Unsteady Flow Model of Priest Rapids Dam Releases at Hanford Reach, Columbia River, Washington

**S. C. Sneider
R. L. Skaggs**

February 1983

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COLUMBIA RIVER, WASHINGTON

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Richland, Washington 99352

PREFACE

This report is part of a program entitled "Riverine Ecology: Effects of Energy Technologies," which is conducted in the Applied Ecology Section, Environmental Sciences Department of Pacific Northwest Laboratory (PNL). Research related to the effects of water-level fluctuations have been part of the program for several years. Many ecological research studies at PNL have been made in the Hanford Reach of the Columbia River. The observations and data obtained to date suggested the need for a method to correlate available hydrological data with water-level changes and, in turn, with ecological effects. The model described in this report is a way to predict water-level fluctuations using discharges recorded during routine operation at a hydroelectric dam.

Development of a reliable model to describe physical phenomena in a river system is only an initial step, and often the easiest step, in determining ecological impact on fish and other riverine organisms. Detailed field studies are necessary to correlate the relationship between stream flows and habitat requirements of important fish species. For realistic impact assessment, the effects of changing stream flows during critical life stages of fish (such as spawning and rearing of young) must be determined under field conditions. Key references that discuss instream flow methods include Bovee (1981), Bovee and Milhouse (1978), Orth and Maughan (1982), and Osborn and Allman (1976).

Environmental studies on the effects of water-level fluctuations at Hanford are site-specific; however, results of this research are generic and can be applied to other river ecosystems affected by alternating flow regimes. Over the years, Hanford has proven to be a convenient and useful location for field research related to DOE needs.

SUMMARY

A model was developed to simulate water levels at three locations on the Columbia River between Priest Rapids Dam River Mile 396.1 (River Kilometer 639.0) and River Mile 361.50 (River Kilometer 581.7). The model was calibrated and verified over a range of flows. The results of calibration and verification indicate that the model, with reasonable accuracy, simulates stages to within ± 0.08 m (± 0.25 ft) and surface wave timing to within ± 20 min. The model can be used by researchers, river system managers, planners, and decision makers as a tool to predict fluctuating water levels at locations downstream of dams.

Data produced by the model can be used to evaluate and quantify possible impacts on aquatic organisms, water supply, navigation, irrigation, recreation, and additional hydropower enhancement. Although the results of this model calibration and the model simulations presented are site-specific, the methodology is generic. Therefore, the model can be adapted to reflect dam discharges and resulting river flows at other river systems affected by water-level fluctuations.

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INTRODUCTION

This report describes the development, calibration, and verification of a mathematical model designed to simulate water levels at the mouth of selected sloughs within the Hanford Reach of the Columbia River. The model uses an existing computer code called DWOPER (Dynamic Wave OPERational model) developed by the National Weather Service. The model was formulated with Priest Rapids Dam (River Mile 397.1) discharge data and a rating curve at River Mile 361.50 that represent the upstream and downstream boundaries, respectively.(a)

The model was calibrated and verified for Priest Rapids releases ranging from 36,000 to nearly 200,000 cubic feet per second (cfs).(b) The model was able to simulate water levels at River Mile 361.5 to within +0.08 m (+0.25 ft) and surface wave timing to within +20 minutes. This precision indicates that the model may be accurate enough for investigating ecological impacts associated with fluctuating water levels. The model, when fitted with applicable parameters, can also be adapted to reflect dam discharges and resulting river flows at other locations in the Hanford Reach and in other river systems.

The need for the DWOPER model, as described in this report, arose from ecological studies in the Hanford Reach of the Columbia River. Many of these studies concerned the effects of water-level fluctuations produced by hydro-electric generations at an upriver dam on valued fish species. For example, backwater and slough areas along the Hanford Reach are important spawning and rearing areas for smallmouth bass (Micropterus dolomieu). The annual success of bass spawning in these areas is adversely affected by fluctuating water levels. During low flow periods, some sloughs are isolated above the main river level, allowing slough water temperatures to increase from insolation.

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- (a) U.S. customary units of measurement are the typical units of measurement for river systems in the U.S. To convert miles to kilometers, multiply by 1.609.
 - (b) To convert cubic feet per second (cfs) to cubic meters per second (m^3/sec), multiply by 0.0283.

When river levels rise, the sloughs are flooded by the colder main river, thereby significantly reducing water temperatures, increasing current velocities, and disrupting bass spawning.

To study the ecological function of the sloughs and the effects of hydro-power operations on fish, it is necessary to determine the frequency and duration of slough flooding. This can be done directly by in situ monitoring, or indirectly based on upriver discharges. In situ measurements involve studies that are long-term and expensive; therefore, some indirect means were needed to correlate available hydrological data with water levels. Hourly discharge data from Priest Rapids Dam above Hanford were available. A mathematical model was required that could compute water levels at downriver locations based on discharges at Priest Rapids Dam.

HANFORD REACH DESCRIPTION

The Hanford Reach is the only unimpounded section of the Columbia River above Bonneville Dam in the United States (Figures 1 and 2). The Reach extends from Priest Rapids Dam at River Mile (RM) 397 downstream to the backwater of McNary Dam. Discharges in the Hanford Reach are regulated by releases from Priest Rapids Dam. The minimum instantaneous flow required by the Federal Energy Regulatory Commission for the Hanford Reach is 36,000 cfs (Washington State Department of Ecology 1980). The powerhouse hydraulic capacity at Priest Rapids Dam is 170,000 cfs; discharges in excess of this are passed through the dam spillway. Priest Rapids Dam is operated primarily for producing baseload hydropower. This mode of operation results in downstream flow releases that have a distinct diurnal cycle (Figure 3). Water levels in the Hanford Reach, therefore, follow a similar cycle, and may fluctuate several feet over the course of a day.

Generally, the Hanford Reach is characterized by a substrate of gravel, cobble, and rock, and by swift currents. Some of the shoreline is steep, particularly along the east bank where bluffs extend to the river margin. The main channel of the study reach includes a number of islands. Ancient floods from glacier melt generated significant scour that produced submerged rock ledges and gravel bars. The result today is a series of pools and crossings. The large material in the bed acts as an armor layer, maintaining the main morphological features of the channel. As the flow passes through the ripples and pools, the cross-sectional flow area and the flow depth change dramatically. These changes in flow area and depth are illustrated in Figure 4, which shows the channel thalweg profile based on the available cross-sectional information. Significant longitudinal variation in depths occurs.

A number of sloughs and backwater areas are located on both sides of the river. Substrates in these areas are largely rocky but some are also composed of sand, silt and mud. In the spring, water tends to warm earlier in the sloughs than in the mainstream Columbia River. Due to insolation, peak summer temperatures in isolated sloughs often exceed 30°C, while temperatures in the

FIGURE 1. Columbia River and Environs (Fickeisen et al. 1980)

FIGURE 2. Hanford Reach of the Columbia River. Numbers represent islands (Fickeisen et al. 1980).

FIGURE 3. Discharges from Priest Rapids Dam from June 15, 1980 Through July 1, 1980

river peak near 20°C. The rocky areas of the sloughs are used for spawning by smallmouth bass (Micropterus dolomieu), and the sandy and muddy areas are used for spawning by largemouth bass (Micropterus salmoides) and catfish (Ictalurus spp.).

The fluctuations in river flows and pool levels significantly affect fish and riparian wildlife. As the water level rises from increased dam discharges, small ponding areas (known as "sloughs") become inundated. Cold water from the main channel flows into the warm slough areas and produce abrupt temperature

FIGURE 4. Channel Thalweg Profile, and Slough Location and Elevation

changes. Subsequently, as the water level drops, the sloughs become relatively isolated from the main flow of the river until the water level rises again.

Recent studies (Montgomery, Fickeisen and Becker 1980; Fickeisen, Becker, and Neitzel 1981; Becker, Fickeisen and Montgomery 1981) confirm that small-mouth bass spawning occurs within the Hanford Reach in at least three backwater slough areas: Hanford Slough (RM 367.0), 100-F Slough (RM 367.3), and White Bluffs Slough (RM 369.6) (Figure 2). The sloughs provide gravel and rock substrate suitable for nest construction, slower currents, and warmer water

than occurs in the mainstem river. These habitat conditions are necessary for successful smallmouth bass reproduction, including the building of nests, protection of developing eggs, and rearing of fry.

Of the three sloughs where spawning commonly occurs, Hanford Slough is open directly to the river and undergoes the most extreme changes in water level and temperature. The 100-F Slough (Figure 2) is usually isolated from the main river channel by low water conditions, which trap adult bass after the spring spate for periods of up to a year. During low flow periods, water-level fluctuations in the 100-F Slough are minimal. The 100-F Slough is flooded during the spring spate, resulting in high currents and low temperatures that disrupt optimal spawning conditions. White Bluffs Slough is partially open, and water moves in and out in relation to changing river levels. Since this slough is intermediate in elevation above the mainstem river, some fluctuations in water level and temperature result from moderate changes in river flow. Slough locations, inlet elevations, and channel thalweg profiles are shown in Figure 4.

UNSTEADY FLOW ANALYSIS

A surface wave is a temporal, spatially propagated change in a water surface. Unsteady flow analysis is the procedure of determining the shape of a surface wave at one point in a stream from a known wave shape at an upstream location. A surface wave moving through a short, regular channel with little resistance will not change shape significantly. However, a surface wave moving over longer distances in an irregular channel with much resistance can change shape appreciably. The change in shape is due to physical attenuation. The longer the reach, the more irregular the channel, and/or the greater the flow resistance, the greater the attenuation. Attenuation lowers the peak of a discharge and broadens the time base of surface waves.

PHENOMENON DESCRIPTION

The National Weather Service's Dynamic Wave OPERational model (DWOPER) was applied to the Hanford Reach of the Columbia River. The calibrated and verified DWOPER model developed for the Hanford Reach has been used to demonstrate the attenuation phenomenon. An artificial event consisting of an inflow for 24 hours at low flow, 4 hours at high flow, and 42 additional hours at low flow has been simulated. This inflow is a step function in which flow at the upstream boundary increases from 36,000 cfs to 150,000 cfs between Hours 24 and 25, remains at 150,000 cfs between Hours 28 and 29, and returns to 36,000 cfs for the remainder of the event. A hydrograph of this inflow at the upstream boundary is shown in Figure 5. The stage response to this inflow at the upstream boundary is shown in Figure 6.

The simulation shows that a surface wave becomes elongated and develops a lower peak as it travels downstream. The peak reaches the downstream boundary nine hours after it passes the upstream boundary. By this time the magnitude of the peak has been reduced from 150,000 cfs to 95,000 cfs, and the time base has broadened from 4 to about 30 hours. A hydrograph showing the flow at the downstream boundary is in Figure 7. Figure 8 shows the stage response to this flow.

TIME INTO SIMULATION (HOURS)

FIGURE 5. Hydrograph of Inflow at Upstream Boundary

FIGURE 6. Hydrograph of Stage Response to Inflow

TIME INTO SIMULATION (HOURS)

FIGURE 7. Hydrograph of Outflow at Downstream Boundary

FIGURE 8. Hydrograph of Stage Response to Outflow

FIGURE 9. Stage Profile of Hanford Reach During the Artificial Event

The stage of the reach at fixed points in time is shown in Figures 9 and 10. Figure 9 shows the river stage at Hours 24, 29, and 34. The entire reach at Hour 24 is stable at low flow. The upper portion of the reach at Hour 29 is under the influence of the surface wave, while the lower portion of the reach is still at low flow. The flood wave at Hour 34 the flood wave has reached the lower portion of the reach, and the upper portion has returned to low flow. Flows at fixed times from Hours 24 through 44 are shown in Figure 10. The flow at Hour 24 is at 36,000 cfs throughout the reach. As time progresses, the surface wave moves downstream. The peak flow is reduced and the time base is broadened with each subsequent time increment.

FIGURE 10. Flow Profile of Hanford Reach During the Artificial Event

MODEL DESCRIPTION

Selecting the appropriate unsteady flow model to solve a flow routing problem requires careful consideration of the nature of the problem and the river being studied. After considering the Hanford Reach of the Columbia River and the problem of simulating water-level fluctuations for a range of flows at the sloughs, we chose to use a generally applicable dynamic wave model. The National Weather Service's Dynamic Wave OPERational model (DWOPER) is a widely used model that has been successfully applied in similar situations.

DWOPER is a dynamic wave routing model based on an implicit finite difference solution of the complete one-dimensional St. Venant unsteady flow equations. These equations, written in terms of one-dimensional mass and momentum conservation, are given in Equations (1) and (2), respectively.

$$\frac{\partial Q}{\partial x} + \frac{\partial (A + A_0)}{\partial t} - q = 0 \quad (1)$$

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2/A)}{\partial x} + gA \left(\frac{\partial h}{\partial x} + S_f + S_e \right) - qV_x + W_f B = 0 \quad (2)$$

where

Q = discharge

x = channel distance

A = cross-sectional flow area

A₀ = cross-sectional area not contributing to flow

t = time

q = lateral inflows (or outflows)

g = acceleration due to gravity

h = stage

S_f = friction slope

S_e = expansion (or contraction) loss

V_x = x-component of the velocity of lateral inflows

W_f = wind friction

B = channel top width.

The friction slope is further defined by the Manning equation:

$$Q = \frac{1.486}{n} R^{2/3} S_f^{1/2} A \quad (3)$$

where Q, S_f and A are as previously defined

n = Manning's coefficient of roughness

R = hydraulic radius.

Solving Equation (3) for S_f gives

$$S_f = \frac{n^2 |Q| Q}{2.208 R^{4/3} A^2} \quad (4)$$

The expansion loss term is further defined as

$$S_e = K_e \frac{\partial}{\partial x} \left(\frac{V^2}{2g} \right) \quad (5)$$

or

$$= \frac{K_e}{2g} \frac{\partial (Q/A^2)}{\partial x} \quad (6)$$

where x , g , Q , and A are as previously defined

V = cross sectional averaged velocity

K_e = expansion (or contraction) loss coefficient.

The partial differential equations in (1) and (2) are solved by DWOPER through the use of implicit finite difference techniques. A four-point weighted scheme is employed. This scheme can be effectively used with unequal distance steps, and the convergence properties can be controlled. Stage (h) and discharge (Q) are calculated in the continuous space-time (x - t) region by representing this region as a net of discrete points. Each discrete point is identified by a subscript (i) and a superscript (j) representing the space and time positions, respectively. Figure 11 represents this scheme. If K represents any particular variable, then the temporal derivative of K is given by

$$\frac{\partial K}{\partial t} = (K_i^{j+1} + K_{i+1}^{j+1} - K_i^j - K_{i+1}^j) / 2\Delta t \quad (7)$$

A weighting factor (θ) is introduced into the calculation of spatial derivatives. A spatial derivative is calculated as a finite difference quotient, which is positioned between two adjacent time lines in proportion to the weighting factor. This can be seen in Equation (8):

$$\frac{\partial K}{\partial x} = \theta (K_{i+1}^{j+1} - K_i^{j+1}) / \Delta x + (1 - \theta) (K_{i+1}^j - K_i^j) / \Delta x \quad (8)$$

Other variables are calculated in the same manner with the same weighting factor. Equation (9) gives this result:

$$K = \theta (K_{i+1}^{j+1} + K_i^{j+1}) / 2 + (1 - \theta) (K_{i+1}^j + K_i^j) / 2 \quad (9)$$

FIGURE 11. Finite Difference Mesh Space (x-t) Domain

Solving the unsteady flow equations in Equations (1) and (2) by substitution of the finite difference quotients in Equations (7), (8), and (9) generates two sets of equations, which are nonlinear in stage (h) and discharge (Q). If there are n cross sections, there will be n - 1 computational subreaches and 2(n - 1) (or 2n - 2) equations with 2n unknowns, h and Q at each cross section. This leaves two fewer equations than unknowns. The number of unknowns is reduced by specifying boundary conditions at the upstream and downstream boundaries of the study area. A Newton-Raphson method is used to solve the resultant system of 2n equations.

The Newton-Raphson method is an iterative procedure in which trial values of stage and discharge are substituted into the system of equations and new values of stage and discharge are calculated. If the new values differ by more than a specified amount from the trial values, then the new values become the trial values for the next iteration. The procedure is repeated until convergence upon a solution occurs. The initial trial values are determined by linear or parabolic extrapolation from previous time-step solutions.

Values of initial conditions, at time $j = 1$, must be specified. The values can be calculated by steady flow step-backwater methods. The initial conditions are then substituted into the unsteady flow equations with the boundary conditions held constant for several time steps. This results in a set of calculated initial conditions, which become the solutions to the partial differential equations at time $j = 1$.

MODEL DEVELOPMENT

As used here, model development is the application of a general computer code to a specific river reach. For the DWOPER code, model development involves defining channel geometry, initial conditions, boundary conditions, off-channel storage, roughness coefficients, lateral inflows, local losses, and wind effects.

For the application of DWOPER to the Hanford Reach, wind effects were assumed to be insignificant. The climate is semi-arid so, in general, runoff from the surrounding area is not significant during a given event; thus, there are no significant lateral inflows to the reach. The channel was cut by historic flows much greater than existing flows; therefore, off-channel storage is not significant, even at high flows.

Channel Geometry

Channel geometry in DWOPER is represented as cross sections. The cross sections indicate surface widths at various elevations and indicate the area below the minimum elevation used. The cross-sectional data used in this study were obtained from the U.S. Army Corps of Engineers and from U.S. Geological Survey 15-minute topographic maps. These cross-sectional data are listed in Appendix A. An example of a cross section is shown in Figure 12.

Channel Roughness

DWOPER uses the Manning's roughness coefficient to represent channel roughness. Initial values of the coefficient, known as Manning's n , were estimated from previous studies (Barnes 1967; Walters, Ecker, and Skaggs 1982), from U.S. Army Corps of Engineers steady flow estimates, and from past experience. A separate n value was estimated at each cross section for

FIGURE 12. Cross Section at RM 382.05 (personal communication, W. H. Walters, PNL, with N. MacDonald, U.S. Army Corp of Engineers, Seattle District, 1978)

discharge values of 36,000, 300,000 and 725,000 cfs. The estimated n values were then calibrated with the DWOPER automatic calibration option and a trial and error procedure. The calibration is discussed in the model calibration section of this report.

Boundary Conditions

DWOPER requires an upstream and a downstream boundary condition. In this application of DWOPER, the upstream boundary condition was based on outflow hydrographs from Priest Rapids Dam. The hydrographs were obtained from the Grant County Public Utility District, the U.S. Geological Survey, and the U.S. Army Corps of Engineers. The downstream boundary condition was specified as a

stage discharge rating curve. An estimate of the rating curve was obtained from a previous study (Wood 1954). The estimated rating curve was then adjusted as part of the calibration procedure. The procedure is described in the model calibration section of this report.

Initial Conditions

Initial conditions are the values of all parameters at time zero. DWOPER requires that initial conditions of stage and discharge be specified at every cross section or computational point in the model. This was accomplished by 1) assuming steady flow throughout the reach at time zero, 2) assuming a steady flow discharge equal to the initial value of the inflow hydrograph, 3) using the downstream boundary rating curve to determine a stage value at the downstream boundary, and 4) using step-backwater procedures to calculate stage at each successive upstream cross section or computational point. The values of the initial conditions obtained in this manner are not intended to be realistic but to give the model a starting point for its calculations.

DWOPER takes the initial values and uses them in the unsteady flow equations with the boundary conditions held constant. The result is a set of initial conditions that satisfy the unsteady flow equations. As a result of the unrealistic assumptions above, however, the initial conditions are not related to the actual conditions in the reach at time zero. Therefore, it is advisable to disregard the model output for the first day. This makes it necessary to begin the simulation at least one day before the period of interest.

Local Losses

Local losses are energy losses due to changes in flow velocity at an expansion or a contraction between two cross sections. The losses are simulated in DWOPER by reducing the velocity head by a fixed percent at each cross section. The fixed percent at each cross section are input parameters into DWOPER. These coefficients can be estimated by comparing the top width of each cross section at a given discharge. An alternative procedure is to increase n values at expansion and contraction locations. After performing both

procedures for the study reach, we discovered that adjusting the n values gave more accurate results. This is an unexpected result and may be due to numerical procedures rather than to a physical phenomenon.

MODEL CALIBRATION

Model calibration is the procedure of fine-tuning a model. In general, calibration is an iterative procedure in which a known event is simulated, the differences between the known event and the simulated event are noted, and any required changes to the model are made. The procedure is repeated until the differences between the known and the simulated are minimized or reduced to an acceptable level.

In this application of DWOPER, several known events were available for calibrating the model. During May and September 1980, stage was measured for intermittent periods at River Mile 360.5, the downstream boundary of this study. The time period September 17, 1980 through September 25, 1980 was selected as a calibration event. Steady flow profiles obtained from the U.S. Army Corps of Engineers were also used for the calibration.

The calibration procedure was performed in three steps. First, the local loss coefficients were determined; second, Manning's n values were determined; third, the downstream boundary condition was adjusted. The local loss coefficients were estimated with top width and Froude criteria and then adjusted. A Manning's n calibration procedure was used to adjust the relative magnitude of the n values based on the steady flow profiles. The absolute (not relative) values of Manning's n were adjusted based on timing differences at the downstream boundary for the September event. In addition, Manning's n values were not allowed to be less than a minimum value. The rating curve at the downstream boundary was calibrated on the basis of stage differences during the September event.

Local Losses

Ideally, local losses should be accounted for in the model through the use of local loss coefficients. The calibration of these coefficients should be the first step in the calibration procedure.

Initially, local loss coefficients were estimated with top width and Froude criteria at a steady flow of 36,000 cfs. As part of the calibration, the estimated coefficients were adjusted. The attempted adjustment revealed that the choice of the local loss coefficient had little effect on the results. We decided that local losses should be accounted for as part of the Manning's n calibration. Inherent in the calibration procedure is the idea that each subsequent step attempts to account for all differences between simulated and observed events not previously accounted for. Manning's n generally accounts for friction losses. Because local losses were not accounted for before the Manning's n calibration procedure, the local loss coefficient is lumped in with the final calibrated n values.

This observation is substantiated by the final results of the n value calibration. The final n values at the cross sections, where significant expansion or contraction takes place, are greater than one would expect. This seems to indicate that the n value is raised at these cross sections to account for local losses. Errors may be introduced in the dynamics at individual cross sections because DWOPER varies n values with discharge, whereas local losses vary with velocity head. These errors are expected to be insignificant because of the strong positive correlation between velocity head and discharge and because any errors will dampen with increasing distance from the expansion or contraction. Most points of interest within the reach are some distance downstream of the major expansions and contractions.

Manning's n

The only available information pertaining to the relative values of Manning's n was contained in the Army Corps' steady flow profiles. The only information readily available pertaining to the actual n values was contained in the observed events. The first step in the calibration of Manning's n was to match steady flow profiles. These profiles are given in Table A.1 in Appendix A and in Figure 13. This step was accomplished through use of DWOPER's automatic calibration feature and through trial and error.

The automatic calibration feature requires that hydrographs at several locations along the reach be known. The model will match the observed and the

FIGURE 13. Steady Flow Profiles (U.S. Army Corps 1970)

simulated hydrographs at those locations by adjusting the n values. Using the Army Corps' profiles as known hydrographs provides known values at 22 locations along the reach for three different steady flow hydrographs. DWOPER is capable of accepting a maximum of one known hydrograph at eight different locations. This limitation forced the automatic calibration feature to be applied to separate sections of the reach for each steady flow situation. The result is a set of three n values at each cross section, corresponding to the three steady flow values.

The next step was to adjust the n values to account for their effect on flow dynamics. The n values obtained in matching the steady flow profiles were adjusted up if the surface wave arrived too early, down if it arrived too late. In general, the smaller an n value, the faster a surface wave will travel; the greater an n value, the slower a surface wave will travel. Since calibration data were limited [the September event is the only unsteady event available for calibration, and the one (and only) hydrograph measured had only intermittent values], all n values were adjusted by a multiplicative factor. The multiplicative factor was calculated on the basis of the timing differences between the simulated and the observed hydrograph during the September event. If this adjustment forced any n values for the 36,000 cfs profile below 0.025, that value was set equal to 0.025. This minimum value was selected from U.S. Army Corps' observations. The final calibrated n values are listed in Table 1.

The actual values of Manning's n obtained by matching the steady profiles are, at best, reproductions of U.S. Army Corps' estimates. Also, it is important to realize how and why these steady flow profiles were developed: they were used to delineate floods. Because flood studies, by their very nature, tend to be conservative, the n values obtained from matching a flood study are expected to be greater than the n values used in studies such as this one, where the attempt was to simulate hourly unsteady flow. Application of the multiplicative factor, calculated by examining timing differences in the September event, did in fact reduce the steady flow n values by 50%.

Downstream Boundary

Rating curves were developed in 1954 at several locations along the Hanford Reach (Wood 1954). One of the rating curves was used as the initial downstream boundary for this model. Although rating curves can change radically with time, any changes in the study reach should be slight due to the stability of the channel. The initial downstream boundary was adjusted on the basis of stage differences between simulated and observed events. The final calibrated downstream boundary condition is given in Table 2.

TABLE 1. Final Calibrated Manning's n Values

River Mile	Manning's n at 36,000 cfs	Manning's n at 300,000 cfs	Manning's n at 725,000 cfs
396.55	0.0250	0.0157	0.0245
394.49	0.0250	0.0143	0.0384
393.42	0.0250	0.0152	0.0151
390.10	0.0250	0.0141	0.0245
388.34	0.0250	0.0154	0.0325
386.66	0.0250	0.0161	0.0377
384.72	0.0463	0.0201	0.0326
382.05	0.0250	0.0194	0.0229
380.13	0.0256	0.0106	0.0216
378.34	0.0424	0.0114	0.0462
375.89	0.0250	0.0123	0.0323
374.17	0.0250	0.0227	0.0363
372.34	0.0250	0.0227	0.0377
371.61	0.0623	0.0247	0.0428
369.63	0.0520	0.0168	0.0281
368.13	0.0250	0.0117	0.0205
367.00	0.0250	0.0168	0.0272
366.09	0.0338	0.0161	0.0176
364.38	0.0253	0.0169	0.0304
362.51	0.0337	0.0209	0.0264
362.30	0.0250	0.0131	0.0323

Summary

The model was calibrated by adjusting Manning's n values and the downstream boundary condition based on differences between simulated and given events. A comparative adjustment of Manning's n was accomplished by simulating several steady flow events. An absolute adjustment of n values was performed

TABLE 2. Final Calibrated Downstream Boundary Condition
Rating Curve at River Mile 361.50

<u>Elevation, MSL</u>	<u>Discharge, cfs</u>
350.80	36,000
351.10	40,000
352.20	50,000
353.60	60,000
354.00	70,000
355.20	80,000
357.30	100,000
359.00	120,000
360.70	140,000
362.20	160,000
363.50	180,000
364.70	200,000
365.80	220,000
366.80	240,000
367.70	260,000
368.60	280,000
369.30	300,000
370.80	320,000
371.60	340,000
372.40	360,000
313.30	380,000
314.30	400,000

performed based on timing differences at the downstream boundary for a simulation of a September 1980 flow event. The downstream boundary was adjusted based on differences in stage at the downstream boundary during the September event. Local losses were unavoidably lumped into the calibration of Manning's n .

Local losses are usually kinetic energy losses caused by a rapid expansion or contraction in flow. Kinetic energy and local losses are both functions of flow velocity.

Friction losses are usually total energy losses due to surface friction at the flow boundaries. Friction losses increase with increasing surface roughness, increasing boundary length, and increasing flow velocity.

The differences between the two types of losses may cause errors in the simulation when they are lumped together. These errors are believed to be small. This also means that the product of the final n value calibration is not Manning's n but rather a parameter representing the lumped effect of friction losses and local losses. A hydrograph simulating the final calibration event at the downstream boundary is shown in Figure 14.

FIGURE 14. Calibration Event Hydrograph at River Mile 361.5

MODEL VERIFICATION

Model verification is the process of testing how well a model has been calibrated. Verification is accomplished by simulating known events that were not included as calibration events.

Although the calibrated model accurately represents the calibration events, it does not necessarily represent actual events in the system as a whole. Ideally, verification would include a full range of events. Usually, this is not possible because data are limited and because higher priority (in terms of time and money) is given to the calibration effort.

Only two actual measured hydrographs were available in this study. The September 1980 event, the longer of the two, was used in the calibration procedure, leaving the May 1980 event for verification. The results of the simulation of this event are given in Figure 15. The May event flows ranged from 80,000 cfs to 170,000 cfs. The model performed well within this range of flows.

FIGURE 15. Verification Event Hydrograph at River Mile 361.5

GENERAL APPLICATIONS

To quantitatively analyze the impacts of fluctuating upstream hydropower releases on a river reach, a determination must be made of the frequency and duration that elevations of interest (e.g., slough inlets, irrigation diversion inlets, boat ramps) are exceeded. The conventional approach to obtaining this information is to establish monitoring stations that continuously record elevations of the water surface at the location(s) of interest. From these records, a stage hydrograph can be developed for the monitoring period. The conventional approach, however, has two major limitations. First, a monitoring station must be established and maintained at every location of interest. Second, the monitoring stations must remain in place for several years to accumulate a representative sample of water-level fluctuation measurements. Such monitoring efforts are usually expensive.

Although results of the model are site-specific for the Hanford Reach, the general methodology is applicable to practically any river ecosystem affected by fluctuating flow regimes. Once developed, such a model provides the capability to simulate discharges and water levels at any downstream location within a study reach. Additionally, the resultant period of record at any study location is equivalent to the period of record for releases from the upstream dam. This information is then available to aid in the management of water resources for a number of uses downstream from a dam (including aquatic and riparian habitats, navigation, recreation, and irrigation). The following paragraphs describe how the model can be used to help determine aquatic impacts from water-level fluctuations.

As discussed by Becker, Fickeisen and Montgomery (1981), the aquatic impacts of water-level fluctuations in shoreline zones depend on the range of vertical fluctuation and the shoreline topography. For example, pools and sloughs within the Hanford Reach vary in their entrapment potential, depending upon their size, inlet elevation, depth, vegetative cover, and other features. These conditions are illustrated in Figures 16 and 17. As shown in Figure 16a

FIGURE 16. Typical Situations Existing in Pools Subject to Filling During River-Level Fluctuations in the Hanford Reach (modified from Becker, Fickeisen and Montgomery 1981)

Figure 16a, less area is exposed to water-level changes for steep shorelines (left bank) than for gradual slopes (right bank). Thus, a steep shoreline has less potential for fish entrapment. In addition, a gradual slope usually contains more shallow depressions that create temporary pools for entrapment of fish fry. Larger depressions in the area affected by fluctuating water levels (fluctuation zones) are periodically filled when water levels rise, and then become pools as water levels recede. Pools situated below the fluctuation zone (Figure 16b) normally remain filled during extended periods of low flows.

FIGURE 17. Typical Situations Existing in Sloughs Subject to Filling During River-Level Fluctuations in the Hanford Reach (modified from Becker, Fickeisen and Montgomery 1981)

River water enters the pool during peak flow periods and fish can move in and out. Such pools may become semi-permanent and may persist for months during extended periods of low flow.

The typical slough is open at one location, and river water enters during high flow periods. Some small, shallow sloughs fill and drain with each rise and fall of the river's water level (Figure 17a). Larger and deeper sloughs retain water and become temporarily isolated pools as water levels recede (Figure 17b). Such isolation may persist for months during extended periods of low flows. River water may enter at the head of a slough during peak flow periods, leaving behind a series of connected pools that retain water as flows decline (Figure 17c). When river level fluctuations become cyclic on a daily and weekly basis, water may enter only at the "outlet" of a slough and extend inward only as far as the rise permits (Figure 17c).

An approach to quantifying these effects on shoreline areas is to first simulate the water-level fluctuations at the location of interest. For example, Figure 3 shows the time-varying releases from a dam upstream of the Hanford Reach. Based on DWOPER simulations, Figure 18 presents the time-varying water levels at the mouth of White Bluffs Slough. Figure 18 can be used to determine the depth and duration of flooding for the slough, as illustrated in Figure 19. Other details such as periods of rise and recession, which are indicated in Figure 18, are also easily determined. These factors can then be used to identify directly the correlations between aquatic impacts and upstream flow releases.

FIGURE 18. Percent Time from June 15 Through June 20, 1980, for Which Water Levels Exceeded the White Bluffs Slough Inlet Elevation by a Given Depth

FIGURE 19. Stage Hydrograph for Columbia River at White Bluffs Slough, June 15 Through June 20, 1980

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APPENDIX A

MODEL DEVELOPMENT DATA

APPENDIX A

MODEL DEVELOPMENT DATA^(a)

Cross Section No. 1 at River
Mile 396.55 Area Below 384 ft
MSL = 525 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
384	630
393	710
400	980
440	2580
460	5400
480	7800
540	9090
600.	13000

Cross Section No. 2 at River
Mile 394.49 Area Below 379 Ft,
MSL = 2590 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
379	740
382	790
392	1220
400	1340
440	2160
480	6180
520	8160
600	11600

Cross Section No. 3 at River
Mile 392.42 Area Below 382 ft
MSL = 1675 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
382	480
396	1050
400	1500
420	2380
440	2800
480	7280
520	8230
560	8600

Cross Section No. 4 at River
Mile 390.10 Area Below 383 ft
MSL = 5925 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
383	560
385	800
397	960
400	1040
420	2850
440	4760
460	12000
600	12700

(a) Source: personal communication, W. H. Walters, PNL, with N. MacDonald, U.S. Army Corps of Engineers, Seattle District, 1978.

Cross Section No. 5 at River
Mile 388.34 Area Below 372 ft
 MSL = 250 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
372	250
383	680
387	1100
395	1240
400	1500
420	2000
425	2900
440	3500

Cross Section No. 6 at River
Mile 386.66 Area Below 380 ft
 MSL = 5220 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
380	870
384	1140
390	1350
400	1900
420	3400
440	3700
445	10900
520	15800

Cross Section No. 7 at River
Mile 384.72 Area Below 342 ft
 MSL = 150 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
342	150
361	330
372	540
383	950
391	1160
400	1970
420	4300
500	15000

Cross Section No. 8 at River
Mile 382.05 Area Below 373 ft
 MSL = 9750 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
373	780
378	1200
385	1280
400	1640
420	4000
425	5500
460	7500
480	10500

Cross Section No. 9 at River
Mile 380.13 Area Below 350 ft
 MSL = 540 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
350	360
371	730
380	1340
400	1820
430	5040
440	8040
455	16400
480	19600

Cross Section No. 10 at River
Mile 378.34 Area Below 367 ft
 MSL = 1895 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
367	820
371	1080
379	1490
400	2110
420	3600
425	12400
440	14000
460	19400

Cross Section No. 11 at River
Mile 375.89 Area Below 350 ft
 MSL = 150 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
350	170
356	780
360	1210
376	1560
400	6600
420	7920
460	11800
465	23200

Cross Section No. 12 at River
Mile 374.17 Area Below 359 ft
 MSL = 2100 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
359	300
362	650
373	1700
375	2800
380	3500
400	4400
420	8640
440	20300

Cross Section No. 13 at River
Mile 372.34 Area Below 362 ft
 MSL = 1650 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
362	550
365	1250
368	3320
372	4300
400	5000
405	10200
420	22800
460	23600

Cross Section No. 14 at River
Mile 371.61 Area Below 355 ft
 MSL = 2100 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
355	600
365	1800
375	4080
395	7200
420	21360
460	26400
480	27720
500	28100

Cross Section No. 15 at River
Mile 369.63 Area Below 332 ft
 MSL = 8100 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
332	560
360	1050
380	1460
400	4590
405	15100
420	17120
440	20480
460	21920

Cross Section No. 16 at River
Mile 368.13 Area Below 358 ft
 MSL = 4000 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
358	780
360	970
366	1400
370	2170
380	2500
400	2730
402	15600
420	16500

Cross Section No. 17 at River
Mile 367.00 Area Below 356 ft
MSL = 3625 ft²

<u>Elevation (ft, MSL)</u>	<u>Surface Width (ft)</u>
356	825
362	1410
367	2570
382	2865
402	7665
412	17000
422	18000
500	22000

Cross Section No. 18 at River
Mile 366.09 Area Below 354 ft
MSL = 3250 ft²

<u>Elevation (ft, MSL)</u>	<u>Surface Width (ft)</u>
354	840
360	1630
362	2640
365	2990
380	3170
400	12500
440	20800
500	21300

Cross Section No. 19 at River
Mile 364.38 Area Below 345 ft
MSL = 3395 ft²

<u>Elevation (ft, MSL)</u>	<u>Surface Width (ft)</u>
345	970
346	1140
349	1700
354	1880
360	2160
380	2460
400	6160
405	9060

Cross Section No. 20 at River
Mile 362.51 Area Below 330 ft
MSL = 350 ft²

<u>Elevation (ft, MSL)</u>	<u>Surface Width (ft)</u>
330	350
333	530
347	840
353	1120
354	1940
360	2240
400	2970
420	8850

Cross Section No. 21 at River
Mile 362.30 Area Below 337 ft
 MSL = 1525 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
337	410
341	560
344	790
349	1230
360	1630
380	2534
400	3168
420	8448

Cross Section No. 22 at River
Mile 361.50 Area Below 340 ft
 MSL = 2870 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
340	840
343	1070
350	1280
360	1500
380	5100
400	10600
420	18200
440	24700

Cross Section No. 23 at River Mile 360.19
Area Below 331 ft MSL = 112 ft²

<u>Elevation</u> <u>(ft, MSL)</u>	<u>Surface</u> <u>Width (ft)</u>
331	225
334	500
341	785
350	1870
360	2490
380	3860
400	7090
460	10400

TABLE A.1. U.S. Army Corps of Engineers' Steady Flow Profiles

<u>River Mile</u>	<u>Elevation at 36,000 cfs</u>	<u>Elevation at 300,000 cfs</u>
394.49	395.2	418.1
392.42	393.0	416.0
390.10	390.0	413.0
388.34	388.0	410.0
386.66	386.1	408.0
384.72	385.0	405.0
382.05	380.2	398.0
380.13	379.6	393.2
378.34	378.0	391.1
375.89	374.0	389.2
374.17	373.2	388.0
372.34	369.0	385.0
371.61	368.0	384.0
369.63	365.0	380.9
368.13	363.0	377.0
367.00	362.0	376.0
366.09	359.2	374.3
364.38	354.1	373.0
362.51	352.0	371.0
362.30	351.5	370.3
361.50	350.8	369.3
360.19	349.0	368.0

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APPENDIX B

CALIBRATION AND VERIFICATION DATA

APPENDIX B

CALIBRATION AND VERIFICATION DATA

A. Observed Stage at River
Mile 361.50
May 8, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
1200	361.73
1300	361.75
1400	361.70
1550	361.54
1600	361.33
1700	361.10
1800	360.92
1900	360.80

B. Observed Stage at River
Mile 361.50
May 9, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
1000	357.86
1100	357.51
1200	357.36
1300	357.31
1400	357.36
1500	357.46

C. Observed Stage at River
Mile 361.50
September 17, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
1000	355.81
1100	354.92
1200	354.16
1300	353.80
1400	354.02
1500	354.73
1600	355.68

D. Observed Stage at River
Mile 361.50
September 17, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
900	355.20
1000	354.33
1100	353.84
1200	353.64
1300	353.94
1400	354.79
1500	355.98
1600	356.95
1700	357.73
1800	358.26
1900	358.63

E. Observed Stage at River
Mile 361.50
September 19, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
1000	353.67
1100	353.41
1200	353.32
1300	353.28
1400	353.29

F. Observed Stage at River
Mile 361.50
September 21, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
1000	353.05
1100	352.76
1200	352.53
1300	352.55
1400	353.18
1500	354.33

G. Observed Stage at River
Mile 361.50
September 22, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
830	355.4
930	354.8
1030	354.1
1130	353.5
1315	353.8
1415	354.7
1545	356.4

H. Observed Stage at River
Mile 361.50
September 23, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
1100	353.22
1200	352.96
1300	353.06
1400	353.67
1500	354.77

I. Observed Stage at River
Mile 361.50
September 24, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
900	354.54
1000	354.06
1100	353.76
1200	353.68
1300	354.06
1400	354.94
1500	355.96

J. Observed Stage at River
Mile 361.50
September 24, 1981

<u>Time</u>	<u>Stage (ft, MSL)</u>
800	355.46
900	354.83
1000	354.20
1100	353.58
1200	353.07
1300	352.86
1400	353.31

TABLE B.1. Inflows From Priest Rapids Dam, May 1981 Event

<u>Time</u>	<u>Discharge, cfs</u>	
	<u>May 8</u>	<u>May 9</u>
0000	168,700	132,200
100	163,100	116,500
200	154,700	115,200
300	157,500	116,500
400	157,500	116,600
500	156,100	101,600
600	157,500	98,300
700	140,000	89,000
800	158,900	89,000
900	171,600	100,500
1000	160,300	101,600
1100	158,900	101,600
1200	150,500	104,900
1300	145,200	122,400
1400	143,900	140,000
1500	141,300	147,800
1600	141,300	149,100
1700	147,800	140,000
1800	150,500	129,600
1900	147,800	124,800
2000	141,300	127,200
2100	137,400	138,100
2200	137,400	137,400
2300	147,800	80,300

TABLE B.2. Inflows From Priest Rapids Dam, September 1981 Event

Time	Discharge, cfs		
	<u>September 17</u>	<u>September 18</u>	<u>September 19</u>
0000	129,690	108,200	94,000
100	129,600	99,400	74,600
200	126,000	89,000	60,940
300	110,400	78,280	52,500
400	81,160	47,780	45,540
500	62,540	51,890	44,880
600	48,940	44,880	48,240
700	42,960	45,540	52,500
800	40,500	53,980	56,240
900	48,940	64,200	56,240
1000	70,160	89,000	53,980
1100	91,000	109,300	57,780
1200	102,700	118,800	67,560
1300	111,600	121,200	81,160
1400	108,200	129,800	95,000
1500	100,500	122,400	104,900
1600	99,400	120,000	110,400
1700	99,400	122,400	106,000
1800	102,700	127,200	99,400
1900	106,000	129,600	97,200
2000	108,200	136,100	87,000
2100	109,300	141,300	81,160
2200	110,400	122,400	75,520
2300	109,300	106,000	74,600

TABLE B.2. (contd)

Time	Discharge, cfs		
	September 20	September 21	September 22
0000	75,520	53,980	103,800
100	80,200	64,200	101,600
200	82,120	68,400	92,000
300	75,520	66,720	72,800
400	61,740	57,780	55,480
500	53,980	47,560	46,880
600	44,880	42,340	42,860
700	39,300	39,900	40,500
800	38,720	41,100	40,500
900	38,140	44,240	59,340
1000	41,100	67,560	89,000
1100	50,346	98,300	104,900
1200	65,040	107,100	111,600
1300	82,120	111,600	116,400
1400	94,000	115,200	117,600
1500	95,000	117,600	115,200
1600	90,000	117,600	110,400
1700	84,040	115,200	104,900
1800	71,920	111,600	101,600
1900	57,000	106,000	97,200
2000	48,940	97,200	95,000
2100	42,860	92,000	92,000
2200	39,900	94,000	90,000
2300	46,880	99,400	89,000

TABLE B.2. (contd)

<u>Time</u>	<u>Discharge, cfs</u>		
	<u>September 23</u>	<u>September 24</u>	<u>September 25</u>
0000	87,000	94,000	96,100
100	83,080	92,000	89,000
200	74,600	79,240	83,080
300	60,140	61,740	69,280
400	46,880	51,060	59,340
500	42,960	46,880	49,640
600	41,100	50,340	42,960
700	43,600	52,500	41,100
800	48,240	55,480	39,900
900	53,980	65,880	42,340
1000	69,280	85,000	61,740
1100	95,000	110,400	94,000
1200	111,600	114,000	109,300
1300	120,000	115,200	128,400
1400	121,200	120,000	137,400
1500	107,100	120,000	145,200
1600	90,000	118,800	128,400
1700	79,240	115,200	120,000
1800	79,240	107,100	110,400
1900	78,280	99,400	102,700
2000	79,240	91,000	100,500
2100	82,120	89,000	97,200
2200	90,000	94,000	94,000
2300	96,100	99,400	85,000

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