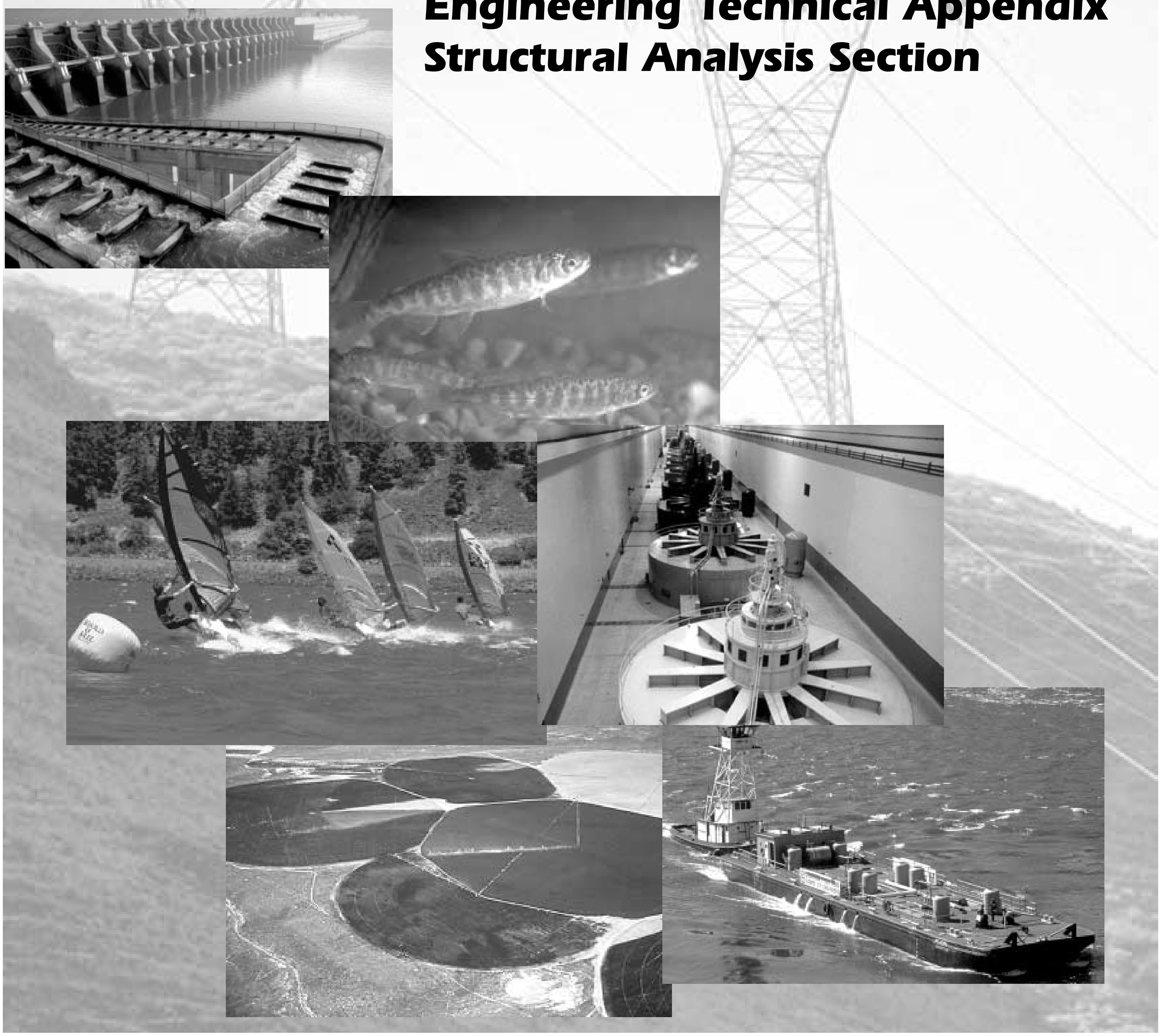
US Army Corps of Engineers ®
Portland District

Salmon Recovery through John Day Reservoir

John Day Drawdown
Phase I Study

Engineering Technical Appendix
Structural Analysis Section



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JOHN DAY DRAWDOWN STUDY

Structural Evaluation

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LIST OF ACRONYMS USED IN THIS REPORT

AASHTO	American Association of State Highway and Transportation Officials
ABMA	American Bearing Manufacturer Association
ANSI	American National Standards Institute
ARBED	Proprietary Piling Company
ASTM	American Society for Testing an Materials
AWS	Auxilliary Water Supply
BLH	Baldwin-Lima Hamilton Corporation
BPA	Bonneville Power Authority
CBL	Construction Baseline
cfs	Cubic feet per second
DM	Design Memorandum
EIS	Environmental Impact Statement
ESBS	Extended Submersible Bar Screen
EM	Engineering Manual
fps	feet per second
FSL	Feet Sea Level
FTC	Fish Transportation Conduit
HDC	Hydroelectric Design Center
HEC-RAS	Hydrologic Engineering Center water surface profile program for rivers
HP	Hewlett Packard
ICEA	Insulated Cable Engineers Association
IEEE	Institute of Electrical and Electronic Engineers
IES	Illuminating Engineering Society
ISA	Instrument Society of America
JBS	Juvenile Bypass System
khp	kilo-horsepower
KW	Kilowatt
MCE	Maximum Credible earthquake
msl	mean sea level
NEC	National Electrical Code
NEMA	National Electrical Manufacturers Association
NFPA	Life Safety Code
NGVD	National Grid Vertica Datum
NMFS	National Marine Fisheries Service
NSFL	North Shore Fish Ladder
OBE	Operating Basis Earthquake
OSHA	Occupational Safety and Health Act
PMF	Probable Maximum Flood
PSF	pounds per square foot
RM	River Mile
SCO	Synchronous Condensing Operation
SNL	Speed No Load

SSFL	South Shore Fish Ladder
STS	Submersible Travelling Screen
TEFC	totally enclosed, fan-cooled
UL	Underwriters Laboratory
USGS	United States Geological Service
V	Velocity
VAR	Volts Amp - Reactive
VBS	Vertical Barrier Screen
WES	Waterways Experiment Station

EXECUTIVE SUMMARY

Four alternatives were considered for accomplishing the drawdown at the John Day Project. Impacts to the project were identified, and concept level designs were developed for each affected project feature for each alternative.

The main criteria for developing the designs are:

- Fish passage must be in operation at all times except the in-water work period
- All in-water work must be accomplished from 1 December through 1 March
- All alternatives must pass fish for river flows from 80,000 cfs up to the 10-year flood (515,000 cfs)
- Cofferdam and dewatering design flood is the 10-year event (515,000 cfs)
- River traffic must be able to pass the project from 80,000 cfs to 800,000 cfs

The concept designs for each project feature in each alternative are described below.

Alternative 1 - Drawdown to Spillway Crest without Flood Control

In this alternative the gates will be raised and the river will flow uncontrolled over the existing spillway. Modifications are required at several project features due to the lower reservoir level. These include:

- Fish Ladders – Both north and south shore fish ladders will be rebuilt with a vertical slot fishway and two exits to accommodate the wider range in forebay water levels. New pump motors will be required for the auxiliary water supply on the south shore.
- Juvenile Bypass Systems – It is envisioned that new fish collection conduits will be bored in the dam. New extended length bar screens will also be required. Outside the dam the transportation channel, dewatering structure, fish evaluation facilities, and outfall, as necessary, will all be rebuilt at a lower elevation.
- Navigation Lock – The upstream sill on the navigation lock will be lowered and a new gate installed. New lock fill intakes with fish exclusion screens would also be built.
- Hydroturbines – New auxiliary systems would be required, but the existing turbines could be used after drawdown.

This alternative would require eight years to plan and design and about five and one half years to construct.

Alternative 2 - Drawdown to Spillway Crest with Flood Control

In this alternative the gates will be raised and the river will flow uncontrolled over the existing spillway. However, the gates would be lowered for flood control operations about once every two to five years to provide up to 500,000 acre-feet of storage. Alterations are required at several project features due to the lower fluctuating reservoir level. These include:

- Fish Ladders – Both north and south shore fish ladders will be rebuilt with vertical slot fishways with four exits. The two lower exits operate similar to those in Alternative 1. The two upper ones will accommodate the higher forebay water levels during flood control operations. New pump motors will be required for the auxiliary water supply on the south shore.
- Juvenile Bypass Systems – It is envisioned that new fish collection conduits will be bored into the dam similar to those in Alternative 1. However, four new orifices will be required between the gate well and fish collection conduit. Outside the dam the transportation channel, dewatering structure, fish evaluation facilities, and outfall will all be rebuilt at a lower elevation. These will have higher side walls to contain the greater fluctuation in forebay water elevations.
- Navigation Lock – The navigation lock modifications will be the same as those in Alternative 1.
- Hydroturbines – New auxiliary systems would be required, but the existing turbines could be used.

This alternative would require eight years to plan and design and about six years to construct.

Alternative 3 - Drawdown to Natural River without Flood Control

In this alternative the river will be drawn down to approximate natural river conditions. Fish passage and navigation would be accomplished through the breach in the dam. The criteria for this alternative is to provide an average velocity of 10 fps or less through the opening in the dam during the 10-year flood. The size of the opening was calculated using a backwater model by adjusting the opening width until the required velocity was achieved.

It is envisioned that the dam will be removed in two construction stages, which are described below.

- Stage 1 – During Stage 1 construction, a cofferdam will be constructed around the center of the dam. An embankment cofferdam would be built upstream since the water is too deep to allow use of a cellular sheetpile cofferdam, which would be employed downstream. Inside the cofferdam Spillway Bays 13 through 20, the non-overflow section, and powerhouse Units 17 through 20 will be removed. The spillway would be removed down to elevation 135, and the powerhouse units down to elevation 128. The draft tubes would be filled with concrete up to elevation 128. Fish passage baffles will then be built in the area of Unit 17. These baffles would provide upstream passage through the opening in the dam during Stage 2 construction. The reservoir will then be drawn down to free-flowing conditions.
- Stage 2 – The Stage 2 cofferdam will be built to encircle the northern portion of the spillway. This cofferdam will be cellular sheetpile. The remainder of the spillway will be removed to elevation 135. At this time a sheetpile cell will be built in the upstream navigation channel to dewater the upstream portion of the navigation lock. The upstream sill will be lowered to the

floor of the lock and a new upstream lock gate installed. Lock filling and emptying valves would be incorporated into the bottom of the two lock gates.

No modifications of the fish ladders will be required since fish will pass upstream through the breach. The remainder of the powerhouse and the Juvenile Bypass System will be abandoned.

It is estimated that the studies and design for the drawdown will take about eight years to complete, and that construction will take four and one half years.

Alternative 4 - Drawdown to Natural River with Flood Control

In this alternative the river will be drawn down to approximate natural river conditions. However, piers and gates will be built to provide for 500,000 acre-feet of flood storage. The width of the new spillway is based on achieving an average velocity through the breach of less than 10 fps at the 10-year flood.

It is envisioned that the construction would take place in three stages as described below. The existing forebay water level would be maintained during Stages 1 and 2.

- Stage 1 – A cofferdam would be built around the center of the dam. An embankment cofferdam would be built upstream since the water is too deep to allow use of a cellular sheetpile cofferdam, which would be employed downstream. Spillway Bays 14 through 20, the non-overflow section, and powerhouse Units 15 through 20 would be removed. A new spillway would be built with 50 openings and 12-foot wide piers. The southern 10 new spillway bays will be fitted with temporary ogee crests, 50 feet high, to accommodate flows and fish passage during Stage 2 construction when these spillways will be used to maintain the forebay water level.
- Stage 2 – A cofferdam would be built around the northern part of the existing spillway. The forebay would be maintained at existing levels during Stage 2 construction by use of the gates completed in Stage 1. This would again require an embankment type cofferdam upstream. The northern portion of the spillway (Bays 1 through 13) would be removed and new piers built and gates installed. The navigation lock would be modified by lowering the upstream sill to the floor of the lock and installing a new 105-foot high gate. New filling and emptying valves would be installed at the bottom of the lock gates. The reservoir would then be drawn down and the cofferdam removed.
- Stage 3 – A small cofferdam will be completed around part of the north shore fish ladder and the ladder will be rebuilt. A ladder is necessary to pass fish during flood control operations when the forebay water level will be raised as much as 50 feet. The temporary ogees will be removed from one spillway bay at a time using bulkheads for dewatering.

It is estimated that the studies and design for the drawdown will take about eight years to complete, and that construction will take ten and one half years.

Drawdown Effects at the McNary Project

All four alternatives are assumed to have an identical effect on the tailwater and fish passage at the McNary Dam for flows less than 600,000 cfs. The McNary Dam was in operation prior to the completion of the John Day Dam. Consequently, modifications were required to the fish passage facilities to provide for their continued operation when the John Day pool raised the tailwater at the McNary Project. These modifications were outlined in *John Day Design Memorandum No. 30, Modifications to McNary Fish Facilities.* This memorandum was used as the basis for anticipating the changes required to ensure fish passage at McNary Dam in the event that the tailwater returns to pre-John Day Project levels. Changes required include reestablishing sill elevations at the fish ladder entrances, modifying the auxiliary water systems, adjusting several weir crests in the lower reaches of the north and south shore fish ladders, modifying juvenile fish return outfalls, and perhaps some minor modifications to the fenders and fish loading system at the barge facilities. Additionally, all alternatives for the drawdown would strand the spill deflectors that were installed after the John Day Project was completed. The deflectors would be removed and relocated. No changes to the hydroturbine operations are anticipated.

SECTION 1 INTRODUCTION

1.1 General

In 1991, Snake River wild sockeye, spring/summer Chinook, and fall Chinook salmon were proposed by National Marine Fisheries Service (NMFS) for endangered or threatened status under provisions of the Endangered Species Act. The National Marine Fisheries Service, in *Reasonable and Prudent Alternative Action #5* of its *Biological Opinion on Operation of the Federal Columbia River Power System*, recommended that the Corps of Engineers investigate the feasibility of lowering John Day reservoir to spillway crest.

Lowering the John Day reservoir may decrease juvenile salmonid travel time and create a more natural shoreline and benthic community structure. Furthermore, lowering the reservoir may result in conditions similar to those of the unimpounded reach of the Columbia River where mainstem spawning populations of fall Chinook salmon appear to be healthy and productive. It has been proposed that drawdown of the 76-mile John Day reservoir may provide substantial improvements in migration and rearing conditions for juveniles by increasing river velocity, reducing water temperature and dissolved gas, and restoring riverine habitat. Drawdown of John Day may improve spawning conditions for adult fall Chinook by restoring spawning habitat and the natural flow regimes needed for successful incubation and emergence.

The regional goals for a drawdown of John Day reservoir, as identified in NMFS' draft *Recovery Plan for Snake River Salmon*, the *Tribal Restoration Plan*, and the Northwest Power Planning Council's Fish and Wildlife Program are to: (1) improve migration and rearing conditions for juvenile spring, summer, and fall chinook, sockeye, and steelhead, (2) reduce water temperature and total dissolved gas to comply with Clean Water Act criteria and standards, and (3) improve spawning conditions for fall chinook.

In response to direction provided in the Energy and Water Development Appropriation Bill, 1998, the Corps of Engineers prepared a scoping document for studying drawdown of the John Day reservoir to spillway crest and natural river. Normal reservoir elevation is 265 feet. MSL; operation at spillway crest would result in a reservoir elevation of about 220 feet MSL; and natural river elevation would be about 170 feet MSL. The scoping document recommended a two-phased approach to the study. Phase I of the study will use existing information to evaluate biological, social and economic benefits and costs of the two proposed alternatives, spillway crest and natural river, and identifies the potential physical impacts of drawdown. The Phase II study, if conducted, will be a multi-year study in which the Corps will develop a feasibility-level evaluation of all the benefits, costs and physical impacts associated with a range of reasonable drawdown alternatives. These alternatives would be identified during the Phase II scoping process.

The Phase I report has several components. These include fishery, economic, hydrology and hydraulic, reservoir, wildlife, recreation, irrigation, hydropower, sediment and engineering studies. This study provides information on the impacts of the drawdown on the structural features at the John Day Dam and the fish passage and hydropower facilities at the McNary Project.

1.2 Purpose

The purpose of the Structural Alternatives Appendix is to define the configuration of the project features and the construction and operational requirements at John Day Lock and Dam for each of the four drawdown alternatives. The study provides adequate information from which to make a decision to continue with Phase II. Since the Structural Alternatives Appendix will provide input to the overall Phase I report, its information is coordinated with that of the other study components.

1.3 Scope

The Structural Alternatives study is an engineering report to identify impacts at the John Day and McNary project facilities and structures except for those in the reservoir area. The four alternatives are:

1. Spillway Crest Drawdown
2. Spillway Crest Drawdown with Flood Control
3. Natural River Drawdown
4. Natural River Drawdown with Flood Control

The assumptions, constraints and criteria used in this study are given in Section 2. The analysis and its results for Alternatives 1, 2, 3, and 4 are given in Sections 3, 4, 5, and 6, respectively. Section 7 describes impacts at the McNary Project. The project components studied in one or more of the alternatives are the powerhouse, spillway, navigation lock, embankment section, and upstream and downstream fish passage. See Plate 1 for the location of the major project features. For the natural river drawdown alternatives an analysis of the channel through the dam's location was made to insure adequate passage of fish and river navigation traffic. Quantity calculations were assembled for use in developing a cost estimate. The cost estimate will be provided separately for the Phase I report.

All work in the Structural Alternatives Appendix was performed by the CH2M HILL/Montgomery Watson Joint Venture except for the turbine operation subsections, which were written by the Hydroelectric Design Center (HDC). The Geotechnical Engineering Branch of the Portland District supplied geotechnical support for the report. Geotechnical information is supplied in another appendix to the Phase 1 report.

The Phase I Study, of which this Appendix is a part, is a reconnaissance level study. Therefore, all designs are conceptual in nature, and there could be other more efficient designs. However, the designs in this report have been developed to determine the impacts, potential design, construction and operational characteristics of a drawdown of the John Day reservoir.

1.4 Authorization

The John Day Lock and Dam were 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.

This study is authorized under Contract No. DACW57-97-D-0004, Delivery Order No. 0008 between the CH2M Hill/Montgomery Watson Joint Venture and the U. S. Army Corps of Engineers, Portland District.

SECTION 2 ASSUMPTIONS AND CONSTRAINTS

2.1 General

2.1.1 Purpose

The purpose of this section is to set forth a detailed list of all assumptions and constraints used throughout the Structural Alternatives Study. This section states the assumptions, constraints, and criteria used to develop the concept designs for changes at both John Day and McNary under all four of the alternatives.

The assumptions and constraints were derived from a variety of sources. Fisheries criteria were obtained from standard operating criteria at the dams and from agency criteria for fish passage. The structural criteria were obtained from the Appendix A of Feature Design Memorandum No. 52, John Day Lock and Dam Surface Bypass Spillway.

2.1.2 Background

The purpose of the Structural Alternatives Study is to define the construction and operational requirements for each of the four drawdown alternatives. To insure an equal treatment of all alternatives, a consistent set of assumptions, constraints, and criteria are required at the outset. These will involve all features of the John Day Project and some features of the McNary Project. The features affected are the powerhouse, spillway, navigation lock, embankment section, and both upstream and downstream fish passage. Therefore, hydraulic, biological, and structural assumptions and criteria must be considered.

2.2 Design Life

The changes contemplated for the projects under the four alternatives considered in this report will have the same design life as that of the existing structures. This is estimated to be about 60 years.

2.3 Hydraulic Assumptions

2.3.1 General

The hydraulic assumptions state the water levels and flows used as constraints in developing the concept designs for the features at both the John Day and McNary Projects. The flows and water levels are divided into two types, maximum design and operating. The maximum design values are those used in designing the structure and assessing the stability and forces acting on it. The operating values are those for which the structure is designed to operate and perform its intended purpose. These can be both minimum and maximum values.

2.3.2 Flows

The following are the design flows for which the structures are designed.

Maximum Design Flows	
Standard Project Flood	1,060,000 cfs
Spillway Design Flood	2,250,000 cfs
Operating Flows	
Maximum for Fish Passage	515,000 cfs (10-Year Flood)
Minimum for Fish Passage	80,000 cfs
Maximum for Navigation	800,000 cfs
Minimum for Navigation	80,000 cfs

The maximum flow for upstream fish passage was originally set at the five year flood event of 450,000 cfs to be consistent with the Lower Snake Drawdown Study Assumption. During the 30 percent reivew, the criteria was changed to the 10 year event of 515,000 cfs. This was decided due to the fact that the five year event duration was considered to be too long and could preclude fish run survival. The 10 year event duration was near one week and it was decided that it would not severly impact the run in any given year.

2.3.3 Water Levels

The operating water levels for fish passage are shown on Table 2-1 and those for navigation are shown on Table 2-2. See Subsection 2.5.3 for water surface elevation extremes used for structural design.

Operating Water Levels

The operating tailwater levels at both John Day and McNary Dams were taken from backwater hydraulic analyses being performed in support of the Phase I drawdown study. The water levels upstream of the dam were derived from the same backwater analyses for the drawdown to natural river. The upstream water levels for drawdown to spillway crest were taken from two sources: 1) a hydraulic model study (Northwest Hydraulic Consultants, September 1998) for flows up to about 300,000 cfs, 2) the spillway rating curve on Plate 54 of the *John Day Design Memorandum No. 16*.

The maximum operating water surface elevation for the forebay in Alternative 1 depends on the number of units operating during the maximum flow of 515,000 cfs. Each unit can take about 20,000 cfs at heads expected after drawdown. So, for each operating unit the flow over the spillway is reduced by about 20,000 cfs, and the head on the uncontrolled spillway is reduced. Since it is likely that several units will be operating when there is a flow of 515,000 cfs in the river, a value of 230 feet was selected as a design value for the maximum forebay operating pool. This corresponds to a condition with 10 units operating.

The maximum John Day pool elevations for flood control were obtained by assuming a flow of 515,000 cfs to be flowing in the river before flood control operations start. Under Alternative 2, the water surface elevation upstream of the dam would be 237 feet, assuming no turbines are operating. Under Alternative 4, the upstream water surface would be 169 feet Then flood control would commence and achieve the required 500,000 acre feet of storage with the flow at 515,000 cfs. The resulting pool elevations would be 252 feet and 223 feet for spillway crest and natural river drawdown, respectively. Flood control operations are typically triggered by bank-full conditions at the Vancouver, WA gage which normally correspond to a discharge of 450,000 cfs as measured at the USGS gage at The Dalles. However, under certain conditions, such as high tributary flows, bank-full conditions can occur at discharges as low as 360,000 cfs (2 yr. event). This corresponds to a tailwater elevation of 165 feet at John Day.

Table 2-1
Fish Passage Design Operating Water Levels

	Alternative 1 (ft msl)	**Alternative 2** (ft msl)	**Alternative 3** (ft msl)	**Alternative 4** (ft msl)
Maximum				
John Day Tailwater (1)	169	169	169	169
John Day Pool (Headwater)(2)	230	252	--	223
McNary Tailwater	270	270	270	270
Minimum				
John Day Tailwater	155	155	155	155
John Day Pool (Headwater)	213	213	--	--
McNary Tailwater (3)	251	251	251	251

(1) Assumes a flow of 515,000 cfs and a water surface at The Dalles of 160 ft msl.
(2) Elevation 230 is based on 10 units operating.
(3) The tailwater elevations are the same for all alternatives for flows less than 600,000 cfs as verified by backwater modeling.

Water levels for different discharges used for design of all features at the John Day Dam were taken from the same sources as those for fish passage, and are included in the Navigation and Flood Control Appendices. Additional assumptions used in the design for navigation are shown in Subsection 2.9.

For the purposes of this study, fish passage will be provided for upstream and downstream passage for the full year. Passage will also be provided all year during construction except that passage may be suspended during the in-water work period from December 1 through February 28.

Table 2-2
Navigation Operating Water Levels

	Alternative 1 (ft msl)	Alternative 2 (ft msl)	Alternative 3 (ft msl)	Alternative 4 (ft msl)
John Day, Minimum Tailwater	155	155	155	155
John Day, Minimum Operating Pool	213	213	155 (1)	155 (1)
John Day, Maximum Operating Tailwater	175	175	175	175
John Day, Maximum Operating Pool	230	252	177 (1)	223

(1) In Alternatives 3 and 4, there is no operating water surface for these values. Those shown are for natural river conditions.

2.4 Biological Assumptions

2.4.1 General

This section deals with biological and fish behavior characteristics of the target species, both juvenile and adult, at the John Day Project. The assumptions stated below deal with seasonality of passage at John Day, swimming speeds, and project operational criteria.

2.4.2 Juvenile Passage Period

2.4.2.1 Seasonal Timing

Table 2-3 shows the time frame for juvenile fish migration at the John Day Dam as given in the John Day Dam section of the *Fish Passage Plan for Corps of Engineers Projects, March 1998*. The wild and hatchery steelhead data were consolidated.

Table 2-3
Juvenile Passage Timing

% Past Project	*Year/Date of Passage*							*Earliest/Latest Passage*
	1991	1992	1993	1994	1995	1996	1997	
Yearling Chinook								
10%	4/26	5/2	5/6	5/2	4/29	4/21	4/20	4/20
90%	6/7	6/10	6/1	6/18	5/29	5/28	5/28	6/18
Subyearling Chinook								
10%	6/6	6/24	6/21	7/8	6/8	5/12	5/1	5/1
90%	8/15	8/15	8/17	8/2	7/24	8/19	8/16	8/19
Steelhead (all)								
10%	5/4	5/3	5/5	5/3	5/5	4/26	4/23	4/23
90%	5/29	5/28	5/26	6/1	5/25	5/26	5/25	6/1
Coho								
10%	5/11	5/2	5/9	5/12	5/8	4/27	4/30	4/27
90%	6/4	5/27	5/30	5/29	5/21	5/21	6/9	6/9
Sockeye								
10%	5/16	5/8	5/16	5/11	5/9	5/3	5/10	5/3
90%	6/1	5/27	5/31	6/5	5/26	6/3	6/21	6/21

2.4.3 Adult Passage Period

2.4.3.1 Seasonal Timing

Table 2-4 shows the period of earliest and latest dates of peak adult passage for the years between 1968 and 1997. These data are taken from the *Fish Passage Plan for Corps of Engineers Projects, March 1998*.

Table 2-4
Adult Migration Timing

Species	*Migration Period*
Spring Chinook	4/17 – 5/22
Summer Chinook	6/7 - 8/2
Fall Chinook	9/2-9/25
Steelhead	9/6-10/6
Sockeye	6/23-7/10
Coho	8/4-10/15

As defined in the John Day Project section of the *Fish Passage Plan (March 1998)*, the in-water work period is defined as December 1 through February 28. The ladders and JBS facilities can be taken out of service during this period. In addition, all work in the water will be scheduled for this period.

2.4.4 Juvenile Passage Criteria

2.4.4.1 Swimming Speed

Swimming speed values were obtained from the *Fisheries Handbook of Engineering Requirements and Biological Criteria (1991)*.

Three aspects of swimming speed are considered in the criteria for design of fish facilities. They are:

- Cruising - a speed that can be maintained for long periods of time (hours).
- Sustained - a speed that can be maintained for minutes.
- Darting - a single effort burst of speed that is not sustainable.

The assumed design criteria for swimming speeds is shown in Table 2-5. Because the sustained speed is the most applicable swimming speed for fish traveling through the Project after drawdown, the sustained speeds for the affected species were assembled. These values provide the basis for the facility design criteria.

Table 2-5
Juvenile Swimming Speeds

Species	*Speed (fps)*
	Sustained
Chinook (2")	0.5-1.2
Chinook (>2")	1.0-2.1
Coho (2")	0.5 – 1.2
Coho (>2")	1.0 – 2.1
Sockeye (5")	1.8 – 2.2
Steelhead	1.8-2.2

2.4.4.2 John Day Juvenile Passage Operation

The following operating criteria were adapted from the *Fish Passage Plan (March 1998)* as listed for John Day Project.

- Gatewell drawdown shall be maintained at <1.5 feet
- Units 1-5 shall be raked biweekly between 4/1 and 7/1. At the same time that units 1-5 are being raked, units 6-10 and units 11-16 shall be raked alternately.
- Forebay debris accumulation of 500 ft^2 or more shall be removed within 48 hours.

- The trash shall be raked weekly at affected units in forebay if debris loads are obvious.
- Additional raking shall occur if differential across the trash racks reaches 1.5 feet During raking, gatewell orifices of the unit being raked shall be closed; furthermore, the Submerged Traveling Screen (STS) units will be run continuously through the raking.

2.4.5 Adult Passage Criteria

Swimming speed values were obtained from the *Fisheries Handbook of Engineering Requirements and Biological Criteria (1991)*.

2.4.5.1 Adult Swimming Speed

Table 2-6 lists the recorded swimming speeds of several species of adult fish.

Table 2-6
Adult Swimming Speeds

Species	*Speed (fps)*		
	Cruising	Sustained	Darting
Chinook	0 – 4	4 – 11	11 – 22
Coho	0 – 4	4 – 11	11 – 21.5
Sockeye	0 – 4	4 – 11	11 – 22
Steelhead (2' - 2.7')	0 - 5	5 - 14	27

2.4.5.2 John Day Adult Passage Operation

The following operating criteria were adapted from the *Fish Passage Plan (March 1998)* as listed for the John Day Project. We will use these as the criteria for this study.

- Slot width, vertical slot ladders: 1.25 feet (minimum)
- Energy dissipation per pool: 4 to 5 ft-lb/s/cf
- Drop between pools, vertical slot ladder: 1.0 feet target, operational limits 0.4 feet to 1.5 feet corresponding to average slot velocities of 5 fps to 10 fps, respectively
- Floor slope: 1.0 feet per pool
- Head on all fish ladder entrances shall be between 1.0 feet and 2.0 feet with a preferred head of 1.5 feet.
- A transportation velocity of 1.5 to 4.0 fps (2.0 fps preferred) shall be maintained in all transportation channels and at the lower ends of the fish ladders which are below the tailwater.
- A maximum head of 0.5 feet shall be maintained on attraction water intakes and trashracks at all ladder exits. A maximum head of 0.3 feet shall be maintained on all picket leads.
- Staff gauges and water level indicators shall be calibrated and readable at all water levels encountered.

- Main entrance weir depths shall be 8.0 feet or greater below the tailwater. Set gates at 8.5 feet when possible.
- Forebay at The Dalles Dam shall be controlled as necessary to regulate the tailwater at John Day for fish ladder operation
- Count station crowders shall remain in operating position while visual counting and/or video taping. The crowder shall be closed to allow count slot width to be no less than 15 inches.

2.4.5.3 Dam Breach Flow Velocities (Alternatives 3 and 4)

To set the minimum size of the breach width required to pass upstream migrants, a flow velocity of 10 feet per second was selected. Although this is faster than a fish can swim through the length of breach, it was assumed that the boundary layer on the floor and walls of the breach would provide suitable velocities for passage.

2.5 Structural Design

2.5.1 General

For conceptual design of structures considered in the drawdown study, stability analyses and general structural computations are required. To be consistent with the Corps' design criteria and other structures at the John Day and McNary Projects, the following criteria are used in this study. These design criteria were taken from *John Day Project Design Memorandum No. 52*.

2.5.2 Structural Materials

2.5.2.1 Rock Bearing Capacity: 69 kips per sq feet

2.5.2.2 Sliding Safety Factor: 2.0 normal operating, 1.7 for OBE

2.5.2.3 Overturning Stability Criteria: Normal operation resultant in middle 1/3 of foundation, OBE not to exceed 75 percent of bearing capacity

2.5.2.4 Concrete 28-day compressive strength for all concrete except spillway slab: f'c=4,000 psi

2.5.2.5 Concrete 28-day compressive strength for all spillway slab: f'c=6,000 psi

2.5.2.6 Reinforcing steel : ASTM 615 Grade 60

2.5.2.7 Structural steel : ASTM A36

2.5.2.8 Steel plates : ASTM A242

2.5.2.9 Structural bolts : ASTM A325

2.5.2.10 Post tensioning rods : Minimum ultimate strength 145,000 psi

2.5.3 Water Surface Elevations

These apply to all alternatives unless otherwise noted.

2.5.3.1 Maximum operating reservoir level elevation: 268 feet (Alternatives 1 and 2)
: 223 feet (Alternative 4)

2.5.3.2 Maximum pool elevation : 276.0 feet (Alternatives 1 and 2)
: 228.0 feet (Alternative 4)

2.5.3.3 Maximum John Day tailwater elevation : 202.0 feet

2.5.3.4 Maximum McNary Tailwater elevation: 310.0 feet

2.5.3.5 Minimum John Day tailwater elevation : 155.0 feet

2.5.3.6 Minimum McNary tailwater elevation: 251.0 feet.

2.5.4 Design Loads

2.5.4.1 General

a. Densities
   - Concrete : 150lbs/ft^3
   - Steel : 490 lbs/ft 3
   - Aluminum : 165 lbs/ft^3

2.5.4.2 Spillway Bridge

a. Dead Load: Weight of the structure
b. Deck Crane: Crane Capacity of 175 kip plus crane dead load. 25 percent crane impact
c. AASHTO H2O Truck, 30 percent Truck impact
d. Flat Bed Tractor-Trailor carrying one concrete stoplog. Loading as described in *Design Memorandum No. 16*.

2.5.4.3 Spillway Gate

a. Closed gate with water at normal pool
b. Closed gate with water at normal pool plus ice load of 10 kip/feet with a 1/3 stress increase
c. Water at normal pool with wave pressure based on a 6-foot wave with a 1/3 stress increase
d. Gate hoist
e. Post tensioned trunnion anchor

2.5.4.5 Wind : 30 PSF

2.5.4.6 Seismic : 0.1g for OBE
0.2g for MCE

2.5.4.7 Water : 62.5 lbs/ft^3

2.5.4.8 Ice : 10 kip/foot at water surface

2.5.5 Spillway Sectional Stability

2.5.5.1 Uplift Pressures

a. Uplift is applied over 100 percent of foundation area; the intensity being equal to the pressure of tailwater head at the toe, varying uniformly to a pressure at the heel equal to tailwater pressure plus 2/3 of the difference between the headwater and the tailwater pressures.

b. Uplift within the structure is applied over 100 percent of the horizontal section being investigated and the intensity 2/3 the amount obtained by considering uniform variation from full headwater pressure at the heel to full tailwater pressure at the toe.

2.5.5.2 Load Case

- Maximum Credible Earthquake (MCE), 20 percentg. Normal Operating Condition: Reservoir at elevation 268.0 feet with tailwater at elevation 155.0 feet, with uplift.

2.6 Cofferdams Design

2.6.1 Upstream Cofferdam

2.6.1.1 Design river flow: 515,000 cfs (10-year flood).

2.6.1.2 Reservoir elevation at maximum pool : 268.0 feet.

2.6.1.3 Normal reservoir pool elevation : 265 feet.

2.6.1.4 Wave forces shall be determined in accordance with the *Shore Protection Manual*.

2.6.1.5 Ice forces equal 10 kip/foot at water surface

2.6.2 Downstream Cofferdams

2.6.2.1 Normal tailwater range : elevation 156 feet to 165 feet.

2.6.2.2 Design flow for overtopping : 515,000 cfs (10-year flood).

2.6.2.3 Tailwater at 515,000 cfs : 169.0 feet with The Dalles pool at 160.0 feet.

2.6.2.4 Wave forces shall be determined in accordance with *Shore Protection Manual*.

2.6.2.5 Ice forces equal 10 kip/foot at water surface.

2.7 Mechanical Design

2.7.1 Spillway Crest Gates

2.7.1.1 Tainter gates will be designed in accordance with manuals *EM 1110-2-2702, Design of Spillway Tainter Gates*, and *EM 1110-2-1603, Hydraulic Design of Spillways*.

2.7.1.2 Wheel gates will be designed in accordance with manuals *EM 1110-2-2701, Vertical Lift Crest Gates,* and *EM 1110-2-1603 , Hydraulic Design of Spillways*.

2.7.1.3 Wave forces on the gates will be determined in accordance with *Shore Protection Manual*.

2.7.1.4 Ice forces on the gates will be 10 kip/foot

2.7.1.5 The gates will be fabricated from structural steel.

2.7.1.6 All gate seating surfaces will be provided with rubber seals.

2.7.1.7 Corrosion protection for gate components will consist of a high-build epoxy painting system. No supplemental cathodic protection system is to be provided.

2.7.1.8 Gate guides and sills will be provided with either an electric heat cable or freeze protection system.

2.7.1.9 All gate slots will be streamlined to minimize the impact on passing fish.

2.7.1.10 Gate lifting speed will be a maximum 1 foot per minute.

2.7.1.11 All gates will be designed for local-manual control.

2.7.1.12 Provision is made to permit dewatering of any gate.

2.7.1.13 The existing spillway gate gantry crane will be used to lift the gates.

2.8 Electrical Design

2.8.1 The new gate operators will be served with power from an existing substation located at the north end of the dam.

2.8.2 The control circuits for the new gate operators will be routed back to the main control room, located at the south end of the powerhouse

2.8.3 Grounding System: The existing grounding system from the North Substation will be used for accommodating the new equipment installation.

2.8.4 Motors: All motors will be in locations that are easily accessible for operation and maintenance. Enclosures for motors are to be totally enclosed, fan-cooled (TEFC). Service factors shall be 1.15. Motor insulation shall be Class F with the rise limited to Class B. Bearings are to be rated 100,000-hour ABMA B-10 life. Motor voltages will be 460 V, 3-phase for motors 0.4 kW (0.5 HP) and larger and 120 V, single-phase for motors less than 0.4 kW (0.5 HP). All 3-phase motors will be operated from combination motor starters with overload protection and 120 V control transformers. All motors will have local disconnects at the equipment.

2.8.5 The design shall conform to the latest edition of the following applicable standards and codes:

- National Electric Code (NEC-1996 Edition)
- Life Safety Code (NFPA-101-HB85)
- National Electric Safety Code (ANSI C2-1997)
- American National Standards Association (ANSI)
- Illuminating Engineering Society (IES)
- National Electrical Manufacturers Association (NEMA)
- Institute of Electrical and Electronic Engineers (IEEE)
- Instrument Society of America (ISA)
- Insulated Cable Engineers Association (ICEA)
- Occupational Safety and Health Act (OSHA)
- Underwriters Laboratory (UL)

2.9 Navigation Lock

The following assumptions are based on *John Day DM No. 16*, general knowledge, and inquiries of barge operating companies. Operating water surface elevations and flows are contained in Subsection 2.3.

River traffic will pass the John Day Project during flood control operations. The impact to lock traffic will be kept to a minimum during drawdown.

Drawdown to Spillway Crest

Downstream Approach Width	250 feet
Downstream Approach Channel El.	139 feet msl
Upstream Approach Width	80 feet
Upstream Approach Channel	147.0 feet msl
Water Depth at Sill	15.0 feet
Lock Width	86 feet wide at elev. 242.0

Drawdown to Natural River

Maximum Velocity for Navigation	5 feet per second
Minimum Channel Depth	15.0 feet
Minimum Channel Width	80 feet

SECTION 3 ALTERNATIVE 1 - DRAWDOWN TO SPILLWAY CREST WITHOUT FLOOD CONTROL

In this alternative the spillway gates can be raised out of the flow or removed so the river can run uncontrolled through the spillway. Implementation of this alternative will involve modifying or replacing the navigation lock, the adult and juvenile fish passage facilities, and the power generating equipment. There are no effects to the powerhouse structure or embankment section on the north and south sides of the project. Modifications to other project features might also be required. The options for modification or replacement of project features are described below.

3.1 North Shore Fish Ladder

3.1.1 North Shore Fish Ladder Description

The modification option chosen for the North Shore Fish Ladder (NSFL) consists of the full reconstruction of the fish ladder from the entrance channel to the new fish ladder exit. The ladder is designed for operation over the design operating water levels in the forebay and tailrace as described in Section 2 of this report (213 feet to 230 feet).

It is envisioned that a vertical slot fish ladder will be constructed with a low level and a high level fish ladder exits to allow operation over the range of headwater and tailwater fluctuations. The fish ladder provides 52 pools when using the low level outlet, and 60 pools when configured for the high level outlet. A plan view and section view are shown on Plates 3 and 4.

Under this alternative the existing fish ladder would be demolished from the existing construction joint between weirs 155 and 156. The south wall of the existing fish ladder and water supply conduit would be preserved and used for the new ladder. Diffusers Nos. 1 and 2 are maintained as well as the existing fish ladder entrance gates. The floor elevation would be lowered to elevation 148 (two feet) between the start of the new ladder and Diffuser No. 2. Relocation of the water supply conduit bulkhead gate at the west end of the ladder will be required. The existing ladder exit would be abandoned and filled with concrete. The existing trash fender and exit gates will be salvaged for reuse on the new ladder exit. The interpretive center above the NSFL will remain, but all counting and viewing facilities will be removed. Public access to the new viewing area will be provided.

The new ladder will be a vertical slot fish ladder similar to the ladder used at Hells Gate in British Columbia, Canada. Pools would be 14 feet long center to center and 16 feet wide. Fish ladder baffles are 24 feet high with two 15-inch (1.25 feet) wide slots. The floor slopes up from invert 148.0 at a one foot rise per pool (7.1 percent). The alignment of the ladder, shown on Plate 3, extends about 45 feet past the west end of the existing ladder wall. Taking two 90-degree bends, the new ladder follows the alignment of the old ladder and penetrates the dam near the existing ladder exit. A new counting and viewing facility is provided at pool 38 at invert elevation185. Counting and viewing facilities are assumed to be similar to the existing facilities. The fish ladder exit channel tapers to seven feet wide to maintain two fps transportation velocity. A 7 x 24 foot tunnel bored through the dam conveys fish to Exit No. 1, the new low level exit at invert elevation 200 feet. This is 26.5 feet below the existing fish ladder exit. When operating conditions dictate, the low level exit gate can be closed and fish guided through another eight pools. Here, another 7 x 24 feet exit channel and tunnel bored through the dam will convey the fish to Exit No. 2, the new high level exit at invert elevation 208 feet.

The auxiliary water system would be modified to fill existing Diffuser Nos. 3 through 15 with concrete. The water supply conduit bulkhead structure on the water supply conduit at the west end of the ladder would be relocated. No changes are required to the auxiliary water pumps.

The majority of the new fish ladder would be below the current deck level of 185 feet. Walls in the lower part of the ladder provide flood protection to this level. Starting at the counting and viewing structure the ladder would be elevated. Construction will be similar to the existing ladder.

Two other fish ladder options were considered at the 30 percent level: partial reconstruction with a new regulating section (1N2) and partial reconstruction with fish locks (1N3). The option of partial reconstruction with a new regulating section could not accommodate the large operating range and maintain self regulation. Partial reconstruction with fish locks was not brought forward because there were too many fish to handle. Experience at other projects indicated that lock systems were not as successful as ladders in passing large quantities of fish without delay and stress.

3.1.2 North Shore Fish Ladder Operation

Project operation described in the scope of work assumes a normal operation to maintain the pool at elevation 215 ±2 feet between April and August and at 220 feet +10 feet, -2 feet for the remainder of the year. It is assumed that some regulation of John Day pool level will be provided through the operation of the turbines. Even during the low pool operation in April through August, high flow conditions will occur which would raise the pool level above the "normal" levels indicated. The fish ladder can be operated using Exit No. 1 (52 pools) between forebay elevations 213 feet and 224 feet and tailwater elevations between 158 feet and 169 feet with a 1 feet drop between fish ladder pools. This requires coordinated operation of the John Day Project and The Dalles Project to maintain an overall water differential across the John Day Dam of 55 feet. This assumes the fish ladder entrance is operated so the head loss between the tailwater and the ladder is two feet (1.5 feet entrance and 0.5 feet channel loss). The 52 pool configuration would operate within stated operating ladder criteria (0.4 feet – 1.5 feet drops between pools) at overall water differentials from 48 to 58 feet depending on the tailwater levels. The 52 pool ladder using Exit No. 1 is limited to a high forebay elevation of 224 feet. Above this level the exit becomes submerged. In the 60 pool configuration the fish ladder can be operated between forebay elevations 215 feet and 230 feet, and tailwater elevations between 158 feet and 169 feet with a one foot drop between fish ladder pools. The 60 pool ladder will operate within stated criteria (0.4 feet – 1.5 feet drops between pools) at overall water differentials from 56 to 67 feet depending on the tailwater levels. The high fish ladder exit is limited to a forebay elevation of 232 feet allowing two feet of freeboard above the maximum operating level of 230 feet.

Figure 3-1 shows the forebay-tailwater operating ranges for the 52 pool and 60 pool ladder configurations. The dashed lines represent the forebay-tailwater combinations at which the ladder can be operated at the target one foot drop between pools. The lines to the left and right of the dashed line having the same symbols, represent the criteria range of 0.4 feet drop (left) and 1.5 feet drop (right) between any two pools. A 0.4 feet and 1.5 feet drop corresponds to an average slot velocity of 5 and 10 fps, respectively. If the combination forebay-tailwater falls to the right of the dashed line, then the drop between pools will exceed one foot in the lower parts of the fish ladder. If the combination falls to the left of the dashed line then the drop between pools will be less than the target one foot drop between pools. For example, if the John Day pool is being regulated through operation of the powerhouse to maintain a forebay elevation of 215 feet, then The Dalles pool will need to be regulated to maintain a John Day tailwater of 160 feet in order for the fish ladders to be operating with the preferred 1.0 feet drop between pools. If The Dalles pool and river flow remain unchanged and generation at John Day is reduced, then the forebay level will raise. The forebay can be raised to about elevation 218 feet, as seen on Figure 3-1, before the drop between pools exceed the 1.5 feet limit. Once the forebay raises above elevation 218 feet then either: 1) The Dalles pool must be raised to increase the tailwater at John Day, or 2) The configuration of the fish ladder must be changed to use the 60 pool configuration.

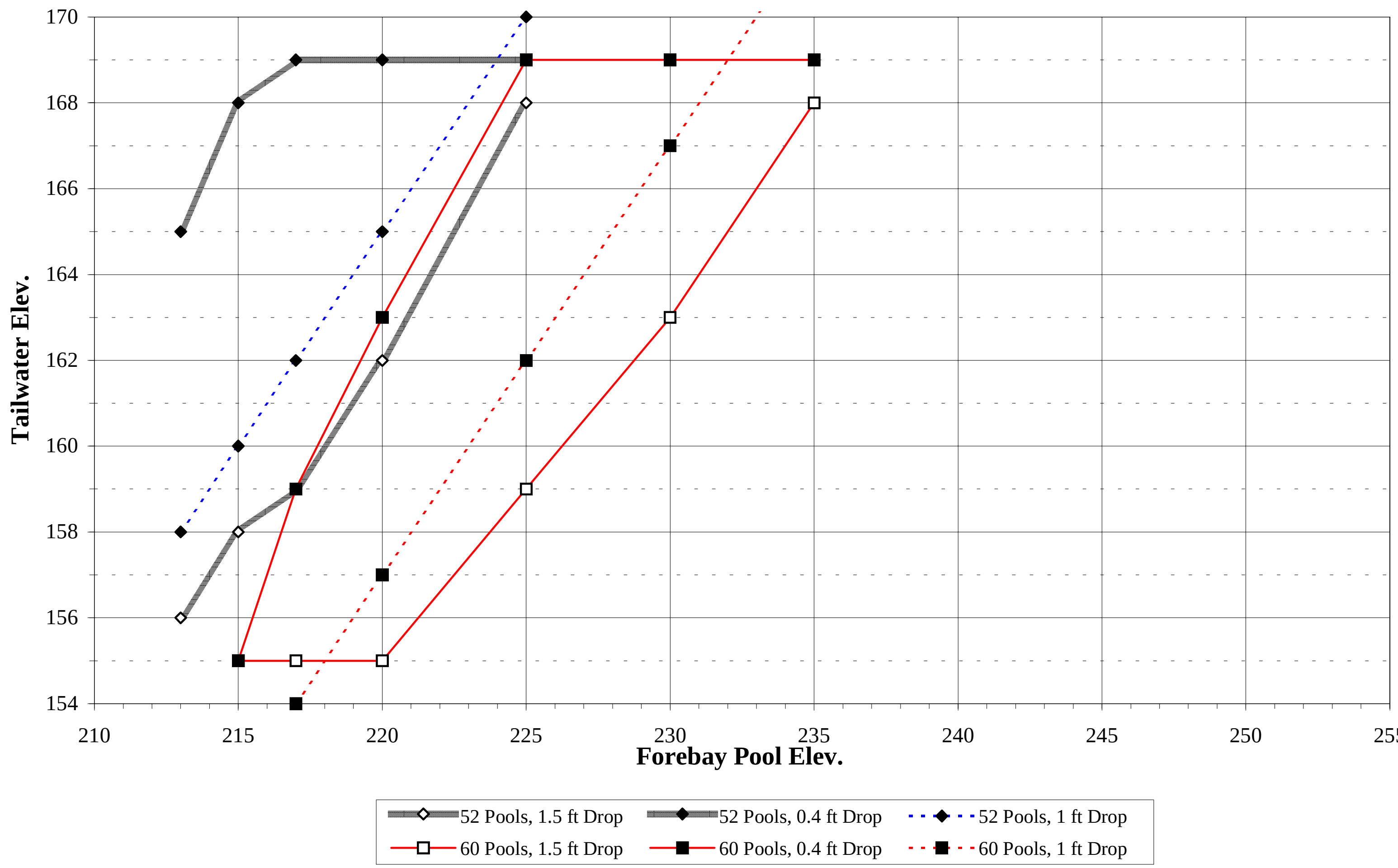
The 0.4 feet to 1.5 feet operating range represents reasonable limits preferred for fish passage. The average slot velocities within these limits (5 fps to 10 fps) are well within the swimming capabilities of the adult fish shown on Table 2-6. At drops less than 0.4 feet delay in the ladders can become an issue. Drops above 1.5 feet (10 fps) can create a hydraulic barrier to some weaker fish and create excessive turbulence within the pools resulting in erratic jumping. For weaker swimming species, not listed on Table 2-6, the upper limit of 1.5 feet may be too high, and poor passage may be observed. In this case, the John Day and The Dalles Projects should be operated so the forebay-tailwater differential remains closer to the dashed line on Figure 3-1 and a 1.0 feet drop between pools.

Flow in the fish ladder is dependent on the depth at the upstream end of the ladder. Over the full range, 213 feet to 230 feet the ladder flow varies between 160 cfs and 320 cfs. Transport velocity using the full channel width, in the fish ladder pools varies between 0.9 fps and 1.2 fps. Energy dissipation is 4 ft-lb/s/cf at a one foot drop between pools.

Exit channels are seven feet wide and provide a two fps transport velocity. The transport velocity stays the same for varying forebay levels.

Auxiliary water for attraction flow would be pumped via the water supply conduit to the entrance chamber and diffused to the channel through Diffuser Nos. 1 and 2. Current attraction flows can be maintained between 450 cfs and 1,100 cfs. This is within the capacity of the existing pumps. Attraction flow and entrance weir setting should be coordinated to maintain a 1.5 foot drop at the ladder entrance.

Figure 3-1
Alternative 1 - Fish Ladder Operating Ranges



3.2 South Shore Fish Ladder

3.2.1 South Shore Fish Ladder Description

The modification option chosen for the South Shore Fish Ladder (SSFL) consists of the full reconstruction of the fish ladder from the entrance channel to the new fish ladder exit. See Plate 5 showing a plan of the existing facilities. The ladder is designed for operation over the design operating water levels in the forebay and tailrace as described in Section 2 of this report, (213 feet to 230 feet).

In this alternative a vertical slot fish ladder would be constructed with a low level exit, Exit No. 1, and a high level exit, Exit No. 2, as described for the NSFL in Section 3.1.1. Exit No. 1 will be used with the 52 pool ladder configuration and Exit No. 2 will be used with the 60 pool configuration. A plan view and section view are shown on Plates 6 and 7.

The existing fish ladder would be demolished from the existing construction joint between weirs 155 and 156. The north wall of the existing fish ladder and water supply conduit would be preserved and used for the new ladder. Diffuser Nos. 2 and 3 are maintained as well as the existing fish collection channel and entrances. The floor elevation at weir 155 would be lowered to elevation 148 (2 feet) between the start of the new ladder and the Diffuser No. 3. Modification to the water supply conduit at the west end of the ladder would be required. The existing water supply conduit to Diffuser Nos. 4 through 16 would be demolished and filled with concrete.

Construction of the new ladder is as described for the NSFL. A new counting and viewing facility is provided at pool 38 at invert elevation 185.0 feet (existing ground level). Counting and viewing facilities are assumed to be similar to the existing facilities but would be completely reconstructed. Two exits are provided at the same elevations as on the NSFL.

The new ladder fish exits are below the existing grade upstream of the dam. Excavation will be required with riprap slope protection. A trash fender and bulkhead gate will be furnished for each exit.

The auxiliary water system would be modified to add attraction flow to the fish collection channel below the last fish ladder pool at Diffuser Nos. 1, 2 and 3. Diffuser Nos. 4 through 16 would not be required. The fishwater pumps would provide auxiliary water. With the lower operating pool the existing turbine pumps will not provide sufficient flow and would need to be supplemented with new motor driven pumps. These are described in Section 3.4.5.

Construction of the new fish ladder would be mostly below the current ground elevation of 185 feet. Walls in the lower part of the ladder will provide flood protection to this level. Starting at the counting and viewing structure, the ladder would be elevated. Construction would be similar to the existing ladder.

Two other fish ladder options were considered at the 30 percent level: partial reconstruction with a new regulating section (1S2) and partial reconstruction with fish locks (1S3). The option of partial reconstruction with a new regulating section could not accommodate the large operating range and maintain self regulation. Partial reconstruction with fish locks was not brought forward because there were too many fish to handle. Experience at other projects indicated that lock systems were not as successful as ladders in passing large quantities of fish without delay and stress.

3.2.2 South Shore Fish Ladder Operation

It is envisioned that the SSFL operation will be the same as described for the NSFL in Section 3.1.2. Operating conditions will be as indicated on Figure 3-1.

Auxiliary water for attraction flow would be supplied to the collection channel below the fish ladder. Current attraction flow would be maintained at 2,570 cfs to 3,220 cfs. Attraction flow and entrance weirs would be coordinated to maintain a 1.5 foot drop at the ladder entrances to maintain current operating conditions.

3.3 Downstream Passage

3.3.1 Feature Description

Downstream passage for juveniles is provided by using the existing spillways and a reconstructed juvenile bypass system (JBS) in the powerhouse.

Structural modifications required for the new lower forebay level and larger operating range include:

- Modifications to the bar or submerged traveling screens (screens). Extended length bar screens (ELBS) are assumed to be required
- Modifications to the vertical barrier screens (VBS)
- Bore new fish transportation conduits (FTC) the length of the dam
- Bore new orifices
- Bore a new service tunnel from which to maintain the orifices
- Construct new transport channels and dewatering screens
- Construct a new elevated bypass transportation flume
- Modifications to the existing evaluation and monitoring building

Structural modifications to the existing screens and VBS are included in this alternative. If the existing equipment will not function properly with necessary modifications, new screens and VBS's will be constructed. The existing JBS collection system has been modified several times since the initial construction to improve the efficiency of the collection system for operation at current forebay levels. Reducing the forebay levels as proposed in these drawdown alternatives is expected to require a similar development period to optimize the screens and flows for the new conditions which include not only hydraulic changes but also expected differences in fish distributions and tendencies. For juveniles the attraction conditions of the intake would be significantly different than under current conditions. About 33 percent less flow would be passing through the turbines after drawdown to spillway crest, and the ELBS would create about half as much head loss. This reduces the head driving flow up the gate well and through the VBS. In addition, the water level in the gate well and on the barrier screen would be much lower. At the minimum water surface elevation 60 percent less area on the VBS would be available to pass flow. To maintain the same approach velocities on the barrier screen only about 40 percent of the existing flow would be diverted into the gate well. This could reduce the effectiveness of the system to guide fish away from the turbines and into the JBS. Hydraulic and physical modeling would be required of the ELBS and VBS to determine the scope of the modifications required. Expected modifications include reducing the porosity control behind the VBS and screen and modifications to improve cleaning.

A new system is also required to transport fish downstream of the dam from the gate well. In this arrangement two new FTC's and a new service gallery will be constructed the full length of the active powerhouse through Unit 16. See Plate 8 for the location and details. Two orifices would be bored from the gate well to the new FTC's from each of the 48 intake bays. The centerline of the orifices that are connected to the lower FTC are at elevation 208 feet and at 217 feet for the upper. A submergence allowance of five feet is provided to reduce the potential for vortex formation. Both FTC's increase from a three feet square section at Unit 16, to 8 feet square section at the south end of the powerhouse. The top of the lower FTC is at elevation 208 feet and the top upper FTC is at elevation 217 feet. The floor slopes downward to the south from elevation 205 feet to elevation 200 feet. It is assumed that a single channel will be excavated and then a reinforced concrete floor constructed to form the upper FTC. A new service gallery is provided at elevation 227 feet. Orifice gate controls and lighting equipment would be located in this gallery.

Both conduits exit the south end of the dam and transition to an elevated transportation flume. The transportation flume would follow the same alignment as the existing elevated flume to the dewatering facility. The open concrete flume is five feet wide inside and 32 feet high and would be elevated between 10 and 13 feet above grade. Transportation velocities vary between 4 fps and 10 fps. A telescoping weir gate would be installed near the transition from the FTC to the concrete flume. The weir gate would be automated to regulate the head in the FTC and through the orifices. The dewatering facility can be demolished and rebuilt in its current location but at a lower elevation.

The juvenile bypass outfall pipe and evaluation features can be lowered to match the new dewatering structure. The outfall discharge location will remain unchanged. The evaluation structure would require modification to collect fish at the lower elevation.

Three additional options were considered at the 30 percent level: existing spillway with no JBS (1J2), surface bypass spillway with a JBS (1J3), and surface bypass spillway with no JBS (1J4). Option 1J2 was discarded since it would not meet the 80 percent fish passage efficiency requirement. Options 1J3 and 1J4 were not selected because they involved a surface by-pass spillway similar to that in DM No. 52. With a lower water surface upstream, juveniles coming down river along the left bank would be more likely to enter the turbine inlet before reaching the surface bypass spillway. In addition, a surface bypass on the front of the dam was not judged to be satisfactory since criteria and design of a successful surface bypass system has not been developed to a sufficiently reliable level yet.

3.3.2 Operation

Operation of the JBS requires monitoring of the forebay level and selection of the correct orifices and FTC to use. At forebay levels between 213 feet and 222 feet the lower FTC would be used and from 222 feet to 230 feet the upper FTC should be operated. Orifices are sized for 8 fps. The FTC size would increase to maintain transport velocities between 4 fps and 10 fps. It is assumed the hydraulic design of the new concrete flume, dewatering structure and transportation flume would match the existing features.

3.3.3 Biological Considerations

Predicting the passage characteristics of the JBS has proven to be a challenge under current operations and is expected to be the same for this alternative. A long evaluation program would be required to properly configure and test the new JBS system.

3.4 Hydroturbine Operation

There are four alternatives under study – two involve drawdown to spillway crest and the other two call for drawdown to natural river. For the spillway crest option, the river would be drawn down and will fluctuate between 213 and 230 fmsl. The spillways can be used to lower the reservoir water surfaces to near the spillway crest elevation, 210 fmsl. Below the spillway crest, there are no low-level outlets other than the turbine passages through the powerhouse.

3.4.1 Background

Previous to this investigation by the Portland District, the Walla Walla District has been working on a reconnaissance level report on the drawdown of the Lower Snake Dams. The Walla Walla District contracted with Voest-Alpine Machinery Construction Engineering (Voest-Alpine), in Linz, Austria, to evaluate intermediate-head and low-head turbine operation using an existing 1:25 scale turbine model for a Lower Granite Lock and Dam (Lower Granite) turbine, Unit 4. At the same time, the Corps' Waterways Experiment Station (WES) in Vicksburg, Mississippi, conducted tests using a bladeless runner in the Lower Granite 1:25 scale sectional turbine model.

The Lower Granite turbines, Units 4-6, are of identical diameter, 312 inches, and operate at the same approximate head as the John Day turbines. The Lower Granite low-head and intermediate-head (25 feet – 60 feet) performance was used to validate the model extrapolation for the John Day turbines.

This report will summarize the pertinent powerhouse systems affected by the drawdown of the John Day reservoir. Included will be the effect of the drawdown on the power output and discharge through the hydroturbines. Also included will be the effect on the fish turbines and the John Day powerhouse water systems.

John Day turbines are rated at 212,400 horsepower when operating at 90 rpm and a net head of 94 feet. The turbines were manufactured by Baldwin-Lima-Hamilton Corporation (BLH). The design operating range is 83.5 to 110 feet net head. Normal operation range has been between 88 and 108 feet net head. This section will attempt to predict turbine performance for the John Day project from 80 feet gross head down to 20 feet gross head, which is the Speed No Load (SNL) lower limit for these turbines. The SNL lower limit is the minimum head at which the turbines are capable of spinning at the synchronous speed of 90 rpm without generating electrical power. Operation below SNL is possible, but requires manual operation and is not recommended due to increased uncertainties and risks. Information for these predictions were taken from an extrapolation of the original 1962 model test for the John Day turbines. Information was also taken from the Voest-Alpine model testing performed for the Lower Granite drawdown reconnaissance report.

3.4.2 Assumptions

1. Since the Lower Granite turbines, Units 4-6 are the same diameter and approximate head as the John Day BLH turbines, the low head (25 feet – 60 feet) performance was used to validate the model extrapolation for John Day.

2. The current turbine synchronous speed of 90 rpm will not change.

3. The turbines will be capable of operating down to speed no load. Cavitation limits are impossible to predict at the lower heads without additional model testing, which is beyond the scope of this study. However, steep blade angle operation at low heads will increase cavitation.

4. The turbines will be capable of operating from the gate at which speed no load begins to a maximum gate opening of 100 percent.

5. Low head (below about 45 feet gross head) performance is assumed to be at flat blade (18.5 degrees).

3.4.3 Turbine Performance for Spillway Crest Option

Turbine performance information presented in this report has been put together from the Original Model test for John Day and from the Voest-Alpine model testing recently performed for Lower Granite drawdown study. The John Day Original Model Hill curve was extrapolated to the minimum operable turbine head to find the maximum head range of the turbines. This minimum point was established by first developing the SNL line for the turbines. The SNL point is the discharge and associated gate opening at each head where the prototype turbines are producing no electric power while spinning at synchronous speed, in this case 90 rpm. The SNL line was determined from the model test and from this, the minimum head was determined to be about 19 feet (the head at 100 percent gate). Efficiencies and flows at the heads between the minimum head and design heads were estimated by using the information available in the Lower Granite Model test by Voest-Alpine. It is estimated that for heads above 50 feet the error is about +- 10 percent. For heads below 50 feet down to the minimum head, where the extrapolation is greatest, the estimated error is +-20 percent.

Figures 3-2 and 3-3 are the SNL line for Head vs. Gate Opening percent and Discharge vs. Gate Opening percent for both flat blade angle, about 18.5 degrees, and for steep blade angle at 33 degrees. The lower line identifies the minimum heads and flows at which the turbine can produce power as a function of gate opening. Notice that at a gate opening of 100 percent the minimum head is about 19 feet. According to the model test this is the minimum head at which power can be produced, however this does not take into account increased cavitation at lower heads. More than likely this is a very liberal estimate and the actual minimum power head is higher.

Figure 3-4 is again the SNL line for Head vs. Discharge for flat and steep blade angles. The flat blade curve, which is the lower of the two, shows the minimum discharge required to produce power at each head.

Figures 3-5 and 3-6 are the expected Discharge Rating Curve and the Power Rating Curve at five foot increments from the current design heads down to 50 feet. It is difficult to show the discharge and power rating curves below 50 feet, however the estimated minimum and maximum power are included in Table 3-1.

Finally, Table 3-1 shows the expected minimum and maximum performance between 20 and 80 feet net head. At each head, minimum performance is at or above the SNL line, whereas maximum performance is considered to be 100 percent gate opening. This does not necessarily mean that the turbine will perform and produce power at these lower heads. These are more or less the absolute minimums and maximums. In actual performance the range may be substantially less.

3.4.4 Results of the Lower Granite Model Testing in Low Head Conditions

In addition to the measured data from the Lower Granite testing there was valuable qualitative information which was gathered by direct observation. These observations showed that a vortex was formed on the runner for some of the lower heads. The formation of a vortex is an indication of severe unstable and unsafe operation of the turbines. The vortex formation and collapse causes severe vibration from unstable flow distribution to the runner.

Cavitation was not able to be modeled due to the type of modeling method used, but it is expected that significant cavitation will occur which will cause damage to the machinery and structures.

At head ranges far outside the design operating range of the turbine, there is a significant decrease in efficiency. The poor, i.e. low efficiency, operation means that a substantial amount of energy must be absorbed by the equipment and structure. In the information presented in Table 3-1, one of the columns shows the expected energy dissipation by the turbine and powerhouse structure.

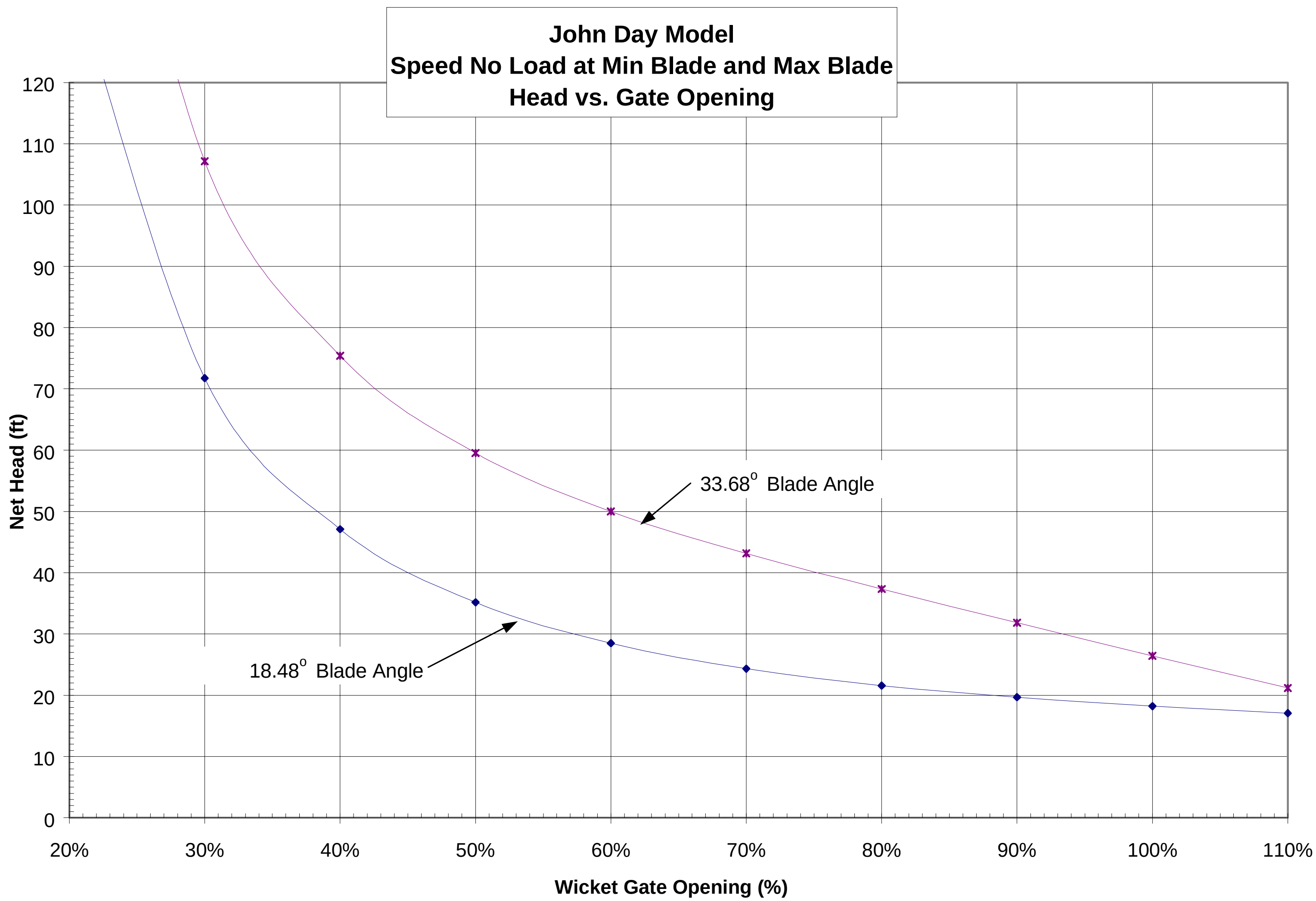


Figure 3-2. - Speed No Load, Head vs. Gate

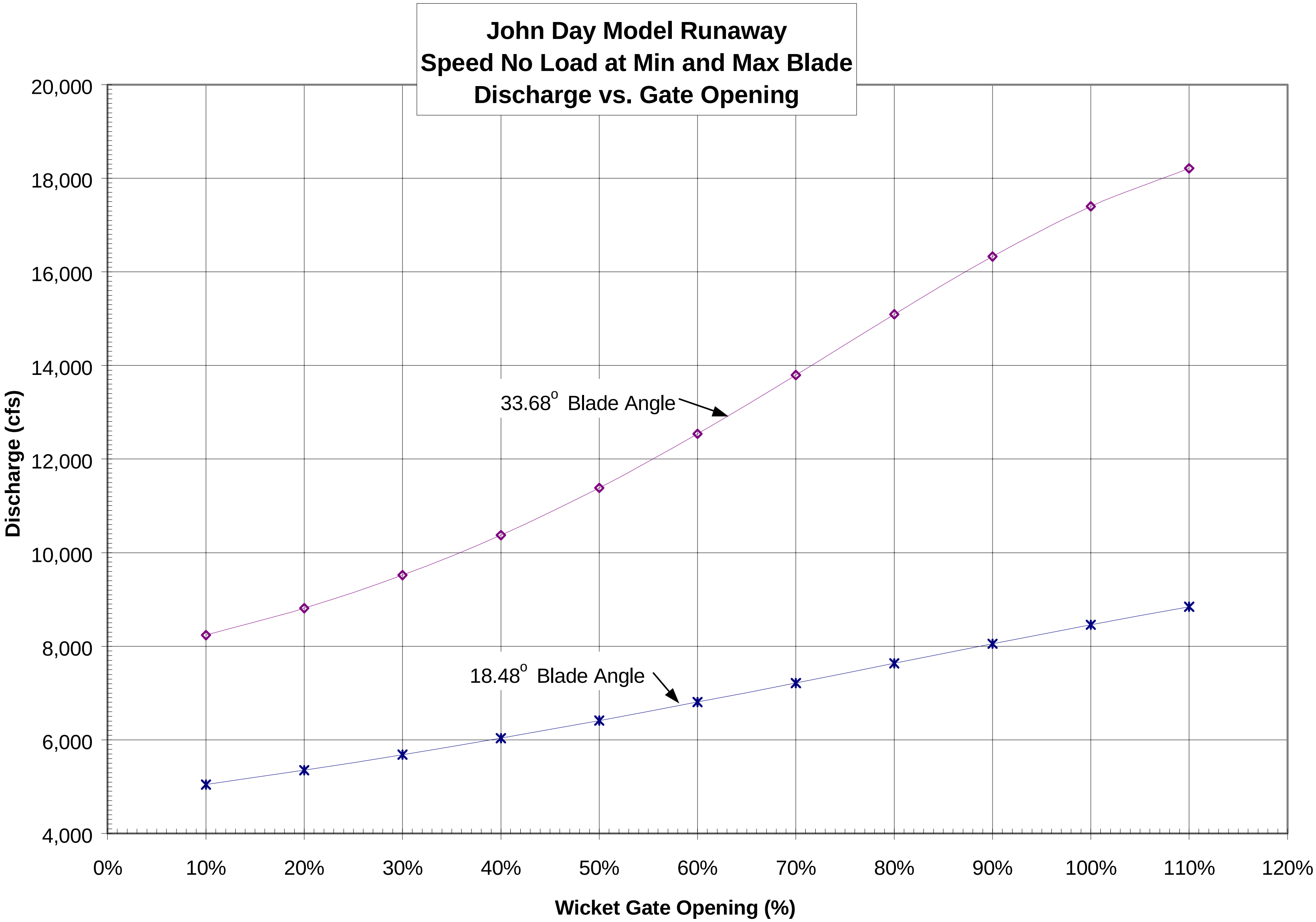


Figure 3-3. - Speed No Load, Discharge vs. Gate

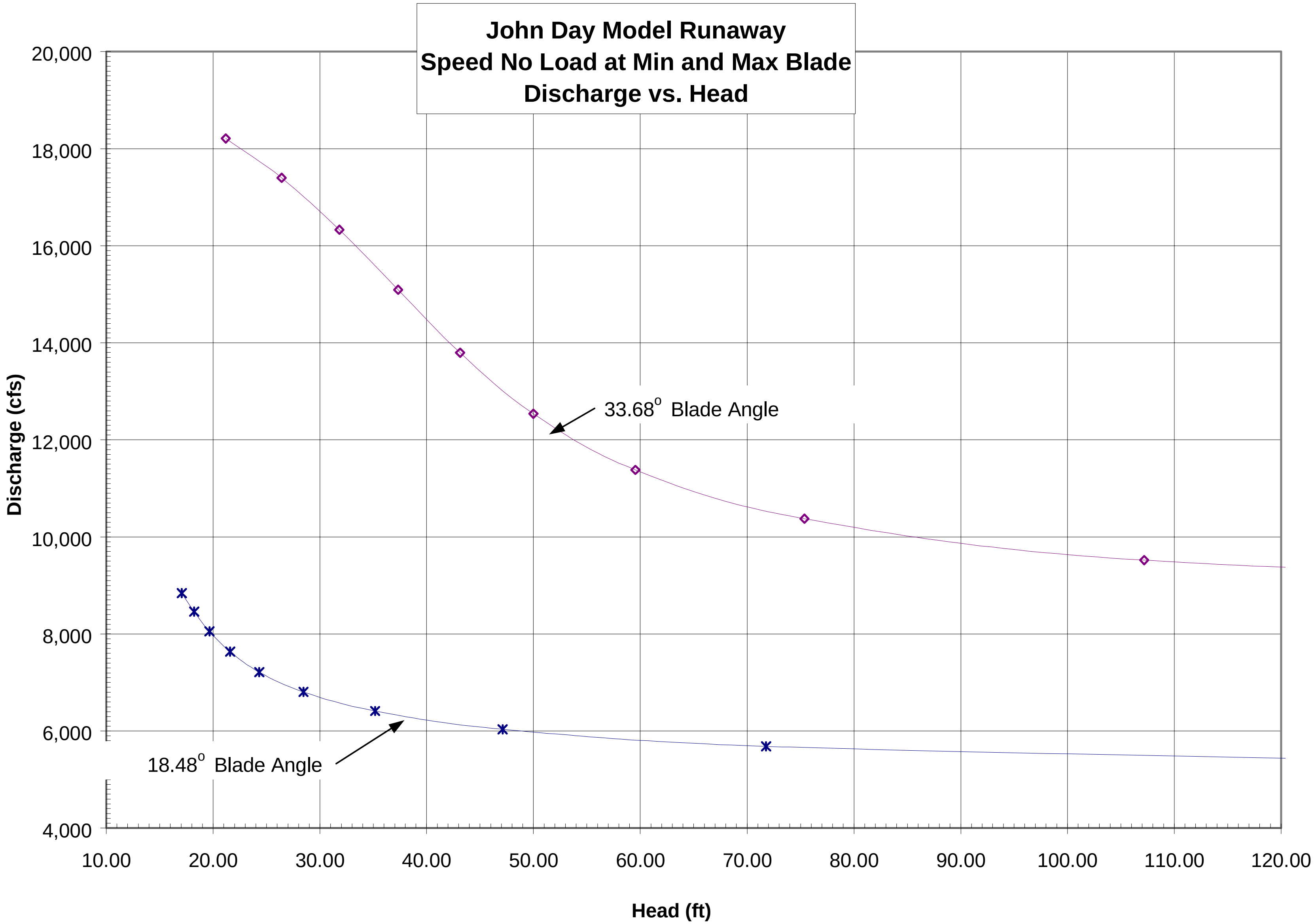


Figure 3-4 - Speed No Load, Discharge vs. Head

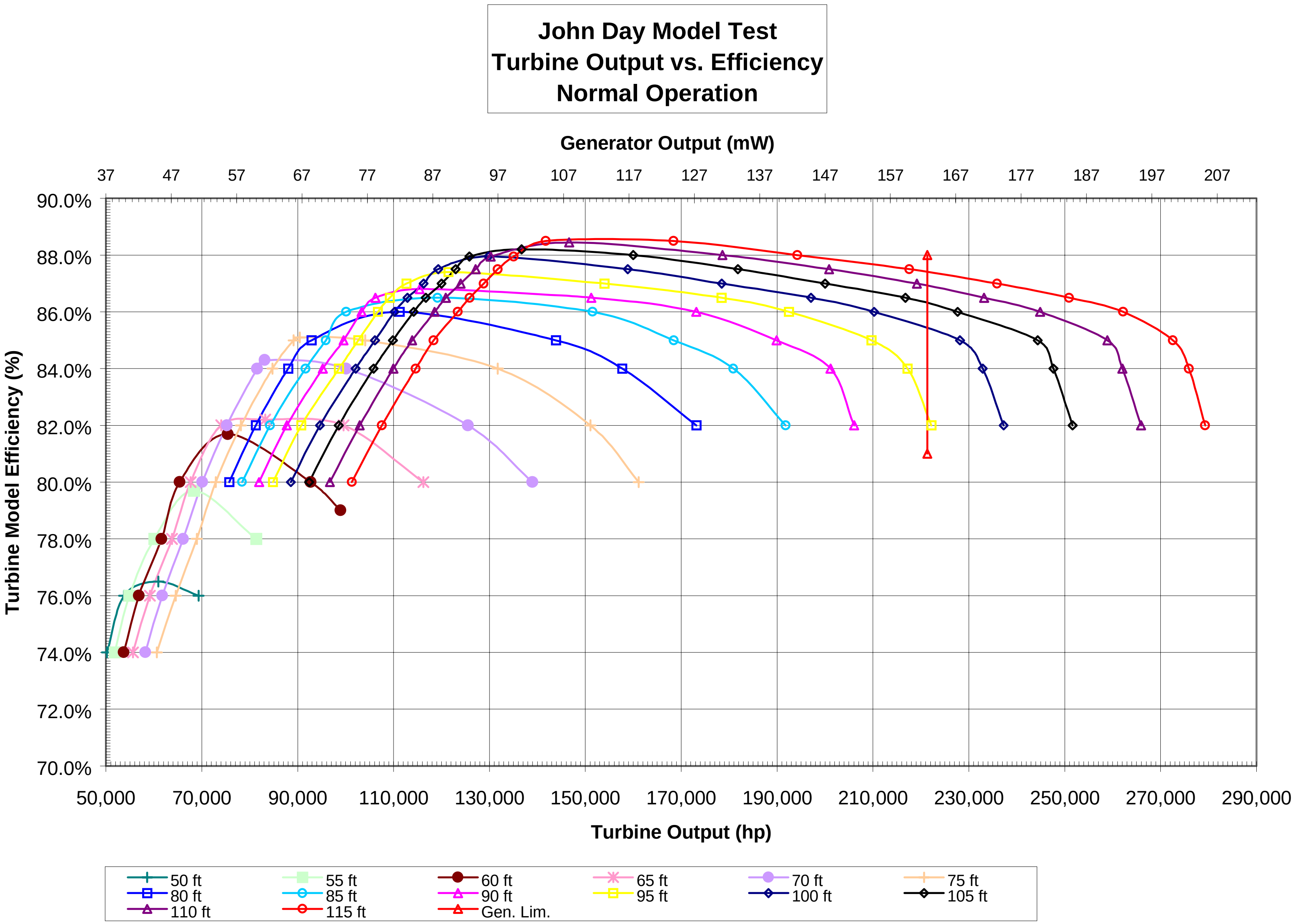


Figure 3-5. - Turbine Output vs. Efficiency over Operating Range

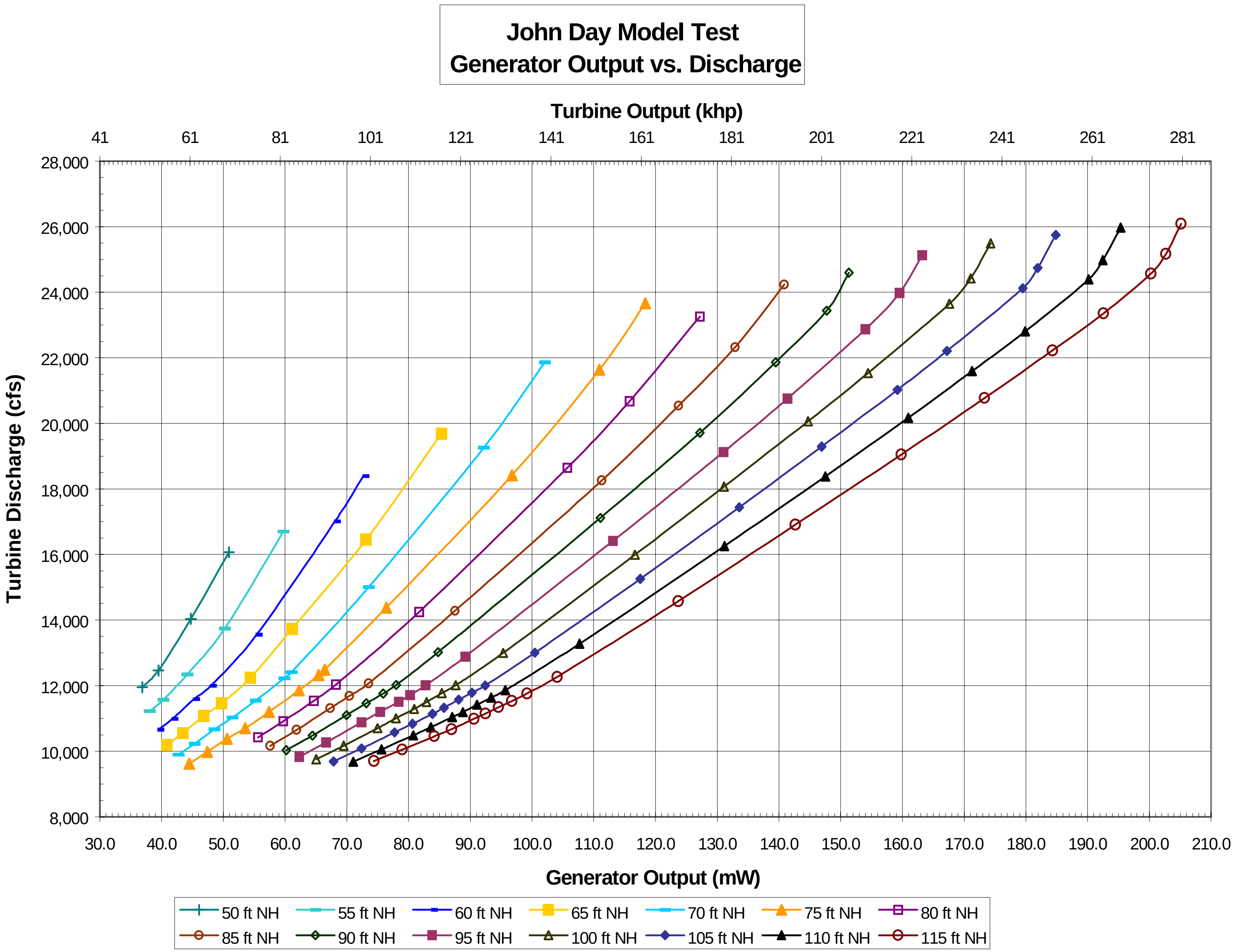


Figure 3-6. - Generator Output vs. Discharge Over Operating Range

Table 3-1
Turbine Performance over 20' – 80' Net Head Range

Net Head	Minimum Turbine Performance			Maximum Turbine Performance					Power to Machinery & Structures
	Gate	Proto Power	Proto Flow	Gate	Model Power	Proto Power	Max Effic.	Flow	@100% Gate
(ft)	(WGO%)	hp	(cfs)	(WGO%)	(hp)	(hp)	(%)	(cfs)	(hp)
20	88.0	1000	9,000	100	0.060	3,628	7.0%	22,829	48,198
22	80.0	1000	8,680	100	0.097	6,780	15.4%	17,631	37,247
24	72.0	1000	8,360	100	0.134	10,682	23.8%	16,476	34,201
26	65.8	1000	8,100	100	0.164	14,734	31.6%	15,799	31,892
28	61.5	1000	7,900	100	0.187	18,749	38.8%	15,205	29,574
30	57.2	1000	7,700	100	0.210	23,326	46.0%	14,892	27,383
32	54.4	1000	7,620	100	0.221	27,068	47.1%	15,815	30,377
34	51.6	1000	7,540	100	0.232	31,146	48.2%	16,730	33,419
36	49.2	1000	7,460	100	0.241	35,248	61.0%	14,141	22,536
38	47.1	1000	7,380	100	0.248	39,303	65.0%	14,019	21,163
40	45.0	1000	7,300	100	0.255	43,609	69.0%	13,920	19,592
41	44.2	1000	7,280	100	0.258	45,752	69.8%	14,085	19,795
42	43.5	1000	7,260	100	0.261	47,951	70.6%	14,247	19,968
43	42.7	1000	7,240	100	0.263	50,207	71.4%	14,407	20,111
44	42.0	1000	7,220	100	0.266	52,521	72.2%	14,565	20,223
45	41.2	1000	7,200	100	0.277	56,526	73.0%	15,160	20,907
46	40.7	1000	7,180	100	0.283	59,728	73.4%	15,585	21,645
47	40.1	1000	7,160	100	0.289	63,036	73.8%	16,011	22,379
48	39.6	1000	7,140	100	0.296	66,453	74.2%	16,438	23,106
49	39.0	1000	7,120	100	0.302	69,978	74.6%	16,866	23,826
50	38.5	1000	7,100	100	0.308	73,613	75.0%	17,294	24,538
51	38.0	1000	7,080	100	0.314	77,309	75.6%	17,665	24,952
52	37.6	1000	7,060	100	0.320	81,115	76.2%	18,035	25,335
53	37.1	1000	7,040	100	0.326	85,031	76.8%	18,404	25,686
54	36.7	1000	7,020	100	0.332	89,059	77.4%	18,772	26,004
55	36.2	1000	7,000	100	0.338	93,198	76.0%	19,643	29,431
56	35.8	1000	6,980	100	0.342	96,828	76.2%	19,991	30,243
57	35.4	1000	6,960	100	0.346	100,539	76.4%	20,340	31,056
58	35.0	1000	6,940	100	0.349	104,331	76.6%	20,689	31,871
59	34.6	1000	6,920	100	0.353	108,205	76.8%	21,038	32,687
60	34.2	1000	6,900	100	0.357	112,161	77.0%	21,388	33,503
61	33.8	1000	6,880	100	0.359	115,460	77.2%	21,600	34,100
62	33.4	1000	6,860	100	0.360	118,806	77.4%	21,811	34,690
63	33.1	1000	6,840	100	0.362	122,198	77.6%	22,021	35,274
64	32.7	1000	6,820	100	0.363	125,639	77.8%	22,230	35,851
65	32.3	1000	6,800	100	0.365	129,126	78.0%	22,438	36,420
66	31.9	1000	6,780	100	0.365	132,444	78.2%	22,608	36,922
67	31.6	1000	6,760	100	0.366	135,799	78.4%	22,776	37,414
68	31.2	1000	6,740	100	0.367	139,192	78.6%	22,943	37,897
69	30.9	1000	6,720	100	0.368	142,622	78.8%	23,109	38,370
70	30.5	1000	6,700	100	0.369	146,090	79.0%	23,274	38,834
71	30.2	1000	6,690	100	0.369	149,312	79.2%	23,393	39,213
72	29.9	1000	6,680	100	0.369	152,561	79.4%	23,511	39,581
73	29.7	1000	6,670	100	0.370	155,834	79.6%	23,627	39,937
74	29.4	1000	6,660	100	0.370	159,133	79.8%	23,741	40,282
75	29.1	1000	6,650	100	0.370	162,458	80.0%	23,854	40,614
76	28.9	1000	6,640	100	0.366	163,747	80.0%	23,727	40,937
77	28.6	1000	6,630	100	0.366	167,263	80.0%	23,922	41,816
78	28.4	1000	6,620	100	0.367	170,812	80.0%	24,116	42,703
79	28.1	1000	6,610	100	0.367	174,392	80.0%	24,310	43,598
80	27.9	1000	6,600	100	0.368	178,004	80.0%	24,503	44,501

Notes: Minimum performance is assumed to be at flat blade (18.5 degrees)
Turbine output is unlikely below a head of 40 feet.
It is not possible to develop cavitation limits between 40 and 70 feet head.
(This could not be determined from the model test.)

Observations indicated that the worst conditions of unstable operation and vortex formation occurred with a blade angle of 32 degrees, wicket gate openings from 100 percent to 75 percent, and heads of 46 feet and below. The worst condition noted during the observational testing was for 100 percent wicket gate opening, 32 degrees blade angle, a low tailwater, and gross head of 28.2 feet (SNL condition). As head on the turbine is reduced with a blade angle of 20 degrees, the model testing indicates acceptable to marginal operating conditions.

Also noted during the observational testing was the effect of lowering the pool elevation on the turbine intake velocity. As the pool elevation was lowered and intake flow area decreased the velocity increased. Higher intake velocities may cause higher loading on the trash racks from debris accumulation, which may affect the turbine discharge capacity.

3.4.5 Recommendations for Using Existing Turbines

1. The operating range limitations identified in Table 3.1 should be used to help define the turbine discharge capabilities for evaluating drawdown alternatives. However, unstable or unacceptable operation may occur at many of the conditions identified in the tables, which may preclude actual operation at the conditions. The magnitude of the response of the prototype turbine to the hydraulic conditions is difficult to quantify for zones of turbine operation far beyond accepted design practice. Personnel within the powerhouse should be limited to persons making structural and operational observations.

2. Because the prototype response to operation far below the design range is uncertain, operation to the SNL condition should be restricted to low blade angles and should be carefully monitored prior to incremental increases in discharge.

3. The turbines and plant should be appropriately instrumented to detect structurally dangerous conditions. This includes accelerometers, shaft run out, increased leakage, bearing temperatures, structural and mechanical vertical displacement measurements, and pressure measurements at the head cover, intake, and draft tube man doors. There should also be instrumentation to detect runner blade impact on the discharge ring. HDC recommends full instrumentation at one turbine unit and conducting several tests to make sure the instrumentation setup is sufficient and working properly. Reduced instrumentation on remaining units would be required.

4. Emergency closure devices should be in operating condition.

5. Trash rack design should be reviewed for adequacy. The trash racks should be inspected and repaired as necessary prior to the testing. As the pool elevation drops and the turbine intake velocity increases, it is anticipated there will be significant effort required to keep the trash racks clear of debris.

3.4.6 John Day Fishwater Turbines

Auxiliary Water Supply (AWS) for the South Fish Ladder is provided by three turbine driven fish pumps. Pump discharge capacity for each of the fish water pumps is 1100-1300 cfs. For the two spillway crest drawdown alternatives under consideration, current AWS pump discharge capability is to be maintained. The head on fish unit turbines would be reduced, and consequently fish pump discharge capacity will be reduced approximately 40 percent. For alldrawdown options, replacement of fish turbines with electric motors would be required. The three replacement motors would be rated at 600 rpm at about 1000 horsepower each. If this method dows not supply adequate flow, the pumps will need to be replaced.

3.4.7 John Day Fishwater Turbines

For both drawdown to spillway alternatives, supplemental cooling water will be required to augment existing water systems, e.g., supplemental water for thrust and guide bearing cooling, gland water and generator cooling would be required. Table 3-2 shows the water systems at John Day that would be affected by drawdown. The estimated costs for equipment supply and installation is included in Attachment B, based on the following system requirements:

- 16 generator cooling water & bearing water booster pumps
- 16 pump bases
- 16 pump motors
- Electric service for the motors
- 2 station service transformer cooling pumps
- 2 station service transformer cooling water pump motors
- 1 heat pump water pump & motor

3.4.8 Synchronous Condensing Operation

During certain times of the year (April-November), six units at John Day are currently dedicated to synchronous condensing operation to provide VAR reserve for improved transmission system stability. For the spillway crest alternatives, with the exception of supplemental cooling water systems mentioned above, there does not appear to be additional modifications required to maintain synchronous condensing operation.

3.5 Navigation Lock

During initial construction, the navigation lock was built in two phases. The First Step cofferdam was built from the north shore and enclosed the spillway and navigation lock locations. The lock walls were built and the floor was constructed to elevation 138.0. At the upstream end of the lock a sill was constructed to elevation 147.0. Behind the Second Step cofferdam on the south side of the project, the powerhouse superstructure was constructed, and the river ran through the navigation lock and the uncompleted spillway. The Third Step cofferdam was constructed on the north shore around the navigation lock and spillway. Behind this cofferdam the navigation lock was completed by raising the upstream sill to elevation 242.0. The upstream lock gate was also installed at this time.

Table 3-2
John Day Water Systems Affected by Drawdown

Powerhouse System	Water Intake Elevation (ft)	Discharge (gpm)	Discharge req'd for Powerhouse (gpm)	Comments
Transformers	N/A	N/A	N/A	Air Cooled
Heat Pump		625	625	Pumps @ Elev. 154
Thrust and Guide Bearing Coolers	140	150	2,400	
Generator Coolers	140	1,500	24,000	
Gland Water		15	240	Potable Water, need backup source
SS Coolers				N/A
Air Compressors		50	50	Potable Water, need backup source
SS Transformers	208	40	80	

3.5.1 Lock Modifications

The existing navigation lock is designed to operate between forebay water surface elevations of 257.0 and 268.0. Therefore, the minimum design water depth across the upstream sill is 15 feet. This is used as criteria for design of the lock modifications. Therefore, the sill would be cut down to elevation 195.0 for the modification assuming that the lowest possible pool would be at elevation 210. The design high flow for lock operation is 800,000 cfs. At this flow the upstream water surface with no spillway gate control is about elevation 246.0. However, to afford the same protection for the lock from overflow, the upstream lock gate would extend up to elevation 268.8. This requires a gate 74 feet high. The present lock gate is about 27 feet high. So, a considerably larger lock gate would be required. It is likely that the only type of gate that would be feasible would be a miter type rather than the present vertical lift gate. Stoplogs will be provided to dewater the gate.

The original design tailwater elevation range, 155.0 to 176.0 at 800,000 cfs, should remain the same. Any changes in flow over the spillway could change the current speeds and directions in the downstream approach channel. A model study would be required to develop a design that would provide safe and reliable transportation through the project.

Two options were considered for the navigation lock at the 30 percent level: modify the existing lock and navigation lock replacement. The option of modifying the existing lock was selected for reasons of cost.

3.5.2 Structural Considerations

Removal of 47 feet of sill will reduce the height of cross bracing at the upstream end of the lock where the earth loads on the north lock wall from the embankment section are highest. A structural analysis was performed on the lock walls.

John Day D.M. No. 16, Plates 22-27 provide stability diagrams which show that monoliths 11, 13, 15, 17, 19 20, 21, 22, 23, 24, 29 & 30 are stable under the water surface elevations for this alternative. See Plate 9 for key to monoliths. For monoliths 5 & 6, which are to the side of the Upstream Sill Block, design drawings JDN-1-4-2/2, 6, 8, 12 &13 and *D.M. No. 16,* Plate 17, provide similar details to the mid-lock monoliths, and it has been assumed that similar design criteria were used for overall stability. This provides the basis for the assumption that monoliths 5 & 6 are stable under this alternative for all sill heights between Elev. 242 and Elev. 147. This is further substantiated by initial operations of the lock during construction when it was operated with a sill elevation of 147.0 feet while the north embankment was at its full height.

Lowering of the embankment behind the monoliths could be done which would reduce the fill pressure. This would help the earthquake stability analysis at the Maximum Credible Earthquake (MCE) level. The MCE level was not part of the original design criteria present in D.M. No. 16, Plate 22. However, for Alternate 1 it is not recommended that this lowering be implemented due to the flood criteria which remains the same at elevation 276. This is consistent with the original design. The overturning moment due to the hydrostatic head still requires a mostly complete design section.

3.5.3 Filling and Emptying System

To provide adequate flow for the filling and emptying culverts, the existing intakes require modification. There are two entrances, one for each culvert. The existing north entrance is located in Monolith 2 and draws water from the upstream approach channel. The existing south entrance is located in Monolith 1 and draws water from the pool just upstream of the non-overflow section between the lock and the fish ladder. Each entrance consists of four openings eight feet wide and 30 feet high. The top of the entrances are located at elevation 200.0, which is just 13 feet below the minimum design water surface. Although the flow and velocities through the entrances should be less than at present, it is likely that vortexes will develop during filling operations. In model studies for the Ice Harbor navigation lock, vortexes formed occasionally at a flow of 10,700 cfs with the water surface 60 feet above the top of the entrances, and air-entraining vortices formed at 17,800 cfs. The maximum flow after drawdown could be about half of the present flow to allow the same lock transit time for river traffic. Therefore, the Alternative 1 maximum lock-filling rate is assumed to be 5,000 cfs. Vortexes will form for intake flows of 5,000 cfs if the existing intakes are used. Therefore, a new intake is required.

The new intake would be located east of the dam embankment and north of the navigation channel. It will be built behind a cofferdam. See Plate 11. The intake would have to be at about the same elevation as the existing intakes since it would have to be placed in an excavation. Therefore, fish screens will probably be required. The intake screen would adhere to all NMFS screening criteria for anadromous fish except that it would not contain a positive bypass. This was deemed not necessary for two reasons. First, the intake would draw water only during lock filling operations, which would last about ten minutes and would occur infrequently. Second, most of the juvenile fish encountering the screen can be expected to be larger than fry. Therefore, they will be stronger swimmers and able to keep themselves from being impinged on the screens. If a bypass were to be required, the intake screens would be located in the same place but have a different configuration. The bypass would pass through the embankment north of the navigation lock.

The intake structure would be constructed of concrete. It would be 50 feet high, 480 feet long, and vary in width from 40 to 90 feet. See Plate 10. The 14 intake openings would be covered with trashrack panels and fish screens located behind them. A platform would be located on top to facilitate maintenance during low flow periods. It is envisioned that the trashracks would be cleaned with a portable backhoe-type vehicle located on top of the platform. The fish screens would be cleaned with a brush or backspray system.

A rectangular concrete conduit would extend from the intake structure to Monolith 2 on the west wall of the navigation lock exit. See Plate 10. A 146 foot transition section would be located at the west end of the intake structure and would decrease the width of the conduit from 90 feet to about 41 feet. The remainder of the conduit would be about 570 feet long and connect to Monolith 2.

Water flowing to the navigation lock would first pass through the trashracks and then through the fish screens. It would then travel parallel to the screens to the west end of the structure at about 2 fps. The flow would pass through the transition and then through the conduit at about 5 fps. The existing lock gate would control the flow into the lock. Its movement might have to be restricted to limit the flow to 5,000 cfs and a 0.4 fps screen approach velocity as directed in NMFS criteria.

3.5.4 Guidewalls

The existing guidewall can be used after drawdown. However, a new slot for the mooring connection would be fabricated from steel and attached to upstream monoliths by divers.

Currents from a redesigned spillway might affect the downstream guidewall. However, it was assumed that no change in the downstream guidewall would be required since the energy of the spillway discharge will produce weaker currents after drawdown. However, extensive physical modeling would be required to assess the entrance and exit hydraulics.

3.6 Spillway and Stilling Basin

The spillway and stilling basin were designed for a flood of 2,250,000 cfs, which requires an upstream water elevation of 276.5. The spillway crest shape was designed at 75 percent of design head on the spillway. In this alternative the spillway would remain the same. However, at other than design flood conditions, the spillway flow would be free surface rather than controlled by tainter gates.

For spillway crest drawdown flows entering the stilling basin would have up to 50 percent less head and, therefore, less energy. This could affect the stilling basin, which was not designed to receive these lower energy spills. Due to the lower head the Froude number would decrease and the jump would occur closer to the spillway. With less energy the spillway would not be as efficient in flushing rocks and debris out of the stilling basin. Rocks drawn into the stilling basin could be retained in it. These could damage the stilling basin increasing maintenance costs. Model studies should be conducted to assess the potential damage to the stilling basin and to develop a design to fix the problem, if possible.

Since the head on the spillway and stilling basin will be lower, the mechanism for entrainment of air into the spill will change. It has been found that the amount of air entrained does not decrease in proportion to the decrease in head on the spillway. The actual effect of the drawdown on spill cannot be determined without further study. For this study it is assumed that existing spillway deflectors will need to be removed and replaced.

It is assumed that the spillway gates and the gate machinery will be removed and disposed of.

3.7 Project Sequencing

This subsection first describes the constraints on construction activities. This is followed by a description of the project schedule from completion of the Phase I Report through completion of construction and full implementation of the drawdown.

3.7.1 Project Constraints

Land access to the project for construction is limited. Much of the access to the project will have to be from the water. It is probable that much of the construction will be staged from barges and that transportation of materials to and from the site will be by barge.

In addition to site and access constraints, the construction schedule is affected by three operational items, which involve fish passage requirements. These restrictions are in-water work periods, fish transport spill, and the requirement that fish passage be provided at all times except during the three-month in-water work window.

Any construction work in the water is allowed only between December 1 and March 1. Construction behind cofferdams is not in-water work and, therefore, is not subject to this requirement.

In the spring and early summer if water is available, more flow is released into the river from upstream storage and is sent over the spillways. This restricts work on and near the spillways. The spill period typically extends from April 1 to July 31.

Another constraint on construction sequencing is to provide for navigation through the project at all times to the extent practical.

3.7.2 Project Schedule

In developing the project schedule it was assumed that the steps to implement the project would be the Phase II feasibility study and EIS, design memorandum, plans and specifications, and construction. For the purposes of this reconnaissance study all work prior to the start of construction is assumed to be the same for all alternatives. The Feasibility Study and EIS are assumed to start in October 2000 and last for five years. Preparing the design memorandum and plans and specifications is assumed to start in October of 2005 and take approximately three and one half years to complete. The construction bid process will start in April 2009.

The construction schedule is shown on Figure 3-7. The paragraphs below describe the construction sequencing by project feature.

3.7.3 North Shore Fish Ladder

The only location for upstream adult fish passage is at the existing north and south fish ladders. Since most of the ladder structure has to be demolished, the only way to provide constant fish passage is to continue one ladder in operation while reconstructing the other. Providing for a

Act ID	Description	Orig Dur	Early Start	Finish
Water Work Windows				
0010	In Water Work Window (09-10)	90	01DEC09 *	01MAR10
0020	In Water Work Window (10-11)	90	01DEC10 *	01MAR11
0030	In Water Work Window (11-12)	90	01DEC11 *	29FEB12
0040	In Water Work Window (12-13)	90	01DEC12 *	01MAR13
0050	In Water Work Window (13-14)	90	01DEC13 *	01MAR14
Spill Periods				
0080	Spill Period (09)	121	01APR09 *	31JUL09
0090	Spill Period (10)	121	01APR10 *	31JUL10
0100	Spill Period (11)	121	01APR11 *	31JUL11
0110	Spill Period (12)	121	01APR12 *	31JUL12
0120	Spill Period (13)	121	01APR13 *	31JUL13
0130	Spill Period (14)	121	01APR14 *	31JUL14
Bid and Award				
4100	Advertise for Bid	0	01APR09 *	
4200	Bid And Award	180	01APR09	28SEP09
4300	Award Construction Contract	0	28SEP09	
North Shore Ladder				
5100	Dewater NS Ladder	5	01DEC10 *	06DEC10
5200	Remove Ladder Structure	150	06DEC10	05MAY11
5300	Build Ladder	400	05MAY11	09JUN12
5400	Mine Ladder Exits	180	05MAY11	01NOV11
5500	Install Exit Gates	60	01NOV11	31DEC11
5600	Place NS Ladder in Operation	15	20JAN13	04FEB13
Navigation Lock				
6100	Mobilize	90	28SEP09	27DEC09
6200	Construct Intake Cofferdam	90	01DEC09 *	01MAR10
6300	Construct Remainder of Intake Cofferdan	90	01DEC10 *	01MAR11
6400	Construct the Lock Fill Intake	639	01MAR11	30NOV12
6500	Remove Intake Cofferdam	90	01DEC12 *	01MAR13
6600	Fabricate New Upstream Lock Gate	450	01SEP11 *	24NOV12
6700	Construct Lock Cofferdam	40	19JAN13 *	28FEB13
6800	Remove Upstream Guidewall	15	19JAN13 *	03FEB13
6900	Remove Upstream Sill to El. 195	45	28FEB13	14APR13
6920	Bore New Cross Over Tunnel	90	01JAN13 *	01APR13
6940	Install New Upstream Lock Gate	45	14APR13	29MAY13
6960	Reinstall Upstream Guidewall	30	14APR13	14MAY13
6980	Remove Lock Cofferdam	30	29MAY13 *	28JUN13
Spillway				
7100	Remove and Replace Flow Deflectors	90	01DEC10 *	01MAR11
7120	Remove and Replace Flow Deflectors	90	01DEC11 *	29FEB12
7130	Remove and Replace Flow Deflectors	90	01DEC12 *	01MAR13
7200	Remove and Dispose - Spillway Gates	500	03FEB13 *	18JUN14
Juvenille Bypass System in Dam				
8100	Award Dam JBS Contract	0	07SEP09 *	
8200	Mobilize	60	07SEP09	06NOV09
8250	Fabricate New Turbine Screens	1050	06NOV09 *	22SEP12
8300	Bore Vertical Shafts	100	06NOV09	14FEB10
8400	Mine Juvenile Collection Conduit and Gallery	350	14FEB10	30JAN11
8500	Bore Orifices	300	30JAN11	26NOV11
8600	Install Orifice Gates and Utilities	150	26NOV11	25APR12
8700	Install Turbine Screens	150	01OCT12 *	28FEB13

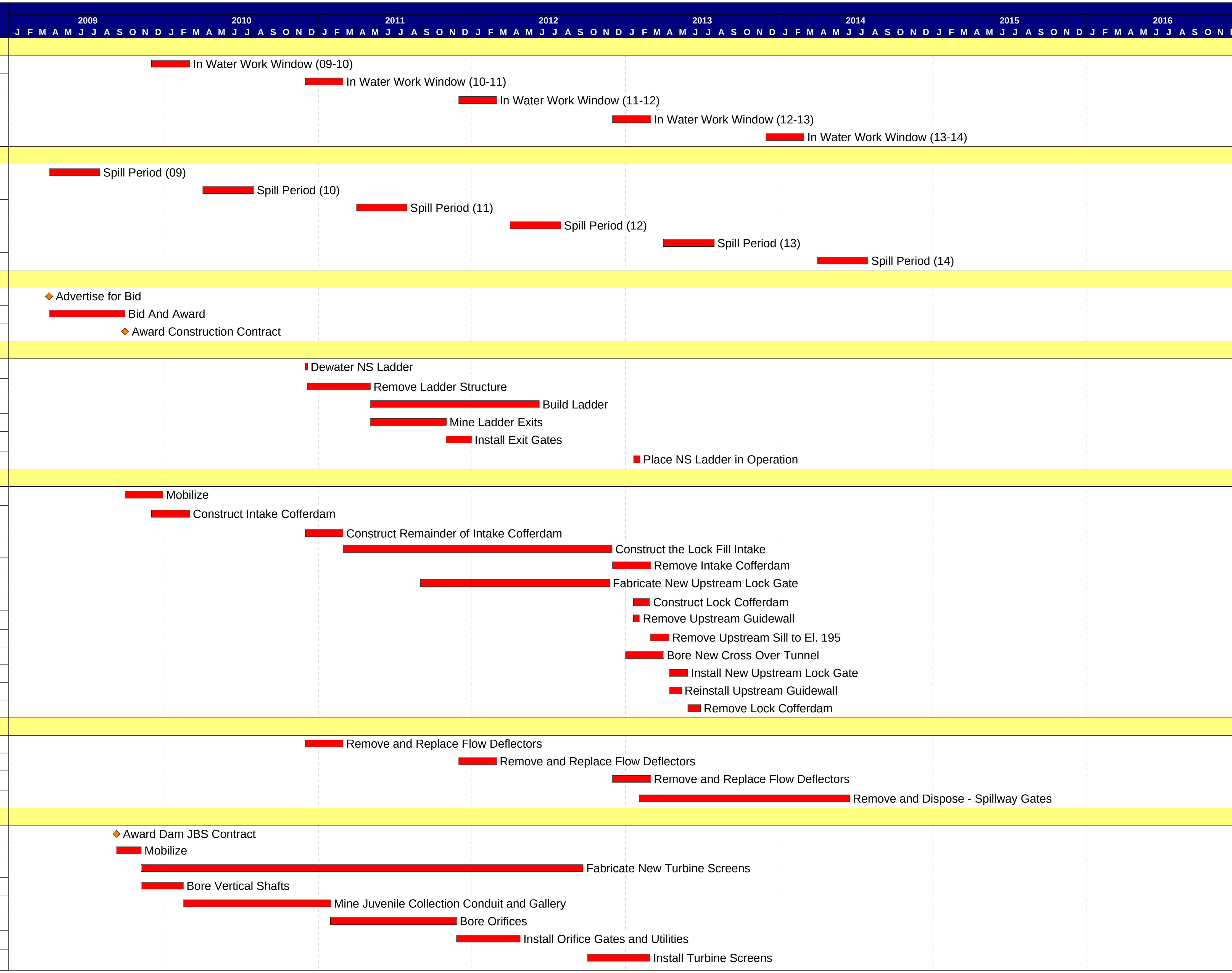


Figure 3-7
Schedule
Alternative 1 - Drawdown to Spillway Crest without Flood Control

Act ID	Description	Orig Dur	Early Start	Finish
Reservoir Drawdown				
9100	Raise Gates and Drawdown Reservoir	50	01DEC12 *	20JAN13
Juvenile Bypass System Outside Dam				
10100	Award South Shore Contract	0	07SEP09 *	
10200	Mobilize	50	07SEP09	27OCT09
10300	Divert Flow to Old Bypass System	10	27OCT09	06NOV09
10400	Remove Dewatering Structure and Flumes	200	06NOV09	25MAY10
10500	Construct Downstream JBS Facilities	400	25MAY10	29JUN11
10600	Complete Juvenille Collection Channels	240	04APR14	30NOV14 *
10700	Connect Upstream JBS Facilities	90	30NOV14	28FEB15
South Shore Ladder				
11100	Install Auxiliary Water Pumps	400	29JUN11	03AUG12
11200	Fabricate Dewatering Bulkhead	200	29JUN11	15JAN12
11300	Dewater South Shore Ladder	10	01DEC12 *	11DEC12
11400	Remove Ladder Structure	180	11DEC12	09JUN13
11500	Place Bulkhead and Mine Ladder Exits	120	09JUN13	07OCT13
11600	Build Ladder Structure	350	07OCT13	22SEP14

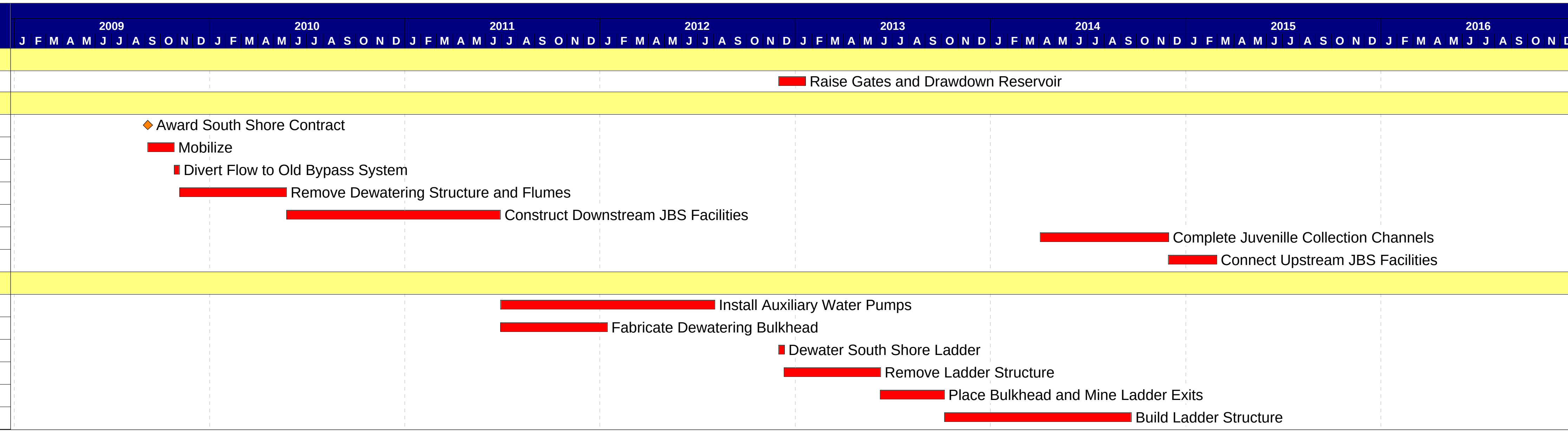


Figure 3-7
Schedule
Alternative 1 - Drawdown to Spillway Crest without Flood Control

trap and haul operation at the ladder while it is under construction is not considered feasible. The limited space available will not support both a trap and haul operation and construction of ladder modifications. In addition, trapping and hauling endangered fish passing the project would not be acceptable.

The NSFL would be rebuilt first while the SSFL remains in operation. For upstream fish passage during the construction period, flow through the turbines and attraction water in the south ladder should be maximized. During the drawdown of the reservoir upstream passage will not be provided under this scenario. One possibility that should be considered in the FDM phase would be to construct a false weir in the SSFL with a temporary flume at the exit to the forebay. The return flume would have to be adjustable to follow the reservoir level down through the full drawdown.

It is anticipated that the NSFL and navigation lock work could occur at the same time during the first part of construction. Work would begin with dewatering the ladder in early December. The ladder would then be demolished, and reconstruction of the new ladder would begin. After the ladder is demolished and the upstream cofferdam around the navigation lock and fish ladder exit is complete, the new fish ladder exits would be mined. Then the gates would be installed over the exits. This work can be coordinated with the navigation lock construction.

3.7.4 Navigation Lock

Work on the navigation lock would take place during both Stage 1 and 2. The first task is to start fabrication of the new upstream lock gate.

To build the new navigation lock water intake the area upstream of the navigation lock must be dewatered. See also Plate 11. The new lock fill intake would be constructed inside the cofferdam during construction Stages 1 and 2. Access for construction would be from the shore north of the navigation lock. Some construction access might be accomplished from the top of the cofferdam supplied by barge.

The modifications to the lock will be made during Stage 2 construction after the reservoir has been drawn down to spillway crest. The first piece of work is to remove the upper 47 feet of upstream sill. The upstream lock gate should be removed. Then the upstream concrete sill would be removed by diamond saw cutting the sill into large blocks. The blocks would be mechanically split into manageable sizes and removed by truck from the north side of the navigation lock. The new lock gate will then be installed, and the cofferdam will be removed.

The upstream floating guidewall will be removed before drawdown. After drawdown new fabricated slots will be installed by divers and the guidewalls will be re-installed.

Navigation would be curtailed for about six months during upstream sill removal and installation of the new gate.

It is assumed that the small one-cell cofferdam can be removed outside the in-water work window to lessen the time the navigation lock is out of service.

The alternative to the six-month out of service time is to build a new navigation lock. It was assumed that moving commodities by rail for six months would cost less than building a new navigation lock. Therefore, the six-month outage was selected as being more economical.

3.7.5 Juvenile Bypass System in Dam

It is envisioned that a new JBS will have to be constructed for the spillway crest drawdown. It would resemble the present one but would be about 45 feet lower. The plan calls for two new FTCs. They would be constructed by boring one tunnel and constructing a concrete floor horizontally in the center of the tunnel. Two conduits are required due to the greater range in pool water surface elevations than are presently experienced. Each conduit would have its own orifices for transferring fish from the gatewell into the collection conduits. These facilities can be built at any time since no in-water work is required. This work would be done while the north shore work is underway and prior to the south shore work to avoid conflicts during construction of the south shore facilities.

During construction, the first step is anticipated to be boring vertical shafts at the north and south ends of the powerhouse deck east of the existing conduit. These shafts would be used to start mining the FTCs and the service gallery for the JBS.

The orifices would be constructed at the individual units by first stopping the unit. Then, the area over the orifice in the gate well would be dewatered by sealing a bulkhead over the orifice location and drilling the orifice from the transportation conduit to the dewatered area in the gatewell. Gates would then be installed over the orifices and the bulkhead removed. This would be repeated at the other powerhouse units. Construction in this manner would allow operation of all units except the one where the orifices are being installed.

3.7.6 Spillway

Due to the reduced head on the spillway after drawdown, the deflectors would not be at their optimum location. For a conservative cost estimate it is assumed that the existing deflectors would have to be removed and new ones added. This work would require dewatering using bulkheads similar to the construction of the existing deflectors. The deflector construction would take place prior to reservoir drawdown.

Since the spillway tainter gates are no longer needed, they can be removed any time after the drawdown is complete. Removal would entail cutting the gate into pieces and lifting them with cranes onto trucks or barges for removal. The trucks would take the gate pieces off the dam to the north shore since work on the SSFL would be in progress.

3.7.7 Reservoir Drawdown

Under this schedule, after the NSFL and navigation lock intake work is completed, the reservoir will be drawn down. It is assumed that the drawdown will take place over 50 days. This is about one foot of drawdown per day to reduce slope stability problems on the reservoir banks. As soon as the upstream water level reaches a level within the NSFL's new operating range , it will be placed in operation.

3.7.8 Juvenile Bypass System Outside the Dam and South Shore Fish Ladder

The existing Juvenile Bypass System (JBS) outside the dam is located adjacent to the SSFL. So, it is anticipated that it would be best to construct the JBS and ladder at the same time and under the same construction contract.

The schedule on Figure 3-7 shows the construction tasks for the JBS and SSFL under different headings. However, these tasks are described together below in chronological order for the sake of clarity.

There is a great deal of construction work to be performed in a small area on the south shore. This increases the construction time. To minimize the length of construction and the time, in which only one ladder is in service, work will begin on the south shore before work on the north shore is complete. First, the crest gate would be opened diverting the JBS flow down the chute and into the tailrace. This dewaters the JBS west of the crest gate. Next the existing dewatering structure, fish transportation flumes, and dewatering facilities west of the crest gate would be demolished. See Plates 5, 6, 7, and 8. Since the new JBS would be the same as the present one but at a lower elevation, some of the mechanical equipment can probably be salvaged for use in the new JBS. Next the elevated fish transportation flume, dewatering structure, and evaluation facilities west of the existing fish ladder would be built. Then the new auxiliary water supply pump motors should be installed inside the powerhouse.

After the NSFL is in operation, the reservoir is drawn down and work can begin on the SSFL. First, the existing ladder would be demolished and removed. A dewatering bulkhead would be fabricated and placed on the upstream side of the dam to dewater for construction of the new fish ladder exits. The two fish ladder exits would be mined in the same manner as those on the north shore. The ladder would then be constructed and connected to the exits. Concurrently, the final reach of the FTCs at the south end of the dam would be constructed from the downstream side of the dam. Finally, the elevated transportation flume adjacent to the fish ladder would be built and connected to the end of FTC at its south end. The connection of the fish transportation flume outside the dam to the FTC will be made during the in-water work period when the JBS will be shutdown.

During construction of the SSFL, powerhouse flow should be minimized to reduce the attraction to the south shore. In addition, spillway gate removal should be coordinated to direct spill to help guide fish to the north shore. Attraction flow at the NSFL should be maximized at all flows until the SSFL is completed.

3.8 Operation and Maintenance Considerations

This subsection describes the operation and maintenance requirements only for those features impacted by the drawdown. These requirements are described in a general manner consistent with a reconnaissance level study.

The operation and maintenance requirements are described below for each feature that will be impacted by the drawdown.

3.8.1 North Shore Fish Ladder

The NSFL is designed to operate over a range of river flows from a low of 80,000 cfs to the 10-year high flow of 515,000 cfs. At forebay levels above elevation 233 feet, the exit gate at the ladder must be closed. Depending on powerhouse output, this water level will correspond to flows of 515,000 cfs or greater. Above this level water could overflow the sides of the ladder. The auxiliary water system would operate the same as it does now. However, only the lower ladder would require auxiliary water to supply the entrances.

The transition from the 52 pool configuration (Exit No. 1) to the 60 pool configuration (Exit No. 2) will occur as follows:

1. Open Exit No. 2 bulkhead
2. Open the swing gate between the ladders to direct fish to the upper ladder
3. Close Exit No. 1 bulkhead gate

Changing back from the 60 pool configuration to the 52 pool configuration will occur as follows:

1. Open Exit No. 1 bulkhead
2. Close Exit No. 2 bulkhead gate
3. Check for stranded fish and crowd into the lower ladder
4. Close the swing gate between the ladders directing fish to Exit No. 1

Depending on forebay levels simultaneous opening and closing of the exit gates may be required to prevent flooding the ladder.

Optimal operation of the fish ladders described for this alternative requires a philosophical change to the current operation of both The Dalles Dam and John Day Dam. Power production and/or spill at both projects will have to be coordinated to maintain the fish ladders at or near the operating conditions for a 1 feet drop between pools (dashed lines on Figure 3-1). Items for consideration during the next phases of this project include:

- Extension of the fish ladder entrance channel to add automatic weirs that will adjust to eliminate the need for controlling the pool at The Dalles.

- Use of auxiliary water system to add and withdrawal water from the ladder pools to extend the forebay range of each ladder.

- Use of adjustable slot gates to extend the forebay range of each ladder.

3.8.2 South Shore Fish Ladder

The SSFL would operate the same as the NSFL. However, the auxiliary water supply is different. Only half of the auxiliary water could come from the turbine pumps which now supply all auxiliary water needs. Electric motors would be provided to drive the existing turbine-driven pumps. Auxiliary water would be supplied to all diffusers except those further up the ladder.

3.8.3 Downstream Passage

Operation and maintenance for the JBS facilities would be the same as for the existing facilities. The dewatering and monitoring structures downstream of the dam would be the same as the existing structure except they would be lower in elevation. Therefore, the operation and maintenance requirements for these facilities will also be the same.

3.8.4 Hydroturbine Operation

The turbine will operate much as it does now, however the spillway crest alternative will cause the head operating range to be less than the minimum design head of the turbine at 83.5 feet. It is uncertain how the turbine will function in this range because it has never been operated there before. It is possible that unstable or unacceptable operation may occur at many of the low head conditions (see para. 3.4.4, Lower Granite model testing results at low head conditions, and para. 3.4.5 Operational recommendations).

3.8.5 Navigation Lock

The navigation lock would be the same as the existing structure except that the upstream gate would be a 74-foot high miter gate rather than the existing vertical lift gate. Filling and emptying the lock would have lower velocities in the system since the head on the structure would be less. Overall, the operation and maintenance effort for the lock would be about the same as at present.

Additional operation and maintenance will be required for the two lock fill intakes. Operational requirements include cleaning both the trashrack and the fish screens. A portable cleaning system would be employed for cleaning the trashrack. An underwater brush sweeping system or a back spray system would automatically clean the fish screens periodically triggered by a timer or differential head across the screens. Either system would require periodic maintenance and inspections by divers.

3.8.6 Spillway

Operation of the spillway would not be required if the gates are removed. However, regulation of the upstream water surface could be accomplished by operation of the turbines at the powerhouse. For example, if the forebay pool is too high more turbines can be brought on line. This would pass more flow through the powerhouse and less over the uncontrolled spillway, lowering the upstream water level.

With spillway crest drawdown fish will pass over the spillway rather than passing beneath the gate under 50 feet of head. So, possible damage to fish at the gate will be reduced. In addition, the energy dissipated in the stilling basin will be greatly reduced. This will subject the downstream migrants to less turbulence and shearing forces in the stilling basin. Once the spillway gates are removed, the ability to shape the spill to aid in attraction to the fish ladders will be lost. Fish moving upstream are expected to favor either side of the spillway. However, it may be necessary to increase attraction flows over present amounts during low flows to help induce fish moving into or through the spillway area toward the ladders. Further consideration should be given in future phases of this project to possibly lowering some crests to shape the spill under run-of-river conditions similar to the shaping done now with the spill gates. Additional study will be required to optimize ladder performance and develop powerhouse protocols that help upstream passage.

Since there are no spillway gates the operation and maintenance budget for the spillway would be considerably less than it is now. General maintenance on the spillway and spillway deck would be unchanged.

Without gates backing up water there will be less head differential across the spillway, and the spill will have less energy entering the stilling basin. Therefore, any rocks drawn into the stilling basin could be less likely to be flushed out. Rocks trapped in the stilling basin can erode the basin causing increased maintenance. The amount of repair work that might be required is impossible to estimate at this time, but it would probably be more than the repair work required at the existing basin.

SECTION 4 ALTERNATIVE 2 - DRAWDOWN TO SPILLWAY CREST WITH FLOOD CONTROL

In this alternative the spillway gates will not be removed. They would be raised during normal operation, and the river will run uncontrolled over the spillway. When the flow at the flood control point at Vancouver is reached, the gates would be lowered to achieve up to 500,000 acre-feet of flood control storage at the John Day Project. The set point for triggering flood control operations at John Day would probably be lower than the 10-year flood design flow of 515,000 cfs used for fish passage. Therefore, the design forebay elevation over the range of operating conditions would be about 25 feet higher to account for use of the full 500,000 acre-foot flood control pool.

With this alternative the spillway gates can be used to control depths upstream of the dam. This would provide a benefit to operating the two fish ladders because the proper flow and head drops between pools could be maintained more closely.

Implementation of this alternative would involve modifying or replacing the navigation lock, the adult and juvenile fish passage facilities, and the power generating equipment. There are no effects to the powerhouse structure or embankment sections on the north and south sides of the project. Modifications to other project features might also be required. The options for modification or replacement of project features are described below.

4.1 North Shore Fish Ladder

4.1.1 North Shore Fish Ladder Description

The modification option chosen for the NSFL consists of the full reconstruction of the fish ladder from the entrance channel to the new fish ladder exit. The ladder is designed for operation over the design operating water levels in the forebay and tailrace as described in Section 2 of this report (213 feet to 252 feet).

For this alternative, a variable length vertical slot fish ladder will be constructed with four exits to allow operation over the range of headwater and tailwater fluctuations. As the forebay level and the differential between the forebay and the tailwater level increases, additional sections of the ladder are used to raise the fish to the forebay level. Exits No. 1 through 4 are provided for each of the four ladder configurations, 52 pool, 60 pool, 70 pool and 80 pool, respectively. The low level outlet is for the 52 pool configuration and highest exit is for the 80 pool configuration. A plan view and elevation view are shown on Plates 12 and 13.

The existing fish ladder would be demolished from the existing construction joint between weirs 155 and 156. The south wall of the existing fish ladder and water supply conduit would be preserved and used for the new ladder. Diffuser Nos. 1 and 2 are maintained as well as the existing fish ladder entrance gates. The floor elevation would be lowered to elevation 148 (2 feet) between the start of the new ladder and Diffuser No. 2. Relocation of the water supply conduit bulkhead gate at the west end of the ladder would be required. The existing ladder exit would be abandoned and filled with concrete. The interpretive center above the NSFL will remain, but all counting and viewing facilities will be removed. Public access to the new viewing area will be provided.

The new ladder will be a vertical slot fish ladder as described in Section 3.1.1. The alignment is shown on Plate 12. A new counting and viewing facility is provided at pool 38 at invert elevation 185 feet. Counting and viewing facilities are assumed to be similar to the existing facilities but will be completely reconstructed. Exit channels are 7 feet wide and 24 feet high. Exit inverts are 200, 208, 218 and 228 for the 52 pool, 60 pool, 70 pool and 80 pool configurations. Exit nos. 1 and 2 are located near the existing ladder exit. Exits 3 and 4 are located close to spillway bay no. 1. To reduce the potential for fall-back through the spillway and maintain acceptable transport velocities, the exit channels will be extended away from the spillway. A trashrack and bulkheads are provided for all exits.

The auxiliary water system would be modified to fill existing Diffuser Nos. 3 through 15 with concrete. The water supply conduit bulkhead structure on the water supply conduit at the west end of the ladder would be relocated. No changes are required to the auxiliary water pumps.

The majority of the new fish ladder would be below the current deck level of 185 feet. Walls in the lower part of the ladder provide flood protection to this level. At the counting and viewing structure the ladder would be elevated. Construction should be similar to the existing ladder.

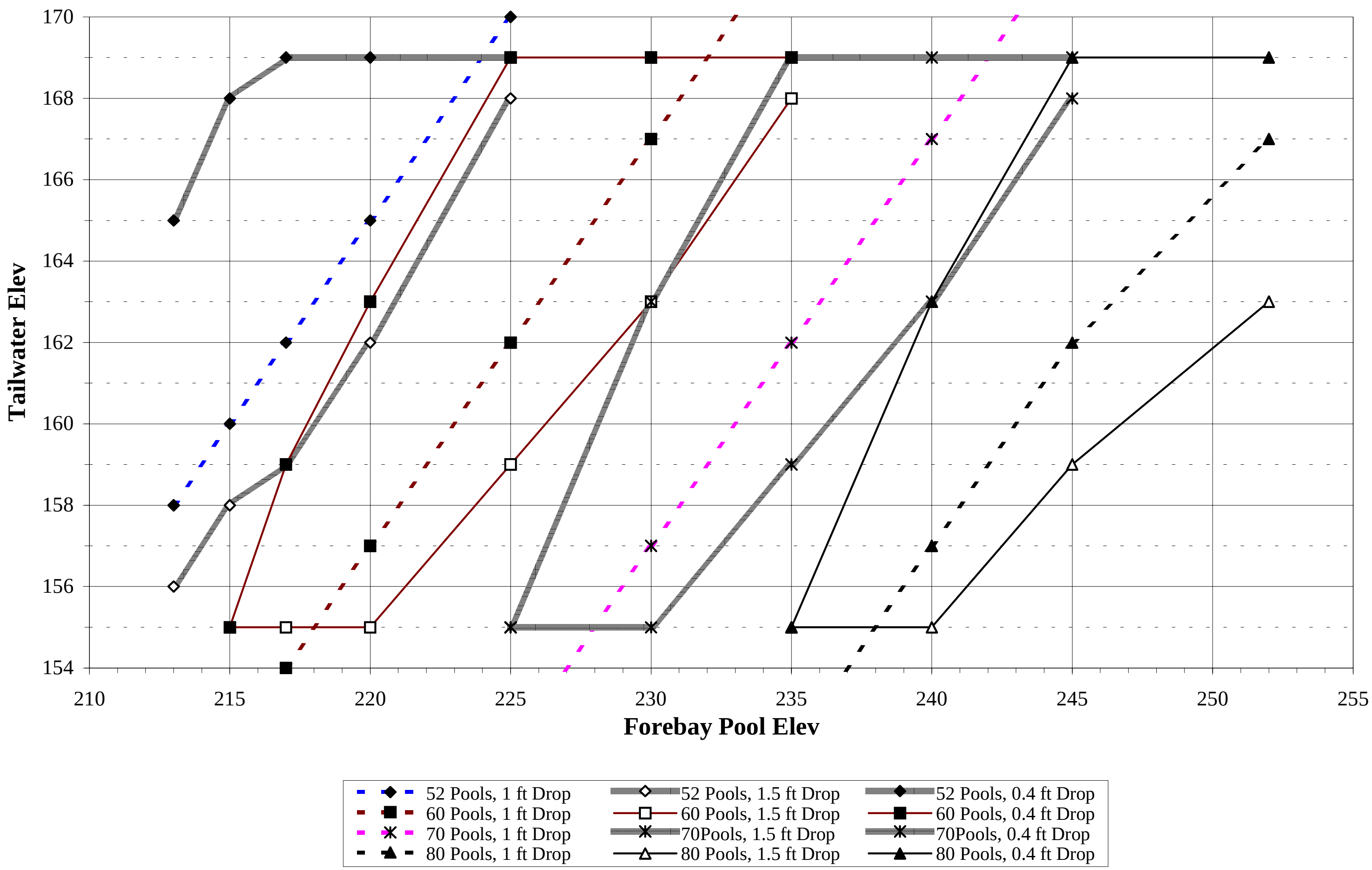
Two other fish ladder options were considered at the 30 percent level: partial reconstruction with a new regulating section (2N2) and partial reconstruction with fish locks (2N3). The option of partial reconstruction with a new regulating section could not accommodate the large operating range and maintain self regulation. Partial reconstruction with fish locks was not brought forward because there were too many fish to handle. Experience at other projects indicated that lock systems were not as successful as ladders in passing large quantities of fish without delay and stress.

4.1.2 North Shore Fish Ladder Operation

Project operation described in the scope of work assumes a normal operation to maintain the pool at elevation 215 ±2 feet between April and August and at 220 feet for the remainder of the year with a maximum forebay of 252 feet with flood storage. The 52 pool and 60 pool fish ladder would be operated as described in Section 3.1.2, and the operation of the 70 pool and 80 pool ladders would be similar.

Operating ranges for each configuration provide for 10 to 20 feet of forebay fluctuation. As with Alternative 1 (refer to Section 3.1.2), coordinated operation of the John Day Project and The Dalles Project would be required to keep the ladders operating within the criteria for drop between pools of 0.4 feet to 1.5 feet with a target of 1.0 feet. Figure 4-1 shows the forebay-tailwater fish ladder operating ranges for all four ladder configurations. A narrative description of how to use this figure is presented in Section 3.1.2. Dashed lines represent the forebay-tailwater combinations at which the ladder(s) can be operated at the target 1 feet drop between pools. If the combination forebay-tailwater falls to the right of the dashed line, then the drop between pools will exceed one foot in the lower parts of the fish ladder. If the combination falls

Figure 4 -1
Alternative 2 - Fish Ladder Operating Ranges



to the left of the dashed line then the drop between pools will be less than the target one feet drop between pools.

Flow conditions in the ladder would be as described in Section 3.1.2. Exit channels are seven feet wide and would provide a two fps transport velocity to the forebay. The transport velocity stays the same for varying forebay levels.

4.2 South Shore Fish Ladder

4.2.1 South Shore Fish Ladder Description

The modification option chose for the SSFL consists of the full reconstruction of the fish ladder from the entrance channel to the new fish ladder exit. The ladder is designed for operation over the design operating water levels in the forebay and tailrace as described in Section 2 of this report (213 feet to 252 feet). A variable length vertical slot fish ladder is provided with 52 pool, 60 pool, 70 pool and 80 pool configurations as described previously in Section 4.1.1. A plan view and elevation view are shown on Plates 14 and 15.

Excavation in the channel of the dam at the ladder exits will be similar to that described in Section 3.2.1, but will be extended to include fish ladder Exit Nos. 3 and 4 as shown on Plate 14.

Auxiliary water system modifications are required as described in Section 3.2.1.

Two other fish ladder options were considered at the 30 percent level: partial reconstruction with a new regulating section 2S2) and partial reconstruction with fish locks (2S3). The option of partial reconstruction with a new regulating section could not accommodate the large operating range and maintain self regulation. Partial reconstruction with fish locks was not brought forward because there were too many fish to handle. Experience at other projects indicated that lock systems were not as successful as ladders in passing large quantities of fish without delay and stress.

4.2.2 South Shore Fish Ladder Operation

SSFL operation would be the same as described for the NSFL in Section 4.1.2.

4.3 Downstream Passage

4.3.1 Impacts to Existing Facilities

Downstream passage for juveniles would be provided by using the existing spillways and a reconstructed juvenile bypass system (JBS) in the powerhouse.

Structural modifications required for the new lower forebay level and larger operating range include:

- Modifications to the bar or submerged traveling screens (screens). Extended length bar screens (ESBS) are assumed to be required
- Modifications to the barrier screens (VBS)
- Bore two new fish transportation conduits (FTC) the length of the dam
- Bore new orifices
- Modify the existing service gallery from which to maintain the orifices
- Construct new transport channels and dewatering screens
- Construct a new elevated bypass transportation flume
- Modifications to the existing evaluation and monitoring building

Structural modifications to the existing screens and VBS are included in this alternative. If the existing equipment will not function properly with necessary modifications, new screens and VBS's will be constructed. Modifications required for this alternative will be the same as described for Alternative 1 in Section 3.3.1.

A new system is also required to transport fish downstream of the dam from the gate well and VBS. See Plate 16 for the location and details. Two orifices, at two different elevations, would be bored from the gate well to new FTC's at each of the 48 bays. The centerline of each line of orifices are at elevation 208 feet, 217 feet, 226 feet and at 235 feet. Allowing for a submergence of five feet, the minimum forebay water level for each orifice is 213 feet, 222 feet, 231 feet and 240 feet, respectively. Only one line of orifices would be used at a time. Similar to Alternative 1, two new FTC's would be constructed the full length of the active powerhouse through Unit 16. However, due to the larger operating range the conduits would be separate bores with the lower invert at 200 feet and the upper conduit invert at 218 feet. Both FTC's increase from a 3 feet square section at Unit 16, to 8 feet square section at the south end of the powerhouse. The floor slopes downward to the south.

A new service gallery would not be constructed for this alternative due to the distance and difficulty in boring to each of the orifices. Orifice gate controls and lighting equipment would be located in the existing service gallery. Conduits would be excavated into the wall of the gate wells from the existing service gallery to each FTC.

Both FTC's exit the south end of the dam above the outfall chute. Each FTC would transition into a separate elevated concrete transportation flume five feet wide. The lower flume is envisioned to have a telescoping weir gate to regulate the head in the lower FTC. The upper flume would be constructed parallel to the lower flume and would also include a telescoping weir gate. The existing dewatering facility would be demolished and replaced with a new dewatering structure having two sets of screens to accommodate both flumes.

The juvenile bypass outfall pipe and evaluation features would be lowered to match the new dewatering structure. The discharge into the outfall chute would remain unchanged. The evaluation structure would require modification to collect fish at the lower elevation.

Three additional options were considered at the 30 percent level: existing spillway with no JBS (2J2), surface bypass spillway with a JBS (2J3), and surface bypass spillway with no JBS (2J4). Option 2J2 was discarded since it would no meet the 80 percent fish passage efficiency requirement. Options 2J3 and 2J4 were not selected because they involved a surface by-pass spillway similar to that in DM No. 52. With a lower water surface upstream, juveniles coming down river along the left bank would be more likely to enter the turbine inlet before reaching the surface bypass spillway. In addition, a surface bypass on the front of the dam was not judged to be satisfactory since criteria and design of a successful surface bypass system in not at a level that can be used as a model for an alternative study.

4.3.2 Operation

Operation of the JBS requires monitoring of the forebay level and selection of the correct orifices and FTC to use. Orifices are sized for eight fps. The FTC size would increase to maintain transport velocities between four fps and 10 fps. It is assumed the hydraulic design of the new concrete flume, dewatering structure and transportation flume would match the existing features.

4.3.3 Biological Considerations

Predicting the passage characteristics of the JBS has proven to be a challenge under current operations and is expected to be the same for this alternative. It is estimated that a long evaluation program will be required to properly configure and test the new JBS system.

4.4 Hydroturbine Operation

Turbine operations and modifications would be the same as those in Alternative 1. See Subsection 3.4.

4.5 Navigation Lock

The operation of the dam and navigation lock under Alternative 2 would be the same as that for Alternative 1 except under Alternative 2 under flood control conditions (flows above about 450,000 cfs), the gates would be lowered raising the water in the John Day pool for flood control. This is expected to occur only once every two to five years. However, fish passage would be maintained for flows up to 515,000 cfs, even during flood control operations.

The mode of operation and the changes to the structures would be the same as for Alternative 1. That is, the upstream sill would be cut down to elevation 195.0 and a 74-foot high miter gate would be installed on the upstream sill. This gate would be high enough to cover all water levels during flood control operation. Stoplogs would be provided for dewatering the upstream lock gate. A new water intake would be constructed upstream as described in Subsection 3.5.

Operationally, there is no difference between Alternatives 1 and 2 until flood control operations commence. Under flood control operations the upstream head could be as much as about 20 feet higher than in Alternative 1. Since the new gate extends up to elevation 269, the lock is protected from overflow under flood control operations to the present level of flood protection.

Two options were considered for the navigation lock at the 30 percent level: modify the existing lock and navigation lock replacement. The option of modifying the existing lock was selected for reasons of cost.

4.6 Spillway and Stilling Basin

The spillway and stilling basin would be operated the same in both Alternatives 1 and 2. However, during flood control the tainter gates would be lowered, and the spill would have as much as 20 feet of additional head and energy. Since about 97 percent of the time it would be operating identical to Alternative 1, the same potential problems would be encountered. That is, rocks could be drawn into the basin potentially eroding the concrete causing higher maintenance costs. A model study should also be performed to assess the need to relocate the spillway flow deflectors to reduce gas super saturation. For this study, it is assumed that the spillway flow deflectors will have to be relocated.

4.7 Project Sequencing

This subsection first describes the constraints on construction activities. This is followed by a description of the project schedule from completion of the Phase I Report through completion of construction and full implementation of the drawdown.

4.7.1 Project Constraints

Land access to the project for construction is limited. Much of the access to the project will have to be from the water. It is probable that much of the construction will be staged from barges and that transportation of materials to and from the site will be by barge.

In addition to site and access constraints, the construction schedule is affected by three operational items which involve fish passage requirements. These restrictions are in-water work periods, fish transport spill, and the requirement that fish passage be provided at all times, except during the three-month in-water work period.

Any construction work in the water is allowed only between December 1 and March 1. Construction behind cofferdams is not in-water work and, therefore, is not subject to this requirement.

In the spring and early summer if water is available, more flow is released into the river from upstream storage and is sent over the spillways. This restricts work on and near the spillways. The spill period typically extends from April 1 to July 31.

Another constraint on construction sequencing is to provide for navigation through the project at all times to the extent practical.

4.7.2 Project Schedule

The feasibility study/EIS is assumed to start in October 2000 and last 5 years. The design memorandum and production of plans and specifications is assumed to start in October 2005 and take about 3-½ years. The project schedule is shown on Figure 4-2 and starts with advertising for construction bids. Alternative 2 is the same as Alternative 1 except that during flood control operations the project must provide fish passage and barge traffic. The differences between Alternative 1 and Alternative 2 are:

- The north and south shore fish ladders will have two additional exits to the forebay pool with the associated additional ladder segments.
- An additional fish collection channel and side by side transport channels are required in the JBS.
- The existing spillway tainter gates will be retained and operated for flood control under Alternative 2.

The paragraphs below describe the construction sequencing by project feature for Alternative 2.

4.7.3 North Shore Fish Ladder

The only location for upstream adult fish passage is at the existing north and south shore fish ladders. Since most of the ladder structure has to be demolished, the only way to provide constant fish passage is to continue one ladder in operation while reconstructing the other. Providing for a trap and haul operation at the ladder while it is under construction is not considered feasible. The limited space available will not support both a trap and haul operation and construction of ladder modifications. In addition, trapping and hauling endangered fish passing the project would not be acceptable.

The NSFL would be rebuilt first while the SSFL remains in operation. For upstream fish passage during the construction period, flow through the turbines and attraction water in the SSFL should be maximized. During the drawdown of the reservoir upstream passage will not be provided under this scenario. One possibility that should be considered in the FDM phase would be to construct a false weir in the SSFL with a temporary flume at the exit to the forebay. The return flume would have to be adjustable to follow the reservoir level down through the full drawdown.

It is anticipated that the NSFL and navigation lock work could occur at the same time during the first part of construction. Work would begin with dewatering the ladder in early December. The ladder would then be demolished, and reconstruction of the new ladder would begin. After the ladder is demolished and the upstream cofferdam around the navigation lock and fish ladder exit is complete, the new fish ladder exits would be mined. Then the gates would be installed over the exits. This work can be coordinated with the navigation lock construction.

More time would be required to construct the NSFL than was required in Alternative 1. The ladder for Alternative 2 will have four exits in order to cover the wider range in pool elevations

Act ID	Description	Orig Dur	Start	Finish
Water Work Windows				
0010	In Water Work Window (09-10)	90	01DEC09 *	01MAR10
0020	In Water Work Window (10-11)	90	01DEC10 *	01MAR11
0030	In Water Work Window (11-12)	90	01DEC11 *	29FEB12
0040	In Water Work Window (12-13)	90	01DEC12 *	01MAR13
0050	In Water Work Window (13-14)	90	01DEC13 *	01MAR14
0060	In Water Work Window (14-15)	90	01DEC14 *	01MAR15
Spill Periods				
0080	Spill Period (09)	121	01APR09 *	31JUL09
0090	Spill Period (10)	121	01APR10 *	31JUL10
0100	Spill Period (11)	121	01APR11 *	31JUL11
0110	Spill Period (12)	121	01APR12 *	31JUL12
0120	Spill Period (13)	121	01APR13 *	31JUL13
0130	Spill Period (14)	121	01APR14 *	31JUL14
Bid and Award				
4100	Advertise for Bid	0	01APR09 *	
4200	Bid And Award	180	01APR09	28SEP09
4300	Award Construction Contract	0	28SEP09	
North Shore Ladder				
5100	Dewater NS Ladder	5	01DEC10 *	06DEC10
5200	Remove Ladder Structure	150	06DEC10	05MAY11
5300	Build Ladder	450	05MAY11	29JUL12
5400	Mine Ladder Exits	270	05MAY11	30JAN12
5500	Install Exit Gates	100	30JAN12	10MAY12
5600	Place NS Ladder in Operation	15	20JAN13	04FEB13
Navigation Lock				
6100	Mobilize	90	28SEP09	27DEC09
6200	Construct Intake Cofferdam	90	01DEC09 *	01MAR10
6300	Construct Remainder of Intake Cofferdan	90	01DEC10 *	01MAR11
6400	Construct the Lock Fill Intake	639	01MAR11	30NOV12
6500	Remove Intake Cofferdam	90	01DEC12 *	01MAR13
6600	Fabricate New Upstream Lock Gate	450	01SEP11 *	25NOV12
6700	Construct Lock Cofferdam	40	19JAN13 *	28FEB13
6800	Remove Upstream Guidewall	15	19JAN13 *	03FEB13
6900	Remove Upstream Sill to El. 195	45	28FEB13	14APR13
6920	Bore New Cross Over Tunnel	90	01JAN13 *	01APR13
6940	Install New Upstream Lock Gate	45	14APR13	29MAY13
6960	Reinstall Upstream Guidewall	30	14APR13	14MAY13
6980	Remove Lock Cofferdam	30	29MAY13 *	28JUN13
Spillway				
7100	Remove and Replace Flow Deflectors	90	01DEC10 *	01MAR11
7200	Remove and Replace Flow Deflectors	90	01DEC11 *	29FEB12
7300	Remove and Replace Flow Deflectors	90	01DEC12 *	01MAR13

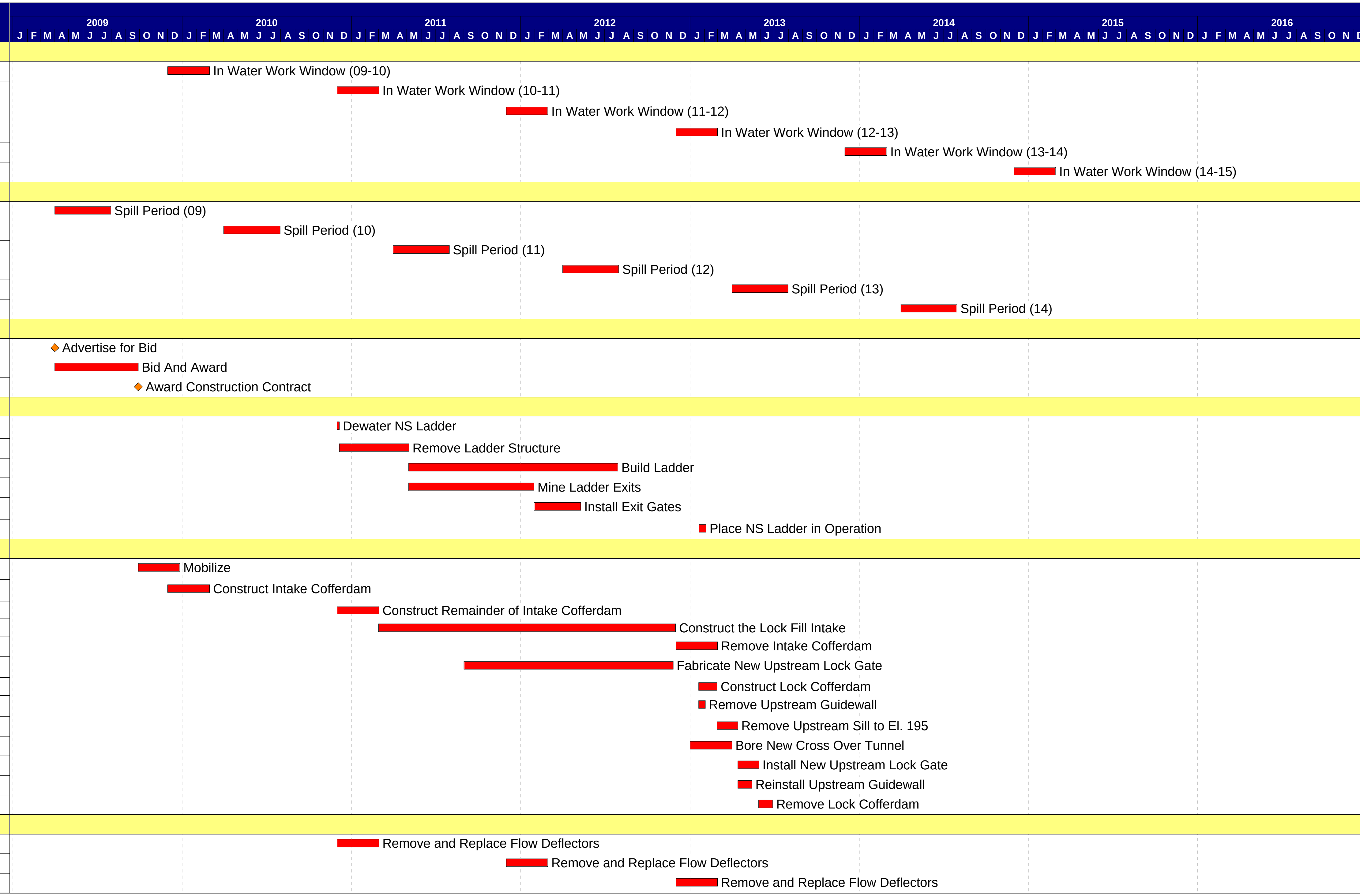


Figure 4-2
Schedule
Alternative 2 - Drawdown to Spillway Crest with Flood Control

Act ID	Description	Orig Dur	Start	Finish
Juvenille Bypass System in Dam				
8100	Award Dam JBS Contract	0	07SEP09 *	
8200	Mobilize	60	07SEP09	06NOV09
8250	Fabricate New Turbine Screens	720	07NOV09 *	28OCT11
8300	Bore Vertical Shafts	100	06NOV09	14FEB10
8400	Mine Juvenile Collection Conduits	300	14FEB10	11DEC10
8500	Bore Orifices	420	11DEC10	04FEB12
8600	Install Orifice Gates and Utilities	240	04FEB12	02OCT12
8700	Install Turbine Screens	150	02OCT12	01MAR13
Reservoir Drawdown				
9100	Raise Gates and Drawdown Reservoir	50	01DEC12 *	20JAN13
Juvenile Bypass System Outside Dam				
10100	Award South Shore Contract	0	07SEP09 *	
10200	Mobilize	50	07SEP09	27OCT09
10300	Divert Flow to Old Bypass System	10	27OCT09	06NOV09
10400	Remove Dewatering Structure and Flumes	200	06NOV09	25MAY10
10500	Construct Downstream JBS Facilities	500	25MAY10	07OCT11
10600	Complete Juvenille Collection Channels	300	03FEB14	30NOV14 *
10700	Connect Upstream JBS Facilities	120	30NOV14	30MAR15
South Shore Ladder				
11100	Install Auxiliary Water Pumps	400	07OCT11	11NOV12
11200	Fabricate Dewatering Bulkhead	200	07OCT11	25APR12
11300	Dewater South Shore Ladder	10	04FEB13	14FEB13
11400	Remove Ladder Structure	180	14FEB13	13AUG13
11500	Place Bulkhead and Mine Ladder Exits	200	13AUG13	01MAR14
11600	Build Ladder Structure	450	01MAR14	25MAY15

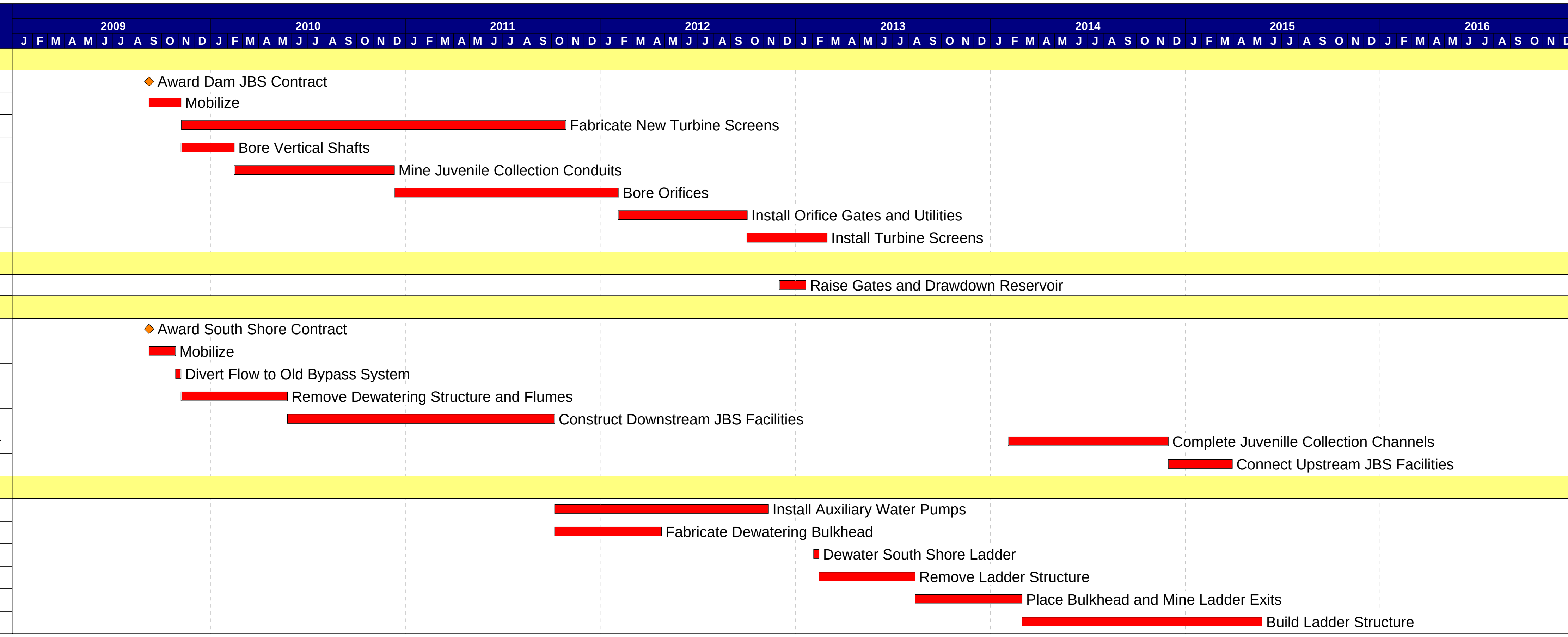


Figure 4-2
Schedule
Alternative 2 - Drawdown to Spillway Crest with Flood Control

required of this alternative. Although additional time is required to build the extra two exits, it can be built within the two years allotted.

4.7.4 Navigation Lock

The navigation lock construction sequencing would be the same as in Alternative 1. See subsection 3.7.4. The navigation lock would be out of service about 6 months. To shorten this period a new navigation lock would have to be constructed. So, a six month outage is employed as being more economical.

4.7.5 Juvenile Bypass System in Dam

It is envisioned that a new JBS will have to be constructed for the spillway crest drawdown. It would resemble the present one but would be about 45 feet lower. The plan calls for two new FTCs. They would be constructed by boring two separate tunnels. Two conduits, further apart than in Alternative 1, are required due to the greater range in pool water surface elevations. Each conduit would have its own orifices (two per intake bay, six per generating unit) for transferring fish from the gatewell into the collection conduits. These facilities can be built at any time since no in-water work is required. This work is scheduled while the north shore work is underway and prior to the south shore work to avoid conflicts during construction of the south shore facilities.

During construction, the first step is anticipated to be boring vertical shafts at the north and south ends of the powerhouse deck east of the existing conduit. These shafts would be used to start mining the FTCs and the service gallery for the JBS.

The orifices would be constructed at the individual units by first stopping the unit. Then, the area over the orifice in the gate well would be dewatered by sealing a bulkhead over the orifice location and drilling the orifice from the FTC to the dewatered area in the gatewell. Gates would then be installed over the orifices and the bulkhead removed. This would be repeated at the other powerhouse units. Construction in this manner would allow operation of all units except the one where the orifices are being installed.

4.7.6 Spillway

Due to the reduced head on the spillway after drawdown, the deflectors would not be at their optimum location. For a conservative cost estimate it is assumed that the existing deflectors would have to be removed and new ones added. This work would require dewatering using bulkheads similar to the construction of the existing deflectors. The deflector construction would take place prior to reservoir drawdown.

4.7.7 Reservoir Drawdown

After the NSFL and navigation lock intake work is completed, the reservoir can be drawn down. It is assumed that the drawdown would take place over 50 days. This is about one foot of drawdown per day to reduce slope stability problems on the reservoir banks as much as possible and still draw the reservoir down in one in-water work period. Therefore, interim fish passage measures would not be required during drawdown. As soon as the upstream water level reaches a level within the ladder's new operating range the NSFL would be placed in operation.

4.7.8 Juvenile Bypass System Outside the Dam and South Shore Fish Ladder

Construction of the SSFL and JBS facilities can follow the same schedule as that for Alternative 1. However, under Alternative 2 there would be two more ladder exits and additional ladder to reach them, an additional FTC exiting the dam, and an additional elevated fish transportation flume. Therefore, the construction time would be greater.

The existing juvenile bypass system outside the dam is located adjacent to the SSFL. So, it is best to construct these at the same time and under the same construction contract.

The schedule on Figure 4-2 shows the construction tasks for the JBS and the SSFL under different headings. However, these tasks are described together below in chronological order for the sake of clarity.

There is a great deal of construction work to be performed in a small area on the south shore. This increases the construction time. To minimize the length of construction and the time, in which only one ladder is in service, work would begin before work on the north shore is complete. First, the crest gate would be opened diverting the flow down the chute and into the tailrace. This dewaters the JBS west of the crest gate. Next the existing dewatering structure, fish transportation flumes, and evaluation facilities west of the crest gate would be demolished. See Plates 14, 15, and 16. Since the new JBS would be the same as the present one but at a lower elevation, some of the mechanical equipment can probably be salvaged for use in the new JBS. Next the juvenile bypass outfall, transport flumes, dewatering structure, and evaluation facilities west of the existing fish ladder would be built. At the same time the new auxiliary water supply pump motors would be installed.

After the reservoir is drawn down and the NSFL is in operation, work can begin on the SSFL. First, the entire ladder would be demolished and removed. A dewatering bulkhead would be fabricated and placed on the upstream side of the dam to dewater the fish ladder exits. The four fish ladder exits would be mined in the same manner as those on the north shore. The ladder would then be constructed and connected to the exits. Concurrently, the last reach of the FTC at the south end of the dam would be constructed from downstream side of the dam. Finally, the fish transportation flumes adjacent to the fish ladder would be built and connected to the end of the FTCs at their south end.

During construction of the SSFL, powerhouse flow should be minimized to reduce the attraction to the south shore. In addition, spillway gates could be operated to direct spill to help guide fish to the north shore. Attraction flow at the NSFL should be maximized at all flows until the SSFL is completed.

4.8 Operation and Maintenance Considerations

This subsection describes the operation and maintenance requirements only for those features impacted by the drawdown. These requirements are described in a general manner consistent with a reconnaissance level study. The operation and maintenance requirements would be the same as in Alternative 1 except that the spillway tainter gates would be operated under Alternative 2.

The operation and maintenance requirements are described below for each feature that will be impacted by the drawdown.

4.8.1 North Shore Fish Ladder

The NSFL is designed to operate over a range of river flows from a low of 80,000 cfs to the 10-year high flow of 515,000 cfs. At forebay levels above elevation 255 feet all exit gates must be closed. If they are not shut water could overflow the sides of the ladder. The AWS would operate the same as it does now. However, only the entrances would require auxiliary water. Refer to Section 4.1.2 for additional operational characteristics. Sequencing for changing between ladder configurations will be similar to that described in Section 3.8.1 except that two additional exits would require operation when the project is under flood control operations.

Optimal operation of the fish ladders described for this alternative requires a philosophical change to the current operation of both The Dalles Dam and John Day Dam. Power production and/or spill at both projects will have to be coordinated to maintain the fish ladders at or near the operating conditions for a 1 feet drop between pools (dashed lines on Figure 4-1). Items for consideration during the next phases of this project include:

- possible modifications to the entrance channels to extend the effective tailwater range to either reduce the operational requirement on The Dalles or improve the operational range of the fish ladders closer to the 1 feet drop line.

- Investigate use of auxiliary water and / or adjustable width slots to improve the operational range of the fish ladders.

- Use of a false weir and chute at the exit to limit or reduce the switching between ladder configurations.

4.8.2 South Shore Fish Ladder

The SSFL would operate the same as the NSFL. However, the auxiliary water supply is different. Only half of the auxiliary water could come from the turbine pumps which now supply all auxiliary water needs. Electric motors will be provided to drive the turbine driven pumps. Auxiliary water would be supplied to all diffusers except those further up the ladder. Refer to Section 4.2.2 for additional operational characteristics. Sequencing for changing between ladder configurations will be similar to that described in Section 3.8.1.

4.8.3 Downstream Passage

Operation and maintenance for the JBS facilities would be the same as for the existing facilities. The dewatering and monitoring structures downstream of the dam would be the same as the existing structure except they would be lower in elevation. Therefore, the operation and maintenance requirements for these facilities would also be the same. Since four orifices and two FTCs are required, operation of these items will require more effort than that required for existing operations.

4.8.4 Hydroturbine Operation

Hydroturbine operation and maintenance considerations would be the same as those presented for Alternative 1. See Subsection 3.8.4.

4.8.5 Navigation Lock

In this alternative the navigation lock will be the same as the existing structure except that the upstream gate will be 74-foot high miter gate rather than a smaller lift gate. Filling and emptying the lock will have lower velocities in the system since the head on the structure will be less. Overall, the operation and maintenance effort for the lock would be about the same as at present.

Additional operation and maintenance would be required for the two intakes. Operational requirements include cleaning both the trashrack and the fish screens. A portable cleaning system would be employed for cleaning the trashrack. An underwater brush sweeping system or a back spray system would automatically clean the fish screens periodically triggered by a timer or differential head across the screens. Either system would require periodic maintenance and inspections by divers.

4.8.6 Spillway

In this alternative the spillway gates would be retained. They would be in the fully raised position most of the time and lowered only during flood control operations. The operational requirements would be greatly reduced since the gates would no longer control the forebay pool. However, the maintenance requirements for the gates and operators would probably remain the same because the gates must be ready for operation during the flood season every year. General maintenance on the spillway and spillway deck would be unchanged.

There would be less head differential across the spillway and less energy entering the stilling basin. Therefore, any rocks drawn into the stilling basin should be less likely to be flushed out. Rocks trapped in the stilling basin can erode the basin causing increased maintenance. The amount of repair work that might be required is impossible to estimate at this time, but it should be more than the repair work required at the existing basin.

Since the intent of this alternative is to only use the spillway gate for flood storage the upstream fish guidance will be affected the same way as described for Alternative 1. During flood storage gates should be used to shape the spill to encourage fish passage.

SECTION 5 ALTERNATIVE 3 – DRAWDOWN TO NATURAL RIVER WITHOUT FLOOD CONTROL

In this alternative, near natural river hydraulic patterns will be re-established. No regulation of the river will take place. Enough of the dam structure will be removed to provide passage of upstream migrants at the 10-year flood of 515,000 cfs. A maximum average velocity of 10 fps has been set as the criteria for upstream fish passage.

During implementation of this alternative fish passage will be maintained at all times except for the in-water work period of December, January, and February.

5.1 Hydraulic Computations

Alternative 3 includes removing the spillway and a portion of the powerhouse to create hydraulic conditions similar to the pre-dam natural river channel. Several configurations were studied to determine the minimum amount of structural modifications required to meet fish passage and barge traffic criteria. The fish passage criteria includes a maximum 10 fps average velocity for a discharge of 515,000 cfs through the removed section of the dam. The barge traffic requirements include a target velocity of about five fps during Phase II construction for the majority of the construction duration. The barge traffic requirements during Phase II construction were the controlling factors in sizing the opening for this alternative. A numerical model was used to estimate flow characteristics along the modified reach of river for use in estimating the potential impact on fish passage. The hydraulic characteristics of the natural river drawdown option were analyzed using a HEC-RAS (Hydrologic Engineering Center's River Analysis System, Version 2.2) backwater model. Attachment B contains a detailed description of the HEC-RAS model used for the hydraulic computations. The model extends from RM 212.510 to RM 217.829 with the dam located between stations 215.636 and 215.535.

The different model runs include the following:

Run 3-1	Remove Spillway and Powerhouse
Run 3-2	Remove Spillway and Powerhouse Units 6-20
Run 3-3	Remove Spillway Only
Run 3-4	Remove Spillway and Powerhouse Units 11-20
Run 3-5	Remove Spillway and Powerhouse Units 16-20
Run 3-6	Remove Spillway and Powerhouse Units 17-20
Run 3-7	Remove Spillway and Powerhouse Units 10-20

The first four runs were modeled to study the sensitivity of the flow characteristics and specifically the velocity to different structural modifications. After studying the first four runs, the options were narrowed to Runs 3-5 and 3-6. These runs were modeled to determine the minimum amount of dam that would have to be removed to provide a velocity of about 10 fps through the removed portion for a discharge of 515,000 cfs and a The Dalles forebay elevation of 155.0 feet. Run 3-7 was completed to determine a configuration that would meet the barge traffic criteria.

Run 3-7 was selected from the seven runs because this run met the velocity criteria for both fish passage and barge traffic and minimized the amount of structural removal. The bottom elevations of the removed portion of the powerhouse and the spillway are 128.0 feet and 135.0 feet, respectively. A riprap dike extends downstream from the south side of the powerhouse until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore to the south side of Unit 10 at a 1:0.9 contraction ratio. Attachment B contains a detailed discussion regarding the ineffective flow areas and other hydraulic characteristics. Table 5-1 provides the average velocities and water surface elevations at the downstream face of the dam for Run 3-7.

Table 5-1
Run 3-7 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.76	1.48
80,000	160	160.43	1.24
515,000	155	167.61	6.36
515,000	160	169.45	6.05

The Run 3-7 model was also run for several other discharges between 100,000 cfs and 500,000 cfs. Attachment B contains the output for the additional discharges. Figure 5-1 provides a plot of the water surface profile for various discharges (The Dalles forebay of 155.0). The thalweg shown in Figure 5-1 represents the recently surveyed cross-sections from the HEC-RAS model developed by WEST Consultants for the Portland District.

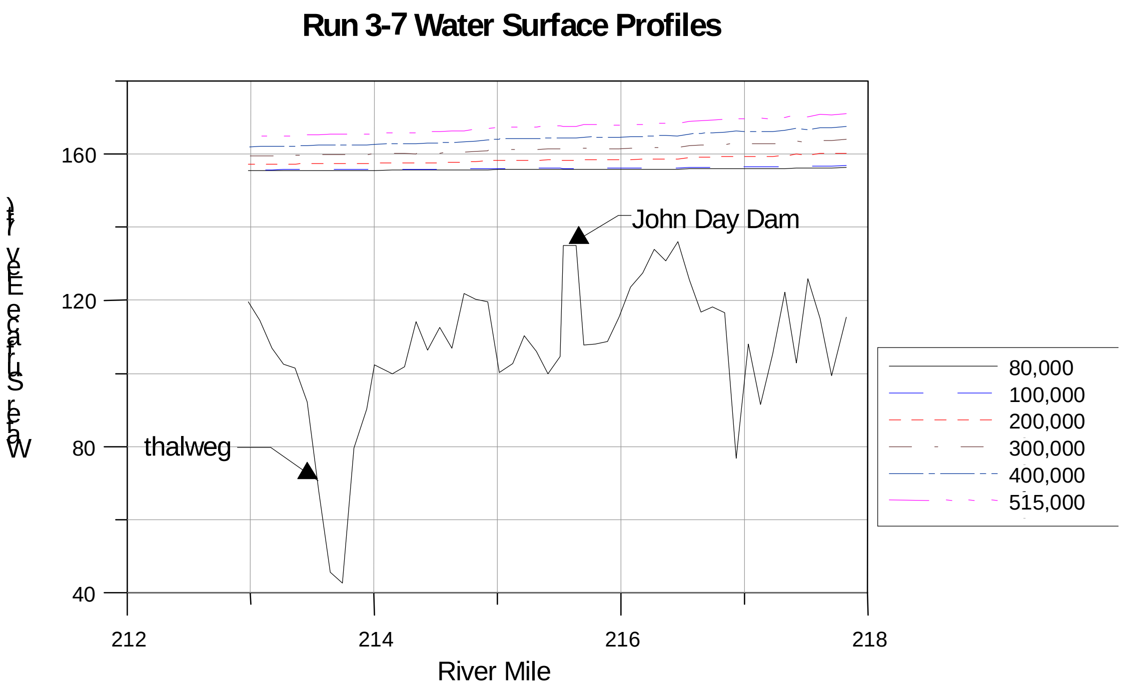


Figure 5-1
Run 3-7 Water Surface Profiles

5.2 Structure Removal

This alternative consists of the removal of the entire existing spillway, which is 1,228 feet long, the 37 feet long non-overflow section, and 996 feet of the powerhouse section (powerhouse units 10 through 20), for a total length of 2,261 feet (CBL Station 28+51 to 51+12). A plan view and cross sections are shown on Plates 17 and 19. See subsection 5.7 for a description of construction sequencing. The spillway concrete would be removed down to elevation 135, and the powerhouse and non-overflow sections would be removed down to elevation 128 as shown on Plates 19 and 20. During modeling of the natural river drawdown, removal of the spillway down to different elevations was tested. It was found that invert elevations lower than 135 in the spillway did not appreciably decrease the velocity through the beach. Much of the area upstream and downstream of the northern portions of the spillway was above elevation 140 and was excavated to 140 during construction. Therefore, cutting down the spillway below 135 provides little hydraulic advantage. The voids in the powerhouse below elevation 128 would be filled in to achieve a uniformly sloping channel. This provides safer hydraulic conditions by preventing local turbulence caused by these voids.

Three additional options for structure removal were considered at the 30 percent level: remove the north embankment and lower the ogee portion of the spillway (1), remove the spillway and north embankment entirely (2), and remove the entire dam (3). Consideration of these options at the 30 percent review meeting centered on where to locate the breach in the dam and whether to remove the dam or lower it. It was decided to locate the breach from the navigation lock to the south and to take out enough turbine bays to achieve the necessary width. Option 1 would require extensive excavation to provide an adequate river channel because the embankment foundation is considerably higher than the river channel. Option 3 would cost more than removing only enough of the dam to meet the fish passage velocity criteria. Removal of the embankment section to the north of the lock (considered at the 30 percent review meeting) would change the channel location too drastically and would require removal of the lock.

5.3 Upstream Passage

For natural river drawdown the structure will be removed as described in the previous sections to obtain satisfactory passage conditions. Upstream fish passage is provided in the boundary layer along the perimeter of the breach.

A narrow breach with added roughness elements was also considered. However, it was not selected because the reliability of the roughness elements to pass fish is unknown.

5.4 Downstream Passage

Downstream passage will be provided through the breach. No special features or operation are necessary.

5.5 Navigation Lock

River traffic was assumed to operate in water velocities of up to five feet per second. For the breach width considered for this alternative, navigation is possible through the breach in flows of about 400,000 cfs and below for water elevation of 160 at The Dalles. This occurs about 97 percent of the time.

For flows over 400,000 cfs river traffic should transit the breach through the navigation lock. The lock would operate without the existing filling and emptying system. See Plate 21. Upstream traffic would enter the lock channel with the upstream lock gate closed. After closing the downstream lock gate the upstream filling valve located in the bottom of the upstream gate would open, filling the lock. The upstream lock gate would then open and traffic would travel upstream in the navigation channel, which would have to be dredged about seven feet deeper. Downstream traffic would pass in a similar manner using the new filling and emptying valves to equalize water levels.

The lock would be modified for this alternative by removing the upstream sill to elevation 140 and installing a new upstream lock gate. Details are shown in Plate 21. A miter type gate about 65 feet tall would be installed. This would prevent overflow for river flows below about 2,000,000 cfs. The downstream gate would remain in service, and the existing inlets and outlets for the fill and drain system would be plugged with structural concrete. Both upstream and downstream lock gates would be fitted with valves at the bottom of the gates for filling and emptying the lock. The total head across the lock is expected to be from 1.5 to 2.5 feet at the design flow of 800,000 cfs. Stoplogs would be provided for dewatering the upstream lock gate.

The upstream navigation channel would be dredged to elevation 140. The area underneath the floating guidewall would also be dredged to 140 to provide space for the floating guidewall during lower water surface operations. A new mooring structure would be built to accommodate the floating guidewall operation at lower water surface elevations for this alternative.

Providing navigation through the breach in the dam was the only option considered at the 30 percent design level. It was then discovered that it was more economical to limit the width of the breach to that required for fish passage rather than for that required for barge traffic so lock modification strategies were pursued.

5.6 Restoration of Synchronous Condensing Operation

During certain times of the year (April-November), six units at John Day are currently dedicated to synchronous condensing operation (SCO) by agreement between Bonneville Power Administration (BPA) and the Corps. Under Alternative 3, portions of the powerhouse are to be removed and the power plant abandoned. Existing transmission system stability benefits and rating of inter-tie, as currently provided by SCO at John Day, is to be maintained. This would require conversion of six similarly sized units to SCO at another project. Costs of installation and maintenance are based on most recent cost data for the John Day Project.

5.7 Project Sequencing

This subsection first describes the constraints on construction activities. This is followed by a description of the project schedule from completion of the Phase I Report through completion of construction and full implementation of the drawdown.

5.7.1 Project Constraints

Land access to the project for construction is limited. Much of the access to the project will have to be from the water. It is probable that much of the construction will be staged from barges and that transportation of materials to and from the site will be by barge.

In addition to site and access constraints, the construction schedule is affected by three operational items, which involve fish passage requirements. These restrictions are in-water work periods, fish transport spill, and the requirement that fish passage be provided at all times except during the three-month in-work period.

Any construction work in the water is allowed only between December 1 and March 1. Construction behind cofferdams is not in-water work and, therefore, is not subject to this requirement.

In the spring and early summer if water is available, more flow is released into the river from upstream storage and is sent over the spillways. This restricts work on and near the spillways. The spill period typically extends from April 1 to July 31.

Another constraint on construction sequencing is to provide for navigation through the project at all times to the extent practical.

5.7.2 Project Schedule

In developing the project schedule it was assumed that the steps to implement the project would be the Phase II feasibility study and EIS, design memorandum, plans and specifications, and construction. For the purposes of this reconnaissance study all work prior to the start of construction is assumed to be the same for all alternatives. The Feasibility Study and EIS are assumed to start in October 2000 and last for five years. Preparing the design memorandum and plans and specifications is assumed to start in October of 2005 and take approximately three and one half years to complete. The construction bid process will start in April 2009.

The project schedule is shown on Figure 5-2. The paragraphs below describe the construction sequencing by task.

5.7.3 Dewatering and Reservoir Drawdown

Requirements for performing the reservoir drawdown are that fish passage occurs during all stages of construction. Fish passage must be by fish ladder and JBS. Trap and haul schemes are not acceptable due to the presence of listed fish and the large numbers of fish that must pass the project site.

Removal of the dam and powerhouse units will be accomplished in two stages. The reservoir drawdown will occur between the two stages. The river will be flowing in its natural channel during Stage 2 construction.

Stage 1

In this step the southern seven spillway bays, the non-overflow section, and Units 10 through 20 in the powerhouse will be removed. It is envisioned that the construction will take place as follows.

The upstream cofferdam will be up to 150 high. Therefore, a cellular sheetpile cofferdam is not feasible, and a dam type of cofferdam would be required. For this study an H-pile cofferdam is selected. See Plate 18. The wall key would be excavated into the riverbed from a barge. This will be accomplished with a clamshell if the riverbed is granular material. If the riverbed material is bedrock, the rock will be drilled and shot, and the shot rock will then be excavated with a clamshell. After the key is constructed, the initial fill will be placed with a clamshell from a barge. A cheaper method would be to simply dump the material from a barge, however turbidity could be a problem. Once the initial fill has been placed, the ARBED shapes can be driven with a vibratory hammer. Temporary support piles will be used to hold the H-piles in position and provide support during installation of shot rock fill. With the ARBED wall in place, the shot rock will be placed with a clamshell in 10-foot lifts up to approximately 80 feet above the original riverbed, and each lift would be vibro compacted to stabilize the material. Beginning at the 80-foot elevation mark, earth reinforcement will be placed in the fill material every 10 feet up to the top. This could be a steel mat grid or other reinforcement. Then

Act ID	Description	Orig Dur	Start	Finish
Water Work Windows				
0010	In Water Work Window (09-10)	90d	01DEC09 *	01MAR10
0020	In Water Work Window (10-11)	90d	02DEC10	02MAR11
0030	In Water Work Window (11-12)	90d	02DEC11	01MAR12
0040	In Water Work Window (12-13)	90d	01DEC12	01MAR13
0050	In Water Work WIndow (13-14)	90d	01DEC13	01MAR14
0150	In Water Work Window (14-15)	90d	02DEC14	02MAR15
Spill Periods				
0060	Spill Period (09)	121d	01APR09 *	31JUL09
0070	Spill Period (10)	121d	01APR10	31JUL10
0080	Spill Period (11)	121d	01APR11	31JUL11
0090	Spill Period (12)	121d	01APR12	31JUL12
0100	Spill Period (13)	121d	01APR13	31JUL13
0110	Spill Period (14)	121d	01APR14	31JUL14
Bid and Award				
4100	Advertise for Bid	0	01APR09 *	
4200	Bid And Award	100d	01APR09	10JUL09
4300	Award Construction Contract	0	07SEP09 *	
Stage 1				
5000	Mobilize	83d	07SEP09	29NOV09
5050	Fabricate Closure Gates, Units 1-4	400d	29NOV09	03JAN11
5075	Remove Machinery, Units 10-20	300d	29NOV09	25SEP10
5100	Construct PH Side of Stage 1 Cofferdam	90d	29NOV09	27FEB10
5200	Construct Rest of Stage 1 Cofferdan	90d	01DEC10 *	01MAR11
5250	Rmve Machinery, Roof, Wall; Units 1-4	400d	29NOV09	03JAN11
5275	Rmve Fish Collect. Ch. and Aux Water;Units 1-4	200d	03JAN11	22JUL11
5300	Remove Spwy Gates and Machinery	180d	01MAR11	28AUG11
5400	Remove Spillways 14-20, PH 10-20	490d	01MAR11	03JUL12
5500	Install Fish Passage Baffles	90d	03JUL12 *	01OCT12
5600	Build Stage 2 Cofferdam Inside	150d	03JUL12 *	30NOV12
5700	Remove Cofferdam	90d	01DEC12	01MAR13
Reservoir Drawdown				
6000	Drawdown Reservoir	100d	20NOV12 *	28FEB13
Stage 2				
7000	Construct Stage 2 Cofferdam	90d	01DEC12 *	01MAR13
7100	Remove Gates and Machinery	150d	01MAR13	29JUL13
7200	Complete Stage 2 Cofferdam	90d	01DEC13 *	01MAR14
7300	Remove Spill Bays 1-12	275d	01MAR14	01DEC14
7400	Remove Cofferdam	90d	01DEC14	01MAR15
Navigation Lock				
8000	Fabricate New Upstream Lock Gate	360d	08SEP12 *	03SEP13
8100	Remove Upstream Guidewall	15d	15NOV12 *	30NOV12
8200	Build Cofferdam	90d	01DEC12 *	01MAR13
8300	Remove Upstream Sill to El. 140	180d	01MAR13	28AUG13
8400	Install New Lock Gate	90d	03SEP13	02DEC13
8500	Dewater D/S Lock Gate	10d	09DEC12 *	19DEC12
8600	Install Emptying Valve in D/S Lock Gate	90d	19DEC12	19MAR13
8700	Remove Cofferdam	30d	02DEC13	01JAN14
8800	Dredge Upstream Navigation Channel	90d	01DEC13 *	01MAR14

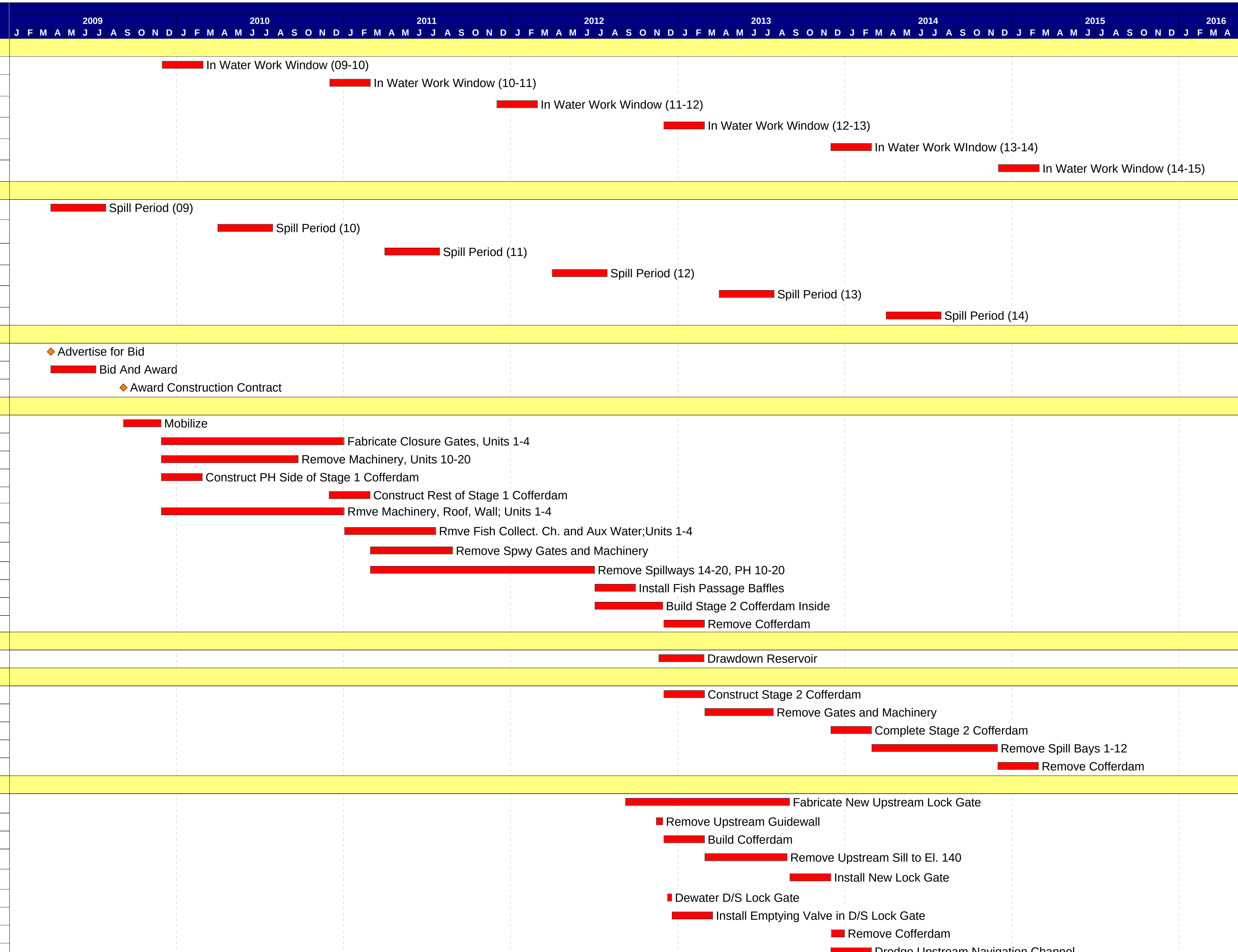


Figure 5-2
Schedule
Alternative 3 - Drawdown to Natural River without Flood Control

Act ID	Description	Orig Dur	Start	Finish
Flow Control Berm				
9000	Place Upstream and Downstream Berm	90d	01DEC11 *	29FEB12
9100	Place Upstream and Downstream Berm	90d	01DEC12 *	01MAR13
9200	Place Upstream and Downstream Berm	90d	01DEC13 *	01MAR14

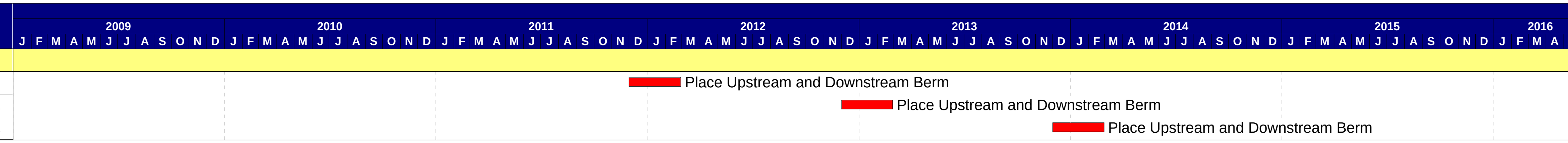


Figure 5-2
Schedule
Alternative 3 - Drawdown to Natural River without Flood Control

subsequent 10-foot lifts and vibro compaction will continue until full height is reached. At the final elevation of 268 an access road and amenities as required can be added. Outer layers of the shot rock should include materials which will not wash away with wave action or reservoir drawdown.

The downstream cofferdam will be constructed of sheetpile cells west of the stilling basin and the adverse slope downstream of the powerhouse. The northern sheetpile cells will be anchored to the stilling basin slab to protect the cofferdam from flow over the spillway. Permanent stoplogs will be placed to close off the draft tubes. Additional fill will be placed behind the draft tubes of Units four through eight to support the cellular sheetpile cofferdam in the deeper area near the powerhouse.

Since the cofferdams are so large, they will have to be built in two in-water work periods. In the first year the cofferdams upstream and downstream of the powerhouse would be constructed. See Plate 18. At this point the powerhouse will be taken out of service since the first year cofferdams will block the flow into and out of the turbines. The auxiliary water system would still operate. In front of the spillway the wall key will be excavated and the initial fill for supporting the H piles will be placed. The initial fill will be far enough upstream of the dam so that it will not be eroded by spillway flows. One year later, during the in-water work period, the rest of the cofferdam will be constructed as described above.

The Spillway Bays 14 through 20, the non-overflow section, and Units 10 through 20 will be removed by blasting. See Plates 18 and 19. The rubble will be placed downstream of the draft tube exits and the rest will be hauled away by barge operating from the downstream cofferdam.

After removal of the powerhouse, permanent reinforced concrete stoplogs will be placed to close off the draft tubes and the turbine cavities will be filled with mass concrete. The area behind the powerhouse will be filled with rock and concrete rubble from spillway demolition. See Plate 19. Six weirs will be constructed at the location of Unit 10 to slow the flow and allow upstream fish passage through the breach. See Plate 18.

As soon as construction starts twelve special closure gates will be fabricated and installed in the gate wells of Units 1 through 4. During Stage 1 construction the turbine, generator, and other machinery on the powerhouse floor of Units one through four will be removed. Then the roof and west wall of the powerhouse and the second stage concrete around Units one through four would be removed. See Plate 18. During this time the South Shore Fish Ladder will remain in operation. Then the SSFL will be taken out of operation, and the fish collection channel and auxiliary water conduit will be removed. Since the turbines will be shut down at the beginning of construction, there will be no need for operation of the JBS.

All Stage 1 work will be accomplished while the upstream water surface is maintained at existing operating levels. This will allow operation of both north and south shore fish ladders except as noted above. The fish units should be operational during the early part of Stage 1 work until the SSFL is taken out of service. The navigation lock will also be operational during Stage 1. The upstream water surface will be controlled by passing flow through Spillway Bays 1 through 7.

This scheme can allow a maximum of about 595,000 cfs to pass the project. This assumes a water surface of 265 and fully open gates at Spillway Bays 1 through 7. Six spillways are required to pass the 10-year flood of 515,000 cfs.

After the dam has been removed the cellular sheetpiles for Stage 2 cofferdaming will be built inside the Stage 1 cofferdam as the last part of Stage 1 construction.

<u>Reservoir Drawdown</u>

Drawing down the reservoir would be accomplished by use of the spillway gates and modified powerhouse intakes. The drawdown to natural river will be done over about 100 days allowing a maximum of 1 foot per day drawdown. The drawdown should be accomplished from late November through February since there will be no upstream passage available. First, the reservoir will be drawn down to spillway crest using the available Spillway Bays 1 through 7. Below spillway crest the drawdown will be accomplished by opening the newly installed closure gates in Units 1 through 4. With these four 4 units open, the powerhouse flow capacity would be about 250,000 cfs for a head differential of 15 feet. This capacity exceeds river flow about 90 percent of the time during February. See Figure A-11.

To remain within the in-water work window, the cofferdam will be removed while the reservoir is being drawn down. First, water will be allowed into the dewatered area equalizing the head with the downstream water level. The H-pile cofferdam will be removed from top down following the reservoir water level during drawdown. The downstream sheetpile cellular cofferdam will also be removed. The upstream cofferdams will be removed from south to north to allow the river to flow through the breach as soon as possible. Some of the cells in the downstream cofferdam would be retained as shown on Plate 18.

<u>Stage 2</u>

During the second stage of construction the remainder of the spillway will be removed. The entire flow of the river will be passing through the breach created during Stage 1. It is planned that the Stage 2 work will be accomplished as follows.

The first portion of the Stage 2 cofferdam will be built inside the Stage 1 cofferdam. The rest of the cofferdam cannot be built while the Stage 1 cofferdam is being removed and the reservoir is being drawn down. Therefore, the Stage 2 cofferdam will be completed during the in-water work period the following year.

The gates and machinery could be removed during the year prior to completion of the cofferdam. After dewatering, the spillway would be removed by blasting and transporting the rubble by barge.

During Stage 2 work a cofferdam would be built in the upstream navigation lock channel. The sill would be removed and the new lock gate would be installed. The emptying valve would be installed in the downstream lock gate after dewatering utilizing the downstream floating bulkhead. Work on the navigation lock would be accomplished during the first year after reservoir drawdown.

The Stage 2 cofferdam would then be removed completing the project.

During drawdown the upstream and downstream flow control berms would be started. They could be built from the shore out with help from barges, if required. The beams would be built in three in-water work periods.

5.7.4 Fish Passage

Fish passage will be provided during construction for all months except December, January, and February, which constitute the in-water work period. The 100 days of drawdown require that fish passage be curtailed starting about November 20 in order to complete drawdown by the end of February. This would be achieved by operating the project at the present headwater and tailwater levels during Stage 1 construction. Therefore, the NSFL would be able to operate during construction. The SSFL would operate for the first part of Stage 1 construction only. The difficulty in implementing such a scheme is the large size of the cofferdams required. The cofferdams are described in subsection 5.7.3, Reservoir Drawdown.

Once the reservoir is drawn down past elevation 257 feet, the existing fish ladders will no longer work, and the ladders will need to be dewatered and stranded fish salvaged. The remainder of the drawdown period will occur within the in-water window and no upstream passage will be needed or provided. Cofferdam removal should begin on the south side and proceed to the north to provide the maximum amount of spillway for upstream passage through the new breach.

After drawdown, during Stage 2, fish will pass through the newly constructed breach. The average velocity will exceed 10 fps for flows above about 450,000 cfs. For flows at 515,000 cfs the velocities through the opening will reach 11 fps, and the drop across the project will be over 1 foot. During higher flows, weirs built into the southern portion of the breach will slow the water velocities and provide a means of upstream passage. See Plates 17 and 18. There will be six weirs spaced 30 feet apart. The weirs will have orifices to pass fish swimming along the bottom. The weirs will vary in height from 20 feet near the south abutment to three feet high at their northern end.

Downstream passage during the Stage 1 construction will be through the north spillway and in the existing JBS. During Stage 2 construction, downstream passage is provided through the new breach and through the powerhouse. Juvenile passage through the powerhouse should have little effect on fish since the units will either be full open or have blades removed.

5.7.5 Navigation Lock

The navigation lock would be modified to provide a channel for river traffic after drawdown is complete. During high flows velocities as high as 10 fps would be flowing in the river through the dam breach, therefore, the navigation lock would still be required.

Since the water level difference across the lock would be small, the lock fill and drain system cannot be used. The lock gates will be fitted with emptying and filling gates.

Prior to construction work on the navigation lock the upstream lock gate would be fabricated. It would be made with a filling valve built into the lower five feet of the lock.

The upstream floating guidewall would be removed and refurbished. The upstream pool would be drawn down allowing placement of a single cell cofferdam across the upstream navigation channel. The sill would be removed and the new lock gate installed. Downstream the floating bulkhead would be installed, and the lock dewatered. The lock-emptying valve would be installed in the bottom of the existing gate. The emptying and filling valve operators would also be installed at this time.

River navigation would be interrupted at the start of drawdown. Navigation would be possible about 100 days later after drawdown is complete and the cofferdam is removed. Navigation during Stage 2 construction would take place through the breach in the dam created during Stage 1. Velocities through the breach are less than five fps for flows less than 180,000 cfs. This means that navigation will be possible about 60 percent of the time during Stage 2 construction. Navigation against seven fps velocities would be possible about 87 percent of the time. Releasing flow through Units 1 through 4 could increase the time when navigation is possible.

The present navigation channel upstream of the lock has a bottom elevation of 147. During Stage 2 construction, the channel upstream of the lock would be dredged to elevation 140. The dredged material would be moved offsite and deposited above the water line.

5.7.6 Dam Removal

Removal of the dam could be performed in two stages. The first stage involves removing Units 10 through 20, the non-overflow section, and Spillway Bays 14 through 20.

During the Stage 2 construction Spillway Bays 1 through 13 would be removed. Removal of the dam would be by blasting and hauling the rubble away by barge or placing it in the fill area behind the powerhouse. See Plates 17, 18, and 19. The barges would operate from the downstream cofferdam.

The units would be filled with mass concrete and rock prior to removing the Stage 1 cofferdam. See Plate 19.

5.8 Operating and Maintenance Considerations

Under Alternative 3 the spillway and part of the powerhouse would be removed, and the river would flow uncontrolled through the breach. The power plant and all fish facilities would cease to function. There would only be two issues related to operation, the navigation lock and security at the project site.

5.8.1 Navigation Lock

For flows less than 400,000 cfs river traffic would travel through the breach in the dam since the velocities would be below five feet per second. For flows above 400,000 cfs river traffic would use the navigation lock. The lock would operate against a relatively low head of about 0.2 feet at 400,000 cfs and 0.3 foot at 515,000 cfs. Instead of the using the normal filling and emptying system, valves located in the bottom of the lock gates would be used to fill and empty the locks. Maintenance would still be required for the lock gates and the emptying and filling valves.

5.8.2 Security

Flow control berms would extend from the north end of the remaining powerhouse structure to the shore on both the upstream and downstream sides of the structure. See Plate 17. The flow control berms would be permeable, and the area between the levees and shore would be filled with water. It is envisioned that fencing and other security facilities would be set up and maintained to keep the public out of the abandoned powerhouse and fish passage facilities on the north and south shores of the project.

SECTION 6 ALTERNATIVE 4 - DRAWDOWN TO NATURAL RIVER WITH FLOOD CONTROL

In this alternative, part of the dam will be removed to approximate natural river hydraulic patterns, and a gate structure would be added to regulate flow for flood control. This entails providing gates on a spillway with a crest near natural river bed elevation. These gates would be raised during normal operation, and the river would run uncontrolled. When the flow at the flood control point at Vancouver is reached, the gates would be lowered to achieve up to 500,000 acre-feet of flood control storage at the John Day Project. The set point for triggering flood control operations at John Day can be lower than the 10-year flood design flow of 515,000 cfs depending on downstream conditions and tributary flow. For the design of fish passage features it is assumed that flood storage operations can be triggered at discharges of 360,000 cfs (2-year) or higher.

Implementation of this alternative will involve building a spillway with gates, modifying or replacing the navigation lock, and providing new adult fish passage facilities. Modifications to other project features might also be required. The modification or replacement of project features are described below.

6.1 Hydraulic Computations

Alternative 4 includes modification of the John Day spillway to reflect natural river conditions while providing flood control. The entire spillway was modified in all of the options to resemble a broad crested weir structure. In addition, different sections of the powerhouse were removed and replaced with gate bays. The minimum amount of structural modifications required to obtain a maximum average velocity of 10 fps at a discharge of 515,000 cfs and a Dalles forebay elevation of 155.0 feet was determined by modeling a variety of options. Since the navigation lock would be rebuilt for this alternative, the barge traffic criteria was not a concern when sizing the opening. Backwater calculations were required to estimate the velocities and water surface elevations along the modified reach of river. The hydraulic characteristics of the natural river drawdown with flood control option were analyzed using a HEC-RAS backwater model. Attachment B contains a detailed description of the HEC-RAS model used for the hydraulic computations. The model extends from RM 212.510 to RM 217.829 with the dam located between stations 215.636 and 215.535.

The different alternatives modeled include the following:

Run 4-1	Modify Spillway and Replace Entire Powerhouse with a New Spillway, Crest Elevation 135.0 feet
Run 4-2	Modify Spillway and Replaced Powerhouse Units 10 through 20 with a New Spillway, Crest Elevation 135.0 feet
Run 4-3	Modify Spillway and Entire Powerhouse with a New Spillway, Crest Elevation 130.0 feet
Run 4-4	Modify Spillway and Replaced Powerhouse Units 10 through 20 with a New Spillway, Crest Elevation 130.0 feet
Run 4-5	Modify Spillway and Replaced Powerhouse Units 16 through 20 with a

New Spillway, Crest Elevation 135.0 feet

Run 4-6 Modify Spillway and Replaced Powerhouse Units 15 through 20 with a New Spillway, Crest Elevation 135.0 feet

The first four model runs provided a range of options from removing a portion of the powerhouse to the entire powerhouse. The last two alternatives were analyzed to determine a configuration that would provide an average velocity of about 10 fps through the revised spillway portion of the dam.

Run 4-6 was selected from the six runs because this run met the velocity criteria and barge traffic requirements and minimized the amount of structural removal. This run includes modifying the spillway and replacing Units 15 through 20 with a broad crested weir with a crest elevation of 135.0 feet. There are 29 spillway bays in this run all at a crest elevation of 135.0 feet. A riprap dike extends downstream at an expansion rate of 1:2.9 from the south side of the powerhouse until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore at a 1.5:1 contraction rate.

Table 6-1 provides the average velocities and water surface elevations at the downstream face of the dam.

Table 6-1
Run 4-6 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.74	2.35
80,000	160	160.41	1.92
515,000	155	167.27	9.74
515,000	160	169.15	9.21

The run 4-6 model was also run for several other discharges between 100,000 and 500,000 cfs. Figure 6-1 provides a plot of the water surface profile for various discharges (The Dalles forebay 155.0). The thalweg shown in Figure 5-1 represents the recently surveyed cross-sections from the HEC-RAS model developed by WEST Consultants for the Portland District.

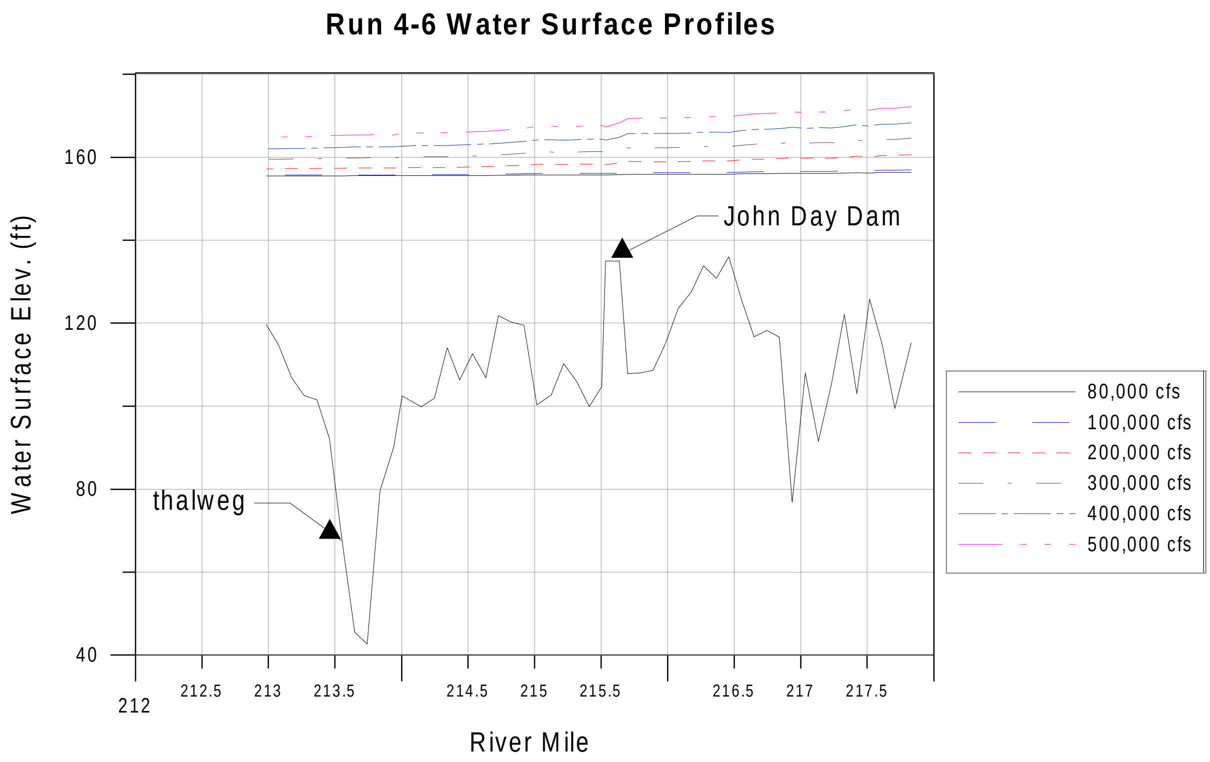


Figure 6-1
Run 4-6 Water Surface Profiles

6.2 Structure Removal

This alternative consists of the removal of Spillway Bays 8 through 20, which is 794 feet long, the 37 foot long non-overflow section, 996 feet of the powerhouse section (powerhouse Units 10 through 20), for a total length of 1,827 feet (CBL Station 28+51 to 46+78). See Plate 22. In the south end of powerhouse Bay 10, 41 feet would be filled in with structural concrete to provide the required opening length. The spillway concrete would be removed down to elevation 125 and new concrete will be placed up to elevation 135.0. The spillway stilling basin would be retained. The powerhouse section would be removed down to elevation 128. The voids below elevation 128 that are formed by the powerhouse intake would be filled in with concrete and built up to elevation 135 with structural concrete. See Plates 24 and 25. An energy dissipater would be constructed downstream of the new spillway on the downstream end of the powerhouse and tailrace fill. See Plates 24 and 25.

At the 30 percent level two options for structure removal included reusing the existing spillway or constructing a new spillway. Reusing the existing spillway was selected. However, during development of the concept structural issues arose that led to a reconsideration of the decision. The peirs in the spillway are battered where they join the slab. The batter inhibits the gate from seating. Stability issues rise if the batter were simply cut off so that rectangular gates could operate. Constructing a new spillway was selected.

6.3 Spillway

The new spillway will consist of 29 bays with 12-foot wide piers to support new spillway gates. Each bay would be 50 feet wide and would be equipped with triple leaf fixed wheel gates that could be fully or partially closed to provide the 500,000-acre feet of flood storage.

Construction of the new spillway will be done in two stages. During Stage 1 a portion of the new spillway (Bays 13 through 29) will be constructed. See Plate 24. Bays 20 through 29 will be built with a deck elevation of 281. New Spillway Bays 13 through 19 constructed in Stage 1 and Bays 1 through 12 constructed in Stage 2 will have a deck elevation of 245. See Plate 25.

To withstand the moments at the base of the piers a structural concrete base is required. At the existing spillway the concrete will be excavated to elevation 125 and new reinforced concrete placed up to elevation 135. See Plate 25. This slab will form the foundation for the spillway piers. In the area of the powerhouse, a reinforced concrete slab will be placed on the intake floor up to elevation 135. This will form the foundation for the piers located on the spillway.

Each of the new Spillway Bays 20 through 29 will be equipped with triple leaf fixed-wheel gates and a set of 50-feet high stoplogs with a temporary ogee section (see Plate 24). These stoplogs will be used as dewatering stoplogs for gate maintenance and replacement after the project is built. During Stage 2 construction these gates will regulate flows to maintain the forebay elevations between 260 and 265 feet msl. Each gate leaf will be furnished with a separate 50 horsepower motor-driven cable drum hoist located on the spillway deck so that they can be operated under unbalanced conditions to maintain the reservoir water level elevation between 260 and 265. In addition to the stoplogs shown, additional stoplogs will be provided to permit complete dewatering of any single spillway bay to service the fixed-wheel gates. A 150-ton gantry crane will be provided to permit installation and removal of the stoplogs and gate leafs.

During Stage 1, the gates, stoplogs and ogee section downstream of the stoplogs will be installed as shown on Plate 24. A new energy dissipater will be constructed in new Spillway Bays 14 through 29 which are located within the limits of the removed powerhouse. The gates will be in the closed position to maintain a reservoir elevation of 265. Flow releases will be made by raising the lower gate leaf, permitting water to discharge over the top of the stoplogs and the ogee section. After completion of the Stage 2 construction, the stoplogs and ogee section will be removed and the gates will operate in a similar fashion to the Stage 2 spillway gates shown on Plate 25.

The Stage 2 spillway will be constructed with a deck elevation of 245 in the space presently occupied by Spillway Bays 1 through 19. A 300-ton gantry crane will be installed on this portion of the spillway to operate the fixed-wheel gates. These gates will be operated only under balanced head during flood conditions up to a reservoir elevation of 223, so a cable drum hoist will not be required for the new Spillway Bays 1 through 19.

During normal operation of the project, the gates will be in the up position with all leaves fully raised. The gates will be lowered into position during a flood event to provide up to 500,000-acre feet of flood storage up to reservoir elevation 223. During normal operation, the three gates will be in the up position with the bottom of the gates above the maximum water surface elevation of 205 during the PMF. When closed for flood storage, the top of the gates will be at elevation 228, or 5 feet above the anticipated maximum water surface elevation for the required flood storage of 500,000-acre feet. Two 5-foot high stoplogs will have to be installed below the gates in Spillway Bays 20 through 29 to accomplish this.

Modifications to the spillway considered but not selected at the 30 percent level, included installation of a Bascule Gate.

6.4 Upstream Fish Passage

6.4.1 North Shore Fish Ladder Description

Upstream fish passage for this alternative, during non-storage, would be through the breach in the dam similar to the previous alternative. When it becomes necessary to begin storing water for flood control, and the gates are lowered into the water, upstream fish passage through the breech is impacted. During flood control operation a new fish ladder on the north shore would be used to provide upstream passage during flood control operations.

The new, variable length, vertical slot fish ladders with five sections would be constructed to provide upstream passage for forebay elevations between 176 feet and 222 feet. Five sections of 10 pools, 20 pools, 30 pools, 40 pools and 50 pools can be used depending on the forebay and tailwater elevations. Thus, upstream fish passage would be provided for all water levels during flood control operations. Details are shown on Plates 26 and 27.

A new fish ladder would be the same size and constructed starting at the same location as the NSFL described in Sections 3.1.1 and 4.1.1. The invert of the low level exit is at elevation 162 feet. The remaining four exits are located at 10-foot intervals up to elevation 202. Exit channels are seven feet wide by 24 feet high. Due to width restrictions between the lock and spillway, the alignment of the 40 pool and 50 pool ladder would switch back over the lower portions of the 10 pool and 20 pool ladders, respectively. The exit channel from the two upper ladders would be sloped to maintain a two fps transport velocity.

Auxiliary water for attraction flow is provided by the existing pumps. Modifications to the water supply conduit to relocate the bulkhead would be required.

A counting and viewing structure is not planned due to the infrequent use of the fish ladder.

Construction of the new fish ladder would be mostly below the current deck level of 185 feet. Walls in the lower part of the ladder would provide protection to this level. Elevated portions would be constructed similar to the existing ladder.

The SSFL will be abandoned and cut off from the river by the flow control berms. See Plate 22.

At the 30 percent level a narrow breach with roughness elements and a fish ladder were considered for alternative 4. However, the reliability of the roughness elements to pass fish is not known.

6.4.2 North Shore Fish Ladder Operation

When operation for flood storage is initiated the dam gates would be closed to raise the forebay elevation to a minimum level of 176 feet, to allow operation of the fish ladders. After reaching a forebay water level of 176 feet, forebay levels would then be varied depending on flood control needs. Depending on the river flow and tailwater from The Dalles Dam, gate closures in excess of that required for the flood control may be required to keep the fish ladders operating within criteria. Figure 6-2 shows the forebay-tailwater operating ranges for this ladder. Refer to Section 3.1.2 for a narrative description of how to read Figure 6-2.

Sequenced operation of spillway gates during flood control will need to be developed to prevent potential fallback of fish exiting the ladder. Reduced use of Spillway Gates 1 and 2 are expected, but will need to be balanced with fish ladder attraction.

Flow conditions, energy dissipation and drop between pools in the ladder will be as described in Section 3.1.2. Exit channels are seven feet wide and will provide a two fps transport velocity to the forebay. The transport velocity would stay the same for varying forebay levels.

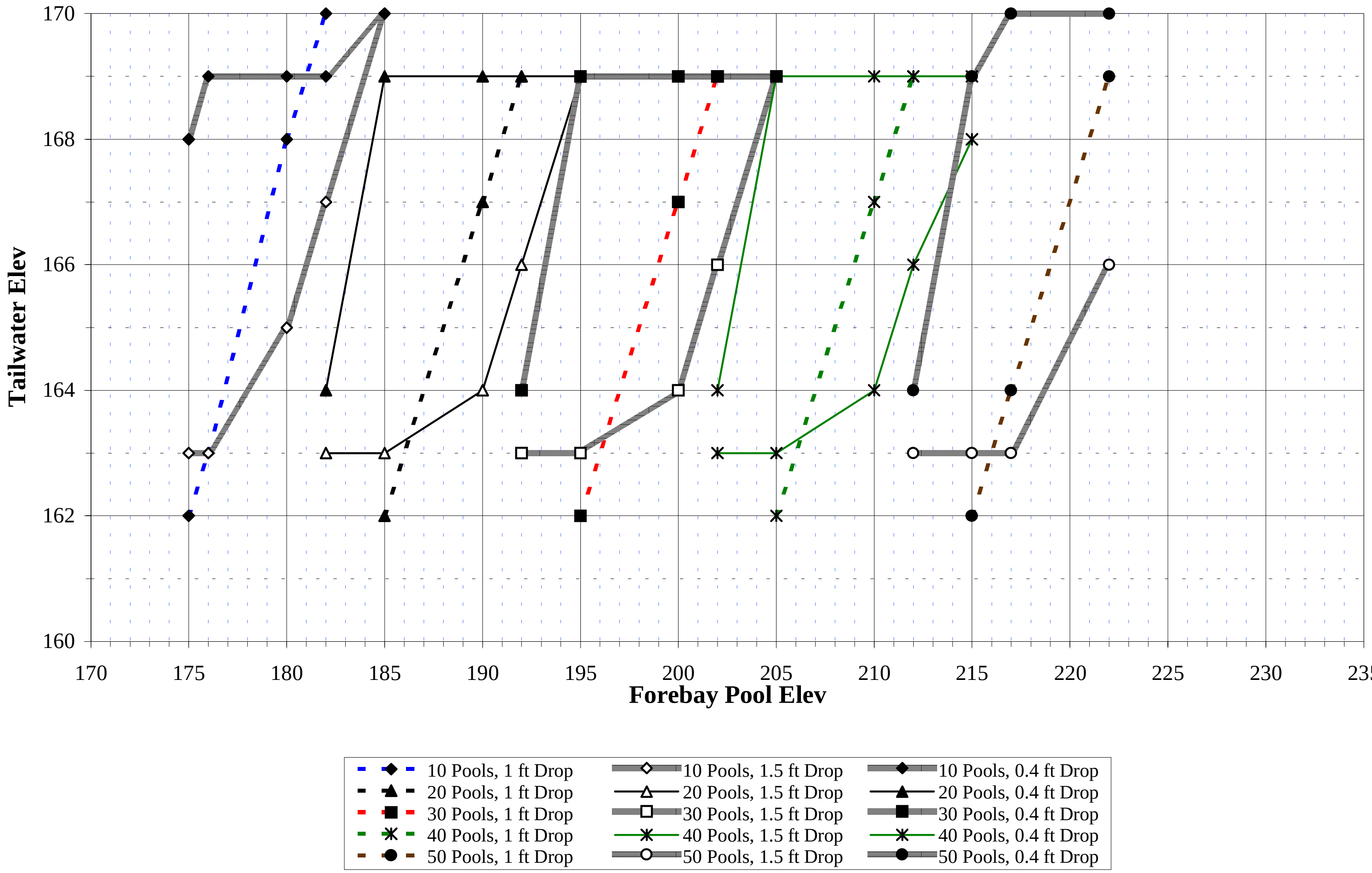
6.5 Downstream Fish Passage

Downstream passage is provided through the gated breach. For non-storage conditions, the gates should not affect flow or drop through the spill section. During storage conditions, the design and operation of the gates and spill sections are assumed to provide safe downstream passage and minimize dissolved gas concentrations. No JBS facilities are proposed due to the intermittent and short duration of the storage conditions.

6.6 Navigation Lock

It is envisioned that a new navigation lock would be built through the embankment north of the existing lock. The existing navigation lock would not be modified because to do so would stop navigation for more than one year because of the extensive cofferdamming requirements. Building a new lock would curtail navigation for less than a month while the downstream approach channel to the new lock is connected to the existing channel. The new lock would be similar to the existing one, however the upstream sill would be at elevation 140. The bottom of the lock would be at elevation 138. A new 105-foot high miter gate would be installed on the upstream sill. See Plate 28. A shallow screened intake would be constructed for lock filling flows. A new channel extending from the lock to the existing upstream channel would be dredged to an elevation of 140. Stoplog slots and stoplogs will be provided for dewatering the upstream lock gate. The existing downstream floating bulkhead will be used for dewatering the downstream gate.

Figure 6-2
Alternative 4 - Fish Ladder Operating Ranges



All river traffic will travel through the navigation lock under this alternative because the gate bay openings of 50 feet are too narrow for navigation. For river flows of about 130,000 cfs and below the velocity through the breach would be 3 fps or less, and traffic would travel through the lock without operation of the lock gates. If it is assumed that velocities through the open lock are the same as through the breach, navigation through the open lock without operating the gates can only occur at flows below 130,000 cfs. This occurs about 30 percent of the time. For flows over 130,000 cfs river traffic will transit the project through the operating lock. The head across the locks is expected to be about 1.5 feet at a flow of 515,000 cfs with no flood control operation in effect. During flood control operations the lock could operate up to the design river flow of 8000,000 cfs. At the design flow of 800,000 cfs under flood control, the head across the project would be about 50 feet.

Providing navigation through the breach in the dam was considered at the 30 percent design level. It was then discovered that it was more economical to limit the width of the breach to that required for fish passage rather than for that required for barge traffic so lock modification strategies were pursued. Widening the spillway bays for navigation would be very expensive and would not provide navigation during flood control and a lock would be required. However, developing the details of the construction sequence revealed that navigation would be curtailed for an unacceptable length of time during construction. Therefore it was decided to proceed with navigation lock replacement.

6.7 Project Sequencing

This subsection first describes the constraints on construction activities. This is followed by a description of the project schedule from completion of the Phase I Report through completion of construction and full implementation of the drawdown.

6.7.1 Project Constraints

Land access to the project for construction is limited. Much of the access to the project will have to be from the water. It is probable that much of the construction will be staged from barges and that transportation of materials to and from the site will be by barge.

In addition to site and access constraints, the construction schedule is affected by three operational items, which involve fish passage requirements. These restrictions are in-water work periods, fish transport spill, and the requirement that fish passage be provided at all times except during the three-month in-water work period.

Any construction work in the water is allowed only between December 1 and March 1. Construction behind cofferdams is not in-water work and, therefore, is not subject to this requirement.

In the spring and early summer if water is available, more flow is released into the river from upstream storage and is sent over the spillways. This restricts work on and near the spillways. The spill period typically extends from April 1 to July 31.

Another constraint on construction sequencing is to provide for navigation through the project at all times except during the drawdown period.

6.7.2 Project Schedule

In developing the project schedule it was assumed that the steps to implement the project would be the Phase II feasibility study and EIS, design memorandum, plans and specifications, and construction. For the purposes of this reconnaissance study all work prior to the start of construction is assumed to be the same for all alternatives. The Feasibility Study and EIS are assumed to start in October 2000 and last for five years. Preparing the design memorandum and plans and specifications is assumed to start in October of 2005 and take approximately three and one half years to complete. The construction bid process will start in April 2009.

The project schedule is shown on Figure 6-3.

6.7.3 Dewatering and Reservoir Drawdown

Requirements for performing the reservoir drawdown are that fish passage occurs during all stages of construction. Fish passage must be by fish ladder and JBS. Trap and haul schemes are not acceptable due to the presence of listed fish and the large numbers of fish that pass the project site.

Removal of the dam and powerhouse units and constitution of the new spillway will be accomplished in two stages. The reservoir drawdown will occur after both are complete. The spillway gates constructed during Stage 1 will be used to regulate the upstream water level between 260 and 265 during Stage 2 construction. During a third stage of construction after drawdown is complete, modifications to the NSFL will be completed. A more detailed explanation of the construction sequencing is provided below.

Stage 1

In this stage the Spillway Bay 20, the non-overflow section, and Units 10 through 20 would be removed. It is envisioned that the construction will take place as follows.

The upstream cofferdam will be up to 150 high. Therefore, a cellular sheetpile cofferdam is not feasible, and a dam type of cofferdam will be required. For this study an H-pile cofferdam is selected. See Plate 23. The wall key would be excavated into the riverbed from a barge. This will be accomplished with a clamshell if the riverbed is granular material. If the riverbed material is bedrock, the rock will be drilled and shot, and the shot rock will then be excavated with a clamshell. After the key is constructed, the initial fill will be placed with a clamshell from a barge. A cheaper method would be to simply dump the material from a barge, however turbidity could be a problem. Once the initial fill has been placed, the ARBED shapes can be driven with a vibratory hammer. Temporary support piles would be driven to hold piles in position and provide support during installation of the shot rock. With the ARBED wall in place, the shot rock will be placed with a clamshell in 10-foot lifts up to a point of approximately 80 feet above the original riverbed, and each lift would be vibro compacted to stabilize the material. Beginning at the 80-foot elevation mark, earth reinforcement will be placed in the fill material

Act ID	Description	Orig Dur	Start	Finish	Predecessors
Water Work Windows					
0010	In Water Work Window (09-10)	90d	01DEC09 *	01MAR10	
0020	In Water Work Window (10-11)	90d	02DEC10	02MAR11	0010
0030	In Water Work Window (11-12)	90d	02DEC11	01MAR12	0020
0040	In Water Work Window (12-13)	90d	01DEC12	01MAR13	0030
0050	In Water Work Window (13-14)	90d	01DEC13	01MAR14	0040
0060	In Water Work Window (14-15)	90d	01DEC14	01MAR15	0050
Spill Periods					
0110	Spill Period (09)	121d	01APR09 *	31JUL09	
0120	Spill Period (10)	121d	01APR10	31JUL10	0110
0130	Spill Period (11)	121d	01APR11	31JUL11	0120
0140	Spill Period (12)	121d	01APR12	31JUL12	0130
0150	Spill Period (13)	121d	01APR13	31JUL13	0140
0160	Spill Period (14)	121d	01APR14	31JUL14	0150
Bid and Award					
4100	Advertise for Bid	0	28MAY09 *		
4200	Bid And Award	100d	28MAY09	05SEP09	4100
4300	Award Construction Contract	0	07SEP09 *		
Stage 1					
5000	Mobilize	83d	07SEP09	29NOV09	4300
5100	Construct Spwy Side of Stage 1	90d	29NOV09	27FEB10	5000
5200	Construct Rest of Stage 1 Cofferdam	90d	01DEC10 *	01MAR11	
5300	Remove Gate and PH Machinery	250d	01MAR11	06NOV11	5200
5400	Remove Spillway 20, PH 10-20	420d	01MAR11	24APR12	5200
5500	Construct New Spillways	600d	01OCT11 *	23MAY13	
5600	Install Stoplogs and Gates	192d	23MAY13	01DEC13	5500
5700	Build Stage 2 Cofferdam Inside	120d	01AUG13 *	29NOV13	
5800	Remove Stage 1 Cofferdam	90d	01DEC13	01MAR14	5600
Stage 2					
7000	Construct Stage 2 Cofferdam	90d	01DEC13 *	01MAR14	
7100	Remove Gate Machinery	150d	01MAR14	29JUL14	7000
7200	Complete Stage 2 Cofferdam	90d	01DEC14 *	01MAR15	
7300	Remove Spill Bays 8-19	360d	01MAR15	24FEB16	7200
7400	Construct Spillway	600d	01OCT15 *	23MAY17	
7500	Install Gates	192d	23MAY17	01DEC17	7400
7600	Remove Cofferdam	90d	01DEC17	01MAR18	7500
Navigation Lock					
8000	Fabricate New Lock Gates	600d	01DEC09 *	24JUL11	
8100	Start Upstream Cofferdam	90d	01DEC09 *	01MAR10	
8200	Complete Upstream Cofferdam	90d	01DEC10 *	01MAR11	
8300	Construct Navigation Lock	900d	01MAR11	17AUG13	8200
8400	Construct Lock Intake Structure	900d	01MAR11	17AUG13	8200
8500	Install Lock Gates	150d	17AUG13	14JAN14	8400
8600	Dredge Upstream Navigation Channel	90d	01DEC14 *	01MAR15	
8700	Dredge Upstream Navigation Channel	90d	01DEC15 *	29FEB16	
8800	Remove Cofferdam & DS Berm	90d	01DEC17 *	01MAR18	

Figure 6-3
Schedule
Alternative 4 - Drawdown to Natural River with Flood Control

Act ID	Description	Orig Dur	Start	Finish	Predecessors
Reservoir Drawdown					
6000	Drawdown Reservoir	100d	20NOV17 *	28FEB18	
Stage 3					
9000	Construct Stage 3 Cofferdam	90d	01DEC17 *	01MAR18	
9100	Demolish Northshore Ladder	150d	01MAR18	29JUL18	9000
9200	Rebuild Northshore Ladder	490d	29JUL18	01DEC19	9100
9300	Remove Temporary Ogee Crests	270d	01MAR18	26NOV18	9000
9400	Remove Stage 3 Cofferdam	90d	01DEC19	29FEB20	9200
Flow Control Berm					
10000	Place Upstream and Downstream Berm	90d	01DEC17 *	01MAR18	
10100	Place Upstream and Downstream Berm	90d	01DEC18 *	01MAR19	
10200	Place Upstream and Downstream Berm	90d	01DEC19 *	29FEB20	

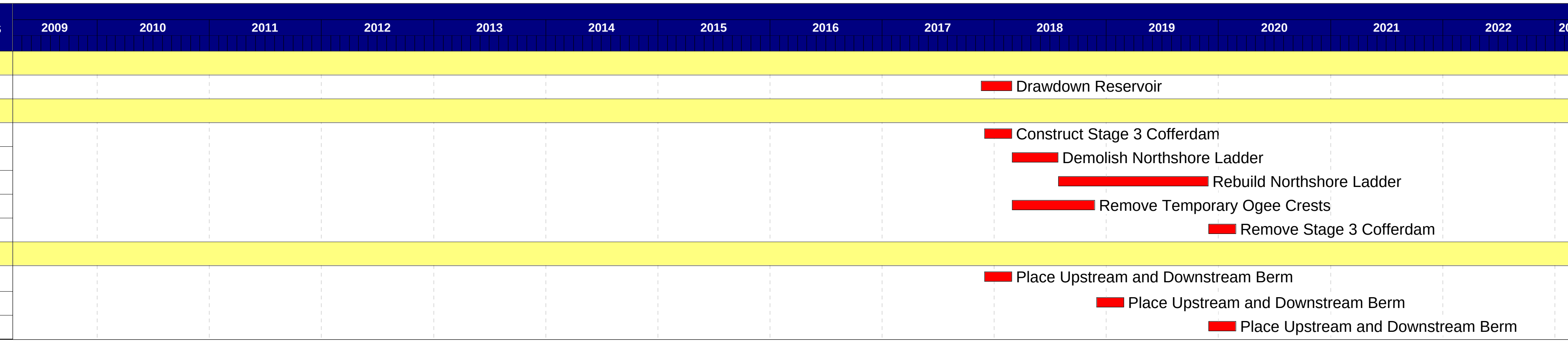


Figure 6-3
Schedule
Alternative 4 - Drawdown to Natural River with Flood Control

every 10 feet up to the top. This could be a steel mat grid or other reinforcement. Then subsequent 10-foot lifts and vibro compaction will continue until full height is reached. At the final elevation of 268 an access road and amenities as required can be added. Outer layers of the shot rock should include materials which will not wash away with wave action or reservoir drawdown.

The downstream cofferdam will be constructed of sheetpile cells west of the stilling basin and the adverse slope downstream of the powerhouse. The cofferdam will be anchored to the stilling basin slab to withstand the spillway discharge turbulance during Stage 1 construction. Permanent stoplogs will be placed to close off the draft tubes. Additional fill will be placed behind the draft tubes of Units 4 through 9 to support the cellular sheetpile cofferdam in the deeper area near the powerhouse.

Since the cofferdams are so large, they will have to be built during two in-water work periods. In the first year the cofferdams upstream and downstream of Spillway Bays 14 through 20 and powerhouse Units 17 through 20 will be constructed. In front of powerhouse, Units 5 through 16, the wall key will be excavated and the initial fill for supporting the H piles will be placed. The initial fill will be far enough upstream of the powerhouse so that it will not be eroded by turbine flows. During the in-water work period the following year the rest of the cofferdam will be constructed as described above.

Spillway Bay 20, the non-overflow section, and Units 10 through 20 will be removed by blasting. The rubble will be hauled away by barges loaded from the downstream side of the cofferdamed area.

After removal of the powerhouse, permanent reinforced concrete stoplogs will be placed to close off the draft tubes and the turbine cavities will be filled with mass concrete. The area behind the powerhouse will be filled with rock. An energy dissipater will be built on the rock and the downstream side of the powerhouse as shown on Plates 24 and 25.

All this work will be accomplished while the upstream water surface is maintained at existing operating levels. This will allow operation of both north and south shore fish ladders and the existing JBS. Units 1 through 3 in the powerhouse should be operational for much of the construction period. The navigation lock will also be operational during Stage 1 work. The upstream water surface will be controlled by passing flow through powerhouse Units 1 through 3 and Spillway Bays 1 through 12.

This scheme can allow a maximum of about 1,100,000 cfs past the project. This assumes a forebay water surface of 265, fully open gates at Spillway Bays 1 through 12, and operation of powerhouse Units 1 through 3.

After the dam has been removed the new spillway and its piers would be constructed from CBL Station 28+51 to about 39+34. This will allow construction of 17 of the 29 new spillways. The existing powerhouse will be demolished, the cavity will be filled with concrete, and a concrete cap will be placed up to elevation 135. In the existing spillway, the concrete will be removed to elevation 125 and structural concrete will be placed up to elevation 135. In new Spillway Bays

20 through 29, stoplogs 50 feet high will be placed in the bottom of the spillway, and three gate leaves will be installed above them. Temporary ogees will be built behind the stoplogs in these southern 10 spillway bays. See Plate 24. The spillway piers will be built up to elevation 281 at new Bays 20 through 29 and up to elevation 245 at Bays 13 through 19.

At the beginning of Stage 1 a cofferdam will be constructed east of the embankment and north of the navigation channel. Inside this cofferdam the upstream lock entrance and lock water supply intake will be built. The rest of the lock and downstream approach channel would be constructed north of the existing lock. No downstream cofferdam would be required since it will be built behind the present shoreline. See Plate 23.

The southern sheetpile cells for the Stage 2 cofferdams will be built inside the Stage 1 cofferdam prior to removing it. The upstream H-pile cofferdam will also be started inside the Stage 1 cofferdam. With the new gates closed the cofferdams upstream and downstream will be removed. At the same time Stage 2 cofferdam construction will start.

Stage 2

During the second stage of construction the forebay will be maintained at existing levels and existing Spillway Bays 8 through 19 will be removed. During Stage 2 construction the entire flow of the river will be passed through powerhouse Units 1 through 3 and the southern ten new spillway bays (Bays 20 through 29). Flow through the new spill bays would pass over the temporary ogees providing a safer passage for downstream migrants. See Plate 24. It is planned that the Stage 2 work will be accomplished as follows.

The cofferdam construction would be started in the same in-water work period that the Stage 1 cofferdams are being removed. Due to the size of the cofferdams required the second stage cofferdams would have to be completed during the in-water work period in the next year. Prior to completing the Stage 2 cofferdam the gate lifting machinery will be removed since operation of the existing spillway is no longer required. In the second year the cofferdam would be tied into the dam at existing Spillway Bays 1 through 7. Temporary NSFL exit and entrance channels will be constructed during the in-water work period. A temporary concrete wall between the existing entrance channel and Spillway Bay No. 1 will be constructed and tied into the cellular cofferdam downstream.

The existing gates would have to be removed after completion of the cofferdam. After dewatering the spillway would be removed by blasting and transporting the rubble by barges which will operate from the downstream cofferdam. The 12 remaining spillway bays and their piers would be constructed and the gates installed. See Plates 23 and 25.

Construction of the new navigation lock would be completed during Stage 2. The lock gates would be installed, and the navigation channel upstream would be dredged. The upstream cofferdam and the shoreline berm downstream would be removed during reservoir drawdown.

During and after drawdown the upstream and downstream flow control berms would be built. They could be built from the shore out with help from barges, if required. They would be built during three in-water work periods.

Construction of the NSFL can begin during the Stage 2 on work outside the existing ladder. This work would consist of portions of the new ladder west of the existing ladder, boring of the new exit channels and construction of some of the exit features.

After construction is complete inside the cofferdam the reservoir would be drawn down as far as possible using the 10 operating spillway bays. The drawdown will be done over about 100 days allowing a maximum of 1 foot per day drawdown. As the pool is lowered the cofferdams will be removed. The temporary wall and the right bank portion of the cofferdam at the NSFL will remain for construction of the new fish ladder. See Plate 23.

Stage 3

Stage 3 of construction involves removing the temporary ogee crests at new Spillway Bays 20 through 29 after the drawdown is complete. The upstream stoplog weirs would provide the upstream cofferdam, which would protect the work for flows up to about 700,000 cfs. A removable bulkhead similar to those used for construction of the spillway flow deflectors will be required to dewater the downstream side. The temporary ogee crests would be removed down to elevation 132, and a concrete cap would be placed to bring the final spillway elevation up to elevation 135. After the temporary ogee crests have been removed the project would be complete.

Construction of the NSFL can be completed during the year and in time to remove the remaining cofferdams during the following in-water work period.

6.7.4 Fish Passage

Upstream fish passage during both Stage 1 and Stage 2 of construction will take place through the fish ladders. An extension of the fish ladder entrance channel will be required through the downstream cofferdam. The extension should be designed with adjustable gates and bulkheads of sufficient size for dewatering of the lower parts of the ladder during Stage 3 construction. Maximum attraction flow through the north ladder will be needed to increase the attraction out of the backwater created by the cofferdam. Passage through the SSFL would be cut off during Stage 3 construction. The auxiliary water system for the ladder will require no modification since it is obtained by pumping from the tailrace downstream of the cofferdam.

During the construction of the NSFL, during Stage 3, upstream fish passage will be provided through the breach in the dam. Therefore, for the construction period (one year) flood control operations at John Day Dam will not be possible without impacting upstream passage.

Downstream passage will be through the existing juvenile bypass during both stages of construction. During Stage 2 work downstream migrants will also pass through the open spillway gates. The fish will pass over the 50-foot high weir and under the gate leaf. A temporary ogee will be placed behind the weir to provide a better trajectory for the flow entering the tailrace.

6.7.5 Navigation Lock

A new navigation lock will be constructed in the embankment north of the existing lock. A cellular sheetpile cofferdam will be established at its eastern end for construction of the navigation lock and intake structure. Construction of the western part of the lock and approach channel will be protected by leaving the shoreline in place. At the end of Stage 2 and during drawdown in the in-water work period, the cofferdam and shoreline berm will be removed placing the lock in operation.

The upstream floating guidewall would be removed prior to construction of the cofferdam. New guidewalls will be built.

The navigation channel will be dredged to obtain a channel bottom at elevation 140. Dredge spoils will be removed to the shore for disposal. New channel markers will also be installed.

The navigation lock would be out of commission for three months during drawdown.

6.7.6 Dam Removal and Spillway Construction

During the first year when the Stage 1 cofferdam is under construction the generators, turbines and associated equipment for the northernmost units will be removed from the powerhouse. After the Stage 1 cofferdams are complete Units 10 through 20 would be removed. The non-overflow section between the powerhouse and spillway will be removed down to elevation 125. Concrete fill would be placed in the turbine/draft tube cavities, and temporary ogees would be placed up to elevation 185. See the powerhouse section on Plate 24. The ogees would reduce potential downstream migrant injury during Stage 2 construction. At the same time the 12-foot wide spillway piers would be built up to elevation 281 at new Spillway Bays 20 through 29. A spillway deck would be built on top of the piers at elevation 281 and connected to the existing powerhouse. Thus, the existing powerhouse crane would be available to assist in operating the southern part of the new spillway during Stage 2 construction.

The remaining six new spillway bays to be constructed in Stage 1 (new Spillway Bays 14 through 19) would be behind the Stage 2 cofferdam and have piers built up to elevation 245. These bays will not pass flow during Stage 2 and will not require full height piers to elevation 281 or temporary ogee crests. Special stoplogs will be inserted in these bays to provide an interface and support for the end of the Stage 2 cofferdam.

After reservoir drawdown is complete the temporary ogees will be removed from behind the upstream stoplog weirs. The stoplog weirs will provide an upstream cofferdam. A removable bulkhead similar to those used for construction of the spillway deflectors will be required to dewater the downstream side. The ten temporary ogees would be removed sequentially across the dam.

6.8 Operation and Maintenance Considerations

This subsection describes the operation and maintenance requirements only for those features impacted by the drawdown. These requirements are described in a general manner consistent with a reconnaissance level study. The operation and maintenance requirements for Alternative 4 are a combination of the requirements for Alternatives 2 and 3.

The operation and maintenance requirements are described below for each feature that would be impacted by the drawdown.

6.8.1 North Shore Fish Ladder

The NSFL is designed to operate over a wide range of forebay elevations, from natural river conditions through full flood control operations at 515,000 cfs. When flood control is not in effect fish would pass upstream through the spillway. During the approximately 3 percent of the time when flood control operations are occurring, the fish ladder should be in operation. Depending on the level of the forebay pool one of the five exits would be in operation. During this time the gates to all other exits would be closed. As the water level increases or decreases exit gates would be opened and closed to bring the proper exit into operation to effectively pass fish during periods of flood control. The auxiliary water system would operate the same as it does now. However, only the lower ladder would require auxiliary water to supply the entrances. Sequencing between ladder configurations will be similar to that described for Alternative 1 in Section 3.8.1.

Optimal operation of the fish ladders described for this alternative requires a philosophical change to the current operation of both The Dalles Dam and John Day Dam. Power production and/or spill at both projects will have to be coordinated to maintain the fish ladders at or near the operating conditions for a one foot drop between pools (dashed lines on Figure 6-2). Items for consideration during the next phases of this project include:

- Extension of the fish ladder entrance channel to add automatic weirs that will adjust to eliminate the need for controlling the pool at The Dalles.
- Use of auxiliary water system to add and withdrawal water from the ladder pools to extend the forebay range of each ladder.
- Use of adjustable slot gates to extend the forebay range of each ladder.

6.8.2 South Shore Fish Ladder

Under this alternative the SSFL will be abandoned.

6.8.3 Downstream Passage

Downstream passage would be over the spillway during normal operation. During flood control operations, downstream migrants would pass under the spillway gates.

6.8.4 Hydroturbine Operation

As discussed in subsection 6.7, upstream water surface will be maintained during the first two stages of construction. Units 1 through 3 are expected to be operational and during Stages 1 and

2. Drawdown of the reservoir would be accomplished using 11 new spillway bays built in Stage 1. At the beginning of Stage 1 Units 4 through 16 will be abandoned. Following drawdown, the rest of the powerhouse will be abandoned. Restoration of existing synchronous condensing operation will be required, as described in subsection 5.6.

6.8.5 Navigation Lock

The navigation lock would be the same as the existing structure except that the upstream gate would be 105-foot high miter gate rather than a smaller lift gate. There would be three modes of operation as explained below.

1. Under 130,000 cfs - At these flows water velocities through the open locks would be at or below 3 fps, and traffic would travel through the open locks using it as a channel.
2. Over 130,000 cfs without flood control - At higher flows under normal operation, the locks would be used. There would be a low head across the structure, from 0.5 to 1.5 feet. The lock would operate normally using the emptying and filling system.
3. Flood Control Operations - When flood control operations are in effect the normal lock facilities would be used. However, the water level upstream should be above about 205 to provide submergence on the lock filling intakes.

6.8.6 Spillway

There would be 29 spillway bays each equipped with triple leaf roller gates. During normal operations the gates would not be used and would be dogged at the spillway deck. See Plates 24 and 25. The gates would be lowered only during flood control operations, which would occur once every two to five years on average. A gantry crane would be utilized to accomplish this in Spillway Bays 1 through 19 to lower the gates. This could take approximately 24 hours. To end flood control operations the gantry crane would raise the gates, and they would be dogged off at the deck level. For Spillway Bays 20 through 29, 50 horsepower cable drum hoists would be used to raise and lower each gate.

Maintenance on the gantry cranes, gates, and spillway deck would be required. To maintain the gate the gantry crane would lift the gate leaf and place it on deck for inspection and painting.

SECTION 7 DRAWDOWN EFFECTS AT THE MCNARY PROJECT

7.1 Upstream Passage

7.1.1 Effects on Upstream Passage

McNary Dam fish passage facilities were in operation prior to the completion of the John Day Dam and the filling of the John Day Pool. The original upstream passage facilities included: the Washington shore fish ladder, a pressure fish lock and the Oregon shore fish ladder. The Oregon shore ladder collects both Oregon shore fish as well as fish collected at entryways along the downstream face of the powerhouse. There were no pre-John Day downstream passage facilities.

Fish ladders operated over a pre-John Day tailwater range from minimum tailwater elevation 248 at a river discharge of 30,000 cfs, to a tailwater elevation of 278.2 at a river discharge of 700,000 cfs. With the construction of the John Day project, modifications to the McNary facilities were implemented to accommodate an increase in tailwater elevations. The John Day Project raised the minimum tailwater to elevation 262 and the maximum tailwater to elevation 280.5. These modifications are described in John Day DM No. 30.

For all four drawdown alternatives under consideration in this study, the tailwater is assumed to behave as it did before the John Day Project was built up to flows of about 700,000 cfs for all Alternatives. See Figure A-9 in Attachment A. As noted in Section 2.3.3, the design flows at the McNary Dam are 80,000 cfs minimum and 515,000 cfs maximum. The tailwaters associated with these flows are at elevation 251 and elevation 270 respectively.

7.1.2 Washington Shore Fish Ladder

A return to the lower tailwater range of operation would require modifications to the fish ladder entrances and lower portions of the fish ladder. Modifications to the fish entrances 1, 2 and 3 include the reestablishment of the entry sills at elevation 242 from their present elevation at 254. The telescoping weir gates and slots would be rehabilitated in order to restore operating capability at the low tailwater. Similar work is required for the stoplog slots. This would require removal of the concrete that fills the slots from elevation 245.33 down to elevation 235. Additionally, the operation and effectiveness of Entrance 4 must be evaluated. (Figure 7-1)

Modifications to the lower portions of the fish ladder involve weir extensions that were added to raise the effective floor elevation to accommodate the increased tailwater. The weir extensions at weirs 248, 252 and 256 would be removed. The telescoping gates on these weirs were not used when the ladder was previously modified. It is assumed that the gates would not be replaced.

Auxiliary water is supplied by gravity from a new hydropower plant. The auxiliary water diffuser system should be evaluated and modified. Flow control to the water diffuser channels associated with the reduced tailwater should be evaluated and reestablished. The attraction water system on the Washington Shore Fish Ladder should provide 1,000 cfs to 2460 cfs attraction flow as originally designed.

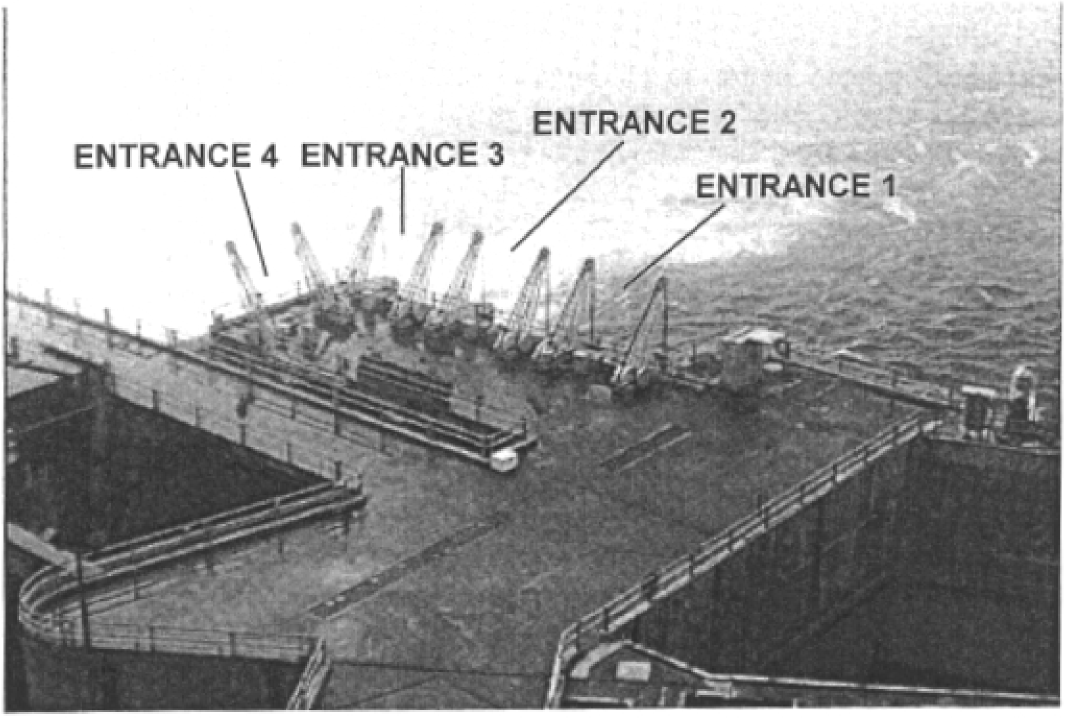


Figure 7-1
McNary Dam Washington Shore Fish Ladder Entrance

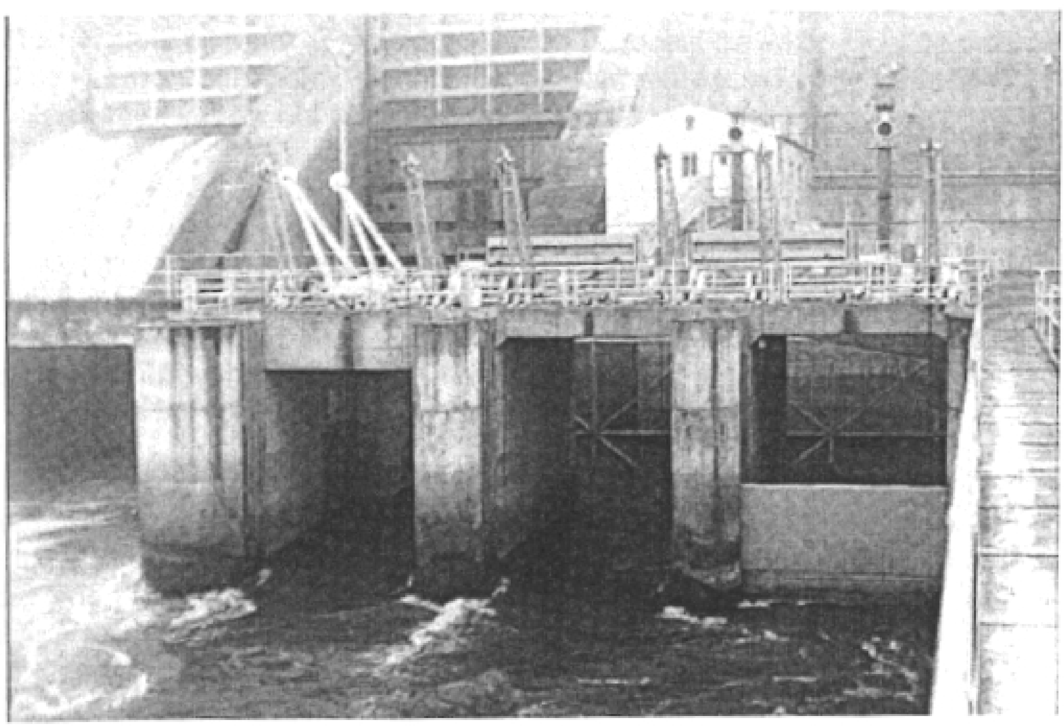


Figure 7-2
McNary Dam Non-Overflow Entrance Between Spillway and Powerhouse

7.1.3 Oregon Shore Fish Ladder

Lowering the tailwater elevations to their original levels would require modifications to entrances and lower portions of the fish ladder similar to those described for the Washington shore.

Modifications to the non-overflow fish entrances (Figure 7-2) between the powerhouse and the spillway include the reestablishment of the entry sills at elevation 242 from their present elevation at 255. The telescoping weir gates and slots would be rehabilitated in order to restore operating capability at the low tailwater. Similar work is required for the stoplog slots. This would require removal of the concrete that fills the slots from elevation 246.25 down to elevation 233.33.

At the time that McNary Dam was modified to accommodate the increased tailwaters, 14 of the original 44 entrance gates remained in place along the powerhouse collection channel. These entrances were not used. The gates were salvaged and the openings were sealed. The powerhouse collection channel however, was unchanged during the modifications. It is assumed that no modifications to the channel should be needed.

No modification was required to the fish ladder entrances at the Oregon shore to accommodate the tailwater increases associated the John Day Project. It is assumed that no further modifications would be required when the tailwaters return to their pre-John Day values.

Modifications to the lower portions of the fish ladder involve weir extensions that were added to raise the effective floor elevation to accommodate the increased tailwater. Existing orifices were sealed, and new orifices built below the new crests. The weir extensions at weirs 254, 257 and 260 would be removed and the old orifices re-opened.

Concrete fill (3feet ±) was placed in the fish collection channel in the Oregon entrances and over floor diffusers. The concrete was added to reduce wall stresses under dewatered conditions. This concrete fill would be removed to restore the original floor elevations. When the floors of the fish collection channel and the Oregon fish entrance channel are lowered, the three wing gates in these two channels would be lengthened. Sluice gates for diffuser water supply removed when the diffusers were filled with concrete would be replaced.

7.1.4 Fish Lock

The pressure fish lock was originally constructed to operate at tailwater elevations below 262. When the modifications were constructed for the increased John Day pool the fish lock was abandoned. It is presently used for flow bypass at the new small hydropower facility. No modifications are required for the fish lock.

7.2 Downstream Passage

7.2.1 Effects on Downstream Passage

The JBS facilities were constructed after the John Day pool was increased. These facilities consist of dewatering screens, sorting and handling facilities, several fish return pipelines and a barge loading structure (see Figures 7-3 through 7-5). Modifications to the facilities are required for operation at the lower tailwater range. Each drawdown alternative at John Day would require the same modifications to the JBS facilities at McNary.

7.2.2 Fish Transportation Pipelines

Three fish transportation pipelines extend into the tailwater pool near the Oregon shore fish ladder entrances. See Figure 7-3. These pipelines are supported on pile frames above the tailwater with pipe inverts at approximately elevation 273. The rock river bottom at the discharge area is at about elevation 240. The operating tailwater range should be from 251 to 270. For low operating tailwater elevations the discharge containing downstream migrants should fall about 22 feet into a depth of about 11 feet. Another set of discharge pipes might be required for use during low tailwater conditions.

7.2.3 Barge Facilities

The average design water level for the barge loading structure is 268 feet and the top of the cells are at elevation 275 feet. The river bottom at the barge facility is at elevation 240 feet providing a depth of 11 feet at the low operating water level (251 feet). Although the water surface is expected to be 5 to 10 feet lower the barges should have ample depth, therefore, no major modifications to the facility are contemplated. However, some minor modifications might be required to the fenders and fish loading system.

7.2.4 Evaluation Facility Drains

Drain pipes from the evaluation facility are also used for emergency fish release and would have to be extended to preclude the discharge of fish onto bare river banks. See Figure 7-4. Drains not potentially discharging fish would not require extension.

7.3 Spill Deflectors

Spillway deflectors were installed at McNary Dam after the tailwater was raised by the John Day Project. Lowering the tailwater would strand these improvements. All alternatives require removal of the deflectors and relocation to a lower elevations.

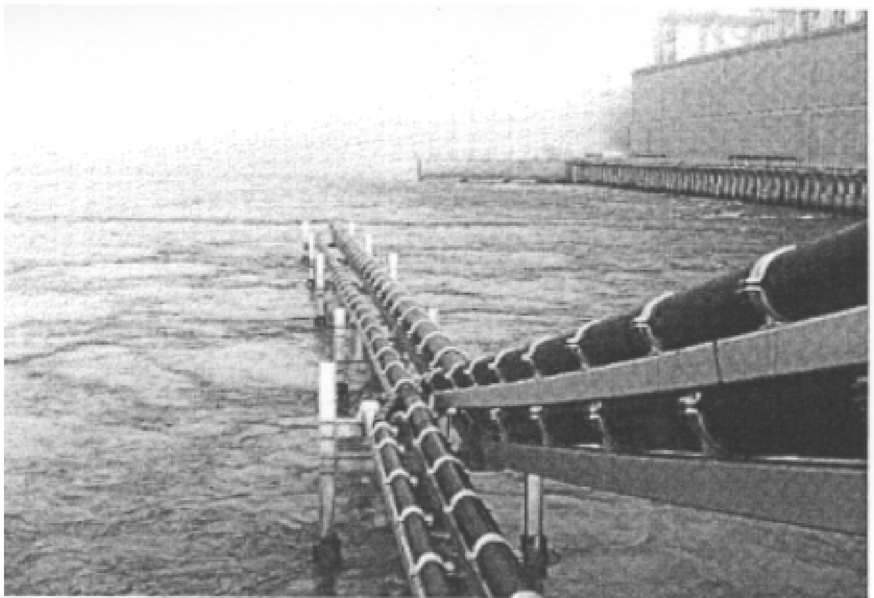


Figure 7-3. McNary Dam Juvenile Bypass System Outfalls

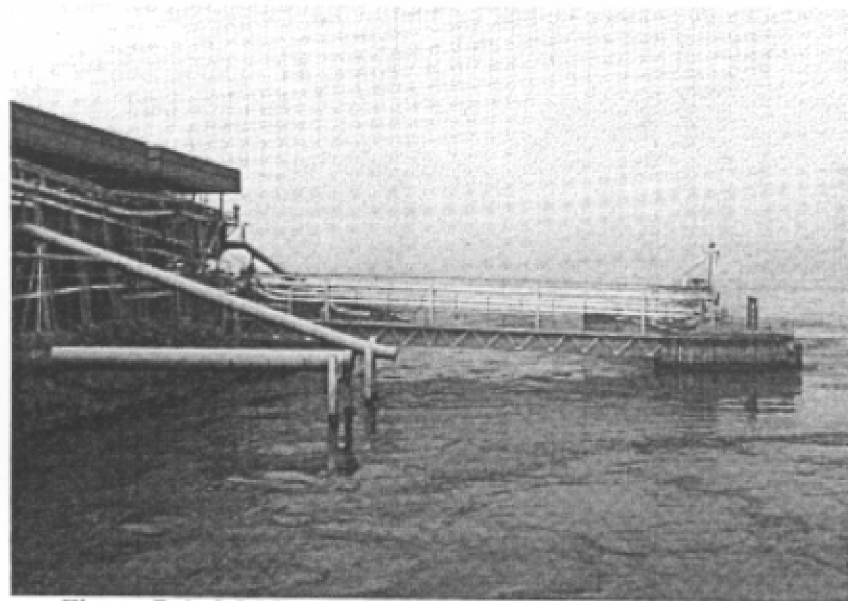


Figure 7-4. McNary Dam Juvenile Bypass System Drains

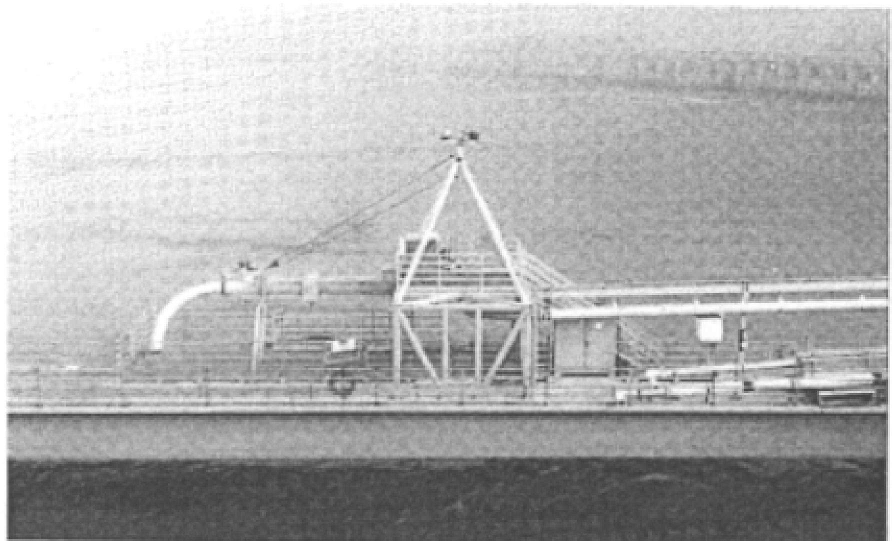


Figure 7-5. McNary Dam Juvenile Bypass System
Barge Loading Facility

7.4 McNary Hydroturbine Operation

The 14 main units at McNary are rated at 80 feet net head. The normal operating range is 62 to 77 feet net head. The tailwater can be lowered five feet and possibly up to 10 feet with no undo problems or change in turbine operation at McNary Project. Lowering the current normal tailwater by five feet would increase the net head operating range to 67-82 feet.

7.5 McNary Navigation Lock

With any form of drawdown at the John Day Project, the navigation channel at the McNary project is affected. The McNary navigation lock was in operation prior to the raising of the John Day pool, and a different type of navigation fleet was in operation at that time (these differences are discussed in the Navigation Appendix). All the locks on the Columbia River are designed for a 15 foot minimum draft vessel. With a minimum river discharge of 50,000 cfs through the McNary project, the downstream navigation lock sill has 15-17 foot of water over the sill at minimum river flow. This is not ideal for navigation, but was assumed to be acceptable at this level of study.

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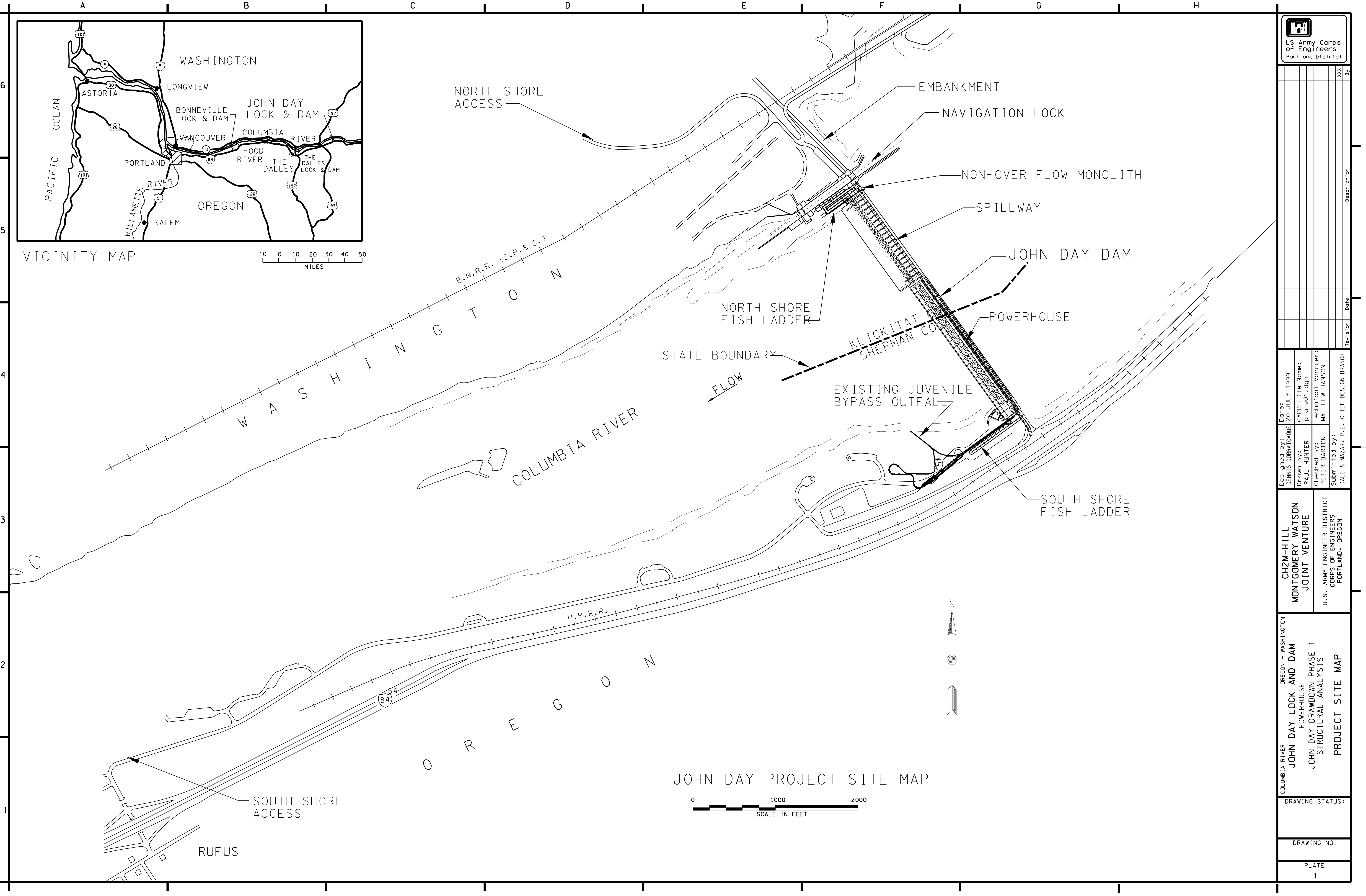
US Army Corps of Engineers, Portland, OR and Walla Walla, WA, January 1999. *Dissolved Gas Abatement Study, Phase II, 60% Draft Technical Report.*

PLATES



WASHINGTON
LONGVIEW
ASTORIA
OCEAN
PACIFIC
BONNEVILLE
LOCK & DAM
JOHN DAY
LOCK & DAM
VANCOUVER
COLUMBIA
RIVER
HOOD
RIVER
THE
DALLES
THE
DALLES
LOCK & DAM
PORTLAND
RIVER
WILLAMETTE
OREGON
SALEM
VICINITY MAP
10 0 10 20 30 40 50
MILES
NORTH SHORE
ACCESS
EMBANKMENT
NAVIGATION LOCK
NON-OVER FLOW MONOLITH
SPILLWAY
JOHN DAY DAM
B.N.R.R. (S.P.& S.)
NORTH SHORE
FISH LADDER
KLICKITAT CO.
SHERMAN CO.
POWERHOUSE
STATE BOUNDARY
FLOW
EXISTING JUVENILE
BYPASS OUTFALL
W A S H I N G T O N
COLUMBIA RIVER
SOUTH SHORE
FISH LADDER
U.P.R.R.
N
84
O R E G O N
SOUTH SHORE
ACCESS
RUFUS
JOHN DAY PROJECT SITE MAP
0 1000 2000
SCALE IN FEET
US Army Corps
of Engineers
Portland District
Revision
Date
Description
By
Designed by:
DENNIS DORRATCAQUE
Drawn by:
PAUL HUNTER
Checked by:
PETER BARTON
Submitted by:
DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date:
20 JULY 1999
CADD File Name:
plate01.dgn
Technical Manager:
MATTHEW HANSON
CH2M-HILL
MONTGOMERY WATSON
JOINT VENTURE
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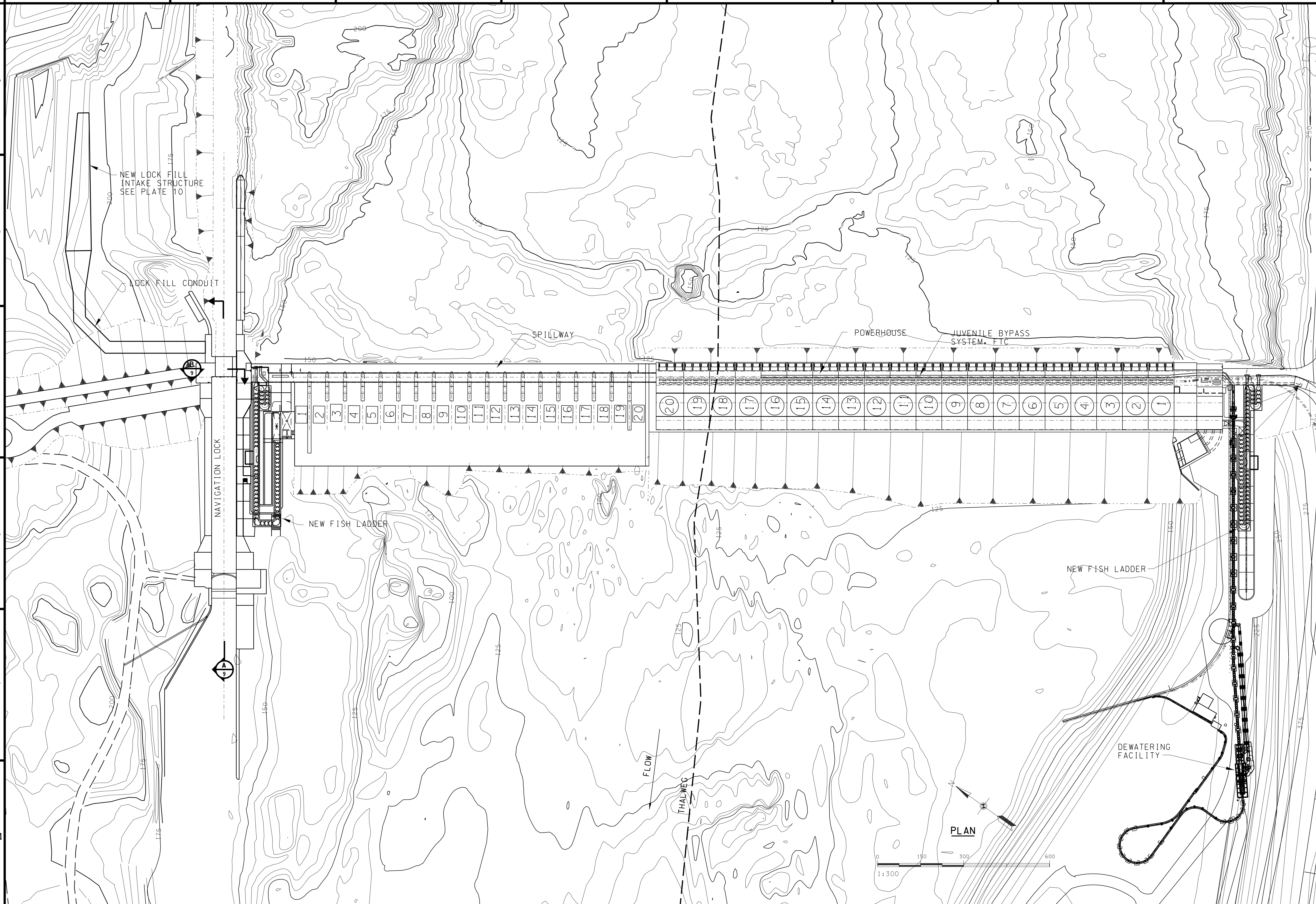
COLUMBIA RIVER
OREGON - WASHINGTON
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
PROJECT SITE MAP
DRAWING STATUS:
DRAWING NO.
PLATE
1



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Portland District

Revision	Date	Description	By
			XXX

Designed by: DENNIS DORRATCAQUE
Date: 20 JULY 1999
Drawn by: PAUL HUNTER
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Checked by: PETER BARTON
Technical Manager: MATTHEW HANSON
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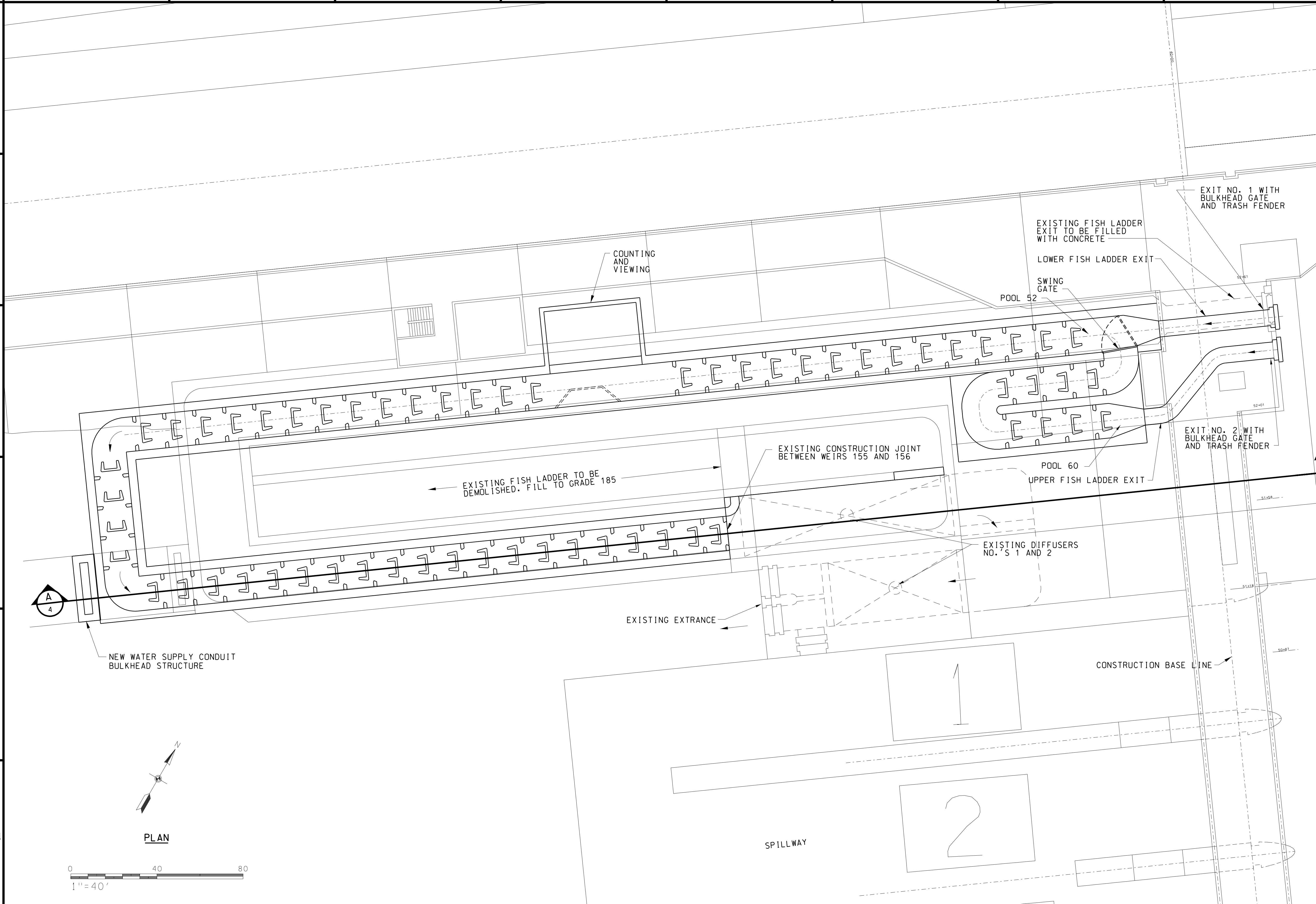
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 1
ALTERNATIVE 1 PLAN

DRAWING STATUS:

DRAWING NO.

PLATE
2



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Designed by: CLINT SMITH
Drawn by: PAUL HUNTER
Checked by: DENNIS DORRATCAQUE
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate03.dgn
Technical Manager: MATTHEW HANSON

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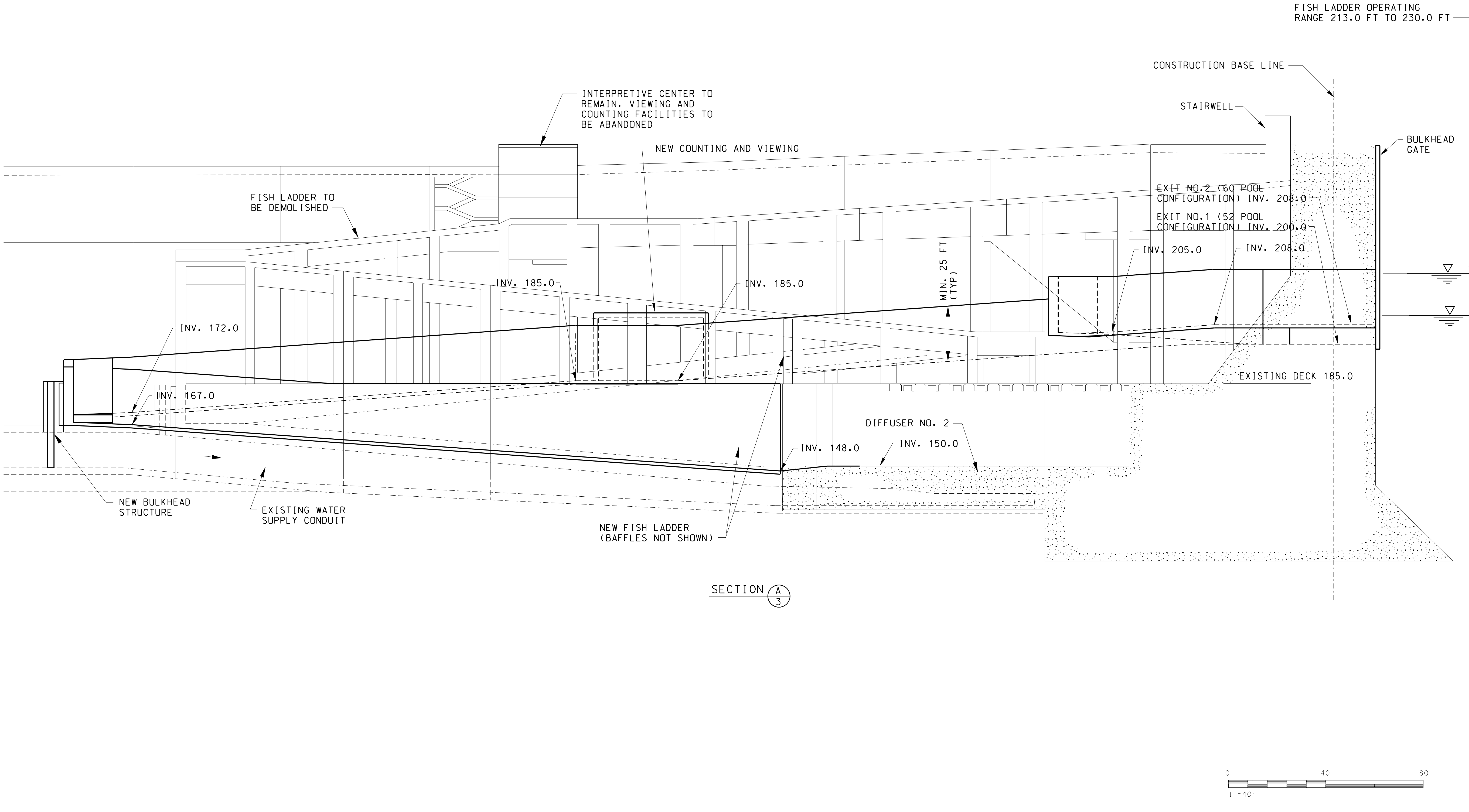
OREGON - WASHINGTON
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO.1
NORTH SHORE FISH LADDER PLAN

DRAWING STATUS:

DRAWING NO.

PLATE
3



SECTION A/3

US Army Corps of Engineers
Portland District

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			XXX

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Drawn by: PAUL HUNTER
Checked by: DENNIS DORRATCAQUE
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Date: 20 JULY 1999
CADD File Name: plate04.dgn
Technical Manager: MATTHEW HANSON

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JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO 1
NORTH SHORE FISH LADDER SECTIONS

DRAWING STATUS:

DRAWING NO.

PLATE
4

PLAN OF EXISTING FACILITIES

US Army Corps of Engineers
Portland District

Revision	Date	Description	By
			xxx

Designed by: DENNIS DORRATCAQUE
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Checked by: PETER BARTON
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate05.dgn
Technical Manager: MATTHEW HANSON

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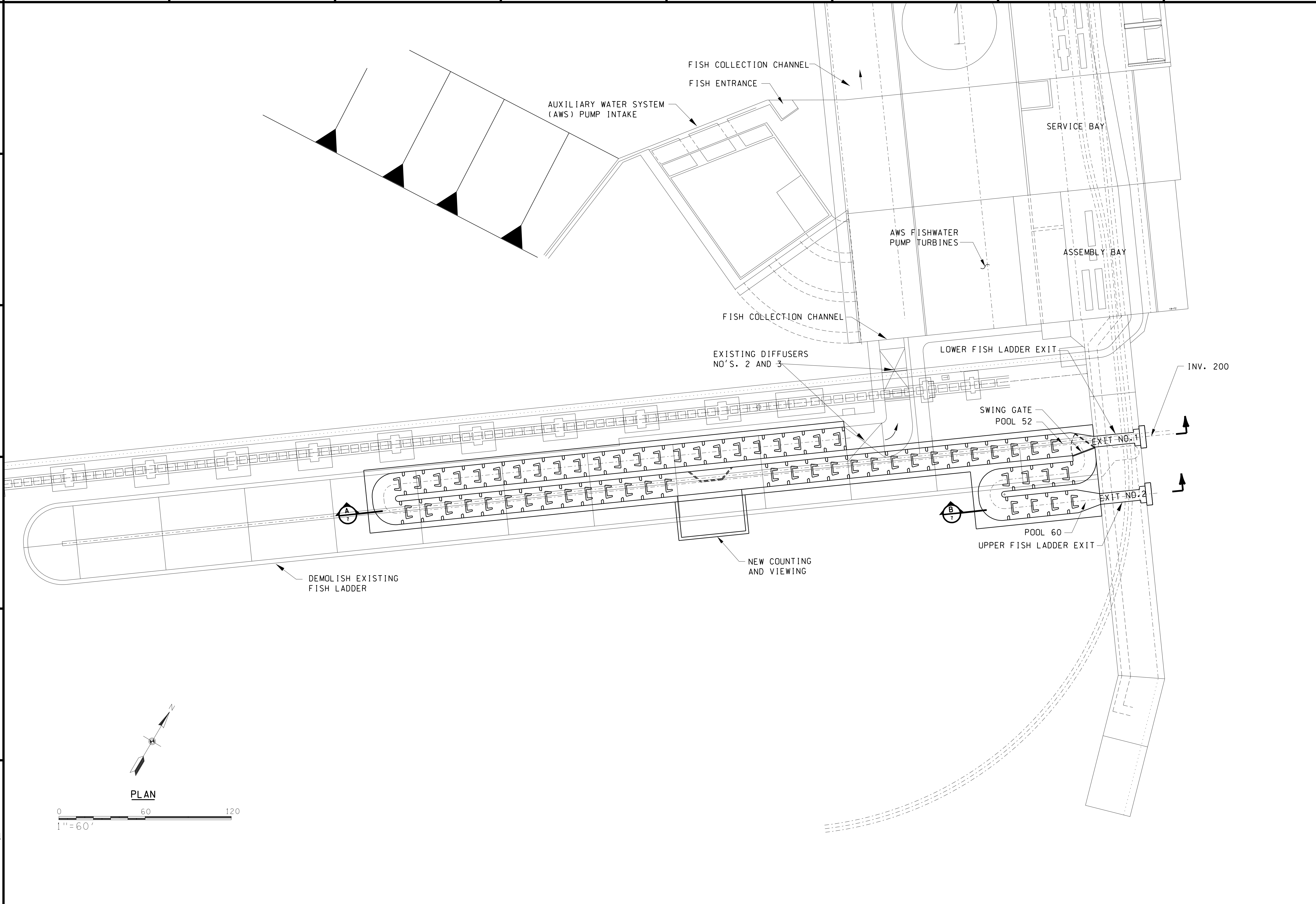
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 1
SOUTH SHORE PLAN

DRAWING STATUS:

DRAWING NO.

PLATE
5



US Army Corps of Engineers
Portland District

Revision	Date	Description	By
			XXX

Designed by: CLINT SMITH
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Checked by: DENNIS DORRATCAQUE
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate06.dgn
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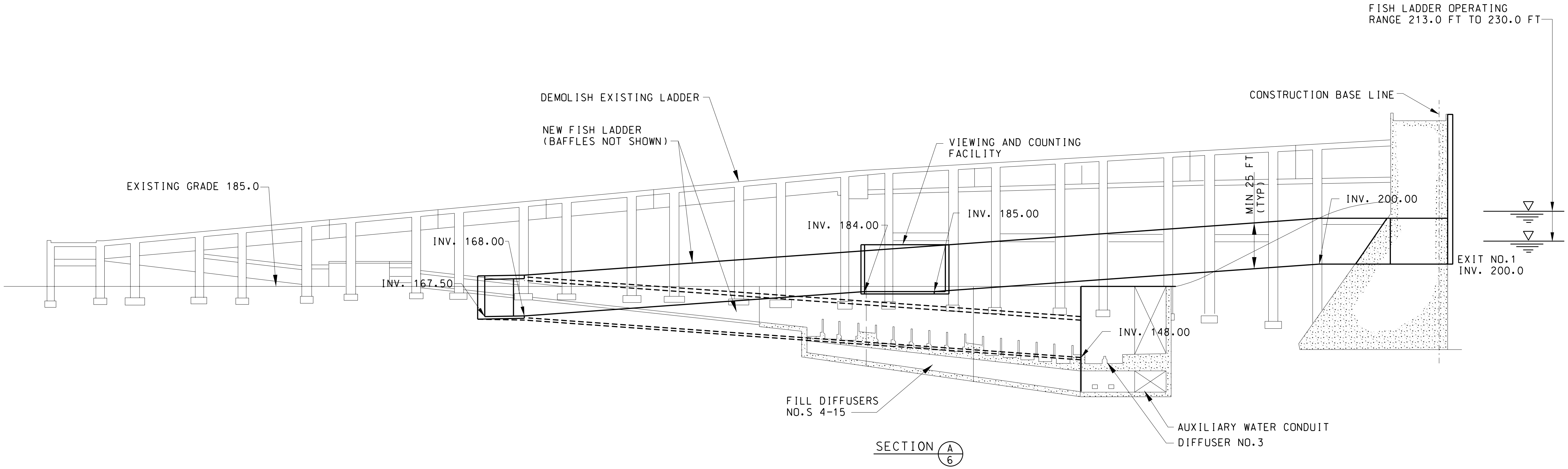
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 1
SOUTH SHORE FISH LADDER PLAN

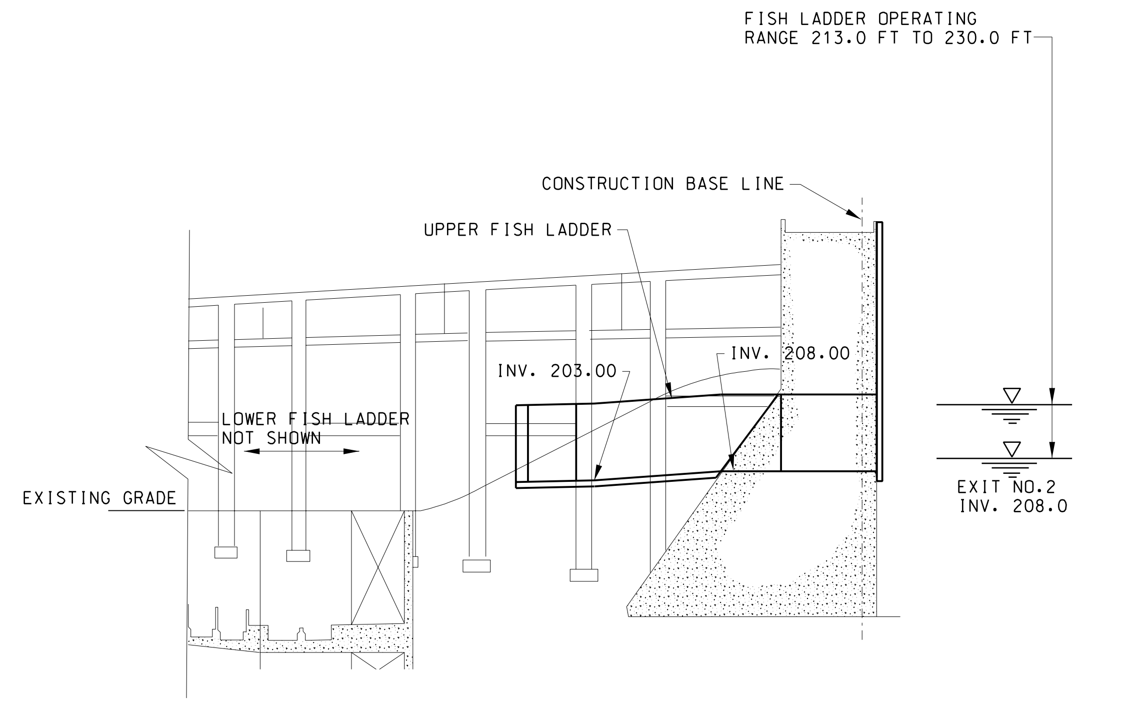
DRAWING STATUS:

DRAWING NO.

PLATE
6



SECTION A/6



SECTION B/6

0 60 120
1"=60'

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Portland District

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			xxx

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Drawn by: PAUL HUNTER
Checked by: DENNIS DORRATCAQUE
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Date: 20 JULY 1999
CADD File Name: plate07.dgn
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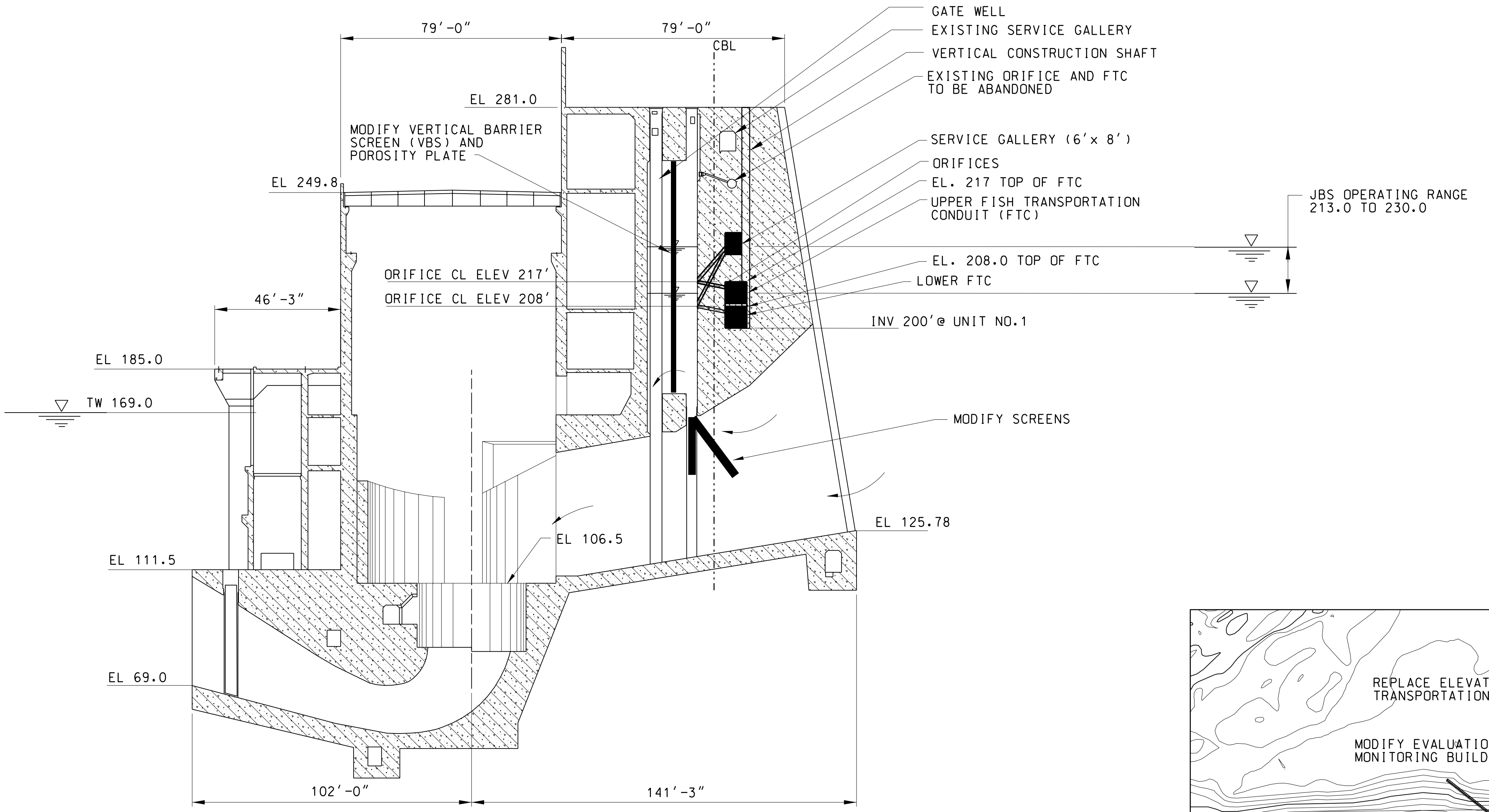
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 1
SOUTH SHORE FISH LADDER SECTION

DRAWING STATUS:

DRAWING NO.

PLATE
7



SECTION A/4

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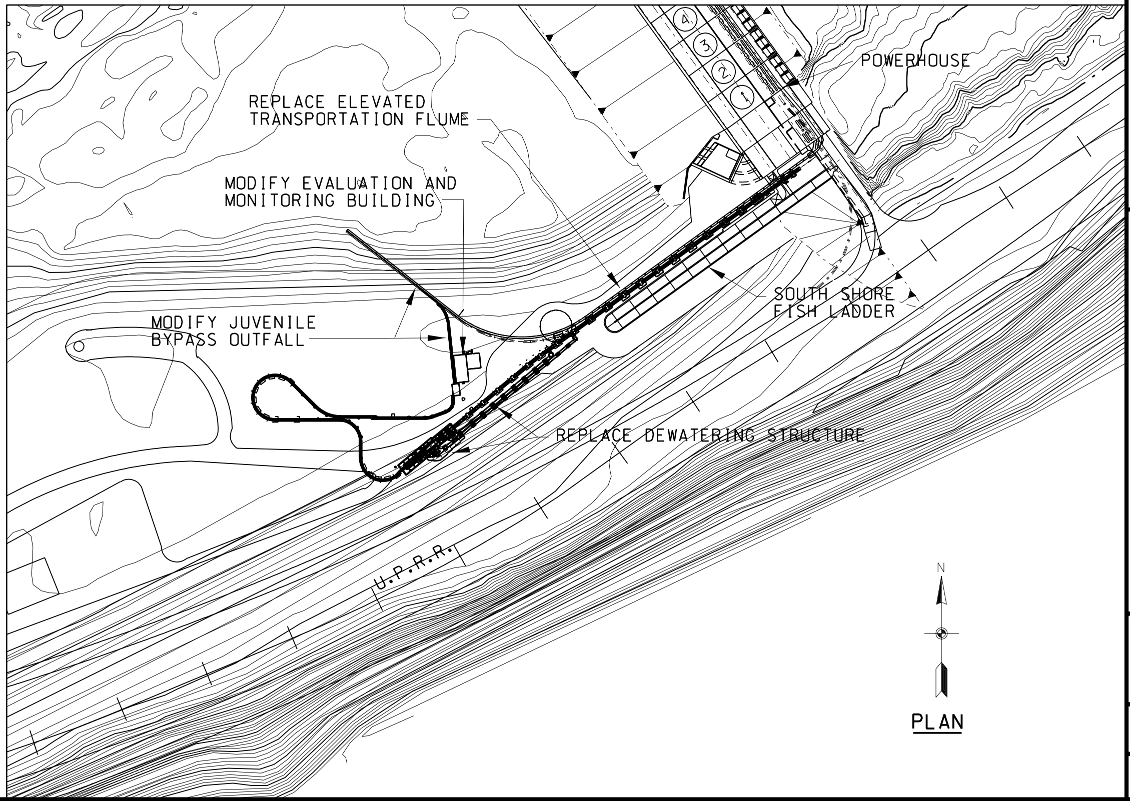

PLAN

US Army Corps of Engineers
Portland District

Revision	Date	Description	By
			XXX

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Checked by: DENNIS DORRATCAQUE
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Date: 20 JULY 1999
CADD File Name: plate08.dgn
Technical Manager: MATTHEW HANSON

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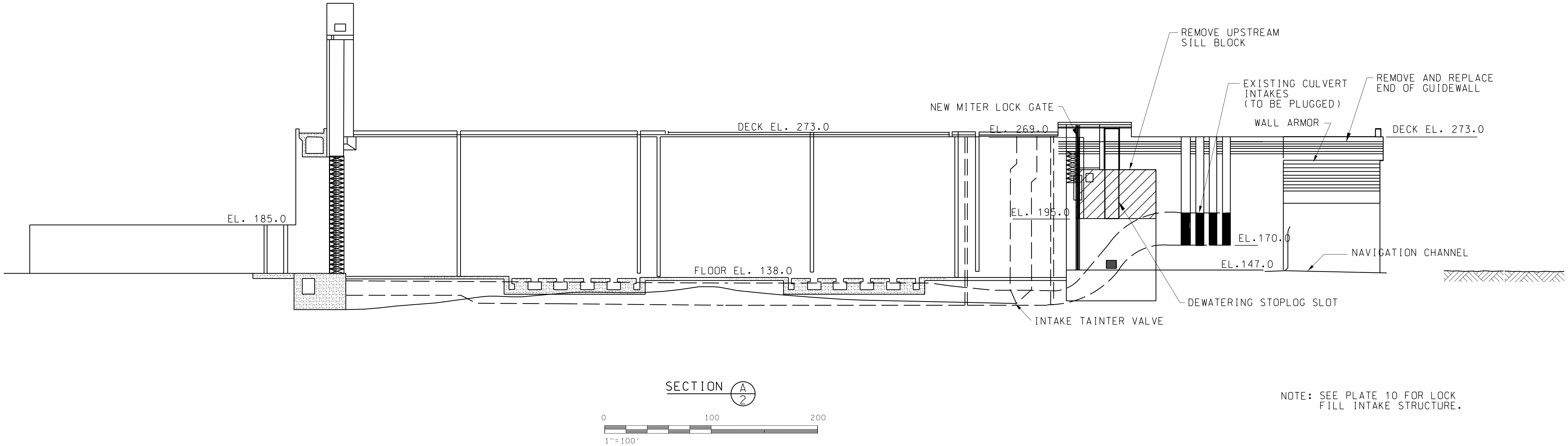
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 1
JUVENILE BYPASS SYSTEM

DRAWING STATUS:

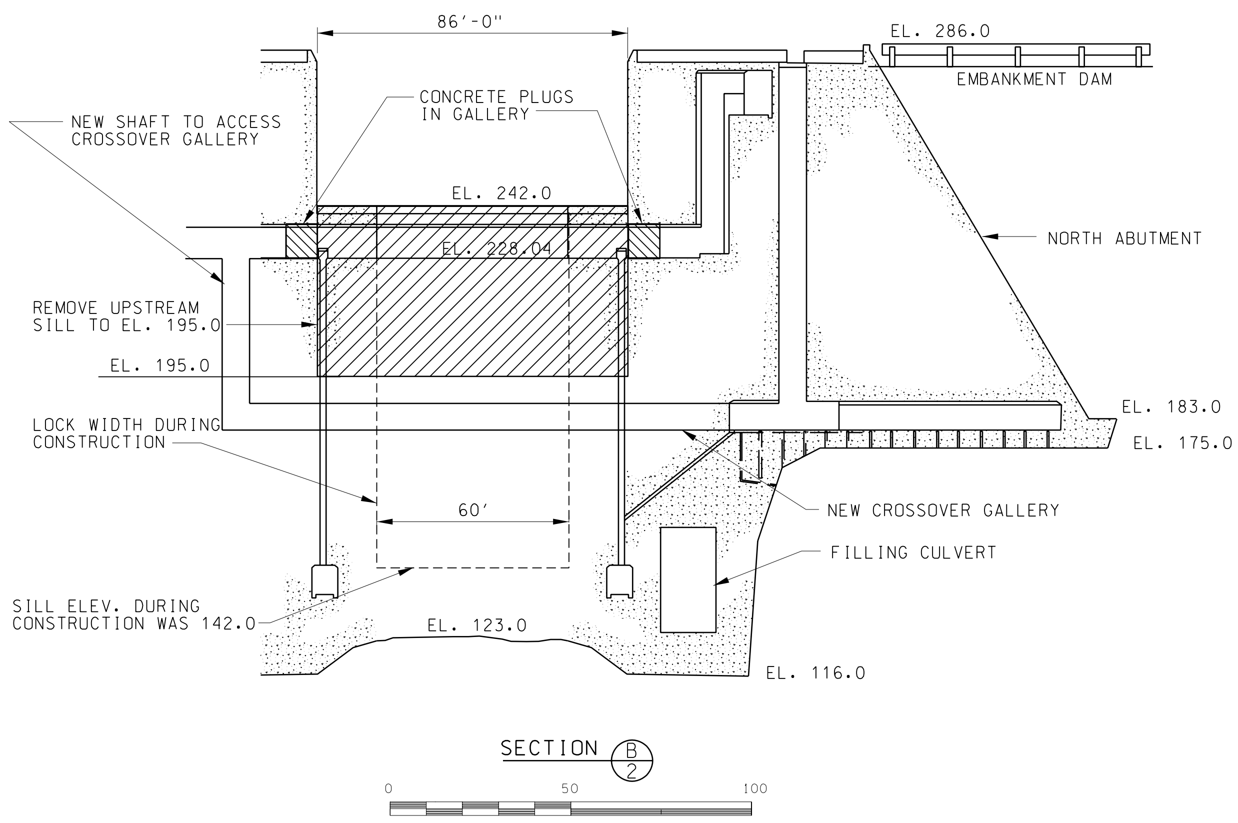
DRAWING NO.

PLATE
8

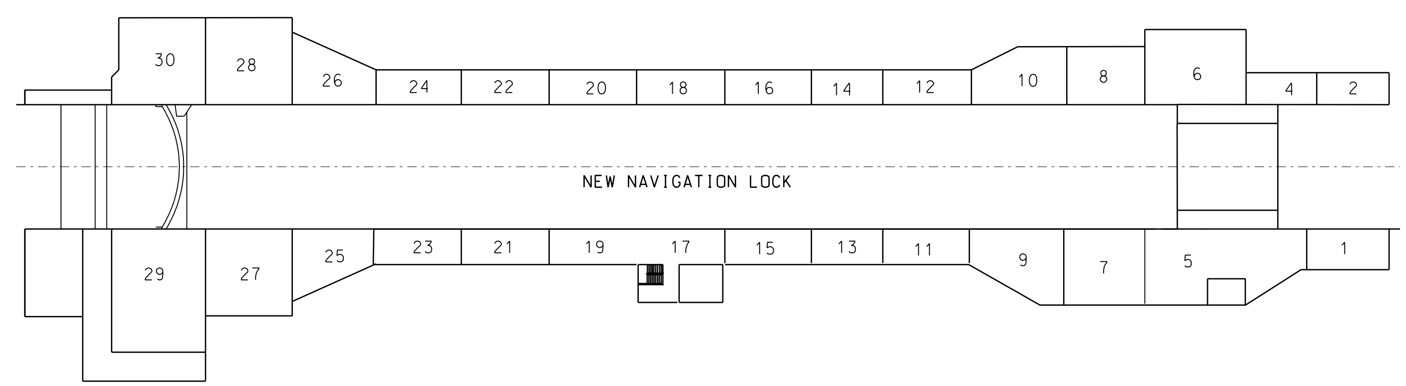


SECTION A/2

NOTE: SEE PLATE 10 FOR LOCK FILL INTAKE STRUCTURE.



SECTION B/2



NAVIGATION LOCK MONOLITH KEY

NTS

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Checked by: PETER BARTON
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate09.dgn
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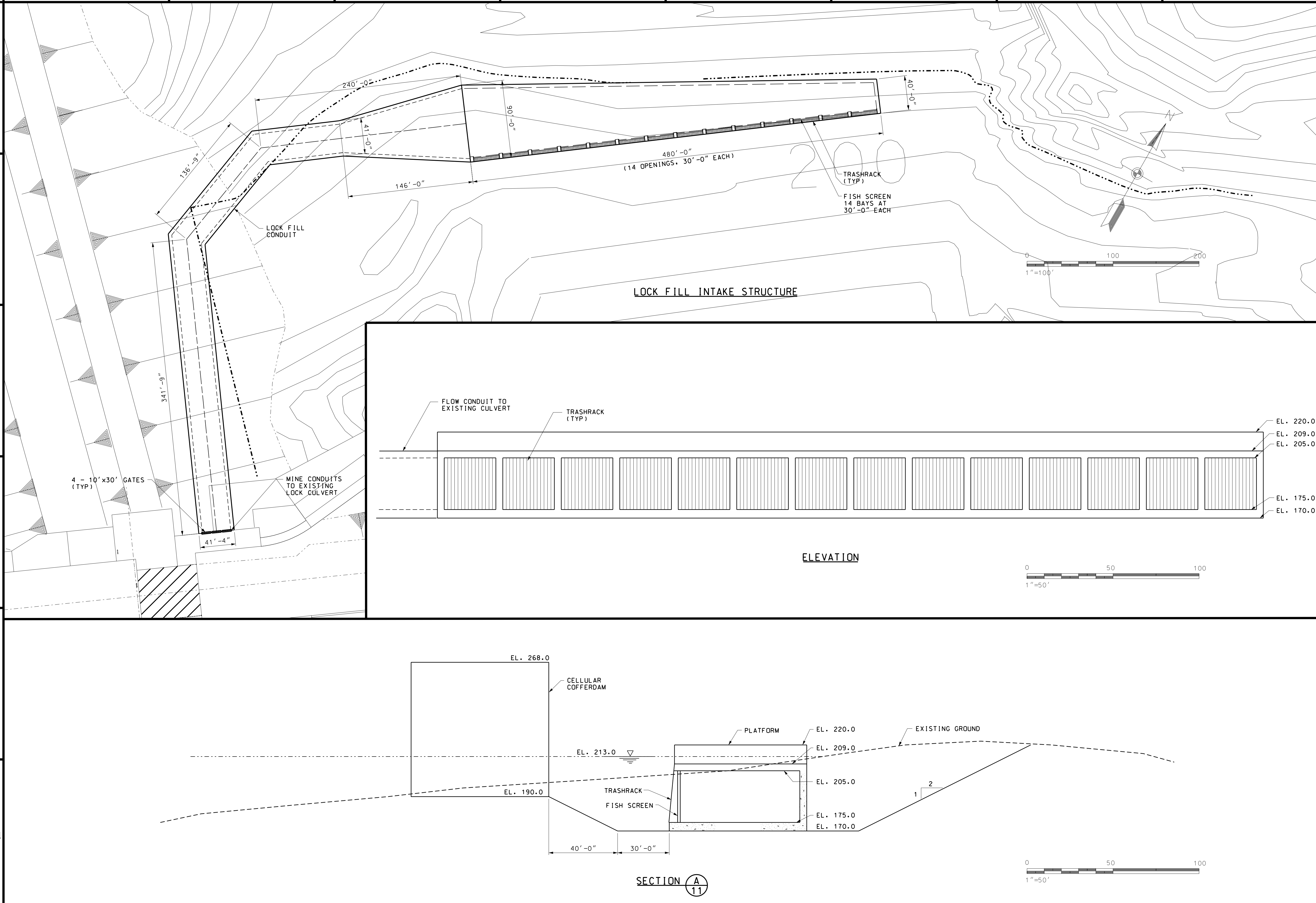
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 1
NAVIGATION LOCK SECTIONS

DRAWING STATUS:

DRAWING NO.

PLATE
9



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Drawn by: PAUL HUNTER
Checked by: DENNIS DORRATCAQUE
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate10.dgn
Technical Manager: MATTHEW HANSON

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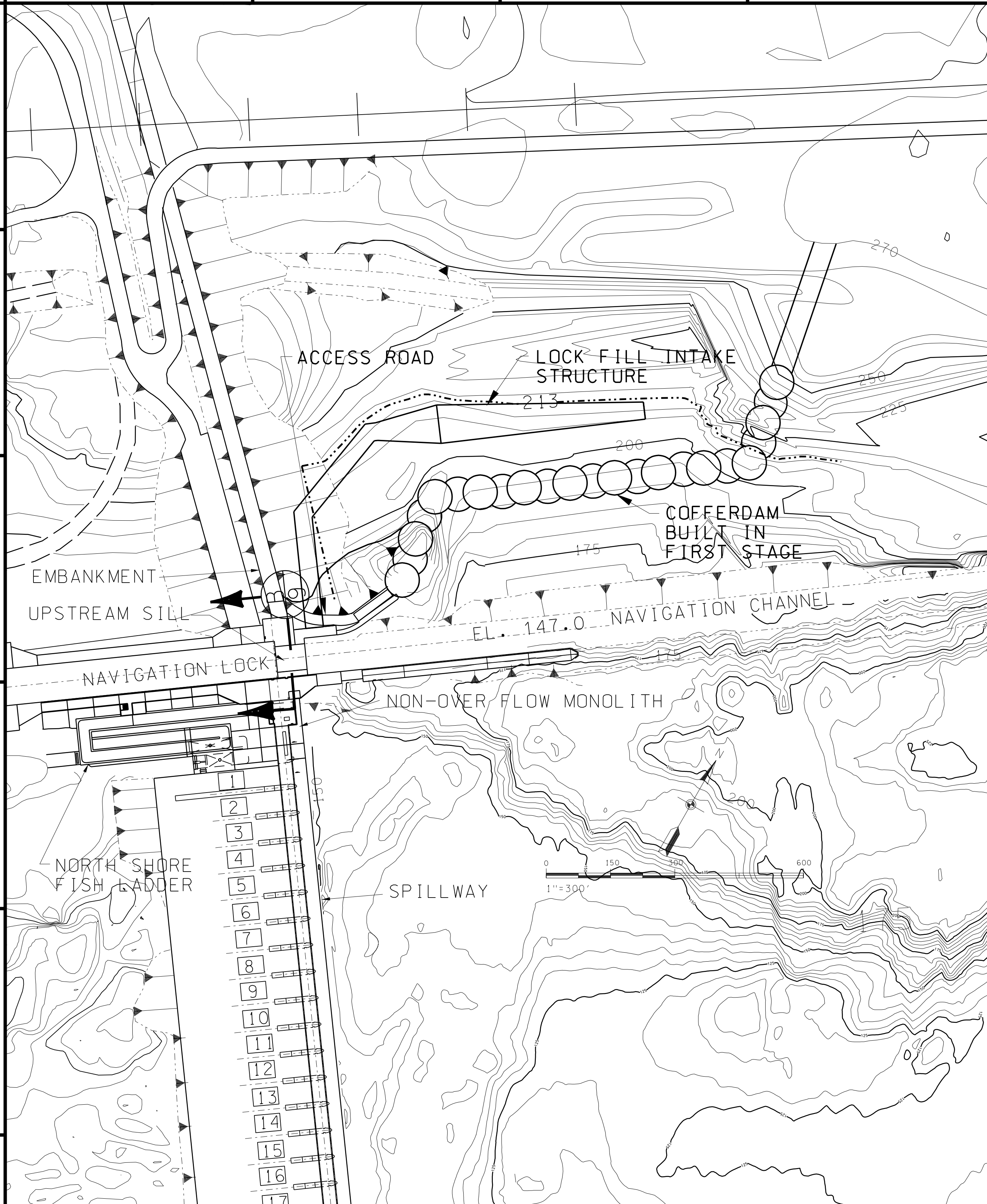
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO.1
NAVIGATION LOCK WATER SUPPLY INTAKE

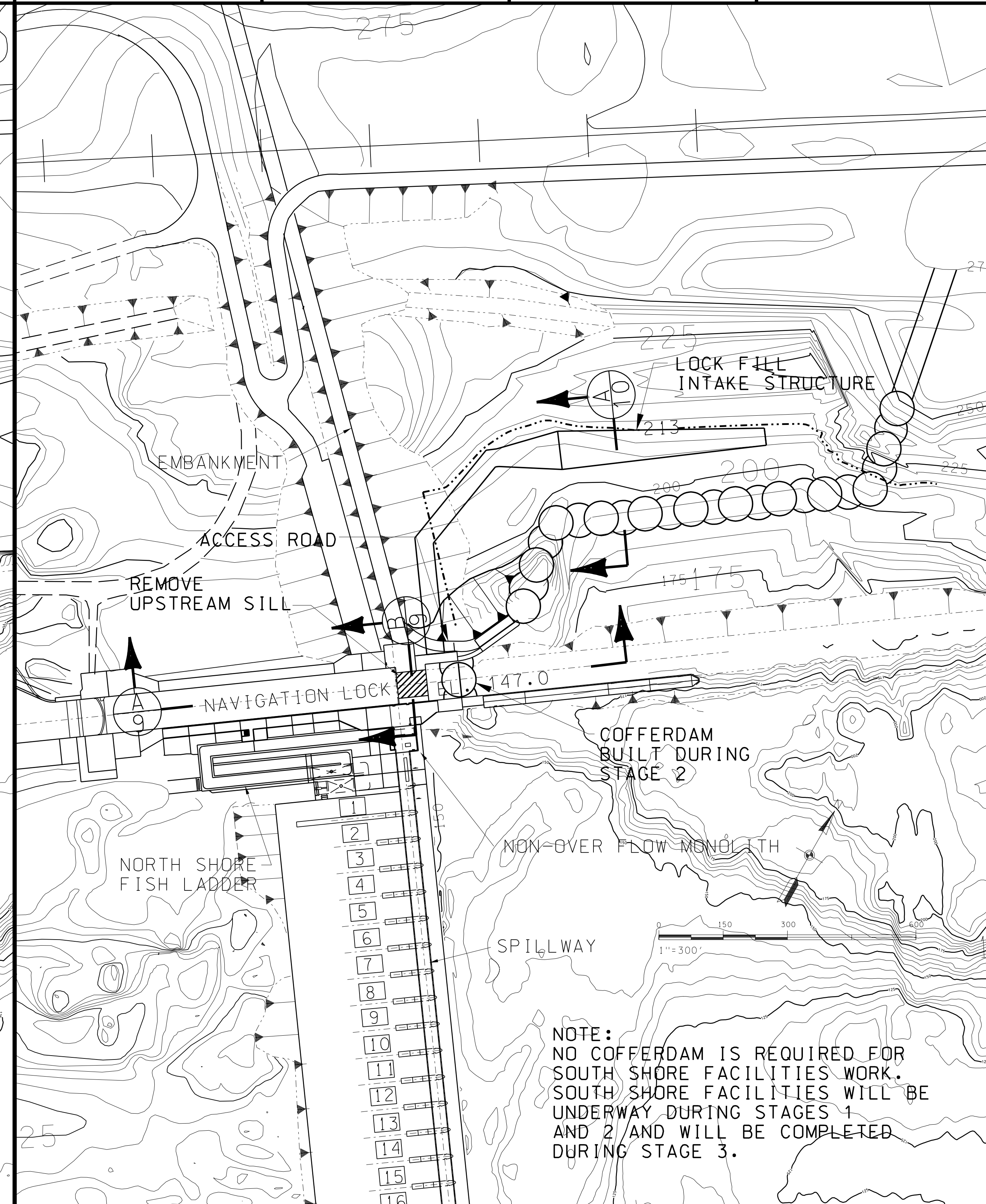
DRAWING STATUS:

DRAWING NO.

PLATE 10



PLAN
COFFERDAM STAGE 1



NOTE:
NO COFFERDAM IS REQUIRED FOR SOUTH SHORE FACILITIES WORK. SOUTH SHORE FACILITIES WILL BE UNDERWAY DURING STAGES 1 AND 2 AND WILL BE COMPLETED DURING STAGE 3.

PLAN
COFFERDAM STAGE 2

US Army Corps of Engineers
Portland District

Revision	Date	Description	By
			XXX

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Drawn by: PAUL HUNTER
Checked by: PETER BARTON
Date: 20 JULY 1999
CADD File Name: plate11.dgn
Technical Manager: MATTHEW HANSON
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH

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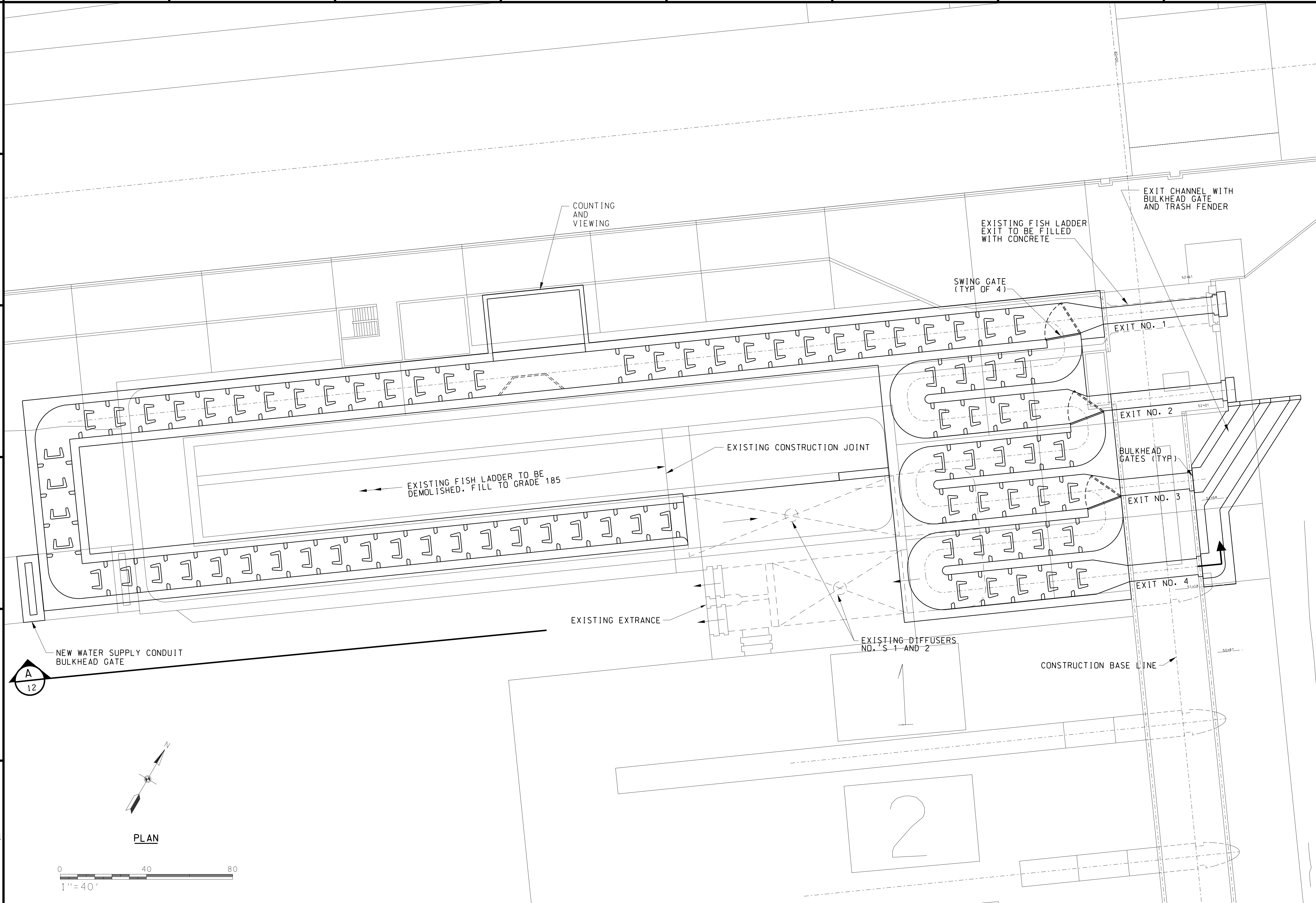
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON

COLUMBIA RIVER — OREGON - WASHINGTON
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 1
CONSTRUCTION SEQUENCING

DRAWING STATUS:

DRAWING NO.

PLATE
11



US Army Corps of Engineers
Portland District

Revision	Date	Description	By
			XXX

Designed by: CLINT SMITH	Date: 20 JULY 1999
Drawn by: PAUL HUNTER	CADD File Name: plate12.dgn
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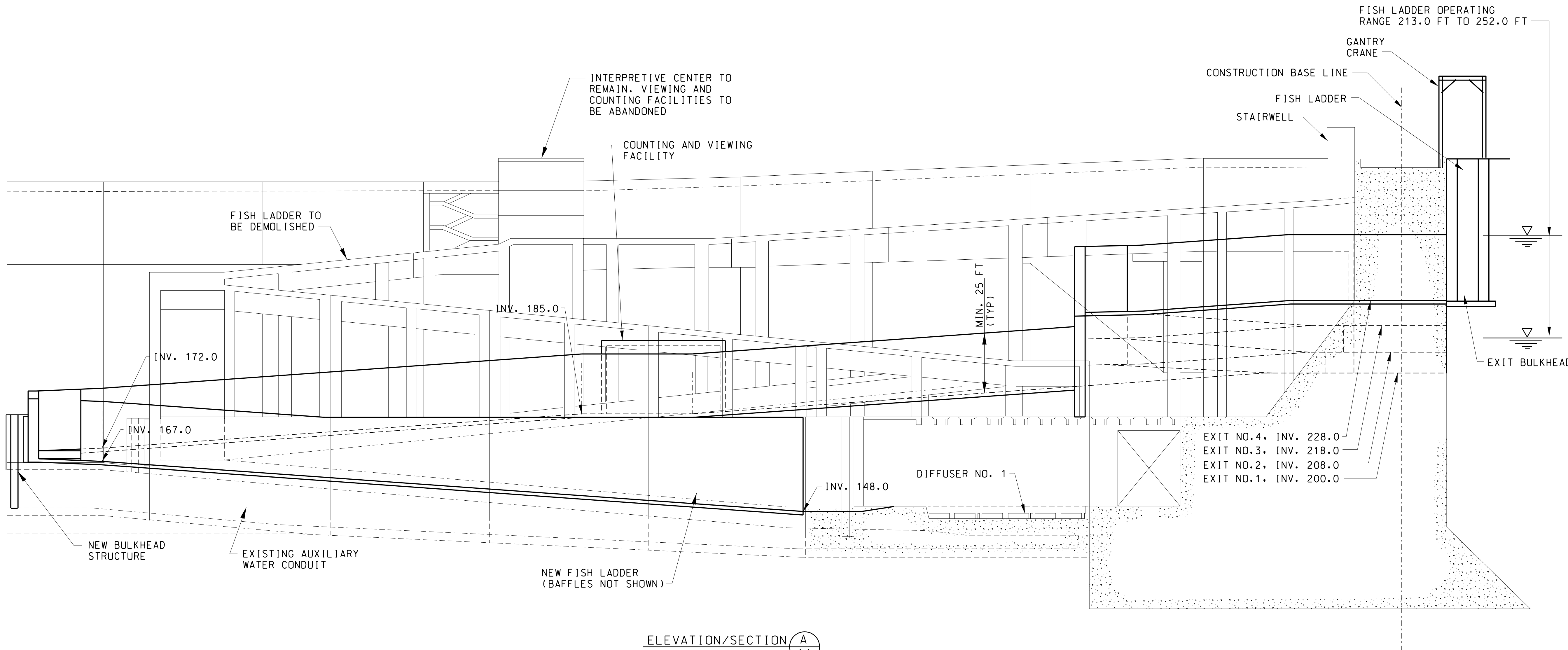
PORTLAND, OREGON

COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO 2
NORTH SHORE FISH LADDER PLAN

DRAWING STATUS:

DRAWING NO.

PLATE
12



ELEVATION/SECTION A/11

0 40 80
1"=40'

US Army Corps of Engineers
Portland District

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CADD File Name: plate13.dgn
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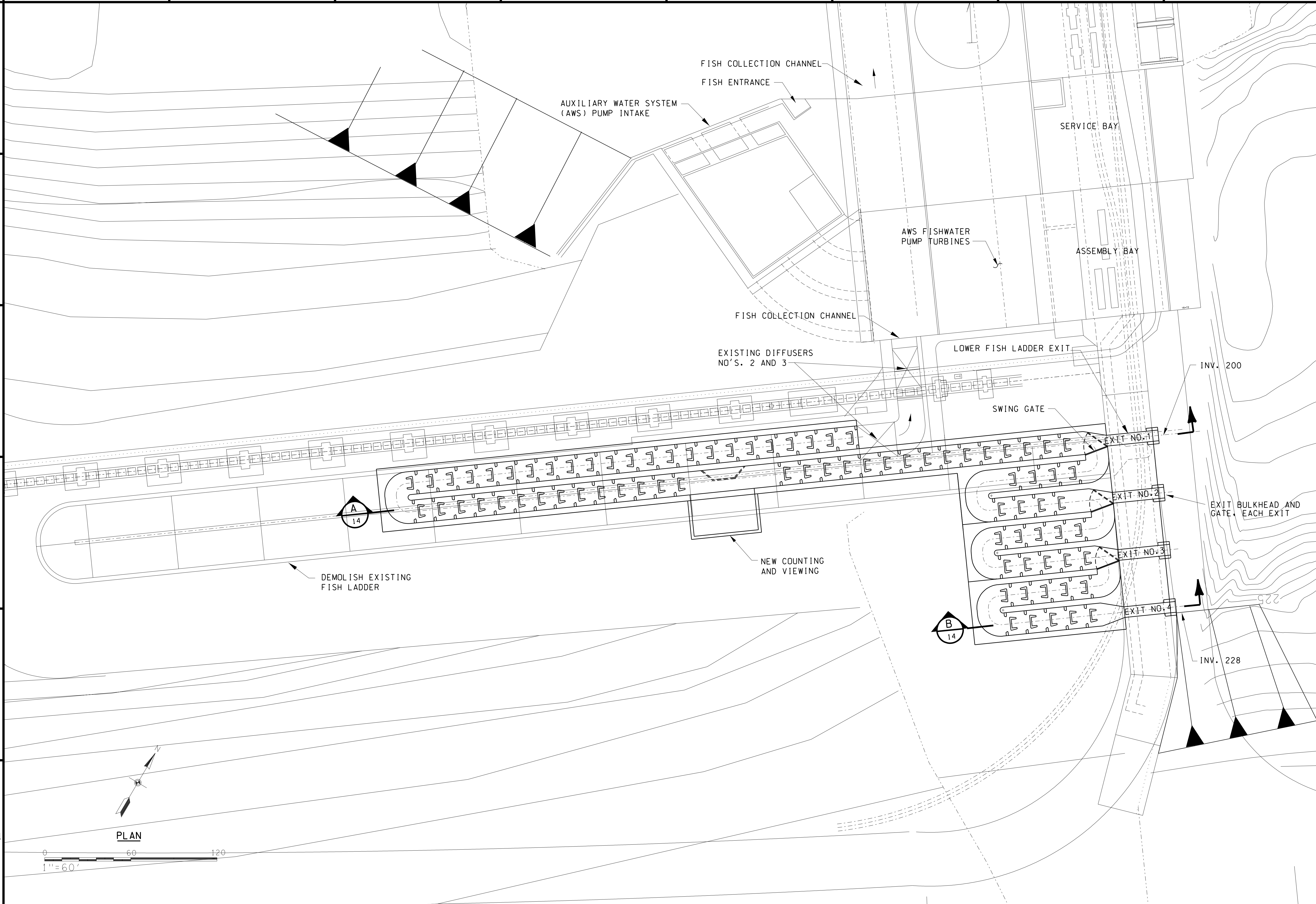
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 2
NORTH SHORE FISH LADDER SECTIONS

DRAWING STATUS:

DRAWING NO.

PLATE
13



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Portland District

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Date: 20 JULY 1999
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CADD File Name:
Checked by: DENNIS DORRATCAQUE
Technical Manager: MATTHEW HANSON
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PORTLAND, OREGON

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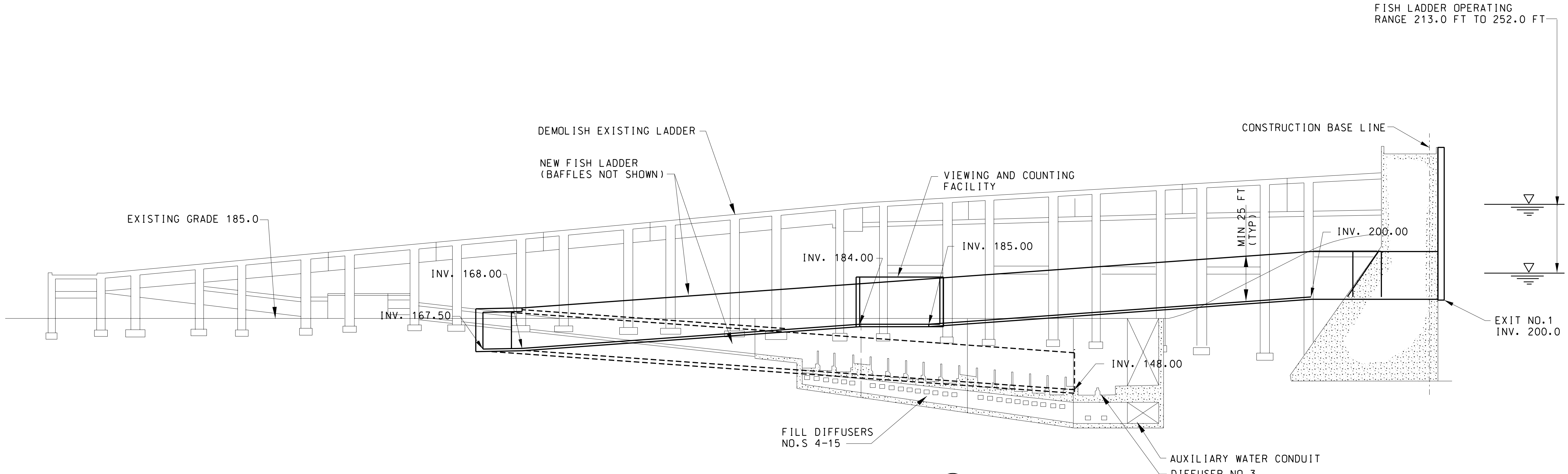
OREGON - WASHINGTON
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 2
SOUTH SHORE FISH LADDER PLAN

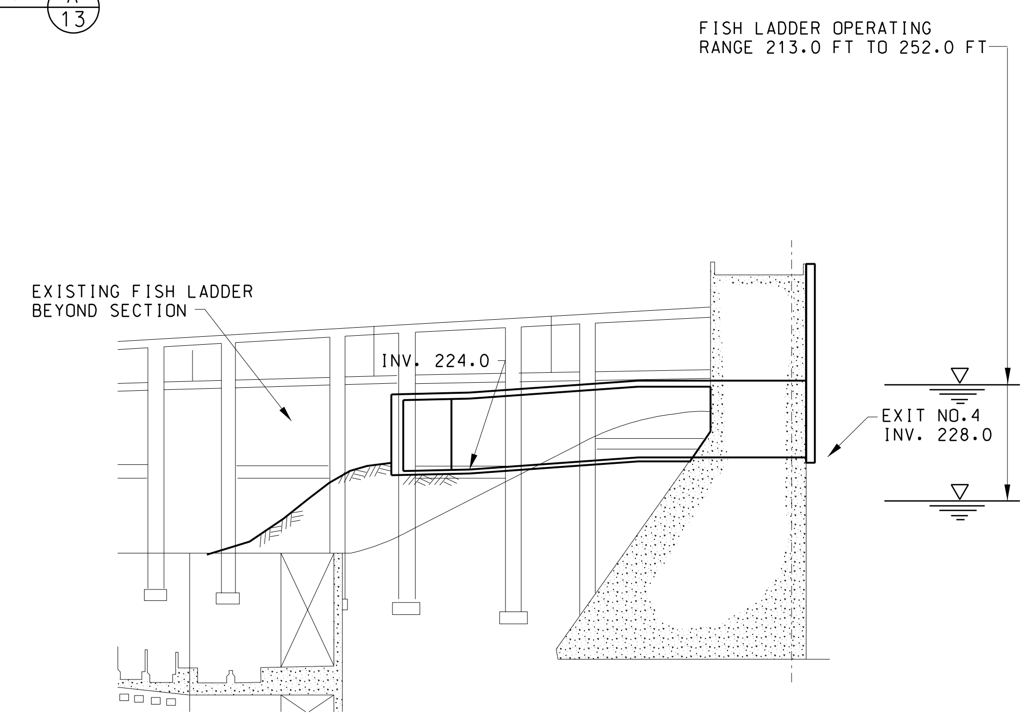
DRAWING STATUS:

DRAWING NO.

PLATE
14



SECTION A/13



SECTION B/13

0 60 120
1"=60'

US Army Corps of Engineers
Portland District

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			XXX

Designed by: CLINT SMITH
Drawn by: PAUL HUNTER
Checked by: DENNIS DORRATCAQUE
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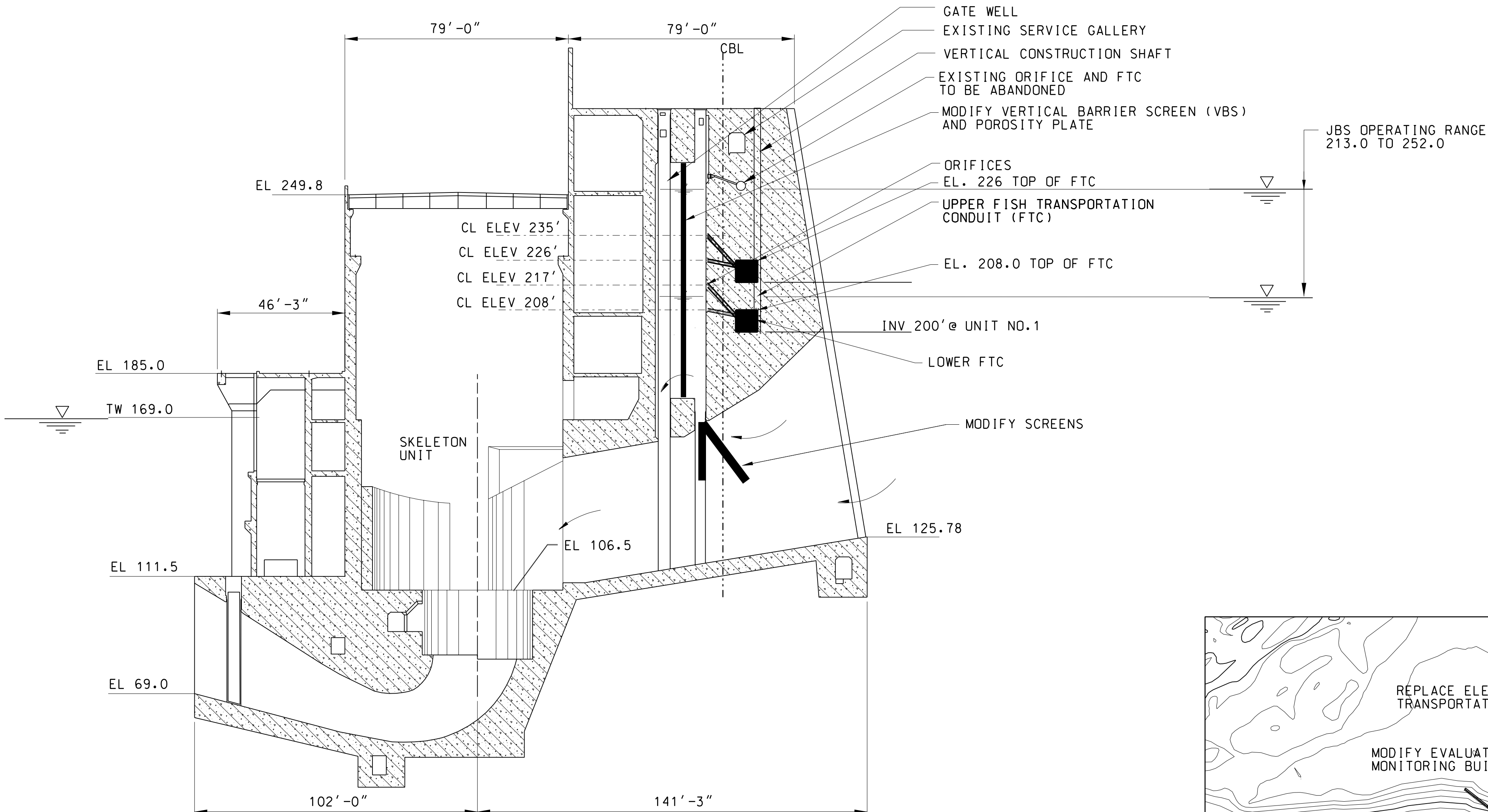
COLUMBIA RIVER — OREGON - WASHINGTON
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 2
SOUTH SHORE FISH LADDER SECTION

DRAWING STATUS:

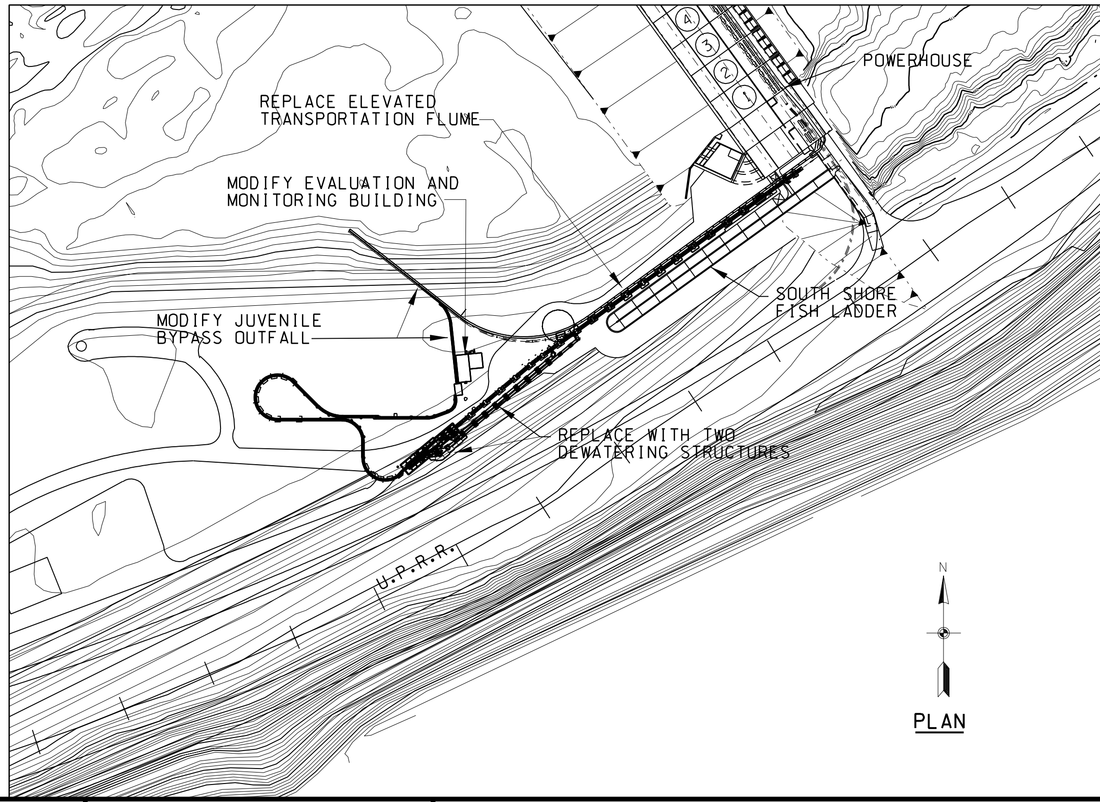
DRAWING NO.

PLATE
15



SECTION A/4

0 50 100
1"=50'



PLAN

US Army Corps of Engineers
Portland District

Revision	Date	Description	By
			xxx

Designed by: CLINT SMITH
Drawn by: PAUL HUNTER
Checked by: DENNIS DORRATCAGUE
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate16.dgn
Technical Manager: MATTHEW HANSON

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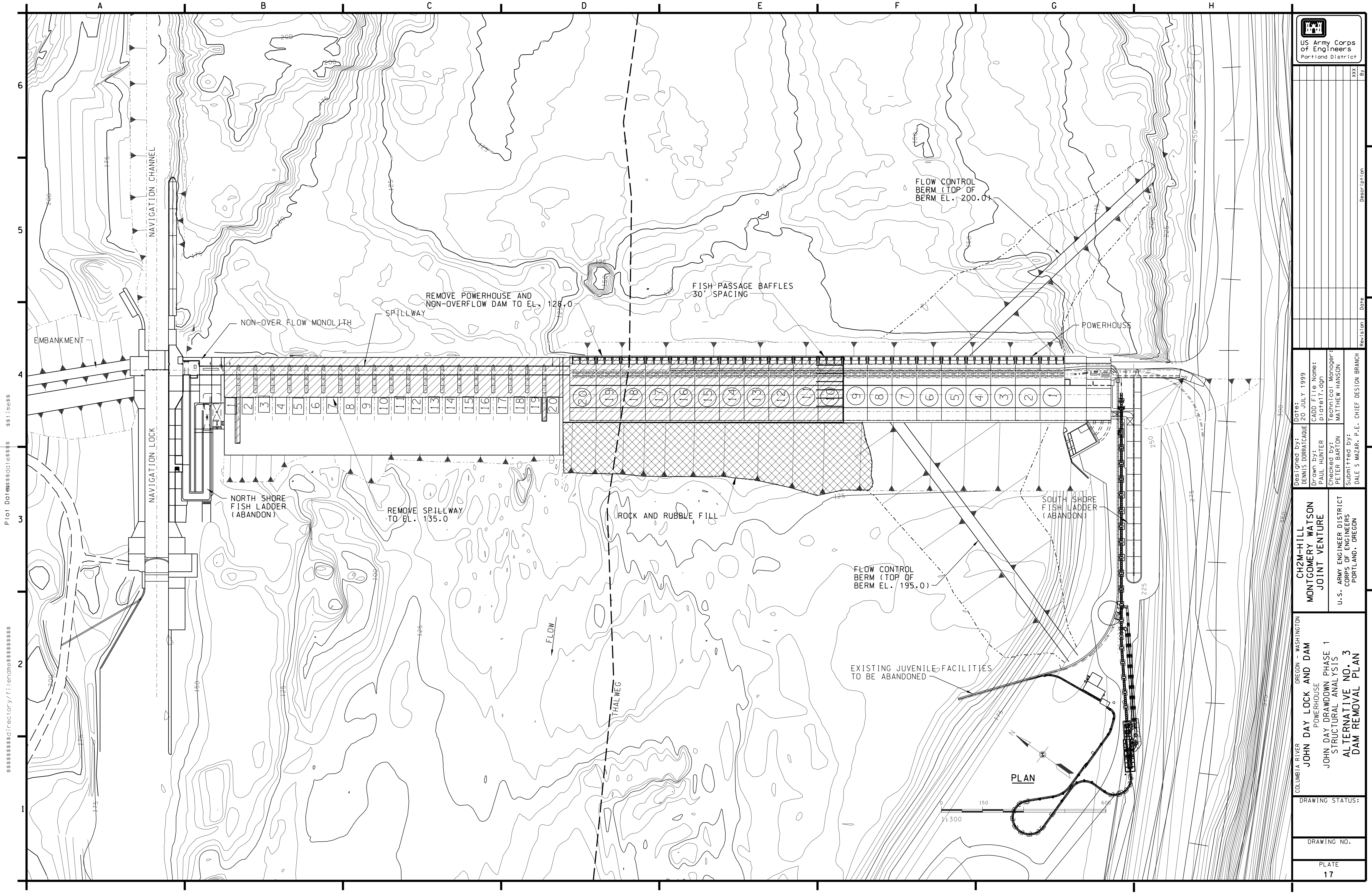
CORPS OF ENGINEERS
PORTLAND, OREGON

COLUMBIA RIVER OREGON - WASHINGTON
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 2
JUVENILE BYPASS SYSTEM

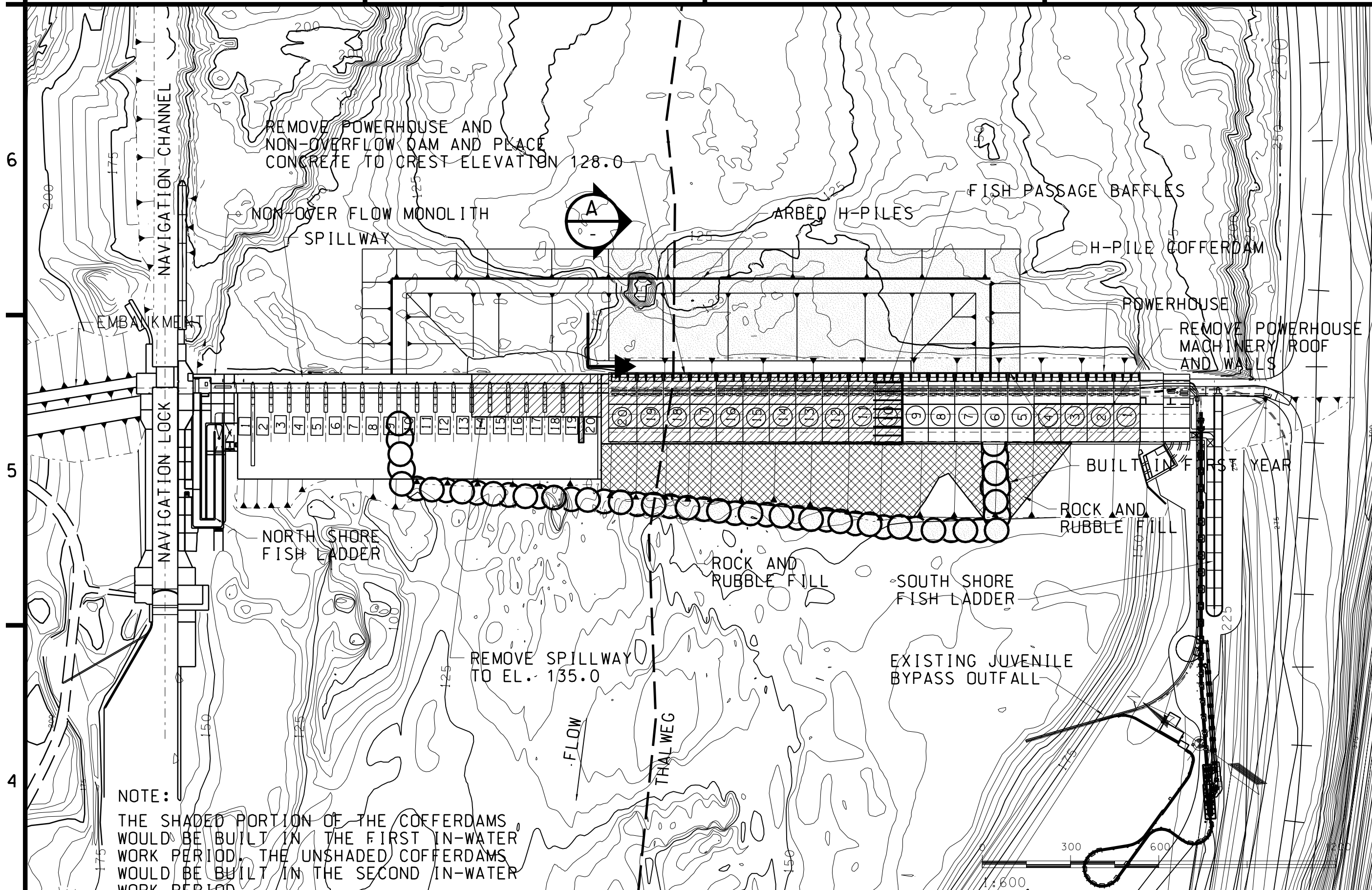
DRAWING STATUS:

DRAWING NO.

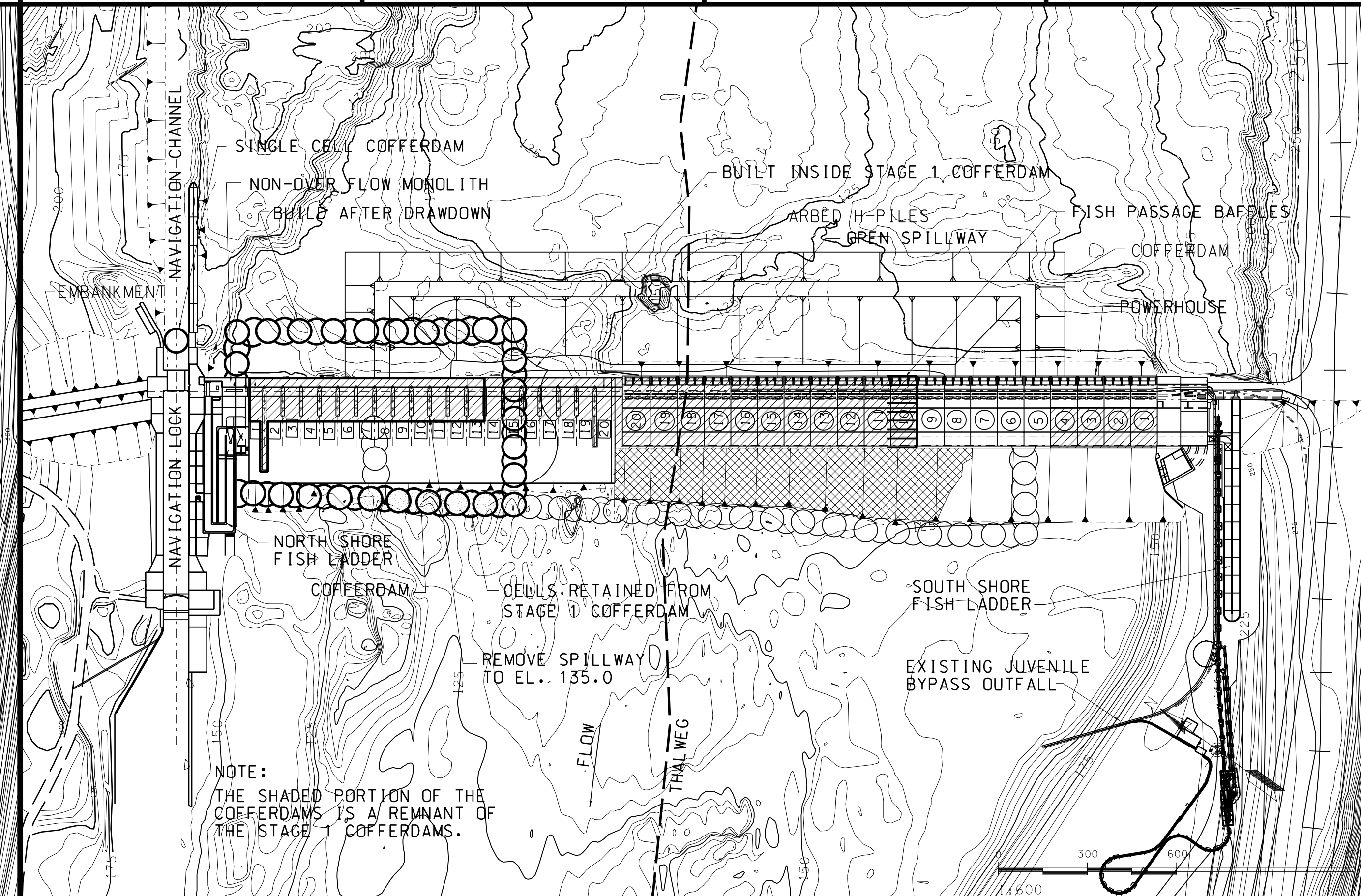
PLATE
16



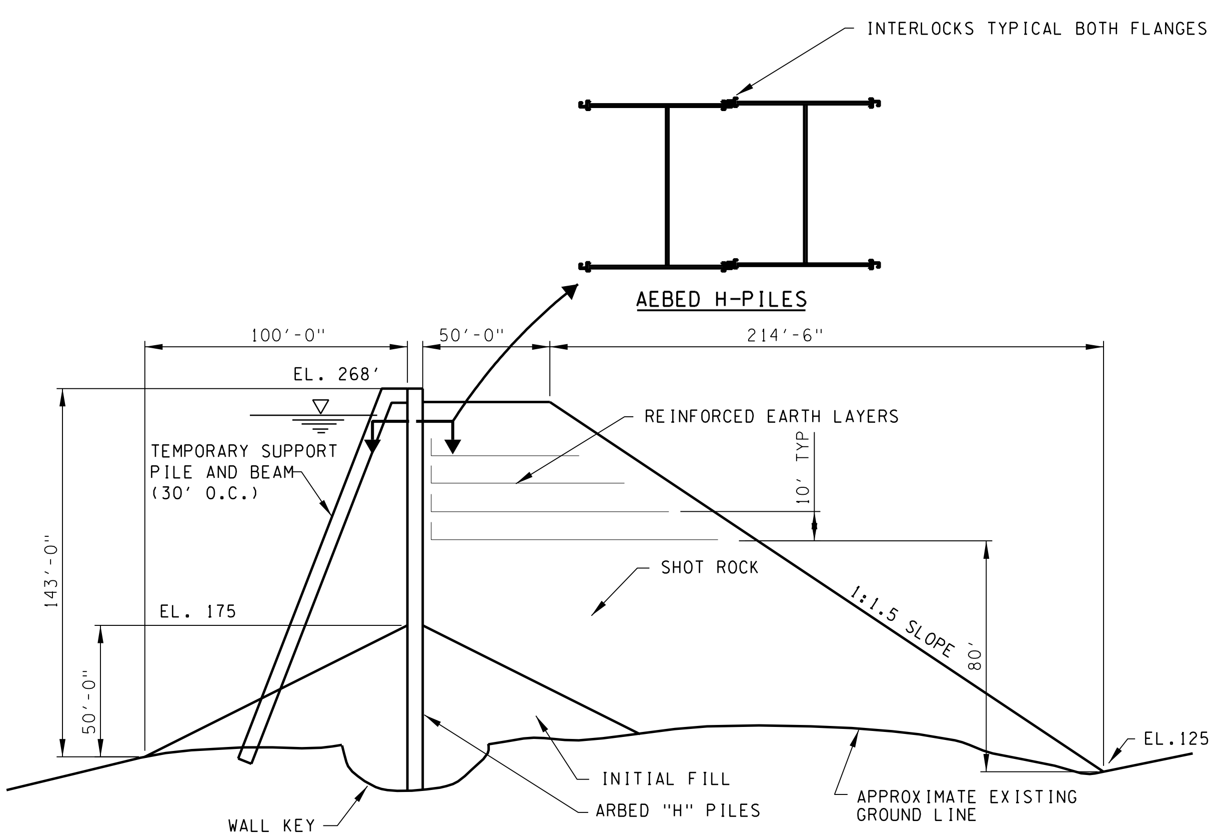
NAVIGATION CHANNEL
NAVIGATION LOCK
EMBANKMENT
NON-OVER FLOW MONOLITH
SPILLWAY
REMOVE POWERHOUSE AND
NON-OVERFLOW DAM TO EL. 128.0
FISH PASSAGE BAFFLES
30' SPACING
FLOW CONTROL
BERM (TOP OF
BERM EL. 200.0)
POWERHOUSE
NORTH SHORE
FISH LADDER
(ABANDON)
REMOVE SPILLWAY
TO EL. 135.0
ROCK AND RUBBLE FILL
SOUTH SHORE
FISH LADDER
(ABANDON)
FLOW CONTROL
BERM (TOP OF
BERM EL. 195.0)
FLOW
THALWEG
EXISTING JUVENILE FACILITIES
TO BE ABANDONED
PLAN
1:300
US Army Corps of Engineers
Portland District
Designed by: DENNIS DORRATCAQUE
Drawn by: PAUL HUNTER
Checked by: PETER BARTON
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate17.dgn
Technical Manager: MATTHEW HANSON
CH2M-HILL
MONTGOMERY WATSON
JOINT VENTURE
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
PORTLAND, OREGON
COLUMBIA RIVER
OREGON - WASHINGTON
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 3
DAM REMOVAL PLAN
DRAWING STATUS:
DRAWING NO.
PLATE
17



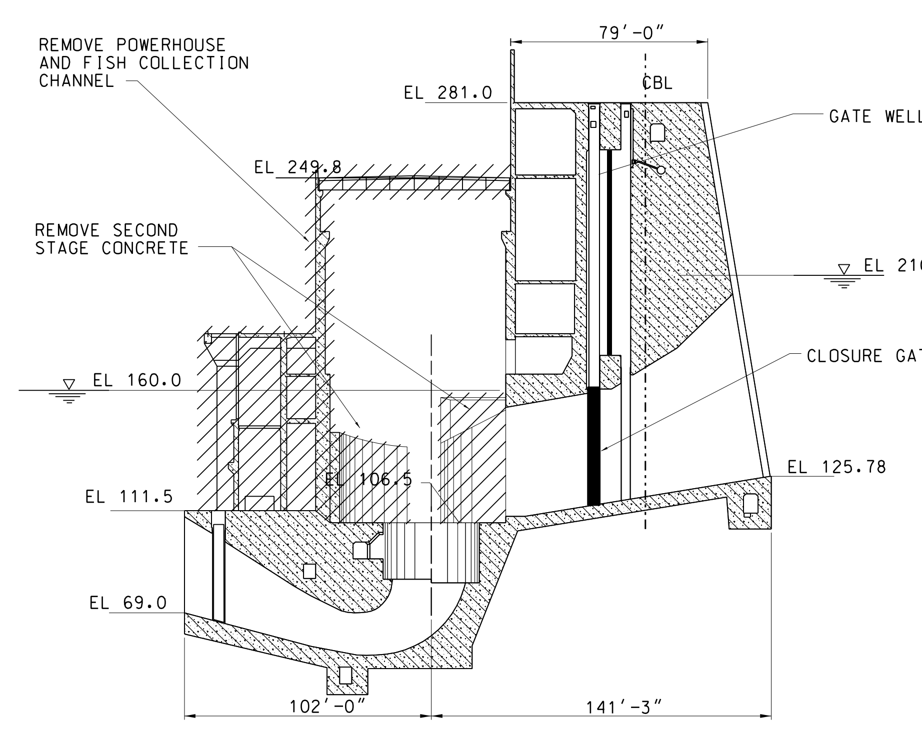
STAGE 1 CONSTRUCTION PLAN



STAGE 2 CONSTRUCTION PLAN



SECTION A
NTS



POWERHOUSE - CROSS-SECTION
STAGE 1 CONSTRUCTION AT
UNITS 1 THROUGH 4

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Portland District

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Date: 20 JULY 1999
CADD File Name: plate18.dgn
Technical Manager: MATTHEW HANSON

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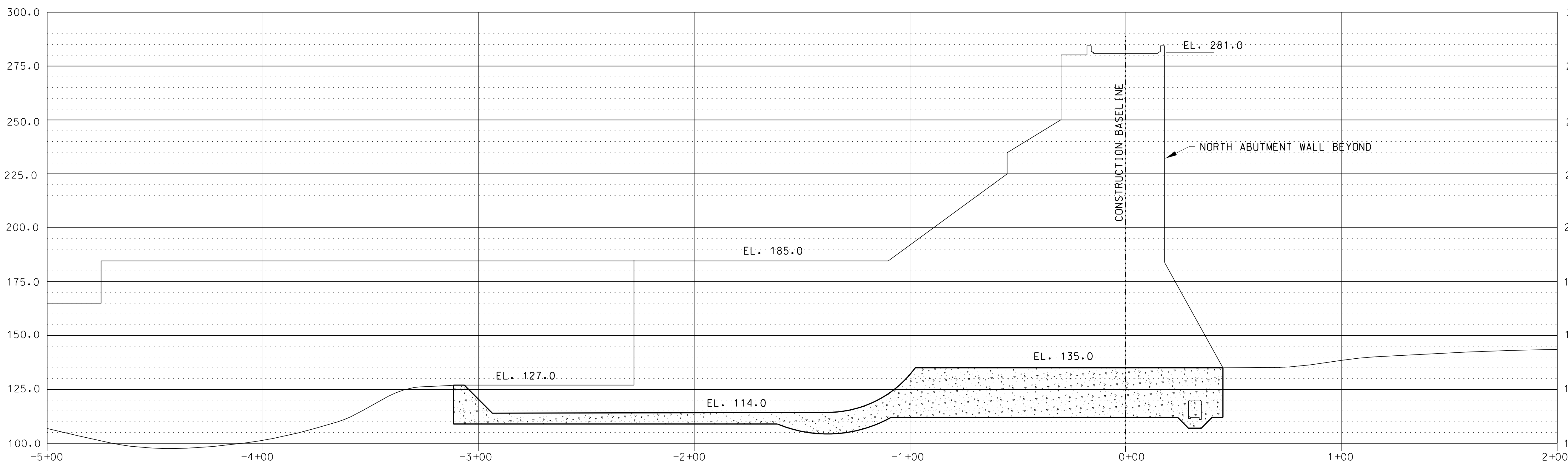
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 3
COFFERDAM PLAN AND SECTION

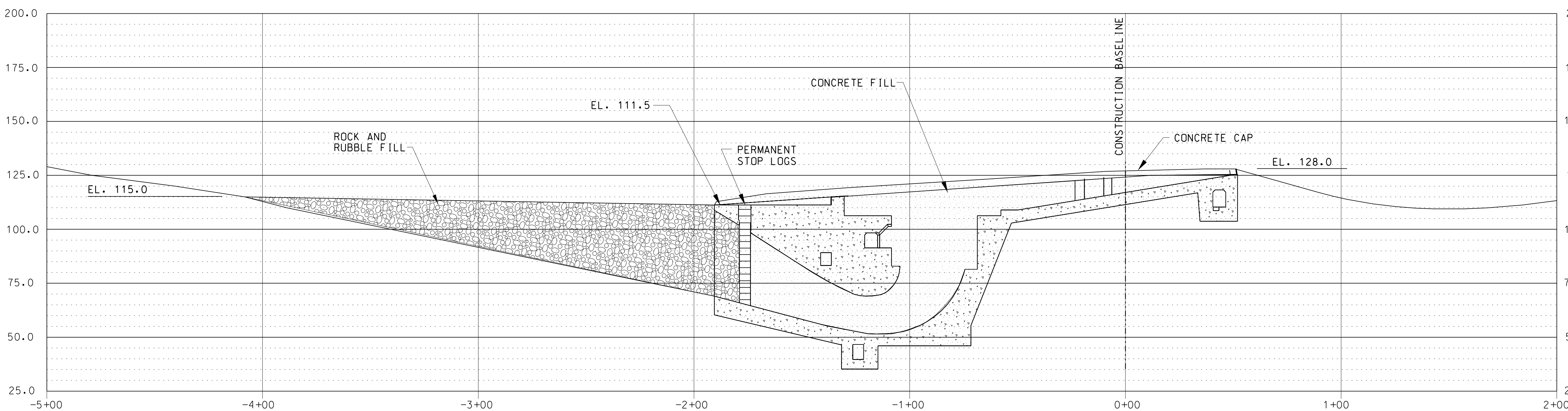
DRAWING STATUS:

DRAWING NO.

PLATE
18



PROFILE THROUGH SPILLWAY BAY 18
(STATIONING FROM CBL)



PROFILE THROUGH SKELETON UNIT 18
(STATIONING FROM CBL)

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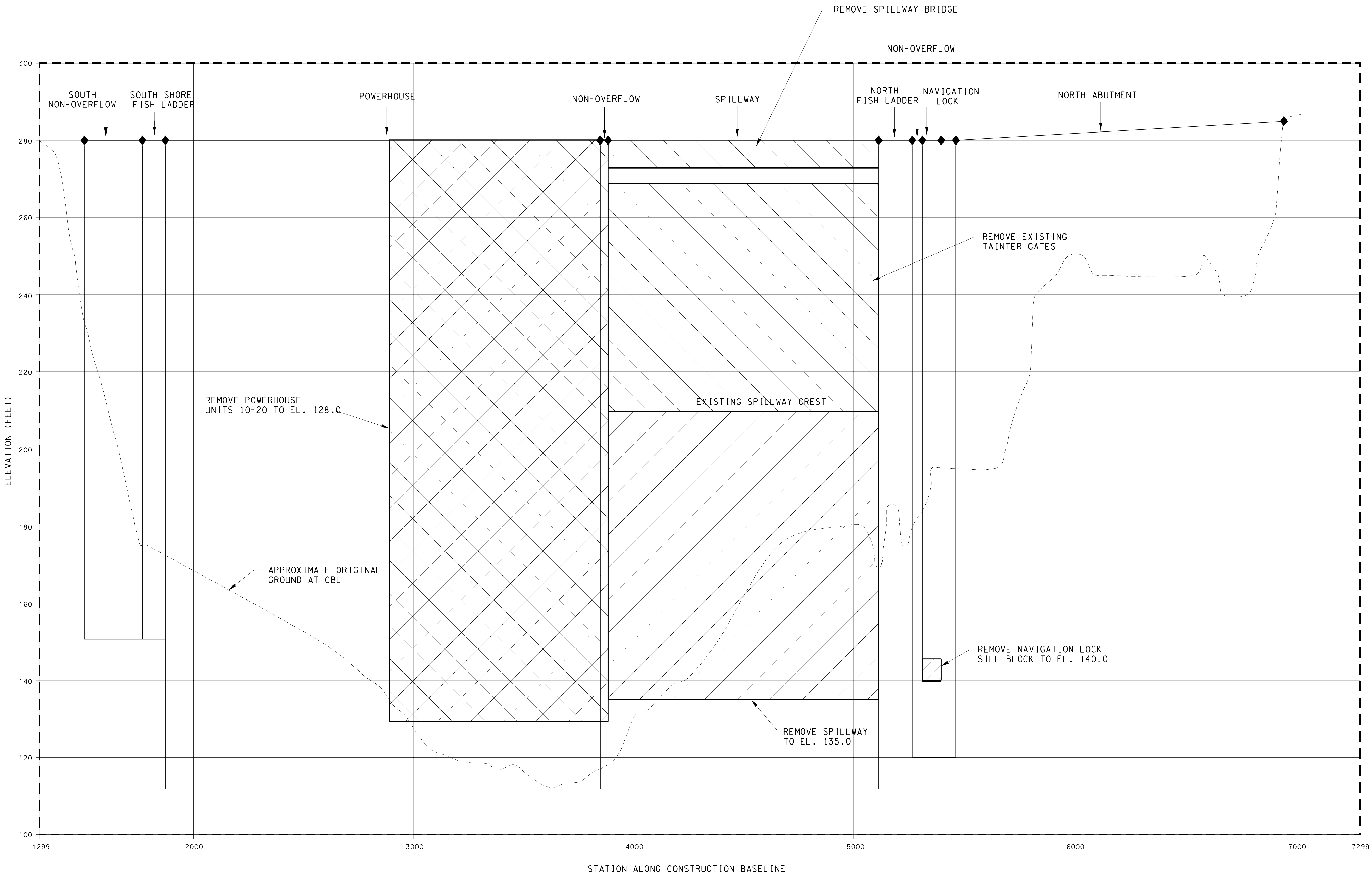
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 3
DAM REMOVAL PROFILE

DRAWING STATUS:

DRAWING NO.

PLATE
19



US Army Corps of Engineers
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xxx

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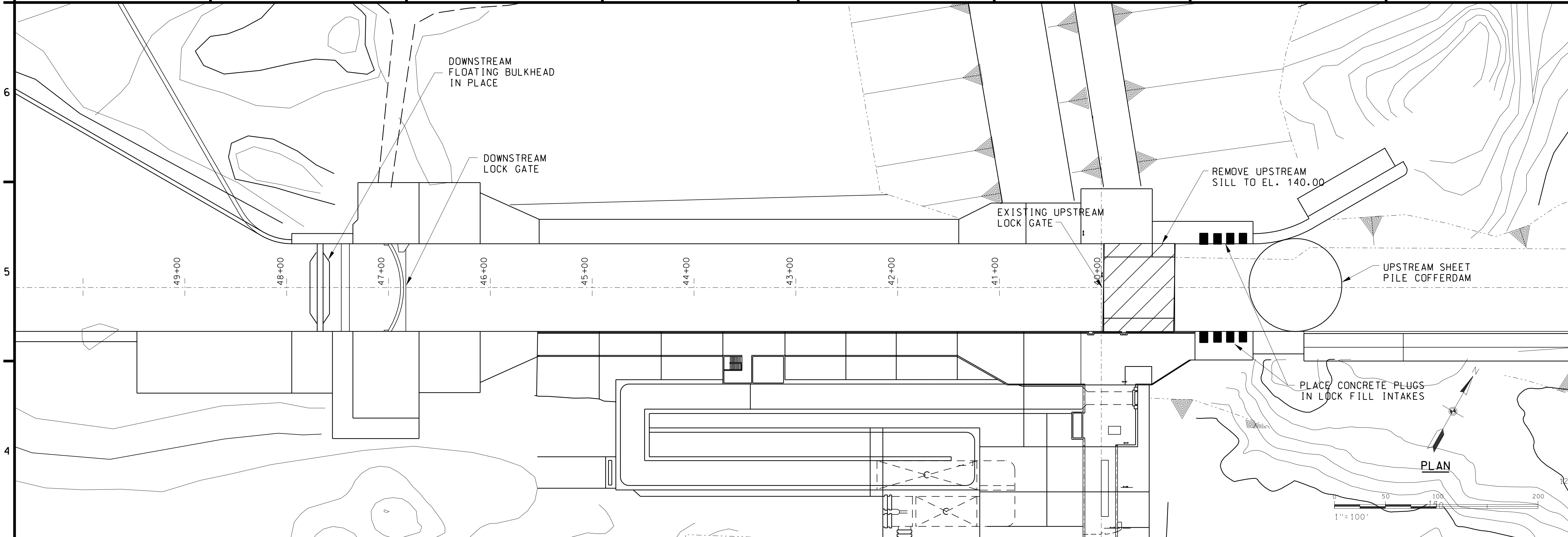
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 3
DAM REMOVAL UPSTREAM ELEVATION

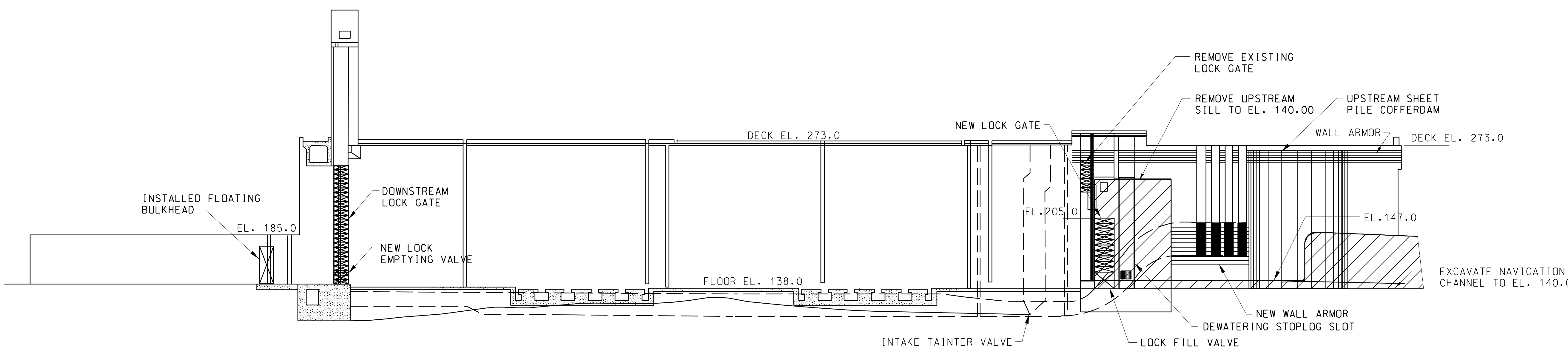
DRAWING STATUS:

DRAWING NO.

PLATE
20



PLAN



SECTION A

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Portland District

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Date: 20 JULY 1999
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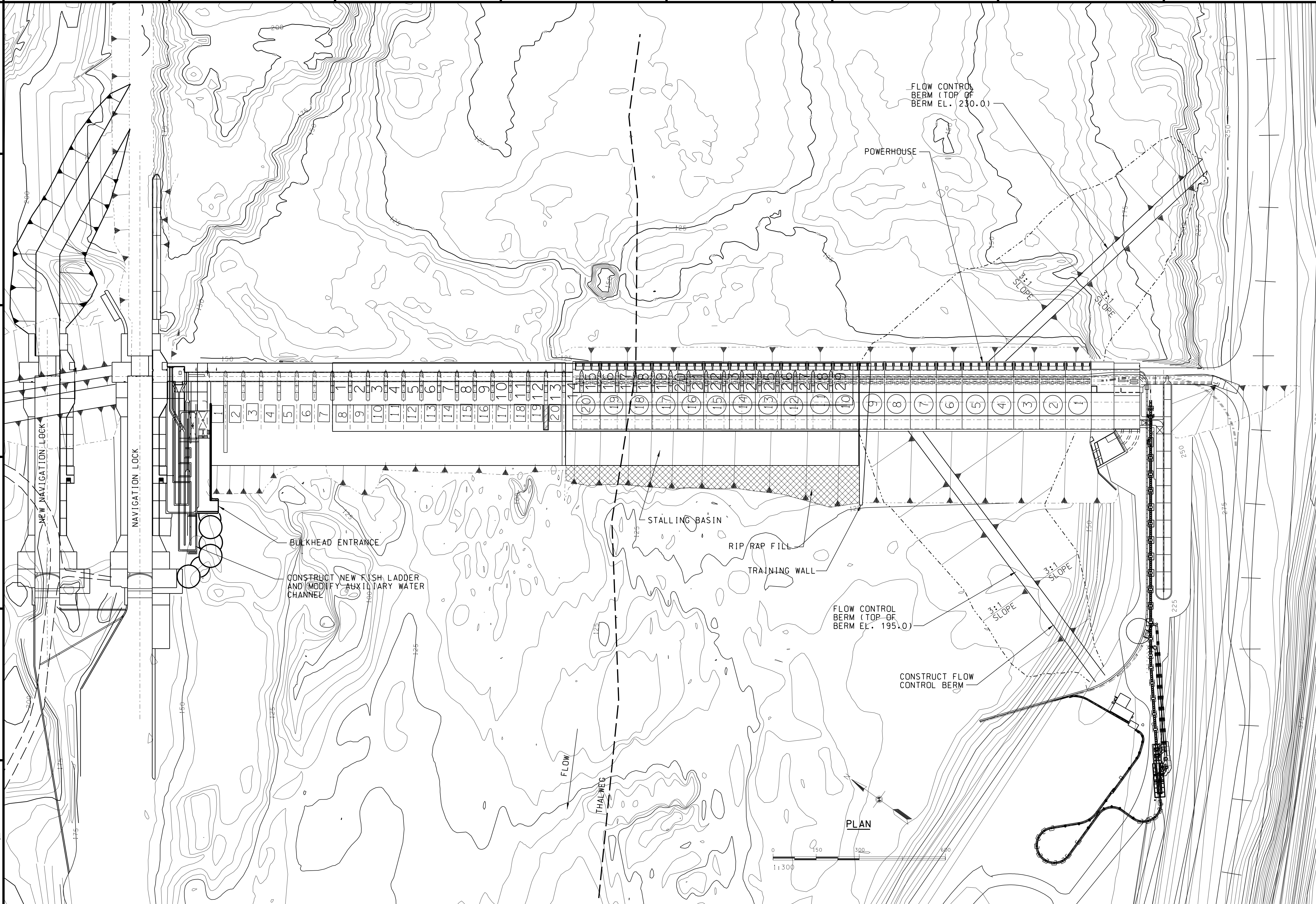
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO.3
NAVIGATION LOCK

DRAWING STATUS:

DRAWING NO.

PLATE
21



US Army Corps
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Portland District

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Date: 20 JULY 1999
CADD File Name: plate22.dgn
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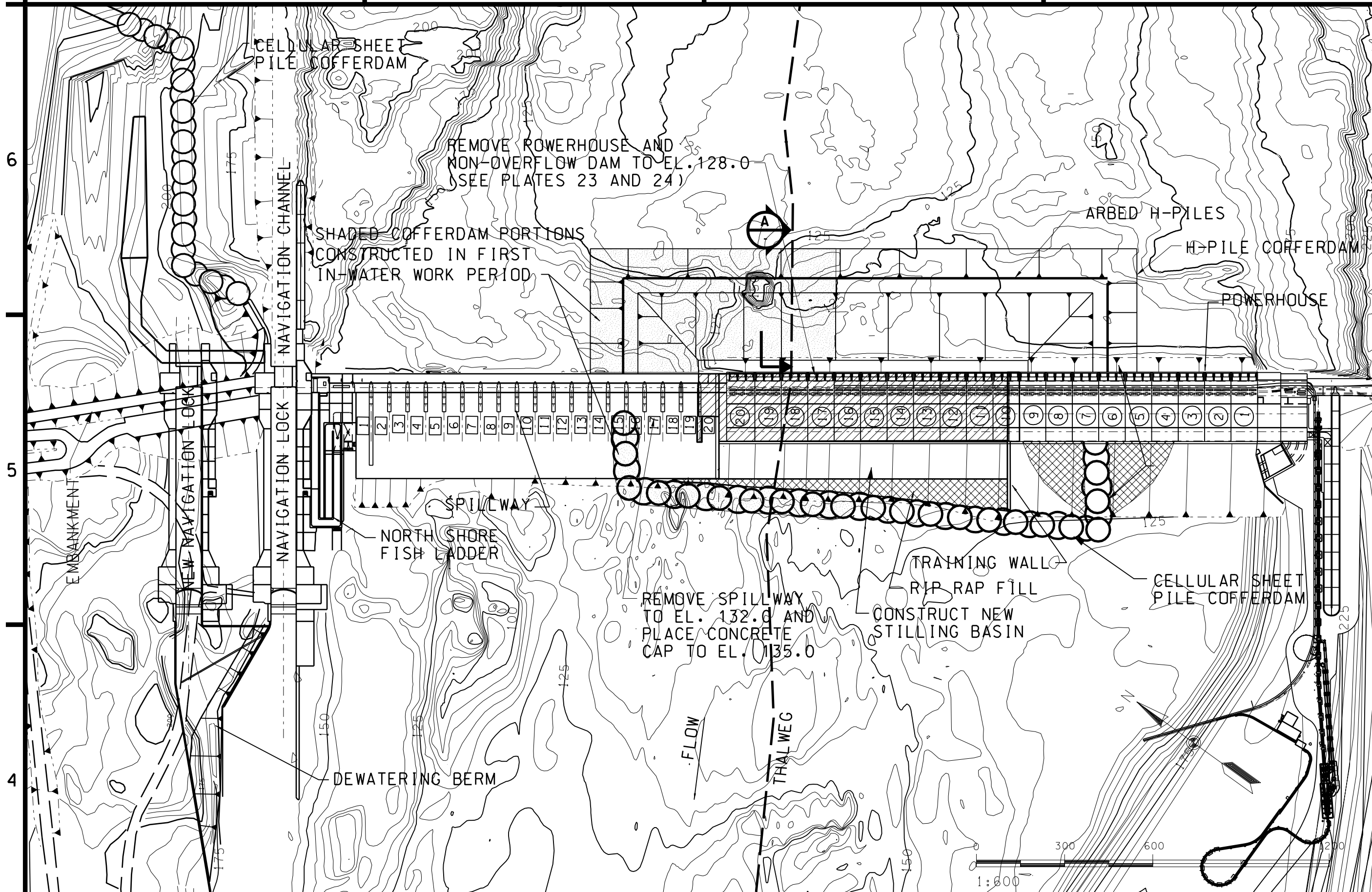
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 4
DAM REMOVAL PLAN

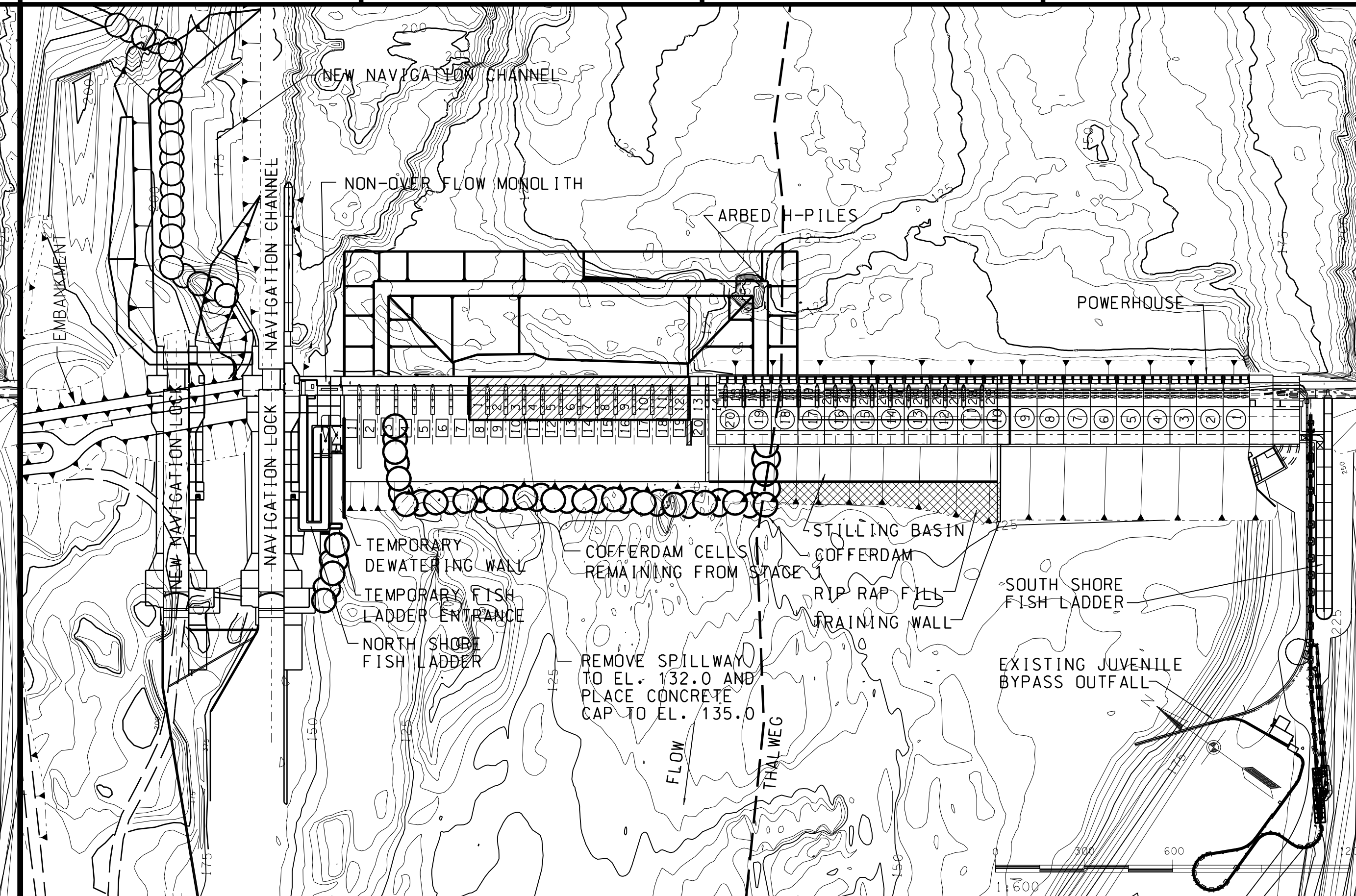
DRAWING STATUS:

DRAWING NO.

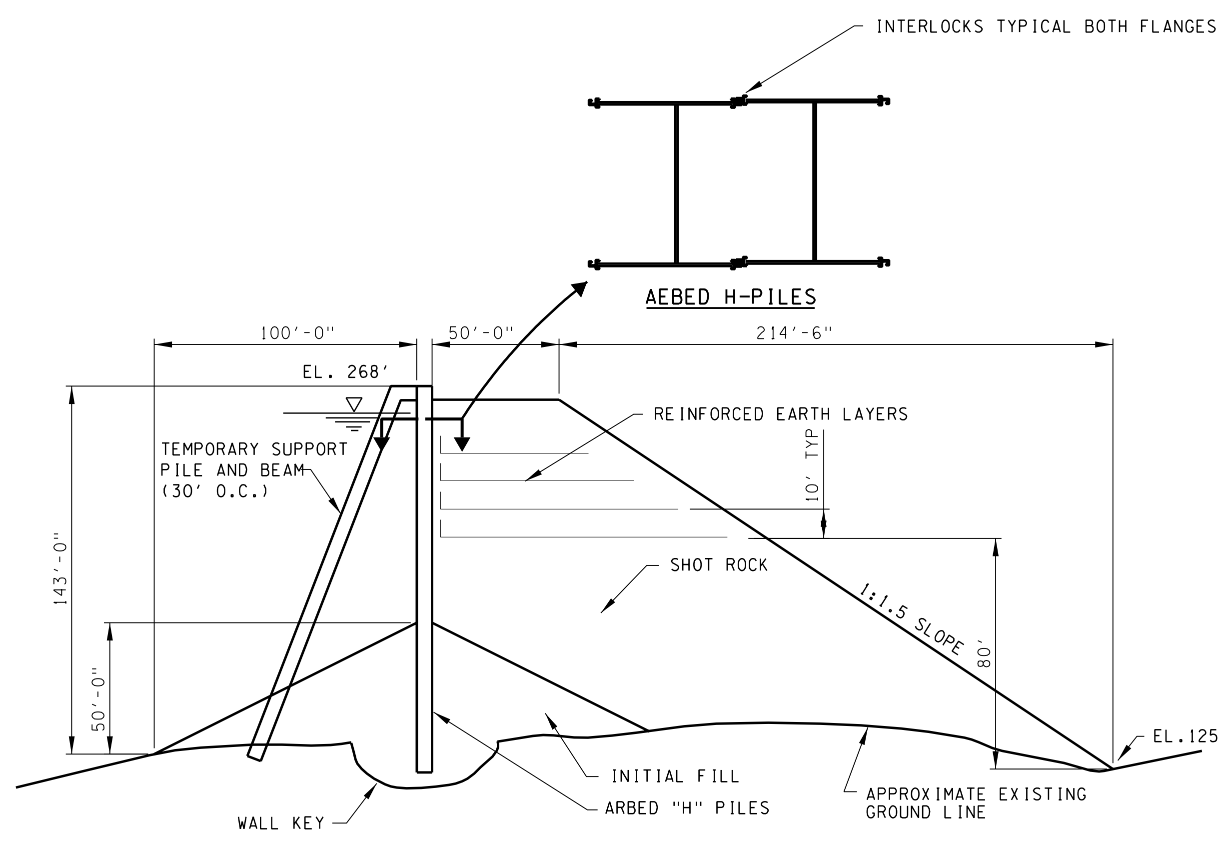
PLATE
22



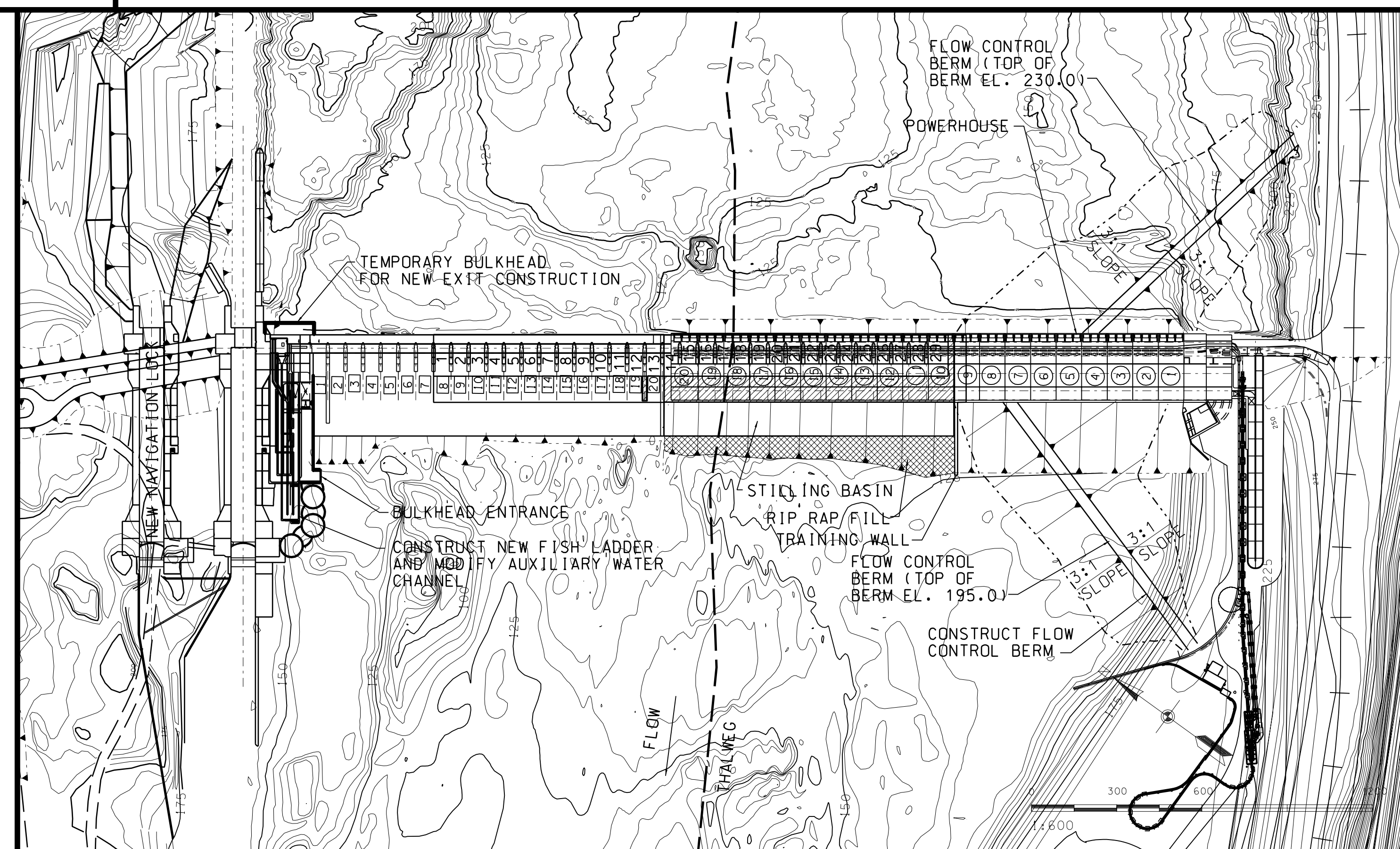
STAGE 1 CONSTRUCTION PLAN



STAGE 2 CONSTRUCTION PLAN



SECTION A
NTS



STAGE 3 CONSTRUCTION PLAN

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Portland District

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Drawn by: PAUL HUNTER
Checked by: PETER BARTON
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate23.dgn
Technical Manager: MATTHEW HANSON

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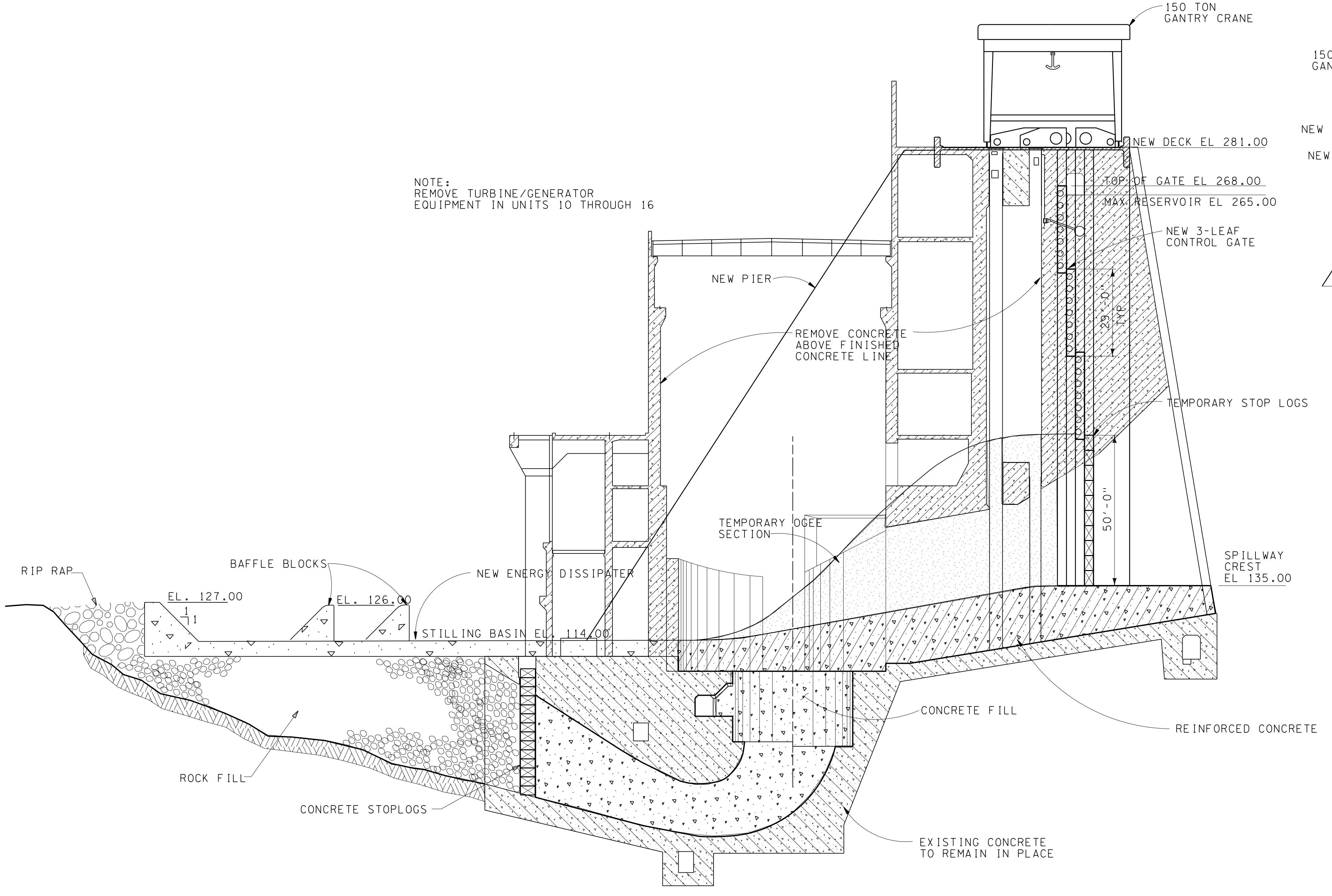
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 4
COFFERDAM PLAN & SECTION

DRAWING STATUS:

DRAWING NO.

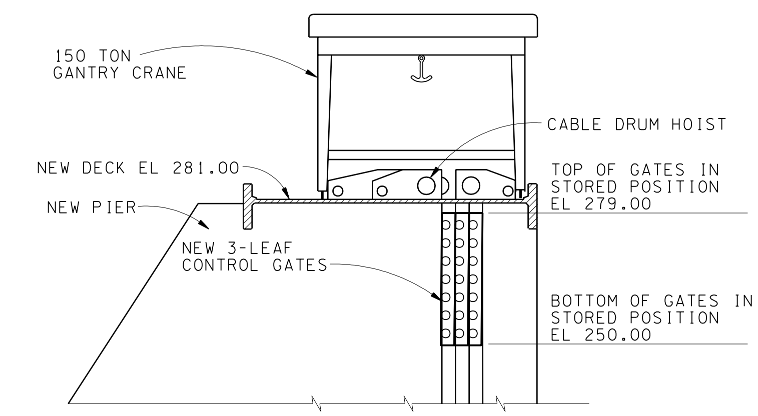
PLATE
23



POWERHOUSE SECTION

NEW SPILLWAY BAYS 20-29

0 20 40 60
Scale In Feet



SPILLWAY SECTION - CONTROL GATES IN STORED POSITION

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Drawn by: PAUL HUNTER
Checked by: DENNIS DORRATCAQUE
Submitted by: DALE S MAZAR, P.E. CHIEF DESIGN BRANCH
Date: 20 JULY 1999
CADD File Name: plate24.dgn
Technical Manager: MATTHEW HANSON

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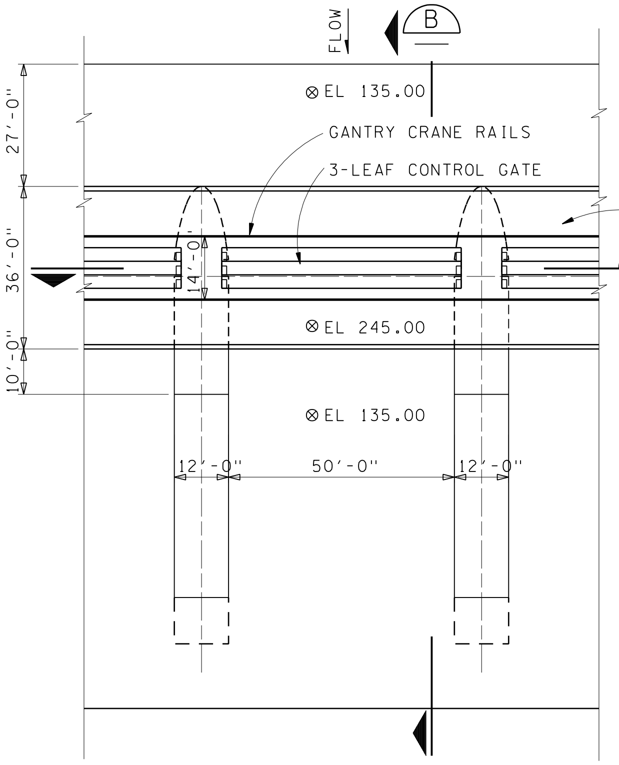
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 4
SPILLWAY AND POWERHOUSE
SECTIONS, STAGE I

DRAWING STATUS:

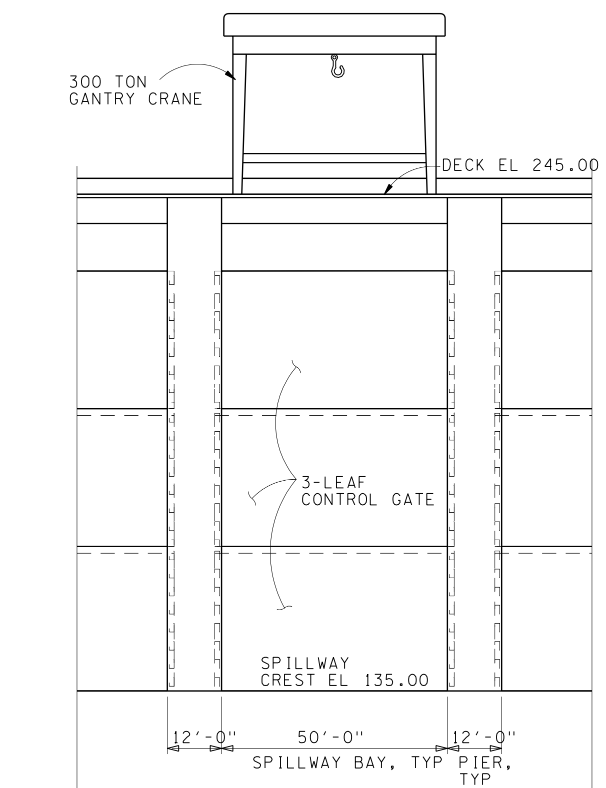
DRAWING NO.

PLATE
24



SPILLWAY PARTIAL PLAN

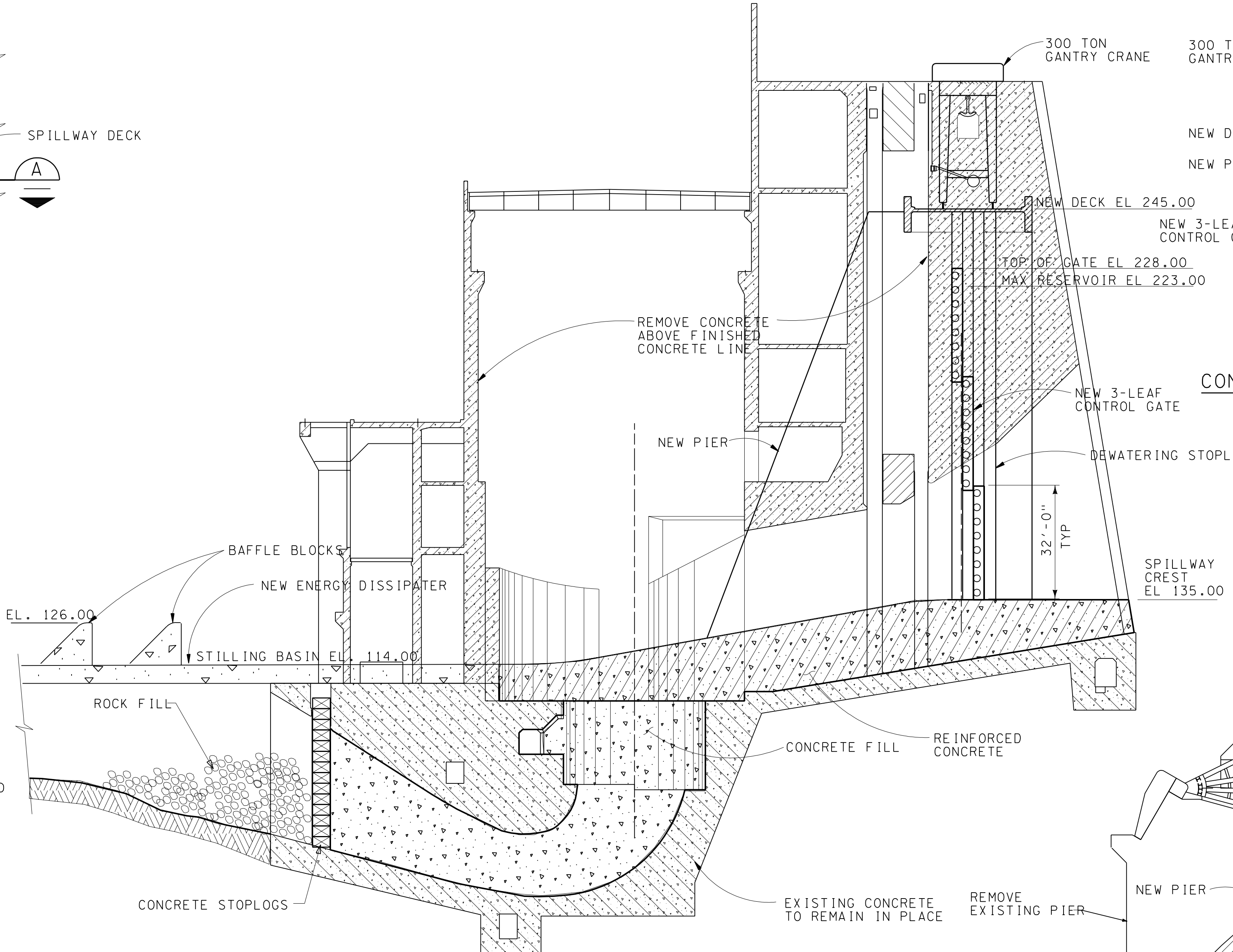
0 20 40 60
Scale In Feet



SPILLWAY

SECTION A 23

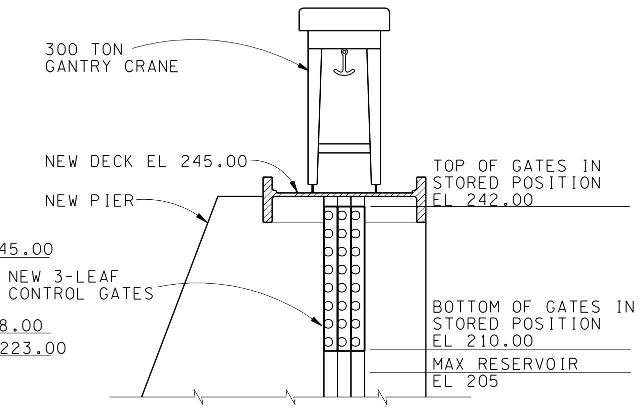
0 20 40 60
Scale In Feet



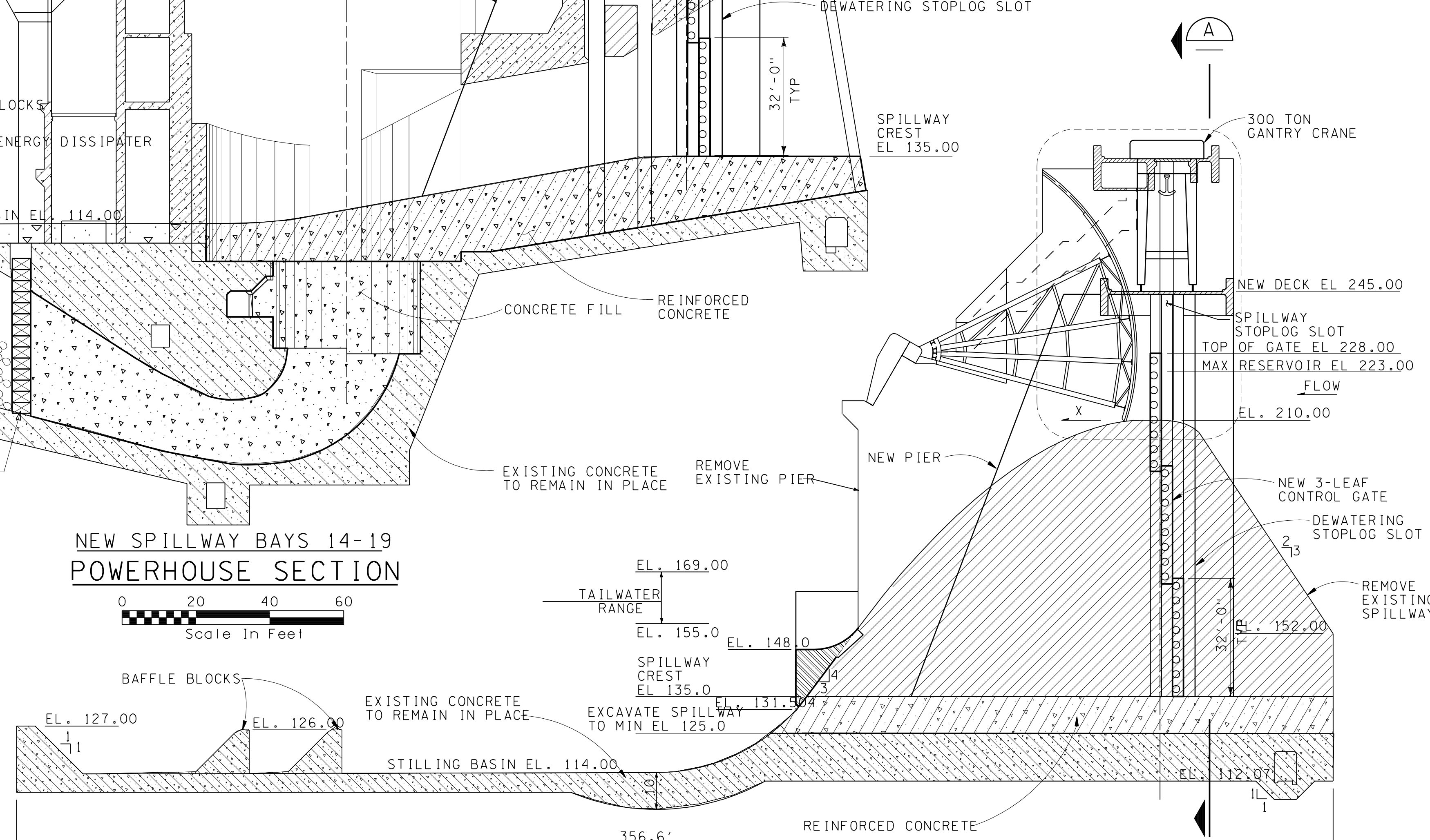
NEW SPILLWAY BAYS 14-19

POWERHOUSE SECTION

0 20 40 60
Scale In Feet



SPILLWAY SECTION -
CONTROL GATES IN STORED POSITION



NEW SPILLWAY BAYS 1-13 - GATES DOWN

STAGE 2 SECTION B

0 20 40 60
Scale In Feet

NOTE:
REMOVE EXISTING RADIAL
GATES, HOISTING EQUIPMENT AND
ALL CONCRETE ABOVE EL 132.00.

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Portland District

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Date: 20 JULY 1999
CADD File Name: plate25.dgn
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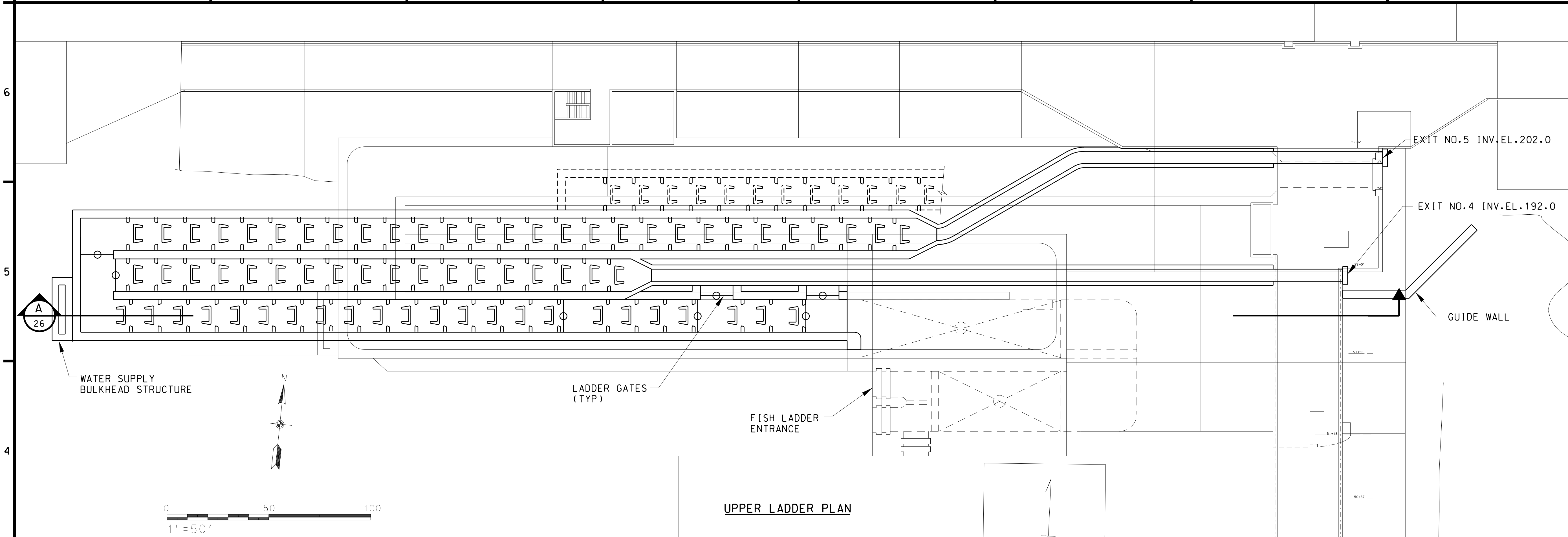
JOHN DAY LOCK AND DAM
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 4
SPILLWAY AND POWERHOUSE
SECTIONS 2

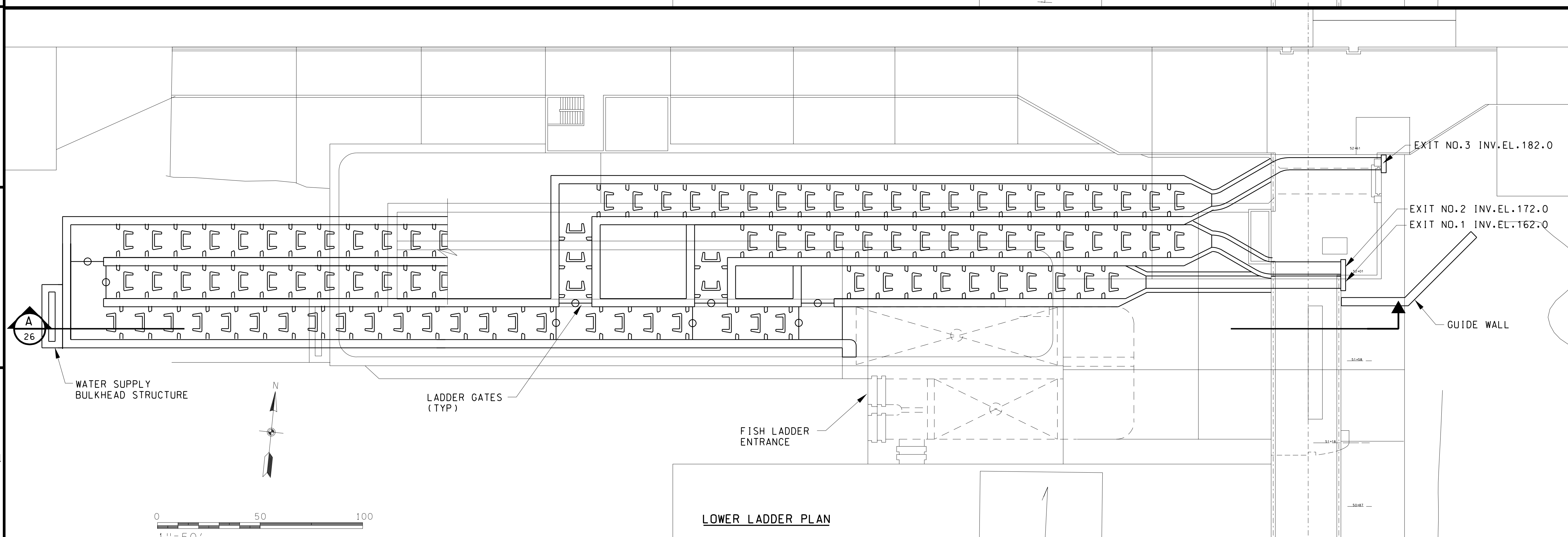
DRAWING STATUS:

DRAWING NO.

PLATE
25



UPPER LADDER PLAN



LOWER LADDER PLAN

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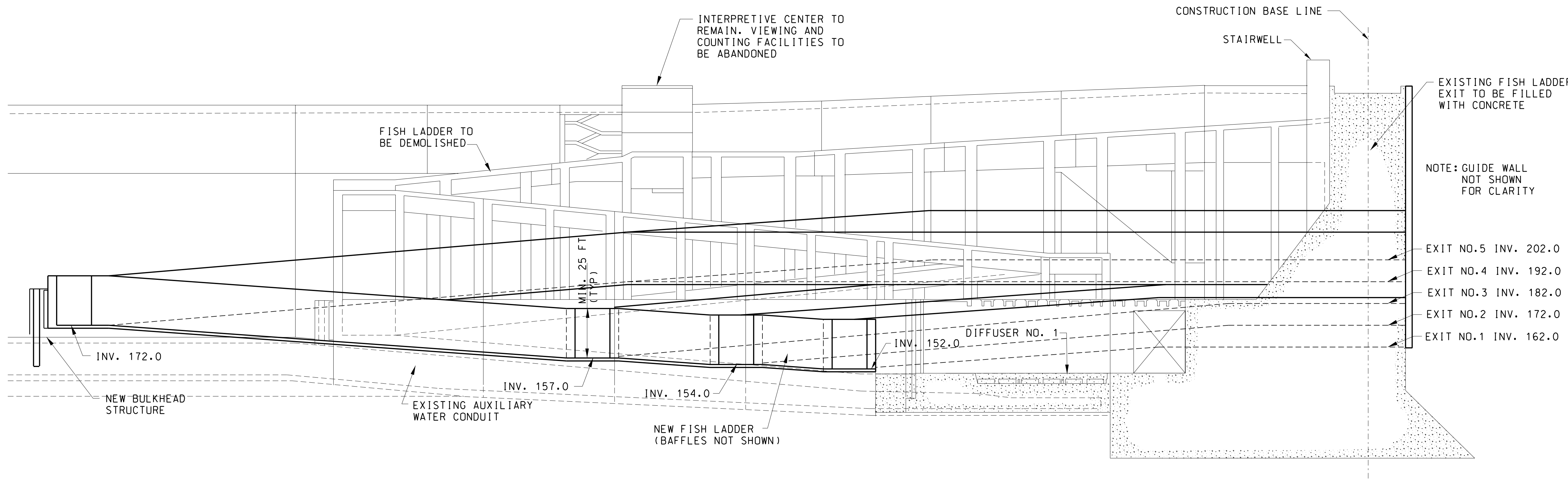
POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 4
NORTH SHORE FISH LADDER PLAN

DRAWING STATUS:

DRAWING NO.

PLATE
26

Plot Date: $$date$$ $$time$$
$$$$$$$$$directory/filename$$$$$$$$$



ELEVATION/SECTION A/22

0 50 100
1"=50'

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POWERHOUSE
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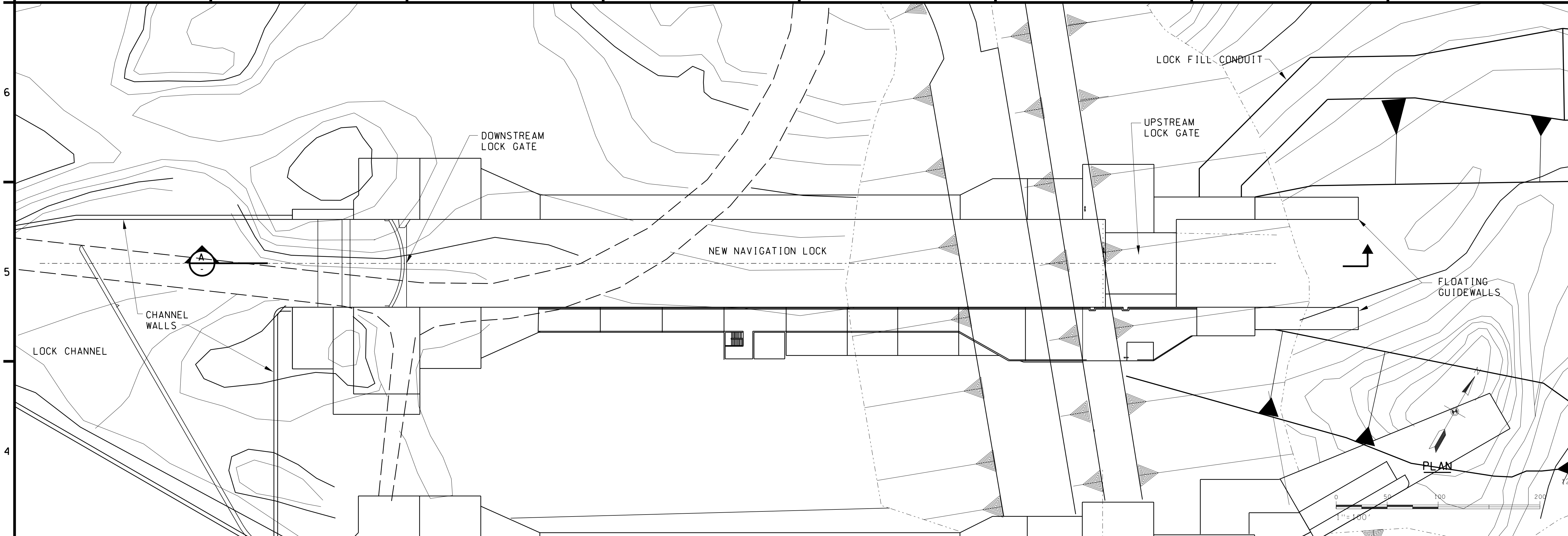
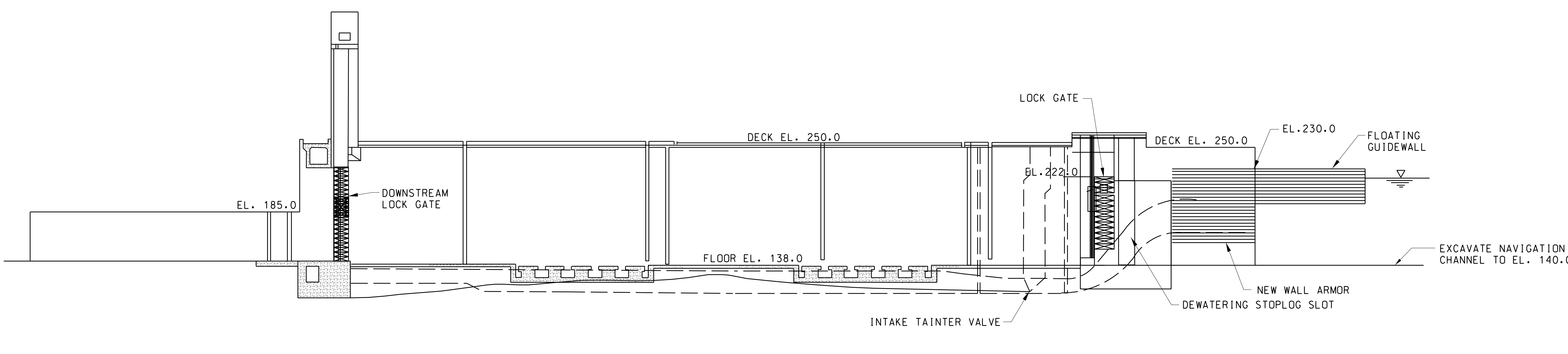
STRUCTURAL ANALYSIS
ALTERNATIVE NO. 4
NORTH SHORE FISH LADDER SECTIONS

DRAWING STATUS:

DRAWING NO.

PLATE
27





SECTION A

0 100 200
1"=100'

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POWERHOUSE
JOHN DAY DRAWDOWN PHASE 1
STRUCTURAL ANALYSIS
ALTERNATIVE NO.4
NAVIGATION LOCK

DRAWING STATUS:

DRAWING NO.

PLATE
28

ATTACHMENT A

HYDROLOGIC AND HYDRAULIC DATA

Figure A-1

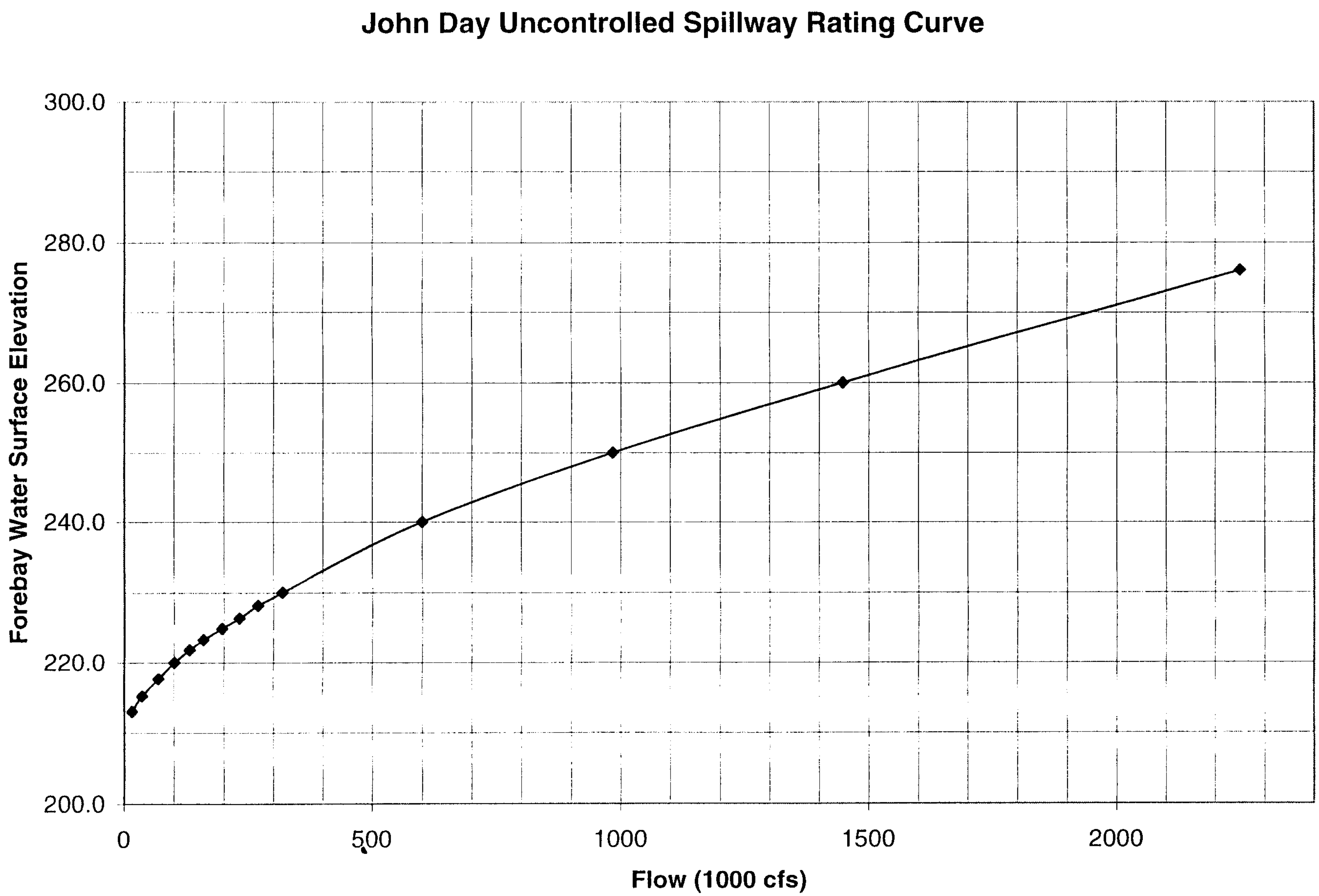

John Day Uncontrolled Spillway Rating Curve
300.0
280.0
260.0
240.0
220.0
200.0
Forebay Water Surface Elevation
0
500
1000
1500
2000
Flow (1000 cfs)

Figure A-2

John Day Uncontrolled Spillway Rating Curve

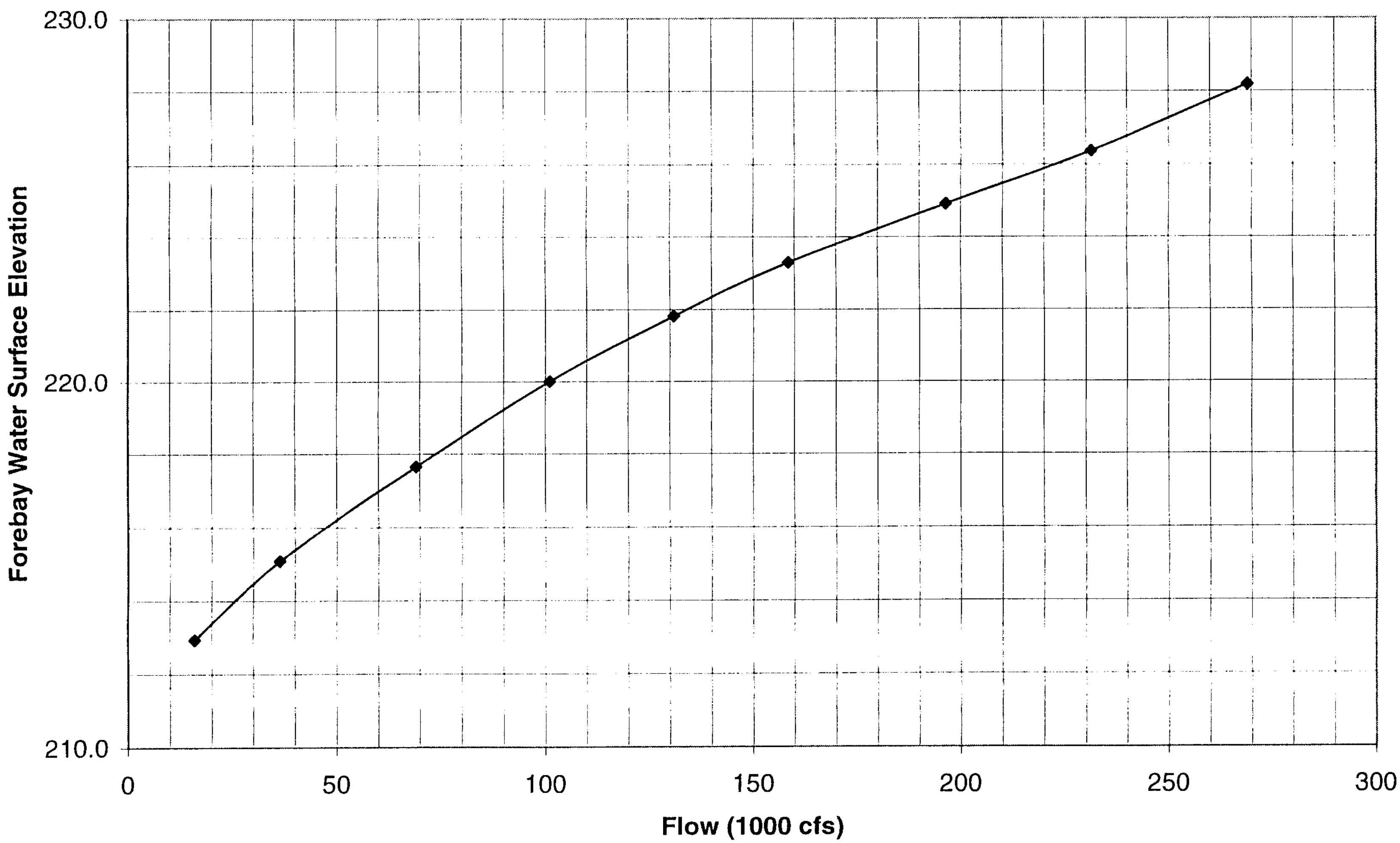


Figure A-3

Forebay Rating Curves at John Day Dam for Natural River without Flood Control

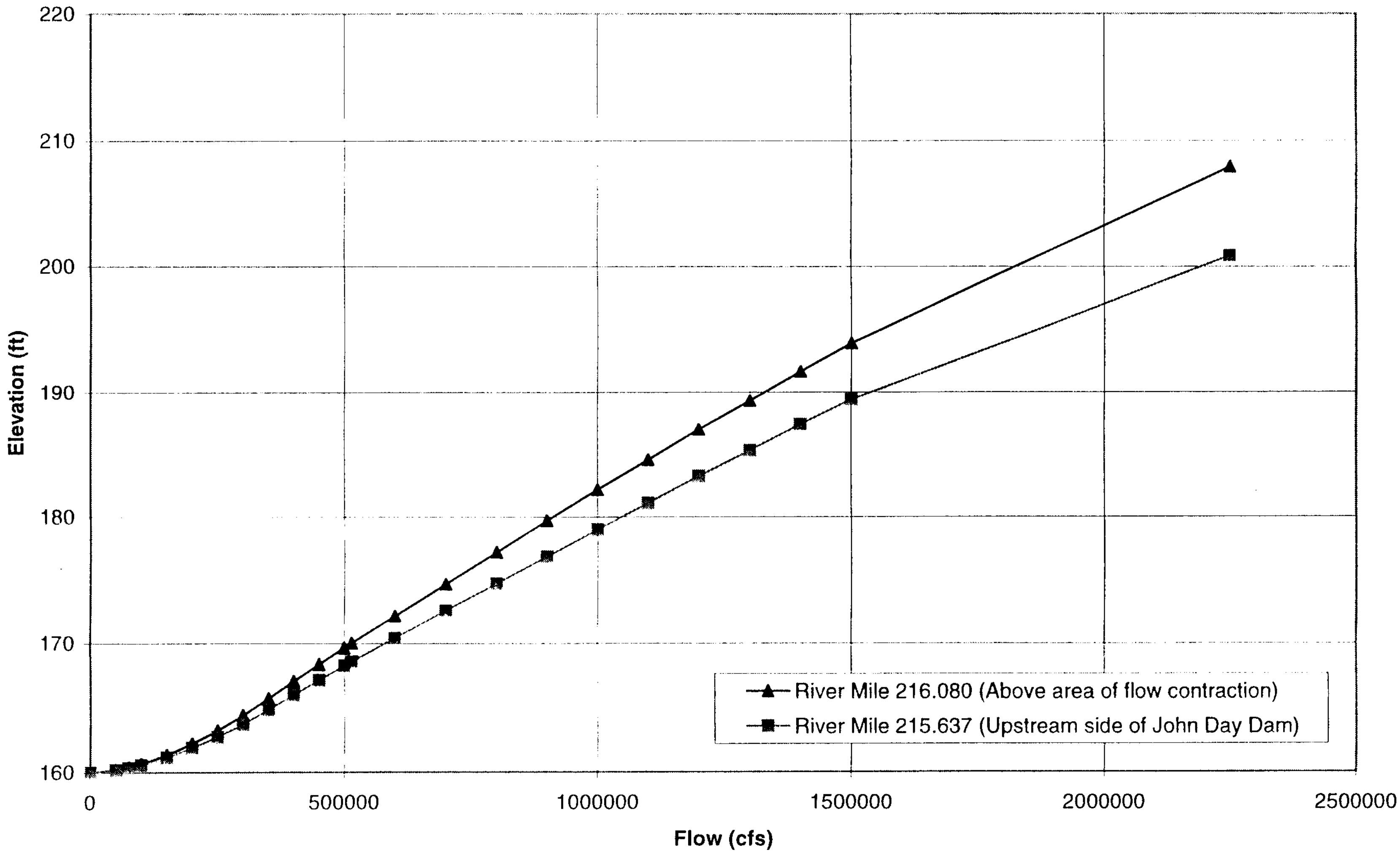


Figure A-4

**Tailwater Rating Curve at John Day Dam Natural River without Flood Control
Station 215.506**

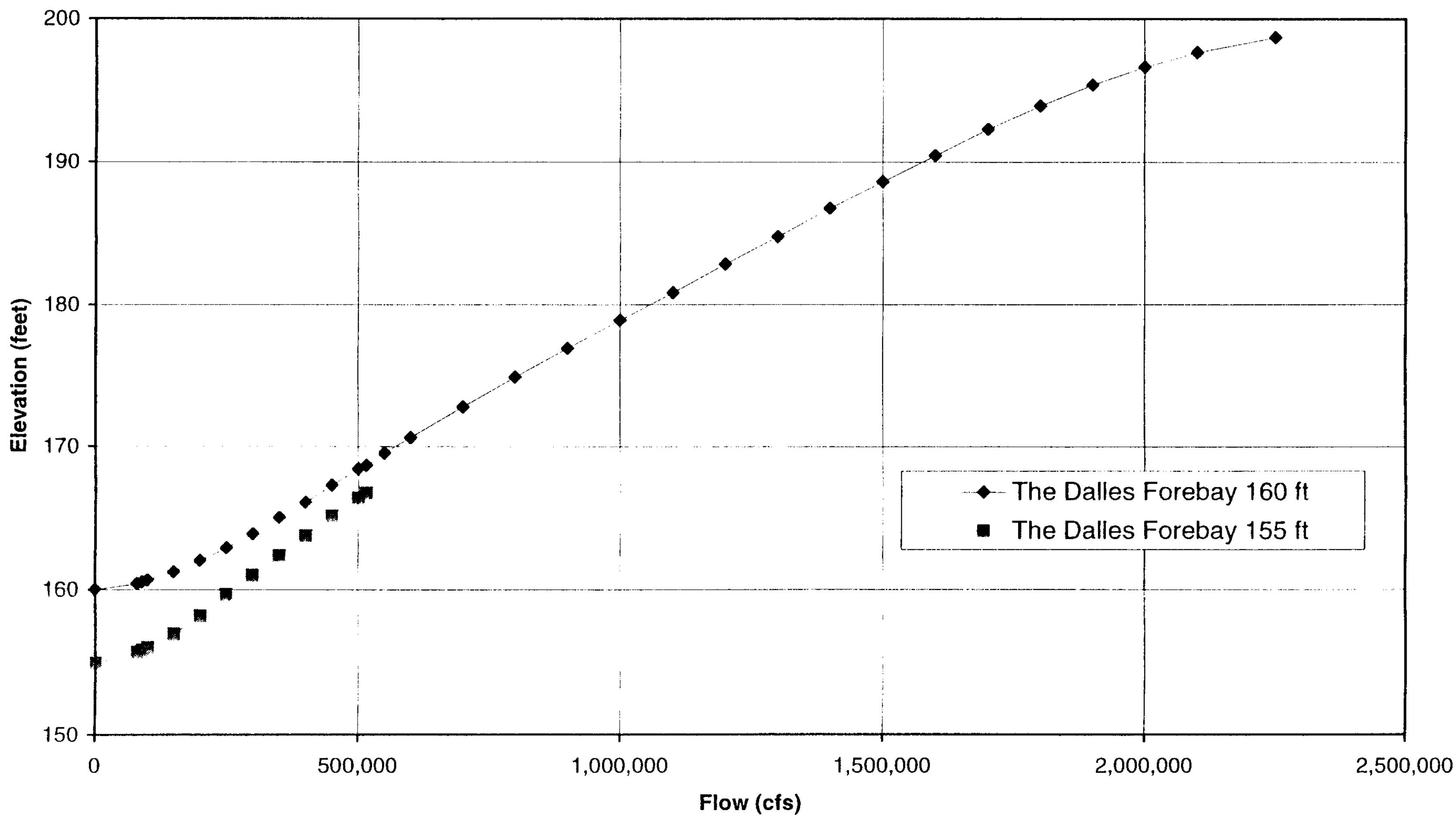
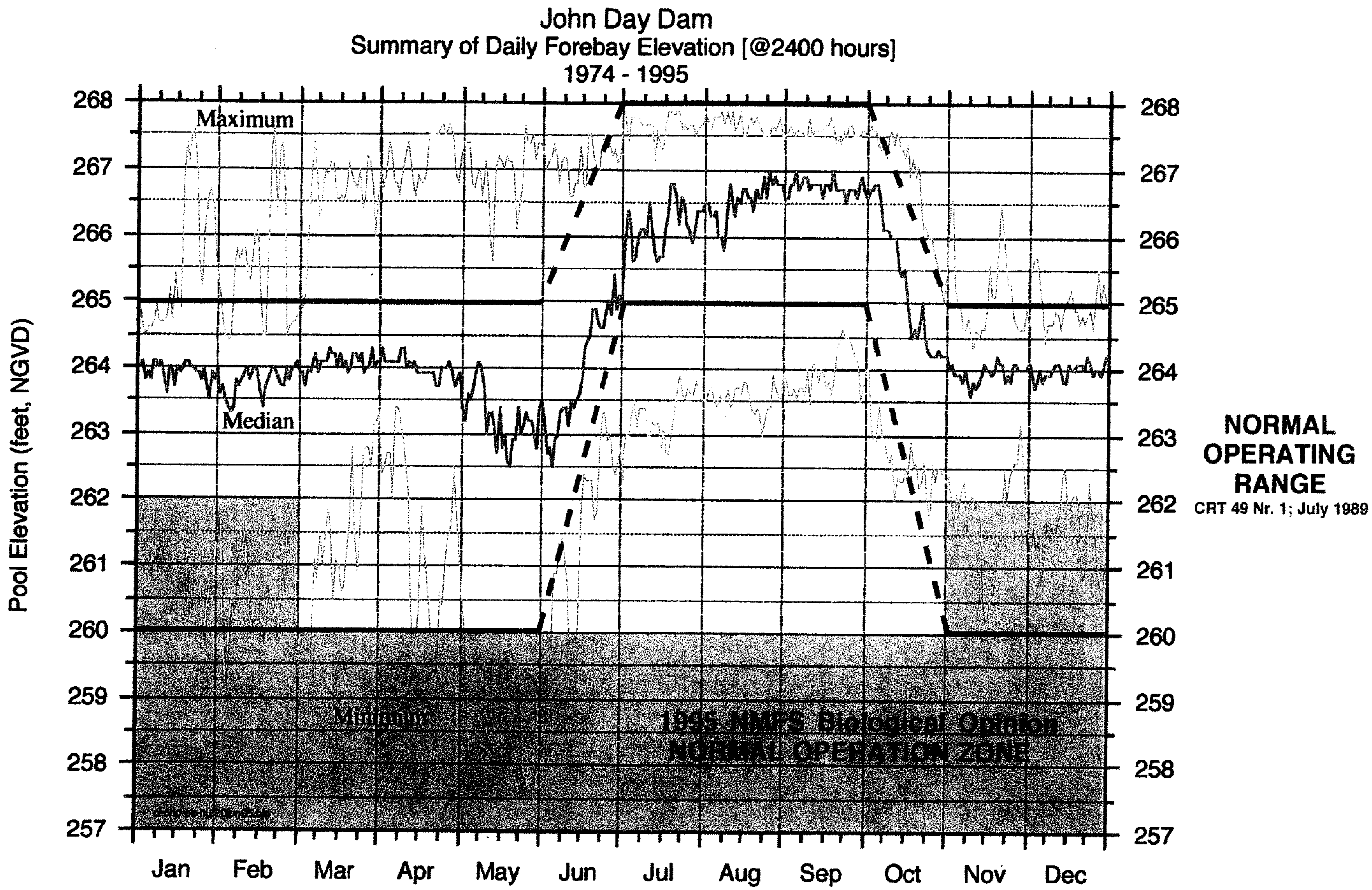


Figure A-5



John Day Dam
Summary of Daily Forebay Elevation [@2400 hours]
1974 - 1995
Pool Elevation (feet, NGVD)
Maximum
Median
Minimum
1995 NMFS Biological Opinion
NORMAL OPERATION ZONE
NORMAL OPERATING RANGE
CRT 49 Nr. 1; July 1989
268
267
266
265
264
263
262
261
260
259
258
257
Jan
Feb
Mar
Apr
May
Jun
Jul
Aug
Sep
Oct
Nov
Dec

Figure A-6

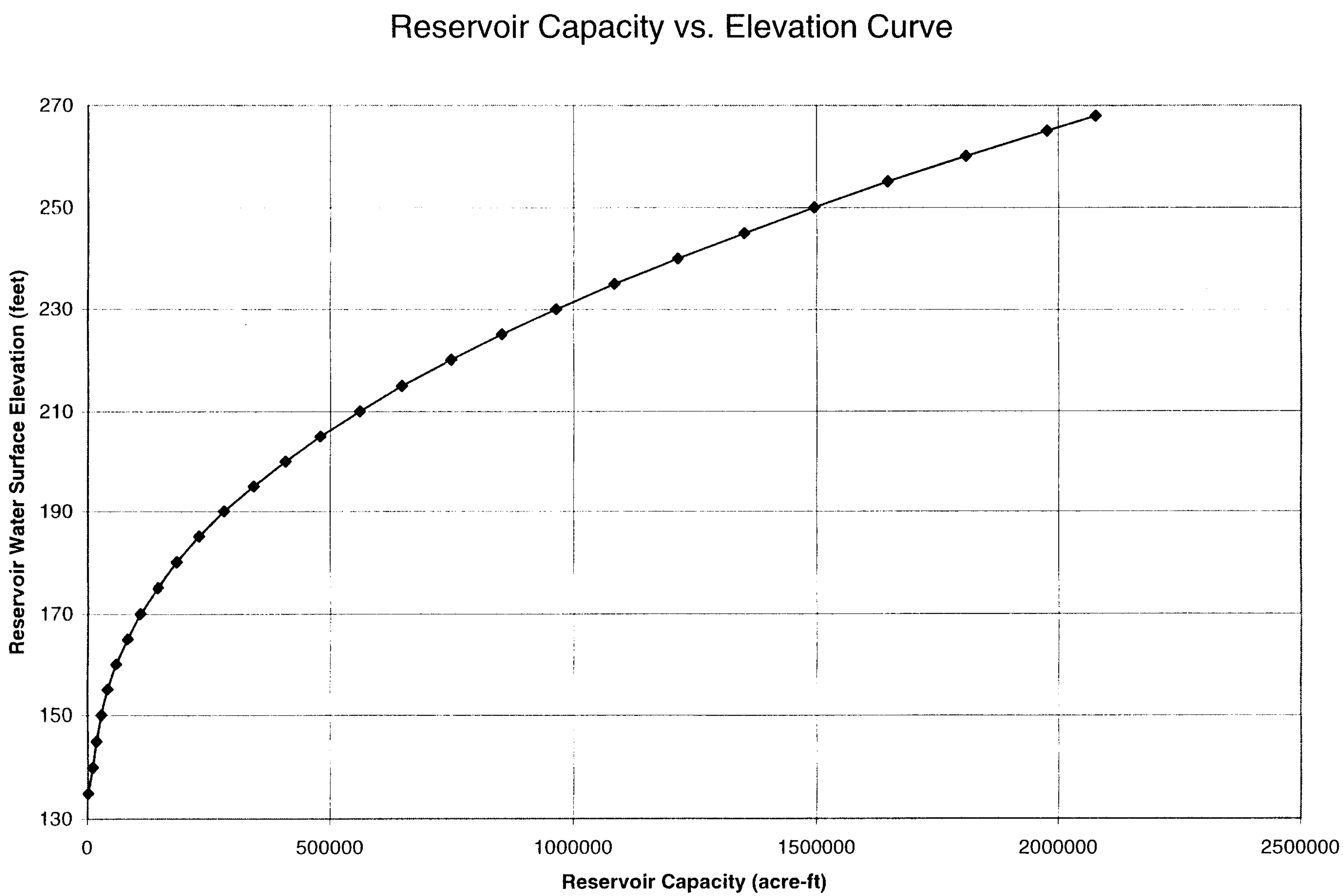

Reservoir Capacity vs. Elevation Curve
270
250
230
210
190
170
150
130
Reservoir Water Surface Elevation (feet)
0
500000
1000000
1500000
2000000
2500000
Reservoir Capacity (acre-ft)

Figure A-7

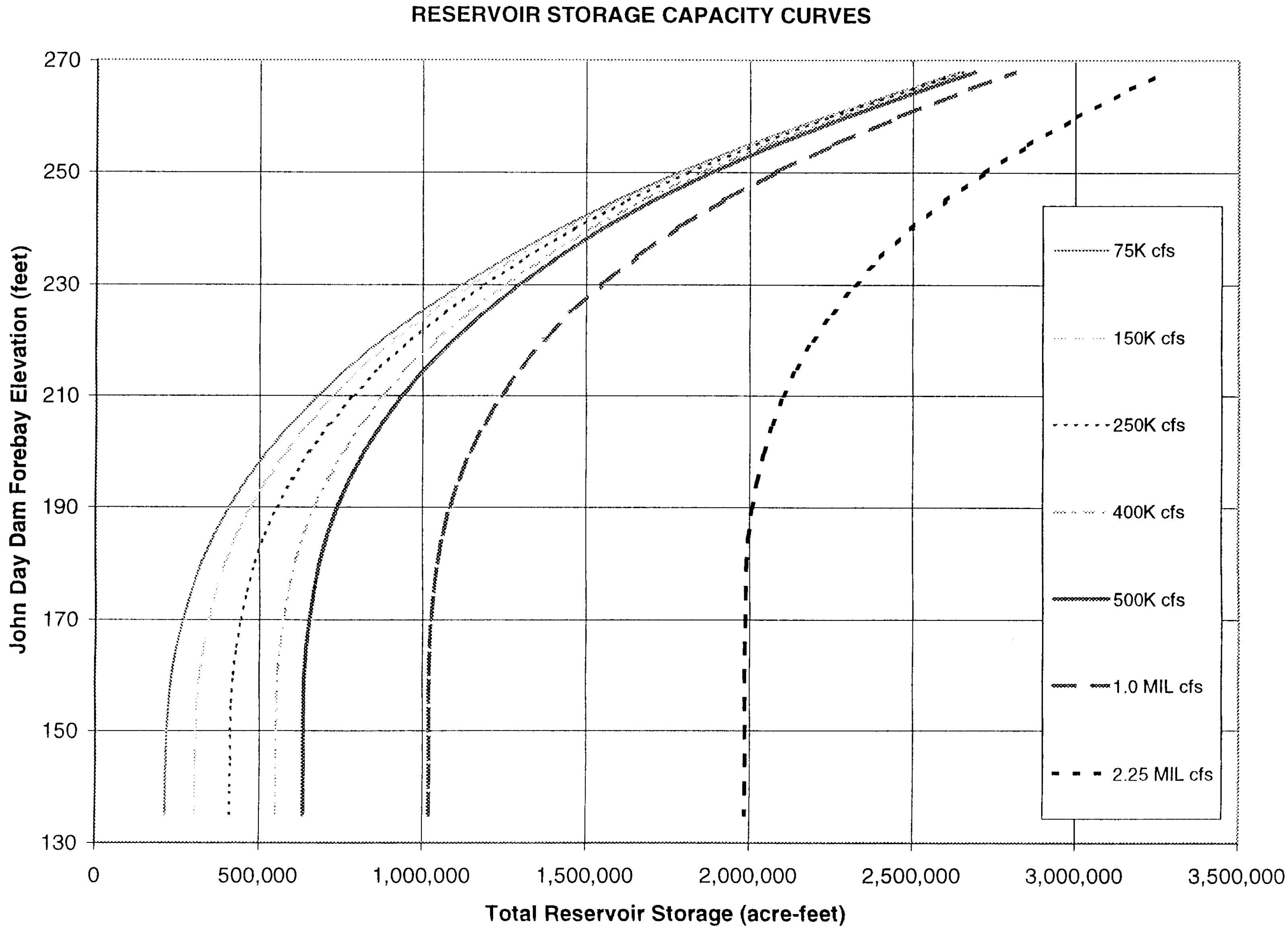

RESERVOIR STORAGE CAPACITY CURVES
270
250
230
210
190
170
150
130
John Day Dam Forebay Elevation (feet)
0
500,000
1,000,000
1,500,000
2,000,000
2,500,000
3,000,000
3,500,000
Total Reservoir Storage (acre-feet)
75K cfs
150K cfs
250K cfs
400K cfs
500K cfs
1.0 MIL cfs
2.25 MIL cfs

Figure A-8

Rating Curve at McNary Dam Tailwater (RM 291.603) for Existing Conditions

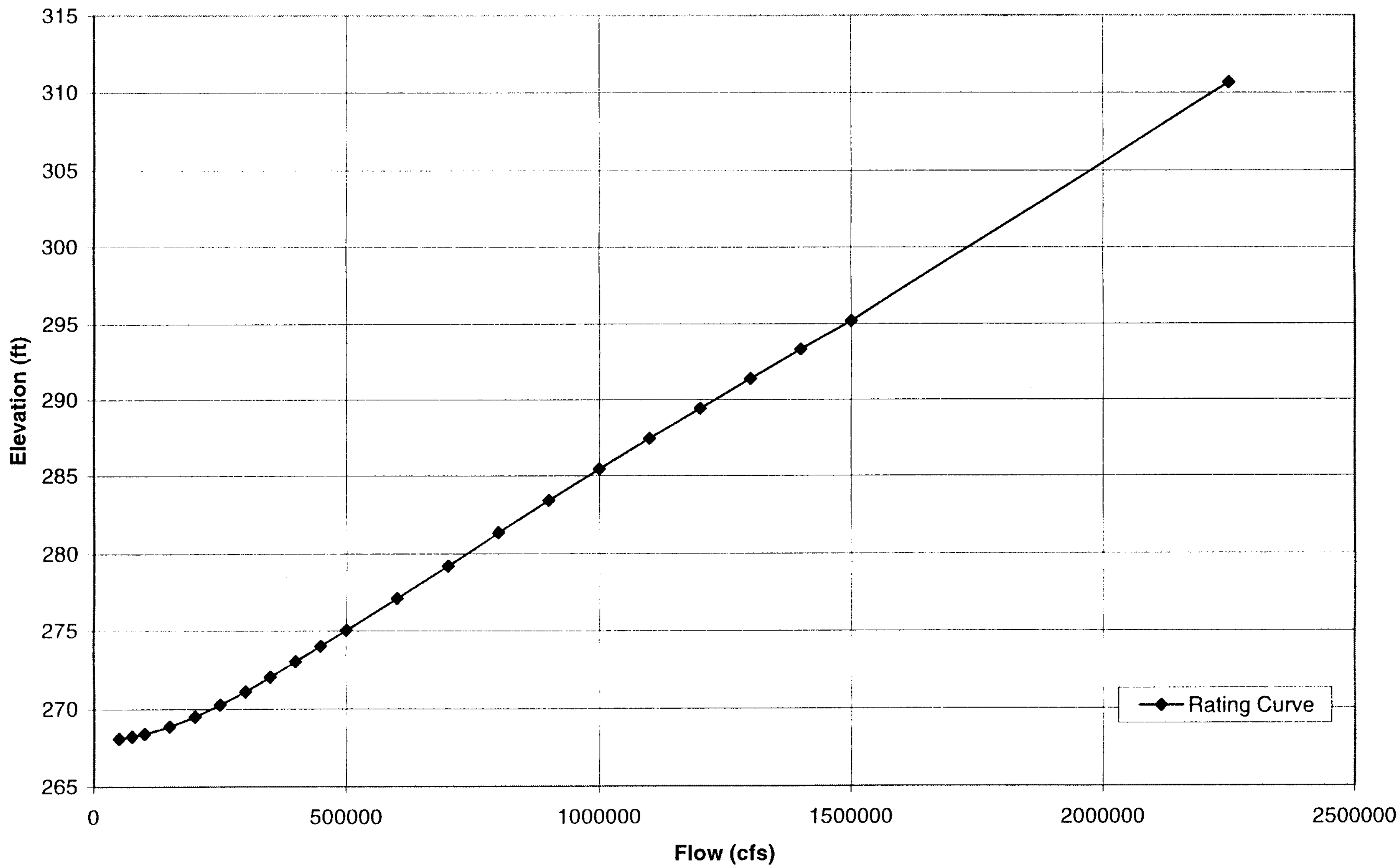


Figure A-9

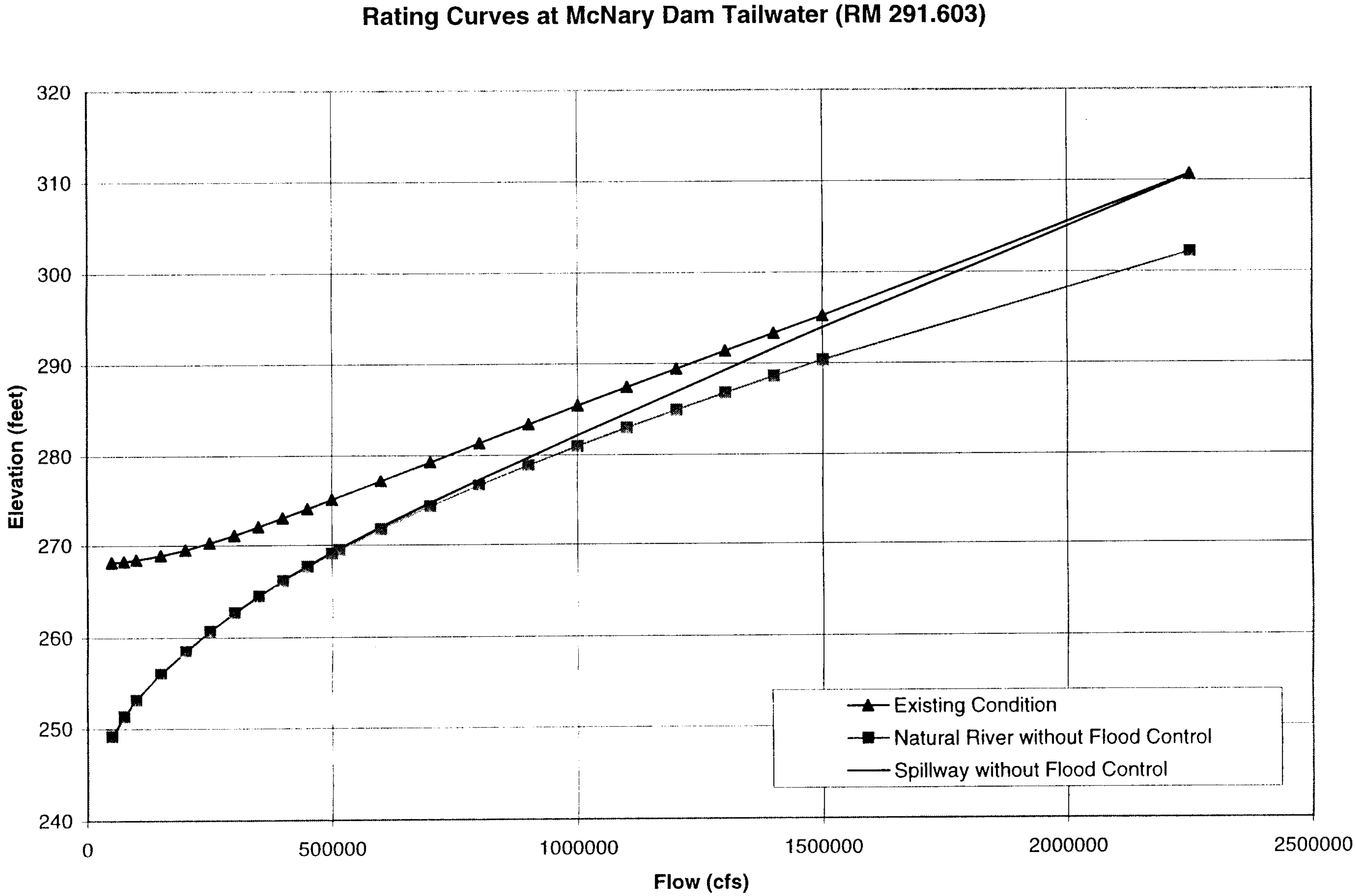
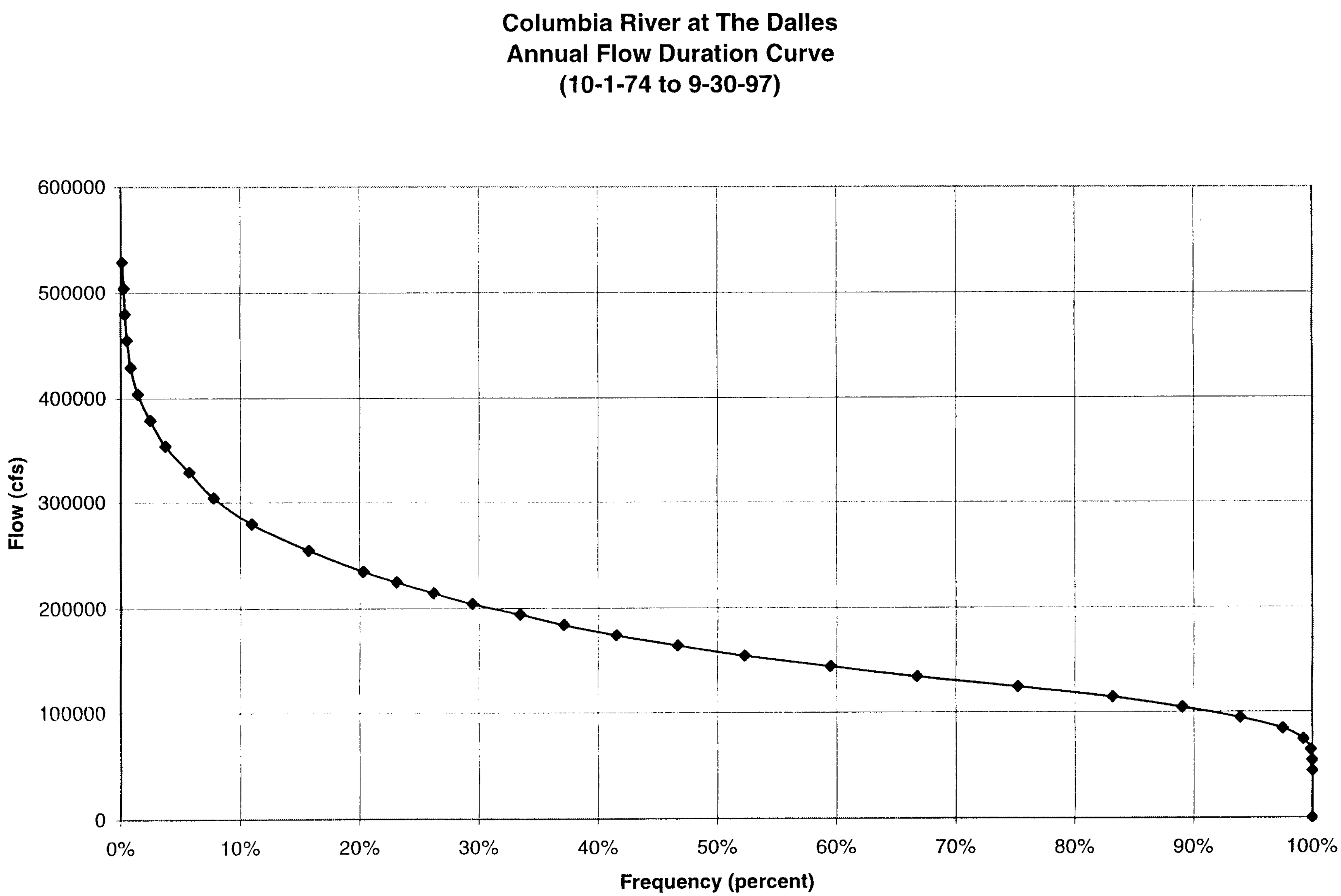

Rating Curves at McNary Dam Tailwater (RM 291.603)
320
310
300
290
280
270
260
250
240
Elevation (feet)
0
500000
1000000
1500000
2000000
2500000
Flow (cfs)
Existing Condition
Natural River without Flood Control
Spillway without Flood Control

Figure A-10



Columbia River at The Dalles
Annual Flow Duration Curve
(10-1-74 to 9-30-97)
600000
500000
400000
300000
200000
100000
0
Flow (cfs)
0%
10%
20%
30%
40%
50%
60%
70%
80%
90%
100%
Frequency (percent)

Figure A-11

Columbia River at The Dalles; USGS Gage

January 1974 - July 1995

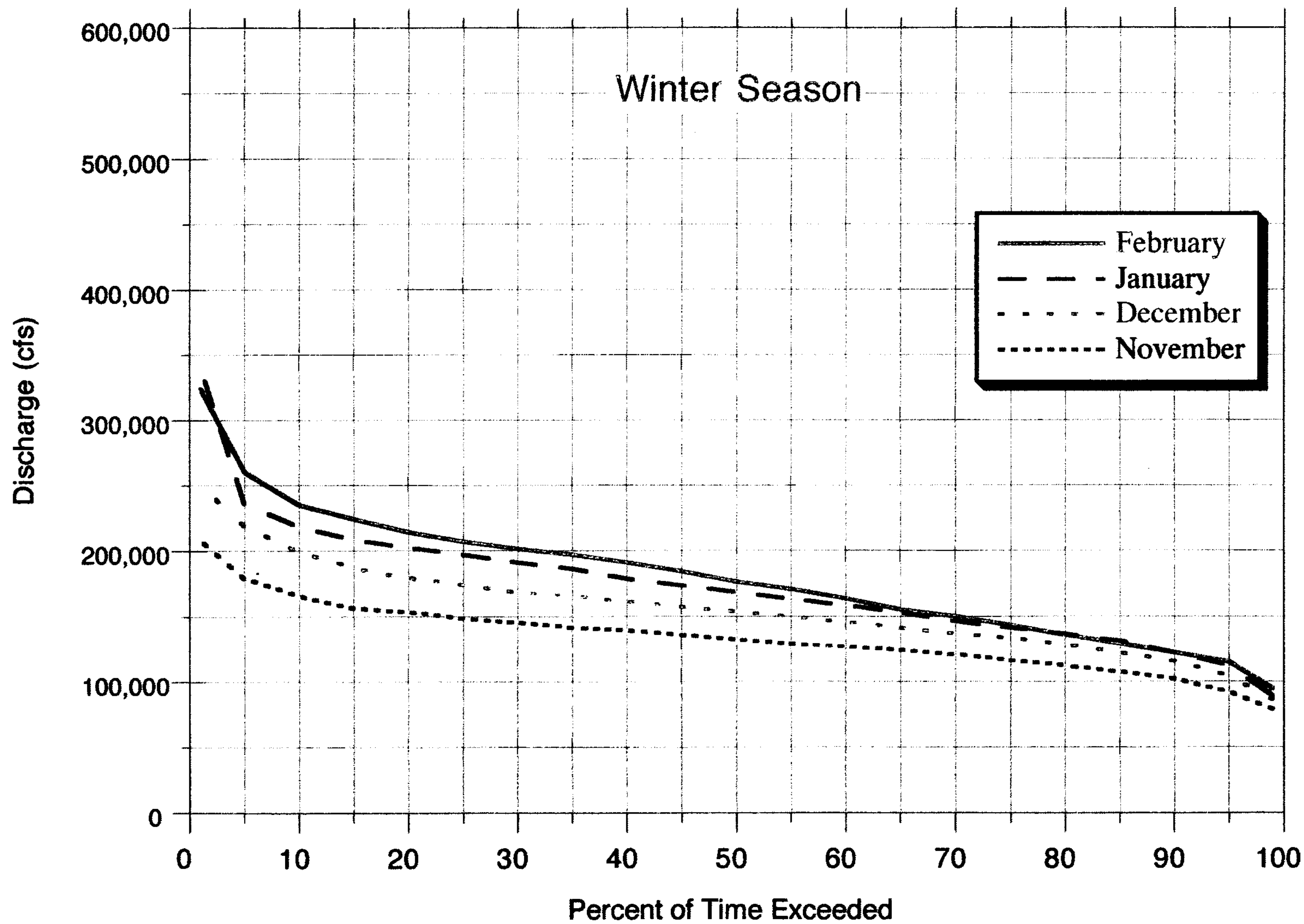


Figure A-12

Columbia River at The Dalles; USGS Gage

January 1974 - July 1995

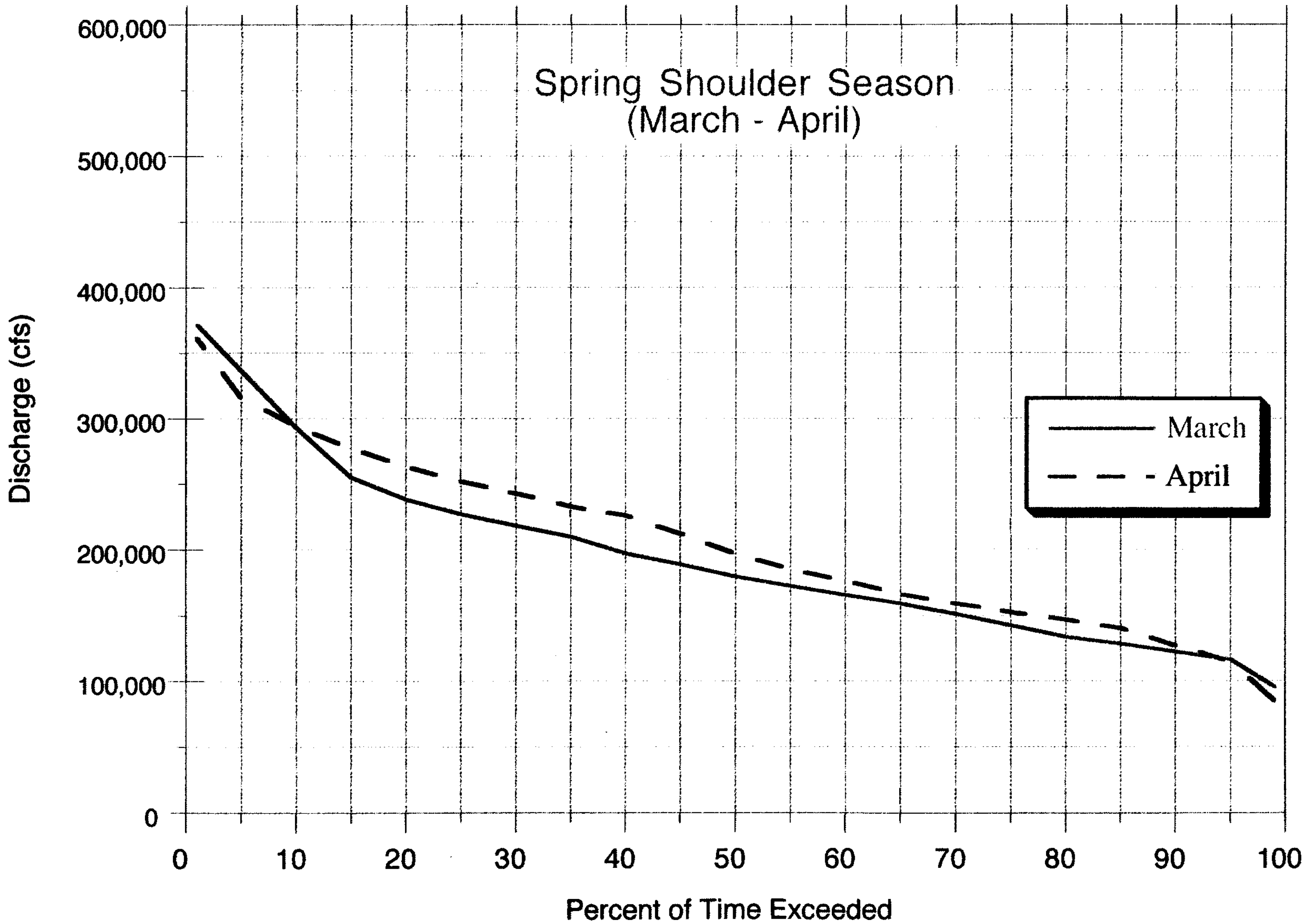


Figure A-13

Columbia River at The Dalles; USGS Gage

January 1974 - July 1995

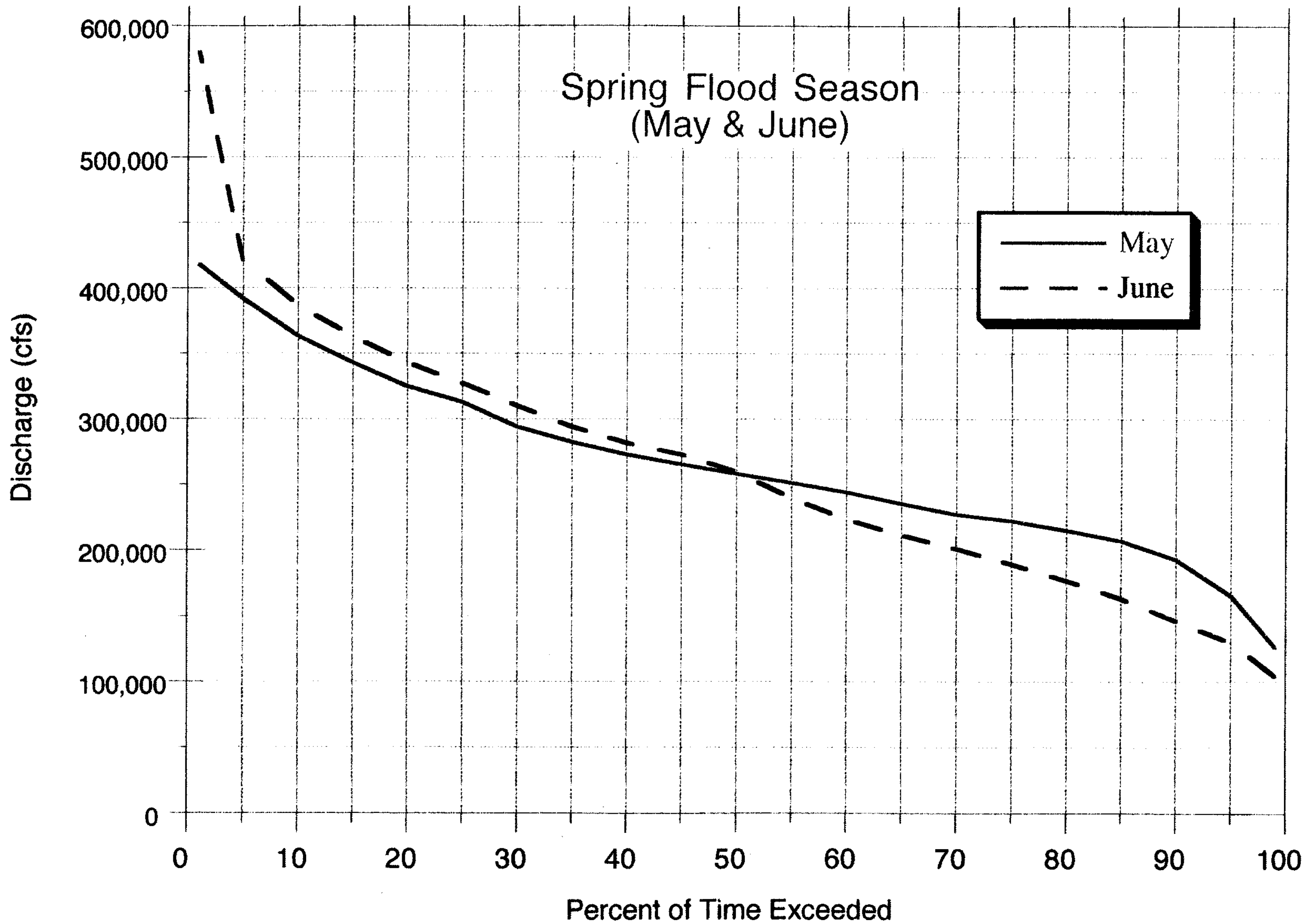
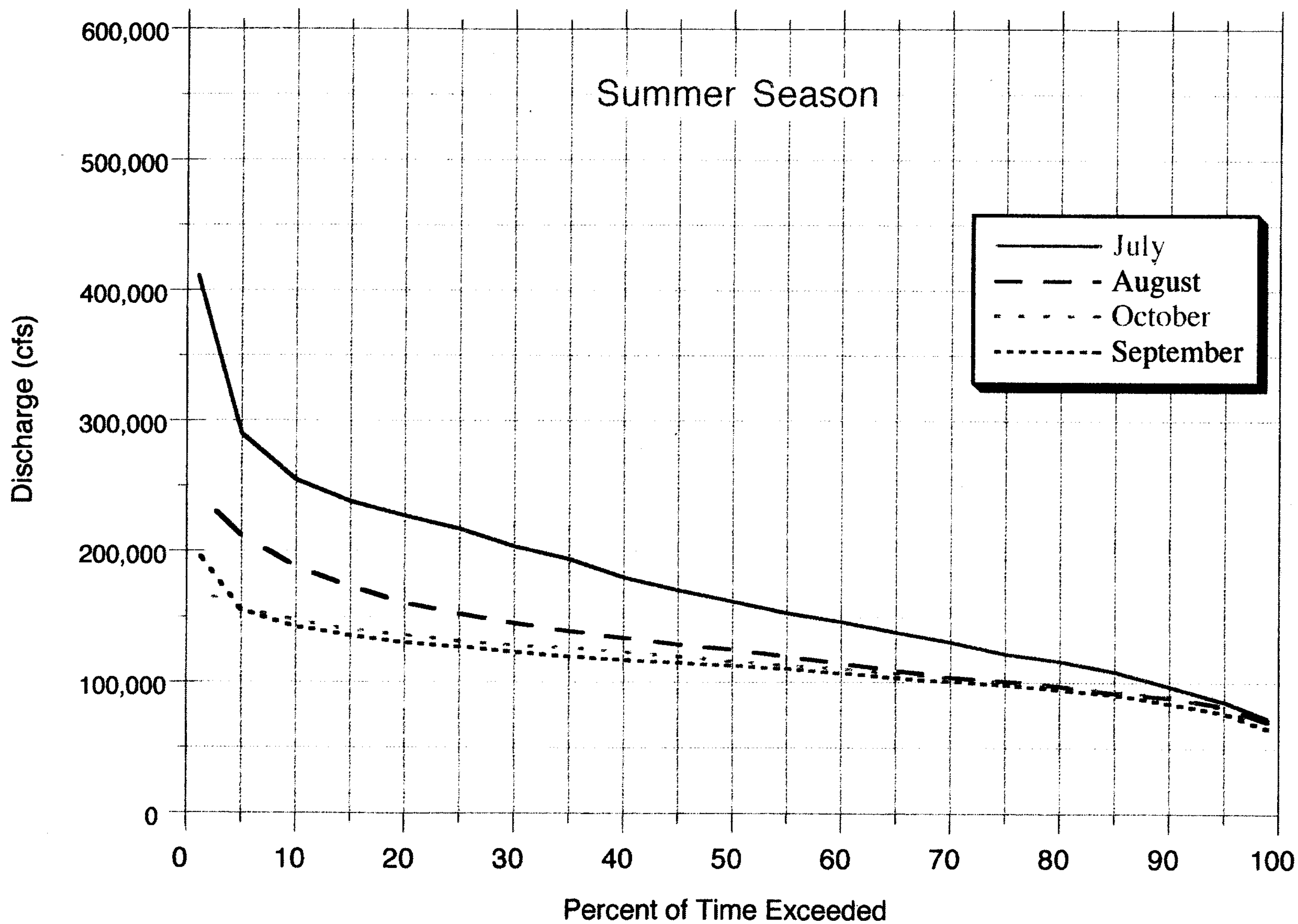


Figure A-14

Columbia River at The Dalles; USGS Gage

Janurary 1974 - July 1995



ATTACHMENT B

ALTERNATIVE 3 HYDRAULIC COMPUTATIONS

Attachment B - ALTERNATIVE 3 HYDRAULIC COMPUTATIONS

This attachment contains a detailed discussion regarding the hydraulic computations for Alternative 3. These computations were required to establish the minimum amount of structural modifications required to obtain a 10 fps maximum average for a discharge of 515,000 cfs and to minimize the impact on barge traffic through the removed section of dam. This alternative includes removing the spillway and a portion of the powerhouse of John Day dam to create hydraulic conditions similar to the pre-dam natural river channel.

B.1 Hydrology

The following rating curves were provided by the Corps of Engineers: Columbia River at Rufus (river mile 213.02), Columbia River at John Day Powerhouse (river mile 215.35), Columbia River at gauges 7 and 8 (river mile 215.48), and a John Day Dam spillway rating curve. The starting conditions for the model require a discharge and a corresponding water surface elevation at the downstream cross-section for a subcritical flow regime. The downstream boundary conditions were obtained from the rating curve for Rufus, which is located at the downstream end of the model. The Rufus and John Day Powerhouse rating curves are presented below in Figures B-1 and B-2 respectively.

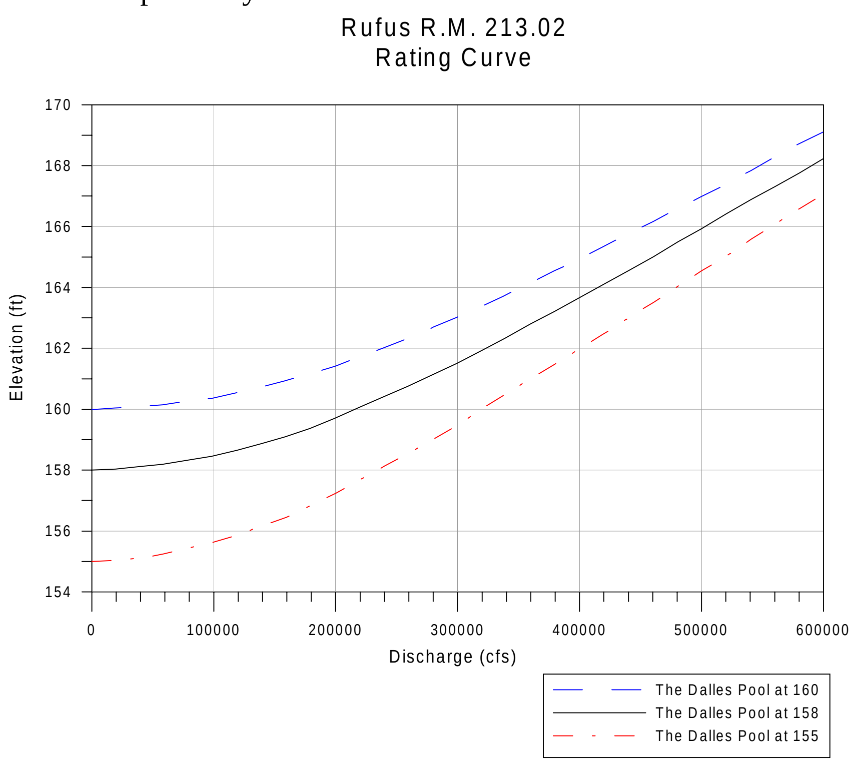


Figure B-1
Rufus Rating Curve

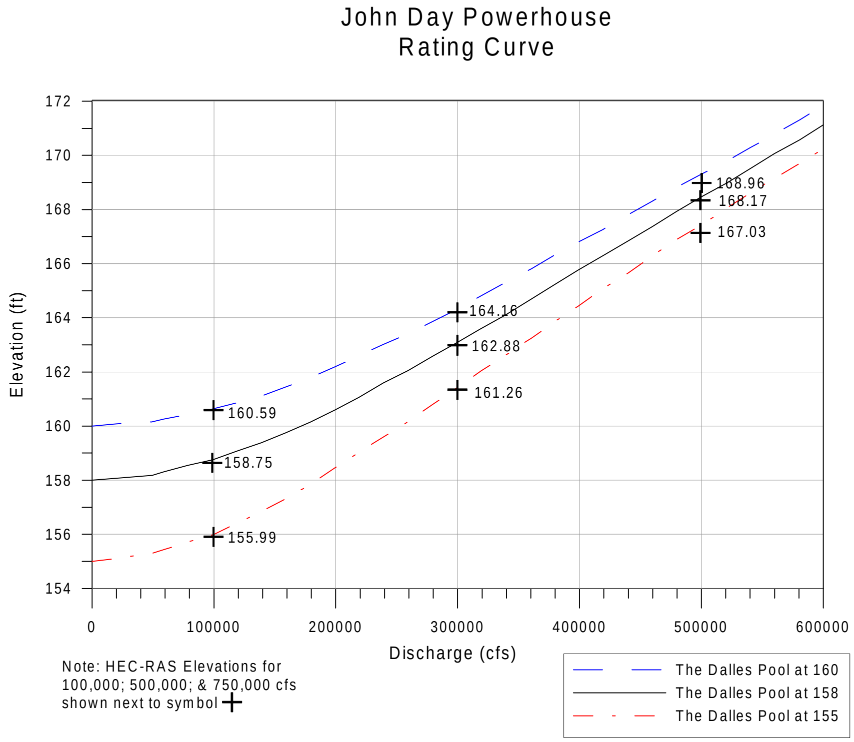


Figure B-2
John Day Powerhouse Rating Curve

B.2 Topography

Modification of the dam site to represent natural river conditions requires topography data representing the existing and pre-dam conditions. Topography available for this analysis include: a pre-dam topography map with 10 feet contour intervals dated December 1955, navigation charts with 20 feet contour intervals (undated), and current topography maps (developed from surveyed cross-sections) with two feet contour intervals (undated). This topography was required to revise the existing HEC-RAS model to reflect the modified river drawdown condition.

B.3 HEC-RAS Model

The hydraulic characteristics of the natural river drawdown option were analyzed using a HEC-RAS (Hydrologic Engineering Center's River Analysis System, Version 2.2) backwater model. Although HEC-RAS was used to predict water surface elevations and velocities, the model does not address flow directions or patterns. A two-dimensional model would provide more information regarding flow directions and patterns; however, this type of model would require more time and data to develop. A two-dimensional model was considered unnecessary for the level of detail required in this study. A HEC-RAS model of the Columbia River extending from McNary Dam to downstream of John Day Dam has been developed by WEST Consultants for the Portland District Corps of Engineers. A portion of this model extending from RM 217.829 to RM 212.510 was modified to analyze the selected natural river drawdown option. The model used for this study includes the river channel between RM 212.983 and RM 217.829

The model contains 59 cross-sections at approximately 500 feet intervals. The downstream and upstream cross-sections in the HEC-RAS model are located at river stations 212.510 and 217.829, respectively. Station numbers correspond to river miles. Therefore, the physical reach of the model is limited to a 5.32 mile reach of the river that extends about 2.48 miles downstream of John Day Dam. The dam is located between stations 215.636 and 215.535. The Manning's roughness values used in the existing model range from 0.025 in the channel to 0.040 along the overbank area. The contraction and expansion coefficients are 0.1 and 0.3, respectively, in all of the cross-sections with the exception of the sections at the dam where the roughness values are 0.3 and 0.5.

B.4 Calibration of Existing Model

Prior to applying the existing model for the hydraulic analysis of the natural river alternative, the model was calibrated by using the rating curves discussed in Section B.1. A calibration error of ±0.5 feet was selected as the maximum acceptable tolerance. The rating curve for the Rufus gage provides downstream boundary conditions for the model. The rating curve for the John Day powerhouse should correlate with the water surface elevations predicted by the model just downstream of the powerhouse. The Rufus and powerhouse rating curves each provide several curves representing forebay elevations at The Dalles Dam ranging from 155.0 feet to 160.0 feet. The model was calibrated for The Dalles Dam forebay elevations of 155.0 feet, 158.0 feet, and 160.0 feet. This provides calibration for a minimum, intermediate, and maximum forebay elevation at The Dalles.

The model was modified to start near the Rufus gage, which provides the starting boundary conditions. The Rufus rating curve was developed for RM 213.02 and the downstream cross-section in the existing model is located at RM 212.51. The head loss between these stations is insignificant; however, the cross-sections downstream of RM 213.02 were deleted to minimize calibration errors. Water surface elevations corresponding to discharges of 100,000 cfs, 300,000 cfs, and 500,000 cfs were obtained from the Rufus rating curve (for the three forebay elevations selected) and used as the starting boundary conditions for the model. The Rufus and Powerhouse Rating Curves used to calibrate the model are shown in Figures B-1 and B-2.

Water surface elevations predicted by HEC-RAS at Station 215.506 (approximately 400 feet downstream of John Day Dam) were compared to the water surface elevations obtained from the Powerhouse rating curve for discharges of 100,000 cfs, 300,000 cfs, 500,000 cfs. This process was repeated for The Dalles Dam forebay elevations of 155.0 feet, 158.0 feet, and 160.0 feet. Calibration errors associated with the existing model are shown in Table B-1. All of the computed values are less than the rating curve values.

Table B-1
Calibration of Existing Model

Discharge (cfs)	Powerhouse Rating Curve WSEL (ft)	WEST Model 400 ft downstream of powerhouse (ft)	Difference Between Model And Rating Curve (ft)
Dalles FB=155			
100,000	156.0	155.88	0.12
300,000	161.4	160.82	0.58
500,000	167.3	166.39	0.91
Dalles FB=158			
100,000	158.8	158.68	0.12
300,000	163.0	162.52	0.48
500,000	168.4	167.6	0.80
Dalles FB=160			
100,000	160.6	160.54	0.06
300,000	164.3	163.85	0.45
500,000	168.44	168.44	0.86

The Manning's roughness value was modified to calibrate the model to within a tolerance of ±0.5 feet. The Manning's roughness value was changed along the entire reach. Manning's roughness value of 0.03 provided the model results shown in Table B-2. As in the previous case, all of the computed values are less than the rating curve values. These results were accepted since the calibration error is within the tolerance target, and a roughness value of 0.03 is a reasonable value for this reach of river. The contraction and expansion coefficients are 0.1 and 0.3, respectively, in all of the cross-sections with the exception of the sections at the dam where the coefficients are 0.3 and 0.5.

Table B-2
Calibration of Model with Modified Roughness Value

Discharge (cfs)	Powerhouse Rating Curve WSEL (ft)	NHC Model 400 ft downstream of powerhouse (ft)	Difference Between Model And Rating Curve (ft)
Dalles FB=155			
100,000	156.0	155.99	0.01
300,000	161.4	161.26	0.14
500,000	167.3	167.03	0.27
Dalles FB=158			
100,000	158.8	158.75	0.05
300,000	163.0	162.88	0.12
500,000	168.4	168.17	0.23
Dalles FB=160			
100,000	160.6	160.59	0.01
300,000	164.3	164.16	0.14
500,000	169.3	168.96	0.34

There are no rating curves available to calibrate the section of the model located upstream of the John Day powerhouse; therefore, direct calibration of this section of the model is not possible. Although this section was not directly calibrated, the Manning's roughness value was changed uniformly throughout the model. The roughness value for the upstream reach under natural river conditions should be similar to that of the downstream reach.

B.5 Model Runs

Several different configurations were modeled including:

- Run 3-1 Remove Spillway and Powerhouse
- Run 3-2 Remove Spillway and Powerhouse Units 6-20
- Run 3-3 Remove Spillway Only
- Run 3-4 Remove Spillway and Powerhouse Units 11-20
- Run 3-5 Remove Spillway and Powerhouse Units 16-20
- Run 3-6 Remove Spillway and Powerhouse Units 17-20
- Run 3-7 Remove Spillway and Powerhouse Units 10-20

The first four runs were modeled to study the sensitivity of the flow characteristics and specifically the velocity to different structural modifications. After studying the first four runs, the options were narrowed to Runs 3-5 and 3-6. These runs were modeled to determine the minimum amount of dam that would have to be removed to provide a velocity of about 10 fps through the removed portion for a discharge of 515,000 cfs and a Dalles forebay elevation of

155.0 feet. The final run was completed to determine a configuration that would minimize the impact on barge traffic requirements.

The model was run for the flows and corresponding starting water surface elevations at Station 213.02 (Rufus location). Results are summarized on Table B-3.

Table B-3
Discharges and Starting Water Surface Elevations used in HEC-RAS model

Discharge (cfs)	Dalles Forebay Elevation (ft)	WSEL at Rufus (ft)
80,000	155.0	155.5
80,000	160.0	160.3
515,000	155.0	164.8
515,000	160.0	167.2

The cross-section geometry representing the dam in the original (WEST) HEC-RAS model was replaced with a pre-dam cross-section. The HEC-RAS model cross-sections were located just upstream and downstream of the project. The pre-dam cross-section was determined from topographic maps and represents a cross-section located at the centerline of the project. The pre-dam cross-section was copied from the upstream to the downstream face of the dam, which is a distance of about 200 feet.

Each run required different modifications to the predam cross-sections located at the upstream and downstream face of the dam. The portions of the powerhouse that remained in place for each run were represented by blocked flow obstructions in the model. Ineffective flow boundaries were used to define areas where water is not actively being conveyed. Determining ineffective flow boundaries requires engineering judgment and experience. The boundaries selected for this project are based on experience with similar configurations; however, the boundaries are only estimates and a physical model test would be necessary to accurately determine the ineffective flow areas. A sensitivity analysis of the ineffective flow boundary locations showed that the velocity through the cross-sections located at the dam were not significantly affected by changing the ineffective flow boundary; however, the ineffective flow boundaries may affect the velocities for some distance upstream and downstream of the project. Some of the runs required a riprap dike to serve as a fish guidance structure, and this dike structure was used as the ineffective flow boundary for several of the runs.

B.5.1 Run 3-1 Remove Spillway and Powerhouse

This run required modifying the geometry file to reflect removal of the entire spillway and powerhouse. The bottom elevations along the powerhouse and spillway sections are reduced to 128.0 feet and 135.0 feet, respectively. The blocked flow obstruction option was used in HEC-RAS to represent the remaining portion of the project south of the powerhouse and north of the spillway. Ineffective flow areas were not required for this run because the constriction caused by the remaining portion of the dam is insignificant.

The upstream and downstream face of the dam are located at Stations 215.635 and 215.535, respectively. This stationing corresponds to the upstream and downstream faces of the dam in the original (WEST) HEC-RAS model. Note that these stations are actually 200 feet apart in the model used for this study instead of 528 feet apart as they were in the original model. All of the other station numbers correspond to river miles. The distance between the stations is provided in the column titled "Length Chnl" in the HEC-RAS output. Table B-4 provides the average velocities and water surface elevations at the downstream face of the dam.

Table B-4
Run 3-1 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.77	1.04
80,000	160	160.44	0.88
515,000	155	167.80	4.53
515,000	160	169.63	4.32

B.5.2 Run 3-2 Remove Spillway and Powerhouse Units 6-20

This run is similar to Run 3-1; however, only Units 6-20 are removed instead of the entire powerhouse. Blocked flow obstructions were required to represent the remaining portion of the project south of Unit 6 and north of the spillway. A riprap dike extends downstream from the south side of Unit 6 for 1400 feet until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore to Unit 6 for about 1000 feet at a 1:1.7 contraction ratio (defined as an expansion width increase of 1 feet for each 1.7 feet in the downstream direction).

Ineffective flow areas were required upstream and downstream of the project to represent contraction and expansion of the flow. For this alternative, the riprap dike was assumed to approximately represent the ineffective flow boundary on the south side both upstream and downstream of the project. The ineffective flow boundary on the north side upstream of the project was set at a 1:1 contraction ratio (defined as a contraction width decrease of 1 feet for each one foot in the downstream direction), and the downstream navigation lock approach represents the downstream ineffective flow boundary.

Table B-5 provides the average velocities and water surface elevations at the downstream face of the dam.

Table B-5
Run 3-2 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.76	1.25
80,000	160	160.43	1.05
515,000	155	167.70	5.39
515,000	160	169.54	5.14

B.5.3 Run 3-3 Remove Spillway Only

The spillway is the only portion of the dam removed for this run. The bottom elevation of the removed portion is 135.0 feet. Blocked flow obstructions were required to represent the portion of the project to the north and south of the spillway. A riprap dike extends downstream from the south side of the powerhouse at a 1:1.5 expansion ratio until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore to the south side of the spillway at a 2:1 contraction ratio.

Ineffective flow areas were required upstream and downstream of the project to represent contraction and expansion of the flow. For this run, the riprap dike was assumed to represent the ineffective flow boundary on the south side downstream of the project. The ineffective flow boundary on the south side upstream of the project was set at 1:1 contraction ratio. The ineffective flow boundary on the north side upstream of the project was set at a 1:1 contraction rate, and the downstream navigation lock approach represents the downstream ineffective flow boundary.

Table B-6 provides the average velocities and water surface elevations at the downstream face of the dam.

Table B-6
Run 3-3 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.70	3.15
80,000	160	160.39	2.57
515,000	155	164.46	13.33 *
515,000	160	168.42	12.55 *

* exceeds 10 fps criteria

B.5.4 Run 3-4 Remove Spillway and Powerhouse Units 11-20

This run is similar to Run 3-2 with the exception of the removal of Units 11-20 instead of 6-20. The bottom elevations of the removed portion of the powerhouse and the spillway are 128.0 feet and 135.0 feet, respectively. Blocked flow obstructions were required to represent the portion of the project to the north of the spillway and south of Unit 11. A riprap dike extends downstream from the south side of the powerhouse until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore to the south side of Unit 11 at a 1:0.83 contraction ratio.

Ineffective flow areas were required upstream and downstream of the project to represent contraction and expansion of the flow. For this run, the riprap dike was assumed to approximately represent ineffective flow boundaries on the south side both downstream and upstream of the project. The ineffective flow boundary on the north side upstream of the project was set at a 1:1 contraction ratio, and the downstream navigation lock approach represents the downstream ineffective flow boundary.

Table B-7 provides the average velocities and water surface elevations at the downstream face of the dam.

Table B-7
Run 3-4 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.76	1.55
80,000	160	160.43	1.29
515,000	155	167.59	6.66
515,000	160	169.43	6.33

B.5.5 Run 3-5 Remove Spillway and Powerhouse Units 16-20

This run is similar to Run 3-4 with the exception of the removal of Units 16-20 instead of 11-20. Fewer units were removed to minimize the structural modifications while obtaining a velocity of about 10 fps through the removed section of the dam for a discharge of 515,000 cfs and a forebay elevation at The Dalles of 155.0 feet. The bottom elevations of the removed portion of the powerhouse and the spillway are 128.0 feet and 135.0 feet, respectively. Blocked flow obstructions were required to represent the portion of the project to the north of the spillway and south of Unit 16. A riprap dike extends downstream at an expansion ratio of 1:2.7 from the south side of the powerhouse until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore to the south side of Unit 16 at a contraction ratio of 1.6:1.

Ineffective flow boundaries were required upstream and downstream of the project to represent contraction and expansion of the flow. For this run, the riprap dike was assumed to approximately represent ineffective flow boundaries on the south side downstream of the project. The ineffective flow boundaries on the north and south sides upstream of the project were set at a 1:1 contraction ratio, and the downstream navigation lock approach represents the downstream ineffective flow boundary.

Table B-8 provides the average velocities and water surface elevations at the downstream face of the dam.

Table B-8
Run 3-5 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.74	2.12
80,000	160	160.42	1.79
515,000	155	167.30	9.27
515,000	160	169.17	8.81

B.5.6 Run 3-6 Remove Spillway and Powerhouse Units 17-20

Since the average velocity was below 10 fps in Run 3-5, Units 17-20 were removed to determine if one more unit could remain in place while still maintaining a velocity of about 10 fps. The bottom elevations of the removed portion of the powerhouse and the spillway are 128.0 feet and 135.0 feet, respectively. Blocked flow obstructions were required to represent the portion of the project to the north of the spillway and south of Unit 17. A riprap dike extends downstream at an expansion ratio of 1:2.5 from the south side of the powerhouse until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore to the south side of Unit 17 at a 1.7:1 contraction ratio.

Ineffective flow areas were required upstream and downstream of the project to represent contraction and expansion of the flow. For this run, the riprap dike was assumed to approximately represent ineffective flow boundaries on the south side downstream of the project. The ineffective flow boundaries on the north and south sides upstream of the project were set at a 1:1 contraction rate, and the navigation lock approach represents the ineffective flow boundary on the right side downstream of the project.

Table B-9 provides the average velocities and water surface elevations at the downstream face of the dam.

Table B-9
Run 3-6 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.74	2.27
80,000	160	160.41	1.90
515,000	155	167.21	9.84
515,000	160	169.09	9.34

Run 3-7 Remove Spillway and Powerhouse Units 10-20

Although the average velocity met the fish passage criteria in Run 3-6, the impact on barge traffic was not acceptable. Therefore, Units 10-20 were removed to determine if one more unit could remain in place while still maintaining a velocity of about 10 fps. The bottom elevations of the removed portion of the powerhouse and the spillway are 128.0 feet and 135.0 feet, respectively. Blocked flow obstructions were required to represent the portion of the project to the north of the spillway and south of Unit 10. A riprap dike extends downstream from the south side of the powerhouse until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore to the south side of Unit 10 at a 1:0.9 contraction ratio.

Ineffective flow areas were required upstream and downstream of the project to represent contraction and expansion of the flow. For this run, the riprap dike was assumed to approximately represent ineffective flow boundaries on the south side downstream of the project. The ineffective flow boundaries on the north and south sides upstream of the project were set at a 1:1 contraction rate, and the navigation lock approach represents the ineffective flow boundary on the right side downstream of the project.

Table B-9 provides the average velocities and water surface elevations at the downstream face of the dam.

Table B-10
Run 3-7 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.76	1.48
80,000	160	160.43	1.24
515,000	155	167.61	6.36 *
515,000	160	169.45	6.05 *

* meets fish passage and barge criteria

The Run 3-7 model was also run for several other discharges between 100,000 cfs and 500,000 cfs. Table B-10 provides output for the various discharges at Station 215.535 (downstream face of the dam).

Table B-11
Run 3-7 Hydraulic Characteristics for Additional Discharges

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
90,000	155	155.92	1.65
90,000	160	160.56	1.38
100,000	155	156.09	1.82
100,000	160	160.69	1.53
150,000	155	157.07	2.63
150,000	160	161.31	2.25
200,000	155	158.73	3.33
200,000	160	162.17	2.91
250,000	155	159.90	3.93
250,000	160	163.15	3.53
300,000	155	161.34	4.49
300,000	160	164.24	4.09
350,000	155	162.82	4.99
350,000	160	165.49	4.60
400,000	155	164.35	5.44
400,000	160	166.65	5.08
450,000	155	165.84	5.85
450,000	160	167.96	5.51
500,000	155)	167.24	6.24
500,000	160	169.17	5.92

B.5.7 Run 3 Phase II Construction

During Phase II Construction, an opening in the dam would extend for 1269 feet from Station 2851 to 4120. The HEC-RAS model was run for a variety of discharges ranging from 80,000 cfs to 515,000 cfs. Blocked flow obstructions were required to the north and south of the 1269 feet opening. Ineffective flow boundaries were calculated for a 1:1 contraction on the upstream side. An ineffective flow boundary also extends from the south end of the opening to the south shore. No riprap dike would be in place during the construction phase. The HEC-RAS results for the low and high discharges at station 215.535 (downstream face of dam) are shown in Table B-12. The computed output for other discharges and stations is included with the calculations under separate cover.

Table B-12
Run 3 Construction, Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.73	2.39
80,000	160	160.41	2.03
515,000	155	167.03	10.76
515,000	160	168.93	10.24

B.6 Summary of Hydraulic Computations

Several runs were modeled to determine the velocities through the removed section of the dam for different structural configurations. The target average velocity for fish passage through the open section of the dam is 10 fps for a discharge of 515,000 cfs and a Dalles forebay elevation of 155 feet; however, the impact to barge traffic must also be minimized. Runs 3-5 and 3-6 are two options that provide velocities in the range of 10 fps; however, these runs do not meet the barge traffic requirements. Alternative 3-7 meets both the fish passage and barge traffic criteria. The removal of the spillway and the northern ten units of the powerhouse is necessary to minimize the impact to barge traffic.

ATTACHMENT C

ALTERNATIVE 4 HYDRAULIC COMPUTATIONS

Attachment C - ALTERNATIVE 4 HYDRAULIC COMPUTATIONS

This attachment contains a detailed discussion regarding the hydraulic computations for Alternative 4. This alternative includes modification of the John Day spillway to reflect natural river conditions while providing flood control. The entire spillway was modified in all of the options to resemble a broad crested weir structure. In addition, different sections of the powerhouse were removed and replaced with spillway bays. The minimum amount of structural modifications required to obtain a maximum average velocity of 10 fps at a discharge of 515,000 cfs and a Dalles forebay elevation of 155.0 feet was determined by modeling a variety of options. Since a new navigation lock would be constructed for this alternative, barge traffic requirements were not a controlling factor in designing the size of the opening. Backwater calculations were required to estimate the velocities and water surface elevations along the modified reach of river.

C.1 Existing HEC-RAS Model

Attachment B, Sections B.2.1 through B.2.4 provide a discussion of the existing HEC-RAS model that was used to analyze modifications made to the river channel for Run 4.

C.2 Conditions Modeled

The following runs were modeled:

Run 4-1	Modify Spillway and Replace Entire Powerhouse with a New Spillway, Crest Elevation 135.0 feet
Run 4-2	Modify Spillway and Replaced Powerhouse Units 10 through 20 with a New Spillway, Crest Elevation 135.0 feet
Run 4-3	Modify Spillway and Entire Powerhouse with a New Spillway, Crest Elevation 130.0 feet
Run 4-4	Modify Spillway and Replaced Powerhouse Units 10 through 20 with a New Spillway, Crest Elevation 130.0 feet
Run 4-5	Modify Spillway and Replaced Powerhouse Units 16 through 20 with a New Spillway, Crest Elevation 135.0 feet
Run 4-6	Modify Spillway and Replaced Powerhouse Units 15 through 20 with a New Spillway, Crest Elevation 135.0 feet

The first four runs provided a range of options from removing a portion of the powerhouse to the entire powerhouse. The last two runs were analyzed to determine a configuration that would provide an average velocity of about 10 fps through the revised spillway portion of the dam.

C.2.1 Run 4-1 Modify Spillway and Entire Powerhouse, Crest Elevation 135.0 feet

This run includes modifying the spillway and replacing the entire powerhouse with a broad crested weir with a crest elevation of 135.0 feet. The 49 spillway bays span 50 feet wide with 12 feet piers on either side. Blocked flow obstructions were required to represent the portion of the project to the north of the spillway and south of the powerhouse. Piers were entered into the cross-section at the dam to reflect the 12 feet spillway piers. Ineffective flow areas were not required for this run because the contraction and expansion would be insignificant.

Table C-1 provides the average velocities and water surface elevations at the downstream face of the dam.

Table C-1
Run 4-1 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.76	1.27
80,000	160	160.43	1.04
515,000	155	167.75	5.20
515,000	160	169.59	4.92

C.2.2 Run 4-2 Modify Spillway and Replace Powerhouse Units 10 through 20, Crest Elevation 135.0 feet.

This run is similar to Run 4-1, however, only half of the powerhouse is modified and the total number of spillway bays is 35 with a crest elevation of 135.0 feet. Blocked flow obstructions were required to represent the portion of the project to the north of the spillway and to the south of station 29+54, which is located just north of Unit 10. The ineffective flow areas are similar to those discussed in Section B.5.4 for Run 3-4.

Table C-2 provides the average velocities and water surface elevations at the downstream face of the dam.

Table C-2
Run 4-2 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.75	1.79
80,000	160	160.42	1.46
515,000	155	167.52	7.34
515,000	160	169.37	6.94

C.2.3 Run 4-3 Modify Spillway and Replace Entire Powerhouse, Crest Elevation 130.0 feet

This run is identical to Run 4-1 with the exception of the spillway crest elevation, which is 130.0 feet instead of 135.0 feet. There are 49 spillway bays for this alternative as discussed in Section C.2.1. Table C-3 provides the average velocities and water surface elevations at the downstream face of the dam.

Table C-3
Run 4-3 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.77	1.03
80,000	160	160.44	0.87
515,000	155	167.80	4.50
515,000	160	169.63	4.29

C.2.4 Run 4-4 Modify Spillway and Replace Powerhouse Units 10 through 20, Crest Elevation 39.62 m (130.0 feet)

This run is identical to Run 4-2 with the exception of the spillway crest elevation, which is at 130.0 feet instead of 135.0 feet. This run includes 35 spillway bays as discussed in Section C.2.2. Table C-4 provides the average velocities and water surface elevations at the downstream face of the dam.

Table C-4
Run 4-4 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.76	1.44
80,000	160	160.43	1.22
515,000	155	167.62	6.34
515,000	160	169.46	6.05

C.2.5 Run 4-5 Modify Spillway and Replace Powerhouse Units 16 through 20, Crest Elevation 135.0 feet

The entire spillway and Units 16 through 20 are modified in this run. There are 28 spillway bays in this run all at crest elevation 135.0 feet. Blocked flow obstructions were required to represent the portion of the project to the north of the spillway and to the south of CBL Station 33+88, which is located between Units 15 and 16. The riprap dike configuration and ineffective flow areas are similar to those discussed in Section B.5.5 for Run 3-5.

Table C-5 provides the average velocities and water surface elevations at the downstream face of the dam.

Table C-5
Run 4-5 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.73	2.42
80,000	160	160.41	1.98
515,000	155	167.02	10.05 *
515,000	160	169.09	9.50

***** exceeds velocity criteria

C.2.6 Run 4-6 Modify Spillway and Replace Powerhouse Units 15 through 20, Crest Elevation 135.0 feet

The average velocity in Run 4-5 exceeded 10 fps for a discharge of 515,000 cfs and a Dalles forebay elevation of 155.0 feet; therefore, one more unit was removed to approach a velocity equal to or less than 10 fps. Blocked flow obstructions were required to represent the portion of the project to the north of the spillway and to the south of CBL Station 33+26, which is located between Units 14 and 15. There are 29 spillway bays in this run. A riprap dike extends downstream at an expansion rate of 1:2.9 from the south side of the powerhouse until reaching the south shore. A dike also extends upstream of the powerhouse from the south shore at a 1.5:1 contraction rate.

Ineffective flow areas were required upstream and downstream of the project to represent contraction and expansion of the flow. For this run, the riprap dike was assumed to approximately represent the ineffective flow boundary on the south side downstream of the project. The ineffective flow boundaries on the north and south sides upstream of the project were set at a 1:1 contraction ratio, and the navigation lock approach represents the ineffective flow boundary on the right side downstream of the project.

Table C-6 provides the average velocities and water surface elevations at the downstream face of the dam.

Table C-6
Run 4-6 Average Velocities and Water Surface Elevations

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
80,000	155	155.74	2.35
80,000	160	160.41	1.92
515,000	155	167.27	9.74 *
515,000	160	169.15	9.21 *

*****meets velocity criteria

The Run 4-6 model was also run for several other discharges between 100,000 and 500,000 cfs. Table C-7 provides the output for the various discharges at Station 215.535 (downstream face of the dam).

Table C-7
Run 6 Hydraulic Characteristics for Additional Discharges

Discharge (cfs)	Dalles Forebay Elev. (ft)	Computed WSEL At Downstream Face of Project (ft)	Computed Velocity at Downstream Face of Project (fps)
90,000	155	155.89	2.62
90,000	160	160.54	2.15
100,000	155	156.06	2.89
100,000	160	160.67	2.38
150,000	155	157.01	4.15
150,000	160	161.27	3.48
200,000	155	158.26	5.24
200,000	160	162.10	4.50
250,000	155	159.76	6.16
250,000	160	163.05	5.44
300,000	155	161.16	6.99
300,000	160	164.09	6.29
350,000	155	162.60	7.74
350,000	160	165.31	7.05
400,000	155	164.09	8.39
400,000	160	166.43	7.77
450,000	155	165.55	8.99
450,000	160	167.71	8.40
500,000	155	166.91	9.56
500,000	160	168.88	9.01

C.3 Summary

Several runs were modeled to determine a range of velocities associated with different structural modifications. The target average velocity through the open section of the dam is 10 fps). Lowering the spillway crest elevation below 135.0 feet to elevation 130.0 feet provided only minimal velocity reduction and is not recommended due to the topography downstream which is at a higher elevation. Runs 4-5 and 4-6 are the two options that provide velocities in the range of 10 fps. Run 4-6 meets the 10 fps criteria with an average velocity of 9.74 fps. The average velocity in Run 4-5 is about 10.05 fps, which exceeds the velocity criteria. The study shows that removing the entire dam is not required; however, removing only the spillway does not reduce the velocities sufficiently to meet the 10 fps criteria. The removal of the spillway plus the northern six units of the powerhouse is necessary to reduce flow velocities below 10 fps through the spillway at the design flow.