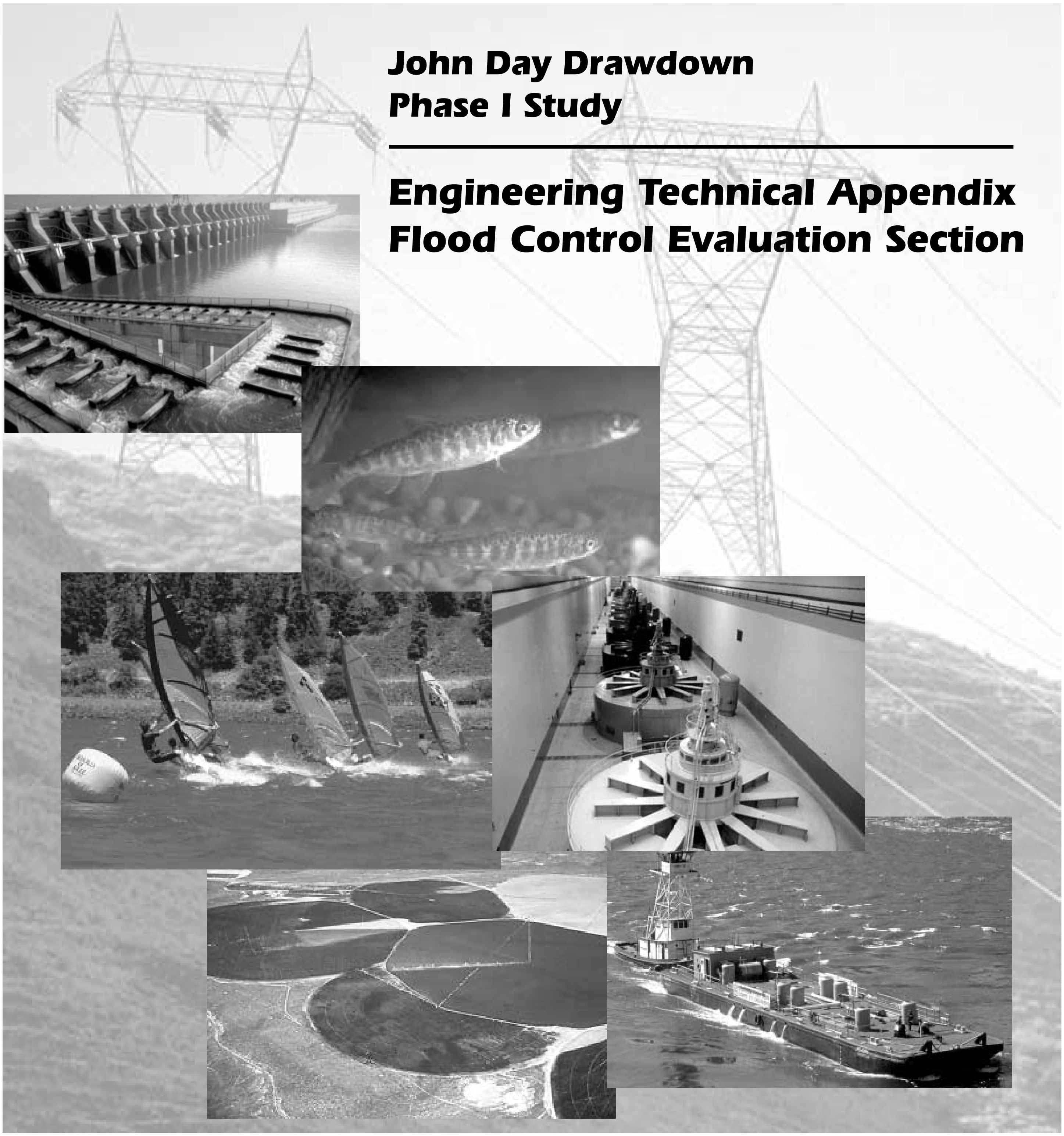
US Army Corps of Engineers ®
Portland District

Salmon Recovery through John Day Reservoir



John Day Drawdown Phase I Study

Engineering Technical Appendix Flood Control Evaluation Section

September 2000

TABLE OF CONTENTS

FIGURES

TABLES

PLATES

1. INTRODUCTION

1.1 General

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

Natural resource agencies believe that lowering the John Day reservoir may decrease juvenile salmonid travel times and create a more natural shoreline and benthic community structure, similar to the unimpounded reach of the Columbia River. The main stem spawning populations of fall chinook salmon appear to be healthy and productive in that reach. 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 spawning habitat. Drawdown of John Day pool may improve spawning conditions for adult fall chinook by restoring spawning habitat and the natural flow regimes needed for successful incubation and emergence.

There are two 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 Programs. Those goals include: (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 of fall chinook.

In response to direction provided in the Energy and Water Development Appropriation Bill, 1998, the Corps of Engineers is studying the potential drawdown of the John Day reservoir to spillway crest and natural river conditions. Normal full pool elevation is 265 feet above National Geodetic Vertical Datum (NGVD); operation at spillway crest would result in a reservoir elevation that will vary from about 217 to 230 ft NGVD; and natural river elevation would be about 170 ft NGVD. The Corps' initial analysis is a reconnaissance-level study evaluating biological, social and economic benefits and costs of the two proposed alternatives, that identifies the potential physical impacts of drawdown. If justified, a feasibility-level evaluation of all the benefits, costs and physical impacts associated with a range of reasonable drawdown alternatives will be performed.

1.2 Goals and Objectives

This reconnaissance study evaluated the impacts of alternative pool configurations, including drawdown to spillway crest and drawdown to natural river, and the extent to which these alternative configurations can provide some measure of flood control.

This report examines the hydraulics of the various configurations and flood control, and evaluates their responses in terms of reservoir storage, ability to store and attenuate floods, and the time of travel through the system.

1.3 Organization of Report

In addition to this INTRODUCTION section, the study report has six additional sections:

Chapter 2, DESCRIPTION OF ALTERNATIVES, describes the base (Existing) condition and four alternative conditions evaluated in this flood control analysis.

Chapter 3, HEC-RAS MODEL OF JOHN DAY POOL, describes the development an application of a steady-state hydraulic model to the John Day reservoir, and the data products produced.

Chapter 4, UNET MODEL OF JOHN DAY POOL, describes the development and application of an unsteady flow model to the John Day reservoir, and the data products produced.

Chapter 5, ANALYSIS OF RESULTS, analyzes and reviews the results of the two model studies, and discusses the changes in system hydraulics and flood control that might be found following implementation of any of the alternatives.

Chapter 6, SUMMARY, describes the conclusions of the flood control evaluation.

Chapter 7, REFERENCES, identifies the sources of information utilized in preparing the study.

2. DESCRIPTION OF ALTERNATIVES

We evaluated a base ("existing") condition and four alternatives – the three "scenarios" and two "conditions" listed in Table 2-1.

Table 2-1: Alternatives

Scenario	***Condition***	
	"without Flood Control"	*"with Flood Control"*
Existing Condition	-	Base
Spillway Drawdown	Alternative One	Alternative Two
Natural River Drawdown	Alternative Three	Alternative Four

2.1 Base Alternative

This is the existing condition that will prevail into the future in the absence of any new Federal action at John Day Dam. The project will incur no structural modifications, and will be operated under current "with flood control" conditions, with the authorized storage of 500,000 acre-feet. It was used as a basis to evaluate the base alternative.

2.2 Spillway Drawdown without Flood Control

The first study alternative is based on requirements for improved downstream fish passage conditions during both low and flood flow conditions on the Columbia River. The existing 20-bay spillway will be operated differently from current operations, but without any structural modifications. All project inflows will be directly passed through the dam spillway with the spillway gates fully opened in free overflow condition, resulting in a pool elevation that will vary from about 217 to 230 ft NGVD. Impacts downstream of the John Day Dam were not studied.

2.3 Spillway Drawdown with Flood Control

The second study alternative is based on requirements for improved downstream fish passage conditions during low flow periods, while maintaining the 500,000 acre-ft of allocated project storage space. The existing 20-bay spillway will be operated differently from current operation, but without any structural modifications. During low flow periods, project inflows will be directly passed through the dam spillway with the spillway gates set in fully open, free overflow, condition. During a flood event, however, the spillway gates will be controlled to reduce downstream flood flows based on using 500,000 acre-ft of allocated project storage space. Ponding will occur upstream of the dam. Impacts downstream of John Day Dam were not studied.

2.4 Natural River Drawdown without Flood Control

The third study alternative is based on a Natural River drawdown scenario for fish passage "without flood control" condition. Natural River conditions pertain to an opening at the John Day Dam that permits acceptable upstream fish passage conditions. The size of the total dam opening must conform to two criteria based on an invert elevation at the dam of 135 feet NGVD. The first criterion is that the opening must be sufficiently large to meet maximum allowable stream velocity criteria for sustained swim speed for the weakest salmon species, which is estimated to be 10 ft per second (Personal Communication with Ken Soderlind, March 1999). The second criterion is that fish passage for this opening must correspond to the 10-year annual flood peak (515,000-cfs). This third alternative would require extensive modifications to John Day Dam even beyond modification of the 1,228-ft long spillway structure. Impacts downstream of John Day Dam were not studied.

2.5 Natural River Drawdown with Flood Control

This fourth study alternative is based on Natural River conditions for fish passage, and includes the "with flood control" condition. It requires natural fish passage conditions for both upstream and downstream directions at the dam, and includes a requirement for full-authorized flood control. The calculated width of the total dam opening will correspond to that previously calculated for Natural River conditions without flood control (Alternative No. 3). Impacts downstream of John Day Dam were not studied.

3. HEC-RAS MODEL OF JOHN DAY POOL

3.1 Source of Information

One-dimensional steady flow models of the John Day reservoir were developed using the Corps of Engineers' model, HEC-RAS (HEC, 1998), for the existing condition, spillway condition without flood control, and natural condition without flood control alternatives. The spillway condition with flood control and natural condition with flood control alternatives were not modeled with HEC-RAS because they represent unsteady flow conditions. Initial versions were developed and run under a previous Task Order No. 0001. The models were updated to provide cross sections extending from the tailwater of the upstream McNary Dam to the gage at Rufus, located about three river miles downstream of the John Day Dam. Information used to develop and update the models included:

- Cross sections, nominally at 500-ft intervals, from Rufus (RM 213.0) to McNary Dam (RM 291.6).
- A rating curve developed at Rufus to specify the water surface elevation at the downstream end of the model (Rufus gage) based on the steady river flow and the operating condition in The Dalles reservoir.
- U.S. Geological Survey quad maps of the reservoir, to possibly extend cross sections to the valley walls, and to identify ineffective flow areas.

3.2 Development of HEC-RAS Models

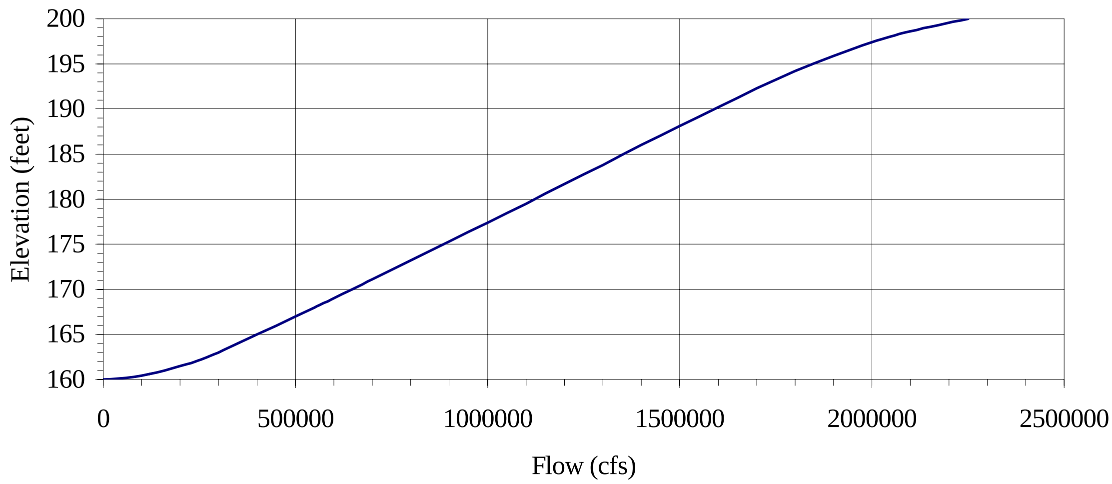
An HEC-RAS model was first developed for the main stem of the Columbia River from Rufus (RM 213.0) to the tailwater of McNary Dam (RM 291.6), nominally at 500-ft intervals for the Existing Condition. Tributaries were not modeled. The model has 824 cross sections, and was provided from an ARC/INFO coverage of the John Day reservoir by the Portland District. The sections were entered into the model and channel and overbank lengths adjusted.

In several areas, particularly at significant bends, some sections did not extend all the way across the river to the valley walls. In these cases, a few sections were removed or lengthened. In other areas, for example, the channel behind Crow Butte Island, ineffective flow areas were defined to limit the river conveyance to the main Columbia River channel.

Global values of Mannings *n* roughness coefficients were selected for the channel and overbank areas, and contraction/expansion coefficients of 0.1 and 0.3 specified to model the effect of variations in cross sectional geometry. A downstream boundary rating curve was specified at Rufus (RM 213.0) using a figure developed by the Corps based on operation of The Dalles Dam and reservoir (Figure 3-1). A "normal" operating elevation in The Dalles Pool of 160 ft NGVD was used. The model was run for the 22 flows: 50,000, 75,000, 100,000, 150,000, 200,000, 250,000, 300,000, 350,000, 400,000, 450,000 (the 5-year recurrence interval flood) 500,000, 600,000, 700,000, 800,000, 900,000, 1.0

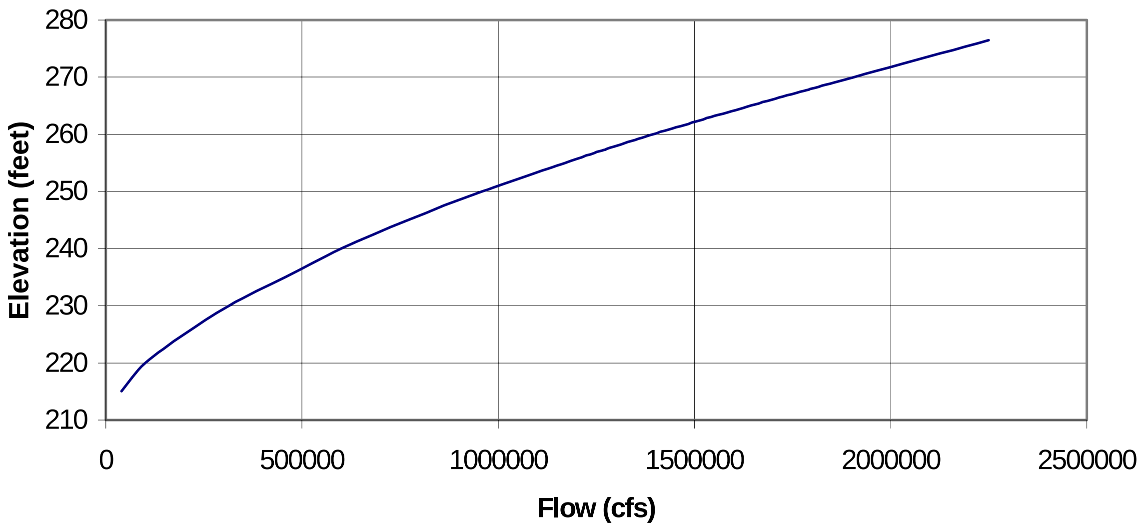
million, 1.1 million, 1.2 million, 1.3 million, 1.4 million, 1.5 million, and 2.25 million cfs.

Figure 3-1: Downstream Rating Curve



An HEC-RAS model was developed for the Spillway Drawdown condition by copying the Existing condition model, and including an internal rating curve at the John Day Dam for free spillway overflow (Figure 3-2). An HEC-RAS model was developed for the Natural River condition by copying the Existing condition model, modifying sections through the dam, and defining ineffective flow areas to describe the contraction to and expansion from the opening. The size of the opening is discussed below.

Figure 3-2: Spillway Rating Curve at John Day Dam

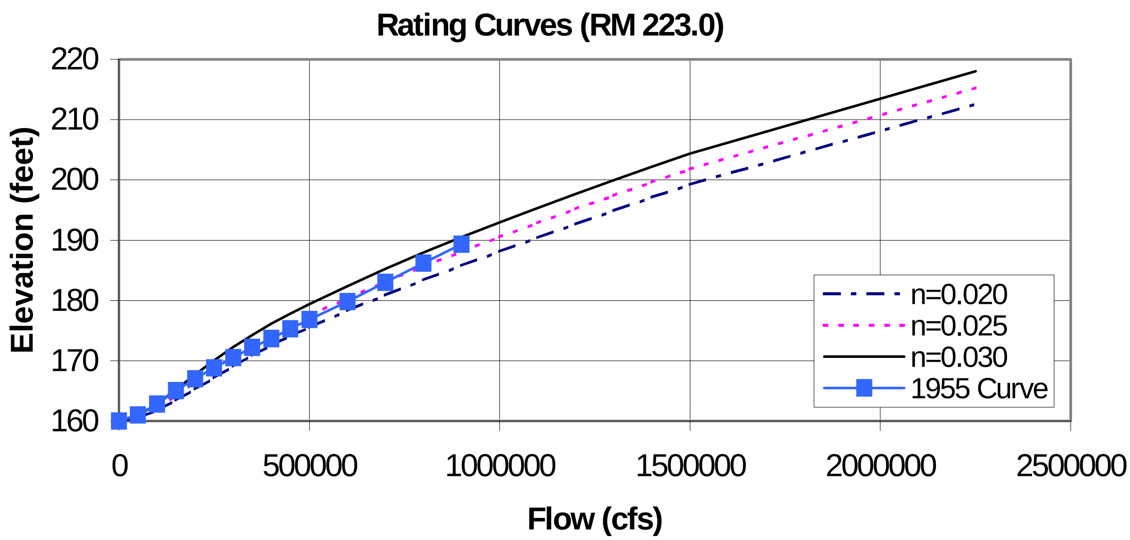


3.3 Sensitivity Analysis of Channel Roughness

In 1955, the Corps had performed a hydraulic investigation of this "natural river" reach of the Columbia River (updated material in archives) prior to closing the John Day Dam. No other "calibration" information was available to the study team.

The HEC-RAS model was run with channel roughness Manning's *n* values of 0.02, 0.025 and 0.03. Figure 3-3 compares the results of these simulations with the results from the previous hydraulic analysis. The results show that *n*=0.025 gives the "best" results, in that the model is at least consistent with the results from the earlier hydraulic analysis.

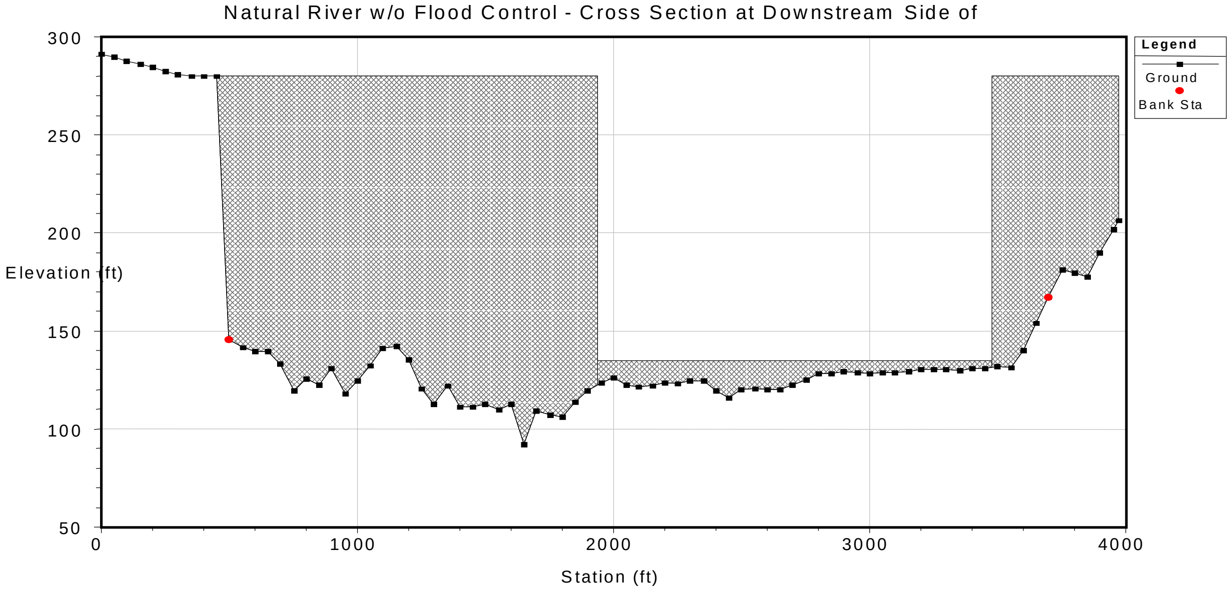
Figure 3-3: Sensitivity Analysis



3.4 Size of Opening for Fish Passage

The Natural River condition was defined as an opening in the existing John Day Dam sufficient for fish passage at the 10-year flow of 515,000 cfs with an average channel velocity not to exceed 10 ft/s. The Natural River HEC-RAS model was developed by modifying the cross sections at the dam to form a channel invert elevation of 135 ft NGVD, and ineffective flow areas to define contraction to and expansion from the opening. Model iterations resulted in an opening of 1,540 ft to meet the fish passage criterion (Figure 3-4).

Figure 3-4: Natural River Opening



3.5 Water Surface and Velocity Profiles

The three HEC-RAS models were run with the 22 flows. Figures 3-5 through 3-7 show the water surface profiles for the Existing, Spillway Drawdown and Natural River conditions, respectively, for some of the 22 flows. Figures 3-8 through 3-10 show the longitudinal channel-average velocities for the Existing, Spillway Drawdown and Natural River conditions, respectively, for the same flows. Table 3-1 shows the forebay elevations at John Day Dam for different flows.

Figure 3-5: Existing Condition Water Surface Profiles

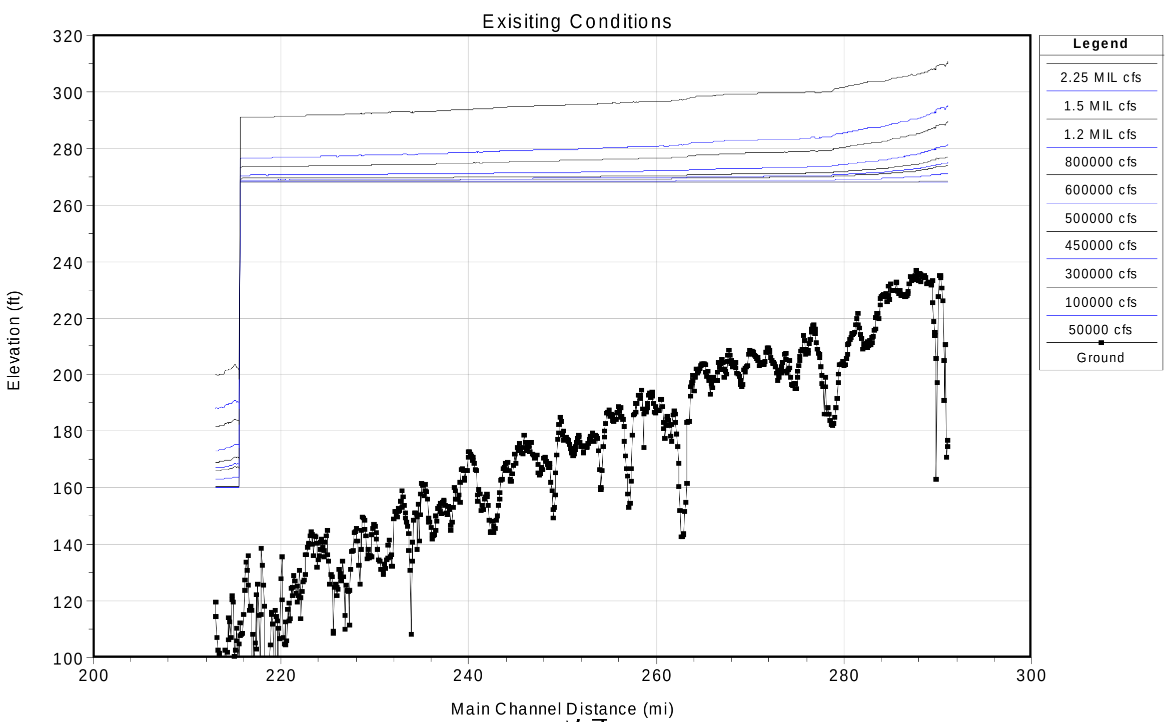


Figure 3-6: Spillway Drawdown Water Surface Profiles

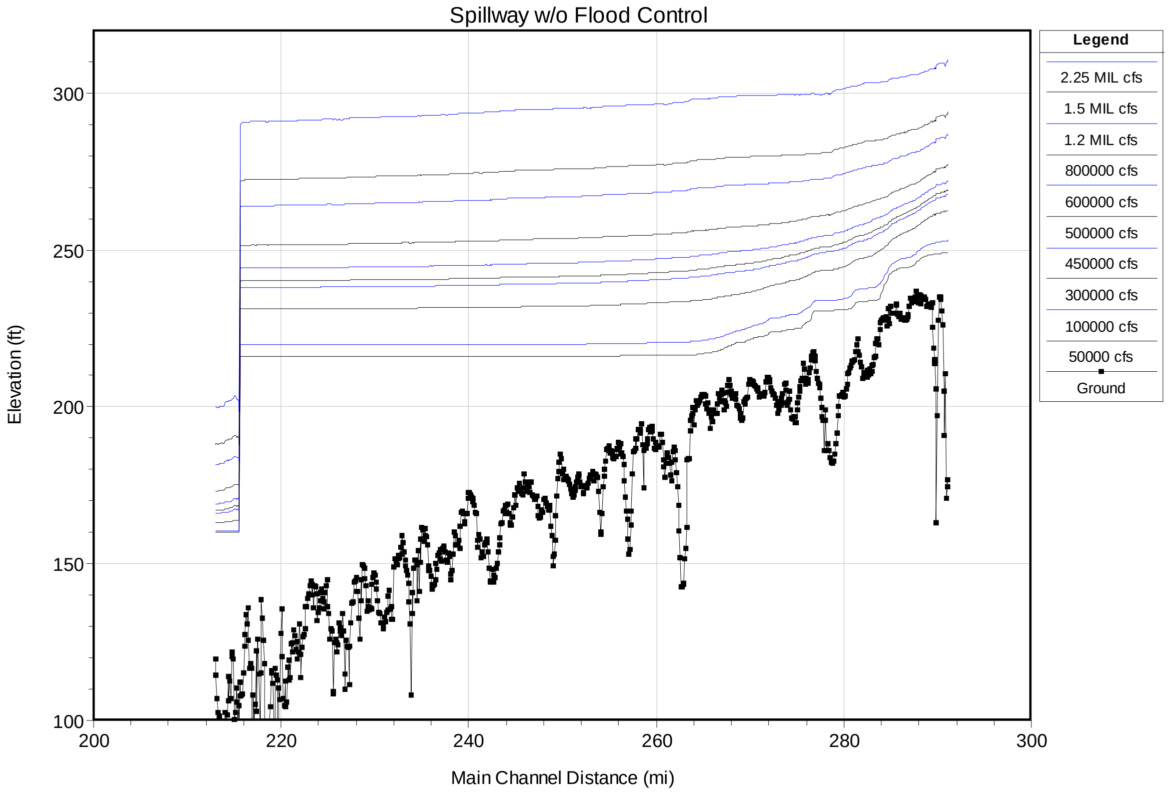


Figure 3-7: Natural River Water Surface Profiles

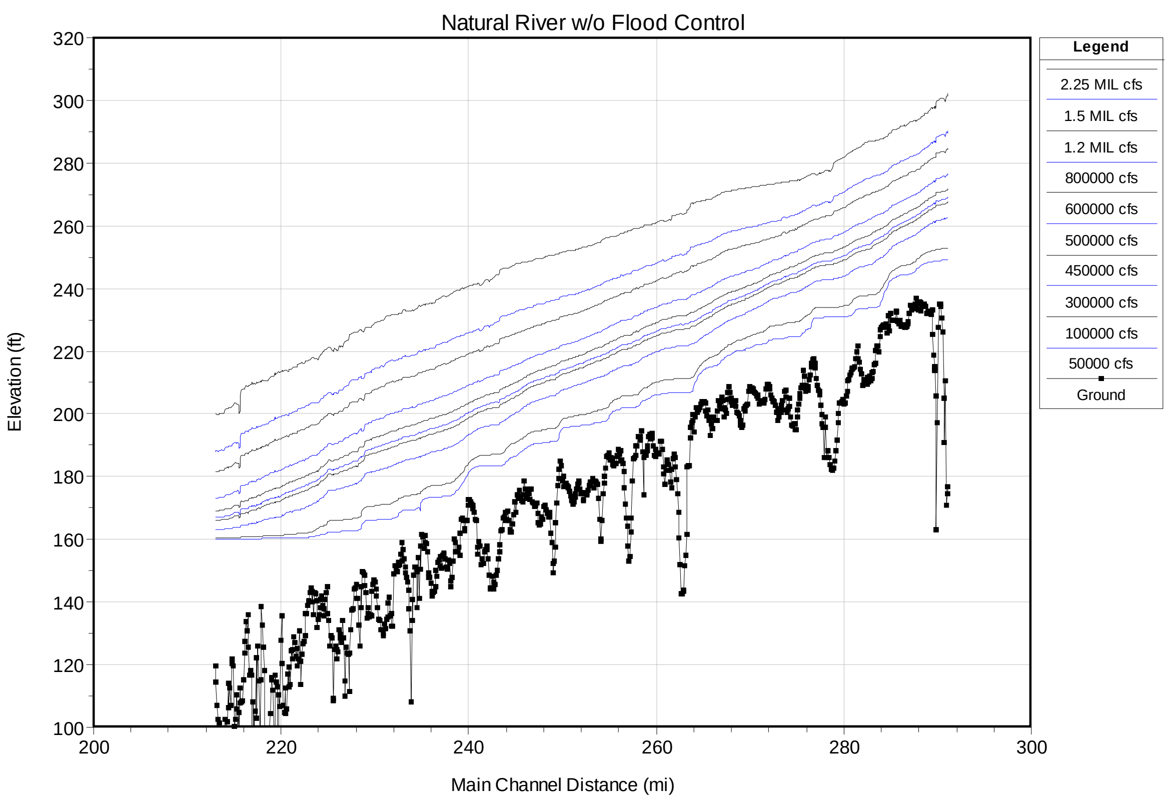


Figure 3-8: Existing Condition Channel Velocity Profiles

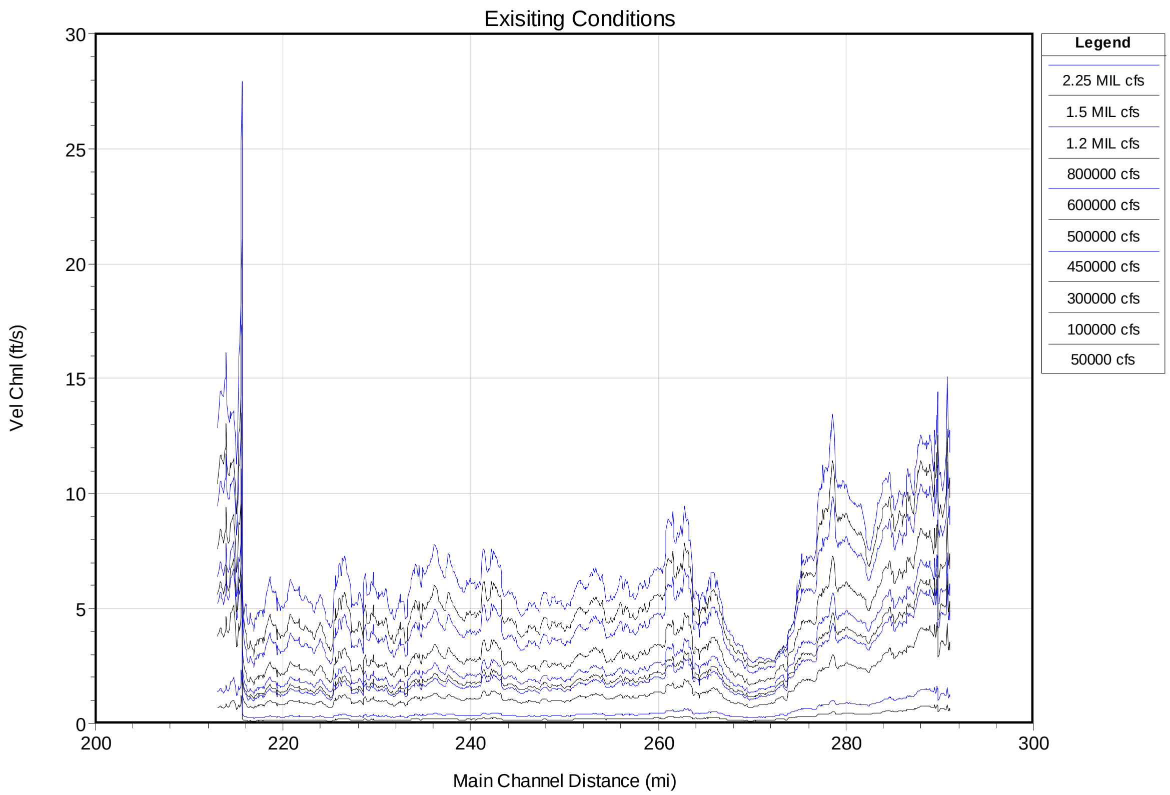


Figure 3-9: Spillway Drawdown Channel Velocity Profiles

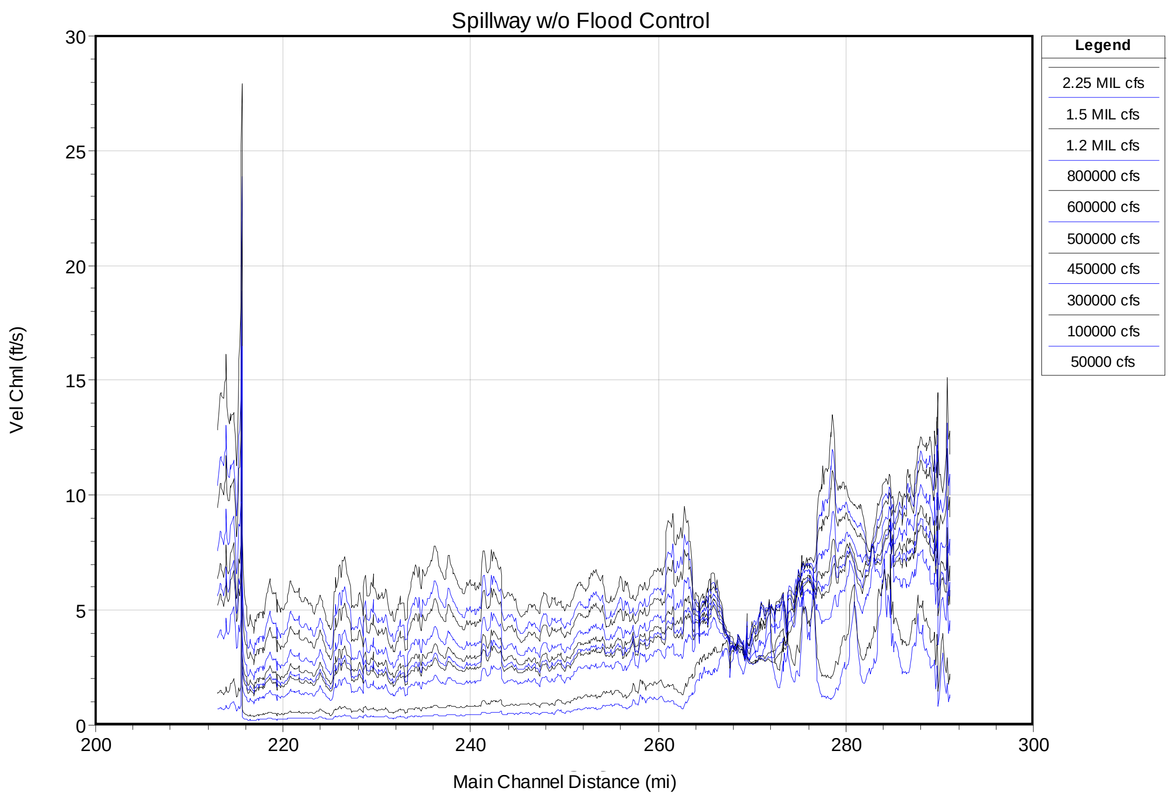


Figure 3-10: Natural River Channel Velocity Profiles

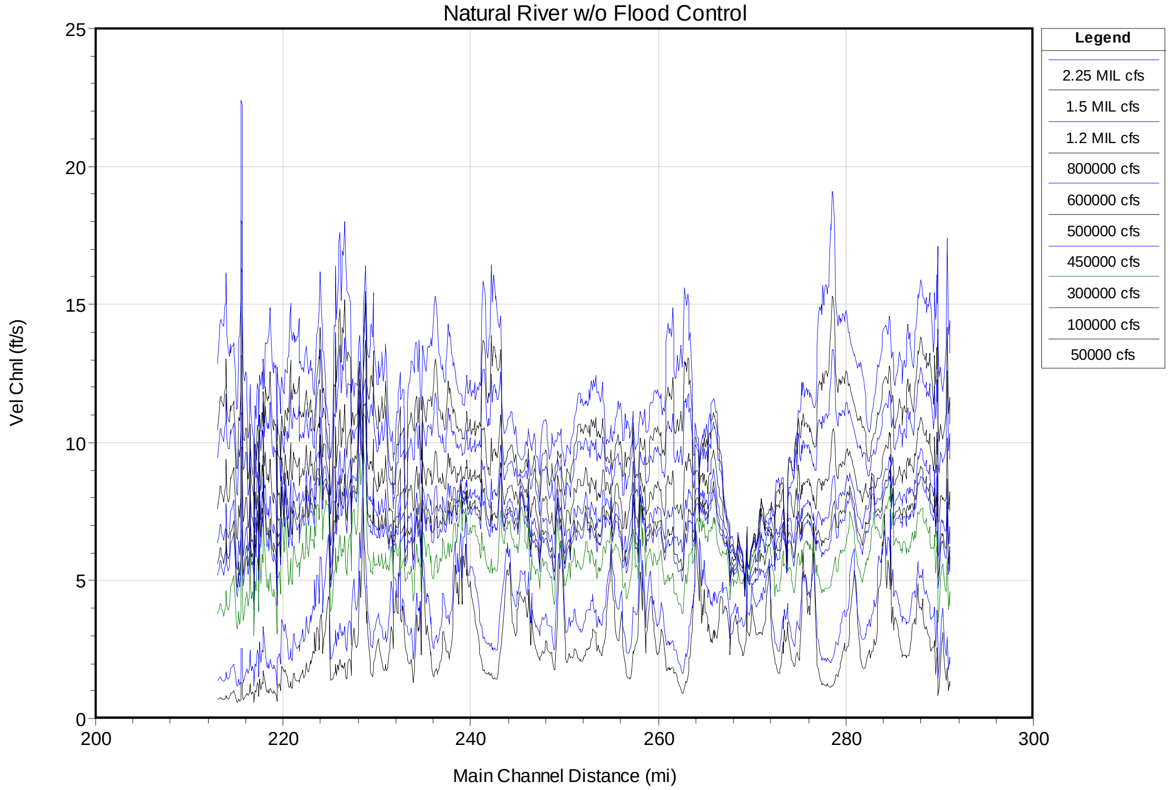


Table 3-1: John Day Dam Forebay Elevation Table

	Forebay Elevation (ft, NGVD)		
Flow (cfs)	Existing	Spillway	Natural
50,000	268	213.71	160.18
75,000	268	214.87	160.34
100,000	268	215.89	160.57
150,000	268	220.00	161.15
200,000	268	222.50	161.91
250,000	268	225.00	162.75
300,000	268	227.19	163.74
350,000	268	229.39	164.88
400,000	268	231.25	166.01
450,000	268	232.99	167.13
500,000	268	234.72	168.23
600,000	268	238.24	170.40
700,000	268	241.47	172.60
800,000	268	244.41	174.77
900,000	268	247.11	176.90
1,000,000	268	249.74	178.99
1,100,000	268	252.14	181.07
1,200,000	268	254.52	183.21
1,300,000	268	256.82	185.24
1,400,000	268	259.04	187.35
1,500,000	268	261.12	189.37
2,250,000	276.5	275.56	200.81

3.6 Time-of-Travel Estimates

Time-of-travel estimates were developed for each of the 22 flows by multiplying the average channel velocity between adjacent cross sections by the distance between them, and summing the results between the McNary tailwater (RM 291.6) to the John Day Dam. This analysis was performed for the Existing, Spillway Drawdown and Natural River conditions. The results are shown in Figure 3-11, and tabulated in Table 3-1.

Figure 3-11: Time of Travel Graph

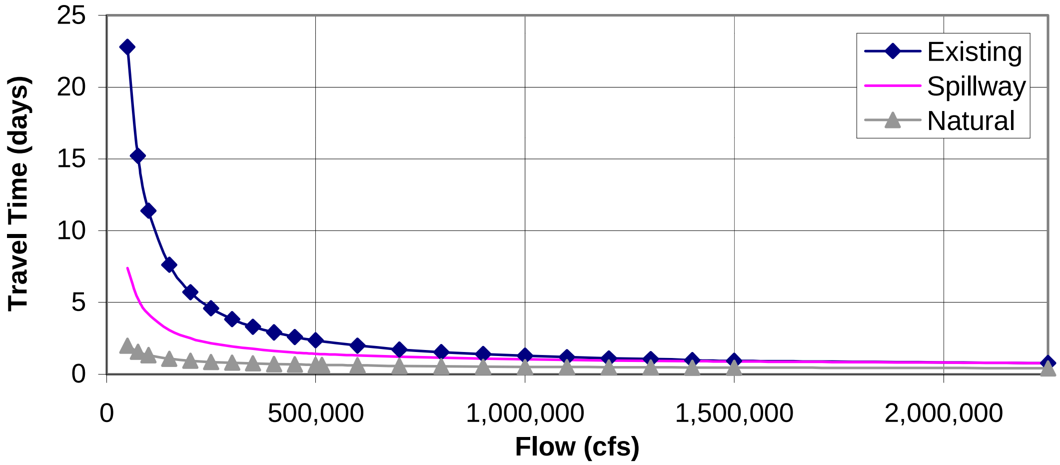


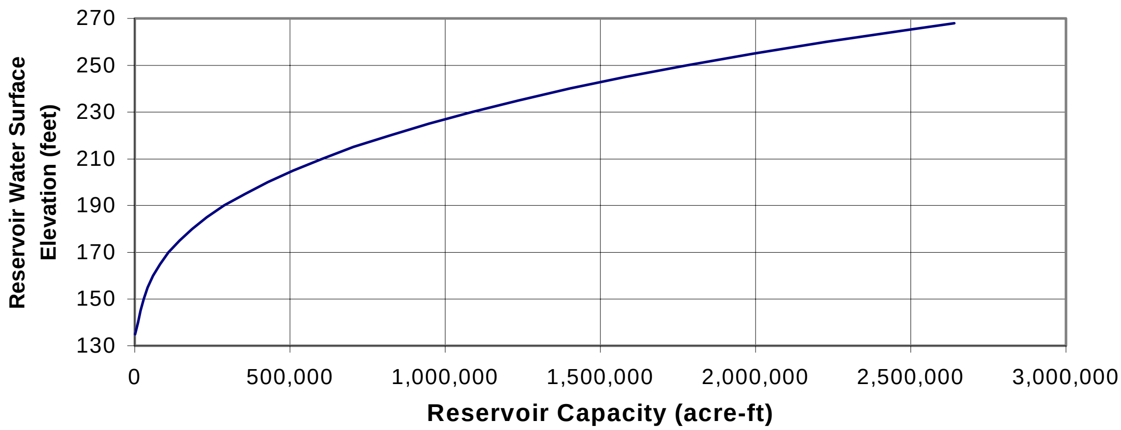
Table 3-2: Time of Travel Table

Flow (cfs)	Travel Time (days)		
	Existing	Spillway	Natural
50,000	22.8	7.4	2.0
75,000	15.2	5.3	1.6
100,000	11.4	4.2	1.3
150,000	7.6	3.1	1.1
200,000	5.7	2.5	0.94
250,000	4.6	2.2	0.86
300,000	3.8	1.9	0.80
350,000	3.3	1.8	0.75
400,000	2.9	1.6	0.71
450,000	2.6	1.5	0.68
500,000	2.4	1.4	0.65
515,000			0.65
600,000	2.0	1.3	0.61
700,000	1.7	1.2	0.58
800,000	1.5	1.1	0.55
900,000	1.4	1.1	0.53
1,000,000	1.3	1.0	0.51
1,100,000	1.2	1.0	0.50
1,200,000	1.1	0.97	0.49
1,300,000	1.0	0.95	0.48
1,400,000	1.0	0.92	0.47
1,500,000	0.95	0.90	0.46
2,250,000	0.79	0.79	0.42

3.7 Elevation-Storage and Reservoir Storage-Capacity Curves

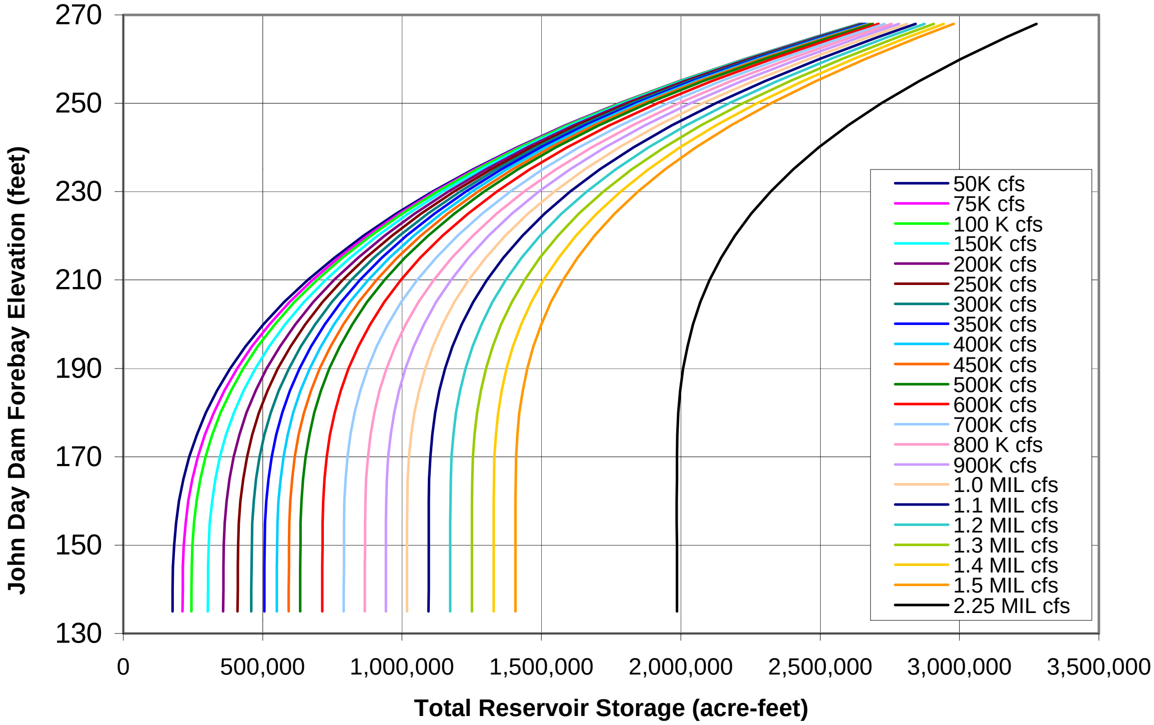
An elevation-storage curve (assuming a flat pool) was developed for the John Day Pool (Figure 3-12). The curve was developed at 5-ft increments from elevation 135 ft NGVD (the invert of the "Natural River" channel) to elevation 268 ft NGVD (the maximum existing pool elevation). For each elevation, the "active" storage was assumed to be the volume downstream of the first intersection of that elevation with the invert of the Columbia River.

Figure 3-12: Elevation – Storage Curve



A reservoir storage-capacity curve, for a given discharge, defines the volume of water upstream of a location for various downstream water surface elevations specified at that location. The upstream volume thus represents both the storage in a flat pool (elevation-storage) plus the volume stored about this elevation due to the slope of the water surface necessary to convey flow. The reservoir storage-capacity curves upstream of the John Day Dam location are shown in Figure 3-13.

Figure 3-13: Reservoir Storage – Capacity Curves



4. UNET MODEL OF JOHN DAY POOL

4.1 Development of UNET Model

HEC-RAS, described in Section 3, is a steady-state model, and as such, cannot examine the passage of hydrologic events (such as flood hydrographs) through the John Day reservoir. Therefore, we also developed an unsteady flow model of the reservoir, using UNET (HEC, 1995). UNET is similar, and fairly compatible with other Hydrologic Engineering Center (HEC) models, in that it uses their input data style and can read from and write to a standard Data Storage System (DSS) database.

UNET models of the Existing, Spillway Drawdown and Natural River Conditions were developed by first exporting each HEC-RAS model into the older model, HEC-2, and then using a HEC-2 –to– UNET conversion program. At this point, the geometries are identical, and only the time-varying information needs to be added. The models were configured with the same rating curve downstream at Rufus (RM 213.0) and the same internal rating curve at the dam for the Spillway Drawdown condition. At the upstream boundary, flows from McNary Dam were specified as a time varying hydrographs. Four major tributary inflows from the John Day River, Rock Creek, Willow Creek, and Umatilla River were specified using average discharges for each year that was modeled.

4.2 Model Calibration and Sensitivity

UNET models for Existing, Spillway Drawdown and Natural River conditions, without flood control, were first run with steady upstream inflows to compare with the results from the HEC-RAS models for consistency. It was necessary to use UNET's "added force" feature to mimic the general contraction-expansion coefficients of 0.1 and 0.3 used in HEC-RAS to simulate the effects of variations in cross section geometries. Some comparisons of the results for Spillway Drawdown and Natural River conditions are shown in Figures 4-1 and 4-2 respectively.

Figure 4-1: RAS vs. UNET Comparison for Spillway Drawdown Condition

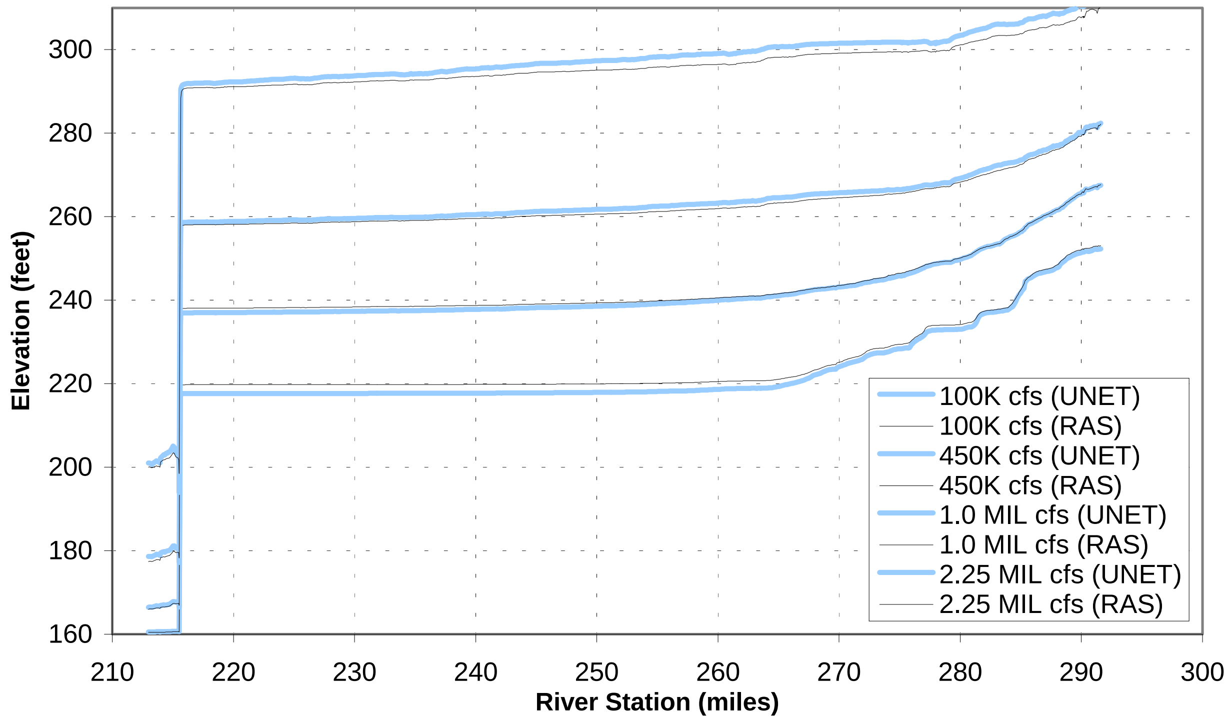
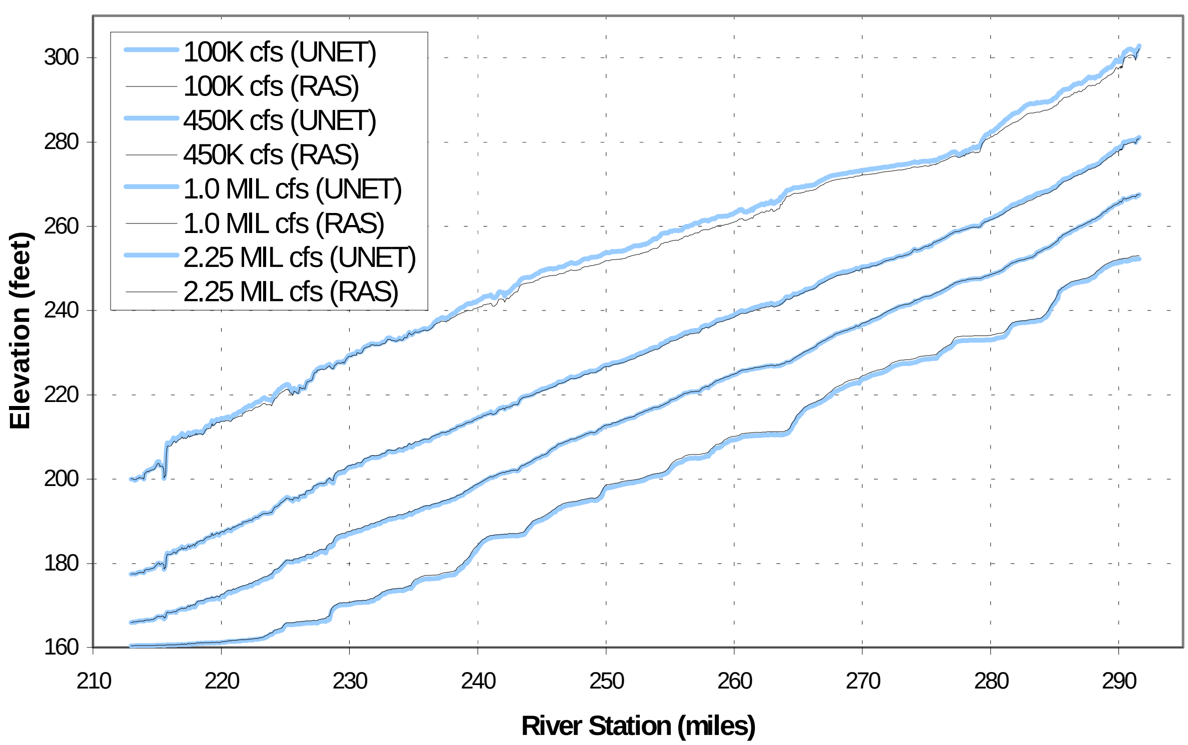


Figure 4-2: RAS vs. UNET Comparison for Natural River Condition



Again, there is very little data available to calibrate a UNET model of the John Day reservoir. The only data provided were observations made in a short reach of the reservoir during a high water (snowmelt) event in May 1996. Flows during this period were specified at the upstream boundary, and UNET run for two conditions – with and without the "added force" coefficients. The results are mixed. However, overall we elected to use the "added force" coefficients throughout the model.

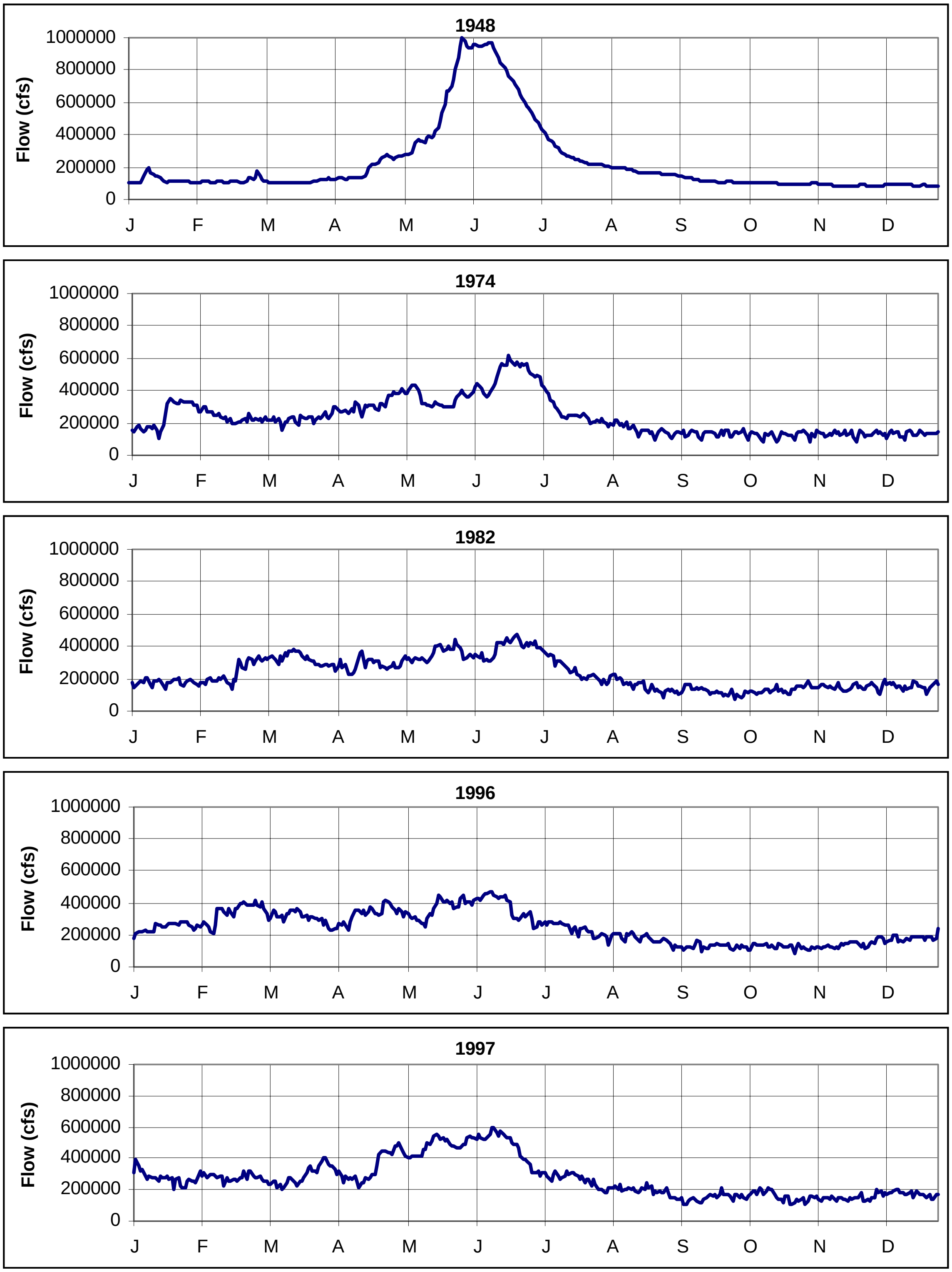
4.3 Flood Events to be Simulated and Flood Control

Five flood events were modeled using UNET:

- 1948, peak flow was 990,000 cfs
- 1974, peak flow was 614,564 cfs
- 1982, peak flow was 470,050 cfs
- 1996, peak flow was 473,000 cfs
- 1997, peak flow was 598,600 cfs

The five hydrographs are shown in Figure 4-3.

Figure 4-3: Flood Hydrographs



The five flood events were simulated for the Existing, Spillway and Natural River condition models, without flood control, specifying the upstream flow hydrographs for each event and constant (annual-average) inflows from the five major tributaries. Specifying flood control, particularly for the Natural River condition, was more complicated.

The criterion used to define “flood control” was to store 500,000 acre-ft around the peak of each flood. For each flood hydrograph at McNary Dam, a maximum flow was determined such that if excess flows were stored, the excess storage would equal 500,000 acre-feet. Then for the Spillway Drawdown condition, the rating curve at the dam’s spillway was simply modified to limit the overflow to the maximum estimated for each flood event.

As there is no spillway in the Natural River configuration, a rating curve was first developed at the location of the John Day Dam. This rating curve was then specified in the Natural River model, at the dam location, and the model re-run to ensure that the introduction of the rating curve did not significantly change the results. Once we were satisfied that the results were consistent, this rating curve was modified to limit the flow to the maximum outflow for the event simulated.

4.4 Maximum Water Surface Elevations, Flows, and Velocities

UNET can output the maximum water surface elevations, flows and velocities along the length of the channel to a DSS database. Figures 4-4 through 4-6 show results for the 1974 flood event for the Existing Condition and four alternatives modeled.

Figure 4-4: Maximum Water Surface Elevation - 1974

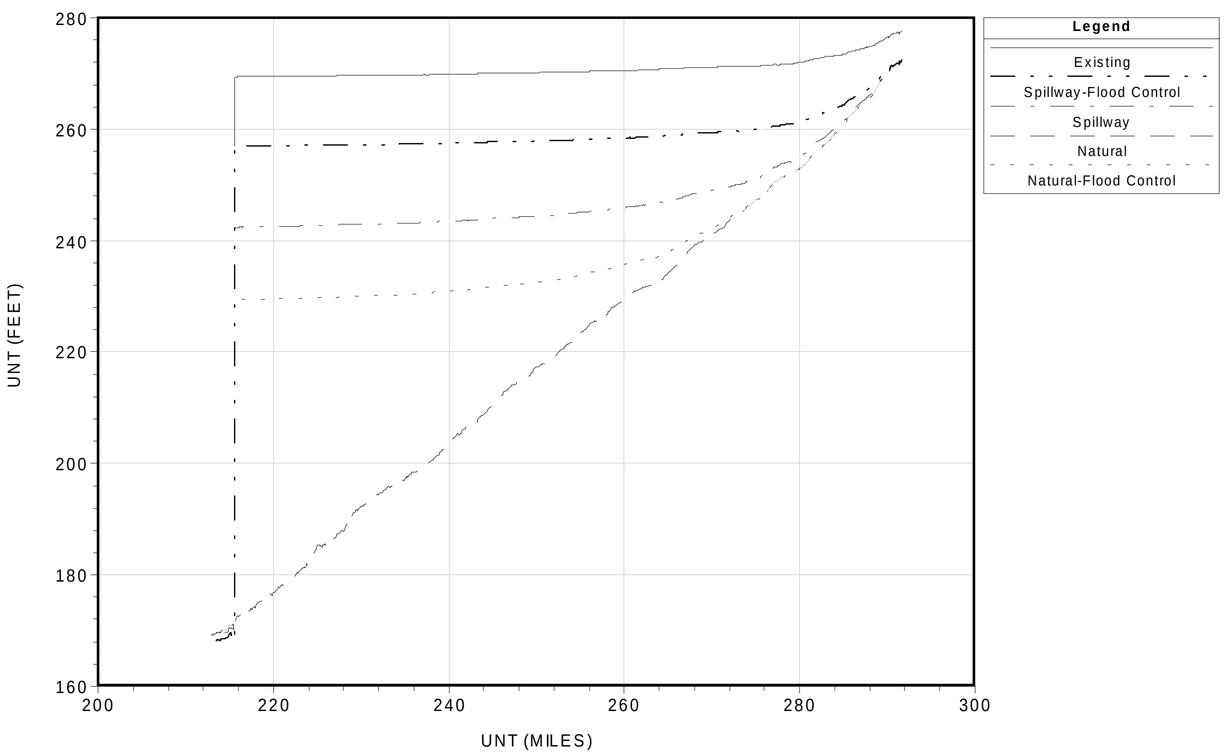


Figure 4-5: Maximum Flow - 1974

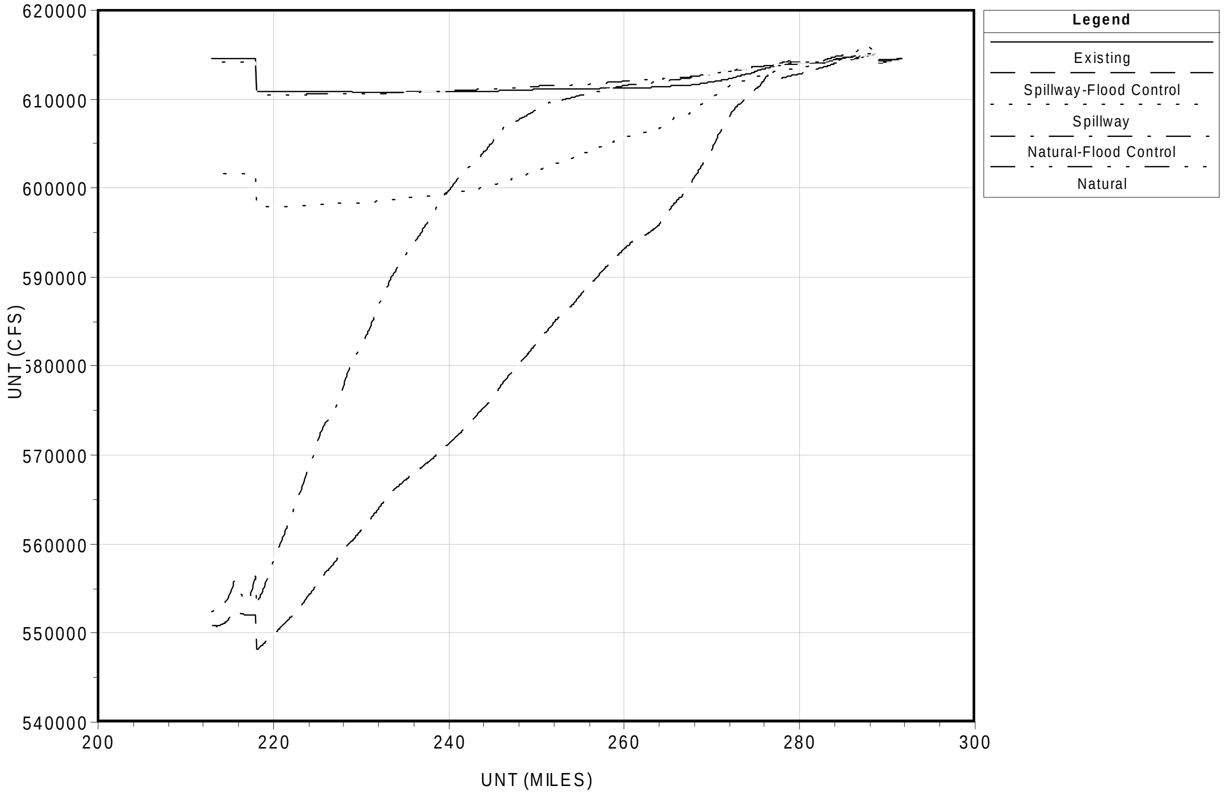
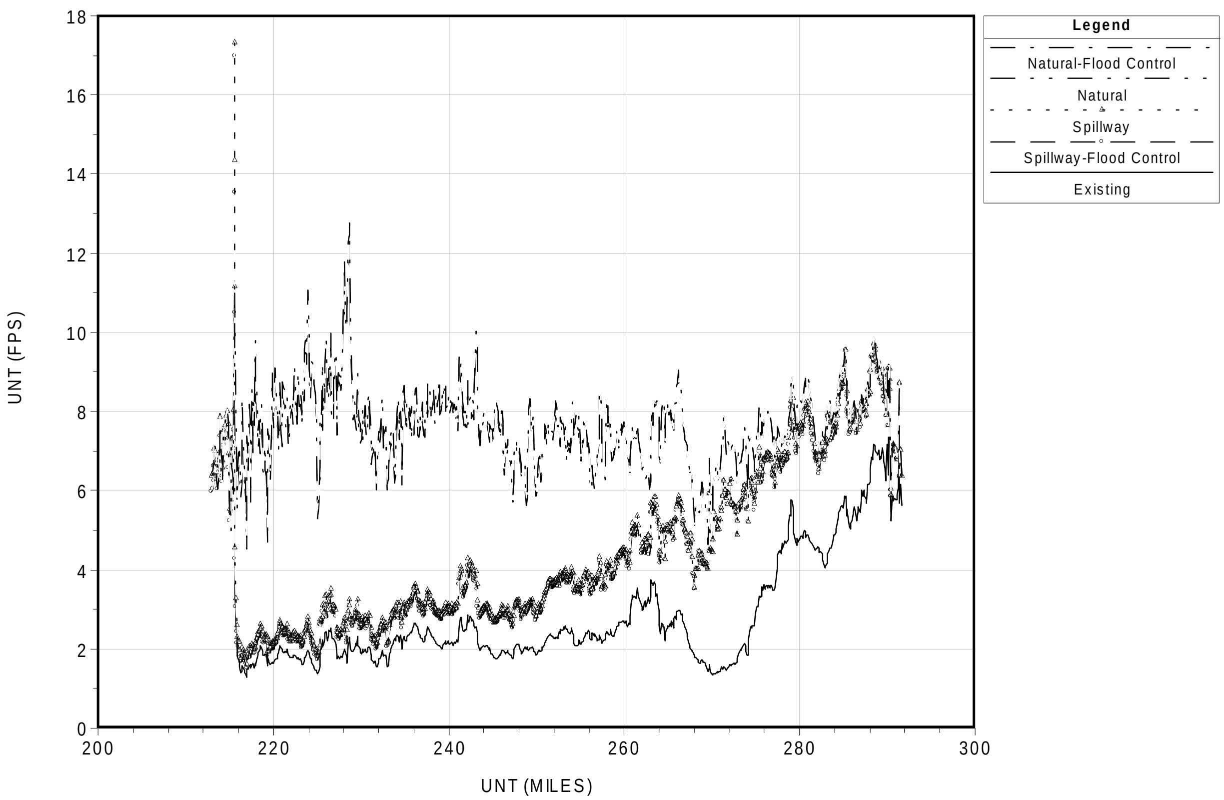


Figure 4-6: Maximum Average Channel Velocity - 1974



4.5 Time Histories of Water Surface Elevations, Flows and Velocities at Key Locations

Twelve key locations, including confluence of tributary streams, ports, and cities, were defined for time history output. The locations in downstream order include (1) McNary Dam Tailwater, (2) Umatilla River Confluence, (3) City of Irrigon, (4) Hogue-Warner Elevator, (5) Port of Morrow, (6) Boardman Marina, (7) Willow Creek Confluence, (8) Wood Gulch Confluence, (9) City of Arlington, (10) Rock Creek Confluence, (11) John Day River Confluence, and (12) John Day Dam Forebay. Time history output for 1974 at the John Day River Confluence and the Port of Morrow, are shown as examples in Figures 4-7 through 4-12.

Figure 4-7: 1974 Flow Hydrographs at Port of Morrow

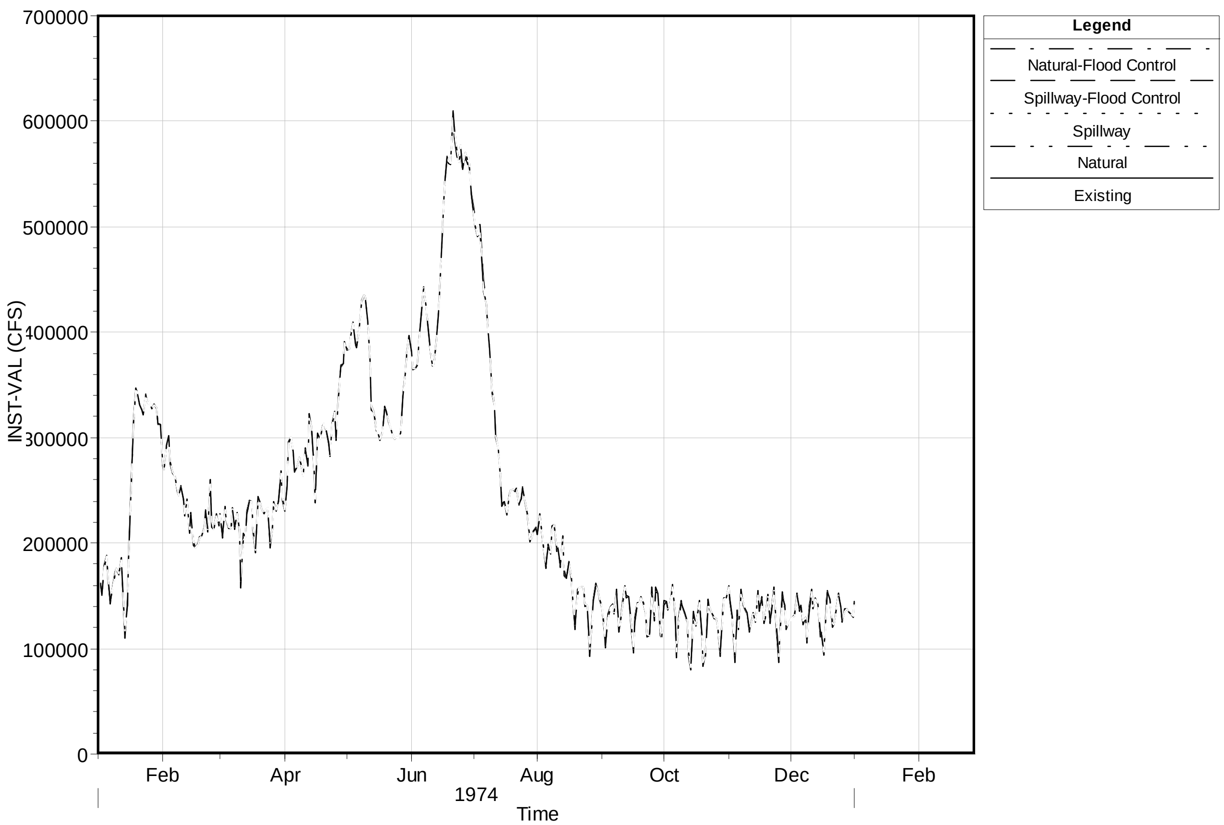


Figure 4-8: 1974 Flow Hydrographs at John Day River Confluence

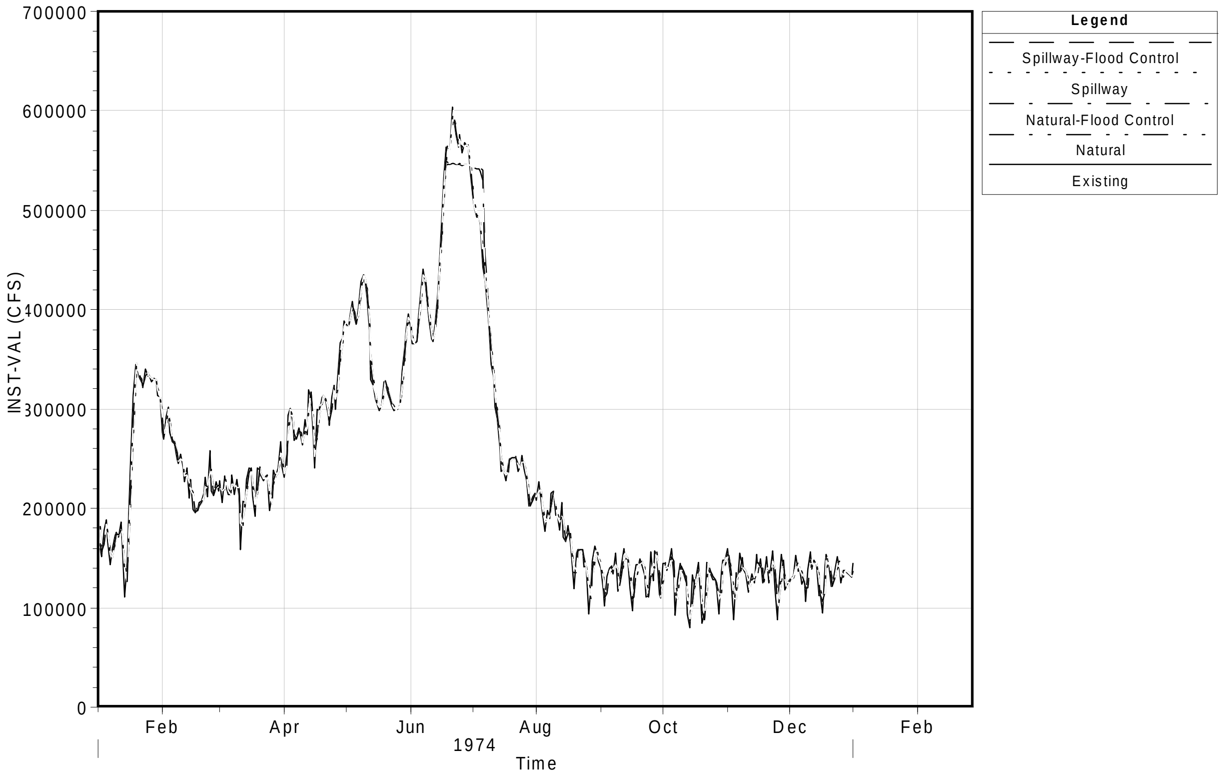


Figure 4-9: 1974 Stage Hydrographs at Port of Morrow

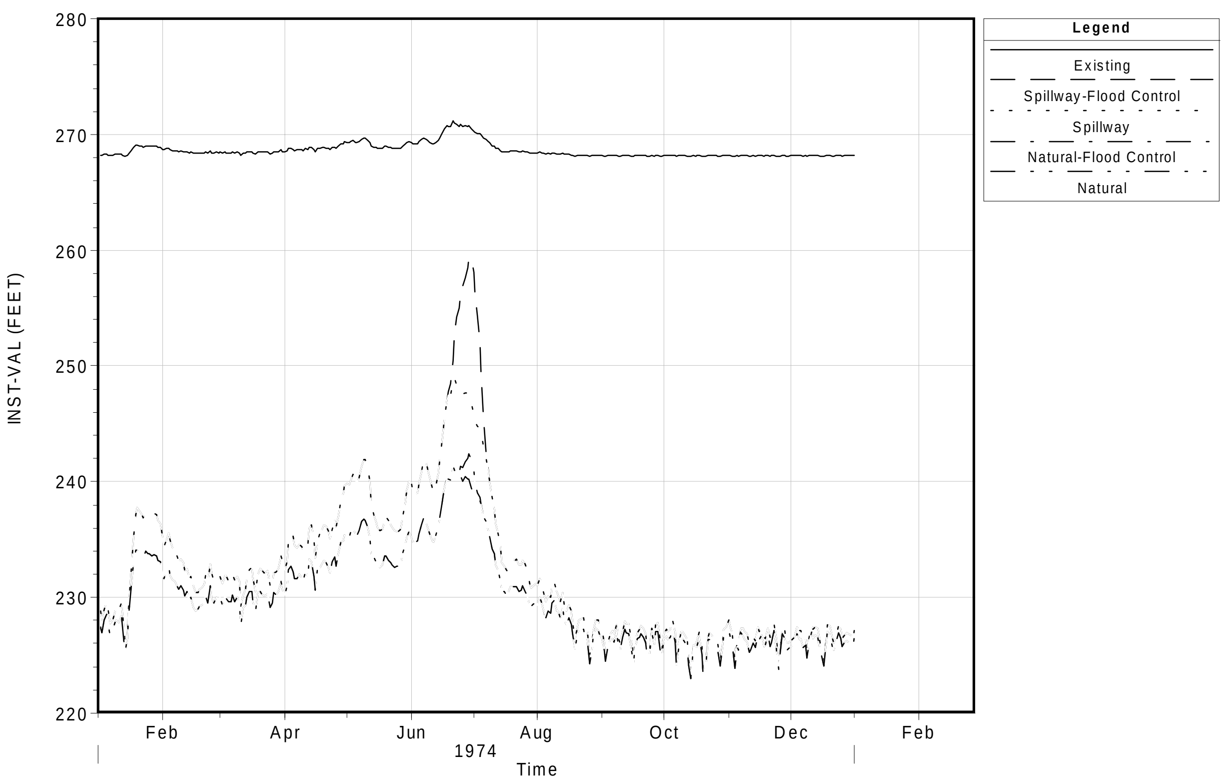


Figure 4-10: 1974 Stage Hydrographs at John Day River Confluence

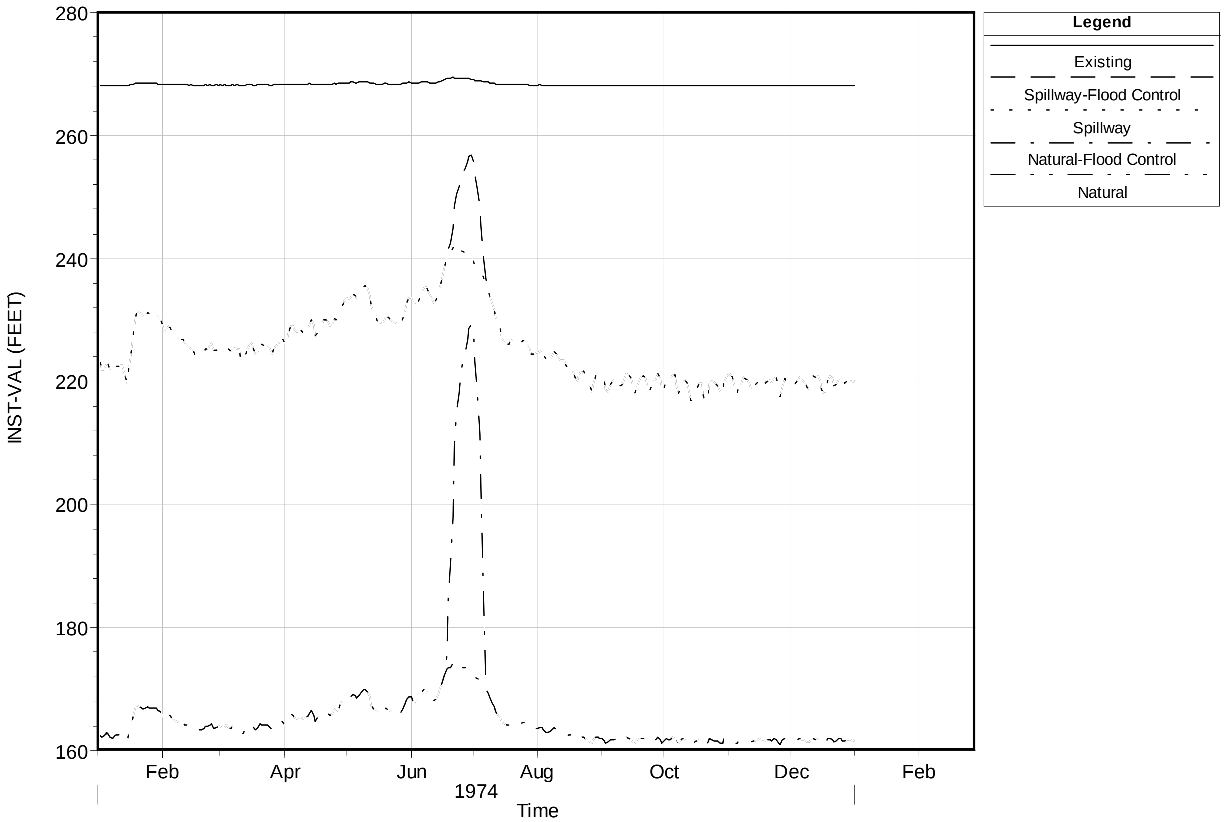


Figure 4-11: 1974 Velocity Hydrographs at Port of Morrow

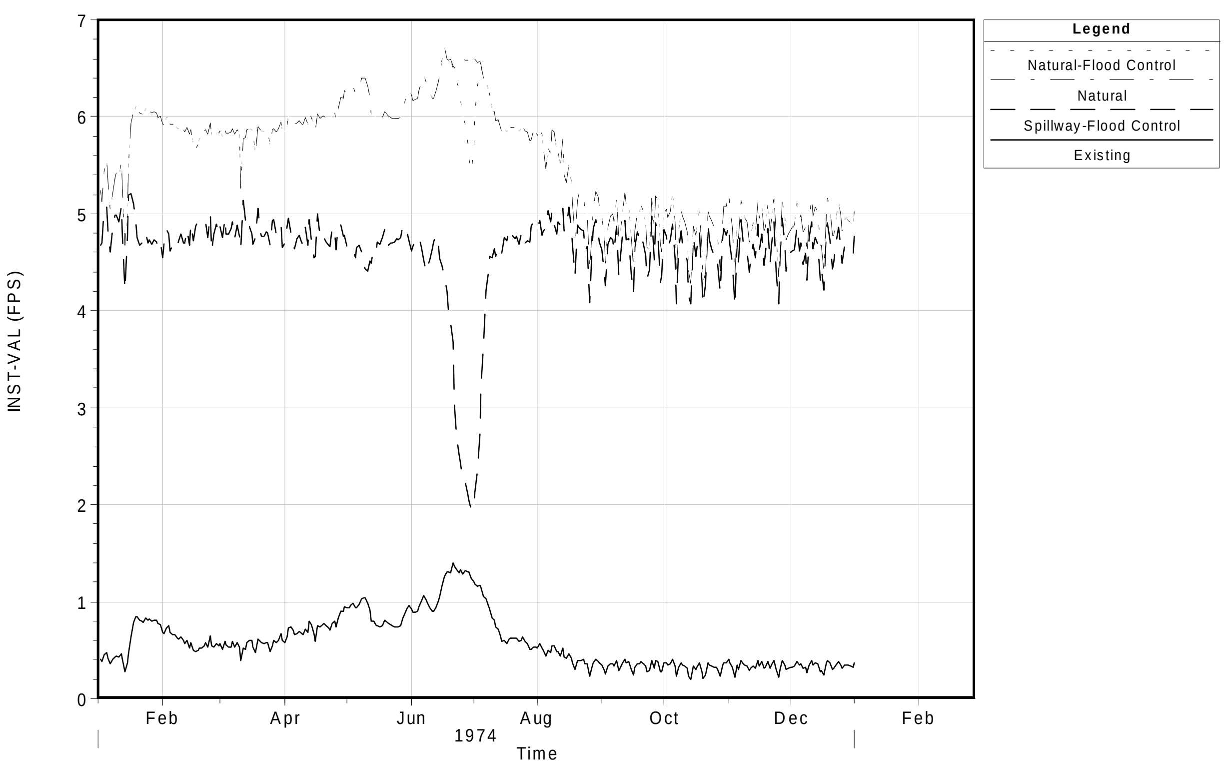
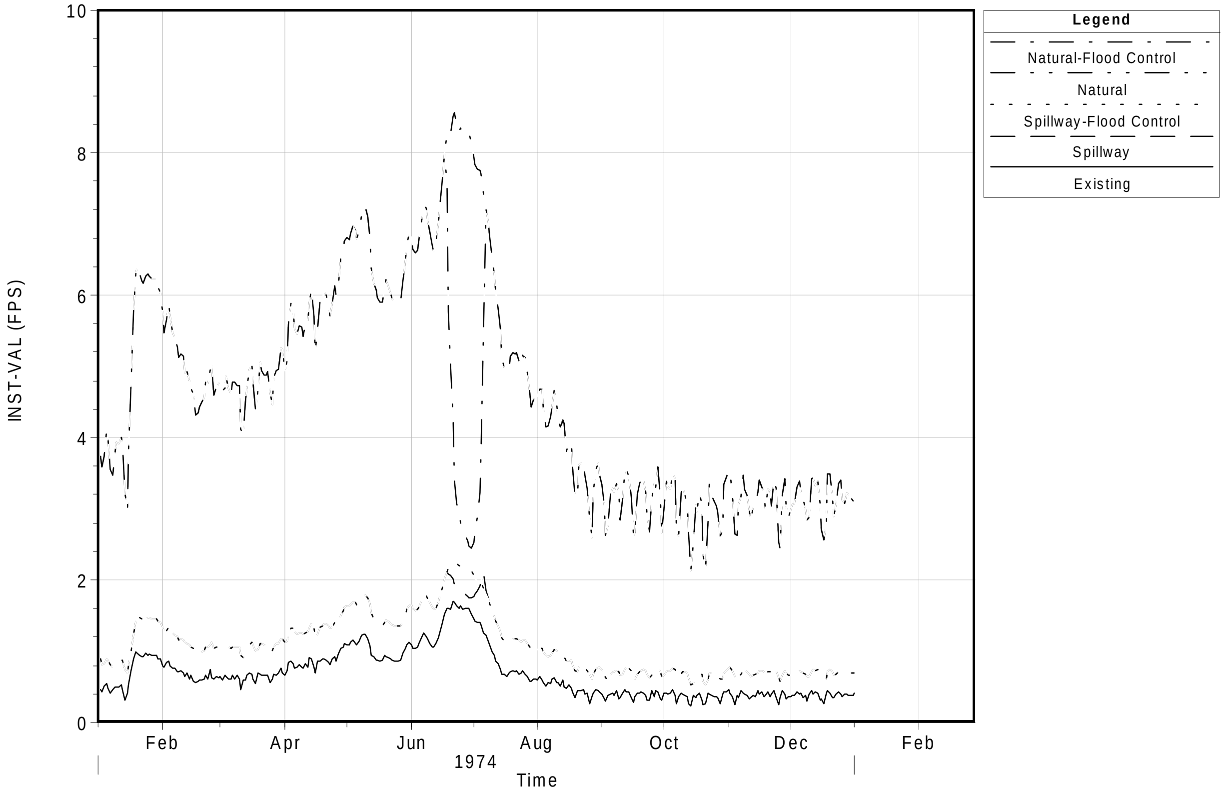


Figure 4-12: 1974 Velocity Hydrographs at John Day River Confluence



4.6 Rating Curves at Key Locations

Rating curves were developed using the 1997 event for three location: (1) the confluence with the Umatilla River (Figure 4-13), (2) the Port of Morrow (Figure 4-14) and (3) Arlington (Figure 4-15). Analyses of other flood events show that the rating curves are consistent, and not a function of the flood event used to generate them.

Figure 4-13: Rating Curve at Umatilla River - 1997

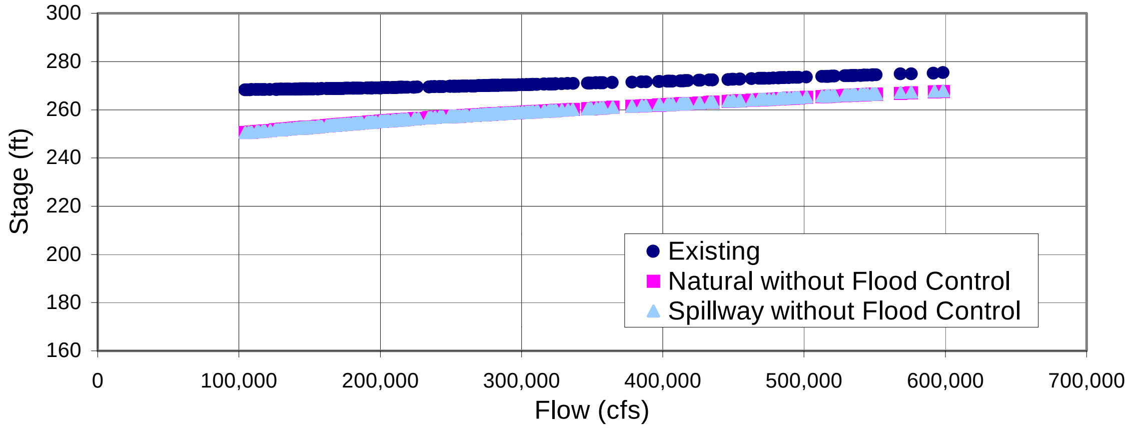


Figure 4-14: Rating Curve at Port of Morrow - 1997

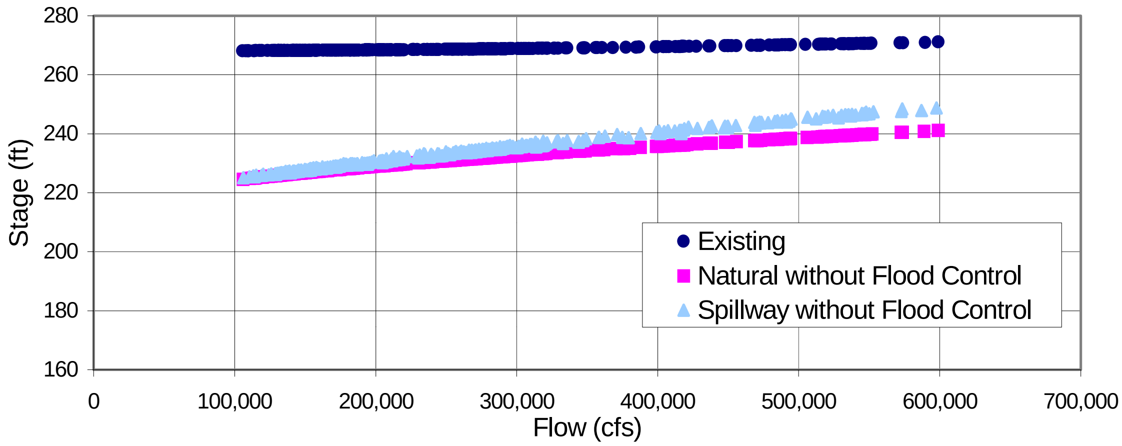
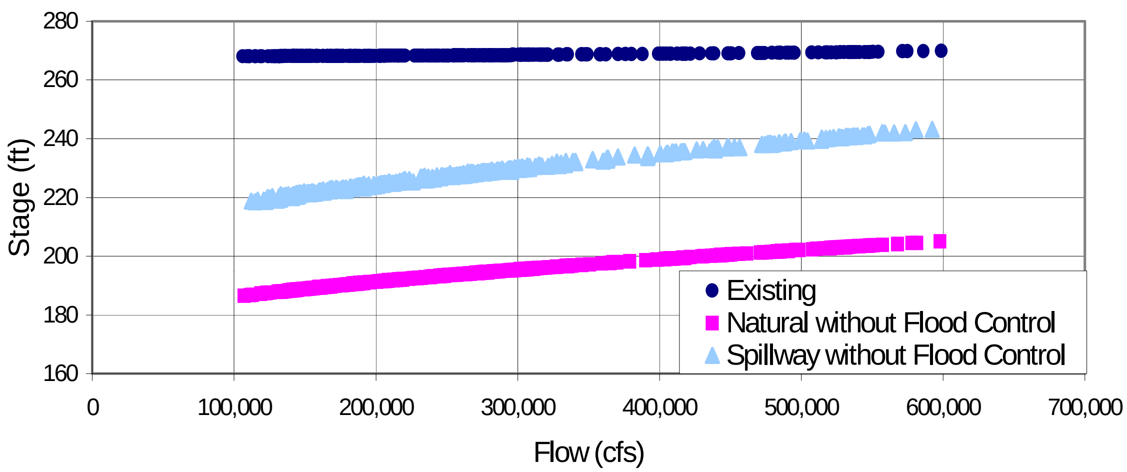


Figure 4-15: Rating Curve at Arlington - 1997



These figures show that the rating curve is essentially single valued, without a “loop” that can be seen on some rivers that denotes the rising and falling limb of the flood hydrograph. The figures only show alternatives without flood control, because flood control tends to yield a range of stages for the same maximum (or near-maximum) flows, as the John Day spillway is only permitted to discharge the maximum prescribed flow while storing an excess.

4.7 Flood Travel Times

Flood hydrographs computed at the McNary Dam Tailwater (RM 291.6) and the John Day Dam, for the five flood events, were used to compute the time of travel of the peak flow. Figures 4-16 through 4-20 show various flood hydrographs around the peak discharge along the length of the reservoir for the Existing conditions, and the four alternatives (with and without flood control) modeled. Table 4-1 summaries the travel times determined from analyzing the model results.

Table 4-1: Travel Time Summary

	Time (hours)				
	Existing	Natural	Spillway	Natural with Flood Control	Spillway with Flood Control
1948					
Port of Morrow	3	3	3	3	6
Arlington	8	9	21	8	16
John Day Dam	9	12	27	13	24
1974					
Port of Morrow	3	3	3	3	5
Arlington	5	8	12	8	17
John Day Dam	6	11	20	12	27
1982					
Port of Morrow	1	1	1	3	4
Arlington	2	4	3	8	20
John Day Dam	3	7	7	14	30
1996					
Port of Morrow	1	3	3	2	4
Arlington	3	8	10	7	15
John Day Dam	4	11	18	14	26
1997					
Port of Morrow	1	1	1	3	3
Arlington	2	4	4	8	19
John Day Dam	3	7	9	12	35

Figure 4-16: 1996 Flood Peak Hydrographs for Existing Condition

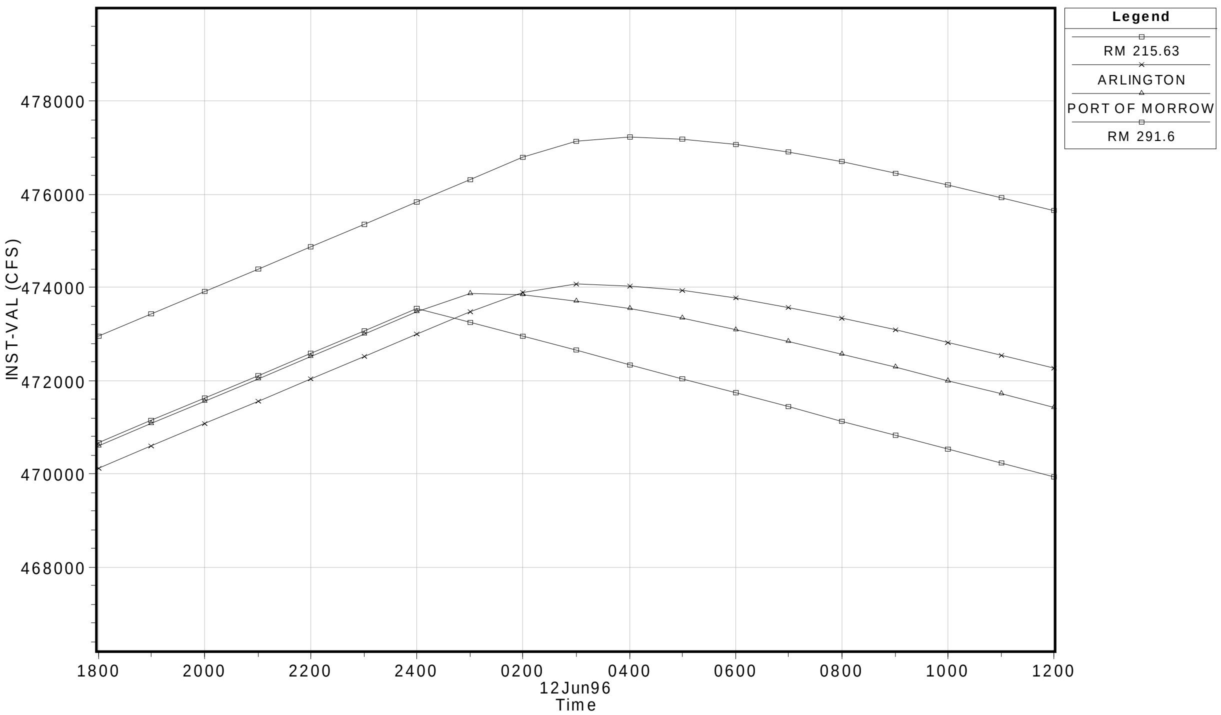


Figure 4-17: 1996 Flood Peak Hydrographs for Natural River

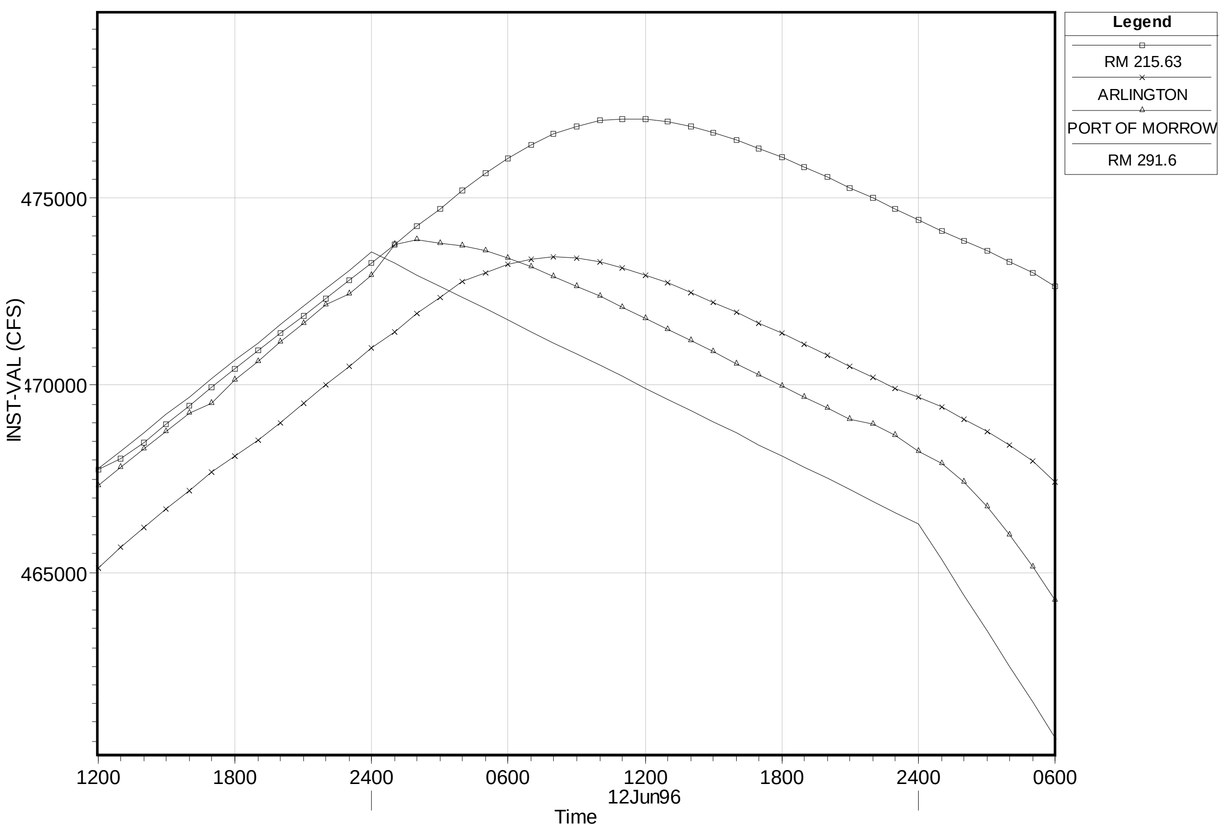


Figure 4-18: 1996 Flood Peak Hydrographs for Spillway Drawdown

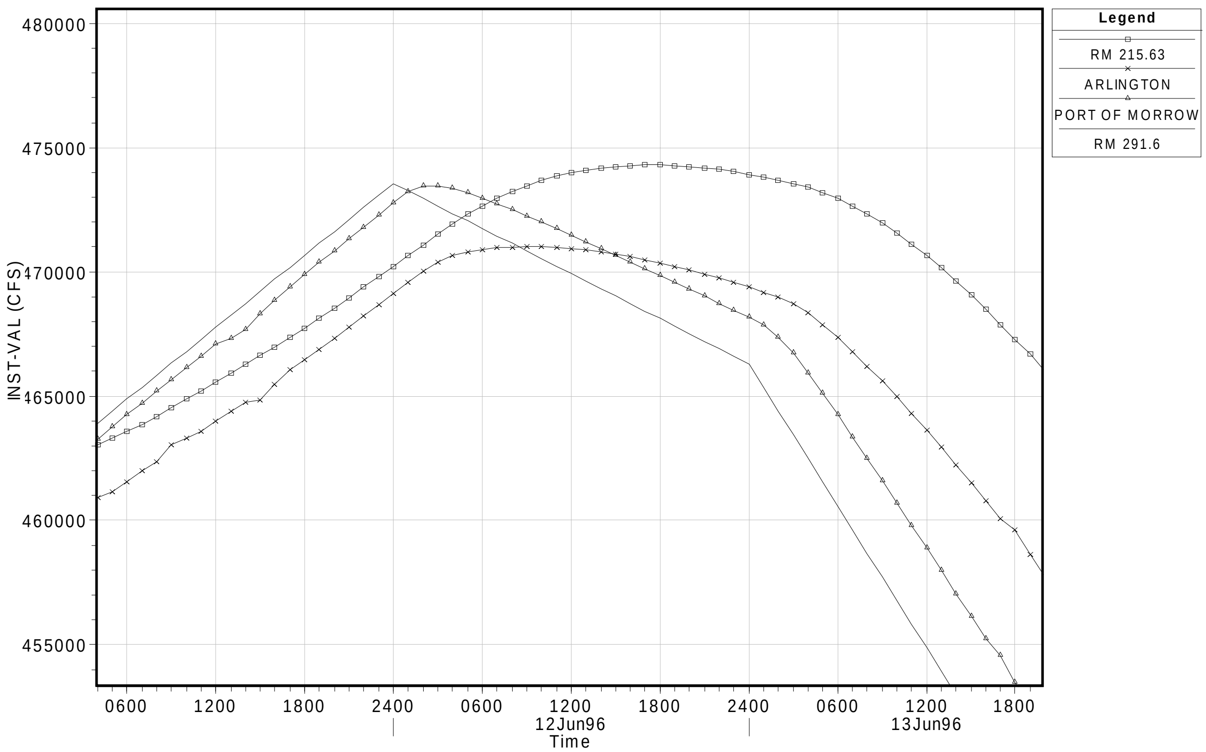


Figure 4-19: 1996 Flood Peak Hydrographs for Natural River with Flood Control

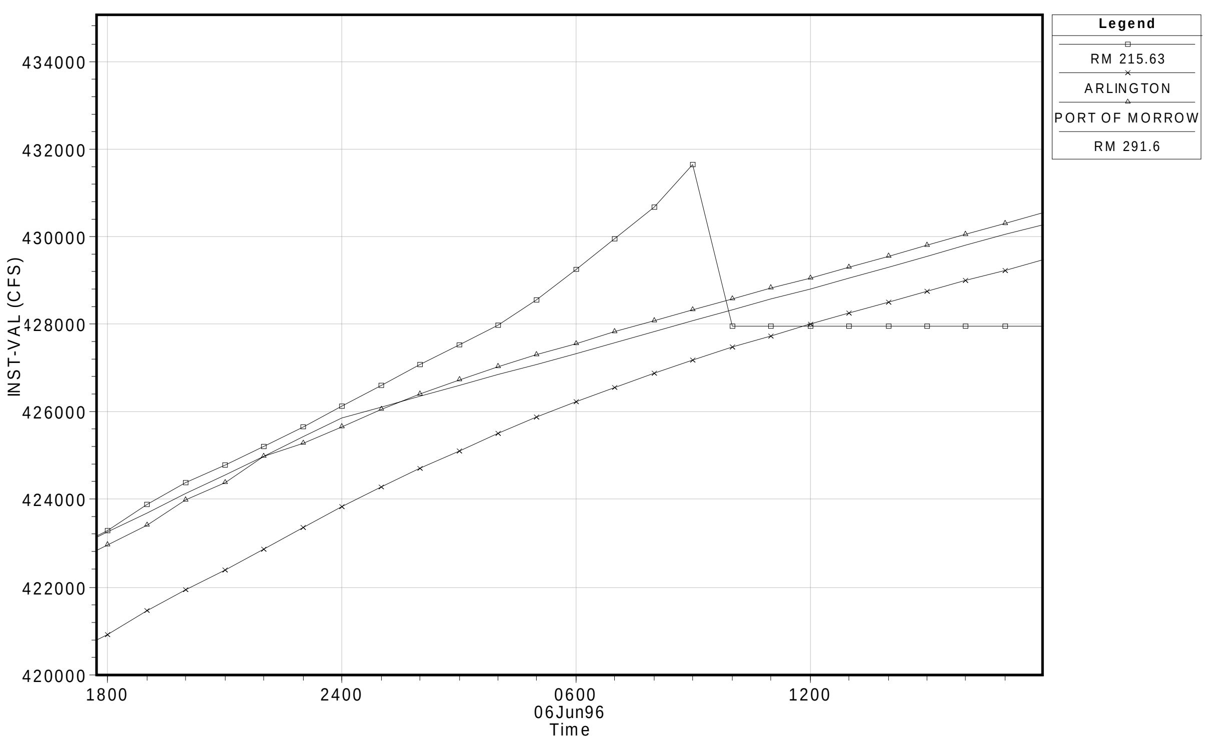
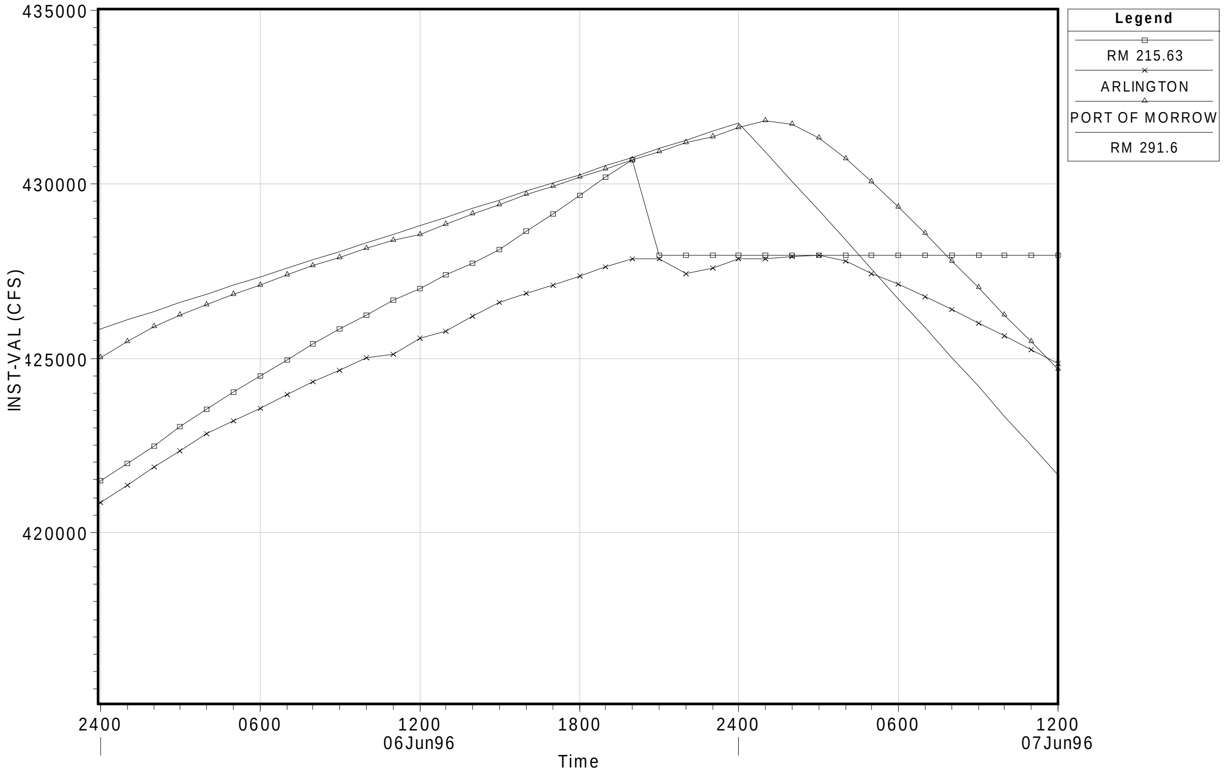


Figure 4-20: 1996 Flood Peak Hydrographs for Spillway Drawdown with Flood Control



5. ANALYSIS OF RESULTS

5.1 Flood Control Operations for Existing Conditions

John Day Dam is the largest of the lower Columbia River Dam and Reservoir projects, and is the only one with allocated flood control space. It provides final regulation of flood peak flows. In addition to flood regulation, the project is operated to provide optimum conditions for navigation and hydroelectric power without creating unnecessary detriment to fish passage, recreation, and other project users.

The primary damage control point for the lower Columbia River and indeed, the entire Columbia River Basin, is the Portland-Vancouver area. Primarily Grand Coulee Dam and other large upstream projects accomplish Columbia River flood regulation. However, the remoteness of these reservoirs to the Portland-Vancouver area makes achievement of the target regulation difficult. Uncertainties in weather and timing of runoff complicate flood control operations. The approximately 500,000 acre-feet of flood control space in John Day Reservoir between elevations 257.0 and 268.0 ft, NGVD, provides an opportunity for final regulation to the target discharges for the downstream control points. The space will be evacuated during the pre-flood period, generally no further in advance than necessary for assured operation based on forecasts of seasonal runoff volumes, short-term inflow forecasts, weather outlook, power demands, and current conditions along the river. Since winter floods may develop rapidly, it is necessary to reserve the space between elevation 265.0 ft and 268.0 ft exclusively for control of sudden winter floods. When the potential of heavy runoff is evident additional space below elevation 265.0 ft will be provided.

5.2 Embankment Riprap Analysis

The U.S. Army Corps of Engineers method (USCOE, 1994) of riprap placement was developed for flow in man-made or natural channels having low turbulence and slopes of less than two percent. The following equation is used with the USCOE method:

$$D_{30} = S_f C_s C_v C_t d \left\{ \left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{0.5} \frac{V}{\sqrt{K_1 g d}} \right\}^{2.5} \qquad (1)$$

where:

- D_{30} = stone size, feet
- S_f = safety Factor
- C_s = stability coefficient for incipient failure
  = 0.30 for angular rock (for rounded rock, increase safety factor)
- C_v = vertical velocity distribution coefficient
  = 1.0 for straight channels, inside of bends
  = 1.283-0.2log(R/W), outside of bends (1 for R/W>26)
- Ct = thickness coefficient
  = 1.0 for thickness = 1*D_{100}(max) or 1.5*D_{50}(max) whichever is greater

d = local depth of flow at same location as V, feet

$(_s$ = unit weight of stone, lbs/ft^3

$(_w$ = unit weight of water, lbs/ft^3

V = local depth averaged velocity, V_{ss} for side slope riprap, ft/s

g = gravitational constant, ft/sec^2

K_1 = side slope correction factor (estimated from a graph [in USCOE, 1994] using the sideslope and the angle of repose of the riprap material)

The USCOE method was incorporated into a computer program, called RIPRAP, developed by WEST Consultants (1996) and supported by the American Society of Civil Engineers (ASCE). The USCOE method can use either “local” depth and velocity, or channel-average values which are then internally converted to “local” values.

Assuming a small average-channel depth of 10 feet (see Figure 3-7) and a very high average-channel velocity of 15 ft/sec (see Figure 3-10) in a straight “natural” channel, a factor of safety of 1.2, a stone of 165 lb/ft^3, and a side slope of 2:1, the USCOE equation gives a D_{30} riprap size of 1.46 feet. Using $D_{50}= D_{30}/0.82$, the median, D_{50}, riprap size is approximately 1.78 feet.

5.3 Time of Travel

One of the crucial questions addressed by the study is how much the various alternatives change the travel time of flows through the John Day reservoir. The concern is that long travel times, representing a relatively quiescent reservoir, are a barrier to downstream fish migration. The results from the steady-state HEC-RAS model are shown in Figure 3-11 and Table 3-1 for the Existing, Spillway Drawdown, and Natural River conditions.

The results show significant differences for flows less than the two-year flow of 353,000 cfs. The Natural River condition gives travel times on the order of one day (perhaps to 1½-2 days for very low flows). By contrast, the Spillway Drawdown condition could vary from more than a week at 50,000 cfs to nearly 2 days at the two-year flow, and the Existing condition varies from more than three weeks at 50,000 cfs to over three days at the two-year flow. For large flows, in excess of the two-year flow, travel times would be relatively short, on the order of 1-2 days.

In general, the Spillway Drawdown would tend to decrease travel times by about a multiple of 2-3 at these lower flows, whereas the Natural River condition would tend to decrease travel times by a multiple of 4-10.

5.4 Flood Warning Analysis

Flood warning was considered in two ways. First, the peak flows of the five flood hydrographs were tracked through the reservoir, and the differences from the time of the peak flow "observed" at McNary Dam noted. The second method was to identify a flood warning "trigger" flow, in this case the two-year flow of 353,000 cfs at the John Day Dam, and track the times of occurrence of the two-year flow as it passed through the reservoir.

The concept of a flood warning, or "trigger", flow was used for two reasons. First, it was found that tracking the flood peak resulted in large differences between the individual flood events, probably caused by the shape of the hydrograph near the peak. Second, realistic flood warning tends to be based on some indicator flow that suggests an impending threat of flooding if it is exceeded.

Figures 4-16 through 4-20 show the 1996 flood peaks for various locations in the John Day reservoir for the Existing, Spillway Drawdown, and Natural River conditions, with and without flood control. Table 4-1 shows the times of peak flow travel from McNary Dam to these same locations. The results show a number of interesting things. First, the travel times through the upper (McNary to Port of Morrow) and lower (Arlington to John Day Dam) thirds of the reservoir tend to be similar, with the travel time through the middle third (Port of Morrow to Arlington) being about twice as long. Second, while the travel times for the Natural River condition are generally longer than for the Existing condition, travel times for the Spillway Drawdown conditions are often significantly longer. However, as can be seen in Table 5-1, there is considerable variability between flood events, which could be related to the shapes of the hydrographs near the event peaks.

Table 5-1 shows travel times determined by tracking the occurrence of the two-year flow through the John Day reservoir. The results show much the same trends as before, however they are more consistent between events, and we believe provide a more accurate estimator of flood warning times through the reservoir. Where the Existing condition might have a two-year-flow travel time of 3.5 hours, the Natural River condition would increase this time by about a multiple of 3. However, the Spillway Drawdown condition would increase this time by a multiple of 5 or more, and seems to depend strongly on the rate of rise of the flood event (see the second column of Table 5-1). The more quickly the flood rises, the more quickly the two-year flow travels through the reservoir.

Table 5-1: 2-Year Flow Travel Time from McNary Dam

	Slope (cfs/hour)	Time (hours)				
		Existing	Natural	Spillway	Natural with Flood Control	Spillway with Flood Control
1948						
Port of Morrow	1181	2	3	4	3	4
Arlington		3	8	15	8	15
John Day Dam		3.5	11.5	22	11.5	22
1974						
Port of Morrow	764	1.5	3	3.5	3	3.5
Arlington		2.5	7.5	14	7.5	14
John Day Dam		3.5	11	21.5	11	21.5
1982						
Port of Morrow	2917	1.5	3	3	3	3
Arlington		3	7	8	7	8
John Day Dam		3.5	9	15	9	15
1996						
Port of Morrow	1597	1.5	3	3	3	3
Arlington		2.5	7.5	11	7.5	11
John Day Dam		3	10.5	17.5	10.5	17.5
1997						
Port of Morrow	2593	1.5	3	3	3	3
Arlington		3	7.5	10	7.5	10
John Day Dam		3.5	10.5	16.5	10.5	16.5

We believe that this is caused by storage behind the reservoir's spillway as the flood rises. Under Existing conditions, as the flood rises, the dam (theoretically) responds by releasing more water, thus increasing the flow almost as soon as it enters the pool. For Natural River conditions, the flood rises relatively slowly compared to the travel time. The depth is very close to "normal depth" for a given flow, and thus again responds more directly to changes in flow with little dynamic storage. However, for the Spillway Drawdown condition, the head has to increase behind the spillway before the flow increases. This increase in head represents dynamic storage that slows down the apparent arrival of flow of a given magnitude (say a two-year "trigger" flow), thus attenuating the downstream rise of the hydrograph, and being dependent on the rate of rise.

5.5 Analysis of Conditions "With" and "Without" Flood Control

Table 5-2 shows the maximum (peak) flows and maximum reservoir stages for the Spillway Drawdown and Natural River conditions, with and without flood control. The table also shows the length of the time that the maximum flow is released over the spillway during flood control operations. The Existing condition was not included in the table as flood control operations were not included in the simulations of this condition.

The results show that during Spillway Drawdown conditions, the pool could be maintained between elevations 233 and 249 feet without flood control, for the five major floods modeled. When flood control (as modeled here) is imposed, the pool increases by almost 20 feet in each case, and 19-26 days of release at the maximum flow would be required before the flood control volume is reduced to zero again. It is also interesting to note that flood control only results in a lessening of the peak discharge by about five percent but increases the length of time of relatively large releases.

Table 5-2: Flow, Stage, and Release Time Comparison at John Day Dam

	Natural	Spillway	Natural with Flood Control	Spillway with Flood Control
1948				
Maximum Stage (feet)	178	249	235	265
Maximum Flow (cfs)	990,000	990,000	940,960	940,960
Release Time (days)			20	20
1974				
Maximum Stage (feet)	170	237	229	255
Maximum Flow (cfs)	605,000	605,000	548,970	548,970
Release Time (days)			20	20
1982				
Maximum Stage (feet)	167	233	226	251
Maximum Flow (cfs)	474,000	474,000	415,820	415,820
Release Time (days)			24	24
1996				
Maximum Stage (feet)	167	233	224	251
Maximum Flow (cfs)	474,000	474,000	427,950	427,950
Release Time (days)			19	19
1997				
Maximum Stage (feet)	170	237	227	254
Maximum Flow (cfs)	600,000	600,000	535,060	535,060
Release Time (days)			26	26

During Natural River conditions, the maximum river stage would vary between 167 and 178 feet without flood control. The influence of flood control, however, is much more dramatic, and maximum stages could increase nearly 60 feet at the dam. Again, the effect of flood control would be to reduce the peak discharge by only about five percent and require 19-26 days to return to a "pre-flood-control" condition.

6. SUMMARY

This reconnaissance study evaluated the impacts of alternative pool configurations, including Spillway Drawdown and Natural River conditions, and the extent to which these alternative configurations can provide some measure of flood control in terms of reservoir storage, the ability to store and attenuate floods, and the time of travel through the system. The study also examined the effects of the various alternative configurations, with and without flood control.

One-dimensional steady (HEC-RAS) and unsteady (UNET) flow models were developed from the McNary tailwater to Rufus, downstream of John Day dam for the various scenarios, and various data products produced. These included elevation-storage and reservoir storage-capacity curves; rating curves; longitudinal profiles of water surface elevations and velocities; hydrographs of water surface elevations, flows and velocities; and various time-of-travel estimates.

Under approximately steady flow conditions, the Spillway Drawdown condition would tend to decrease travel times by about a multiple of two to three at lower flows and the

Natural River condition would tend to decrease travel times by a multiple of 4-10, as compared to Existing conditions.

The model results showed that while the Existing condition might have a two-year-flood travel time of 3.5 hours, the Natural River condition would increase this time by about a multiple of three. However, the Spillway Drawdown condition would increase this time by a multiple of five or more, and seems to depend strongly on the rate of rise of the flood event. The more quickly the flood rises, the more quickly the two-year flow travels through the reservoir. We believe that this is caused by dynamic storage behind the reservoir's spillway as the flood rises, as the head has to increase behind the spillway before the flow increases.

During Spillway Drawdown conditions, the pool could be maintained between elevations 233 and 249 ft NGVD without flood control, for the five major floods modeled. When flood control (as modeled here) is imposed, the pool would increase by about 20 ft, and 19-26 days of release at the maximum flow would be required before the flood control volume is reduced to zero again. It is interesting to note that flood control only results in a lessening of the peak discharge by about five percent but increases the length of time of relatively large releases. During Natural River conditions, the maximum river stage would vary between 167 and 178 ft NGVD without flood control. The influence of flood control, however, is much more dramatic, and maximum stages could increase nearly 60 ft at the dam. Again, the effect of flood control would be to reduce the peak discharge by only about five percent and require 19-26 days to return to a "pre-flood-control" condition.

7. REFERENCES

Hydrologic Engineering Center (HEC), "HEC-RAS, River Analysis System, Hydraulic Reference Manual, Version 2.2", CPD-69, U.S. Army Corps of Engineers, Davis, CA, September 1998.

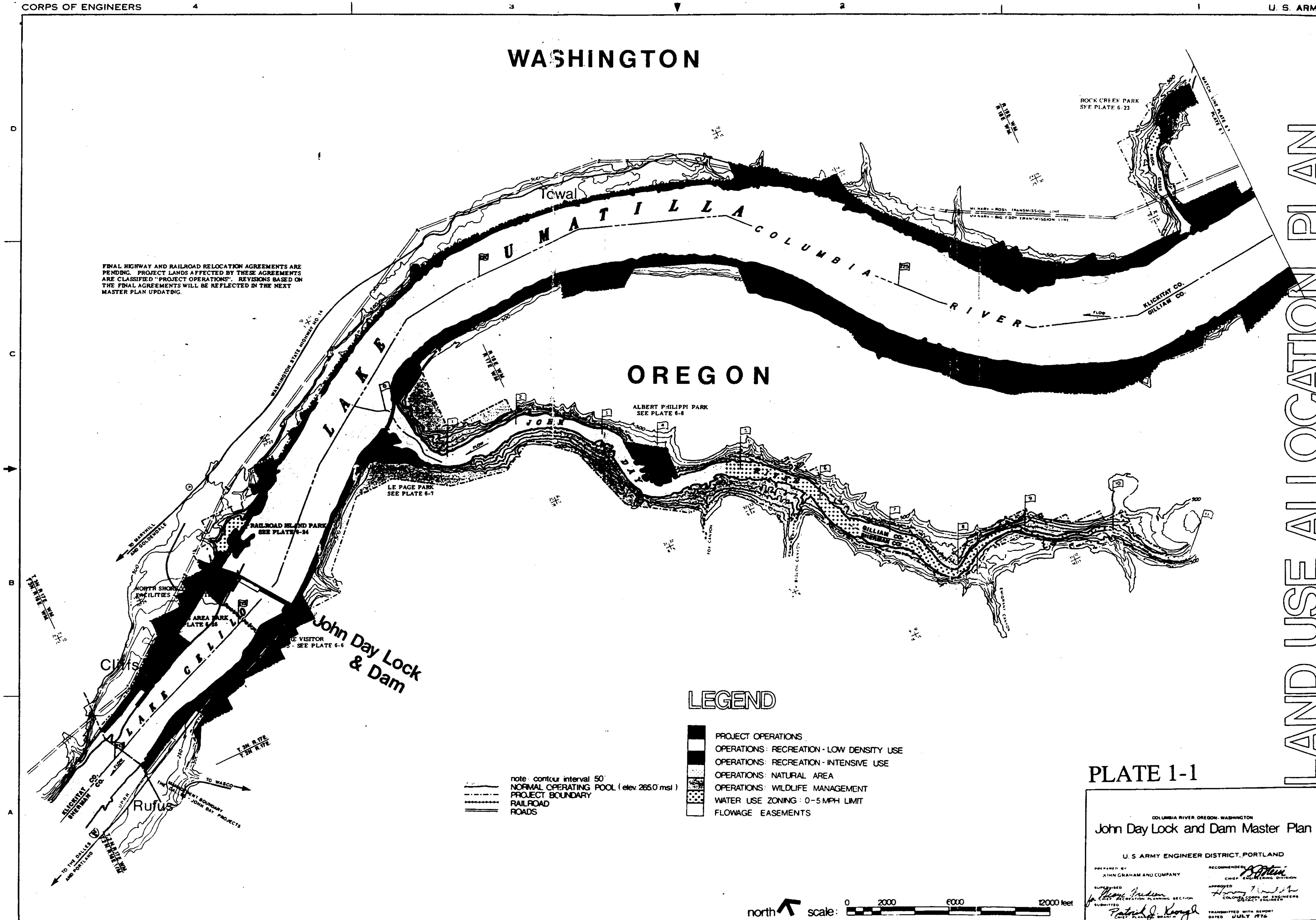
Hydrologic Engineering Center (HEC), "UNET, One-Dimensional Unsteady Flow Through a Full Network of Open Channels, User's Manual, Version 3.2", CPD-66, U.S. Army Corps of Engineers, Davis, CA, August, 1997.

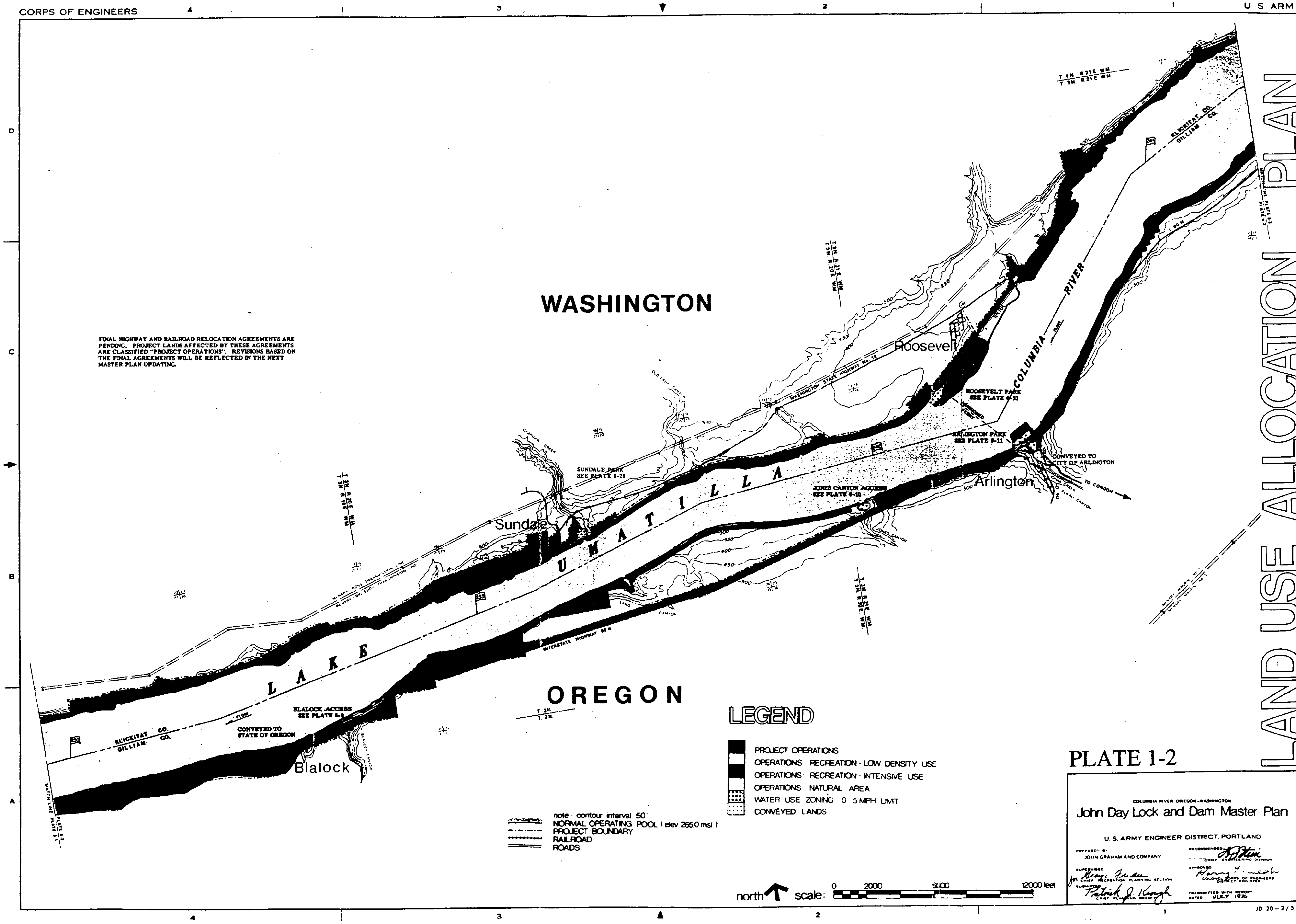
U.S. Army Corps of Engineers, "Hydraulic Design of Flood Control Channels", USCOE, Engineer Manual 1110-2-1601, change 1, 1994.

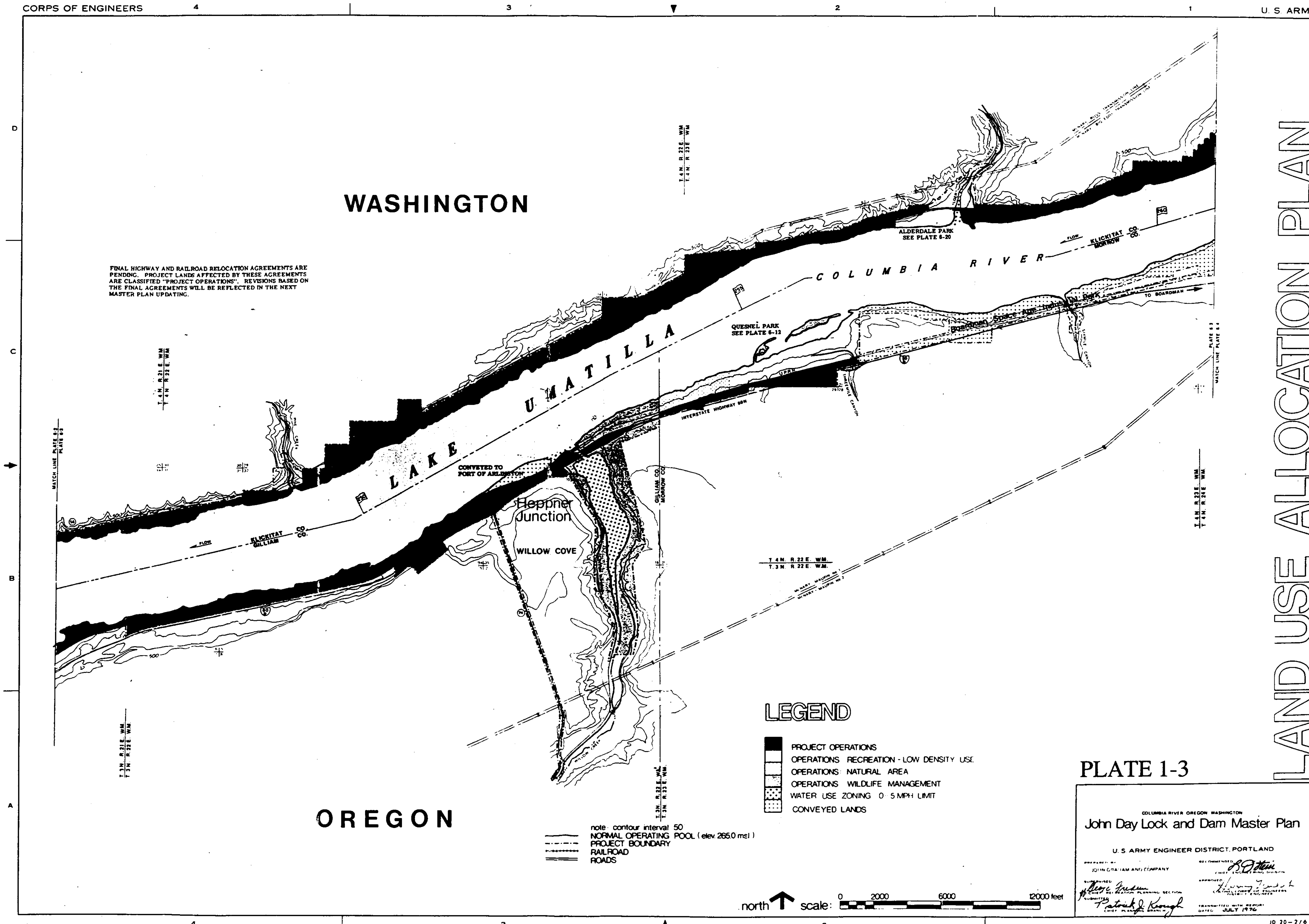
WEST Consultants, Inc., "Riprap Design System, Version 2.0", San Diego, CA, 1996.

Plates

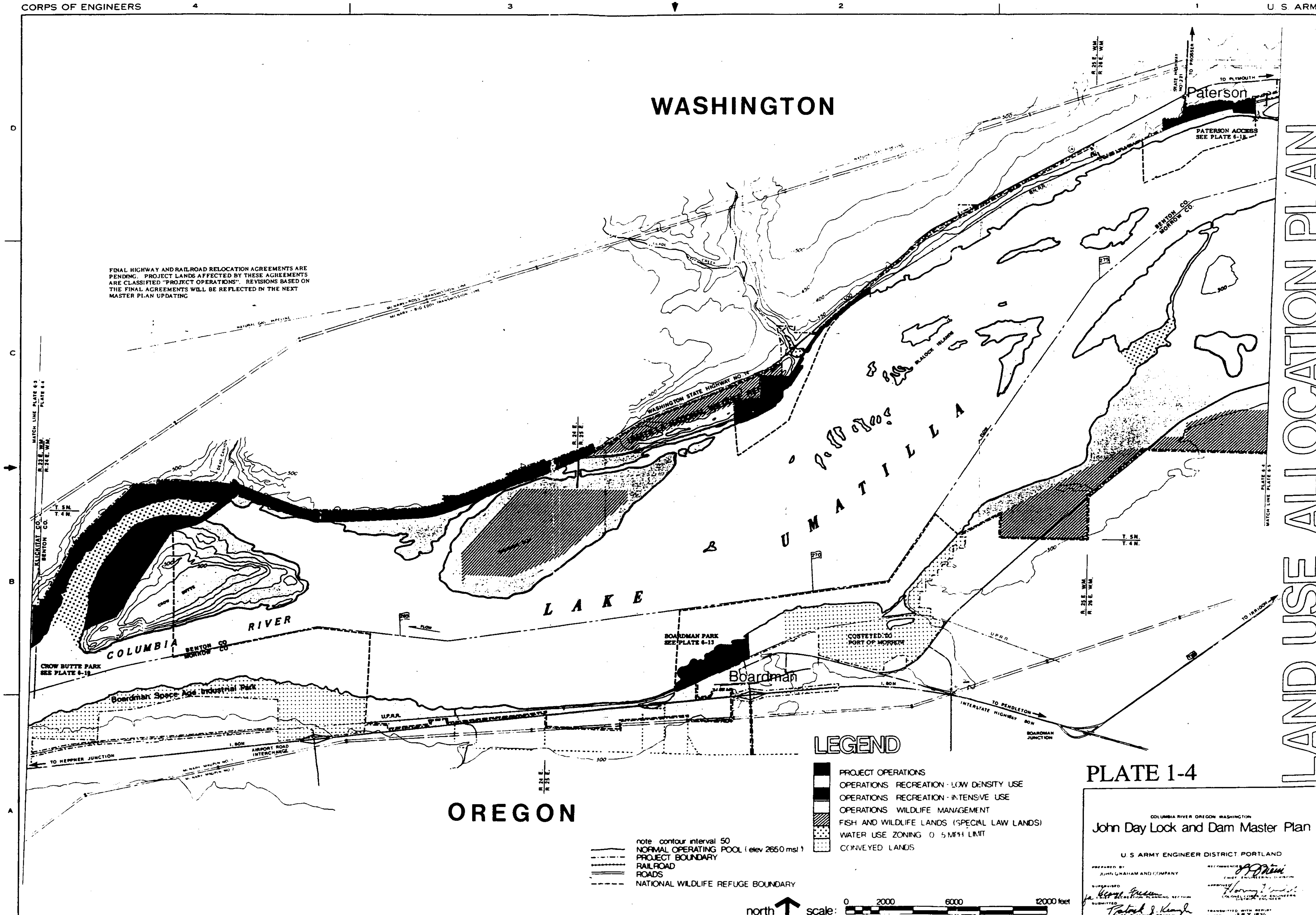
CORPS OF ENGINEERS | U. S. ARMY



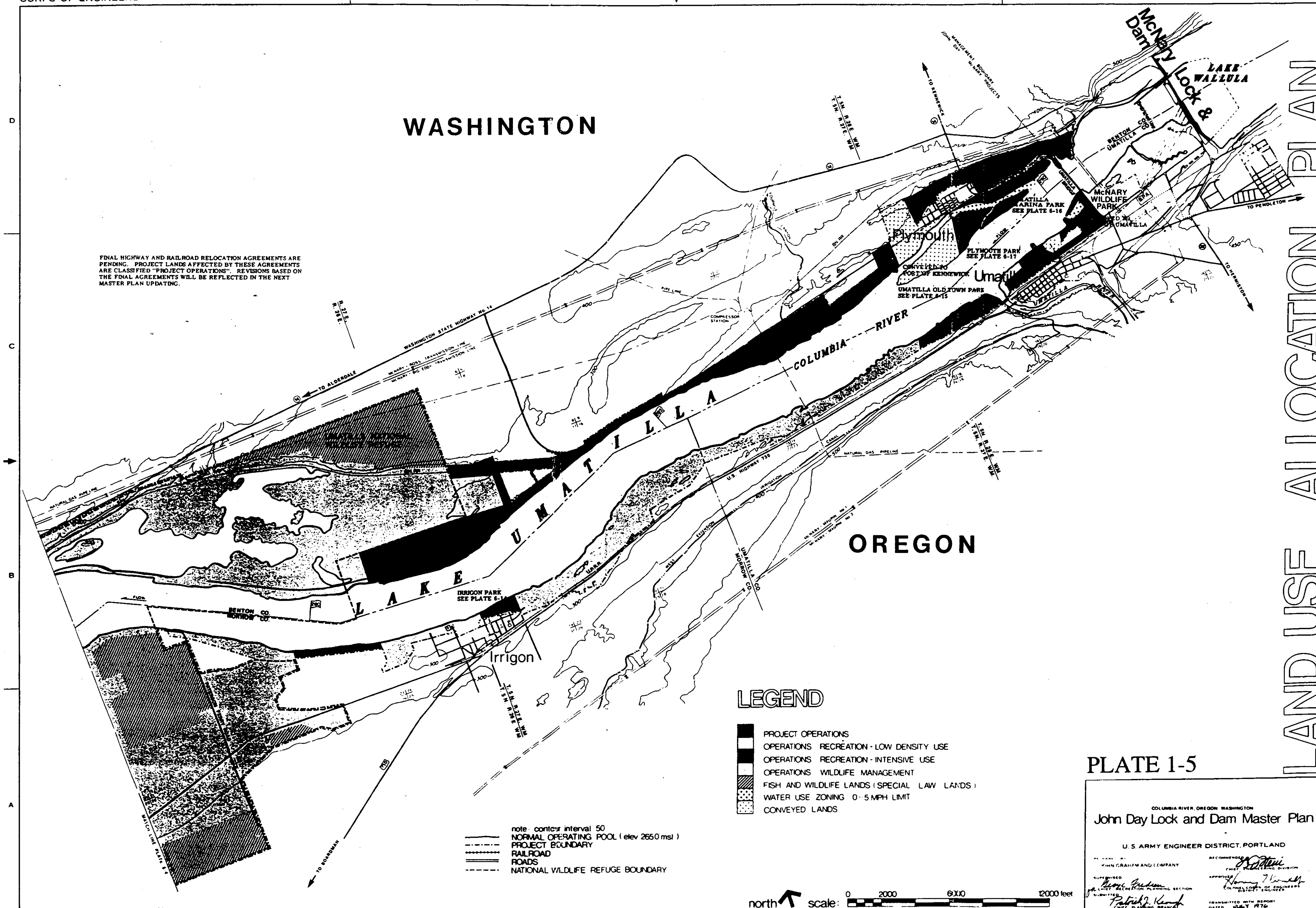

CORPS OF ENGINEERS
U.S. ARMY
LAND USE ALLOCATION PLAN
WASHINGTON
OREGON
FINAL HIGHWAY AND RAILROAD RELOCATION AGREEMENTS ARE PENDING. PROJECT LANDS AFFECTED BY THESE AGREEMENTS ARE CLASSIFIED "PROJECT OPERATIONS". REVISIONS BASED ON THE FINAL AGREEMENTS WILL BE REFLECTED IN THE NEXT MASTER PLAN UPDATING.
LAKE UMATILLA
COLUMBIA RIVER
Roosevelt
Sundale
Arlington
Blalock
ROOSEVELT PARK SEE PLATE 6-21
ARLINGTON PARK SEE PLATE 6-11
SUNDALE PARK SEE PLATE 6-22
JONES CANYON ACCESS SEE PLATE 6-10
BLALOCK ACCESS SEE PLATE 6-9
CONVEYED TO STATE OF OREGON
CONVEYED TO CITY OF ARLINGTON
TO CONDON
KLICKITAT CO.
GILLIAM CO.
INTERSTATE HIGHWAY 80 N
WASHINGTON STATE HIGHWAY NO. 14
LEGEND
PROJECT OPERATIONS
OPERATIONS RECREATION - LOW DENSITY USE
OPERATIONS RECREATION - INTENSIVE USE
OPERATIONS NATURAL AREA
WATER USE ZONING 0-5 MPH LIMIT
CONVEYED LANDS
note: contour interval 50
NORMAL OPERATING POOL (elev 265.0 msl)
PROJECT BOUNDARY
RAILROAD
ROADS
north
scale: 0 2000 6000 12000 feet
PLATE 1-2
COLUMBIA RIVER, OREGON-WASHINGTON
John Day Lock and Dam Master Plan
U.S. ARMY ENGINEER DISTRICT, PORTLAND
PREPARED BY JOHN GRAHAM AND COMPANY
RECOMMENDED CHIEF ENGINEERING DIVISION
SUPERVISED for CHIEF RECREATION PLANNING SECTION
APPROVED COLONEL CORPS OF ENGINEERS DISTRICT ENGINEER
SUBMITTED Patrick J. Keough CHIEF PLANNING BRANCH
TRANSMITTED WITH REPORT DATED JULY 1976
JD 20-2/5


CORPS OF ENGINEERS
U. S. ARMY
LAND USE ALLOCATION PLAN
WASHINGTON
OREGON
FINAL HIGHWAY AND RAILROAD RELOCATION AGREEMENTS ARE PENDING. PROJECT LANDS AFFECTED BY THESE AGREEMENTS ARE CLASSIFIED "PROJECT OPERATIONS". REVISIONS BASED ON THE FINAL AGREEMENTS WILL BE REFLECTED IN THE NEXT MASTER PLAN UPDATING.
LAKE UMATILLA
COLUMBIA RIVER
ALDERDALE PARK
SEE PLATE 6-20
QUESNEL PARK
SEE PLATE 6-12
CONVEYED TO PORT OF ARLINGTON
Heppner Junction
WILLOW COVE
KLICKITAT CO.
GILLIAM CO.
MORROW CO.
INTERSTATE HIGHWAY 80N
TO BOARDMAN
FLOW
MATCH LINE PLATE 6-2
LEGEND
PROJECT OPERATIONS
OPERATIONS: RECREATION - LOW DENSITY USE
OPERATIONS: NATURAL AREA
OPERATIONS: WILDLIFE MANAGEMENT
WATER USE ZONING 0 5 MPH LIMIT
CONVEYED LANDS
note: contour interval 50
NORMAL OPERATING POOL (elev. 265.0 msl)
PROJECT BOUNDARY
RAILROAD
ROADS
north
scale: 0 2000 6000 12000 feet
PLATE 1-3
COLUMBIA RIVER OREGON WASHINGTON
John Day Lock and Dam Master Plan
U. S. ARMY ENGINEER DISTRICT, PORTLAND
PREPARED BY
JOHN GRAHAM AND COMPANY
SUPERVISED
CHIEF RECREATION PLANNING SECTION
SUBMITTED
Patrick J. Keough
CHIEF PLANNING BRANCH
RECOMMENDED
CHIEF ENGINEERING DIVISION
APPROVED
COLONEL CORPS OF ENGINEERS DISTRICT ENGINEER
TRANSMITTED WITH REPORT
DATED JULY 1976
JD 20-2/6

CORPS OF ENGINEERS U S ARMY



JD 20 - 2/7


LAND USE ALLOCATION PLAN
WASHINGTON
OREGON
FINAL HIGHWAY AND RAILROAD RELOCATION AGREEMENTS ARE PENDING. PROJECT LANDS AFFECTED BY THESE AGREEMENTS ARE CLASSIFIED "PROJECT OPERATIONS". REVISIONS BASED ON THE FINAL AGREEMENTS WILL BE REFLECTED IN THE NEXT MASTER PLAN UPDATING.
McNary Lock & Dam
LAKE WALLULA
McNARY WILDLIFE PARK
UMATILLA MARINA PARK SEE PLATE 6-16
Plymouth
PLYMOUTH PARK SEE PLATE 6-17
CONVEYED TO PORT OF KENNEWICK
Umatilla
UMATILLA OLD TOWN PARK SEE PLATE 6-15
COLUMBIA RIVER
LAKE UMATILLA
IRRIGON PARK SEE PLATE 6-14
Irrigon
WASHINGTON STATE HIGHWAY NO. 14
TO ALDERDALE
TO KENNEWICK
TO PENDLETON
TO HERMISTON
TO BOARDMAN
COMPRESSOR STATION
U.S. HIGHWAY 730
NATURAL GAS PIPELINE
BENTON CO.
MORROW CO.
UMATILLA CO.
LEGEND
PROJECT OPERATIONS
OPERATIONS RECREATION - LOW DENSITY USE
OPERATIONS RECREATION - INTENSIVE USE
OPERATIONS WILDLIFE MANAGEMENT
FISH AND WILDLIFE LANDS (SPECIAL LAW LANDS)
WATER USE ZONING 0-5 MPH LIMIT
CONVEYED LANDS
note: contour interval 50
NORMAL OPERATING POOL (elev 265.0 msl)
PROJECT BOUNDARY
RAILROAD
ROADS
NATIONAL WILDLIFE REFUGE BOUNDARY
north
scale: 0 2000 6000 12000 feet
PLATE 1-5
COLUMBIA RIVER, OREGON WASHINGTON
John Day Lock and Dam Master Plan
U. S. ARMY ENGINEER DISTRICT, PORTLAND
JOHN GRAHAM AND COMPANY
RECOMMENDED
CHIEF, ENGINEERING DIVISION
SUPERVISED
CHIEF, RECREATION PLANNING SECTION
APPROVED
COLONEL CORPS OF ENGINEERS DISTRICT ENGINEER
SUBMITTED
CHIEF, PLANNING BRANCH
TRANSMITTED WITH REPORT DATED JULY 1976