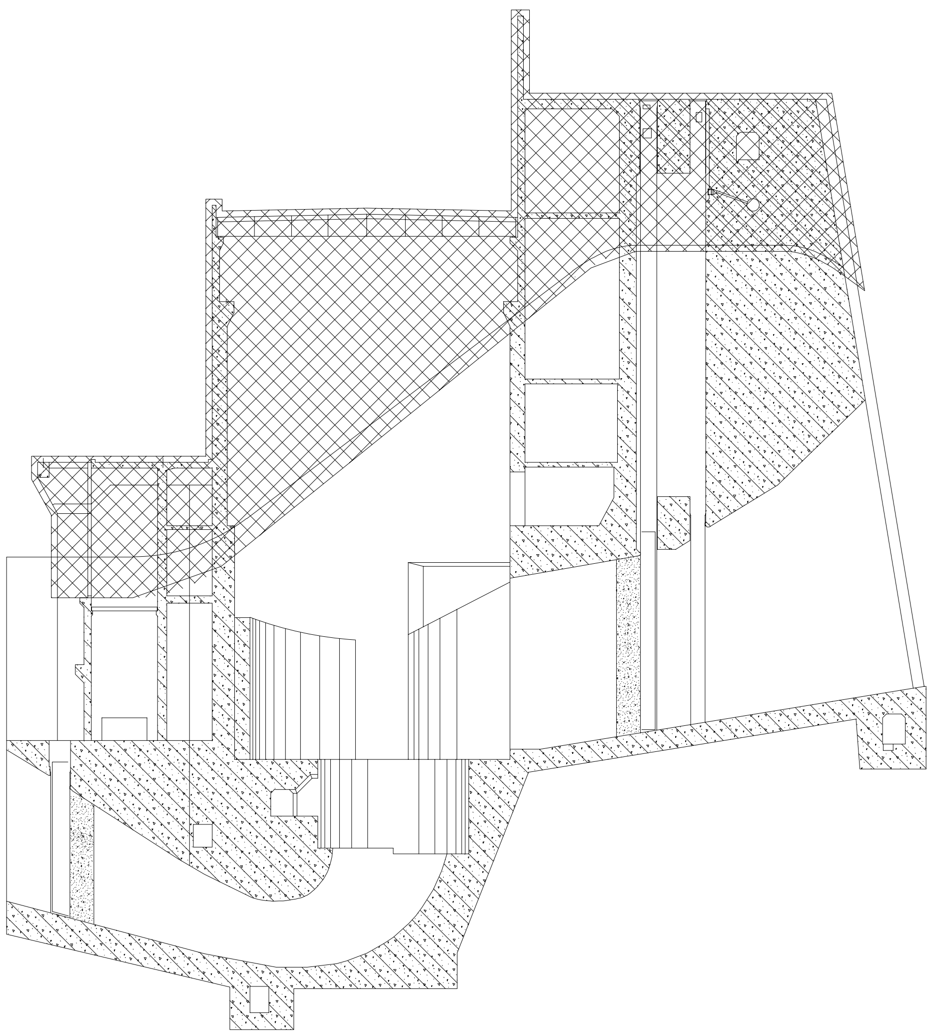
US Army Corps
of Engineers ®
Portland District

Feature Design Memorandum
No. 52

John Day Lock and Dam Surface Bypass Spillway



September 1998

JOHN DAY DESIGN MEMORANDA

Design Memorandum Number	Description	Date
	Project Bulletin # 1 - Wind Wave Investigation	1959
	Project Bulletin # 2 -Wind Wave Investigation	1967
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	Master Plan	1976
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1	Hydrology	1956
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4	First Step, Cofferdam	1958
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5	Supplement # 7 - County Road, Peterson to Plymouth	1965
5.1	Relocation of SP & S Railway, Towel to Rock Creek	1962
5.2	Relocation of Hiway & RR PSH #8 - Rock Creek to Four O' Clock Rapids	1961
5.3	Relocation Hiway & RR #8 - Roosevelt to Pine Creek	1962
5.5	Relocation of Hiway & RR Facility - Pine Creek to Carley	1963
5.6	Relocation, SP & S Rwy, Whitcomb to King	1965
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5.8	SP & S RR Relocation, Miller's Island to Cliffs	1962

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5.9	Relocation of SP & S Cliffs to Towel	1964
5.11	Relocation of SP & S RR, Sundale to Roosevelt	1963
5.12	Relocation of Portions of WA Hiway #8	1960
	Supplement # 1 - Four O'clock Rapids to Chapman	1961
	" # 2 - Towel to Rock Creek	1963
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5.13	SP & S RR Track Laying	1965
5.14	Relocation of SP & S RR, Miller's Island to King	1966
5.16	Instrumentation for SP & s RR & P.S. H	1965
6	North Shore Temporary Project Office and Visitor Facilities	1961
7	Volume I - Relocation OR Shore Earthwork Drainage Pvf.	1959
	Volume II - Relocation OR Highway	1959
	Supplement # 1 - Revision in Design & Cost Allocation	1961
	" # 2 - Earthwork Design Criteria	1962
	" # 3 - Relocation of Columbia Basin Electric	1964
	" # 4 - Relocation of Power & Telephone Facilities	1964
	" # 5 - Protection of County Roads, Rivers, Banks, etc.	1966
7.1	Relocation of Union Pacific Bridge	1959
7.2	Grading & Drainage for UPRR	1962
7.3	UPRR Shoofly & Hiway	1959
7.4	Location of Detour for US Hiway 30 and Railway	1959
7.5	Relocation - OR Shore Interstate Hiway 80 N bridge over John Day River	1961
7.6	Relocation - UPRR & Interstate 80 N, John Day River to Hood Section	1960
7.9	Interstate Highway 80 N - Arlington Viaduct	1963
7.13	Relocation of UPRR Bridge	1961
7.14	UPRR Grading & Drainage & I 80 N East	1962
7.15	UPRR Grading & Drainage US 30	1962
7.17	I 80 N Grading and drainage Between Blalock and Arlington	1962
7.18	UPRR Grading & Drainage - Arlington East to Willows Section	1964
7.19	UPRR Grading & Drainage - Blalock to Boat Ramp Access Road, Blalock to West Arlington	1965
7.20	Grading & Drainage - UPRR & I 80 N	1963
7.21	Temporary Relocation - UPRR & US 30, Blalock Area	1961
7.22	Grading & Drainage for UPRR & I 80 N Quinton to Blalock and Large Canyon Shoofly	1964
7.24	Construction I 80 N and Morrow County Roads	1964
7.25	Grading, Drainage, and Surfacing for UPRR - Castle rock &	

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	Boardman RR	
	Facilities	1964
7.26	Grading, Drainage, and Surfacing for UPRR - Heppner Brdg Facility, I 80 N, Or	1964
	Hiway # 74, Heppner Jet Area	
7.28	Instrumentation for UPRR & I 80 N	1966
8	Relocation of Pacific Telephone & Telegraph Fac.	1959
9	Concrete Aggregate Investigations	1959
	Supplement # 1	1961
10	Real Estate Part I	
	Dam Site Construction Area & North Shore Access Road	
11	Preliminary	1958
	Volume I - Design & Cost Estimates	1958
	Volume II - Real Estate, Relocation of Arlington, Or	1958
	Supplement # 1 - Relocation of Pacific Telephone & Telegraph Co. Fac. City of Arlington	1960
	" # 2 - Relocation Foundation Treatment	1960
	" # 3 - Relocation of Streets & Utilities	1960
	" # 4 - Relocation of Pacific Power & Light Co. Facilities	1961
11	Supplement # 5 - Supplement of Volume I	1962
	" # 6 - Construction of Storm Run -off Drainage System	1963
	" # 8 - Addition to Storm Run-off Systems	1965
	" # 9 - Public Parking	1966
12	Relocation of Boardman, OR	1959
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14	North shore Access Road	1958
15	Power Plant (Preliminary)	1961
15.1	Auxiliary Fishwater Supply - South Shore	1960
15.2	Powerhouse Station Service Power Supply	1961
15.3	Powerhouse Architectural Design	1962
	Supplement # 1 Roof Replacement	1994
15.4	Powerhouse Structure Design	1962
15.6	Powerhouse Air Conditioning	1962
15.7	Powerhouse Piping Design	1963
15.8	Powerhouse Mechanical Equipment	1963
	Supplement # 1 - PH Mechanical Equipment	1963

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	Supplement # 1 - PH Mechanical Equipment	1965
15.9 A	Powerhouse Lighting Design	1962
15.9 B	Powerhouse Grounding System	1963
15.9H	Supplement # 3 - control Switchboards, Sequential Recording Annunciators,and Teletype Communications Systems	1970
15.14	Relocation of SP & s Railway - Millers's Island	
16	Spillway, Navigation Lock, Right Abutment Embankment & Shore Fish Facilities	1959
	Supplement # 2 - Navigation Lock Model Studies	1960
	" # 3 - Navigation Lock Sill Blocks	1963
	" # 4 - Spillway Gantry Crane, Stoplogs and Related Appurtenances	1965
	" # 5 - Extension of Lock Guide Wall "D"	1965
	" # 6 - Temporary Unwatering Facilities for Navigation Lock Monolith Modification	
16	# 7 - Trans - Shipping Facilities	1965
	" # 8 - Modifications to Navigation Lock Gate Monoliths	1965
	" # 9 - Navigation Lock Floating Guide Wall "B"	1966
	" #10 - Downstream Nav Channel Excavation	1969
	" #11 - Upstream Nav Channel Excavation	1969
17	Exploratory Drillings & Grouting - Nav Lock	1969
18	South Non - Overflow Dam	1958
20	Visitor Facilities and Project Beautification	1960
20.1	South Shore Visitor Parking & Misc. Facilities	1973
	Supplement # 2	
20.2	Service Building	
21	Second - Step Cofferdam	1961
22	South Shore Permanent Fish Facilities	1963
	Supplement # 1 - Modification to Fishladder Flow Control	1970
23	Relocation of Boardman Public Schools	1960
23.1	Relocation of Boardman Public Schools	1965
24	Relocation of Arlington Elementary Schools	1959
25.1	South Shore Public Access Facilities	1967
	Supplement # 2 - Quensel Park	1969
	" # 3 - South Shore Public Access Fac.	1967
25.2	North Shore Public Access Facilities	1967
25 B	Master Plan	
	Master Plan (REVISED)	1976
	Appendix 1 - Cost Estimates for Development & Management of Lake Umatilla	1966
25 B	C-1 - Preimpoundment Tree Planting & Fencing - Public Use	1966

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	Areas	
	C-2 - Lepage Park	1966
26	Water Supply, Storage and Distribution	1960
28	Relocation of Municipally-owned Property, City of Boardman, OR	1963
29	Relocation of Municipally-owned Property, City of Arlington, OR	1960
30	Modifications to McNary fish Facilities	1962
31	Relocation of Roosevelt Elementary School	1962
34	Foundation Grouting and Drainage	1962
35	Navigation Lock Fire Protection	1961
36	North Shore Fishway Pumphouse Crane and Trashrack Cleaning Facilities	1961
38	Relocation of city of Umatilla	1962
38.1	Relocation of Municipally-owned Facilities, Umatilla, Or	1966
40	Detailed Plan for Relocation of Irrigon Cemetery in Morrow County, OR	1962
41	Detailed Plan for Relocation for Boardman Cemetery in Morrow County, OR	1963
42	Relocation of Field Office Facilities	1962
43	Cost Allocation Studies	1962
44	Reservoir Clearing	1967
46	Spring Creek National Fish Hatchery	1969
46.1	Bonneville Fish Hatchery	1971
	Supplement #1	1972
46.1	Emergency Incubation Water Supply	
	Supplement # A	
46.2	Real Estate, Spring Creek Fish Hatchery	
	Supplement # 1 and # 2	1970
47	Wind Wave Investigation	1968
48	Navigation Lock, Remedial Repair	Jul-79
	Supplement # 1 - Nav Lock Remedial Repair	Jul-80
	General Letter Report	
	John Day L&D, Juvenile Fish Bypass System	Apr-82
	Juvenile Fish Bypass System - Submerged	
	Traveling Screen Handling Screen - <u>LETTER REPORT</u>	Dec-82
	Suppl. No. 4 to Gen. TLR Report - SUBMERGED	
	TRAVELING SCREEN MAINTENANCE FAC.	Feb-85
Ltr Rpt	Utility Modifications	
Ltr Rpt	Irrigation Pumps Plant Modifications	
49	Juvenile Fish Sampling and Monitoring Facility	Sep-95
*	Preliminary Brochure	1954

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*	Brochure for Meeting of Board of Consultants	1957
*	Letter Report on Main Unit & Total Power Plant Size	1959
*	Supplemental Letter Report on Main Unit & Total Power Plant Size	1960
*	Preliminary Design Study Highway Bridge Relocation	1959
*	Specification, Arlington City Hall	1962
*	Relocation of Union Pacific Railroad 60-190	1962
*	Operation Manual - Potable Water Supply	1963
*	Transcript of Public Hearings on John Day	1960
*	Cemetery Relocations, Final Report, Relocations Irrigon Cemetery	1963
*	Cemetery Relocations, Blalock Cemetery	1960
*	Final Report on WA Shore Cemetery and Burial Sites	1962
*	Cemetery Relocations - Final Report Riverview Cemetery at Boardman	1965
*	Metallurgical & Weld Investigation	1963
*	Metallurgical & Weld Investigation	1964
*	Report on Concrete Operations & Tests	1969
*	Supplemental Report - Concrete Operations and Tests	1970
*	Construction History	1970
*	Foundation Report	
	Part I	1971
	Part II & III	1971
*	Bridge Inspection Report # 1 - Navigation Lock Spillway	1971
*	Inspection Report #1	1969
*	Inspection Report # 5	1973
50	Spillway Flow Deflectors	1996

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. <u>Spillway</u>

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. <u>Powerhouse</u>

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. <u>Navigation Lock</u>

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	South Shore	North Shore

Fish ladder:		
Maximum design river flow, cfs	700,000	
Slope	1 on 16	
Regulation for pool fluctuation	Tilting Weirs	
Fixed weir height, feet	6	
Normal ladder flow, cfs	165	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 orifices	42	---
Overflow weirs	3	---
Velocities, feet per second:		
Through orifices	8	---
Over weirs	4	---
Diffusion chambers, number	42	---
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
First and second step embankments	800,000
Third step embankment	300,000

12. Costs

Initial (12 completed power units and 8 Skeleton Units)	$387,000,000
Ultimate (20 completed power units)	$448,000,000

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PLATES

EXECUTIVE SUMMARY

Existing juvenile fish passage facilities at the John Day Dam include standard length submersible guidance screens upstream of each main turbine unit intake, and a collection and bypass facility that discharges the outmigrants approximately 335 meters (m) downstream of the powerhouse. Fish Guidance Efficiency (FGE) has been estimated at 72% and 26% for yearling and subyearling chinook outmigrants respectively (Brege et al. 1987) (1994 - 1999 Biological Opinion for Operation of the Federal Columbia River Power System (BiOp), National Marine Fisheries Service). A new smolt monitoring facility will begin operation in the spring of 1998 but will not increase FGE at the powerhouse.

Extended length submersible guidance screens are currently under investigation at the John Day Dam and may increase FGE for yearling and subyearling chinook outmigrants to 84% and 60% respectively (Brege et al. 1997). Operation of the extended screen bypass system will not however, meet the fish passage requirements outlined in the BiOp under all conditions, nor will it address the extended juvenile salmonid holding and delay problems identified in the forebay (Sheer et al. 1997) (Holmberg et al. 1997). Spillway flow will therefore be discharged in order to increase the number of outmigrants bypassing the project through non-turbine routes. The volume of spill required to provide both forebay attraction and acceptable tailrace conditions may be excessive.

In 1995, the Corps of Engineers evaluated multiple alternative surface bypass systems for the John Day Dam powerhouse and spillway (Harza 1995). The intent of these investigations was to determine possible structural options for juvenile fish passage facilities that may better utilize salmonid migrational behaviors, forebay flow pattern information, and the physical limitations of the project in order to assist or replace the existing guidance screen bypass system. By 1997, enough biological field data had been collected to identify an appropriate location for a surface flow outlet, that would not interfere with the operation of the screened bypass system. Therefore, the benefits of the screened bypass system can be maintained while additional fish passage measures are implemented. The combined operation of the screened bypass system and a surface flow outlet is thought to provide the most efficient fish passage strategy for the John Day Project. Although spill will not be eliminated, the efficiency of spilled water should be greatly improved under this scenario.

A 1:80 scale general model of the John Day Dam has been utilized to determine the discharge volumes necessary to provide a forebay attraction flow stimulus at the location juvenile fish are delaying. The biological information indicated that approximately 75% of the juvenile outmigrants will pass near the skeleton bay section of the powerhouse, provided that a flow stimulus extends 100 m upstream of the dam. A 1:55 scale sectional model of the modified skeleton bays was used to determine the appropriate discharge characteristics of the new structure, including the necessary total dissolved gas abatement measures.

This feature design memorandum presents the biological rationale and criteria, hydraulic design data, powerhouse modifications, spillway and gate design, cost estimates, and construction schedules for implementing a surface spillway at the north end of the John Day Dam

powerhouse. Information relevant to a complete system, enabling 535 cubic meters per second (m^3/s) surface bypass discharge under total river flow conditions typical to both the spring and summer outmigrations is presented. In addition, a one unit prototype system designed for operation under total river flow of less than 8495 m^3/s has also been presented. The prototype system was designed to maximize operation during the summer outmigration, under lower total river flow conditions, providing the greatest potential improvement to subyearling chinook salmon.

Each option contains three 6.4 m wide spillway chutes, 6 m horizontal crest and a spillway slope of about 60% for each unit. A 6.4 m wide by 7.8 m high tainter gate is located at the downstream end of the crest. The chutes transition to a 6.25 m horizontal apron at the downstream end of the structure. The horizontal apron is at elevation 47.9 m above mean sea level (msl) for all three chutes in Skeleton Unit 20, and at elevation 48.8 m msl for the three chutes in Skeleton Unit 19. These two elevations were necessary to maintain the appropriate gas abatement measures under flow conditions typical to the salmonid outmigration. Each chute under normal operating pool elevation discharges approximately 170 m^3/s. The chutes have been designed to operate in an open or closed manner.

Preliminary structural, mechanical, and electrical designs of the two-unit surface bypass were performed. For the single unit prototype, only Skeleton Unit 20 will be constructed. In addition, a gate alternative study was performed (Appendix B), and dam stability calculations were made for five different load conditions (Appendix F).

Construction of the proposed designs has been divided into two phases to minimize the impact on operation of the project during the salmon outmigration. Prior to phase one, the adult fishway entrances will be relocated downstream of main turbine Unit 16. Phase one will then be performed in three steps: 1) demolition of the powerhouse and construction of the chutes; 2) demolition of the structure between the powerhouse and Construction Base Line (CBL), and construction of the chute and west half of the spillway bridge 3) demolition of the tailrace deck and construction of the chute and tailrace bridge. Phase two consists of demolishing the structure east of the CBL, and construction of the chutes and east half of the spillway bridge.

Construction requires dewatering at both the downstream and upstream sides of the powerhouse. At the downstream side, a dewatering bulkhead will enable the concurrent construction of both units. Bracing the bulkhead against the powerhouse structure is required prior to removing the deck beams. At the upstream side, the hydrostatic force applied by the bulkhead to the structure limits the span of the bulkhead to 16.1 m. Therefore, the construction will take place in seven phases across the front of the powerhouse, each phase requiring a separate bulkhead placement. During each phase , demolition, chute construction, and tainter gate installation will occur prior to moving the bulkhead to the next position.

Construction access will be by truck across the forebay and tailrace decks, or by barge in the forebay and tailrace. For safety reasons, work from the tailrace and much of the work from the forebay will be limited during periods of spill, including the spill for fish season from April 15 to September 1. This limitation, and the relatively small construction site result in an extended construction period. Construction will take approximately three years for the two unit system, and approximately two years for the single unit prototype. If the prototype is completed first however, an additional two years will probably be required to complete the remaining unit.

Fish passage efficiency information should be collected prior to construction of either the prototype or the complete surface bypass system. An assessment of the extended length guidance screens, and a thorough evaluation of the spill program are necessary to determine the maximum potential benefit that can be expected from a surface bypass system. Additional biological studies should also include an assessment of the potential injury associated with the discharge flow characteristics at the outfall. Once this information is compiled, a decision on whether to construct the surface bypass system can be made, and the optimal long range fish passage measures can be determined for the John Day Dam.

Cost estimates of both the two-unit and single-unit options were completed. Construction of the two unit surface bypass spillway would cost $51 million before contingencies, escalation to expected construction date, engineering and construction management. Construction of the single unit surface bypass would be $32 million before contingencies, escalation to expected construction date, engineering and construction management.

SECTION 1.0 INTRODUCTION

1.1 General

The National Marine Fisheries Service (NMFS) declared the Sockeye salmon in the Snake River to be endangered in 1992 and the Chinook salmon in 1994. In the resulting Biological Opinion (March 1995) the NMFS promulgated that the goal for fish passage efficiency (FPE) at each dam be 80% and the goal for fish passage survival be 95%. Recent surface collection successes at other hydroelectric projects has sparked interest in studying surface collection at the Corps of Engineers' Lower Columbia River Projects. Surface collection would be used by itself or in conjunction with the present juvenile bypass system.

It was found in fisheries studies that at the John Day Project downstream migrants swim laterally across the spillway and powerhouse. Since they are swimming in the upper portion of the water column, a surface collector with a strong enough draw would attract the migrants and pass them downstream over the dam.

In subsequent model studies it was found that conversion of only two Skeleton Units into surface bypass spillways produced sufficient flow to attract fish to the spillways under most flow scenarios. It was also determined that the flow did disrupt tailrace flow patterns, however adjustment of the main spillway and powerhouse discharges can solve this problem. It was decided to locate the surface spillways at the north end of the powerhouse at Skeleton Units 19 and 20. The reasons for this location are:

1. The skeleton units provide a good location for attracting and collecting fish and will eliminate the dead zone in the tailrace where spilled fish delay and are subject to predation.

2. Locating the surface outlets at the powerhouse or spillway would reduce the generating and/or spill capacity of the project. The effort associated with generator removal would also increase the program costs substantially.

3. Utilizing Skeleton Units 19 and 20 will maintain future powerhouse expansion possibilities at Units 17 and 18.

1.2 Purpose

In a 1996 reconnaissance feasibility study of spillway passage it was found that a surface bypass spillway located at the north end of the powerhouse is the most attractive alternative for surface passage of juvenile salmon. The purpose of this Feature Design Memorandum (FDM) is to present biological rationale and criteria, spillway and gate designs, cost estimates, and a construction schedule for implementation of a surface bypass spillway at the north end of the John Day Powerhouse. The FDM will identify pertinent technical, operational, and maintenance factors, establish specific design criteria, and develop a preliminary design for the structure.

1.3 Scope

The John Day Powerhouse was originally designed to have the capacity to install up to 20 turbine units. At present, 16 turbines are installed. No turbines are installed in Skeleton Units 17, 18, 19, and 20 at the north end of the powerhouse. This design memorandum presents design and construction information for a surface bypass spillway to be placed at the north end of the powerhouse. Two options are analyzed: three spillway chutes in each of Skeleton Units 19 and 20; and three spillway chutes in Skeleton Unit 20. The work consists of a hydraulic analysis and preliminary structural, mechanical, and electrical design of the spillway and its gates and bulkheads. Other work involves developing a gate alternatives analysis, cost estimate, construction schedule, cofferdam layout, and operation and maintenance considerations. This effort is coordinated with the fisheries agencies through progress review meetings and through the Anadromous Fish Evaluation Program, Fish Facility Design Review Workgroup. Three Progress Review Meetings are being held to obtain comments from all Corps offices and fisheries agencies on the 30%, 60% and 90% drafts of this FDM.

This FDM describes the full surface bypass option of constructing six chutes in Skeleton Units 19 and 20. The second option of using a single Skeleton Unit is merely half of the full bypass option. Section 9.0 describes the single unit option.

The dimensions in this report are given in metric units. All existing features are presented in metric units with English units in parentheses and dimensions of new features are expressed in metric units only.

1.4 Authorization

The John Day Lock and Dam was authorized by the Flood Control Act of 1950, in accordance with the Report of the Chief of Engineers in House Document 531, 81st Congress, 2nd Session, and modified for development of waterfowl management areas by the Flood Control Act of 1965 in accordance with the Report of the Chief of Engineers in Senate Document 28, 89th Congress, 1st Session.

This design Memorandum is authorized under Contract No. DACW57-97-D-0004, Delivery Order No. 0001 between the CH2M HILL Montgomery Watson Joint Venture and the U. S. Army Corps of Engineers - Portland District.

1.5 Previous Studies

Previous studies have been conducted for surface passage of downstream migrating fish. The study report titles are listed below:

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

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

SECTION 2.0 BIOLOGICAL CONSIDERATIONS

2.1 Introduction

Juvenile salmonid bypass system development on the lower Columbia River has evolved considerably over the last three 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 the critical design components for surface bypass technology.

The surface bypass program utilizes juvenile salmon migrational 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.

2.2 Design Strategy

The Portland District, Corps of Engineers has chosen to evaluate the critical assumptions governing bypass system design prior to initiating construction of full scale surface bypass prototypes. The following basic premises govern the development of surface bypass systems at the John Day Dam, and at the other lower Columbia River hydroelectric projects:

1. Bulk Flow: Juvenile outmigrants follow bulk flow as they migrate toward the dam. This dictates their lateral and vertical position in the forebay and predisposes them to pass at certain sites along the dam face. Therefore, locating bypass entrances where the bulk flow enters the dam is advantageous (Johnson et al. 1997).

2. Preference: Juvenile salmon migrate in the upper portion of the water column and prefer not to sound prior to entering a bypass system (Johnson et al. 1997). As a result, the outmigrants tend to delay upstream of dams with bypass outlets below approximately 12 meters (40 ft) deep.

3. Response to Flow Fields: Juvenile salmon maintain themselves in a particular flow field and avoid abrupt changes in the flow net when actively migrating. If the characteristics

of the flow net are abruptly modified, by reducing flow velocity for example, the fish will fall out of the flow net and search for areas of increasing velocity.

4. Zone of Flow Separation: Flownets entering various portals through a structure are distinctive and can be manipulated to attract fish to preferred locations. The specific location where these flow nets diverge constitutes the zone of flow separation. The zone of flow separation can be used to divert the upper potion of the water column, carrying the juvenile salmonid outmigrants, into a surface outlet.

5. Active Avoidance: Downstream migration is governed by avoidance responses to features in the hydraulic environment that are for some reason not conducive to fish passage. It is unlikely that these mechanisms will ever be thoroughly understood therefore, bypass entrances should be designed to limit disruptions in the fluid environment.

6. Zone of Influence: It is necessary to distinguish surface bypass system entrances from the flow nets entering the turbine units. Although many entrances located along a dam structure will increase the opportunities for juvenile outmigrants to enter a surface bypass system by chance, changes in the flow pattern are somewhat more important in order to actively attract outmigrants to the surface bypass system. Therefore, velocities that constantly accelerate towards the surface outlet define the influence of the entrance.

2.3 Background

The Bonneville, The Dalles and John Day hydroelectric projects are the focus of lower Columbia River surface bypass design efforts. Fish passage was considered throughout the design process of each of these dams, beginning with adult passage evaluations and fishladder design at the Bonneville Dam in the early 1930's. In 1952, the Corps initiated a formal fish passage research program which continues today (Ferguson et al. 1997). Screens, lights, louvers, and electricity, among other techniques have been investigated as a means to decrease the total numbers of juveniles passing through turbines. Aside from screens however, most techniques were unsuccessful when applied to the large scale of the lower Columbia River dams. Discharge through each turbine unit ranges from 168 to 616 cubic meters per second (m^3/s), and powerhouse capacities vary between 268 and 9016 m^3/s (Ferguson et al. 1997). Excessive turbidity and debris loads, especially during the spring freshet, and large slack water forebays also complicate system designs.

The bulk flow patterns established by specific dam operations upstream of hydroelectric projects is considered to dictate the juvenile salmonids lateral and vertical position in the forebay, and predisposes them to pass at certain sites horizontally along the dam face (Johnson et al. 1997). Throughout the 1970's and 1980's, screened bypass system designs utilized the bulk flow entering turbine units to consolidate the outmigrants in a manageable volume of water. The distributions of fish vertically in this volume of water was then ascertained, and that information utilized to govern the length of the diversion necessary to guide a small percentage of the total flow, and the entrained fish away from the turbine unit. A porosity panel was designed to control the volume and flow velocity at the screen face to prevent fish from contacting the guidance device.

Although the initial guidance screen evaluations were promising, continued investigations indicated less than satisfactory performance. At the Bonneville Dam, the fish guidance efficiency (FGE), the number of juvenile salmonid diverted away from the turbine unit, for subyearling chinook salmon was estimated as low as 4.4 percent (Gessel et al. 1990). FGE for subyearling chinook at the John Day Dam has been estimated at 25 percent (Brege et al. 1987). Due in part to these estimates, and the lack of systematic success with this type of technology at other locations, guidance screens have not been installed at The Dalles Dam.

Guidance screen development information has been included in this section only to provide a limited description of bypass system evolution over the last several decades and to document some of the basic limitations with guidance screen technology. The shortcomings associated with the operation of screened bypass systems, particularly with the system at the John Day Dam, can likely be mitigated to a degree with surface bypass system technology. Surface bypass system designs utilize the applicable biological information developed during screened bypass system development and expand on that information to account for delay and peaking diel passage rates. A combination of the two systems may therefore, provide the most efficient means of fish passage at the John Day Dam.

The final surface bypass system design for the John Day Dam is a result of information collected through biological evaluations of salmonid migrational behaviors in the John Day Dam forebay, and a coordinated plan of action for fish passage systems at the John Day Project (Appendix C, 2/26/96 Memorandum for the Record). It has been designed to produce a zone of influence 90 m upstream from the dam face, to minimize TDG production and to discharge without disrupting tailwater flow patterns. The design also incorporates the physical constraints of the dam and the limitations of available flow at total river discharges less than 8495 m^3/s.

Data obtained from the Waterways Experiment Station (WES) general model were used to determine the structural dimensions necessary to provide the required attraction flow. The resulting 535 m^3/s surface bypass system flow at a typical pool elevation of 80. 5 m msl was divided into three chutes, each supported by the original framework of the powerhouse. A cross sectional area of approximately 122 square meters (m^2) was necessary to obtain 535 m^3/s through the three chutes. The bypass entrance floor elevation was therefore designed 6 m below normal pool to accommodate this flow level. A total discharge versus air entrainment analysis was conducted, and the volume of water discharged under these conditions is expected to minimize gas supersaturation.

In order to accommodate 510 m^3/s discharge under all tailwater conditions however, two separate outfall elevations are required. Therefore, Surface Bypass Unit 20 was designed with discharge chute elevations at approximately 48 m msl for operation at total river flows of less than 8495 m^3/s, and Skeleton Unit 19 was designed with discharge chute elevations at approximately 49 m msl for total river flows between 5663 m^3/s and approximately 15574 m^3/s. If necessary, both surface bypass units can operate from 5663 m^3/s to 8495 m^3/s total river flow.

Subtleties in the design include operating gate style, shaped pier noses, chute slope, and the length of the downstream, horizontal component of the outfall chute. These features were designed to optimize the hydraulic performance of the bypass, eliminating standing waves, turbulence, and eddies within and downstream of the system. These improvements should assist in reducing flow related stress and mortality, and improve survival of downstream outmigrants.

2.4 Site Description

The John Day Dam is located at river kilometer 347 on the lower Columbia River and is owned and operated by the US Army Corps of Engineers. It was authorized for hydropower production and navigation. Unlike run of the river dams, the John Day Dam is a storage project and can be manipulated to provide additional flood control for the lower river. It impounds approximately 120 kilometers of the Columbia River upstream to the McNary Dam. The initial project was completed in 1971 with 16 of 20 main turbine units in operation. The remaining four units have not been constructed, leaving an unimproved section of the powerhouse approximately 122 meters (m) long between the powerhouse and spillway.

Each turbine unit has three six meter wide intakes with the ceiling elevation located approximately 16 meters below normal pool, at elevation 64 meters above mean sea level (msl). The units have a generating capacity of up to 155 MW at approximately 566 m^3/s of discharge. The total hydraulic capacity of the powerhouse is greater than 9061 m^3/s.

The spillway is located adjacent to the powerhouse and abuts the Washington shoreline. It has 20, 15 m wide spillbays each capable of discharging up to 1416 m^3/s under normal pool elevations. The total spillway discharge capacity in a flood event is approximately 70800 m^3/s. The ogee crest is located at elevation 64 m msl, approximately 16 m below normal pool. Flow through each bay is regulated with a radial gate. The spillway stilling basin is relatively deep and discharge in excess of 90 m^3/s per spillbay increases total dissolved gases to above 120 percent of atmospheric pressure. When possible, total spillway discharge is maintained below approximately 1812 m^3/s due to these limitations.

The forebay pool typically fluctuates between elevation 80 m msl and elevation 81 m msl. The operating range of the project however varies from elevation 78 m msl to 82 m msl. Normal operating pool during the fish passage season is elevation 80.5 m msl. A navigation lock is located along the Washington shoreline.

In a typical flow year, total river discharge at the John Day Dam averages approximately 9911 m^3/s from April through June. By the end of August, river flows have subsided to approximately 5097 m^3/s. Yearling outmigrants, including spring and summer chinook salmon, steelhead, and sockeye and coho salmon pass the project from mid April though early June. Subyearling fall chinook outmigrants pass the dam from mid June through August.

Fish passage facilities include two adult fishladders and a screened juvenile bypass system. The north adult fishladder has two main entrances located adjacent to spillbay one. The ladder exits along the Washington shore. The south adult fishway currently has three main entrances, one at the south end of the powerhouse and two smaller entrances at the north end of the powerhouse. Ten floating orifice type entrances are distributed across the downstream powerhouse face. The south fishladder exits adjacent to the Oregon shoreline.

The juvenile bypass system has undergone several modifications over the last 25 years. Currently each main unit intake has a 6 m submersible guidance screen that diverts approximately 5.7 m^3/s of flow up into the dewatering gate slot. All but 0.4 m^3/s of this flow is removed by a vertical barrier screen located between the dewatering gate slot and the operating gate slot. The remaining 0.4 m^3/s of water and all of the guided fish are discharged through a 0.4 m orifice, into a collection channel, and eventually released approximately 183 m downstream of the powerhouse adjacent to the Oregon shoreline.

2.5 Fish Passage Information

Bulk flow theory has governed the development of screened bypass systems, and is important in the development of surface bypass systems. The ultimate route of fish passage is necessarily with the main body of water moving through the dam (Johnson et al. 1997). Additional characteristics of the hydraulic environment must also be understood however, and incorporated into a surface bypass system design. Ascertaining how juvenile outmigrants move through specific flow fields, and what they are responding to has been a priority in surface bypass system development on the lower Columbia River.

Juvenile salmon are distributed in the upper portions of the water column. At the John Day Dam, the majority of spring outmigrants are located in the upper 10m, with no major changes in vertical distribution between day and nighttime passage (Biosonics 1997). Summer outmigrants are also surface oriented but tend to be distributed slightly lower in the water column during nighttime passage. The majority of summer outmigrants are however, located in the upper 15m of the water column over the 24 hour period (Biosonics 1997).

The horizontal distributions of juvenile salmon across the spillway and powerhouse was monitored with hydroacoustic transducers from 1980 through 1989 (Johnson et al. 1997), and in 1997 (Biosonics 1997). The information was relatively consistent, and largely dependent on dam operations which determine the bulk flow patterns upstream of the project. Distribution was relatively even across the powerhouse for most years, though on occasion an increase in passage was noted towards the south end of the powerhouse. Juvenile outmigrants tended to pass the spillway towards the south end bays, located closest to the powerhouse. In 1997, when the entire spillway was operating, fish passage peaked at both ends of the spillway, with fewer fish passing through the middle bays. Summer outmigrants showed a strong preference for passage over the north end of the spillway (Biosonics 1997).

From 1995 through 1997, yearling and subyearling chinook salmon were radio tagged and monitored as they approached and passed the dam (Sheer et al. 1997) (Holmberg et al. 1997). Horizontal distribution was similar to that noted in the hydroacoustic evaluations, though passage also occurred readily at the north end of the spillway under high river discharges.

In addition to the horizontal distribution information collected throughout the radio telemetry investigations, approach patterns and delay information has also been obtained (Sheer et al. 1997) (Holmberg et al. 1997) (Stuehrenberg and Liscom 1983). Three predominant approach patterns were identified for juvenile outmigrants. Within approximately 457 m of the dam, salmon tended to approach from the north shoreline, move across the spillway and first encounter the dam near the center, unimproved section of the powerhouse (Sheer et al. 1997). Salmon also approached from mid channel and moved directly towards the center of the powerhouse. For juvenile steelhead in particular, a third main approach path was apparent along the south shore.

Whether fish approached the dam from the north shore, mid channel, or from the south shore however, certain characteristics were prominent in all species tested. Juvenile outmigrants tended to delay within approximately 91 m of the dam structure and traverse the powerhouse from shoreline to shoreline. In addition, under normal to low flow years, juvenile outmigrants pass the project almost exclusively during the 8.5 hour period from dusk to dawn (Johnson et al. 1997).

The length of delay is dependent on the total river discharge volume. In 1995 total river flow throughout the test period averaged approximately 9061 m^3/s and the associated delay was 10.3 hours for yearling chinook salmon (Sheer et al. 1997). In 1996, river flows increased to an average of 11327 m^3/s during the spring outmigration and yearling chinook residence times decreased to 2.8 hours. Under reduced flows averaging 8495 m^3/s during the summer subyearling chinook delay increased to a mean of 10.0 hours (Holmberg et al. 1997). In 1997, flows exceeded 14159 m^3/s for much of the spring outmigration, and mean delay was estimated at approximately .5 hours (Hensleigh et al. in preparation). The degree of milling behavior is directly related to the time juvenile outmigrants delay in the forebay, and the diel information indicates that the fish are actively searching for an outlet conducive to their spatial distribution.

The biological information collected over the last three years at the John Day Dam supports research results obtained in the late 1970's and 1980's. Juvenile outmigrants approach the dam in the upper 10m of the water column, and without a surface flownet to bypass the dam, they will delay. For the duration of the time they hold upstream of the dam, juvenile salmon traverse the project, likely in search of a surface flow outlet. At dusk, the fish sound and follow the predominant flow patterns into the turbine unit intakes. A percentage of the outmigrants is unable to avoid the guidance screens and are diverted into the bypass system. The remainder follow constant and increasing velocities under the guidance screens and into the turbine. Under lower total river flow conditions, delay increases. Increasing river flow and spill volumes will expedite forebay passage.

A surface bypass system at the John Day Dam should therefore be designed to reduce the delay associated with dam passage under lower flow conditions, and to supplement the fish guidance efficiency of the screened bypass system. To accomplish this, the bypass system design will utilize the delay and searching behaviors demonstrated under lower flow conditions by creating regions of increasing velocities through the upper 15m of the water column. This zone of influence will likely attract outmigrants to the bypass entrances during the day, though the efficiency of the system will depend largely on our understanding of how juvenile salmonids respond to flow fields.

When river flow exceeds the powerhouse hydraulic capacity, fish passage through the screened bypass system can be supplemented with involuntary spill. Therefore, the John Day Surface Bypass System should be designed to optimize passage conditions during river flows of less than 8495 m^3/s, and for subyearling outmigrants. The system should however, efficiently pass yearling outmigrants as well.

2.6 Forebay Zone of Influence

In 1995, the Corps completed a 1:80 scale general model of the John Day Dam WES in Vicksburg, Mississippi. The initial model tests were intended to reproduce the field conditions that occurred throughout the fish passage evaluations. Velocity and other flow measurements were initially observed with dye releases made at specific points of interest. Point velocity readings were taken with portable flow meters, and the larger bulk flow patterns were measured with weighted corks that were timed as they floated through sections of the forebay and tailwater.

The bulk flow patterns generated in the forebay of the John Day Dam, under typical operating conditions, create a zone of slightly higher velocities that move from the Washington side of the river, to approximately mid channel. Velocities reduce as the bulk flownet moves within approximately 90 m of the dam. Dye released in the area of higher velocity eventually encounters the project upstream of the unimproved section of the powerhouse (refer to Section 3, Hydraulic Design). This information was consistent with the radio telemetry observations of salmon outmigrants conducted in the field.

Under total river flow conditions of less than 7079 m^3/s, bulk flow velocities average .06 meters per second (m/s) in the upper 8 m of the water column. Utilizing the zone of influence theory as discussed above, the goal of the hydraulic evaluations was to create a flow field that would constantly increase in velocity from .06 m/s approximately 90 m upstream of the dam, to a trapping velocity of 2 m/s at the bypass entrance. Based on the biological information, this flow scenario would provide the most appropriate bypass system conditions, maximizing the attraction to the entrances by manipulating the bulk flownet upstream of the project. Trapping velocities at the bypass entrances are necessary to keep outmigrants from rejecting the bypass chutes once they have entered the system. Problems encountered at other prototype surface bypass systems indicate that salmonid outmigrants may move upstream when they encounter areas of unstable flow or reducing velocity inside of the bypass system (Chuck Pevin pers. com. (Rocky Reach Dam)).

The volume of water required to manipulate the bulk flownet 91 m upstream of the dam face, in the upper 15 m of the water column, can exceed 708 m^3/s depending on where the entrances are located along the dam structure. One 708 m^3/s entrance located upstream of approximately every other turbine unit would create a reasonable flow pattern in the upper 9 m but turbine intake flow would likely overwhelm bypass flow at the lower depths. In addition, eight surface bypass entrances, each discharging 708 m^3/s would require a structure large enough to handle 5663 m^3/s. The practical limitations are relatively clear.

A similar system designed in 1995 for the John Day Dam utilized eight bypass entrances, but the total discharge per entrance was reduced to approximately 54 m^3/s each (Harza and ENSR 1995). The projected flow parameters would not manipulate the bulk flow patterns at 91 m, but the size of the system was within reasonable construction efforts. The cost to build a system this size was estimated at over $270,000,000 (Appendix C, 2/26/96 Memorandum for the Record).

The biological information discussed above however, indicates that juvenile outmigrants would likely encounter an attraction flow projected 91 m upstream of the dam, provided they were actively searching for a surface bypass outlet. Therefore, locating the collector entrance at the spillway would require a greatly reduced construction effort, and would provide a zone of influence independent of turbine unit flow. At total river discharges of less than 7079 m^3/s however, the powerhouse could potentially discharge the entire river. It was therefore determined that the zone of influence should project upstream of the powerhouse, at the very least immediately adjacent to the bulk flow pattern generated by turbine unit discharge. A zone of influence in this area should increase the opportunity for juvenile salmonids to encounter the surface bypass attraction flow.

Discharging 708 m^3/s from the water surface at the unimproved, skeleton bay section of the powerhouse has several advantages. The total discharge required to manipulate the target area could be reduced to approximately 510 m^3/s. Eighteen kcfs discharged from the upper 6 m of the water column consistently increases flows from .06 m/s 91 m upstream of the dam to over 6 m/s at the bypass entrance, draws water from the channel bottom at the dam face, and draws water from approximately 8 m deep at a distance 91 m upstream of the structure. Although it would be difficult to locate entrances across the powerhouse, the searching behavior observed throughout the radio telemetry studies indicate that salmonid outmigrants should encounter a zone of influence concentrated adjacent to the bulk flow net created by the powerhouse. The skeleton bays section of the project can be modified to provide this bulk flow net.

2.7 Tailrace Conditions

Discharging 510 m^3/s from the center of the dam (at the skeleton bay section of the project) creates potential problems in both the tailwater flow patterns and increasing total dissolved gas (TDG) levels. State and Federal water quality guidelines require TDG levels to be maintained below 110 percent. Over the last several years however, waivers have been permitted on the lower Columbia River for up to 120 percent TDG during the fish passage season. To minimize TDG levels to the extent practicable, flow exiting the surface bypass system should skim along the downstream water surface. Model evaluations conducted on a 1:40 scale sectional model of the surface bypass system demonstrated that the most desirable skimming flow occurs when the tailwater surface is between 0.3 m and 1.5 m above the chute exit elevation. Increased air entrainment and potentially high TDG saturation result when submergence is greater than 2 m.

Planned operation of the bypass system is from April through September, when mean daily total river discharge typically ranges from 2832 m^3/s to 11327 m^3/s. On occasion, summer flows can be as low as 2265 m^3/s. A chute exit elevation of 48 m msl will provide skimming flow for most tailwater conditions associated with total river flows ranging from 2256 m^3/s to 8495 m^3/s. A chute exit elevation of 50 m msl will provide skimming flow for most tailwater conditions associated with total river flows ranging from 5663 m^3/s to 11327 m^3/s. The surface bypass system was therefore designed to discharge 535 m^3/s at two downstream outfall elevations in order to minimize TDG supersaturation at total river flow levels likely to occur during the juvenile outmigration (refer to Section 3, Hydraulic Design).

The shallow exit chute submergence required to control TDG greatly influences tailwater conditions. Training flow discharged from both the powerhouse and spillway is relatively effective in channeling the surface bypass flow downstream, but careful manipulation of the system is required in order to avoid large tailwater eddies. Juvenile salmon that are delayed in shoreline eddies experience increases in mortality due to predation (Shively et al. 1994). Preliminary general model investigations were conducted to determine the range of spillway and powerhouse discharge levels conducive to surface bypass system operation (Figure 3.11). At total river flows of 5663 m^3/s for example, approximately 1274 m^3/s should be discharged from the spillway and 3879 m^3/s discharged from the powerhouse to optimize both forebay attraction and tailwater conditions. As flow is reduced to either side of the surface bypass system (a reduction in either spill or powerhouse flow), the associated eddy increases in size. A range of acceptable spillway and powerhouse discharge volumes has been calculated for a one unit prototype system (Section 9, Figure 9.1), but the preliminary calculations determined for a complete system need additional verification.

2.8 Biological Design Criteria

The National Marine Fisheries Service (NMFS) has provided guidance for design of bypass systems in the form of fish passage criteria (NMFS 1995)(NMFS 1996). Although these criteria are based on the best available biological information, there may be instances throughout the design process where specific criteria are not available, or are not appropriate in this application. Where the design does not meet NMFS criteria, additional biological investigations will have to occur prior to constructing the prototype bypass system. These areas will be thoroughly identified throughout this document.

The existing NMFS criteria are specific to the development of conventional dewatering and screening facilities, and bypass systems designed for minimal discharge. For reference, those criteria have been included in Appendix C.

2.9 Preliminary Assessment of System Effectiveness

The number of juvenile outmigrants utilizing a surface bypass system is necessarily dependent on their ability to discover the entrances. In order to maximize the opportunity for discovery at the John Day Dam, the surface bypass system has been designed to take advantage of a salmonids positive response to flow, and to minimize potential disruptions in the flow field that may cause a fish to reject the entrances.

The information necessary to accurately predict how fish will respond to accelerating velocities in an open system is not available at this time and therefore impacts any effectiveness assessment. An expected range of efficiency, defined as the total number of salmonid outmigrants utilizing the surface bypass system compared to the total number of outmigrants bypassing the John Day Dam, has been developed however, based on measures of entrance effectiveness determined from evaluations conducted at the Wells and Wanapum dams surface bypass systems.

The Wells Dam has a comparatively effective system, bypassing approximately 75 percent of the outmigrants that discover the entrances. The Wanapum Dam is less efficient, bypassing approximately 57 percent of the outmigrants that discover the surface bypass system entrance. Because these systems have not maximized the zone of influence upstream of the collector however, fish are likely only encountering the attraction flow within a relatively close proximity to the bypass entrance. Therefore, these measures of effectiveness were developed based on the number of fish within three meters of the entrance, that actually entered the bypass systems. A large percentage of the population was able to discover the Wells Dam bypass, likely do to the number and proximity of each entrance in relation to the turbine unit intakes at powerhouse. Therefore, total system efficiency is much higher at the Wells Dam.

Juvenile outmigrants within 90 m of the John Day Project, upstream of the skeleton bay section of the powerhouse, are thought to encounter the zone of influence. Therefore, to develop a range of efficiency for the John Day Dam surface bypass system, 75 percent entrance effectiveness was utilized from the Wells Dam research as an estimate of the potential upper range of effectiveness possible at the John Day Dam. The lower range was based on the 57 percent entrance effectiveness documented through research at the Wanapum Dam. Both of these effectiveness estimates were then applied to juvenile outmigrants passing within 90 m of the John Day Dam surface bypass entrances. This seems to be a reasonable approach given the design parameters of the John Day Dam system. Water velocity in the forebay has been increased, and the direction of flow is consistently towards the bypass entrance in the defined zone of influence.

Based on radio telemetry data collected in 1995 and 1996, 75 percent of the yearling outmigrants, and 77 percent of the subyearling outmigrants will come into contact with the surface bypass system zone of influence while they are holding and searching for a route past the John Day Project. Given an entrance effectiveness range of between 57 percent and 75 percent, the total efficiency of the John Day Dam surface bypass system should be approximately 23 percent to 71 percent for the yearling outmigrants, and approximately 23 percent to 74 percent for the subyearling outmigrants.

The efficiency per volume of water of this system can be determined by estimating the total number of fish utilizing the surface bypass system compared to the total volume of water discharged by the surface bypass system as a percentage of total river flow. During an average spring flow of 9911 m^3/s for example, approximately five percent of the total river flow will be discharged through the surface bypass system. The range of bypass efficiencies for the spring outmigration can therefore be estimated at between 4.6 and 14.25 to one, percent of fish to percent of total river flow. During the summer outmigration, approximately ten percent of the total river flow will be discharged by the bypass system given an average total river flow of 5097 m^3/s. Therefore, bypass efficiencies for the summer outmigration are estimated at between 2.3 and 7.4 to 1, percent of fish to percent of total river flow utilized by the surface bypass system.

2.10 Projected Survival

There are no particular design features associated with the skeleton bay surface bypass system entrance that would indicate a potential cause of injury. In addition, water velocity down the bypass chute is typical of other lower Columbia River spillways, and is not expected to increase mortality. The hydraulic parameters of how flow exits the chute and enters into the tailrace however, are substantially different from the plunging flow typical of most spillways. Plunging flow has been replaced by skimming, undular flow and although the outfall discharge characteristics are more similar to spillway and flow deflector discharge, and may appropriately be designed in accordance with those criteria, the impact velocity is much higher than current outfall criteria allows. The combination of these characteristics may increase injury of bypassed fish due to potential increases in shear.

Discharge flow characteristics from the John Day Dam surface bypass system are similar to flow characteristics demonstrated by spillway flow deflectors. Survival estimates have been calculated for many of the deflectors currently in operation, most recently at the Little Goose Dam on the Snake River (Muir et al. 1997) (Mathur et al. 1997). Although the unit discharge volumes tested were approximately half of the design discharge for the John Day Dam surface bypass system, they can be used as an indicator until additional testing can be completed at discharge volumes of approximately 425 m^3/s. Various spillbay discharge volumes up to 272 m^3/s were evaluated using balloon tagged hatchery steelhead at the Little Goose Dam. There were no significant differences in the survival of hatchery steelhead at 272 m^3/s total flow (Mathur et al. 1997). Total survival at the Little Goose Dam was also evaluated using PIT tagged hatchery steelhead. There were no statistical survival differences between hatchery steelhead spilled over a deflector bay and a non deflector bay (Muir et al. 1997).

The impact velocity at the outfall chute may also be a concern. Although flow deflector evaluations and other spillway survival evaluations indicate comparatively high spillway survival at impact velocities in excess of 15 m/s, the preponderance of information that has been collected relative to shear as a potential causative mechanism for injury to salmonids suggests that velocities in excess of 15 m/s should be avoided (Johnson, 1970) (Ruggles and Murray, 1983). Evaluations conducted by NMFS (Groves, 1972), suggests that shear forces equal to approximately 9 m/s applied directly to the operculum may cause injury. In each of the test cases however, the results appear to be difficult to apply to the conditions represented at spillways or at the John Day Dam surface bypass system outfall. *More definitive tests that represent conditions similar to the outfall should occur prior to assessing a level of mortality to this system.*

Based on dye observations conducted at the Waterways Experiment Station (WES) 1:80 scale general model of the John Day Dam, spillway flow of less than approximately 181 m^3/s per spillbay will delay for an extended period in the stilling basin. Therefore, under low flow conditions, the total amount of flow necessary to maximize tailwater survival will be substantial, and may be difficult to obtain. Discharge from the skeleton bay bypass system however, exits the chute and moves readily downstream. Based on survival estimates conducted at the Bonneville Dam from 1987 through 1990 (Ledgerwood et al., 1990), a 7% survival improvement can be expected if juvenile outmigrants are not subjected to delays in their downstream migration.

In addition to tailrace conditions, the large zone of influence and the benefits associated with surface oriented outlets should reduce the holding and delay demonstrated by juvenile outmigrants in the forebay. A reduction in delay upstream of the project should increase the total survival experienced by juvenile outmigrants bypassing the John Day Dam.

Reducing delay in the forebay, limiting retention times in the tailrace by increasing velocities and improving tailrace conditions under lower total river flows, and reducing TDG levels associated with spilled water should allow for a relatively high bypass and project survival rate. *Field evaluations should be conducted at similar unit discharge volumes however, prior to construction of the bypass system, in order to further evaluate the potential shear injuries that may result from operation of this system.*

2.11 Biological Assessment of the Training Wall

During development of the skeleton bay bypass system at the John Day Dam, an approximately 15 m long training wall was recommended by the dissolved gas abatement team between Spillbay 20 and Skeleton Unit 20. The intent of the training wall was to prevent spilled water, that could potentially be higher in total dissolved gasses than turbine bypassed water, from filling the flow demand created by operation of the surface bypass system (information regarding the interaction of powerhouse, surface bypass, and spillway discharge is limited however, so the reliability of this assumption is questionable). When observed on the general model, a small training wall (less than 76 m) only deflected spillway bypassed flow along the length of the wall. Once the dye released at the spillway moved downstream past the tip of the training wall, it would bend sharply and move upstream to fill in behind the surface bypass system discharge. Dye moving downstream, then back upstream indicates a potential problem with elevated tailrace passage and retention times for juvenile salmonids.

The training wall was therefore increased in length to prevent the spillway dye releases from turning back upstream. The eventual length of the wall was optimized at 76 m. Dye released in the adjacent spillbays, with the 76 m wall in place, continually moved downstream. The training wall also increased the distance dye released through the surface bypass system moved downstream prior to slowing, or entering shoreline eddies. Although dye would move readily downstream to a distance of approximately 1219 m without the training wall, the training wall prevented dye from entering the shoreline eddies approximately 1609 m downstream of the dam. The downstream movements of dye exiting the spillway and surface bypass system was enhanced by the 76 m training wall, but at the distances the improvement was noted, it is questionable whether fish would respond in similar fashion.

Additional evaluations on the general model revealed a large eddy downstream of the powerhouse that was more pronounced under certain conditions by installation of the training wall. Because the training wall prevented spillway flow from turning into the surface bypass system and satisfying the flow demand, powerhouse flow was drawn over in greater quantities. This set up an eddy at the screened juvenile bypass system outfall that caused dye releases through this system to delay on the shoreline, and either move across channel or back upstream until it was flushed out by surface bypass system flow. By forcing powerhouse channel water to fill the surface bypass system demand, conditions at the juvenile bypass system outfall were impacted. Developing optimal project operational scenarios should occur prior to construction of the surface bypass system, especially for total river discharges of less than 7079 m^3/s (preliminary investigations are presented in Section 9 for one skeleton unit prototype operation). The spillway, powerhouse and surface bypass system will all require certain discharge volumes in order to maintain appropriate tailrace conditions, and to not impact the existing juvenile bypass system outfall.

The screened juvenile bypass system will likely pass a substantial number of outmigrants, even after the surface bypass system is installed. The outfall for the screened bypass should therefore, not be impacted by operation of the surface bypass system. Although improvements were made to dye released from the spillway and surface bypass system with the installation of the training wall, the improvements were not substantial and would likely not result in additional system survival improvements. Impacting the screened bypass system outfall may however, result in decreased survival for juvenile outmigrants at the John Day Dam. *Given this information, and the uncertainties of the flow interactions, construction of a training wall is not recommended at this time*.

Under both conditions evaluated, with and without the training wall in place, spillway flow exited the area more readily if spillbay 20 was equipped with a flow deflector.

2.12 Biological Assessment of the Covered Drafttube Outlet

The Skeleton Bay Surface Bypass Spillway will be constructed over what is now the intake and drafttube section of two main turbine units. On the downstream side of the dam, the drafttube outlet has been excavated to elevation 20 m msl to optimize turbine unit operation. When the bypass system is constructed, the outfall will be at approximately elevation 49 m. This leaves an area of slack water approximately 27 m deep (depending on tailwater) by 31 m long by 32 m wide immediately underneath the surface bypass system outfall. Although general model observations did not indicate a potential problem with this area, filling this whole was investigated as an option that may limit potential predator holding area.

Drafttube covers were evaluated at elevation 34 m msl, equal with the bottom of the river channel downstream of the excavated area, and sloping from approximately elevation 49 m msl at the outlet structure down to elevation 34 m msl joining the river bed. There were no obvious changes in the hydraulic performance of the surface bypass system, or with flow entering behind the surface bypass system discharge from either the spillway or powerhouse, with the hole filled to elevation 34 m msl. When the floor was raised however, to eliminate all of the potential predator area (to the extent practicable), discharge from the surface bypass system concentrated towards the center chute, rolling over on either side. Although turbulence increased downstream of the of the two outside chutes, the concentrated spill enabled dye released in the surface bypass system to move further downstream without contacting shoreline eddy areas, in much the same manner as that noted during the training wall investigations. The elevated, sloping floor however was not considered further due to the increase in turbulence and shear.

The level floor at elevation 34 m msl had no affect on flow, and only had questionable potential benefits to juvenile outmigrants. It is unlikely that predators would hold underneath the surface bypass system outfall, given the velocity of the discharged flow. If they did, the velocities are high enough to eliminate predation on salmon bypassing the dam in this area (Steve Rainey, memorandum for record). There does not appear to be a biological benefit to filling the drafttube outlet, it is therefore not recommended.

2.13 Potential Impacts To Adult Passage

The south adult fishway has two main entrances currently in operation at Skeleton Unit 20. Each of the entrances has a two meter wide telescoping weir that is maintained approximately two meters below the tailwater surface. Each entrance discharges auxiliary flow of approximately 8.5 m^3/s, for a total system discharge capacity of 17 m^3/s at the two north entrances. The auxiliary flow is utilized for adult fish attraction water to the south fishway powerhouse collection channel. In addition to the two main entrances, main units 19, 18 and 17 have submerged orifice entrances into the adult collection channel. Each of the 2 m by 0.6 m orifice entrances discharges approximately 1.7 m^3/s of auxiliary attraction water. Construction of the surface bypass system will require that both the main entrances, and the three floating orifice entrances be relocated.

General model observations of the existing entrances indicate very little downstream attraction. As flow from the two main entrances exits the collection channel, it dissipates readily into the slack water area downstream of the skeleton bays. Attraction flow does not enter an area of potential adult fish activity, the hydraulic boundary adjacent to the powerhouse discharge ending at Unit 16 for example, or to the hydraulic boundary adjacent to spillway flow. Orifice flow is almost non existent downstream of the powerhouse. The existing conditions are likely not effective at attracting adult migrants to the south fishway. Radio telemetry and electronic tunnel studies conducted in 1979 and 1980 (Johnson et al. 1982), and additional radio telemetry studies conducted from 1995 through 1997 (Bjornn et al. in preparation) also indicate very little passage through the north powerhouse entrances to the south fishway, or through the submerged orifices in general.

Moving the entrances to an operational unit, and increasing discharge capacity to the extent possible, will likely improve adult attraction to the south fishway. This will afford migrating salmonids, that are likely traveling in or adjacent to the bulk flow net of the powerhouse, an opportunity to more readily discover the fishway entrances. Therefore, the two main fishway entrances were evaluated at Main Unit 16 on the general model, and total fishway entrance discharge was increased by approximately 5 m^3/s, the total volume of additional auxiliary flow available if the three orifice entrances at turbine Units 19, 18 and 17 were removed.

When dye was released through the new entrances, it remained concentrated as it exited the collection channel moved approximately 15 m downstream then laterally to the surface bypass discharge prior to being flushed out. These dye concentrations indicated that if upstream migrants followed the boundary zone between the surface flow of the bypass system and the submerged flow of the powerhouse, attraction flow being discharged from the new main entrances should be apparent. In addition, because flow exits the powerhouse at an operating unit, immediately adjacent to the bulk flow net of the powerhouse, adult salmonids migrating along the powerhouse slack water boundary should more readily encounter the two main entrances.

From the observations made on the general model, adult attraction to the south fishway should benefit from these modifications (see also Section 9, a one unit prototype would also benefit from these modifications, even though the north powerhouse fishway entrances would be 20 m further from the chute exits).

2.14 Conclusion

In order to increase confidence in the preliminary evaluations of the John Day Dam surface bypass system, additional information should be collected in the following areas:

1. Juvenile salmonid responses to accelerating attraction flows, especially regarding the forebay zone of influence.

2. Juvenile (and adult) salmonid survival under the flow discharge characteristics presented by the surface bypass system.

3. The influence of downstream eddies on juvenile salmonid distribution and tailrace egress.

4. Optimal ranges and distribution of spillway, powerhouse and surface bypass system flow, especially in relation to downstream eddies and impacts to the existing juvenile bypass system outfall.

5. Evaluation of surface bypass discharge as false attraction and a potential source of adult salmonid delay.

6. Possible impacts to adult survival as a result of head burn and other potential injuries.

Additional WES modeling, and field and laboratory evaluations will increase confidence in the estimates provided in this memorandum. Most of the evaluations can be completed in one to two years without actually constructing the prototype. Some of these issues however, item one for example, will likely remain elusive until a prototype system can be constructed and evaluated at the John Day Dam.

It may be beneficial to design and construct one surface bypass unit and evaluate its performance prior to constructing the entire system. Therefore, Section 9 of this memorandum outlines a one unit prototype system. Existing fish passage indices at the John Day Dam are maximized under the higher river flows typical of the spring outmigration. Fish guidance efficiency (FGE) and spillway passage efficiency are both higher for yearling outmigrants, and delay in the forebay is reduced. The greatest potential benefit for a system of this design is during the lower total river flows of the summer outmigration, when FGE and spillway efficiency are reduced, and forebay holding and delay is high. Design and construction of a prototype system should therefore be optimized at total river flows of 8495 m^3/s and less.

Although not thoroughly assessed in the development of this memorandum, the John Day Dam surface bypass system also has potential benefits regarding the abatement of total dissolved gasses (TDG) during involuntary spill. The design of the structure enables a much higher total flow to dissolved gasses ratio than either the existing spillbays or spillbays equipped with flow deflectors. Additional spill capacity of 510 m^3/s during the spring outmigration may enable project operations to stay within water quality parameters during the ten year flood event. A TDG structure would operate exclusively under higher tailwater and total river flows in excess of 8495 m^3/s. Although operation of the system at this level would benefit gas abatement, it would likely not provide passage benefit for the summer outmigration (refer to paragraph 3.5).

Given the available biological information, between 23 percent and 71 percent of the yearling outmigrants, and between 23 percent and 74 percent of the subyearling outmigrants should utilize the surface bypass system at the John Day Dam. Survival through the surface bypass system is estimated at 98 percent, including improvements in tailrace survival. Forebay retention time is also expected to decrease, increasing total project survival.

2.15 References

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SECTION 3.0 HYDRAULIC DESIGN

3.1 General

This section discusses the hydraulic criteria used, and the hydraulic design of the Surface Bypass Spillway. Skeleton Bay Units 19 and 20 at John Day Dam will be converted into six surface bypass chutes for passing smolts. The six chutes will operate independently, with each respective chute controlled by a tainter gate that will be either fully opened or fully closed under normal conditions. There will be two different chute exit elevations, so that the chute discharge performance can be optimized for dissolved gas criteria at all tailwater elevations. The existing adult collection channel will be shortened to Unit 16 and the adult fishway entrances at Units 19 and 20 will be closed down and replaced at Unit 16. The amount of flow that the SBC's discharge will be high enough to affect overall tailrace conditions significantly. A strict project operation plan for running the chutes to maintain acceptable tailrace conditions will be required and is contained in Section 3.11.

3.2 Existing Project

The existing project consists of sixteen fully operational turbine units (1-16) and four skeleton bay units (17-20). There are twenty spillway bays numbered 1-20 from north to south. The spillway and powerhouse are adjacent to each other and form a straight line perpendicular to the axis of the Columbia River. Each powerhouse intake bay is 6.4-m (21-ft) wide and draws from elevation 37.19 to 61.57-m (122 to 202-ft), msl. There are three adult fishway entrances at the downstream side of skeleton Unit 20 and ten floating orifice gates along the fishway; one each at Units 19, 18, 17, 15, 12, 9, 6, 3, 2, and 1. The tailrace is approximately 15-m (50-ft) deep on the downstream side of the powerhouse with a 30-m (100-ft) deep draft tube.

3.3 Design Criteria

The criteria used for the hydraulic design of the Surface Bypass Spillway are based on Corps of Engineers EM-1110-2-1603 (Hydraulic Design of Spillways), EM-1110-2-1602 (Hydraulic Design of Reservoir Outlet Works), Waterways Experiment Station (WES) model reports, gas abatement reports, and National Marine Fisheries Service (NMFS) Fish Passage Design Criteria.

3.4 Surface Bypass Chute Crest

The SBC crest is a 6.0-m long, 6.4-m (21-ft) wide broad crested weir at elevation 73.9-m.

The length of the weir crest in the initial stages of this design was 12.192-m (40-ft). In an effort to decrease the exit velocity on the downstream end of the chute, decreasing the crest length was investigated. Originally, it was thought that the tainter gate position might limit the extent at which the crest length could be reduced. After further study, increasing the height of the gate and having the gate seal placed on the sloped portion of the spillway was considered acceptable and could be incorporated into the design.

With that known, there was no reason a regular sharp-crested weir could not be designed by eliminating the crest length altogether and having the gate seal moved upstream 6-m from the construction base line (CBL). This layout was acceptable, and the slope was much shallower at about 0.55337. The drawback to this geometry is that the sharp crested weir design is much more hydraulically efficient than the broad crested design. That is, the discharge coefficient value for the sharp crested weir was approximately 4.0 and the broad crested discharge coefficient value was approximately 3.0. This corresponded to an increase in Q for the sharp crested weir of 56 cms or approximately 2000 cfs. With this increase in discharge, and with the shallower slope of .55337, the chute exit velocity remained about the same as the original design (about 25-m/s, or low 80's ft/s). A broad-crested weir was therefore selected as the preferred design due to the slightly lower exit velocities: This geometry will allow easier access to the gate and will allow the gate to rotate out from under the spillway deck. In addition, the broad crest is believed to reduce juvenile rejection due to the longer transition length so the minimum crest length for a broad crested weir was investigated. The next step then was to find out how much the crest length could be reduced while still retaining the characteristics of a broad crested weir. According to King, in Handbook of Hydraulics, page 5-24, "When the head reaches one to two times the breadth (length), the nappe becomes detached and the weir becomes essentially sharp-crested." Also, from EM 1603, the crest length should be designed to 1 to 2 times the design head or 6.55-m to 13.10-m. Since the head fluctuates from 6.10 to 6.55-m under normal conditions, and the slope of the chute needs to be minimized, the horizontal portion of the crest length will be 6-m. This geometry produces a slope of .60050 and an exit velocity around 24.156-m/s (79.25-ft/s) at chute exit elevation 47.9-m (157-ft). During high flow events, the head on the weir could increase to as much as 10.23-m. Although this is not two-times the crest length, some or all of the broad-crested weir characteristics could be lost during such an event, creating more of a sharp-crested weir (additional model studies are required to determine the characteristics of the weir during high flow events). However, during rare extreme high flow events, fisheries issues are not primary design concerns. Figure 3.1 shows the projected water surface profiles over the chute for normal and spillway design flood pools.

The crest elevation was chosen to provide approximately a 6.4-m (20-ft) deep surface attraction for smolts as they enter the chutes. Fisheries biologists considered this depth optimal for surface bypass from past surface bypass studies. Since each chute will be operated free-flowing (no gate control) during normal conditions, the crest is the hydraulic control. The Surface Bypass Model at WES was used to determine the discharge in a single SBC and a coefficient of discharge (Cd) was back calculated to be 3.00. At Spillway Design Flood (Pool El. 84.12-m), one chute discharges 347.05-m3/s. At Normal Full Pool (Pool El. 80.47-m), one chute discharges 178.40-m3/s. Figures 3.2 shows the rating curves for one, two, and three chutes, respectively.

The amount of flow, determined by the crest and forebay elevation, defines the hydraulic zone of influence in the forebay. This is important as it helps to predict the reaction of smolts to the SBC. Forebay velocity plots generated on the John Day General model at WES are shown in Figures 3.3 and 3.4. Radio tracking of smolts in the John Day forebay indicate that they mill back and forth about 90-m upstream of the dam, potentially searching for a good surface outlet.

Three chutes operating simultaneously discharges 535.2-m3/s at pool elevation 80.47-m (264-ft) and creates a zone of influence that reaches over 90-m upstream.

On the upstream side of the crest, the entrance is bounded by elliptical curves on the floor and the intermediate piers that were designed in accordance with EM 1110-2-1603. The floor curve is a 1.010 by 1.845-m ellipse and the two intermediate pier curves are 1.067 by 1.845-m ellipses. The main and north and south abutment piers have a circular curve with a radius of 1.981-m. A circular curve was chosen for the main pier over an elliptical curve in an effort to reduce the distance it extended into the forebay. With circular piers, a smaller bulkhead can be used during construction, realizing a significant cost savings. An ogee curve of y = x1.85 /9.730 connects the downstream side of the crest to the chute slope.

An 8.6-m high radial gate controls the SBC discharge. The radius of the gate is 8.2-m and its gate seal is located on the ogee curve approximately 1.6-m downstream from the construction baseline. The trunnion elevation is 76.0-m. The radial gate will operate in either the fully opened or fully closed position. However, if gas levels downstream of the Surface Bypass Spillway increase to an unacceptable level, throttling of the gate may be used as an alternative method to decrease gas levels. Figure 3.5 shows the rating curve for partially opened gates. An emergency bulkhead slot is located about 5.2-m upstream of the gate seal. The slot is 0.580-m wide and will hold bulkheads for emergency shutdowns and routine maintenance.

3.5 Surface Bypass Chute Exit

The Surface Bypass Spillway is designed to attract smolts successfully and deliver them to a safe downstream environment. The design of the chute exit is critical in providing overall acceptable fish conditions.

a. Total Dissolved Gas

Total dissolved gas (TDG) supersaturation results when spillway discharge and entrained air plunge to depth in the stilling basin. Different chute exit elevations were evaluated in physical hydraulic models at WES in order to find the optimal design for minimizing TDG supersaturation. In addition, the chute exit length and the unit discharge were evaluated to determine the optimal gas production conditions. The chute has a unit flow of 27.87-m3/s/m at Normal Full Pool, which provides a stable, horizontal discharge out of the chute. The degree of submergence of the chute exit and its length determines whether the exit flow is a plunging, skimming, undular, or hydraulic jump condition. Figure 3.6 and 3.7 show the gas performance curves for both exit elevations that were developed on the Surface Bypass Model at WES. Skimming flow is considered the ideal discharge condition and this occurs at .3 to 1.5-m (1 to 5-ft) submergence at normal pool elevations. At 1.5 to 2.1-m (5 to 7-ft) a shallow undular discharge occurs and is considered acceptable. All other operational conditions are undesirable and may produce excessive dissolved gas. Refer to Section 3.11 for ramifications of operation outside recommended ranges.

For the purpose of this FDM, acceptable gas levels have been defined as the total dissolved gas pressure in the tailrace less than or equal to 120% of atmospheric pressure. The Oregon State Department of Environmental Quality and Washington State Department of Ecology have issued temporary wavers to allow for an increase in Total Dissolved Gas to 120%. Chutes 20a, b, and c have an exit elevation of 47.9-m with a length of 6.250-m and can produce acceptable gas levels at tailwater elevations from 48.2 to 50.0-m. Chutes 19a, b, and c have an exit elevation of 48.8-m and a length of 7.756-m and can produce acceptable gas levels at tailwater elevations from 49.1 to 50.9-m. From tailwater elevations of 49.1 to 50.0-m, all six chutes can be operated and still produce acceptable gas levels with 0.3 to 2.1-m. of submergence. This allows the 2-unit surface bypass spillway to operate under dissolved gas criteria over a range of tailwaters from 48.2 to 50.9-m, at normal pool elevation. Figure 3.8 shows a summary of 30 years of John Day tailwater events. The average tailwater elevation is well within the Surface Bypass Spillway operational range with a maximum of about 50-m and a minimum of about 48.7-m. As shown in the figure, there is a 12.86% chance that the tailwater elevation will go outside the ideal range of 48.2 to 50.3-m and a 4.84% chance that the tailwater will go outside the acceptable range of 48.2 to 50.9-m once during a given fish season (April 1 - August 31). The hundred-year high tailwater and the hundred-year low tailwater during the fish season are 53.7-m and 47.2-m, respectively. Figure 3.9 shows the range at which each chute can be operated.

When the submergence criteria is met, the surface bypass spillway can be expected to produce dissolved gas levels of 117 to 121% under normal pool elevations (Wilhelms, U.S. Army Corps of Engineers Waterways Experiment Station) (see Figure 3.10). When the submergence goes outside the criteria of 0.3 to 2.1-m submergence, the gas levels can be expected to rise significantly and the surface bypass spillway will produce gas similar to a deflectored spillway. The actual percentage of gas supersaturation during these conditions is unknown, but can be expected to surpass 130%. Additional model tests would be required to produce actual percentages.

b. Chute Exit Piers

Rounded piers at the chute exit were developed to reduce shearing zones that occur where high velocity flow encounters slower or stagnant water as recommended by fisheries biologists and EM 1110-2-1603. The rounded curves for the intermediate piers are 6.250 by 1.067-m ellipses. The main piers have 6.250 by 1.981-m ellipses.

c. Hydraulic Jump

When the tailwater elevation increases above elevation 51.82-m (170-ft), a hydraulic jump will occur inside the piers of the surface bypass chute. When this happens, the tailrace bridge designed in this FDM may be impacted, damaged, or destroyed. A test was performed using the John Day Surface Bypass Sectional model at WES to determine the river flow and tailwater elevation where the integrity of the tailrace bridge is compromised. To reduce the complexity of the test, a constant pool elevation of 84.12-m (276-ft) and chute flow of 347 m3/s (12256-cfs) was used. At a river flow of about 19,820 m3/s (700 kcfs) and a tailwater elevation of 52.73-m (173-ft) a hydraulic jump is formed within the piers, but remains under the bridge without directly impacting it. However, at this point, the inconsistent water surface under the bridge becomes significant and could cause damage from excessive spray. At a river flow of 23,360 m3/s (825 kcfs) and a tailwater elevation of 53.64-m (176-ft), the bridge is fully impacted by the hydraulic jump. To alleviate this problem, the surface bypass units will be shut down and bulkheaded off when the river flow reaches 19,820 m3/s (700-kcfs) with a tailwater elevation of 52.7-m. Stoplogs stored under the deck will be installed in the bulkhead slots when the pool elevation surpasses the elevation of the top of the tainter gate in the closed position. These tests serve only as a guideline for operational procedures. Upon completion of the project, strict observation of the surface bypass chutes is recommended under high flow conditions to prevent damage to the bridge. An event that could impact the bridge is unlikely as can be seen in Figure 3.8. There is a 4.0% chance that the tailwater could reach the potentially damaging elevation of 52.7-m once in any given year and only a 1.3% chance of the tailwater reaching an elevation that would force a hydraulic jump to directly impact the bridge.

3.6 Surface Bypass Chute

The chute slope connects the downstream side of the ogee curve with the chute exit. It has a slope of 0.59974 and transitions into the exit with a 15.0-m radius. See Plates 7 and 8 for the geometry of each chute. In Chutes 20a, 20c, and 19b, the chute slope length is 6.250-m. Chutes 20b, 19c and 19a have a chute slope length of 7.751-m. The force exerted on the curved portion of the chute due to the change in momentum of the maximum flow, the hydrostatic pressure and the weight of the water is equal to 5743 kN.

3.7 Water Surface Profiles

Because the broad-crested weir in each chute essentially acts as a free overfall, it is assumed that critical depth occurs a distance upstream of the brink approximately equal to 1.4 times the critical depth (Chow, page 44). However, because the crest is relatively short in length, some of the overfall characteristics are lost at the high flow condition. As a result, the critical depth at high flow is located approximately 0.5 times the critical depth, upstream of the brink (Figure 3.1). Balancing the energy, a depth is determined at the brink of the crest and on the downstream side of the ogee curve. Energy loss computations were used to develop the water surface profiles down the chute. The water surface profiles were computed for Spillway Design Flood, Normal Full Pool, and Maximum Control Pool. To determine the velocities along the chute, a Manning's n value of 0.012 (k = .0006-m, .0020-ft) was used. For determining wall heights, a Manning's n value of 0.015 (k = .0021-m, .0069-ft) is used. These values are recommended for concrete spillways in EM 1110-2-1603. The actual water surface profile will be somewhere between the two conceptual profiles.

Pier end waves in the chute were observed in the model. These standing waves are created when flow around the upstream piers pulls away from the walls. The model indicated that the waves are not substantial and do not warrant further streamlining of the pier noses.

The water surface profile data is listed in Table 3.1.

3.8 Sidewalls

The sidewalls were designed using the Spillway Design Flood water surface profile. The minimum sidewall height was determined using the following equation from EM 1110-2-1603: Freeboard = 2.0+0.025*V*d1/3 (ft) or 0.6096+0.037*V*d1/3 (m). This equation accounts for cross-waves, roll waves, air entrainment, and bulking.

3.9 Tailrace Conditions

a. Tailrace Patterns

When operating, the Surface Bypass Spillway has a tremendous influence on tailrace patterns. For this reason, it is important to balance the entire project outflow so that the Surface Bypass Spillway outfall flows straight downstream without eddying back upstream. Downstream migrants are expected to be fully entrained in the outfall flow. It is necessary for the outfall to move downstream without eddying back to reduce any downstream migrant delay. The general model at WES indicated that a relatively narrow range of powerhouse discharge and spillway discharge is needed for each river flow to achieve an acceptable Surface Bypass Spillway outfall condition. Optimal flows are shown in Figure 3.11 with the spillway operating under the spill pattern developed for John Day Flow Deflectors.

b. Adult Entrances

In constructing the Surface Bypass Spillway, the entrance to the adult fishway and the fishway itself between Units 19 and 20 will be removed. The adult fishway will be modified to end at Unit 16 with the adult entrances located there. The three floating orifice gates at Units 19, 18, and 17 will be removed. The existing diffusers located at Unit 20 and used to supply 5.66-m3/s (200 cfs) of attraction flow for the existing adult entrances are larger than the rest of the diffusers located in Units 1-19 which supply 1.70-m3/s (60 cfs) of flow to the floating orifice gates (see Figure 3.12). For this reason additional unused diffusers located upstream in the collection channel will be opened up in addition to the diffusers at Unit 16 to supply the new adult entrances with the required attraction flow of 5.66-m3/s (200 cfs). The two diffusers located in Unit 16 will supply 120 cfs of flow and one or two additional diffusers upstream of Unit 16 will supply another 1.70 or 3.40-m3/s (60 or 120 cfs) for a total of either 5.10 or 6.80-m3/s (180 or 240 cfs). The diffusers chosen to augment the diffusers at Unit 16 should be located as close to Unit 16 as possible to minimize any velocity reduction in the fishway. The augmenting diffusers will be selected in a trial and error procedure after the completion of the surface bypass spillway construction. If acceptable biological conditions cannot be met using the prescribed method above, the diffusers at Unit 16 will be modified to match the diffusers in Unit 20 and provide 5.66-m3/s (200 cfs) of flow. This can be done by cutting concrete to increase the water supply orifice from a 0.91-m by 1.22-m (3-ft by 4-ft) opening to a 1.22-m by 1.52-m (4-ft by 5-ft) opening and by increasing the width of the gratings from 4.42-m (14.5-ft) to 5.72-m (18.75-ft) (see Figure 3.12).

The existing draft tubes at Units 19 and 20 produce a void underneath the Surface Bypass Spillway outfall. Filling these draft tubes has been investigated and tested in the general model. Presumably, because of the extreme depth at this location, neither condition (backfilled or not backfilled) appears to have a significant effect on tailrace conditions. For this reason, the draft tubes at 19 and 20 will not be backfilled to the elevation of the tailrace floor.

3.10 Downstream Training Wall

The 76.2-m guide wall originally designed between Chute 20a and the non-overflow section between the powerhouse and spillway was removed from the design at the 60% level of this FDM. The guide wall was included in an effort to reduce gas in the tailrace and to serve as a barrier to separate the spillway discharge from the Surface Bypass Spillway discharge. By varying the powerhouse and spillway discharge ratio, the optimal tailrace conditions can be maintained without the need for a training wall. With the training wall in place, more powerhouse flow is needed to produce acceptable tailrace conditions. Without the wall, the spillway flow will have to increase to produce acceptable tailrace conditions. Given the estimated cost for constructing a 76-m training wall was approximately $18 million, significant cost savings could be realized by eliminating the training wall as part of the surface bypass design.

3.11 Operation

A strict operational plan must be adhered to when operating the Surface Bypass Spillway in order to maintain acceptable tailrace conditions for downstream migrants. As the total river flow increases, the amount of discharge released from the powerhouse must increase relative to the spillway discharge. If the powerhouse discharge is too high, then a large circular eddy is formed downstream of the spillway, resulting in a large percentage of the surface bypass discharge flowing back into the stilling basin. If the spillway discharge is too high, then a large circular eddy is formed downstream of the powerhouse. This not only causes the surface bypass discharge to eddy back on the powerhouse side, but it also disrupts the juvenile bypass outfall. The relationship between the powerhouse discharge and the spillway discharge was determined for normal pool in the general model at WES and is indicated in the Project Operation chart in Figure 3.11.

In addition to maintaining an acceptable project discharge balance, the correct Surface Bypass Chutes must be operated at a given tailwater elevation to provide acceptable gas saturation characteristics. Chutes will be operated under most scenarios by either opening up Chutes 20a, b, and c, or Chutes 19a, b, and c. The first set of chutes has an exit elevation of 47.9-m and will be operated from tailwater elevations of 48.2 to 50.0-m (158.14 to 164.04-ft). The second set of chutes has an exit elevation of 48.8-m and will be operated from tailwater elevations of 49.1 to 50.9-m (161.09 to 166.99-ft). There is an overlap of tailwater elevations from 49.1 to 50.0-m where both sets (all six chutes) can be operated. Outside of the tailwater range of 48.2 to 50.9-m, the surface bypass spillways can be operated but TDG and biological criteria may be compromised under these conditions. Above tailwater 52.7-m, the integrity of the structure itself will be compromised. Results of operating the Surface Bypass Chutes outside of the recommended tailwater ranges could result in the following:

1. Total Dissolved Gas levels above 120%
2. Excessive exit velocities (more than at normal conditions).
3. Non-skimming exit flow.
4. Shearing at exit pier noses.
5. Unacceptable tailrace patterns.
6. Tailrace bridge impacted by chute flow when tailwater exceeds 52.7-m.

3.12 Model Studies

Model studies at WES were performed on two models: a 1:55 sectional model and a 1:80 general model. The sectional model consists of 5 ½ surface bypass units or 16 ½ bays and is situated in a flume that previously contained The Dalles Spillway sectional model. The sectional model was used to determine the rating curve for a single surface bypass chute and the gas abatement performance. The gas performance curve (Figures 3.6 & 3.7) was developed using a single bay and increasing the TW to simulate different exit chute elevations or submergences. By adjusting the submergence, an ideal exit chute elevation could be determined for a given tailwater elevation.

The general model was used to determine the zones of influence for different combinations of bays, and to adjust the powerhouse and spillway discharges to enhance the surface bypass outfall characteristics. In addition, the general model was used to analyze the following Surface Bypass Spillway and project components.

Training Wall.
Draft Tube Cover.
Elliptical pier noses and ends.
Adult entrances.
Juvenile Bypass Outfall.

For each trip to WES where the Surface Bypass Spillway was tested, a trip report was written. These trip reports are included in Appendix D.

3.13 Single Unit Prototype Surface Bypass Spillway-Hydraulic Considerations

As a prototype, the Surface Bypass Spillway will be built as a single unit at Unit 20 with only one exit elevation of 47.9-m. The only difference between this and the full system will be when operating at high tailwaters (high flows). Under the full system, when the tailwater reaches 49.1-m, the chutes in Unit 19 operate and when the tailwater reaches 50.0-m, the chutes in Unit 20 shut off. With the prototype, there will be no chutes in Unit 19 and therefore at tailwaters greater than 50.0-m, the chutes cannot operate and meet dissolved gas criteria.

All design aspects of the prototype system will be the same as the full system-except there will only be three chutes at elevation 47.9-m.

SECTION 4.0 STRUCTURAL DESIGN

4.1 General

All designs will be in accordance with accepted engineering practice. Structural design conforms to applicable Engineering Manuals (EM's). The design criteria is included in Appendix A. The calculations for the stability analysis (herein summarized) are in Appendix E. The other structural calculations are bound separately.

4.2 Foundations

a. General

No new explorations were performed for design of the surface bypass spillway feature. Foundation design assumptions for this report are based on research of existing reports and general references on engineering properties of rock.

b. Foundation Design Values

Existing information in DM 15.4 indicates that design bearing pressures for the generator bays are about 69 kips/square foot for the without-earthquake case and about 83 kips/square foot for the with-earthquake case. Since the powerhouse structure has operated satisfactorily for many years, these values can be assumed as minimum values for the allowable bearing pressures for design of this feature. Because foundation design values from original construction of the dam were unavailable, shear strength assumptions for the sliding stability analysis are based on engineering judgment, knowledge of the type and condition of the existing foundation rock, and typical values determined for other projects with similar foundation conditions. The relatively low assumed values for this foundation as compared to what might be expected for a basalt foundation are based mainly on the highly fractured nature of much of the foundation rock, which is fairly typical of the basalt flow rock in the vicinity of John Day Dam. The values that the FDM design is based on are summarized in Table 4.1 below:

Table 4.1
Foundation Design Values

Type	Value
Allowable bearing strength	3.97 megapascals (83 kips/sq ft)
Coefficient of friction for sliding	tan 50 deg.
Cohesion (structure/ foundation interface)	689 kilopascals (100 psi)
Possible rock failure planes	None identified

Confirmation of these assumed values should be performed prior to proceeding with design for plans and specifications.

4.3 Stability Analysis

4.4 Spillway

The spillway slab is supported in part by the existing powerhouse walls, and by new walls added in the Skeleton Unit cavity. Refer to Plates 6 through 10. The slab is considered to span in a east-west direction across the existing walls at the stop log slots, storage gallery, and at the tailrace. The slab will span in a north-south direction across the new walls in the Skeleton Unit cavity.

The guide walls vary in height to provide the necessary freeboard. Refer to Plates 7 and 8. The required heights and water depths are given in Section 3.

The spillway slab thickness is 1.22 m. The clear span between the new walls is 6.401 m. The center-to-center distance between walls is too conservative for a design span (due to the thick walls), therefore, a more reasonable design span of 7.01 m was chosen. The full slab thickness was used for determining slab weight, but slab stresses and reinforcement were checked considering a 152 mm erosion over the life of the project and thus designed using a 1.068 m thickness. The top layer of reinforcement was designed considering 152mm clearance. The bottom layer has 76 mm clearance since it is not subject to erosion.

The hydraulic analysis showed the water depth varying along the spillway from about 4.236 m (13.9 feet) at the top to 2.035 m (6.68 feet) at the bottom. A dynamic overpressure was indicated at the bottom of the spillway due to the momentum change as the water is redirected in a horizontal direction. The slab was designed for the maximum water of 4.236 m (13.9 feet). The dynamic overpressure at the bottom of spillway will be considered in the final design, but is not felt it will exceed the difference between maximum and minimum water depths. Vibration was not considered in this report.

The concrete strength for the spillway slab is increased to 41.368 MPa (6000 psi) to improve the durability of the concrete, and its resistance to erosion.

4.5 Bridges

a. Spillway Bridge

The bridge spans across each chute and is supported on each chute guide wall. The bridge permits access to the radial gate hoisting machinery, supports the spillway gantry crane, and provides vehicular access from the powerhouse to the spillway. The spillway gallery is one of the primary structural components of the spillway bridge. The deck has two removable sections. One to allow access to the stoplogs, and the other an equipment/man access hatch. Plate 11 shows the cross-sectional geometry. The beam nearest the radial gate was minimized in depth to allow the spillway and gate to be positioned as far upstream as possible. Continuity of the existing service gallery is maintained by the layout shown in Plate 13.

Effective span of the beams for design purposes is 7.315 m rather than the length center-to-center of the supports because the piers constitute a wide support, and to use centerlines would be too conservative.

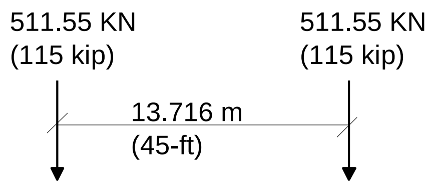
Bridge fixity is assumed to be the same as that in the original design (Design Memorandum No. 16), using fixed at one end and simply supported at the other. This is reasonable over the alternative of continuity over the supports because of the high ratio of stiffness of the pier to the bridge. Each beam is designed as a rectangular beam. Box beam action for the service gallery is considered.

Deflection for the beams supporting the gantry crane is limited to span divided by 1000.

Design loads are assumed to be as follows:

(1) dead load - weight of the structure.

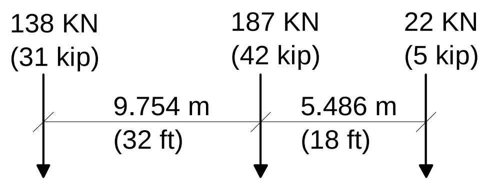
(2) Gantry crane is a 444 KN (100 kip) with a stall-out capacity of 778 KN (175 kip). The crane wheels on each rail are 13.716 m (45 feet) apart. The wheel loads for each beam carrying the crane are assumed as shown on the following diagram. The values shown are the sum of dead plus live loads.



live load impact = 25%

(3) Truck loading according to AASHTO specifications for HS-20 loadings.

(4) As an alternative to the AASHTO truck, a load consisting of one intake stoplog on a flat-bed tractor-trailer is considered as per design memorandum No. 16 as shown below.



live load impact = 0%

(5) Wind load = 1.44 KPa (30 psf)

(6) Seismic = 10% g

To stay within H-20 loading parameters mobile cranes are restricted from operating from the tailrace bridge. However, they can use the bridge to access the non-overflow section.

b. Tailrace Bridge

The purpose of the bridge at the tailrace is to access the two unwatering pump stations in the non-overflow section just north of Skeleton Unit 20. The bridge width is 6.10 m (out-to-out of curbs) and has a low speed guard rail each side to match the existing guardrail on the tailrace deck. The elevation of the deck matches the existing tailrace deck at elevation 56.388m (185 ft). Refer to Plate 11.

The design loads are as follows:

(1) Dead load - weight of the structure.

(2) Truck loading accoarding to AASHTO specifications for HS-20 loadings.

(3) Truck live load impact = 30%.

(4) Wind load = 1.44 KPa (30 psf).

(5) Seismic = 10%g

To stay within H-20 loading parameters mobile cranes are restricted from operating from the tailrace bridge. However, they can use the bridge to access the non-overflow section.

4.6 Tainter Gate Trunnion Supports

The spillway is controlled by six tainter gates, each 6.4 m wide by 8.6 m high. This provides for 0.114 m freeboard over the maximum operating reservoir level elevation of 81.686 m with the top of the gate at elevation 81.8 m. Refer to Plate 15A. The radius of curvature of the gate is 8.2 m measured from the centerline of the trunnions to the downstream face of the skin plate.

There were two design cases considered:

(1) Closed gate with water at elevation 81.686 m.

(2) Closed gate with water at elevation 81.686 m and an ice load of 146 KN/m (10 kip/foot) applied at water surface

At final design other positions of a raised gate should be considered.

The maximum load at each trunnion will be 1660 KN. Two trunnions are located in the guidewalls between chutes. Therefore, the trunnion assembly in these piers will resist 3320 KN. The trunnion assembly will use post-tensioning rods embedded into the concrete pier. Eight to ten 31.25 mm diameter rods will be required.

4.7 Spillway Maintenance Stoplogs

Maintenance stoplogs are placed upstream of the tainter gates to allow maintenance of the gates. They will be placed in guide slots in the spillway chute walls. They are designed to restrain water at the maximum pool elevation 84.125 m (276 ft). See Plates 14 and 15B.

A set of three stoplogs will be fabricated. The three sections, each 3.45 m high, will be stacked on top of each other in the stoplog slot to dewater the tainter gate. They consist of a skin plate and horizontal structural steel beam connected to a vertical steel framing member. Each stoplog will weight approximately 9,900 kg. The three stoplogs will be identical and are designed for the maximum head. They can be placed at any level and are interchangable. See Plate 14. Eighteen stoplogs will be made. During a flood event the tainter gates will have to be closed to prevent damage to the tailrace bridge. During extreme events the upstream water level will overtop the tainter gates. Therefore, the stoplogs will be installed to prevent overtopping of the tainter gates.

The three stoplogs for each chute will be stored in slots below the spillway bridge. One will be hung in the stoplog slot by means of a dog or a hinged clip. See Plate 15B. The other two will be stored in special storage slots under a removeable deck panel immediately upstream of the gallery.

4.8 Modifications to Existing Structures

a. Powerhouse

The north wall of the powerhouse and the south wall of Chute 19c in Skeleton Unit 19 will be a common wall. Refer to plan on Plate 11 and to the cross section on Plate 12.

The new wall will extend up to elevation 77.114 m to match the existing wall and will serve as the closure wall to the powerhouse.

The wall will resist the maximum tailwater flood elevation of 62.575 m (205.3 ft). The spillway slab varies in height along the wall, but will provide lateral support. The maximum tailwater varies in height above the spillway and has a maximum depth of 12.5 m at the west end of the wall.

The roof of the powerhouse will require demolition to the limits of the northern most roof girder in Unit 17. There is a distance of approximately 1 meter between the last girder and the new wall which will require new roof beams and roofing system to be replaced. If this distance is too small for construction reasons then the roof will have to be demolished to the next girder. See Plates 11 and 12.

b. Unit 16 Auxiliary Water System

The amount of auxiliary water introduced at Unit 20 is greater than at the other units because the main entrances are located in Unit 20. The new fishway entrance at the north end of the powerhouse will be relocated to Unit 16. To provide sufficient flows at the new fishway entrances it is necessary to increase the capacity of the auxiliary water system at Unit 16. Two modifications will be made: increase the size of the auxiliary water gate and increase the size of the floor diffusion opening.

The Unit 16 auxiliary water gate is .914 m wide by 1.210 m high (3 feet by 4 feet). It will be expanded to an opening 1.219 m wide by 1.524 m high (4 feet by 5 feet) at the same location. See Plates 4 and 10. A larger opening will be cut and new reinforcing bars added around the periphery of the opening to replace the bars removed. This will retain the same wall strength at the opening. A new four by five foot gate will be installed.

The auxiliary water opening in the floor of the fish collection channel will be widened. This involves removing four portions of the floor as shown on Plate 10. About six additional 305 mm (12 inches) by 305 mm (12 inches) precast concrete beams will be placed at elevation 44.501 m (146.0 feet) in the openings left by partial removal of the floor. Additional floor grating at elevation 45.72 m (150.0 feet) will also be installed in place of the removed floor sections.

4.9 Gallery Plugs

Concrete plugs will be required to seal off the 150 and 168 galleries from the Surface Bypass Spillway. They are designed for the maximum flood tailwater elevation of 62.575m (205.3 feet).

The plugs are designed as a two-way plate and will have a 0.91m thickness. The plug will be connected in each gallery by dowels drilled and epoxy grouted on all four sides. A waterstop will be installed between the plug and existing concrete by sawcutting a groove in the existing concrete and then epoxy grouting the waterstop in the groove.

The 150 Gallery will require one plug at the dividing wall between Units 18 and 19 and extend 0.91m from the construction joint into Unit 18. See Plate 6. The 168 Gallery will require a plug at the entrance to the unwatering pumps. The plug will extend from the construction joint into the non-overflow section. Another plug in the 168 Gallery will be located in the north end of Unit 18 adjacent to the construction joint. Plugs are not required on the Surface Bypass Spillway side of the construction joints since these walls will be demolished and replaced with chute guide walls.

4.10 Dewatering Bulkheads

The upstream dewatering bulkhead is a buoyant bulkhead floated into place against fixed steel piers and a fixed floor section to achieve a water tight seal. The steel piers and floor section will weigh approximately 70,000 kg and will be bolted to the face of the powerhouse. The floating bulkhead will be 16.155 meters long and about 12 meters high and will weigh about 240,000 kg. See Plates 2 and 3A. The floating bulkhead and fixed members will be fabricated from steel plate with structural support provided by wide flanges and channel sections. The floating bulkhead will be supported internally by horizontal trusses. It will contain drain valves and connections for pumps to achieve the correct submergence. An alternative to the floating bulkhead is a fixed bulkhead with or without temporary bracing. *Further analysis during final design will indicate which method is best, or the design can be left to the contractor.*

The downstream bulkhead will be fabricated from steel plate and structural members similar to the upstream bulkhead. The downstream bulkhead will be installed across the entire work area, Units 19 and 20. During bulkhead installation, knee braces will be installed at elevation 44.0 meters by bolts through the columns located on the tailbrace wall between the openings to the fish collection channel. See Plates 2 and 3A. The bulkhead floor panels will then be bolted to the knee braces spanning the 6.4 meters between them. The bulkhead walls will then be bolted to the floor and the north and south end panels bolted to the columns on the non overflow section and north end of Unit 18. The temporary bulkhead bracing members (406 mm diameter pipes) will then be connected to the top of the bulkhead wall and powerhouse. The water will then be pumped out from between the tailrace wall and bulkhead. The hydrostatic load will be borne by the temporary bulkhead bracing at the top and the bulkhead flood and knee braces at the bottom. The load at the bottom will be carried to the powerhouse by the existing beams (five per unit) located between elevation 45.7 meters (150.0 feet) and 44.5 meters (146.0 feet). The total weight of the dewatering bulkhead across both units will be about 340,000 kg.

The new transverse bulkhead to be installed in the fish collection channel will be fabricated from steel plate and structural members. It will be installed in the existing slots between Units 16 and 17 and will form the north end of the fish collection channel. See Plate 4. It will be about 5.8 meters wide and 7.6 meters high and will weigh about 19,000 kg.

SECTION 5.0 MECHANICAL DESIGN

5.1 Operating Gates

Several operating gate alternatives were considered for this project and they are presented in Appendix B, GATE ALTERNATIVE STUDY. Because of the relatively high hydraulic head requirement on the gate, the number of viable options was limited. Based on the results of this study, it is recommended that Tainter gates be provided for the surface bypass spillway. One gate, 8.6 meters high by 6.4 meters wide, will be provided for each chute.

The principal elements of the Tainter gate are the skin plate assembly and supporting structure, the end frames, and the trunnions and anchorage (Plate 15A). The skin plate is rolled to form a segment of a circle and is stiffened and supported by curved vertical beams and horizontal girders. The end frames carry the reactions from the horizontal girders to the trunnions and concrete anchorage. The trunnions will be furnished with self-lubricating bearings, consisting of aluminum bronze backing and a wear surface consisting of a homogenous composite material containing PTFE.

The gate will be operated by a 5 kW motor-driven overhead cable hoist located on top of the gate's concrete end piers. The hoist will consist of an enclosed gear reduction assembly which will drive a line shaft with a cable drum located at each end of the gate. The gate lifting cables will be attached to both ends of the gate, near the bottom.

The motor will be equipped with a spring-set, magnetically released, brake to lock the gate in position when the motor is off. The gate raising and lowering speed will be 0.3 meters per minute.

Each gate is estimated to weigh about 21,400 kg, including the cable hoist operator.

The gate side and bottom surfaces will be provided with rubber seals to minimize flow leakage. The seals will be in contact with steel guide plates imbedded in the concrete structure. An electric heating cable system will also be imbedded in the concrete, adjacent to the side and bottom seals, to prevent the seals from freezing to the steel plates. The heating cable system will consist of a self-regulating heating cable installed in a stainless steel conduit. The conduit will be filled with a water-antifreeze solution. The space between the conduit and the steel seal plates will be filled with a waterproof heat-conducting cement.

5.2 Maintenance Stoplogs

Each operating gate will be provided with a set of fabricated steel stoplogs to permit dewatering of the gate for maintenance.   During very high river flows the hydraulic jump at the bottom of the spillway will impinge on the tailrace bridge. See Section 3. At a flow of approximately 19,800 m3/sec the gates will be closed and the stoplogs inserted to prevent over topping of the tainter gates. See Plate 15B. The stoplogs will be installed in guide slots located upstream of the operating gate, and can be installed and removed by use of the existing spillway gantry crane. The set of stoplogs will consist of three 3.45 meter high sections. When not in use one section can be dogged off at the top of the guide slot and the other two sections can be stored in special slots to be provided just downstream of the stoplog guide slots. Each stoplog section is estimated to weigh about 9,900 kg. A special stoplog lifting beam will be provided to accommodate the gantry crane. The stoplogs will be approximately 482 mm thick and will travel in 580 mm wide guide slots. The downstream side of the stoplog slots will be streamlined to minimize the impact on passing fish.

Using the spillway gantry crane rather than one of the powerhouse intake cranes is preferred because the spillway crane is lighter in weight and thus will impose less structural loading on the new spillway bridge.

5.3 Modifications to Existing Powerhouse Systems

a. Spillway Gantry Crane

The existing spillway gantry crane rails and supporting concrete structure will be extended to the south to permit the crane to serve the new maintenance stoplogs at Units 19 and 20 (Plate 11). The crane will be provided with a separate stoplog lifting beam assembly which will be attached underneath the lifting beam serving the existing spillway gates.

b. Mechanical Piping

The existing 38 mm (1½-inch) intake deck and gate washing potable water piping at Unit 19 will be suitably capped and removed in conjunction with the concrete demolition work. The existing 75 mm (3-inch) drain piping serving the intake deck and 265 Gallery and the existing 75 mm (3-inch) and 150 mm (6-inch) drainage piping serving the tailrace deck and 168 gallery, at Units 19 and 20, will also be suitably capped and removed in conjunction with the concrete demolition work.

The 265 Gallery contains a 200 mm (8-inch) potable water line and a 75 mm (3-inch) 690 kPa (100 psi) compressed air line. Both of these lines serve the entire project. Prior to removal of the 265 Gallery through Units 19 and 20, both lines will be relocated through the new spillway bridge at Units 19 and 20 (Plate 20).

The powerhouse drainage and unwatering galleries, which pass through Units 19 and 20, are at a low enough elevation that they will be unaffected by the new construction.

5.4 Fishway Entrance Gates

Fishway entrances to the fish collection channel are located at various locations on the downstream side of the powerhouse. The channel leads to the south shore fish ladder. Fishway entrances consist of four main entrances and secondary floating orifice entrances. The four main entrances are at Unit 1 and Unit 20 and consist of weir gates operated by drum hoists. The secondary floating entrances are located at various locations between Units 2 and 19.

On the downstream side of the tailrace deck at Unit 20, there are two sets of slots (Figure 5.1). There are four upstream slots for the entrances. Two of these four slots are for fishway entrance weir gates and the other two are for stoplogs. There are two fishway guides for the draft tube stoplogs downstream of the fishway entrances.

The fishway entrance gates at Unit 20 will be moved to Unit 16. See Figure 5.2. Steel frames will be fabricated to accept the weir gates from Unit 20 to form a complete assembly. The two assemblies will be inserted into the two 3.05 meter (10-foot) wide slots at Unit 16. The existing gates will move inside the frames which will be fixed to the concrete bulkhead slots. The hoists at Unit 20 will also be moved to Unit 16.

A new transverse bulkead gate will be constructed and installed in a set of existing gate slots located in the fish collection channel between Units 16 and 17 to keep the fish in the collection channel from migrating north of Unit 16.

SECTION 6.0 ELECTRICAL DESIGN

6.1 Power

The power for the new surface bypass gates will be provided through the substation located at the north non-over flow monolith (DSQ1) (Plate 16). In talking to plant personnel, it is understood that this substation has sufficient spare capacity for this addition. From here the power circuit will run in the existing cable tray located in the 273 spillway gallery to a new 480 volt, 3 phase motor control center to be located near the new gates (Plate 17). At this location there will also be a new low voltage transformer and panelboard installed for providing power to such loads as lights, receptacles, motor heaters and gate heat tracing.

6.2 Grounding

A new grounding system will be imbedded within the new spillway structure to replace the removed portions of the existing system. The new grounding conductors will be sized to match the existing 500MCM/250MCM system.

6.3 Gate Controls

Local (at the gate operator) controls for the new gates will consist of (RAISE/LOWER/STOP) push buttons with lights for both 100% open (OPEN) and 100% closed (CLOSED) indication. At the gate operators there will also be a (LOCAL/REMOTE) control switch to transfer control to the control room (REMOTE) (Plate 18).

Remote operation from the control room will be implemented via a proposed new Panelmate Power Series touch screen operator interface panel connected to a remote I/O rack installed in the gate operator MCC. The touch screen panel will be installed in the control room along with the PLC as part of the planned North Fish Attraction Water Pumps control upgrade to occur in 1998. The remote I/O rack and data cable connection will be installed as part of this project.

The gate control features to be implemented on the Panelmate Power Series touch screen panel via the associated PLC will be RAISE, LOWER and STOP.

6.4 Annunciation, Communication, Instrumentation

The 100% open and the 100% closed status of the gates will be monitored by gate position limit switches at the limit of travel in both directions. Motor starter overload status will also be monitored. All three of these points will provide status to the control room via a remote I/O and the proposed new Panelmate Power Series touch screen.

6.5 Lighting and Receptacles

New outside lighting to match existing fixtures will be provided at the gate operators for use by operations and maintenance personnel. Low level roadway lighting for the new spillway bridge will be provided to match existing fixtures.

New 480 volt three phase 30 amp receptacles to match existing will be provided along the intake deck at the new gates to provide power for the spillway stoplog gantry crane.

6.6 Modifications to Existing Powerhouse Systems

a. Intake Deck

The over head power circuit for the intake gantry crane will be shortened to accommodate the demolition of the Units 19 and 20. Light fixtures, receptacles and associated conduit and conductor for Units 19 and 20 will be removed back to the next adjacent fixture outside of these two Units or if independently powered, they will be removed back to the panelboard from which they originate. For panelboards located in this area serving circuits unaffected by the modifications, they will be relocated to a position just outside the demolition area and their branch circuits rerouted. The 500 MCM grounding system Bus A will be removed for Units 19 and 20 during demolition and the grounding system will be reestablished in construction of the new structure. The grounding loop between Bus A and Bus B will also be reestablished at this time.

b. 265 Gallery

Existing light fixtures and receptacles at Units 19 and 20 will be removed and replaced with new fixtures to match in the new gallery. Existing power circuits will be reused for the new fixtures and receptacles.

Once the new service gallery is constructed and new cable tray is installed, all of the existing control, power and communications circuits will be rerouted through this gallery (Plate 19). The control, communications and low voltage power circuits will be replaced with new conductors one at a time, running them from their present termination point to the North to the nearest existing termination point to the South through the new gallery as a continuous run. Once the new cable is installed the cut over to the new conductor will be made and the old conductor removed.

The 4160-volt circuits feeding the North bank of the dam (DSQ1) will be replaced as follows. New 5KV cables will be installed in the new gallery running from the nearest existing splice to the North to the nearest existing splice to the South in the old cables. One at a time these circuits will be cut over to the new conductors by opening the splices and removing the sections of cables routed through the old gallery and splicing in the new cables.

New taps to feed the unwatering pumps, DSQ2 and DCQ5 will be installed in the 5KV cables in the new section of gallery (Plate 17). Bus No.1 and Bus No.2 feeding the unwatering pump room will be routed from the new service gallery to the electrical equipment room in imbedded conduit. This conduit will be in the new chute wall separating the surface bypass spillways from the center non-overflow section of the dam.

c. 251 Gallery

Light fixtures, breakers, transformers and receptacles as well as associated conduit and conductor for Units 19 and 20 will be removed back to the next adjacent fixture outside of the two units or if independently powered, they will be removed back to the panelboard from which they originate. See Plate 5 for the location of this gallery.

d. 207 Gallery

Any light fixtures, receptacles and associated conduit and conductor for Units 19 and 20, affected by the new construction, will be removed back to the next adjacent fixture outside of these two units or if independently powered, they will be removed back to the panelboard from which they originate. For panelboards located in this area serving circuits unaffected by the modifications, they will be relocated to a position just outside the demolition area and their branch circuits rerouted. See Plate 5 for the location of this gallery.

e. Generator Floor

The high bay light fixtures and associated conduit and conductor for Units 19 and 20 will be removed back to the next adjacent fixture outside of the two units or if independently powered, they will be removed back to the panelboard from which they originate. The bridge crane power buses will be shortened back to skeleton Bay 18 and terminated. Other light fixtures, receptacles and associated conduit and conductor for Units 19 and 20, affected by the new construction, will be removed back to the next adjacent fixture outside of these two units or if independently powered, they will be removed back to the panelboard from which they originate. For panelboards located in this area serving circuits unaffected by the modifications, they will be relocated to a position just outside the demolition area and their branch circuits rerouted.

f. Tailrace Deck

Any light fixtures, receptacles and associated conduit and conductor for Units 19 and 20, affected by the new construction, will be removed back to the next adjacent fixture outside of these two units or if independently powered, they will be removed back to the panelboard from which they originate. The weir gate motors, electrical controls, and bubbler system for the fish collection channel entrance weir gates at Unit 20 will be relocated to Unit 16.

g. 168 Gallery

This gallery is located beneath the tailrace deck. See Plate 5. Any light fixtures, receptacles and associated conduit conductor and cable tray for Units 19 and 20, affected by the new construction, will be removed back to the next adjacent fixture outside of these two units or if independently powered, they will be removed back to the panelboard from which they originate.

For panelboards located in this area serving circuits unaffected by the modifications, they will be relocated to a position just outside the demolition area and their branch circuits rerouted.

h. 150 Gallery

This gallery is located below the 168 Gallery. See Plate 5. Any light fixtures, receptacles and associated conduit and conductor for Units 19 and 20, affected by the new construction, will be removed back to the next adjacent fixture outside of these two units or if independently powered, they will be removed back to the panelboard from which they originate. For panelboards located in this area serving circuits unaffected by the modifications, they will be relocated to a position just outside the demolition area and their branch circuits rerouted.

The weir gate motors, electrical controls, and bubbler system for the fish collection channel entrance weir gates at Unit 20 will be relocated to Unit 16.

SECTION 7.0 CONSTRUCTION CONSIDERATIONS

7.1 Construction Access

Construction of the Skeleton Unit surface bypass spillway will require access to both the forebay and tailrace areas for installation of dewatering systems, demolition, concrete placement, and gate installation. Access would be by barge or truck. A brief discussion of the work activities in the forebay and tailrace areas as well as required equipment access are presented in the following paragraphs.

It is anticipated that operations personnel will require access across Skeleton Bays 19 and 20 during construction. A phased construction schedule has been developed to provide access on the powerhouse deck across the work area. See Section 11 for an explanation of the construction sequencing.

a. Forebay

Access to the forebay will be required to install the dewatering bulkheads, remove demolition materials, and install portions of the new surface spillway, gates, and bridges. Access will be limited to the powerhouse deck at elevation 85.649 meters (281.0 feet) from the south shore or the main spillway access road which requires crossing the navigation lock. Limited work area is available from the powerhouse deck and access with wheeled vehicles will be restricted to minimize impacts to dam operation. Figure 7.1 is a view of the powerhouse deck looking past Unit 20 to the south. It shows the limited access when work is in progress on the deck. Truck access from the north shore is across the navigation lock. Although the bridge across the navigation lock is fixed, there are four sharp 90 degree turns which might limit the length of the trucks. A limited turn around area is available at the north end of the spillway near the navigation lock.

Installation of the dewatering bulkhead will require access to the powerhouse deck as well as barge operations. Once the dewatering bulkhead is in place, demolition of the upstream portion of the existing powerhouse concrete will be initiated.

Removal of demolished materials will be by barge from the forebay or by truck from the powerhouse deck. Installation of new concrete and gates might require temporary closure of the powerhouse deck. Precasting of the new deck and gallery over the new spillway chutes may be feasible to minimize disturbance to vehicle traffic on the intake deck.

Access across Skeleton Units 19 and 20 will be restricted due to demolition activities. However, during Phase 1 construction access across the powerhouse will be over the existing deck east of the CBL. During Phase 2 construction access will be possible over the new spillway bridge west of the CBL constructed in Phase 1. See Section 11 for a discussion of the construction phases.

b. Tailrace

Access to the work area of the powerhouse or tailrace deck downstream of the powerhouse will be over the tailrace deck or by barge from the tailrace (Lake Celilo). The work area is at the north end of the tailrace deck. See Figure 7.2. Access to the tailrace deck is from the south shore by a frontage road which connects with Interstate 84 about 2.7 kilometers west of the dam. See the Location Map on Plate 1. Access by barge is up the Columbia River to the tailrace area. Barge access will be restricted during the spill season from about April 15 through September 1.

c. Staging and Stockpile Area

The contractor will require stockpile and staging areas relatively close to the Skeleton Units during construction. The south shore tailrace area near the recently completed juvenile fish bypass channel has limited area available. Only minor stockpiling and office space would be available on the south shore. There is considerable space available on the north shore for stockpile or a concrete batch plant, if necessary.

Staging larger pieces of equipment such as the radial gates or bulkheads would be by barge. So, the contractor would have to carefully schedule his major equipment deliveries or obtain stockpile areas at a barge loading facility.

7.2 Construction Restrictions

During certain times of the year work in the water near John Day Dam is limited. This work has two main restrictions :

a. Spill Periods

Work in the tailrace is limited during spill from the main spillway. In general, spill for fish passage takes place from April 15 through September 1. Work by boat or barge in the tailrace area is not allowed for safety purposes during spill. Work by barge is also limited in the forebay near the spillway during spill periods. For the purposes of this memorandum it was assumed that barge work in the forebay could only take place south of Unit 20 during spill. During periods when no spill is taking place in-water work is restricted by normal safety considerations. These include scheduling the use of turbines during in-water work to prevent unsafe currents in the work area.

b. Fish Passage

Project features for upstream and downstream migrating anadromous fish must be in operation during certain times during the year. Work restricting use of the fish ladder can only be accomplished between December 1 and February 28. The existing fisheries juvenile bypass equipment might require maintenance at any time of the year. The large gantry cane on the upstream powerhouse deck is used for removing and installing the intake screens and other equipment. This activity would restrict access from the south shore to the work area at Units 19 and 20 as shown on Figure 7.1. At this time the schedule for this activity is unknown. Any construction activities that might interfere with gantry crane access must be coordinated with project operations and fisheries activities.

c. Other Restrictions

Construction will be restricted by operations at the dam and powerhouse. Access across Units 19 and 20 is required by operations personnel at essentially all times. Any access restrictions must be approved by project operations. Demolition work will cause some dust to be released especially during concrete cutting. This dust could be detrimental to the machinery in the powerhouse. Some restrictions might be required to minimize the production of dust. Much of the power and control cables necessary for dam operation will be rerouted through the new galleries. To minimize disruptions each cable will be laid through the new galleries, the old cable disconnected at the ends, and the new cable connected. This will be done cable by cable to minimize down time.

7.3 Forebay Dewatering

The Skeleton Unit surface bypass spillway design requires removal of a portion of the existing powerhouse concrete above the intake. In order to remove the existing concrete, a temporary dewatering bulkhead will have to be installed on the upstream face of the powerhouse. Two features of the existing dam face present challenges to the bulkhead design: (1) the upstream face slope of 1H:6V, and (2) stoplog slots and piers located on the face. A dewatering system composed of a floating bulkhead is proposed (see Plates 2, 3A and 3B). The bulkhead system will be designed to span approximately 16.15 meters allowing demolition of the existing concrete, construction of the surface spillway, and gate installation to occur in a phased arrangement. The bulkhead span is limited by the ability of the existing concrete piers to carry the hydrostatic forces transmitted from the bulkhead to the piers. Plate 3B outlines a phased construction approach and dewatering bulkhead placement. Construction scheduling is described in Section 11.

Steel piers will be installed on the north and south ends of the area to be dewatered as shown on Plate 2 and 3A. These piers will provide sufficient working space and a vertical face for the floating bulkhead to seal against. Floor sections will be attached to the traskrack slot piers and will fill the area between the piers. The configuration of the trashrack slot piers are shown on Figure 7.3. The steel piers and floor sections will be provided with seals to prevent leakage between them and the face of the powerhouse wall and piers. Bolting the steel piers and floor to the dam and final sealing will be performed by divers.

A floating bulkhead will provide final closure and will be fitted with fill valves and pumps to adjust its floating depth. The bulkhead will span approximately 16.15 meters. The floating bulkhead will be designed with a one meter lip which will rest against the fixed floor and pier wall sections. Seals between the fixed pier and wall will provide positive closure. Divers will bolt the floating bulkhead to the fixed steel piers and floor section.

When the pier walls, floors, and floating bulkhead are in place and sealed, the water inside will be pumped out.

The top of the dewatering bulkhead will be set at elevation 82.9 meters (272.0 feet) which is 1.2 meters above the maximum operating pool elevation of 81.686 meters (268.0 feet) to allow for protection against wave action. The floor of the bulkhead is set approximately at elevation 71.63 meters (235 feet). This is 2.3 meters below the chute crest.

7.4 Tailrace Dewatering

The tailrace area of construction consists of the tailrace deck, fish collection channel, and the 150 and 168 Galleries. Since it contains open areas of water conduits and galleries, the dewatering must be carried out in stages and the north and south ends of these "open areas" must be sealed. In addition, care must be taken to insure that the structural integrity of the walls are maintained throughout dewatering and construction. A detailed dewatering and construction sequence has been devised for work in the tailrace area. See Section 11.0 for a description of the construction schedule. Although there are probably several approaches to construction in the tailrace area, an approach using a dewatering bulkhead is outlined below in chronological order of work.

1. Preliminary Work - This includes dewatering Units 15 through 20, relocating the fishway entrances from Skeleton Unit 20 to Generator Unit 16, relocating the water level measuring equipment in the fish collection channel, increasing the auxiliary water capacity at Unit 16, rerouting utilities in the 150 and 168 Galleries, and installing the permanent transverse bulkhead at the north end of Unit 16. After removing the two fishway entrance gates at Unit 20, pre-fabricated bulkheads will be inserted into the existing entrance gate slots prior to dewatering the collection channel. The existing fishway entrance gates would be inserted into new steel frames. The frames with installed gates would then be inserted into the existing slots at Unit 16. Transverse bulkhead slots are located in the fish collection channel at the extreme south end of all odd numbered units. A new transverse bulkhead would be built and inserted permanently at the south end of Unit 17. At this point the transverse bulkhead at the south end of Unit 15 could be removed and the fishway put back in operation.
2. First Stage Demolition - This involves removing the tailrace deck between the deck beams located on 6.477 meter (21 feet, 3 inch) centers. See Figure 7.4. The walls on either side of the fish collection channel would be removed down to elevation 52.0 meters. The deck beams carry the hydrostatic load from the tailrace into the powerhouse and must remain in place until temporary bracing can be erected. Figure 7.5 shows the west wall of the fish collection channel. The deck beams are attached to the concrete piers at their top.
3. Plug the ends of the Galleries - At the north end of the 168 Gallery is the unwatering pump station. This must be isolated from the new spillway. The openings in the 150 and 168 galleries at the south end of Skeleton Unit 19 must also be blocked. Reinforced concrete plugs anchored to the existing structure would be placed at these locations. The

construction joints would carried through the plugs. Since the entrance to the unwatering pump station has been cut off, a new access would be built through the tailrace deck just north of the powerhouse.

4. Dewater Tailrace Area - Specially fabricated steel bulkheads would be installed on the west end of the powerhouse. They would extend from elevation 53.0 at the top to elevation 45.0 at its floor. They would contact the powerhouse and be bolted to it at elevation 45.0 and at the piers immediately north of Unit 20 and south of Unit 19. See Plates 2 and 3A. Temporary braces from the top of the bulkhead to the west powerhouse wall would be installed to carry the hydrostatic loads to the powerhouse. Some holes would have to be cut in the east collection channel wall to allow for installation of the braces. The floor of the bulkhead would conform to the draft tube bulkhead slots projecting from the west powerhouse wall as shown on Plate 2.
5. Remove Deck Beams and Walls - After the bracing is installed the tailrace deck beams would be removed. The temporary braces will carry the hydrostatic load to the powerhouse. Next, the walls on the west and east side of the fish collection channel would be removed down to elevation 46.0 meters and 47.0 meters, respectively. The west wall of the powerhouse would be removed down to elevation 46.5 meters.
6. Construct Spillway Chutes - The stem walls from the top of the cut walls would be cast in place. The spillway chutes and guidewalls would be built next. The chutes and stem walls would not be built between the temporary braces.
7. Replace Bracing - This task involves erecting temporary bracing off the ends of the new spillway guidewalls or chute floor. Once the new temporary bracing is in place the old temporary bracing can be removed.
8. Complete the Spillway and Spillway Bridge - In this step the stem walls, chute floor, and guidewalls would be constructed in the open areas which were formerly occupied by the temporary bracing. Then the spillway bridge would be constructed.
9. Remove the Dewatering Bulkhead - In this last step the area behind the dewatering bulkhead would be flooded and the temporary braceing and bulkheads would be removed.

7.5 Demolition

Construction of the surface bypass spillway will require select demolition of Skeleton Units 19 and 20. Sequencing of the demolition and construction will be required to maintain sufficient structural stability in the remaining dam cross-section to support the upstream dewatering bulkhead.

As shown on Plates 4 and 5, the demolition activities are divided into three groups:

1. Phase 1 Demolition, west of the Construction Base Line (CBL);
2. Phase 2 Demolition, east of the CBL;
3. and Gallery Mining.

a. Phase 1 Demolition

Phase 1 demolition will involve removal of powerhouse at Skeleton Units 19 and 20 and mass concrete west of the CBL. As shown on Plate 5, the powerhouse will be demolished beginning with the roof, then the interior walls. Then the tailrace deck area and the structure between the CBL and the powerhouse will be removed. See Section 11.0, Construction Schedule, for a more complete description of this sequence.

b. Phase 2 Demolition

Phase 2 demolition will occur following construction of the new chute support walls, chute floor and guide walls west of the CBL. Plate 3B presents a phasing plan for placement of the upstream dewatering bulkhead and demolition of mass concrete east of the construction baseline. With this approach, the upstream dewatering bulkhead is placed to span Chutes 20a and 20b. Concrete will be removed behind the dewatering bulkhead upstream of the CBL. The spillway wall between Chutes 20a and 20b will then be constructed. Temporary stoplogs will be placed between the new pier and the existing concrete to the north and south. See Plate 3B, Bulkhead Move No. 1. Based on structural computations the pier will have to reach a compressive strength of 20.7 megapascals (3,000 psi) before removal of the bulkhead and application of hydrostatic forces on the stop logs.

The bulkhead, vertical piers, and floor will then be moved to the north to span Chute 20a. The newly constructed pier between chutes 20a and 20b will have to reach a compressive strength of 20.7 megapsacals (3,000 psi) before the hydrostatic pressure on the bulkhead can be placed on it. The remainder of Chute 20a will be excavated east of the CBL. Then the pier on the north side of 20a will be built and the tainter gate will be installed from the deck of the non-overflow dam north of Chute 20a. The tainter gate can be installed at any time since it is supported by the pier downstream of the CBL built during Phase 1. The temporary stop logs will be installed and bulkhead removed. See Plate 3B, Bulkhead Move No. 2.

The bulkhead, vertical piers, and floor will then be moved to span Chutes 20b and 20c. This sequence will be as shown for Bulkhead Moves No. 3 through No. 7. With this approach, the new concrete piers can be used to support the loads of the upstream dewatering bulkhead.

c. Gallery Mining

A new gallery will be mined through Skeleton Unit 18 to allow relocation of the existing utilities in the 265 Gallery. The new gallery will begin at the construction joint between Skeleton Units 17 and 18 (at elevation 80.772 meters) and extend north and upward to the construction joint between Skeleton Units 18 and 19 at elevation 83.210. The mined tunnel will connect with the bridge gallery across the chutes in Units 19 and 20. A mined tunnel will extend from the north face of Skeleton Unit 20 to connect with the spillway gallery at elevation 83.210 meters. An access shaft may be required to dispose of the mined concrete.

d. Concrete Demolition Methods

Several methods are available for mass concrete removal:

1. Saw or diamond wire cutting
2. Chemical expanders
3. Mechanical “mining”
4. Blasting

Blasting is not a feasible option for this project. Removal of the powerhouse structure and mass concrete requires select demolition techniques. Blasting is not possible due to risk of damage to existing structures.

Demolition of the powerhouse and mass concrete will require a combination of saw or diamond wire cutting and chemical expanders. Saw and diamond wire cutting allows precise tolerances to be met. Chemical expanders can be used to break large blocks already detached into more manageable pieces.

Mining of the new gallery sections using a rotary head cutter is not recommended because of experienced difficulties. A similar technique was used with difficulty at John Day and Little Goose Dams. A drilling and splitting mining technique appears to be the most applicable method and is assumed here for cost estimating purposes.

7.6 Disposition of Removed Materials

Two options are available for disposition of demolished concrete from Skeleton Units 19 and 20:

1. Disposal at an off-site location.
2. The draft tube of Skeleton Units 19 and 20.

a. Disposal at an Off-Site Location

If off-site disposal is required, the material would have to be either barged or trucked to the disposal site. Disposal at the Arlington land fill is assumed. Barging would be the preferred method, however during periods of spill, use of barges would be restricted. Therefore, temporary storage on a barge and trucking to Arlington was assumed. A suitable disposal site at the project was not identified.

b. Disposal in the Draft Tube of Skeleton Units 19 and 20

Disposal from demolition of the powerhouse in Phase 1 can be placed in Skeleton Units 19 and 20 after the concrete plugs have been constructed in the inlet and the draft tubes. This fill could be placed up to the turbine floor at elevation 32.461 meters (106.5 feet). See Plate 5. This would require a slight change in the schedule shown in Section 11.0.

After construction of the two spillway support piers more demolished concrete can be placed up to about elevation 46.0 meters in the Skeleton Unit cavity. However, this would require revamping the construction schedule shown in Section 11.0. The spillway construction in the powerhouse area would have to be delayed to the end of Phase 1. This would allow demolished concrete from the tailrace deck area and upstream of the powerhouse to be deposited in the Skeleton Unit cavity. The spillway in the powerhouse area would be constructed at the same time as the spillway in the tailrace deck area. *A geotechnical analysis should be performed to ascertain whether the increased weight would cause differential settlement*.

c. Access During Demolition

Access during demolition is limited. Demolition of Skeleton Units 19 and 20 will require access from the tailrace deck or crane from a barge. The removed concrete will be loaded directly on trucks or barges.

Mass concrete removal west of the construction baseline will most likely be accomplished from the tailrace deck. This approach affords direct access for removal of demolished concrete as well as minimizes impacts to traffic on the intake deck.

Removal of concrete east of the construction base line will require temporary shutdowns of the intake deck roadway. Demolished concrete can be removed either with trucks or a barge.

7.7 Relocation of Utilities

Utilities are located in the 265 Gallery in the upstream portion of the powerhouse and in the galleries at the rear of the powerhouse. These galleries have been termed the 168 gallery and the 150 gallery. See Plate 5. See Section 6 for an explanation of electrical utilities and their relocation. Relocation of the utilities will be accomplished as explained below:

a. 265 Gallery

The 265 Gallery contains a 76 mm (3-inch) .69 megapascal (100 psi) air line, a 200 mm (8-inch) water line, two high voltage power cables, other low voltage wires, and numerous control and communication cables. These utilities are necessary for operation for the spillway gates, fish orifice gates, and control of the dam. It is essential that these utilities continue in operation during construction.

It is desirable to move these utilities only once. In developing a construction sequence it was determined that the utilities would be moved from their present location to their future location during Phase 1 of construction. This would be accomplished by construction the downstream portion of the spillway and the spillway bridge first. The downstream side of the spillway bridge contains an extension of the 273 gallery from the spillway. After this gallery is completed a tunnel will be bored in Skeleton Unit 18 at a slope upward from its southern end in the 265 gallery to elevation 83.210 m (273 feet) at its northern end. See Plate 13. At the northern end of Unit 18 the gallery would be directed to the northwest to join the spillway gallery across Chute 19c. The gallery boring will pass within 0.669 m of the northern upstream gate slot for Unit 18.

When this task is complete the utilities will be run through the new galleries and connected to the existing utilities in the non-overflow section and at the construction joint between Skeleton Units 18 and 17. In this manner the utilities in the 265 gallery will be out of service only once for a very short period of time.

b. The 150 and 168 Galleries

Utilities in the 168 Gallery will be rerouted through a hole cut in the floor to the 150 Gallery. The utilities will run in the 150 Gallery under the spillway to the north end of the powerhouse. Pedestrian access will be provided through the 150 gallery for maintenance of these utilities. The gallery ceiling will be formed by the spillway. The gallery will be about one meter high along its west side and about two meters high along its east side under the lower chutes (Chutes 20a, 20b, and 20c). See Plate 5.

c. Unwatering Pumps

The center non-overflow section unwatering pumps are located in a pump room below the tailrace deck at the north end of the 168 Gallery. When the northern guide wall to Chute 20a is completed, this room will be cut off to personnel access. Therefore, a new hatch will be built into the tailrace deck in the non-overflow section to permit access down to the pump room. See Figure 7.6. The hatch opening will be approximately 1.4 meters by 1.7 meters and will be furnished with a metal ladder. The hatchway would be constructed to be water tight since the tailrace deck is open to the elements and spray from the spillway. An air vent, protected from the elements, will also be provided in the ceiling deck. A bridge across the lower spillway provides vehicular access to the deck north of the Surface Bypass Spillway and the pump room.

Alternately, the sleeves for the pump shafts would be extended up through the existing access hatches to the pumps which would be mounted on the tailrace deck. A new pump house would be built to house the pumps, motors, and motor controls. The existing pump room would be abandoned. A bridge across the lower spillway provides vehicular access to the deck north of the spillway and the pump room.

7.8 References

The following references were used in the preparation of this section:

EM-1110-2503, Design of Sheet Pile Cellular Structures Cofferdams & Retaining Structures, 29 SEP 89.

EM-1110-2504, Design of Sheet Pile Walls, 31MAR 94.

John Day Lock and Dam, Foundation Grouting and Drainage, DM No. 34, 16 NOV 62.

John Day Dam Lock and Dam, Second Step Cofferdam, DM No. 21, 27 OCT 61.

New sketches showing the spillway geometry, 16 JUN 97.

Materials and Equipment for Marine Construction, Pile Buck, Inc., 1990.

Cellular Cofferdams, Pile Buck, Inc., 1990.

SECTION 8.0 OPERATION AND MAINTENANCE CONSIDERATIONS

8.1 Operating Gates

a. Operations

Each spillway chute tainter gate will be operable from either a control station located by the gate operator or remotely from the powerhouse control room. The local control station will consist of a local-remote selector switch, a set of open-close-stop push buttons and a set of full-open/full-closed indicator lights. The full-open/full-closed gate conditions will be controlled by a pair of gate travel limit switches.

The powerhouse control room will be provided with remote indication and control via a panel mounted Panelmate Power Series touch screen which will communicate with a remote I/O rack mounted in the gate operator MCC via a remote I/O link. The control features that will be available at this location are gate RAISE/LOWER/STOP. The control status available at this location will be gate in REMOTE, RAISING, LOWERING, 100% OPEN, 100% CLOSED and MOTOR OVERLOAD.

The heating cable system for each separate gate will also be manually controlled by a switch provided at the local control station or remotely controlled from the powerhouse control room.

b. Maintenance

Since the gates will only be operated a few times a year, maintenance requirements should be minimal. The cable hoist gear drives should have their oil level checked approximately once a year. The lifting cables should be inspected annually for signs of wear or damage. The drive motors will be totally enclosed and thus will not normally require any attention.

The rubber seals on the gate sides and bottom should be inspected annually and repaired or replaced as required. The gate trunnions will be self-lubricating and will not normally require any attention.

The gate structure will be provided with a high-build epoxy painting system. The coating should be inspected annually and repairs made accordingly.

8.2 Maintenance Stoplogs

a. Operations

The 3.45 meter high maintenance stoplogs will be installed and removed by use of the existing spillway stoplog gantry crane. The stoplogs will be placed using a fabricated steel lifting beam. The lifting beam will be attached to a special adapter beam which in turn will attach to the existing lifting connections on the spillway stoplog lifting beam. See Plate 15B. The adapter beam is required because the existing lifting connections on the spillway lifting beam are spaced too far apart to allow a single lifting beam to fit into the new stoplog guide slots. After installing a stoplog, the operator will pull a lanyard opening the lifting hooks and releasing the lifting beam from the stoplog. The beam will be raised and attached for the next stoplog to be placed.

Since the 8.84 meter (29.0 foot) lifting clearance above the spillway bridge is not sufficient to install the bottom stoplog in one lift, the lifting cables will be segmented into three 3.0 meter sections. This will permit the stoplog to be temporarily dogged off at an intermediate elevation in the guide slot while another cable section is added. Guide bars at the top of the bottom and middle stoplogs will guide a stoplog into the proper position atop another to create a good seal.

To remove the stoplog the lifting beam will be lowered into the stoplog slot. The hooks will be hinged to allow them to automatically engage the lifting eyes on the top of each stoplog. The stoplogs are designed for installation and removal under a balanced hydraulic head.

The stoplogs will be furnished with rubber seals to facilitate sealing. It may also be necessary to open the operating gate slightly to provide a small differential head across the stoplogs to assist in sealing the stoplogs.

The tailrace bridge will be located at the same elevation as the existing tailrace deck at elevation 56.38 m (185.0 ft.). This places the bridge below the tailwater elevation during extreme high flow events and might force a hydraulic jump onto the bridge. To protect the bridge from damage the stoplogs would have to be installed during extreme high flow events. The tailwater level at which the stoplogs should be installed should be determined in the hydraulic model.

b. Maintenance

The stoplogs will also be provided with a high-build epoxy coating system. The coating and rubber seals should be inspected annually and repaired as required.

SECTION 9.0 SINGLE UNIT OPTION

9.1 Introduction

The skeleton bay surface bypass system was designed to provide a flow net from approximately 90 m upstream of the dam face to the surface bypass system entrances (Section 2). The flow net is defined by water velocities that continually increase as they approach the surface bypass spillway, culminating in a trapping flow immediately upstream of the entrances. Only one complete surface bypass bay (three chutes), discharging approximately 535 m^3/s is required to provide these flows. Due to fluctuating tailwater conditions and the associated changes in the chute exit elevations, two units will have to be modified in order to enable operation of a single unit under a full range of total river discharges. Surface Bypass Unit 20 will operate up to total river flows of approximately 8495 m^3/s, and Surface Bypass Unit 19 will operate between total river flows of approximately 5663 m^3/s to 11327 m^3/s (Section 2).

Construction of a single unit prototype is being considered for a number of reasons. The technology, although supported by a relatively extensive data base, remains largely untested at the John Day Dam. A single unit prototype will enable final verification of the concept, provide efficiency and effectiveness evaluations of a full scale prototype, and enable an estimate of the potential effectiveness of the final system. It will also require a reduced initial financial investment. Given the proposed operation of the system, a single unit prototype can be expected to provide maximum benefit, only the benefit will be over a reduced range of total river flows. Therefore, it will be possible to construct a single unit prototype, and obtain an accurate assessment of this technology for either of the two outmigrations. Given the relatively poor guidance of the screened bypass system at the powerhouse during the summer outmigration, and limited available spill due to a typical reduction in total river flows during the summer, the greatest benefit to fish passage for a single prototype system is expected to be during the summer outmigration. Therefore, Surface Bypass Unit 20, designed for lower total river flows, has been selected as the prototype unit.

Although extrapolating fish passage information from the summer outmigration to infer a system effectiveness during the spring is not the best way to evaluate a system, summer outmigrants are considered to be somewhat more difficult to collect and bypass at a dam. Therefore, it can reasonably be assumed that an evaluation of the summer outmigration represents the worst case scenario.

The description of the physical features in Sections 4 through 8 addressed the Full Surface Bypass Spillway Option which involved Skeleton Units 19 and 20. An option under consideration is the single unit option of building a surface bypass spillway at Skeleton Unit 20. The single unit option would be implemented if funding restrictions prevent construction of the full two-unit spillway. The single unit option is considered a prototype to test the potential success of the final two-unit system. This section discusses the single unit option.

A single unit surface bypass spillway at Skeleton Unit 20 would have the same configuration as the Unit 20 spillway in the Full Surface Bypass Spillway option. In the subsections below only the differences between the two options as they apply to the Unit 20 spillway are described. In subsection 9.2 the differences in physical features are described. In subsection 9.3 the changes in the construction process are outlined. Subsection 9.4 addresses the preliminary operating plan. Sections 10 and 11 present the construction costs and schedule of both these options.

9.2 Physical Features

The single unit option requires about half of the demolition, concrete, rerouted utilities, mechanical equipment, and tailrace dewatering bulkhead would be required. The physical features at Unit 20 under the single unit option would be the same as in the full spillway option except as described below.

1. Spillway and Spillway Guidewalls - The chutes in Unit 20 will have the same geometry in both options. See Plate 7. The guidewalls will be the same except that the southern guidewall of Chute 20c will have the same configuration as the southern guidewall of Chute 19c. See Plate 6.
2. Spillway Support Wall - The spillway support walls would be the same except the south wall to Unit 20 would be the same as the south wall of Unit 19 for the full bypass spillway option. See Section C on Plate 9.
3. North Powerhouse Wall - The north powerhouse wall would be located at the very north end of Skeleton Unit 19 instead of 18. See Section C of Plate 9 and Plate 12.
4. Spillway Guidewalls - The spillway guidewalls would be the same as for the full spillway option except that the southern guide wall on Chute 20c would be the same as the southern guidewall shown on Section D Plate 9.
5. Gallery Modifications - The galleries will be modified in the upstream part of the powerhouse to carry the utilities across the dam. In the full bypass spillway option a gallery is to be mined from the 80.772 meter (265 foot) elevation at the south end of Unit 18 to the 83.210 meter (273foot) elevation at the north end of Unit 18. The mined gallery shown on Plate 13 would have the same dimensions but would be moved to Unit 19 under the single unit option. See Plate 13. There is little or no advantage to mining for the two-unit option during construction of the single unit spillway.

9.3 Construction Tasks

Most of the construction work is halved in the single unit option compared to the full spillway option. However, there are some tasks that remain the same for both options even though they might be performed in a different location. These are described below.

1. Electrical and Control Lines - The terminals from the electrical and control cables are located a large distance south and north of the work area. The same length of cable between these terminals would be replaced in both the full and single unit options. See Note 3 Plate 19.

2. Fishway Entrance Gates - The fishway entrance gates would be moved from Unit to Unit 16 under both options. See Plate 4.
3. Fishway Water Level Sensor - The fishway water level sensor used for setting the entrance gates will be moved to the collection channel at Unit 16.
4. Mined Gallery in the Non-overflow Section - The mined gallery in the non-overflow section is required under both options. See Plate 13.
5. Forebay Bulkhead Moves - The sequence of bulkhead moves to build the full spillway option is 1 through 7 as shown on Plate 3B. The sequence of bulkhead moves to construct the single unit option is 1, 2, 3, and 4. However , the work accomplished in Bulkhead Move No. 4 would be the same as in Bulkhead Move No. 7. That is, demolition would occur only in the south portion of Chute 20c.
6. Unwatering Pump Station - Access to the unwatering pump station at the north end of the 168 Gallery will be cutoff by construction of Chute 20a and a new access will have to be built. This will occur under both the full and single unit spillway options.
7. Gallery Plugs - Plugs are required in the 150 and 168 Galleries at the wall between Units 18 and 19 under the full spillway option. The same plugs are required in the single unit option but they will be located in the wall between Units 19 and 20.
8. Access and Stockpile Areas - The same access will be available for both the full and single unit bypass options. The stockpile and contractor work areas will be the same as those described in Section 7 for the full unit option.

9.4 System Operations

Preliminary observations of system operations on the 1:80 scale general model of the John Day Dam have identified several areas of concern regarding surface bypass unit operation. As discussed in Section 2, the shallow exit chute submergence, required to minimize TDG production, greatly influences tailwater conditions. The shallow discharge of the exit chutes has a tendency to create flow patterns in the upper 5 m of the water column that result in large shoreline eddies. Under extreme conditions, discharge exiting from the existing juvenile bypass system outfall can actually move upstream, towards the powerhouse, prior to being drawn into the surface bypass system and flushed out. The general model was therefore used to evaluate operation of a single prototype unit, at total river flows of between 2831 m^3/s and 8495 m^3/s. This range is typical of the summer outmigration and corresponds to operation of Surface Bypass Unit 20.

Under each scenario tested, a range of operations were determined based on minimum and maximum spillway and powerhouse discharges. These discharges are required in order to train surface bypass unit discharge, reduce or eliminate shoreline eddies, and maintain at least a 0.9 m/s velocity at the juvenile bypass system outfall. The outfall velocities were measured from the outfall to a distance of approximately 150 m downstream of the outfall, and from 35 m to 130 m from the Oregon shoreline. The dye released in the juvenile bypass system traveled through this area under all of the acceptable conditions tested (Table 9.1).

Specific spill patterns and main turbine unit operating priorities are required in order to adequately accommodate surface bypass operation. Although the spill patterns have not be finalized, they generally require a relatively even distribution of discharge between Spillbays 1 and 20. A minimum spill volume is required to eliminate tailwater eddies, and at least four of the six main turbine units (MU) on the south end of the powerhouse (MU1 - MU6) must be operating in order to maintain the minimum acceptable velocities at the juvenile bypass system outfall. Under total river flows of less than approximately 4250 m^3/s, surface bypass unit discharge has to be decreased in order to accommodate tailwater conditions, and the range of operation is fixed at approximately 10% spill and 84% powerhouse discharge (Figure 9.1).

As total river flow increases, the range of operation also increases. At 5663 m^3/s total river flow, spillway discharge can range from approximately 6% to 28%, and at 8495 m^3/s total river flow, spill can range from approximately 9% to 41% under maximum surface bypass unit operation. Under all of the conditions tested, typical spillway operation without surface bypass unit discharge ranges from approximately 25% to 60%, maintaining tailwater conditions within acceptable criteria. From these observations, surface bypass system operation can increase the flexibility of project operations by providing non-turbine passage over a wider range of flows. In addition, the high tailwater velocities at the chute exit of the surface bypass units will facilitate tailwater egress, especially under reduced total river flows or a reduction in spillway operations.

SECTION 10.0 COST ESTIMATE

10.1 Project Description - Full Surface Bypass - Units 19 and 20

The John Day Dam Surface Bypass Spillway is located at Skeleton Unit 19 and 20, adjacent to 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 proposed Surface Bypass Spillway will utilize juvenile salmon migratory behaviors to create a more effective dam bypass system. The Surface Bypass Spillway project is a complex demolition and reconstruction project requiring steel bulkhead cofferdams, extensive dewatering and unique coordination of sequence and scheduling.

Some of the major features of the project include:

- A 59 meter long steel bulkhead cofferdam at the tailrace area.
- 12,200 cubic meters of concrete demolition, including 56.7 meters of the power house and a 11.6 meters thick by 9.1 wide by 56.7 meters long mass of concrete at the top of the dam.
- 22,300 cubic meters of new concrete, including a new spillway slab, support and guide walls, and two bridge galleries.
- Six new Tainter gates.
- A new gallery that requires tunneling and rerouting of all major electrical and instrumentation from the existing gallery through the new gallery.

All of the work must be phased and sequenced. The work is currently scheduled to take about three years from notice to proceed to demobilization. The total bid contract construction cost before contingency and including 7% sales tax is estimated at $51 million. A 20% contingency is recommended for the construction phase. Engineering, construction management, legal and administration are not included in this cost.

a. Steel Bulkhead at Tailrace

A 59 meter long steel bulkhead will be installed at the tailrace area of the dam to allow demolition and new spillway construction to take place below elevation 52.0 meters. The steel bulkhead was estimated to weight 338,835 kilograms with a fabricated cost of $1.8 million. The bulkhead will be bolted to existing piers at the tailrace. Divers and a 150 ton crane will be required to install the bulkhead in sections. The installation of the bulkhead is scheduled to take thirty working days. The steel bulkhead is expected to have a salvage value of $0.13 per kilogram, based on conversations with contractors.

The work is currently estimated at $3.2 million to install, dewater and remove the steel bulkhead at the tailrace area.

b. Demolition

Demolition will be a combination of saw cutting and diamond wire cutting. A cost of $1,023 per square meter for diamond wire cutting was used in the estimate and was based on the average of quotes received from five demolition contractors, including one currently working at John Day Dam. Because the demolition contractors usually gave a range of costs for diamond wire cutting, the cost used in the estimate was based on the high range quoted by the contractors. The 8,220 square meters of diamond wire cutting in the project is valued at $8.5 million before markups.

A production rate of 9.3 square meters of diamond wire cutting per 10 hour shift is typical. For a job this size, as many as four crews working two shifts, or 74.3 square meter per day, may be required to achieve the schedule.

Demolition is phased throughout the project. The first phase is demolition of the north and west wall of the powerhouse. After draft tube and intake plugs are installed, the east wall and mass concrete east of the powerhouse and west of the construction base line (CBL) will be the demolished.

After new spillway support walls, spillway slabs, spillway guide walls and spillway bridge construction starts, the tailrace deck and walls to elevation 52.0 will be demolished. The steel dewatering bulkhead will then be installed at the tailrace and demolition of the tailrace walls below elevation 52 will proceed.

The total volume of 7,320 cubic meters of demolished concrete West of the CBL was assumed to be hauled to the Columbia Ridge Landfill in Arlington, Oregon, about 40 miles from John Day Dam. A cost of $33 per metric ton for hauling and disposal was quoted by the landfill operator. The demolished concrete will be loaded from the dam to a barge in 18 to 40 metric ton blocks. The concrete blocks will be broken into smaller pieces on the barge and then loaded by crane onto trucks stationed on the tailrace or powerhouse deck.

Demolition east of the CBL takes place after a tunnel is constructed to connect the new spillway bridge gallery to the existing gallery and all utilities are rerouted. Demolition east of the CBL is primarily mass demolition. The work will be done in increments of about 7.6 meters in sequenced phasing based on moving a dewatering bulkhead attached to the face of the dam.

The total volume of demolition of mass concrete for Phase 2 is 4,970 cubic meters. The demolition was divided by seven to evenly distribute the total quantity into the seven bulkhead moves. Diamond wire cutting was assumed to break the mass concrete into 36 metric ton blocks. A 230 metric ton crane was included in this area of the estimate to handle the large blocks of concrete. It was assumed the crane will be working from a barge located upstream of the dam. The demolished concrete will be broken into smaller pieces for hauling and disposal at the Columbia Ridge Landfill.

Total demolition is estimated at $14.7 million before prime contractor markup and contingencies. The total volume of concrete demolition is 12,300 cubic meters.

c. New Concrete

The new concrete work is constructed in two phases which require complex sequencing that is tied to demolition and bulkhead moves. Work west of the CBL starts with sealing the draft and intake tubes with 2,150 cubic meters of concrete valued at $826,000.

The new powerhouse end wall includes 1,785 cubic meters of concrete; the Spillway support walls include 6,100 cubic meters of concrete; the Spillway Chute includes 3,510 cubic meters of concrete; the Spillway Chute guide walls include 6,400 cubic meters of concrete; and the new Spillway and Tailrace Bridges includes 458 cubic meters of concrete. These quantities represent work West of the CBL and are part of Phase I work. The Phase I new concrete work is sequenced into work east and west of the powerhouse. The total volume of concrete for the powerhouse and Spillway concrete is 18,253 cubic meters. The work is valued at $13.7 million before contingencies. Phase I new concrete work is scheduled to take about 18 months.

Phase 2 work East of the CBL includes 1,700 cubic meters of Spillway guide wall concrete and 200 cubic meters of Spillway Bridge concrete. The total volume of concrete was divided by the number of bulkhead moves required for the work. The total estimated cost of the Phase 2 concrete work is $2.2 million. The work is scheduled to take about a year and a half.

The total cost of concrete work for the entire project is estimated at $16.7 million, or $750 per cubic meter for 22,300 cubic meters of new concrete.

d. Tainter Gates

Six new tainter gates will be installed in the new Bypass Spillways. The direct cost of $395,000 per gate is based on information from past projects and calls to suppliers. The cost includes a 25% contingency. $80,000 labor was estimated to install each gate. The total material cost of the six gates was estimated at $2.4 million and installation at $0.48 million. The gates are 7.9 meters high by 6.4 meters wide and include hoist motors that can be manually or automatically operated.

e. New Gallery and Tunneling

The new gallery is a complex phased construction sequence that requires two 2.1 meter x 1.8 meter tunnels through existing concrete. The two tunnels will connect with each end of the new spillway gallery and will be built during Phase 1 construction. The two tunnels are approximately 8.2 and 27.7 meter long. The tunneling work assumed a drill and split operation and was estimated at $0.64 million. The relocation of electrical, water and air lines was estimated at $1.21 million. The total cost of the tunneling and relocation of utilities is $1.9 million.

1. Basis of Design

FDM Report dated March, 1998 (Under Contract No. DACW 57-97-D-0004).

2. Construction Schedule

Construction is scheduled to start in May 2003 and End in February 2006. The following is assumed.

a) Overtime: No overtime is anticipated but double shifts may be necessary.

b) Construction Windows: Restrictions on in-water work apply between April and September during periods of spill. Currently the schedule includes in-water work in the forebay south of Unit 20 during the April to September spill period related to moving the upstream bulkhead and working off a barge adjacent to the dewatering bulkhead. The work will not be endangered by spill at the spillway if the barge is located south of Unit 20. Except for bad weather, there are no other restrictions.

c) Acquisition Plan: This project will be accomplished using one construction contract.

3. Subcontracting Plan

Not Applicable

4. Project Construction

a) Site Access: Site Access will be by paved road to the John Day Dam Site and by barge on the Columbia River.

b) Borrow Areas: It is assumed that borrow materials required for construction will be imported to the site. Concrete and aggregates will be batched from an existing commercial concrete batch plant off-site that has several material supply sources near the project site.

c) Construction Methodology: Construction of the Bypass Spillway and modifications to the existing structure will require civil, structural, mechanical, and electrical work to be performed in sequenced and coordinated fashion. Demolition will start immediately upon mobilization. When demolition and new construction at the tailrace area of the Bypass Spillway are complete the steel bulkhead will be removed.

   New construction and demolition will continue in two phases. The phases are defined by work on the West and East side of the construction base line (CBL) that runs North/South at the top of the dam. Work West of the CBL, Phase I, will take place first, followed by work on the East side of the CBL.

   Work East of the CBL requires installation of a floating steel bulkhead.

Construction of a phased and sequenced connection of a new gallery to the existing gallery and rerouting new electrical and instrumentation conductors is an essential and complex element of the project.

d) Unusual Conditions (Soil, Water, Weather): Cold winter weather, high winds and rough water are conditions that make working on the John Day Dam extremely difficult.

e) Unique Construction Techniques: Mass concrete demolition using diamond wire cutting and removing large blocks of concrete; and installation and removal of a 59 meter long steel bulkhead will require unique construction techniques.

f) 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, but a project this size could attract firms from throughout the United States. It is assumed that demobilization and mobilization costs will be similar.

   Labor was assumed to be available without restriction.

5. Environmental Concerns

Restrictions on when in-water "wet" work can occur during the months of April through the end of August. 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.

6. Contingencies

An overall contingency of 20 percent for all construction was based on uncertainty and risk.

7. Effective Dates for Labor, Equipment, Material Pricing

Davis-Bacon Decision Number WA970001, dated 9/05/97 was used for labor rates. The 1995 Equipment Rates for Region I, Aug. 95 were used for equipment rates. Material quotes were obtained during October to December, 1997. Some pricing in the MCACES Unit Price Book was revised to reflect estimator's experience and historical cost information.

8. Current Working Estimate - Full Surface Bypass Spillway - Unit 19 and 20

The current working estimate for the John Day Dam Bypass Spillway Project - Unit 19 and 20 is $51 million before contingencies. The cost includes 7% sales tax but does not include engineering, construction management, legal or administration. A 20% construction contingency is recommended. Table 10.1 shows the summary of the total cost including a 20% contingency, engineering, and construction management.

10.2 Project Description - Bypass Spillway - Unit 20 Only

An alternative project that would include a spillway only at Skeleton Unit 20 is also proposed. The project would include all elements of the spillway described for the Units 19 and 20 but the quantities would be reduced by half. See Section 9.

An estimate for a Bypass Spillway at Unit 20 was compiled by reducing most quantities for the Unit 19 and 20 Spillway in half. Some areas of the work remain constant, therefore the total cost is more than 50% of the cost of the Unit 19 and 20 Bypass Spillway.

All assumptions for the Unit 19 and 20 Bypass Spillway apply to the Unit 20 only Spillway.

A schedule for the construction of Unit 20 is provided in Section 11, and a description of the differences between the two-unit and one-unit options is given in Section 9.

a. Current Working Estimate for Bypass Spillway - Unit 20 Only

The current working estimate for the John Day Dam Bypass Spillway Project - Unit 20 Only is $32 million before contingencies. The cost includes 7% sales tax but does not include engineering, construction management, legal or administration. A 20% construction contingency is recommended. Table 10.2 shows the summary of the total cost including a 20% contingency, engineering, and construction management.

SECTION 11.0 CONSTRUCTION SCHEDULES

11.1 Introduction

This section describes the schedules for implementation of the two alternatives - 1) Full Surface Bypass Spillway (Skeleton Units 19 and 20); 2) Skeleton Unit 20 Surface Bypass Spillway. The design of the full surface bypass spillway was described in Sections 4.0 through 8.0. The design of the Unit 20 surface bypass spillway is essentially the same as the full spillway. The differences in the Unit 20 surface spillway design are described in Section 9.0. The schedule of the full surface bypass spillway is described in Subsection 11.2 and the schedule of the full surface bypass spillway is described in Subsection 11.3.

11.2 Full Surface Bypass Spillway

Figure 11.1 presents a construction schedule for the full surface bypass spillway. The schedule was developed based on a two-phase construction sequence. Phase 1 consists of demolition and construction activities west of the dam CBL. Phase 2 construction occurs east of the CBL. This phased approach was developed to ensure adequate stability of the powerhouse during construction and strength to support the hydrostatic loads from the dewatering bulkheads. Phase 1 includes all work west of the Construction Base Line (CBL), and Phase 2 consists of all work east of the CBL. To provide for a more compact schedule, it was assumed that the contractor could work east of the powerhouse and west of the powerhouse concurrently during Phase 1.

A brief summary of the major design and construction tasks are presented in the following paragraphs.

a. Final Design

Final Design is assumed to begin on March 1, 2002, the construction documents would be completed by February 3, 2003 With final design beginning on March 1, 1998, the construction documents would be completed by February 1, 1999.

b. BCOE Review and Revise Plans and Specifications

As soon as the design is complete the formal Corps of Engineers' review process will start. Review comments will be incorporated and final bid documents will be produced and copies made.

c. Mobilization and Site Preparation

Mobilization will begin with notice to proceed. Approximately 6 weeks was allowed to mobilize the contractor's trailer, equipment, and prepare the site storage and laydown areas.

d. Gate and Hoist Procurement

As soon as the contractor receives notice to proceed, the gate supplier will begin preparation of the gate and hoist shop drawings. Allowing 6 weeks for shop drawing review, the gate supplier has approximately 1 calendar year to fabricate and deliver the tainter gates and hoists. With the phased construction approach, the gate fabrication is not a critical path item.

e. Phase 1 - Work West of the Construction Base Line

Powerhouse - Phase 1 construction activities at Skeleton Units 19 and 20 will begin with demolition of the powerhouse roof over Skeleton Units 19 and 20. The north and west walls of the powerhouse would then be removed to the tailrace deck at elevation 56.388 meters (185 feet). Inside the powerhouse, construction of the plugs in the three inlets and two draft tube outlets would be constructed. Then the spillway supports inside the Skeleton Unit cavity would be constructed followed by the spillway itself and the spillway guidewalls. Lastly the new north wall of the powerhouse would be built at the south end of Unit 19 and the powerhouse roof connected to the wall. See Plates 9 and 12.

East of Powerhouse - This is the area east of the powerhouse and west of the Construction Base Line (CBL) including the east powerhouse wall. By completing the "East of Powerhouse" work as soon as possible, Phase 2 work can begin even if the Phase 1, West of Powerhouse work is not yet completed. This work would be concurrent with the work on Powerhouse explained above. Work in this area involves the following tasks.

- Demolish the east wall of the powerhouse and mass concrete west of the CBL. This work would be performed from barges on the upstream side of the powerhouse and from the powerhouse deck east of the CBL.
- Construct the spillway chutes and guidewalls on the west side of the CBL.
- Construct the new spillway bridge with its gallery west of the CBL.

West of Powerhouse - West of the powerhouse includes the tailrace deck, fish collection channel, and the 150 and 165 Galleries. Work in this area would be serviced by barge from the tailrace and from the tailrace deck. It would be done at the same time as the work between the powerhouse and the CBL. Work in this area would consist of the following tasks.

- Fabricate tailrace dewatering bulkheads and fishway entrance gate frames. Work on this would begin with notice to proceed.
- Install transverse bulkhead in the fish collection channel between Units 14 and 15. See Plate 4.
- Construct access to the unwatering pumps at the northwest end of the powerhouse.
- Dewater the fish collection channel.
- Install larger auxiliary water system supply gate and increase floor diffuser openings.
- Relocate the fishway entrances from Skeleton Unit 20 to Generator Unit 16.
- Relocate the fishway stilling well and bubbler to Unit 16.
- Reroute utilities in the 150 and 168 Galleries.

- Demolish the tailrace deck and walls on either side of the collection channel down to elevation 52 meters. The beams and sections of walls beneath them would be left in place to carry the hydrostatic forces from the tail water. See Plates 2 and 3A.
- Plug the ends of the 150 and 168 Galleries. Three plugs would be constructed at: 1) the entrance to the unwatering pumps at the north end of the 168 Gallery; 2) 168 gallery at the dividing wall between Units 18 and 19; 3) 150 gallery at the dividing wall between Units 18 and 19. The construction joint between Units would be incorporated into the three plugs.
- Place the dewatering bulkheads and supports. The bulkheads would be placed over the west wall of the fish collection channel from the north end of Unit 18 to the north end of Unit 20. Bracing to take the hydrostatic loads would be placed as shown on Plate 3A. Some holes in the wall on the east side of the collection channel would have to be cut to accommodate the beams.
- Remove the tailrace deck and the walls on either side of the collection channel down to elevation 46.0 meters.
- Construct the stemwalls, chutes, and chute guidewalls. These can only be constructed between the bracing supports. When the concrete for the stemwalls and chutes has cured new supports would be placed and anchored to the spillway chutes or guidewalls. Then the original bracing anchors would be removed and the spillway chutes would be finished.
- Construct the tailrace spillway bridge. See Plate 7 and 8.
- Remove the tailrace dewatering bulkheads.

f. Gallery Construction and Utilities Relocation

Mining of the new gallery will occur in conjunction with Phase 1 construction of the spillway chutes. The mined tunnels in Skeleton Unit 18 and the overflow section between Skeleton Unit 20 and Spillway Bay 20 would be completed prior to the new spillway gallery constructed at the end of Phase 1. Once the new spillway gallery is complete, the existing utilities located in Gallery 265 will be relocated to the mined tunnel and new spillway gallery. These utilities include an 8-inch diameter potable water line, 3-inch diameter compressed air line, and electrical power and control wiring. Drainage pipes and the wash water supply pipes will be capped. With completion of the utilities relocation, Phase 2 work east of the CBL line may begin. The existing 265 Gallery in Unit 18 can be backfilled with concrete at this point.

g. Phase 2 - Work in Powerhouse East of Construction Base Line

The Phase 2 work begins with fabrication of the upstream dewatering bulkhead. As discussed in Section 8, the upstream dewatering bulkhead is designed to span across approximately 1-1/2 of the new chute bays. This span was required to meet structural design criteria and provide adequate bearing capacity on the existing concrete and the new chute walls.

Figure 11.1 breaks the Phase 2 construction work into seven movements of the dewatering bulkhead. See Plate 3B. Beginning at Chutes 20a and 20b, the dewatering bulkhead will be placed to allow construction of the chute guide wall between Chutes 20a and 20b (Bulkhead Move No. 1). Mass concrete east of the Construction Base Line will be demolished and the new guide wall constructed. Temporary stoplogs will be installed in Chutes 20a and 20b and the dewatering bulkhead relocated to Chute 20a (Bulkhead Move No. 2). The mass concrete will be demolished for the north pier wall and new concrete placed. Once the pier wall is constructed, Chute 20a and the spillway bridge east of the CBL will be constructed. The new tainter gate will be installed in Chute 20a. See Section 8 for a more complete description.

With the completion of Chute 20a including the spillway bridge, these construction activities will be repeated for Chutes 20b, 20c, 19a, 19b, and 19c moving from north to south (Bulkhead Moves No. 3 through 7). The sequencing plan allows access to the immediate work area from the new spillway bridge. As construction advances, the tainter gates will be installed. The temporary stoplogs will be relocated to allow movement of the dewatering bulkhead to each successive chute. The concrete placement schedule is based on providing sufficient cure time to allow the concrete to reach a minimum of 20.7 megapascals (3000 psi) compressive strength. This strength is required to support the dewatering bulkhead.

Spill will take place at the main spillway between April 15 and September 1. During periods of spill bulkhead placement at Unit 20 will not be allowed. Therefore, Bulkhead Move No. 1 will be delayed until September 1, 2004. It is assumed that Bulkhead Move No. 4 can take place during spill in 2005 since it straddles Units 19 and 20.

Once Phase 2 construction work is complete, the gate hoists will be installed as well as the power supply and instrumentation for the hoists. A single bulkhead will be fabricated which will be used to test each gate in a dry condition. The full movement of the gate will be tested prior to removing the bulkhead. Wet tests will be conducted to verify the operation of the gate. The downstream cofferdams will have been removed allowing full operation of the spillway chutes. Depending on the concrete cure time the gates in the northern chutes could be tested before work on the southern chutes is complete.

11.3 Unit 20 Surface Bypass Spillway

The preliminary construction schedule for Skeleton Unit 20 Surface Bypass Spillway is presented in Figure 11.2. Construction of the tailrace cofferdam and dewatering system is essentially identical to the full surface option. Demolition activities are limited to Skeleton Unit 20. Mining of the new gallery is assumed to be identical to the full surface bypass option except that the horizontal angle in the tunnel at elevation 83.21 meters would be in the area between Bays 19 and 20 instead of Bays 18 and 19. This approach would provide the flexibility to construct the Skeleton Unit 19 Surface Bypass in the future.

The construction of the Unit 20 Surface Bypass Spillway involves the same construction activities as the full surface bypass spillway. As shown in Figure 11.2, construction in Phase 1 will have the same tasks as in the full surface bypass spillway.

The schedule for Phase 1 construction west of the dam CBL will be decreased by approximately 50%. With only Skeleton Unit 20 surface bypass spillway to construct, demolition and concrete placement will be reduced. The construction activities and time periods for mining the new gallery will be identical to the full surface bypass spillway.

The total construction effort east of the CBL, Phase 2, will be reduced by about 40% with a corresponding decrease in the project schedule.

Work in the forebay on the upstream face of the dam at Unit 20 cannot take place during spill for safety reasons. After Bulkhead Move No. 1 work will have to be suspended until spill is stopped around September 1.

A more complete description of the differences between the Unit 20 Surface Bypass Spillway and the Full Surface Bypass Spillway can be found in Section 9.0.