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## THESIS

AN ANALYSIS OF A NUMERICAL TIDAL MODEL  
APPLIED TO THE COLUMBIA RIVER

by  
Richard B. Koehler

September 1988

Thesis Advisor:  
Co-Advisor:

Edward B. Thornton  
Chung-Shang Wu

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**AN ANALYSIS OF A NUMERICAL TIDAL MODEL  
APPLIED TO THE COLUMBIA RIVER**

by

**Richard Bruce Koehler**  
Lieutenant , National Oceanic and Atmospheric Administration  
B.S., University of Arizona, 1978  
M.S., University of Arizona, 1986

Submitted in partial fulfillment of the requirements for the  
degree of

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from the

**NAVAL POSTGRADUATE SCHOOL  
September 1988**

Author:

*Richard B Koehler*

**Richard Bruce Koehler**

Approved by:

*Edward B. Thornton*

**Edward B. Thornton, Thesis Advisor**

*Chung-Shang Wu*

**Chung-Shang Wu, Co-Advisor**

*Curtis A. Collins*

**Curtis A. Collins, Chairman, Department of Oceanography**

*Gordon E. Schacher*

**Gordon E. Schacher, Dean of Science and Engineering**

## ABSTRACT

An implicit finite difference model for predicting flood routing is applied to the lower Columbia River, where tidal forcing causes flow reversals interacting with upstream dam flow during small river flow periods. The model is one-dimensional, unsteady, including lateral inflow and variable bed friction for different channel sections. A comparison of stages at six stations was made for a sensitivity analysis. The analysis used a total of 2209 hours of simulated river stages.

Downstream boundary changes of ~~±0.5~~ <sup>+ or -</sup> feet and ~~±2.0~~ feet were made to the Astoria tide stages. Model simulations showed that 70% of the tide difference appears at Vancouver and Portland, 80% at St. Helens, 85% at Longview, 93% at Wauna and 95% at Skamokawa. Varying the upstream boundary condition (Bonneville Dam discharges) by ~~±10%~~ and ~~±25%~~ were markedly different from the downstream boundary changes. Upstream, where the tide influence is weakest, the tidal cycle is more likely to be "washed out" by the higher flows of the Columbia. Also these changes fluctuated with the tide cycle. Downstream stations did not show such differences because of the larger cross section areas of the Columbia River nearer the mouth and the proximity to the downstream boundary condition.

The river system was calibrated in a downstream to upstream direction and used a total of 606 hours of observed river stages. Three periods with distinct river flow conditions were used in the calibration. Regression analyses of the computed residual values for each of the stations gave correlation coefficients ( $r^2$ ) less than 0.360. However, cross correlations between residual and computed stages showed that the two series were highly sinusoidally correlated for all stations. A spectral estimation of the residuals exhibited very strong peaks at frequencies of  $0.081 \text{ hr}^{-1}$  (12.3 hrs),  $0.042 \text{ hr}^{-1}$  (24.0 hrs) and subsequent harmonics of these frequencies. The residual components are strongly associated with the tidal cycle.

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## I. INTRODUCTION

### A. BACKGROUND INFORMATION

The accuracy of forecasted stages is critical for both navigational safety and commerce on any river system. This is especially true on the lower Columbia River as ship traffic increases. The Northwest River Forecast Center (NWRFC, part of the National Oceanic and Atmospheric Administration, NOAA) presently uses the Dynamic Wave Operational Model (DWOPER) to forecast hourly river stages on the lower Columbia River from the Vancouver-Portland area to Astoria, Oregon (Figure 1). This model was developed by Fread (1976), of the National Weather Service Office of Hydrology.

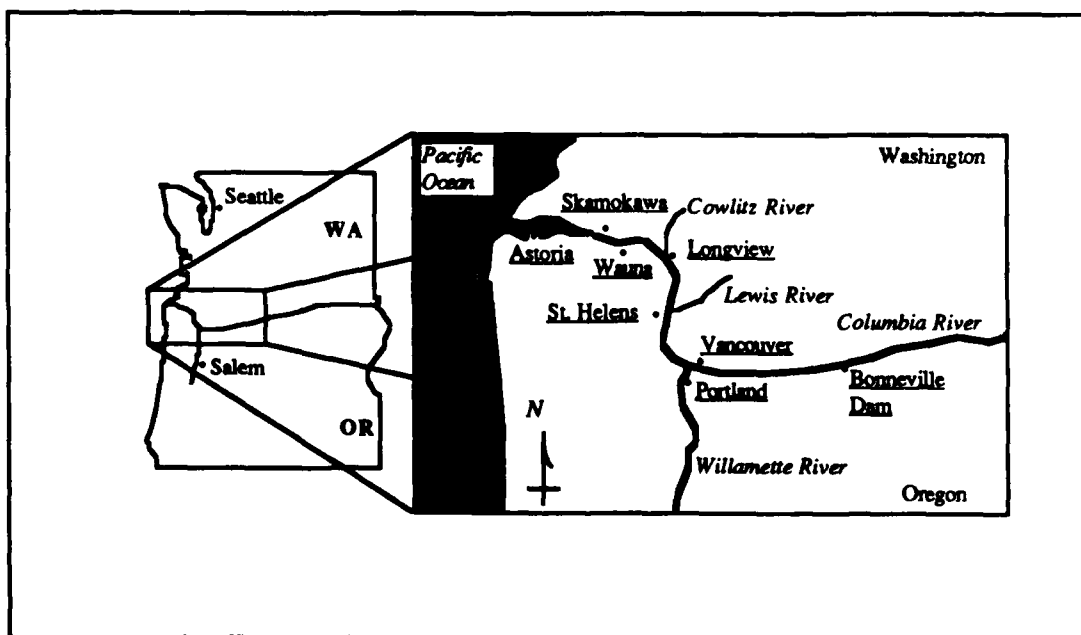


Figure 1. DWOPER Simulation Area (not to scale)

Currently, DWOPER is being used on the Lower Mississippi River System (470 km or 292 miles), the Mississippi-Ohio-Cumberland-Tennessee River System (633 km or 393 miles) as well as the Columbia-Willamette River System (230 km or 143 miles). It is the Columbia-Willamette System that has the most pronounced tidal influences.

The Columbia-Willamette River system is a major waterway in the northwest United States and has a shallow slope (0.011 m/km) below Bonneville Dam. Tidal effects on the river extend as far upstream as the Bonneville Dam tailwaters, 230 km (143 mi) from the ocean, during periods of low flow. Flow reversals can occur as far upstream as the Vancouver-Portland area, 170 km (106 mi) from the ocean (Fread, 1976, 1978). As an additional part of the river optimization process for ships, the U.S. Army Corps of Engineers has deepened the bar crossing at the Pacific Ocean to 16.8 m (55 ft) and maintains a 12.2 m (40 ft) channel to the Vancouver-Portland port area, a distance of 193 km (120 miles) from the ocean to accommodate larger draft vessels (Rooks, 1986).

## **B. OBJECTIVE**

This thesis examines the DWOPER computer model (program version 4/11/79) to find parameter values needed to best match computed stages with observed stages. Calibration periods come from the 1987 Water Year (October 1986 to September 1987). These comparisons are made at six stations for three different time periods.

Confidence intervals for forecasted stages are developed. Residual errors between model output and observed stages are examined as well. The modelling of

these residual errors and applying confidence intervals will aid in more accurate river stage forecasts and improve the DWOPER application.

### **C. AREA DESCRIPTION**

The Columbia River dominates the Pacific northwest from its source, Columbia Lake in Canada's Selkirk Mountain Range at 809 m (2650 ft) above mean sea level. It is the fourth largest river in North America with a course length of 1,953 km (1,214 mi) and a drainage area of 671,000 km<sup>2</sup> (259,000 mi<sup>2</sup>). This river system is important to agriculture, fisheries, power production, recreation and commerce in the surrounding area. The Columbia is one of the world's most managed rivers. Portland, Oregon, at the confluence of the Columbia and Willamette rivers, is the third busiest port on the west coast of the U.S. in terms of total tonnage shipped per year. There are 38 ports that are served by the Columbia system (including the Snake River) with the farthest inland being Lewiston, Idaho.

The Columbia River Management Report for 1987 stated that precipitation and runoff was significantly below normal in 1987 for the Columbia River Basin. Winter precipitation was generally 60% to 80% of normal. The spring runoff of the Columbia at The Dalles, Oregon, was about 70% of normal. The regulated peak flow was 7,950 m<sup>3</sup>/sec (284,000 ft<sup>3</sup>/sec) compared to a bankful flow of 12,600 m<sup>3</sup>/sec (450,000 cfs). This was the lowest runoff since 1977 and it ranks as the fifth lowest compared to the 1928-1978 period of record. Snake River runoff was only 46% of normal and ranked the second lowest during this same 50 year period (Columbia River Management, 1987 ).

Despite the low runoff, reservoirs in the power system refilled due in part to the Bonneville Power Administration (BPA) conservative marketing policy. However, irrigation reservoirs were at very low levels and had little carryover for

1988. The dry weather in the Cascade region caused several reservoirs in the Willamette basin not to refill in the spring. August and September were unusually dry in some areas and resulted in greater than usual reservoir drawdown by the end of the 1987 water year (Columbia River Management Group, 1987 ).

The Pacific Ocean tides are characterized as semi-diurnal (two highs and two lows per day) with a mean range of 2.4 to 3.0 m (8 to 10 ft). No unusual tidal phenomena (i.e. tsunami) occurred. The Tongue Point gauge at Astoria, Oregon is a primary station for the National Ocean Service (NOS) with more than 19 years of continuous record and is a published prediction site. Values for this study used observed Tongue Point records. These values were obtained from the Tidal Analysis office of National Ocean Service (NOS Pacific Tide Tables, 1987).

#### **D. PRESENT USES AND USERS**

The most dramatic use of DWOPER on the lower Columbia River occurred during the May, 1980, Mount Saint Helens volcano eruption in Washington state. The NWRFC used DWOPER to aid the U.S. Coast Guard in timing vessel traffic arrivals at the Longview constriction for safer passage (Orwig,1980). Vessels arrived at or near high water to have maximum water depth over the constriction created by the mud and debris flows from the volcano into the Columbia River. The use of the model during this crisis showed the benefits of forecasting river stages.

In February 1984, a Sea Use Marine Services conference was held in Portland, Oregon. Representatives of the Port of Portland (POP), U.S. Army Corps of Engineers (COE), National Weather Service (NWS) and National Ocean Service (NOS), both part of the NOAA, as well as public and private organizations attended. Two action items resulting from this meeting were the need to establish an automatic real-time river gauging network and the need to develop an hourly fore-

cast model for the tidal stages on the lower Columbia River. The real-time data and forecasts would be extremely valuable to the commercial river traffic and port authorities along the lower Columbia. It was at this time that preliminary work for using DWOPER for forecasting these hourly stages on the river began. Eventually it was decided that the Port of Portland would operate and maintain the gauging stations and the automatic data collection system. The NWRFC agreed to prepare and issue a three day forecast for the six sites on the river (Orwig et al., 1986).

The Port of Portland and the Columbia River Pilots Association, through the local merchants exchange group, have been distributing the three day forecasts from the NWRFC to area shipping interests. David Nessel, director of marine services for the Port of Portland has stated, "A foot of difference (0.3048 m) on a ship is a whole bunch of cargo and a whole bunch of revenue." He estimated the extra revenue at \$10,000 to \$20,000 for most grain ships. The extra feet are helping change the image of the Columbia River, which used to be thought capable of handling only smaller draft ships. Lawrence H. Bogle, plant manager of the Peavey Grain Company of Kalama, Washington explained that if the Columbia can accommodate the so-called Panamax ships, the maximum size of ship that can transit through the Panama Canal, it would mean that the local river shipping system "can compete with anybody, including the Mississippi River." Panamax ships have drafts up to 13.1 m (43.0 ft). In 1984, 28% of the cargo ships had a draft of more than 39 feet and the percentage of ships with larger drafts increased to 68% in 1985 (Rooks, 1986).

## **II. DWOPER MODEL DESCRIPTION**

### **A. BACKGROUND INFORMATION**

Unsteady flow in rivers, reservoirs and estuaries is caused by the motion of long waves from floods, tides, storm surges and reservoir releases. Since this flow can be considered as one-dimensional, the acceleration and velocity components of the flow in the transverse and vertical directions can be neglected, being small compared to the acceleration and velocity components in the direction of river flow. This allows the flow motion to be described by the one dimensional Saint-Venant equations of mass and momentum conservation (Fread, 1976).

The Dynamic Wave Model (DWOPER) allows for dynamic routing of rivers in such cases as

- upstream movement of tides and storm surges,
- reservoir backwater effects,
- flood waves on shallow sloped rivers, or
- abrupt waves from reservoir releases or dam failures.

### **B. THEORETICAL DEVELOPMENT**

Fread (1976) developed the Dynamic Wave model, assuming the following conditions:

- depth, velocity and acceleration vary only in the longitudinal axis of the river,
- velocity is constant for any cross-sectional area of the river and that the water surface is horizontal to any section perpendicular to the longitudinal axis of the river,
- the longitudinal axis of the river is approximately a straight line,
- channel bottom slope is small,

- no erosion or deposition along the channel bottom occurs,
- flow is gradually varied with the hydrostatic pressure,
- the Manning equation describes the resistance effects and
- the fluid is incompressible and homogeneous.

The model equations are:

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

(Note: The conservation of mass for steady state flow is expressed as  $Q = VA$ ).

conservation of momentum: 
$$\frac{\partial Q}{\partial t} + \frac{\partial \left( \frac{Q^2}{A} \right)}{\partial x} + gA \left[ \frac{\partial h}{\partial x} + S_f + S_e \right] - qv_x = 0 \quad (2)$$

friction slope: 
$$S_f = \frac{n^2 |Q| Q}{2.2 A^2 R^{4/3}} \quad (3)$$

energy loss slope: 
$$S_e = \frac{K_e \partial \left( \frac{Q}{A} \right)^2}{2g \partial x} \quad (4)$$

where,

$Q$  = discharge,  $A$  = cross sectional area,  $x$  = distance along the channel,

$A_0$  = off channel cross sectional area (flow velocity is considered negligible),

$q$  = lateral inflow or outflow,  $t$  = time,

$v_x$  = velocity of lateral inflow in the  $x$  direction,

$g$  = gravity acceleration constant,  $B$  = channel top width,  $S_f$  = friction slope,

$K_e$  = contraction/expansion factor.

The  $K_e$  term is used to account for local, small scale head losses that may form in the river flow at abrupt cross section changes along the river such as a bridge support column. For a surface cross section of width  $b$ , the affect is usually limited to a distance of about  $10 \cdot b$  (personal conversation, Fread). The  $K_e$  term was set to zero for this study.

Equations (1) and (2) are nonlinear partial differential equations that can be solved by explicit or implicit finite difference techniques. Explicit techniques are generally not well suited for application to long period, unsteady flow situations such as flood or tide waves. These techniques are restricted by numerical instability reasons to very small computational time steps (the order of a few minutes). This would be a poor and inefficient use of computer resources. On the other hand, implicit finite difference techniques have less restriction on time step size, except for accuracy considerations.

The "weighted four point" method used in DWOPER was first used by Preissmann (1961). This method has the advantage of using unequal distance steps and controlled stability-convergence properties (Fread, 1978). A continuous  $x, t$  region represented by a grid of discrete points allows for solutions of both  $h$  and  $Q$  at equal or unequal intervals of  $\Delta x$  and  $\Delta t$  along the  $x$  and  $t$  axes where  $i$  is the  $x$  position and  $j$  the time position (see Figure 2). The time derivative for any variable,  $k$ , is approximated by equation (6).

$$\frac{\partial k}{\partial t} \approx \frac{[k_i^{j+1} + k_{i+1}^{j+1} - k_i^j - k_{i+1}^j]}{2\Delta t} \quad (6)$$

The spatial derivative is approximated by a finite difference quotient positioned between two adjacent lines (see Figure 3) according to the weighting factors  $\theta$  and  $(1-\theta)$ , such that:

$$\frac{\partial k}{\partial x} = \theta \frac{[k_{i+1}^{j+1} - k_i^{j+1}]}{\Delta x} + (1-\theta) \frac{[k_{i+1}^j - k_i^j]}{\Delta x} \quad (7)$$

with the non-derivative terms approximated by:

$$k = \theta \frac{[k_i^{j+1} - k_{i+1}^{j+1}]}{2} + (1-\theta) \frac{[k_i^j - k_{i+1}^j]}{2} \quad (8)$$

When  $\theta$  equals 1.0, a fully implicit scheme is formed (Baltzer and Lai, 1968) and when  $\theta$  equals 0.5, a box scheme is formed (Amein and Fang, 1970). Fread (1975) studied the stability and convergence properties and concluded that the accuracy decreases as  $\theta$  departs from 0.5 and approaches 1.0. This effect is more pronounced as time step size increases. The model version used in this study allows  $\theta$  to be an input variable with 0.55 used as the default value. This value is used to minimize loss of accuracy.

Substitution of equations (6), (7), (8) yields two algebraic equations that are nonlinear with respect to  $h$  and  $Q$  at the nodal points on the  $j+1$  time line. All terms from the  $j$  time line are known from either initial conditions or previous calculations. The initial conditions are values of  $h$  and  $Q$  at each computational point or node along the  $x$ -axis for the  $j+1$  time line.

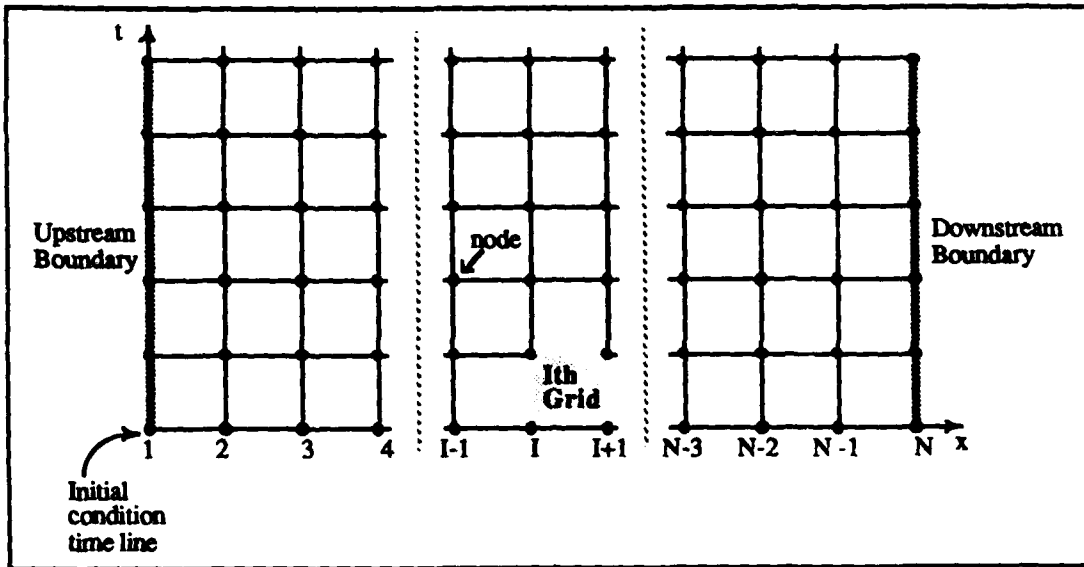
The two algebraic equations cannot be solved directly since there are four unknowns ( $h$  and  $Q$  at points  $i$  and  $i+1$  on the  $j+1$  time line). If, however, similar

equations are formed for each of the  $N-1$   $\Delta x$  reaches ( $N$  equals the number of cross section computational points on the river), a total of  $(2N-2)$  equations with  $2N$  unknowns results. These equations along with the upstream and downstream boundary conditions yield a system of  $2N$  nonlinear equations with  $2N$  unknowns, which are solved by the Newton-Raphson iteration procedure method (Fread, 1976). The tolerance of the residuals produced by the iteration method can be specified as a model input parameter and was 0.001 in this study.

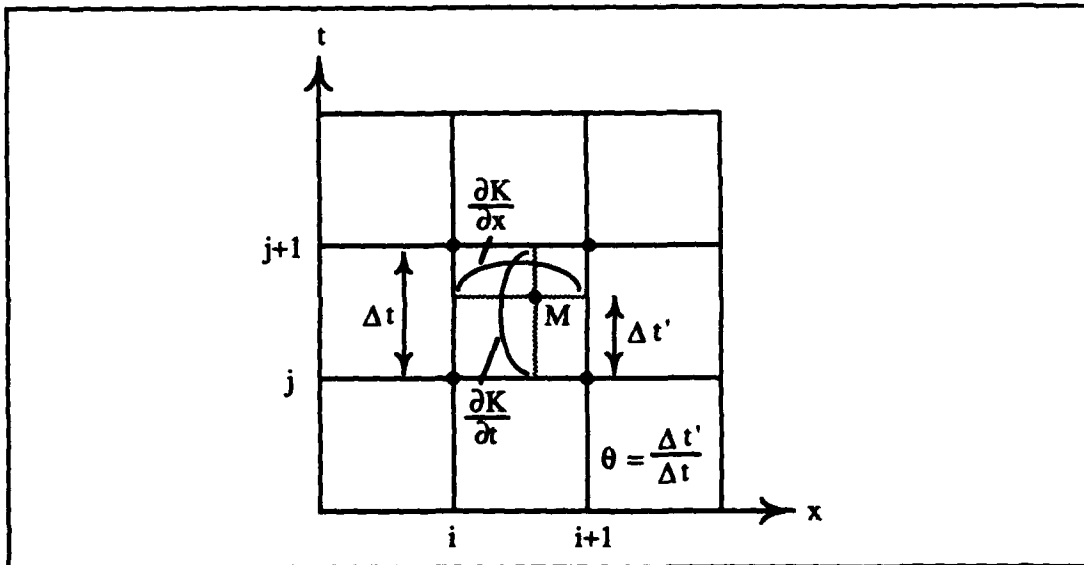
The Dynamic Wave model allows initial conditions to be obtained from:

- estimated stages and discharges at each cross section,
- observed stages at each cross section where a gauge may be located with
  - 1) stages at intermediate cross sections linearly interpolated within the model,
  - 2) observed discharges from the upstream boundary, or
  - 3) with downstream discharges found by summing flows from the upstream boundaries of the main river and any dynamic tributaries plus any lateral inflows,
- saved values of stages and discharges from previous simulations, and
- assumed steady state flow to obtain discharges and backwater computations to obtain stages (Fread, 1978).

In each case, the unsteady flow equations are solved for several time steps using the initial conditions together with boundary conditions which are held constant during the time steps. This allows the errors in the initial conditions to be dampen out which results in the initial conditions being more nearly error free when the actual simulation begins and transient boundary conditions are used.



**Figure 2. Numerical Grid Configuration and Boundary Conditions**



**Figure 3. Weighted Four-Point Implicit Finite Difference Method**

### C. BOUNDARY CONDITIONS

Boundary conditions must be specified in order to solve the Saint-Venant equations. In most unsteady flow problems, the unsteady disturbance is introduced into the flow at the boundaries or extremities of the river system. DWOPER can accommodate either known  $h$  or  $Q$  [ $h_1(t)$  or  $Q_1(t)$ ] as an upstream condition. The model can also accommodate either known  $h$  or  $Q$  as a downstream condition [ $h_N(t)$  or  $Q_N(t)$ ] or use a rating curve option (flow versus stage relationship). If the rating curve option is used, the rating may be single-valued and read in as tabular values. Intermediate points are linearly interpolated. The rating may also be a loop rating curve generated internally from cross section and roughness coefficients for the downstream extremity that have already been entered into the program and the instantaneous water surface slope at the previous time step. However, the rating curve option was not used in this study. Ocean tidal affects on the river makes any flow-stage relation changeable with time (Fread, 1978).

Boundary conditions in this study are Bonneville Dam discharge, Willamette River flow below the Estacada River confluence and the Pacific Ocean tides at Tongue Point, Oregon (near Astoria, Oregon). Flow data are instantaneous values at one hour time intervals. If the Willamette River observed flow record had irregularly spaced data, intermediate flow values were linearly interpolation from the observed data. There was no missing data from Bonneville Dam or the Tongue Point gauge.

A sample printout from the DWOPER model is shown in Figure 4. Both computed flow (Q-FCST) and computed stage (H-FCST) are displayed with the observed stages for Skamokawa, Washington.

RIVER 1, STATION 21 SKAMOKAWA FLOOD STAGE = 14.00 FEET									
* COMPUTED	+---OBSERVED								
TIME	-2.	0.	2.	4.	6.	8.	Q-FCST	H-FCST	H-OBS
10/16/13.	\$	.	.	.	.	**	-260.84	7.67	7.86
10/16/14.	\$	.	.	.	.	+	-11.91	7.33	7.41
10/16/15.	\$	.	.	.	+	.	311.14	6.02	6.09
10/16/16.	\$	.	.	.	+	.	493.99	4.54	4.63
10/16/17.	\$	.	.	+	.	.	508.14	3.23	3.18
10/16/18.	\$	.	+	.	.	.	483.79	1.94	1.96
10/16/19.	\$	*	+	.	.	.	486.42	0.61	0.96
10/16/20.	*\$	+	.	.	.	.	416.19	-0.27	0.32
10/16/21.	\$	*	+	.	.	.	226.16	0.44	0.72
10/16/22.	\$	.	.	**	.	.	27.99	2.18	2.37
10/16/23.	\$	.	.	*	+	.	-144.99	4.11	4.42
10/17/ 0.	\$	.	.	.	*	+	-236.32	5.62	6.02
10/17/ 1.	\$	.	.	.	.	**	-257.95	6.74	7.01
10/17/ 2.	\$	.	.	.	.	**	-106.87	6.97	7.13

**Figure 4. Example Hydrograph Output from DWOPER**

#### **D. OTHER MAJOR FACTORS**

The model uses a total of 29 separate river reaches (24 for the Columbia River and 5 for the Willamette River), each with cross sectional information and river mile location and Manning n (roughness coefficient) values. An explanation of each of these items follows.

##### **1. Cross Sectional Areas**

Cross sections can be entered as either regular or irregular shaped areas. Each is read in as tabular values of channel width and elevation, together giving a piece-wise linear relationship. A low flow area which can be zero is used to describe the cross-section below the minimum elevation inputted. During the solution of the Saint Venant equations, areas or widths associated with a particular water surface elevation are linearly interpolated from the piece-wise linear relationships inputted. Cross sections from gauging station locations are always used as computational points in the x-t plane. The average cross sectional area is a weighted average of the cross sectional properties of the intervening reach. The

distance between each cross section is used as the weighting factor. Observed gauging stations and cross section locations used in this study are in Table 1.

## **2. Lateral Inflow Sources**

The dynamic wave model allows lateral inflow tributaries (non-dynamically routed tributaries) that account for the  $q$  term in equations (1) and (2). These flows are considered independent of the main river flow and are read in the program at constant or varying time intervals. Any location along the main river or a dynamically routed tributary may be specified. Flow units are cubic feet per second (cfs) with outflows being negative values. Flows at other times are linearly interpolated between the appropriate inputted time values. Lateral inflow locations are shown in Table 2. The Vancouver local inflow term in Table 2 simply refers to the cumulative inflows from all streams and creeks between Bonneville Dam and Vancouver, Washington

## **3. Friction terms**

The Manning  $n$  roughness coefficient is used to describe the friction along the river bed caused by vegetation, obstructions, bend effects or eddy losses and is defined for each group of reaches bounded by gauging stations (see Figure 5). These roughness coefficients can be entered either as a function of discharge or stage height. Simulations can vary markedly with small changes in the roughness coefficients and any trial and error type calibration requires a knowledge of how this input data will affect model output.

In this study, all roughness coefficients are functions of the river flow and not stage. The  $n$  values are read into the program as tabular values for a specific flow with intermediate values linearly interpolated. All input parameters used in this study are listed in the appendix.

**TABLE 1. GAUGING STATION AND CROSS SECTION LOCATIONS**

Columbia River			Willamette River		
#	Mile	Description	#	Mile	Description
1	145.5	Bonneville Dam	1	24.2	Gladstone, OR
2	143.3	Ives Island	2	18.6	Milwaukie, OR
3	141.1	Warrendale, WA	3	12.8	<i>Portland, OR</i>
4	132.0	Sand Island	4	6.4	Saint Johns, OR
5	122.9	Washougal, WA	5	0.0	Columbia Conf.
6	114.7	East Government Island			
7	106.5	<i>Vancouver, WA</i>			
8	103.8	East Hayden Island			
9	101.6	<i>Willamette River*</i>			
10	101.1	<i>Willamette River**</i>			
11	92.5	Bachelor Point			
12	87.0	Lewis River*			
13	86.9	Lewis River**			
14	86.1	<i>Saint Helens, OR</i>			
15	75.0	Kalama, WA			
16	67.6	Cowlitz River*			
17	67.5	Cowlitz River**			
18	66.1	<i>Longview, WA</i>			
19	53.9	Bradbury Slough			
20	41.6	<i>Wauna, OR</i>			
21	33.7	<i>Skamokawa, WA</i>			
22	30.2	Three Tree Point			
23	23.4	Miller Sands			
24	17.5	<i>Tongue Point</i>			

Gauged stations are in italics

\* = Upstream Confluence  
\*\* = Downstream Confluence

**TABLE 2. LATERAL INFLOW LOCATIONS**

Columbia River		
#	Mile	Description
1	114.7	Vancouver Local
2	86.9	Lewis River
3	67.6	Cowlitz River

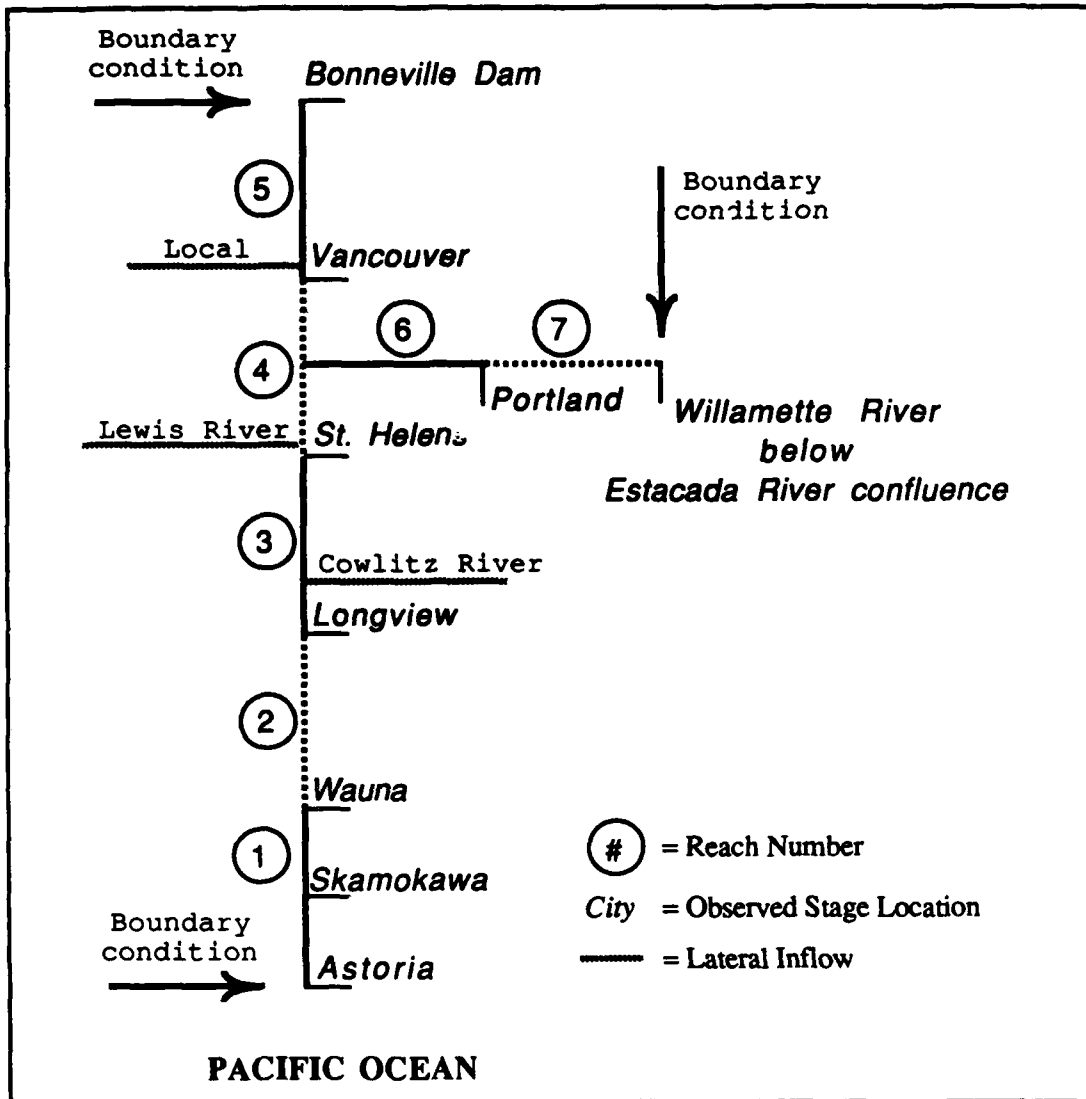


Figure 5. Schematic Diagram of the Lower Columbia River

### III. MODEL ANALYSIS

#### A. COMPARISON DATA

As with any mathematical model, the accuracy of the model results can only be as good as the information that goes into the program. In this study of DWOPER, data for all lateral flows and boundary conditions, except Astoria tides, were obtained from the Northwest River Forecast Center, Portland, Oregon. Ocean tide values were obtained from the NOAA's National Ocean Service (NOS) tidal analysis office. Data obtained from NOS must be adjusted to the datum the user is interested in using. Figure 6 shows different gauge readings for the same water level using three different datums; Mean Lower Low Water (MLLW), Mean Higher High Water (MHHW) and the National Geodetic Vertical Datum (NGVD) of 1929. In this study, all gauges are referenced to NGVD by means of the Columbia River Datum.

The Columbia River Datum is a sloping line that connects the zero gauge readings of the observation stations (see Figure 7) and represents the approximate lowest water surface plane. Water surface levels for the Columbia River seldom go below this plane. Any dredging operations for maintaining the navigational channel are also referenced to this plane and so any location on the river will have some cross sectional area below this low water surface. The Columbia River Datum can also be viewed as the amount added to a gauge reading to reference that reading to NGVD.

Since the model allows for an offset to be applied to any gauge, it was decided that the Astoria gauge values would be read into the model referenced to MLLW.

Within the model, an offset of -3.22 feet was added to adjust the Astoria gauge values to NGVD. By using this method, NOS data could be compared to predicted tides (also referenced to MLLW) and any questionable values could be easily located (no bad values were found however).

The observed stage values acquired are not as reliable as the boundary condition values. Figure 8 is an example of the periods with missing or bad data (Wauna, Oregon in this case). Because some observed stage data are questionable at certain periods, a full year of data for the model were not obtained from the NWRFC office. This was a problem for all of the upstream stations on the Lower Columbia River at one time or another. Only daily high and low stages were gathered for Portland during the 1987 Water Year. Daily high and low stages were gathered for Vancouver prior to May 13, 1987. After that time, hourly stage readings were available for Vancouver.

Since DWOPER does not use observed upstream station stages in simulation computations, questionable observed stage data will not affect the model output. Because the computed stages are independent of the observed values, a continuous series of computed stages were used as comparison values in the sensitivity analysis. The set of comparison data was computed from 1 October 1986, 0000 hrs to 1 January 1987, 0000 hrs (all time is referenced to Pacific Standard Time). These dates represent 2209 simulation hours. Since error-free, continuous records for upstream stations was not possible for the 1987 Water Year, three periods of distinct flow conditions were used in calibrating the model.

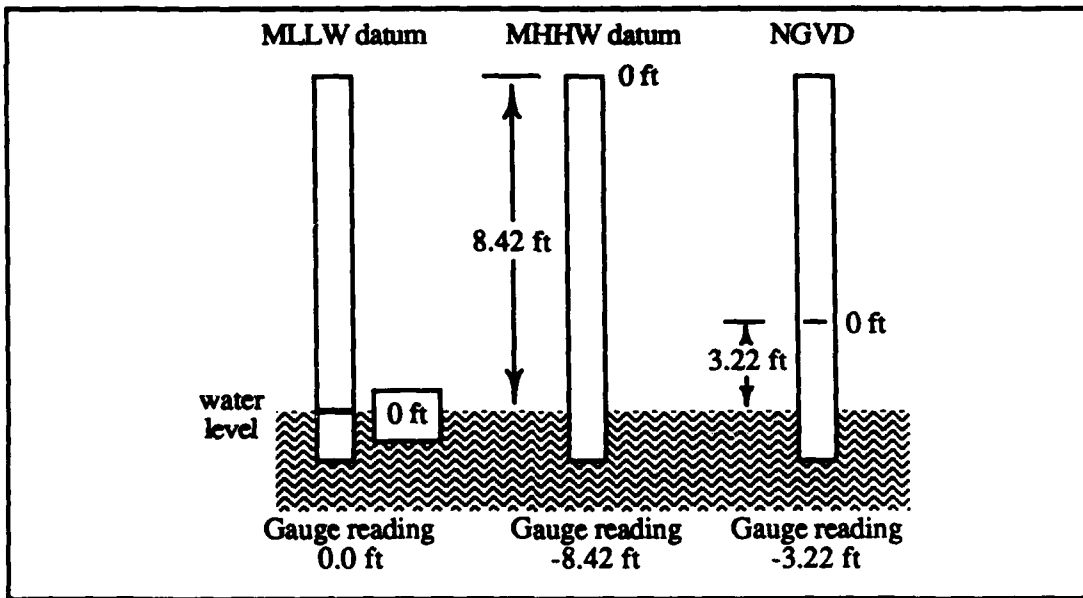


Figure 6. Datum Comparisons for Astoria, Oregon

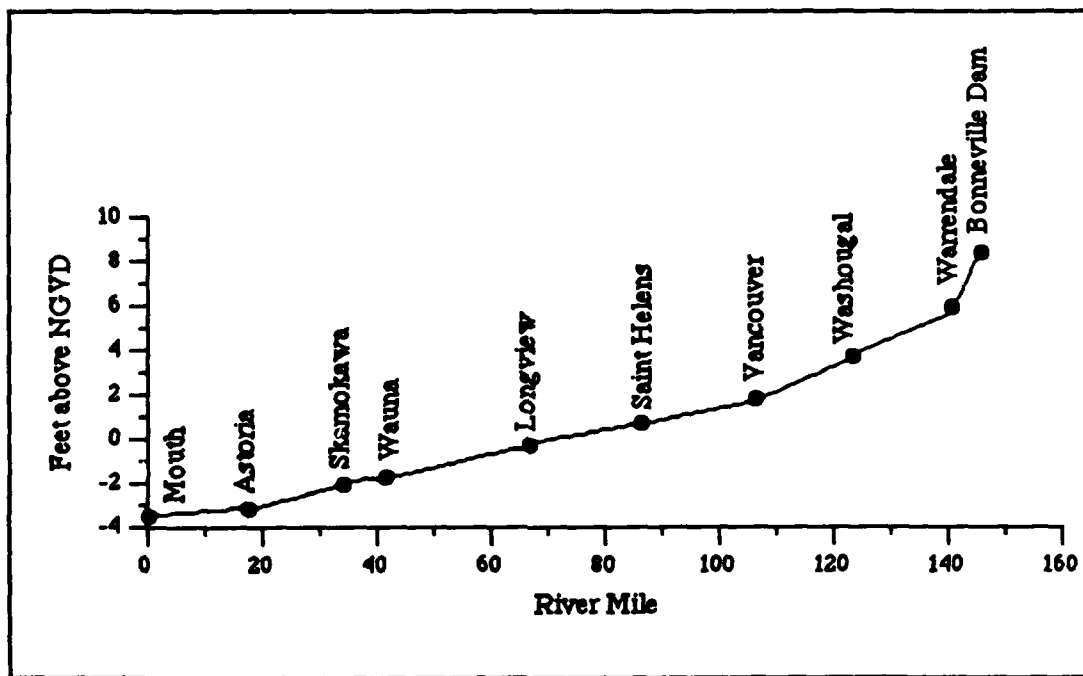


Figure 7. Station Relations for the Columbia River Datum

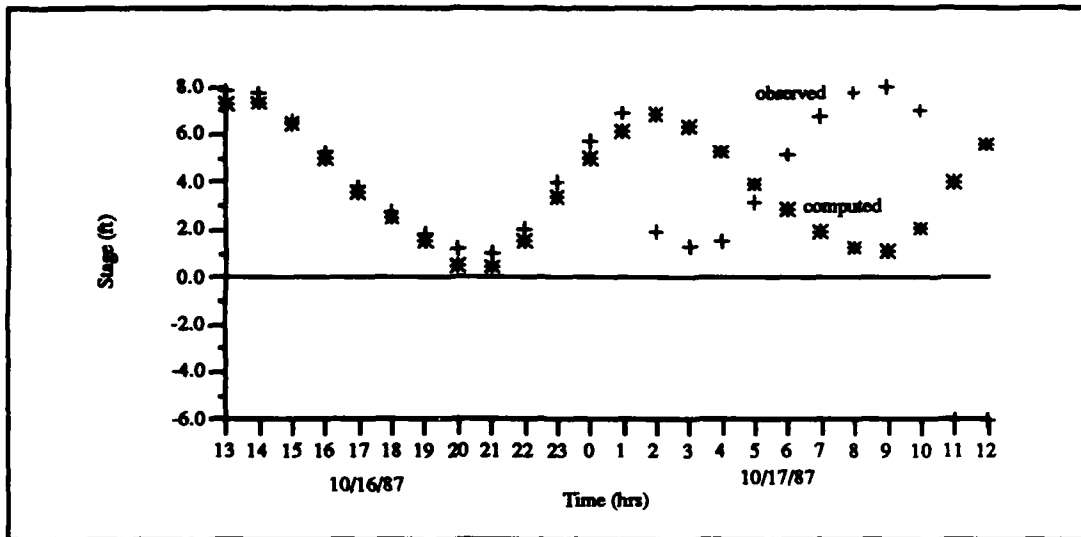


Figure 8. Missing and Questionable Observed Stages at Wauna, Oregon

## B. SENSITIVITY ANALYSIS

### 1. Friction Term Variations

To examine how DWOPER output changed with varying roughness coefficients, the Manning n values for a single river reach were increased by 0.01 and a simulation conducted. After the simulation, the original Manning n values were decreased by 0.01 and another simulation was conducted. In each simulation only one reach had changed n values, all other reaches remained unchanged. In order to evaluate these changes, computed values made with unaltered coefficients were used as comparison values. This was done to isolate changes due to the different Manning n's and not to be confused with the inability of the model to match actual conditions. In all cases, a constant of  $\pm 0.01$  was applied to existing data (Figures 9 and 10). In the case of decreased n values, the computed low water

stages are much lower than the comparison stages but high waters changed very little. Increasing the roughness coefficients caused low water stages to be much higher with high water stages slightly increased. The summaries presented in Tables 3 and 4 show the results of these variations with Figures 11 and 12 showing how these roughness coefficient changes affect model results for Vancouver and Skamokawa. In all cases, the largest rms and bias occurred when the reach just downstream of a station had altered roughness coefficients. Roughness coefficients changes caused the greatest modification of stages just upstream of the adjusted reach.

The equations used to compute the bias (mean error) and root mean square (rms) are as follows ( $O_i$  = observed stage,  $C_i$  = computed stage) :

$$\text{bias} = \frac{\sum_{i=1}^n (O_i - C_i)}{n} \quad (9)$$

$$\text{rms} = \sqrt{\frac{\sum_{i=1}^n (O_i - C_i)^2}{n}} \quad (10)$$

The conclusion from these simulations is that upstream stations are influenced by downstream Manning n changes but downstream stations are little affected by upstream changes. Any optimizing procedure that is a trial and error adjustment of the roughness coefficients must begin at the most downstream station and progress up the river system, with the dynamic tributaries the last to be calibrated.

**TABLE 3. DECREASED MANNING n SIMULATIONS**  
(n values increased by 0.01, values in feet)

Station		Reach				
		1	2	3	4	5
Portland	bias	0.110	0.566	0.598	0.566	-0.012
	rms	0.268	0.629	0.624	0.626	0.152
Vancouver	bias	0.102	0.564	0.596	0.655	-0.022
	rms	0.237	0.627	0.621	0.722	0.151
St. Helens	bias	0.195	0.731	0.737	0.058	0.008
	rms	0.373	0.817	0.770	0.161	0.090
Longview	bias	0.177	0.769	-0.045	0.017	0.020
	rms	0.430	0.914	0.227	0.159	0.094
Wauna	bias	0.834	-0.073	0.009	0.014	0.001
	rms	1.553	0.250	0.084	0.055	0.028
Skamokawa	bias	0.570	-0.038	0.008	0.008	0.000
	rms	1.148	0.148	0.058	0.037	0.018

**TABLE 4. INCREASED MANNING n SIMULATIONS**  
(n values increased by 0.01, values in feet)

Station		Reach				
		1	2	3	4	5
Portland	bias	-0.277	-0.707	-0.874	-0.736	0.005
	rms	0.351	0.768	0.910	0.807	0.092
Vancouver	bias	-0.269	-0.702	-0.873	-0.846	0.010
	rms	0.329	0.763	0.908	0.921	0.093
St. Helens	bias	-0.364	-0.835	-1.012	-0.057	-0.003
	rms	0.458	0.910	1.055	0.143	0.060
Longview	bias	-0.355	-0.884	0.040	-0.023	-0.010
	rms	0.482	0.984	0.250	0.136	0.060
Wauna	bias	-0.841	0.054	-0.010	-0.012	0.000
	rms	1.245	0.179	0.092	0.048	0.018
Skamokawa	bias	-0.601	0.028	-0.008	-0.006	0.001
	rms	0.949	0.107	0.066	0.032	0.011

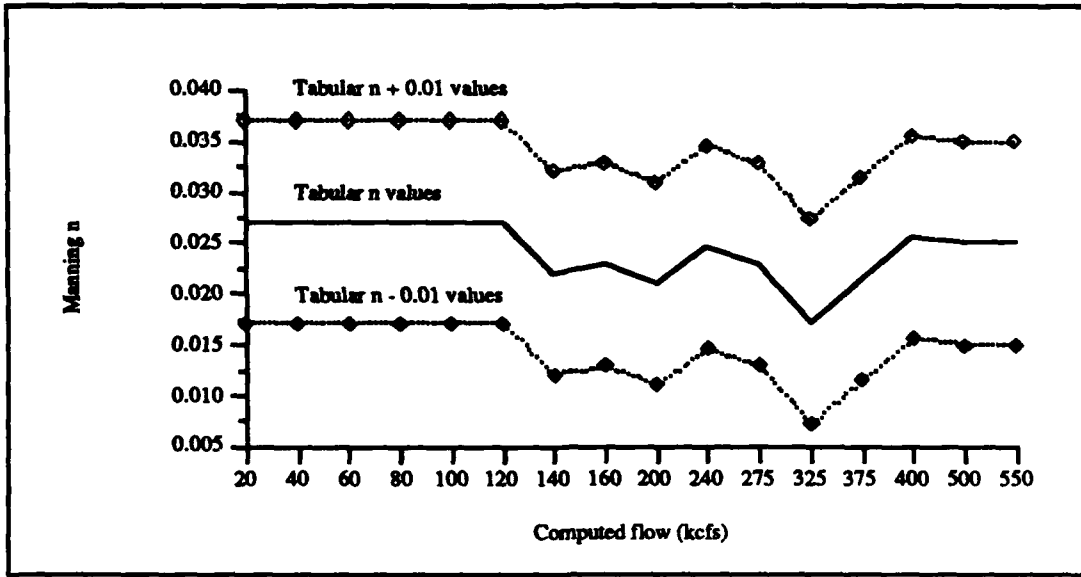


Figure 9. n-Q Relations for Vancouver, Washington

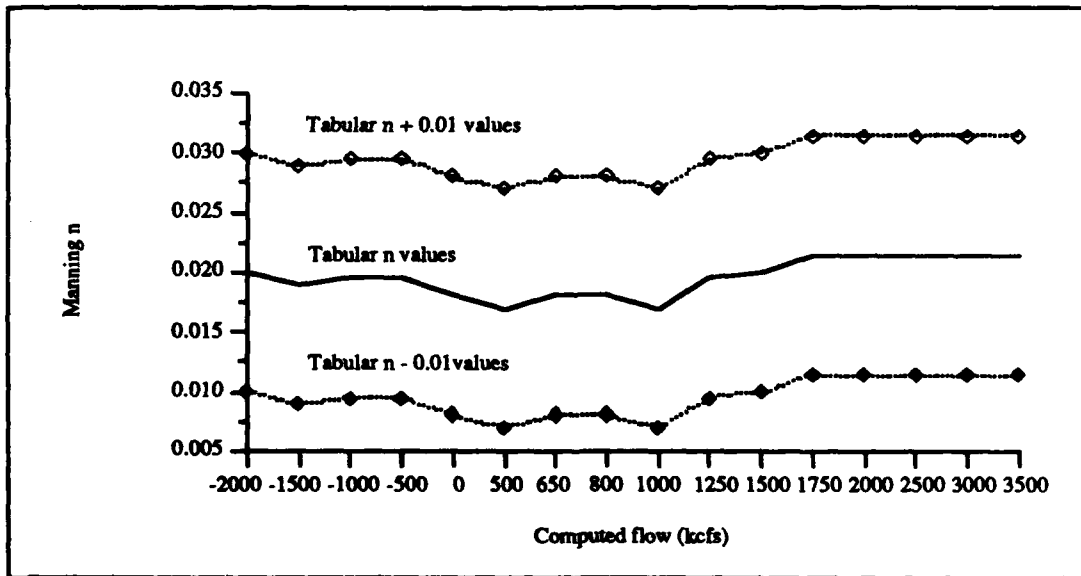


Figure 10. n-Q Relations for Skamokawa, Washington

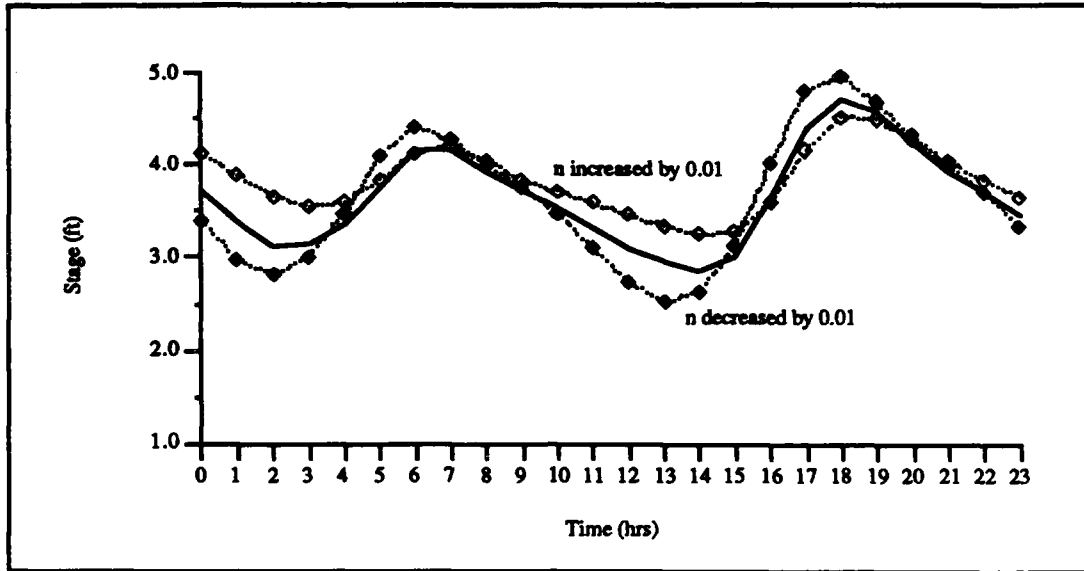


Figure 11. Vancouver Stages, Reach 1 Manning n Variation

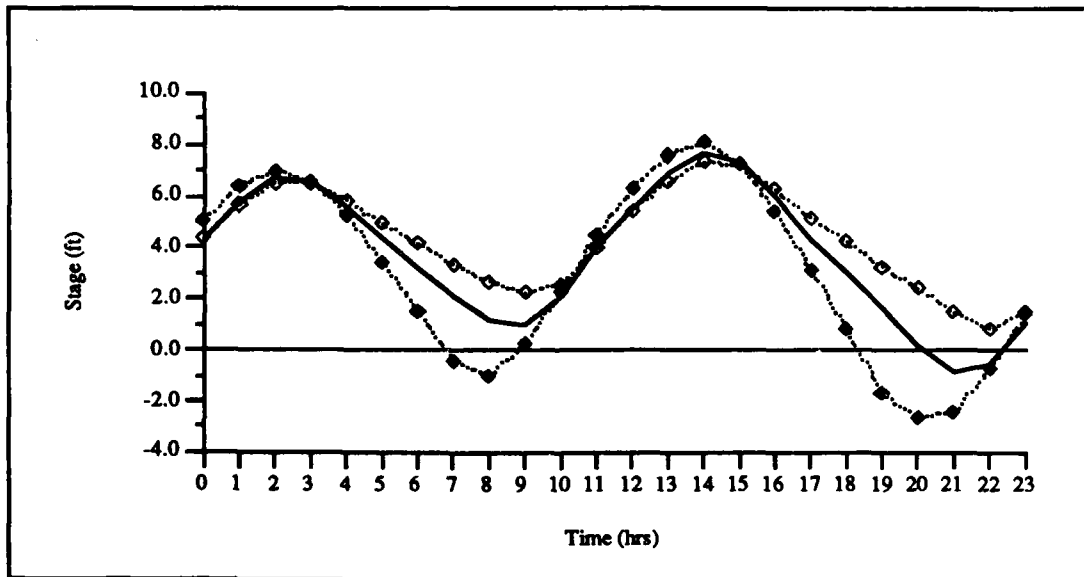
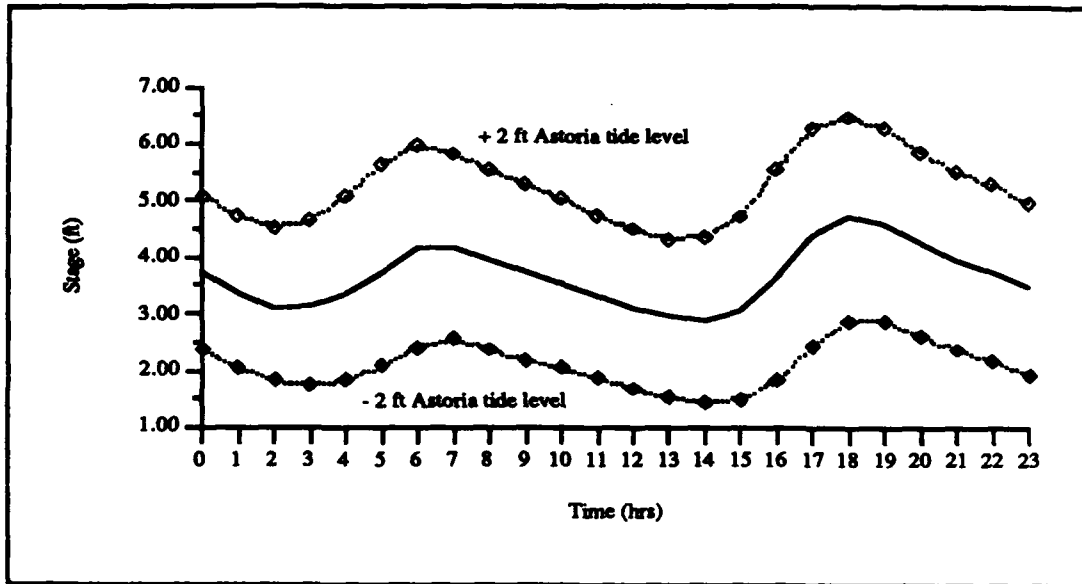


Figure 12. Skamokawa Stages, Reach 1 Manning n Variation

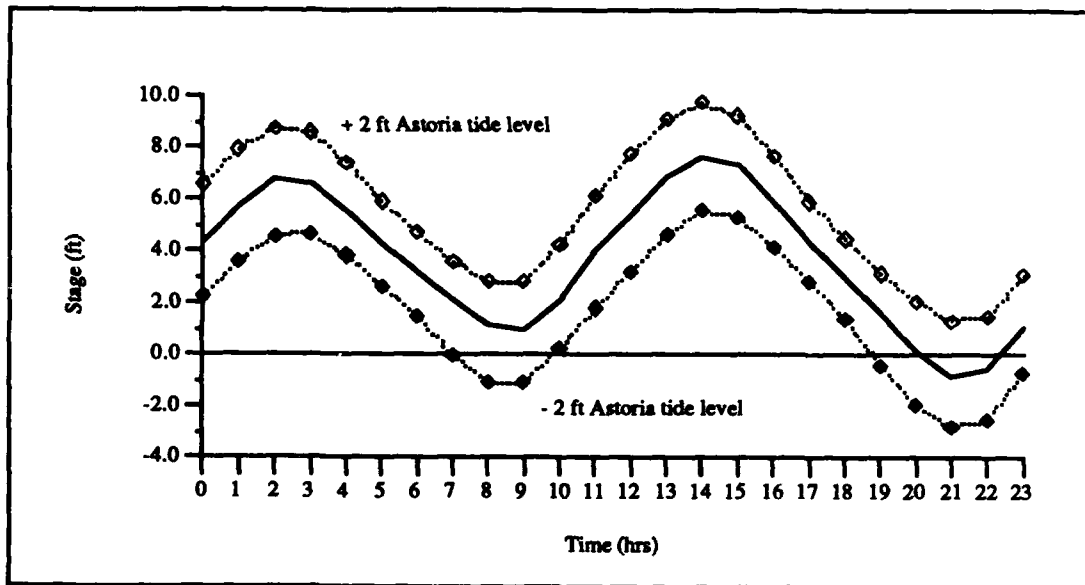
## **2. Boundary Condition Variations**

### ***a. Astoria tide variations***

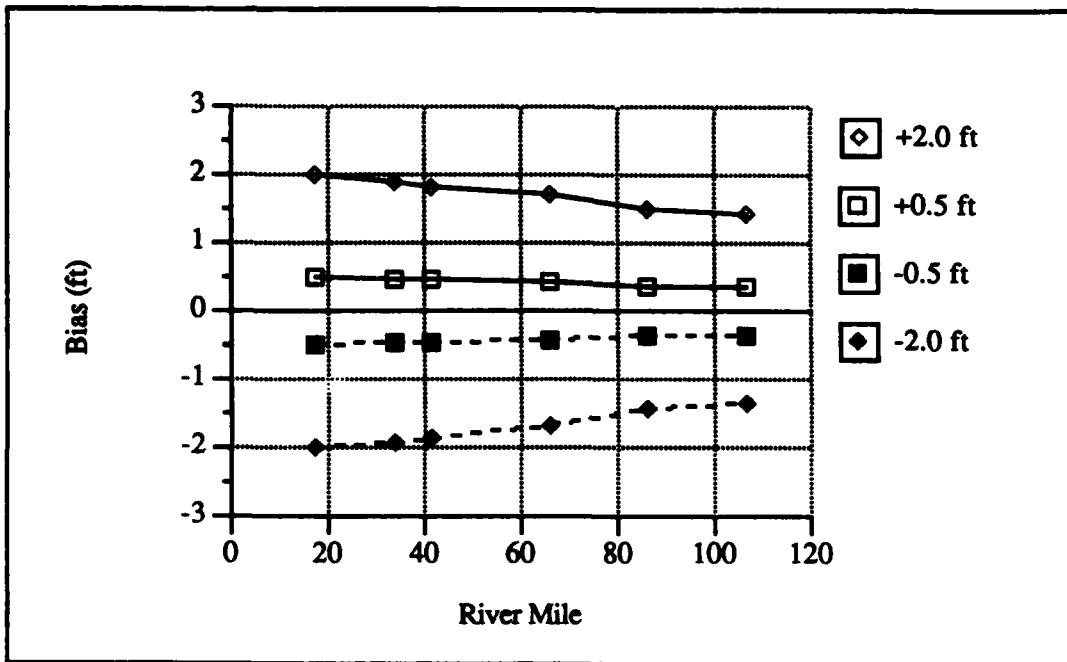
The Tongue Point tide gauge near Astoria, Oregon is the downstream boundary for DWOPER. As part of the sensitivity study, the tide values were uniformly changed by  $\pm 0.5$  feet and  $\pm 2.0$  feet. Figures 13 and 14 show how the  $\pm 2$  ft. change at Astoria affected model output at Vancouver and Skamokawa over a one day period. Figure 15 summarizes mean values of the upstream stations and how they are affected by these changes in regard to river mile. Note that in Figure 15, the y axis is tide difference, not actual tide. Any change in the input tide values will influence model computations far upstream. Model simulations with the new downstream boundary conditions showed that 70% of the tide difference appears at Vancouver and Portland, 80% at St. Helens, 85% at Longview, 93% at Wauna and 95% at Skamokawa. In all cases, the differences are between previously computed comparison values and newly computed values. These percentages are approximately the same with a  $\pm 0.5$  feet or  $\pm 2.0$  feet boundary condition change. This is due in part to the boundary condition being stage and not flow values. Any increase (or decrease) of the stage will considerably change the amount of water entering or leaving the system. As the tide wave travels up the shallowed sloped Columbia, it will be modified by decreased cross sectional areas and channel widths. Also dampening the tide wave will be the freshwater runoff.



**Figure 13. Downstream Boundary Variation Effects at Vancouver, Washington**



**Figure 14. Downstream Boundary Variation Effects at Skamokawa, Washington**



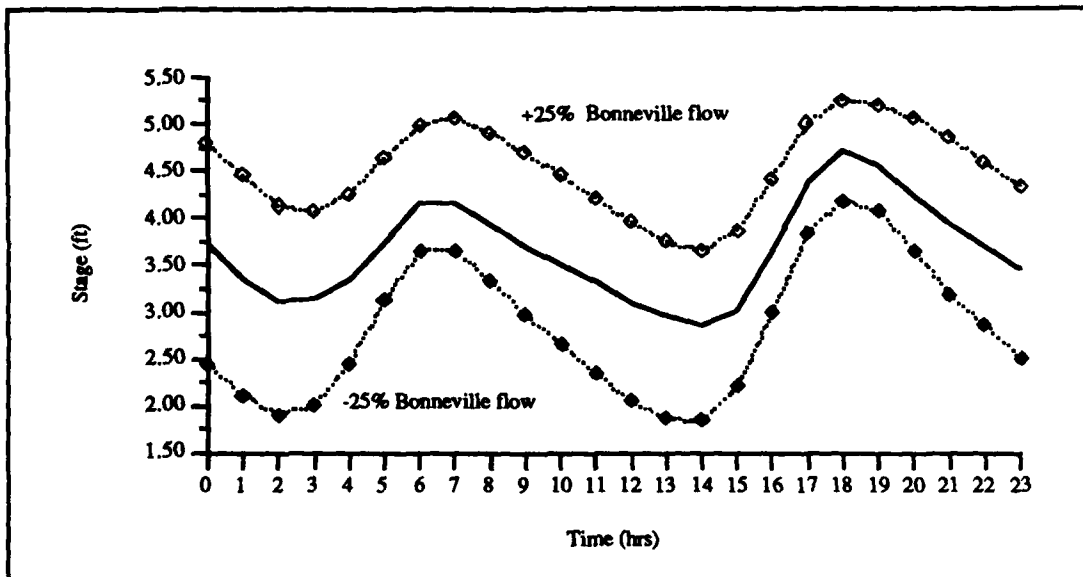
**Figure 15. Summary of Downstream Boundary Variation Effects on Upstream Stations**

***b. Bonneville Dam discharge variations***

The affects of varying Bonneville flows (the upstream boundary condition) by  $\pm 10\%$  and  $\pm 25\%$  are markedly different than Astoria tidal stage changes as shown in Figures 16 and 17. The decreasing bias is caused by a combination of the increased river cross sections further downstream and proximity to the downstream boundary condition. Upstream, where the tide influence is weakest, the tidal cycle is more likely to be “washed out” by the higher flows of the Columbia. Also, the change from comparison stages by these Bonneville flow variations are not constant over time and fluctuate with the tide cycle. Figure 16 shows a larger difference for the -25% Bonneville discharge

variation at low water than at high water with Figure 18 summarizing these differences for the entire river. Figure 18 also shows that stage biases caused by varying Bonneville outflows are proportional.

As stations further downstream are analyzed, the affects of the Bonneville flow variation are almost insignificant. In Figure 17, the three different curves are virtually indistinguishable. This is due in part to the larger cross section areas of the Columbia River nearer the mouth and the proximity of Skamokawa, Washington to the downstream boundary condition (Astoria, Oregon).



**Figure 16. Upstream Boundary Variation Effects at Vancouver, Washington**

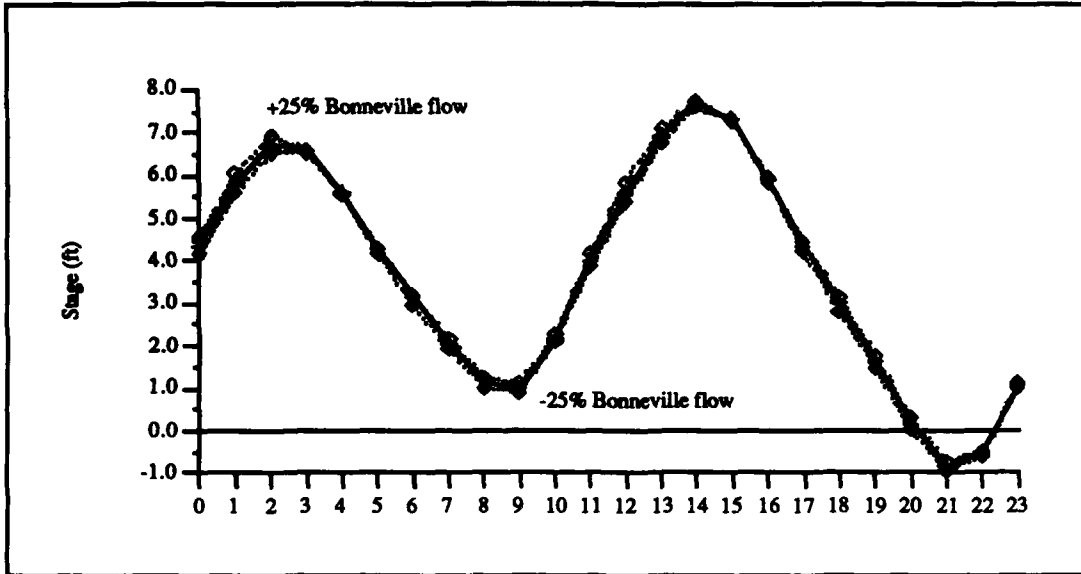


Figure 17. Upstream Boundary Variation Effects at Skamokawa, Washington

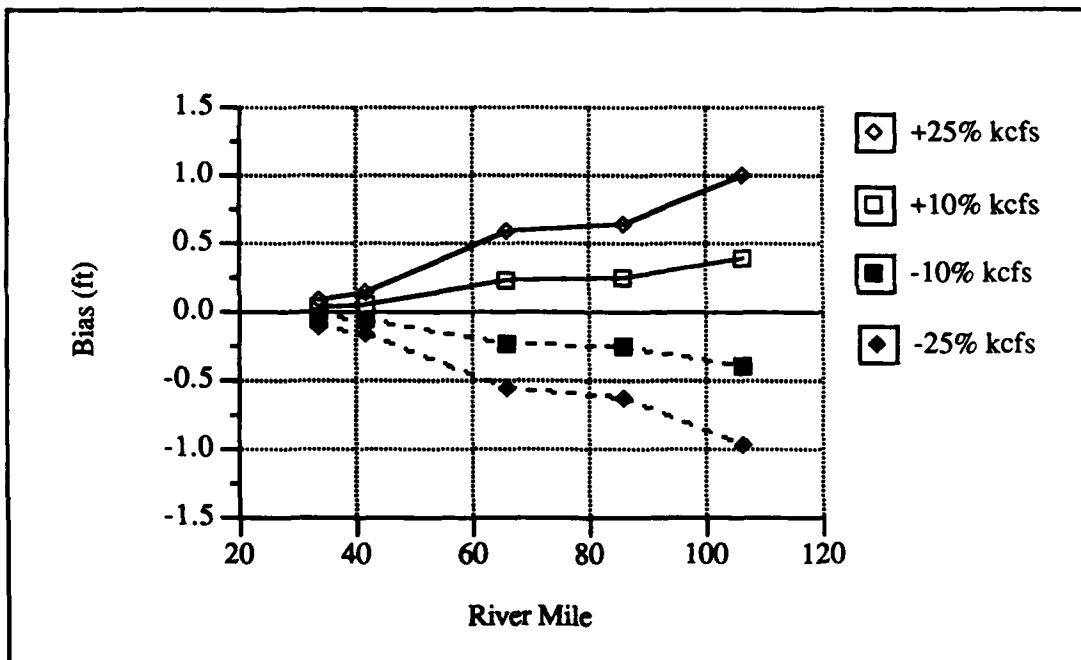
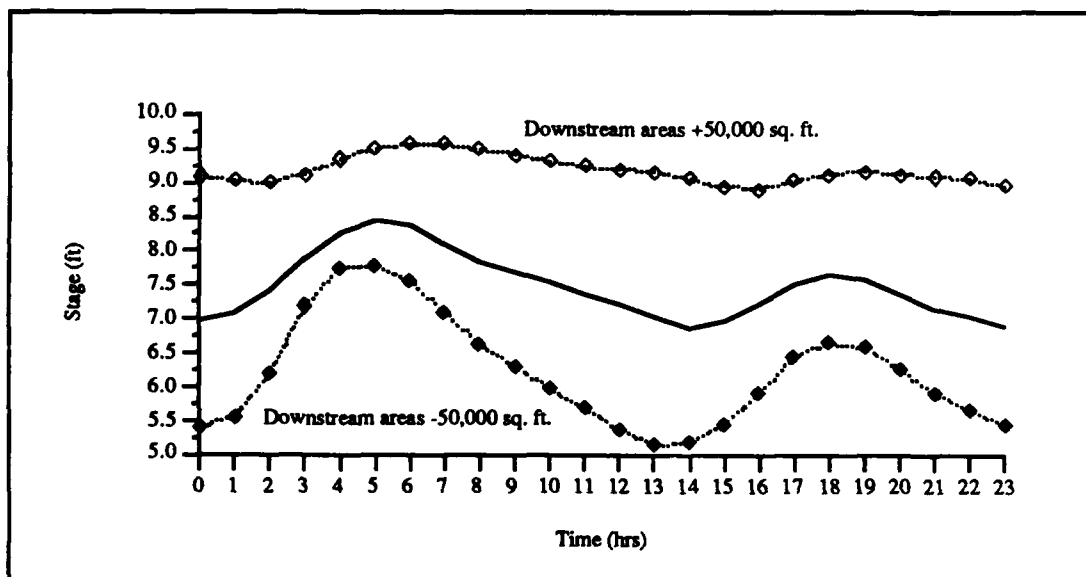


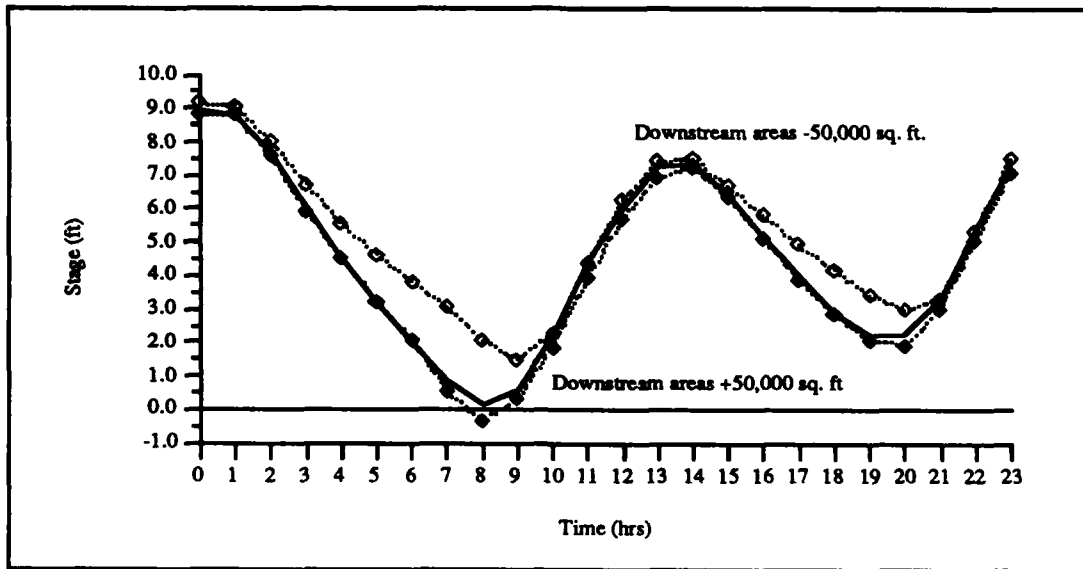
Figure 18. Summary of Upstream Boundary Variation Effects on Downstream Stations

*c. Cross-section area variations*

The five cross-section locations closest to the downstream boundary were varied by  $\pm 50,000$  ft<sup>2</sup> for two simulations and the results compared to previously computed values with unchanged areas. As can be seen in Figures 19 and 20, low waters are affected the most by the increased areas with high waters less affected. At the Skamokawa gauge site, the station is within the part of the river with changed areas and so does not show the "dam" affect seen at the Vancouver gauge site and other upstream sites. Also important is the resultant decreased amplitude for the upstream station.



**Figure 19. Downstream Area Variation Effects at Vancouver, Washington**



**Figure 20. Downstream Area Variation Effects at Skamokawa, Washington**

### C. CALIBRATION PROCEDURE

DWOPER has an automatic calibration feature. This subroutine will determine the optimum roughness coefficients which will minimize the difference between observed and computed stages. This procedure starts at the most upstream station and progresses downstream reach by reach. Dynamically routed tributaries are calibrated before the main river with the flows added to the main river as lateral flows. Upstream and downstream stage value for each reach are needed as well as upstream discharge rates. A steady state river condition is assumed by this procedure.

In the case of the Columbia River, tidal influences make assumptions of a steady state condition invalid. For any station there is no unique stage-discharge relation (as in the case of flow reversals). It is because of this complex situation that

a trial and error approach was taken. The previously described sensitivity analysis was made, in part, to act as a guide in the calibration of the model.

Simulations for three periods of distinct flow conditions were used in calibrating the model. All stations had error free, continuous observed records during the following dates:

- 21 October 1986, 1600 to 31 October 1986, 0700 (232 hours)
- 9 January 1987, 1600 to 16 January 1987, 1600 (169 hours)
- 8 May 1987, 1600 to 17 May 1987, 0400 (205 hours)

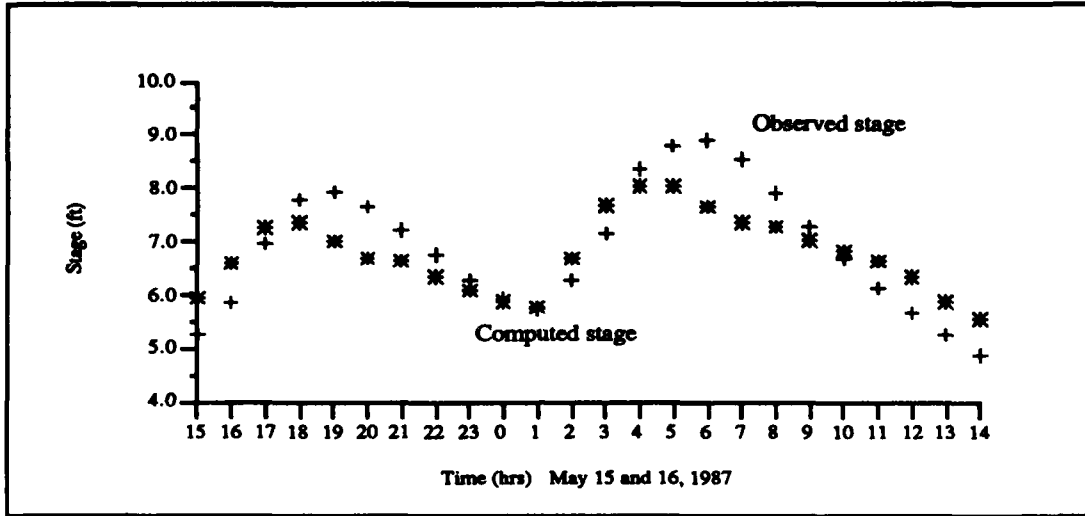
The first period is a typical low flow interval occurring in the fall with maximum tide influence along the entire course of the river. The second period is during a winter storm runoff event and is shorter in duration than the spring snowmelt runoff season, represented by the third period listed.

In each case, simulations for each of the above periods were made. Upon examination of the printout, only the Longview station showed a peculiar shaped falling tide hydrograph at different times during the May calibration period. All other stations had expected or typical tide curves in the output. Figures 21 and 22 illustrate the Longview curve before and after calibration.

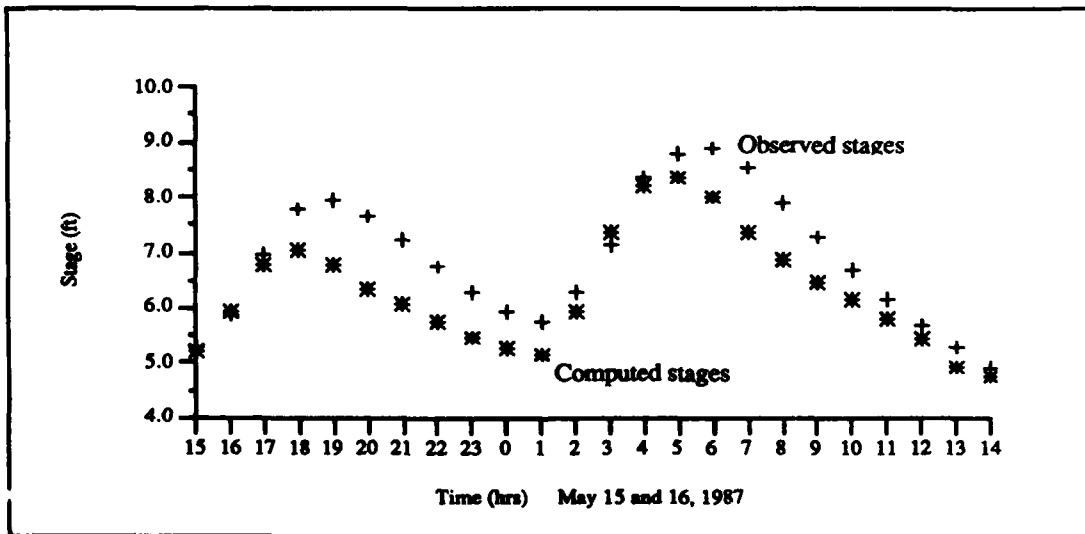
From the sensitivity analysis, most stations needing calibration were simply a matter of adjusting the appropriate Manning  $n$  or roughness coefficient. However, with the peculiar situation at Longview, trial and error adjustment of the cross section area just above and just below the station was needed. After Longview, the river system was calibrated from a downstream to upstream direction.

The adjusted cross-sectional areas reflect a general constriction of the river at this point although no data were available to verify that this is the actual situation. However, it was at this point, just upstream of Longview at the confluence of the

Cowlitz River, that debris from the 1980 eruption of Mount Saint Helens entered the Columbia River.



**Figure 21. Pre-Calibration Longview Computed and Observed Tidal Curves**



**Figure 22. Post-Calibration Longview Computed and Observed Tidal Curves**

The cross-sectional area variations showed that the only way to increase a station high water value was to constrict the river downstream of the station of interest. Varying Manning n factors would not affect the high water very much and boundary conditions were all screened to eliminate errors from those values. Whenever a station seemed to need an adjustment to the cross-sectional areas, the resulting decrease in amplitude was increased by reducing the roughness coefficients (in effect, smoothing the river bed to allow low waters to be lower). A list of the parameters obtained from the calibration exercise can be found in the appendix.

The following tables summarize the descriptive statistics found for each of the stations for each of the different calibration periods both before and after parameter adjustments.

**TABLE 5. PRE-CALIBRATION AND POST-CALIBRATION BIAS (FT) AND ROOT MEAN SQUARE (FT)**

Station		October		January		May	
		Pre-calibration	Post-calibration	Pre-calibration	Post-calibration	Pre-calibration	Post-calibration
Portland	bias	-0.780	-0.413	-0.789	-0.295	-0.046	0.438
	rms	0.887	0.537	0.838	0.420	0.225	0.504
Vancouver	bias	-0.380	-0.031	-0.472	0.113	0.140	0.585
	rms	0.483	0.188	0.564	0.318	0.330	0.655
St. Helens	bias	-0.722	-0.285	-0.355	-0.072	-0.169	0.141
	rms	0.768	0.375	0.478	0.282	0.303	0.311
Longview	bias	0.547	0.362	0.200	0.318	-0.005	0.374
	rms	0.588	0.513	0.613	0.523	0.632	0.588
Wauna	bias	0.327	0.201	0.399	0.116	0.473	0.036
	rms	0.374	0.349	0.455	0.350	0.594	0.371
Skamokawa	bias	0.109	0.146	0.151	0.134	0.221	0.143
	rms	0.180	0.205	0.296	0.278	0.398	0.291

#### D. RESIDUAL ANALYSIS

The residual (or stage error) is the difference between the computed ( $C_i$ ) and observed stages ( $O_i$ ) as denoted by  $R_i$ . By defining the residual this way, any correction to the computed stages will always be algebraically added ( $R_i + C_i = O_i$ ). The statistics calculated by the DWOPER program defines the residual to be computed minus observed stage ( $C_i - O_i = R_i$ ).

Figures 23 through 28 show a least squares fit of observed versus computed stage and residual error versus computed stage. In no case were the correlation coefficients ( $r^2$ ) greater than 0.360 for the residual error versus computed stage graphs (most values between 0.00 and 0.15). This would indicate no correlation or a non-linear relationship between computed stages and the resultant residual. If the residuals are random and normally distributed, it would be difficult to improve the DWOPER output by modelling these residuals. However, a time series of the residuals was made and spectral estimations of the residuals for all stations and all calibrations are shown in Figures 29 through 31. No spectral estimates were made for Portland or Vancouver because the observed records for these stations were only composed of daily high and low readings and not hourly stage readings. All other stations had hourly stage values. If the residuals were random in nature, a spectral analysis should show a white noise distribution over all frequencies.

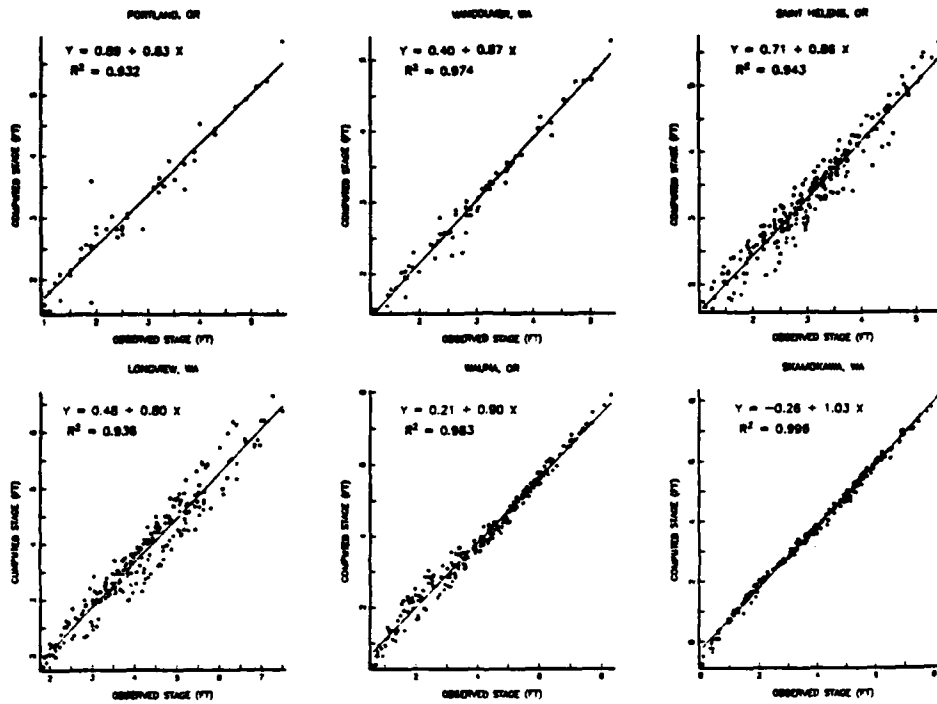


Figure 23. October Observed versus Computed Stage Scatter Plots

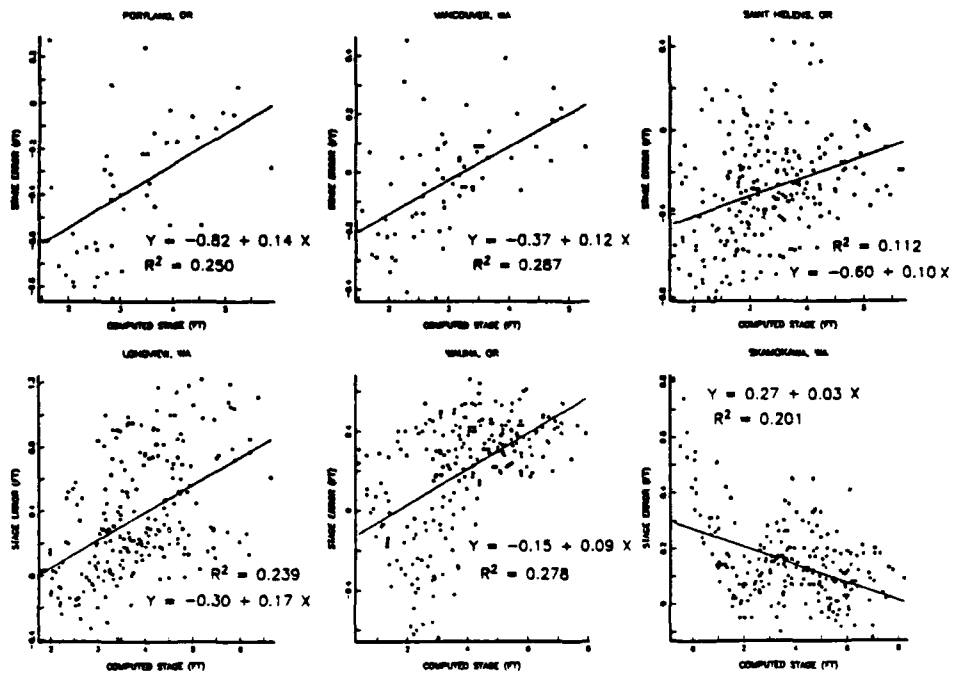


Figure 24. October Residual versus Computed Stage Scatter Plots

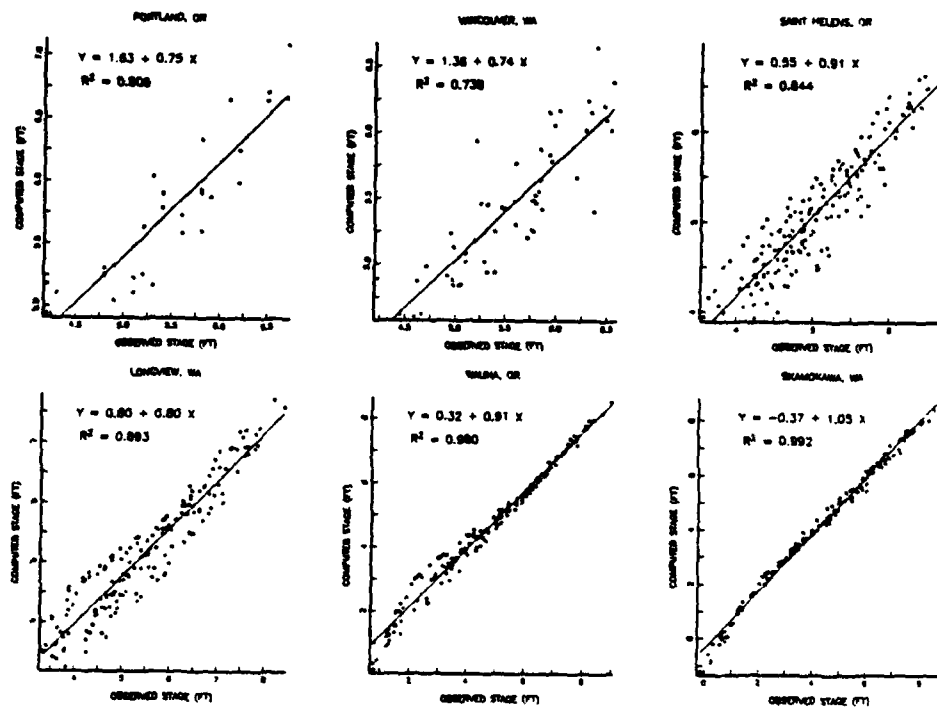


Figure 25. January Observed versus Computed Stage Scatter Plots

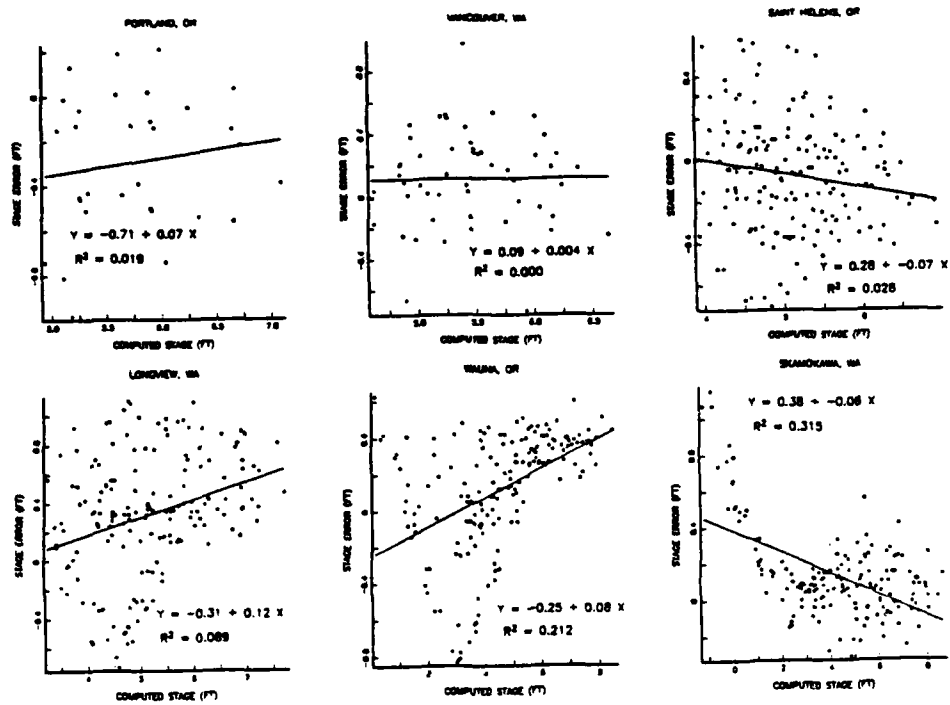


Figure 26. January Residual versus Computed Stage Scatter Plots

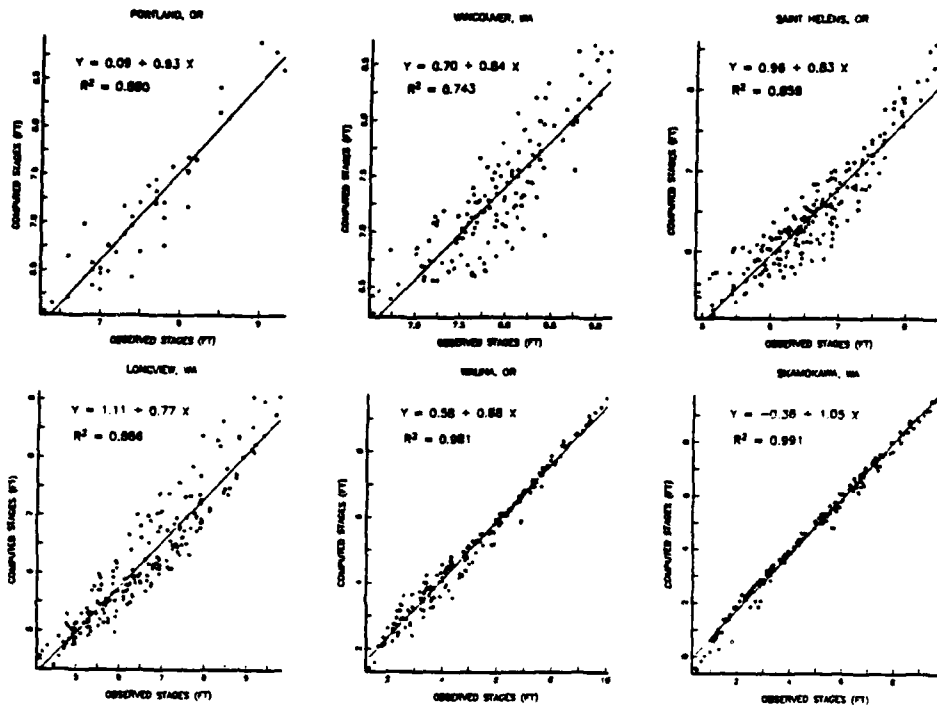


Figure 27. May Observed versus Computed Stage Scatter Plots

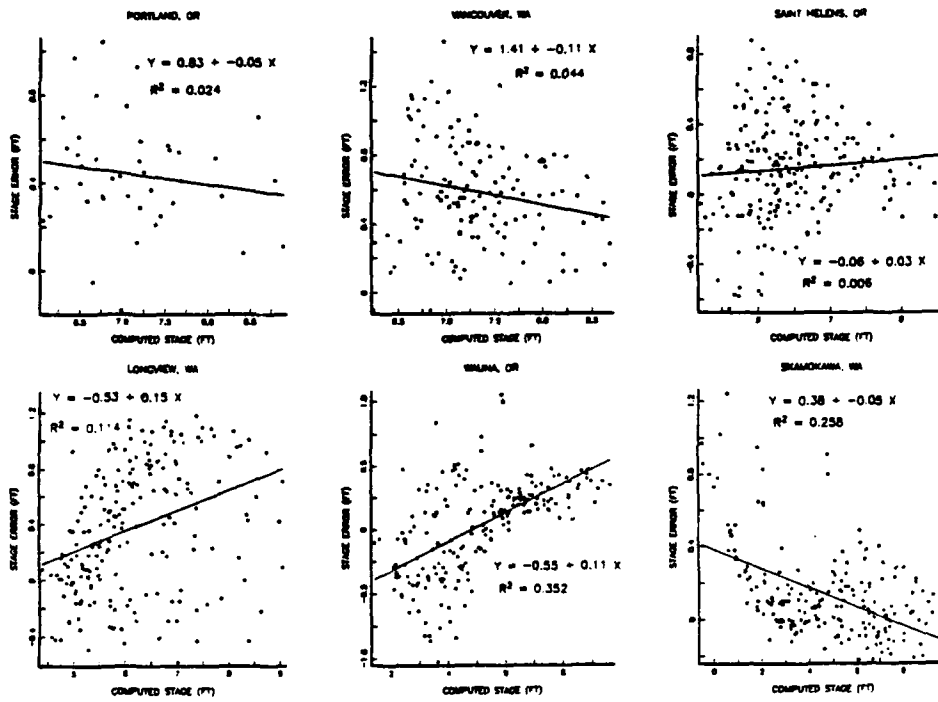


Figure 28. May Residual versus Computed Stage Scatter Plots

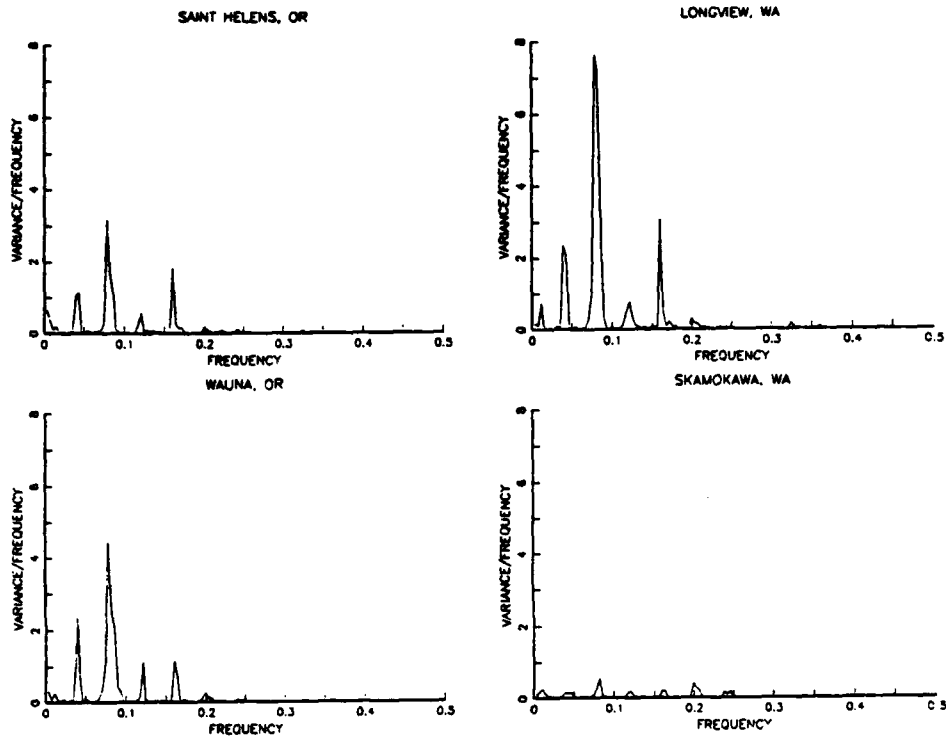


Figure 29. October Residual Spectral Estimate

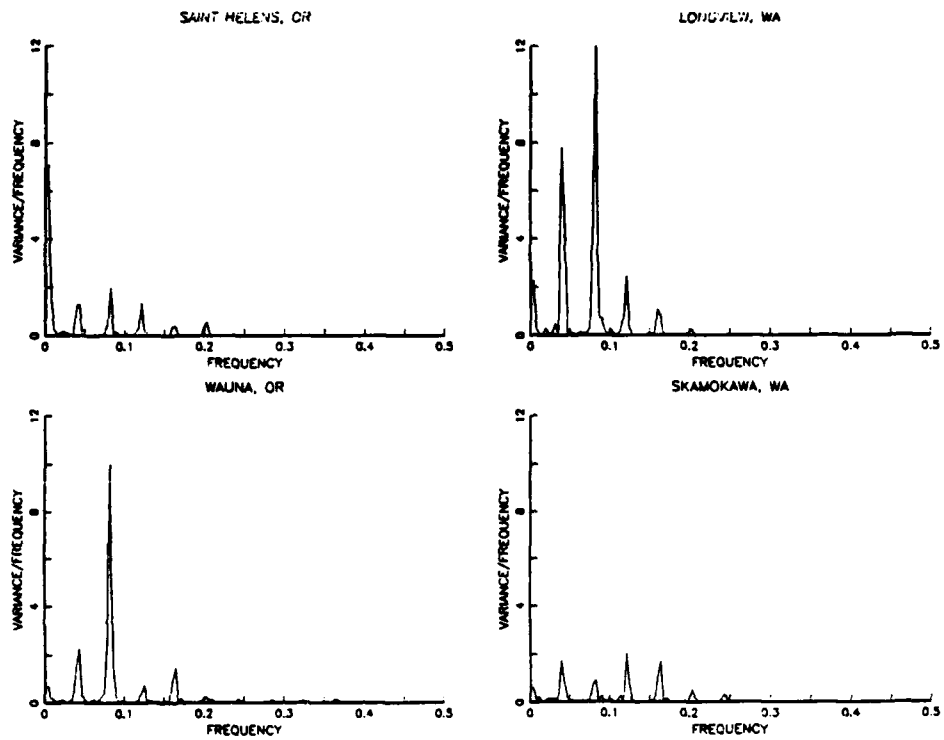
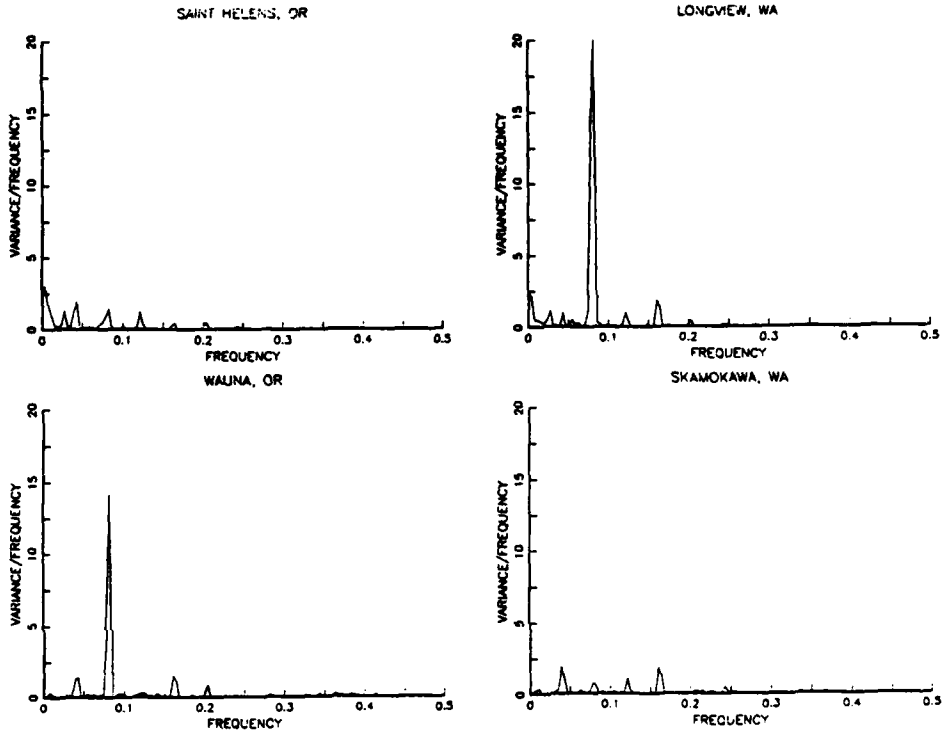


Figure 30. January Residual Spectral Estimate



**Figure 31. May Residual Spectral Estimate**

The very strong peaks at specific frequencies indicates that the residual components are strongly tied to the tidal cycle frequencies of  $0.081 \text{ hr}^{-1}$  (12.3 hrs),  $0.042 \text{ hr}^{-1}$  (24.0 hrs) and subsequent harmonics of these frequencies. In most cases, the 12.3 hr peak is dominate and has the largest part of the residual associated with

it. Because the peak values vary from station to station and between calibration periods, the residual components are distributed over specific frequencies on a seasonal basis for each site along the river. Thus, no one set of correction values can be applied to all the different flow conditions on the Columbia River.

During the low flow October period, both the 0.081 and 0.042 Hz frequencies are present at St. Helens. However, no one frequency dominates during the January and May calibration periods. The January period does have a pronounced peak near the frequency related with the length of the calibration period itself (169 days). This may be associated with the flow regulation of Bonneville dam and Willamette River dams during the winter storm event. This frequency also shows up at Longview but diminished as flow becomes less a factor the further a station is downstream. At Skamokawa, all stages for all calibrations matched extremely well. The cross correlations between computed stages and residuals in Figures 32 to 34 show a sinusoidal, not linear, correlation exists.

A one to three hour phase shift between the observed stages and the stage error is present in each of the calibration periods for all stations. The phase shift becomes evident when an observed stage time series is back-lagged with respect to the corresponding stage error time series. All the stations, except for Skamokawa, have similarly shaped cross correlation curves with respect to each other, especially during the October calibration period. In each of the calibration periods, Skamokawa is always 180° out of phase with the other stations, except for Longview during the May calibration period. All stations had larger correlations (either negative or positive) at the 12 to 13 hour and 24 to 25 hour forward and back-lag times.

It should be noted that for the October calibration, the observed and computed lower low water times for all stations matched extremely well (0 to 1 hour difference). For Skamokawa, times for computed and observed high and low waters nearly always matched no matter which calibration period was examined. It is evident that even though DWOPER handles the timing of the extreme parts of the tide cycle well, intermediate stages are not modeled as well.

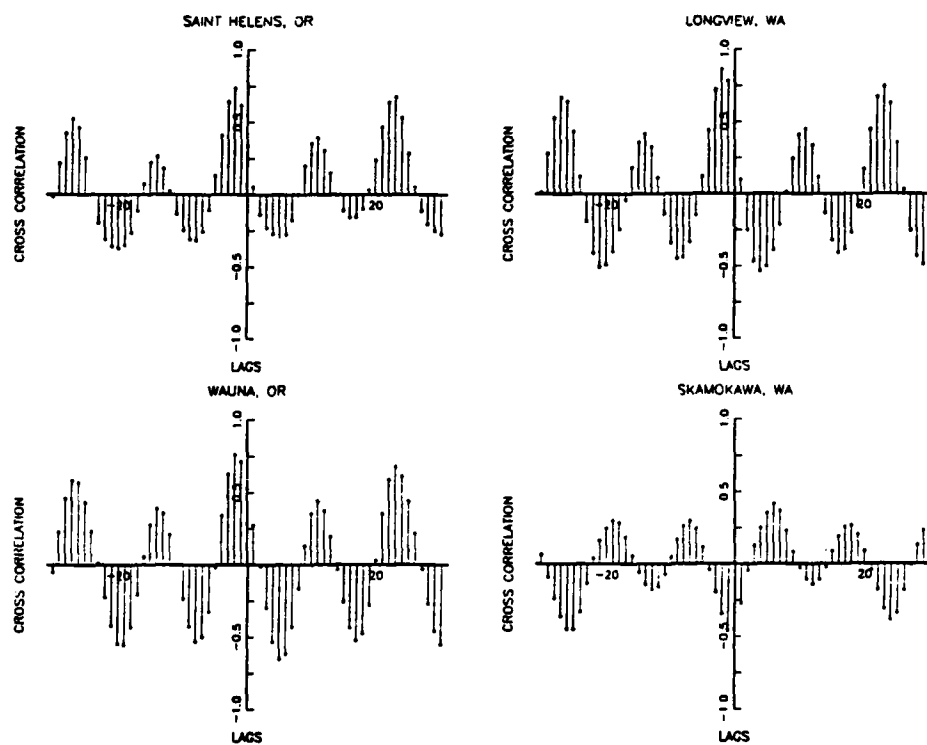


Figure 32. Cross Correlations for October

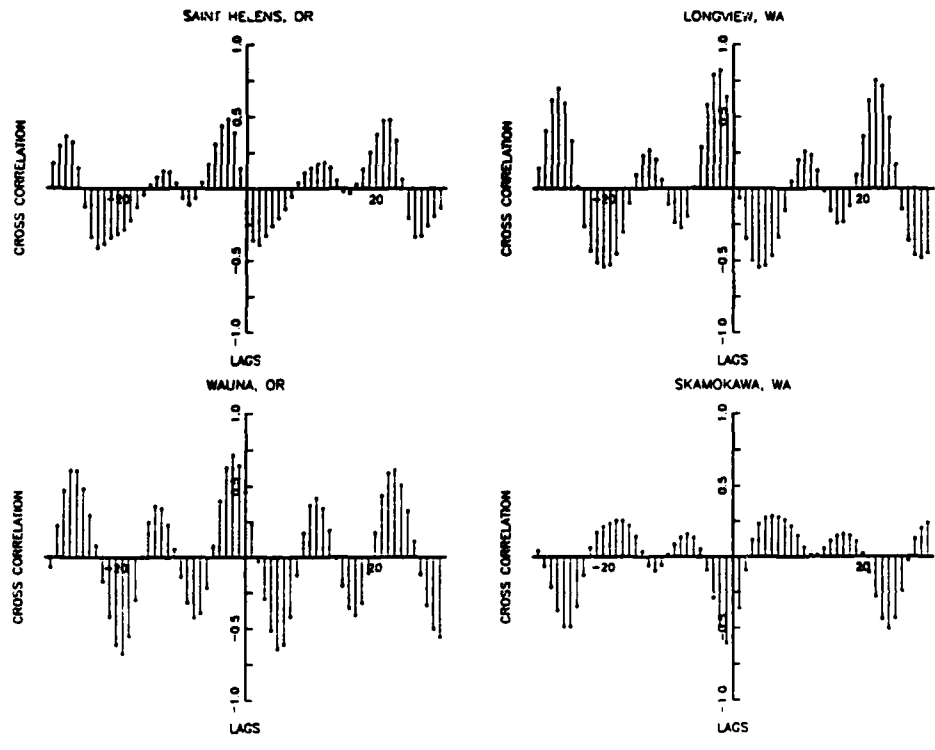


Figure 33. Cross Correlations for January

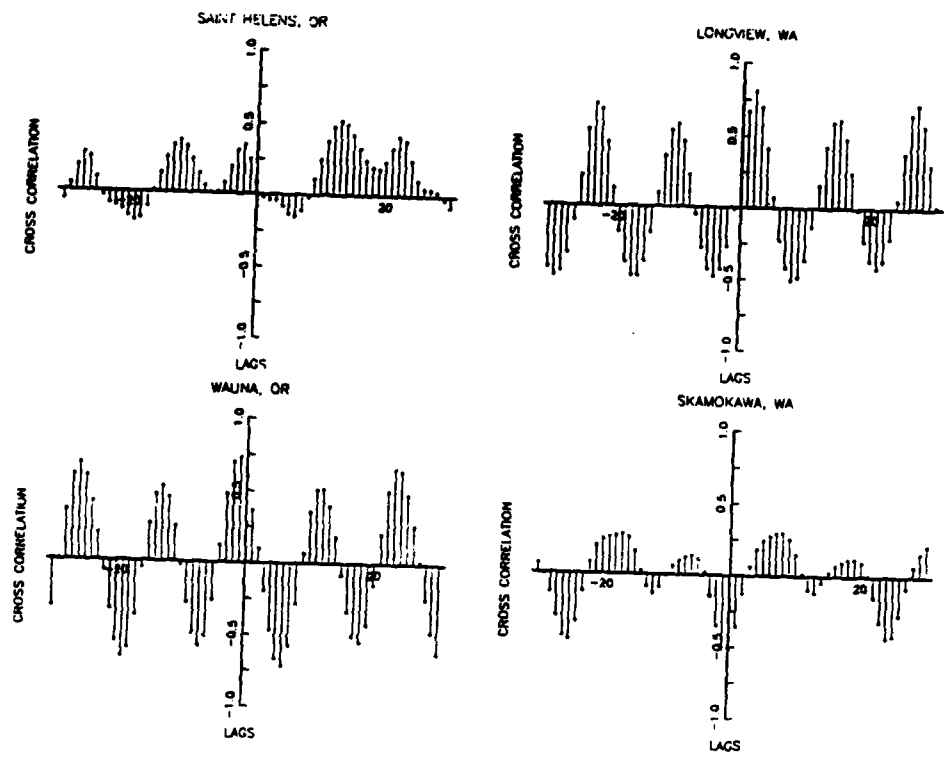


Figure 34. Cross Correlations for May

The Northwest River Forecast Center is presently simulating river stages for the three days prior to a forecast period (a three day "backup period") as well as the forecast period itself. An analysis of how well the computed stages are reproducing actual stages is possible by having a backup period. The backup period data also

allows for a method of adjusting the computed stages based on the most recent river conditions. Adjustment factors are the average differences between the observed and computed stages for four stage ranges. The adjustment factors are trended into the future and are used as corrections to the calculated river stages. Each stations is corrected separately with the forecasted stages placed in the CROHMS regional database for outside users (Orwig, 1986).

It has been shown that the residuals occur at the same frequency as the tide cycle and implies that as the tide changes, so will the residuals. Figure 35 illustrates how the residuals vary with the computed stage over a single tide cycle. The Longview looped error curve was developed from the October calibration period. The first part of the curve (points 1 to 2) is from the higher high to lower low tide, the second part (2 to 3) is from lower low to lower high, the third part (3 to 4) is from lower high to higher low and the fourth part (4 to 5) is from higher low to the higher high stage of the next tide cycle.

For any one stage value, the adjustment factor to be added to the computed stage will depend on what part of the tide cycle that stage falls. The current procedure at NWRFC does not take this tide cycle variation into account but such a procedure may prove useful at higher river flow periods where tidal influences are weakened or washed out altogether.

The looped error curves vary in shape from station to station but are similar at a station for comparable tide cycles. As the tide cycle gradually changes (when the lower high becomes the higher high for example) the error curve changes shape as well. Any adjustment scheme using looped error curves will still need some sort of a backup period analysis to allow error curves to be based on the most recent river conditions. An historic record of curves for differing tidal cycles at a station can be

be obtained as these curves are generated. When forecasted river flow conditions change markedly from the backup period or if observed data is not available, any backup period curves become useless and the historic record of curves could be used to find appropriate adjustment factors.

In the case of Portland and Vancouver, the method described above cannot be applied because only high and low tide stages were recorded. To have an accurate and functional adjustment procedure, hourly stage data must be available. Any plan to decrease the amount of data gathered at any of the lower Columbia River sites would lessen a hydrologist's ability to produce an accurate, knowledgeable forecast for the river.

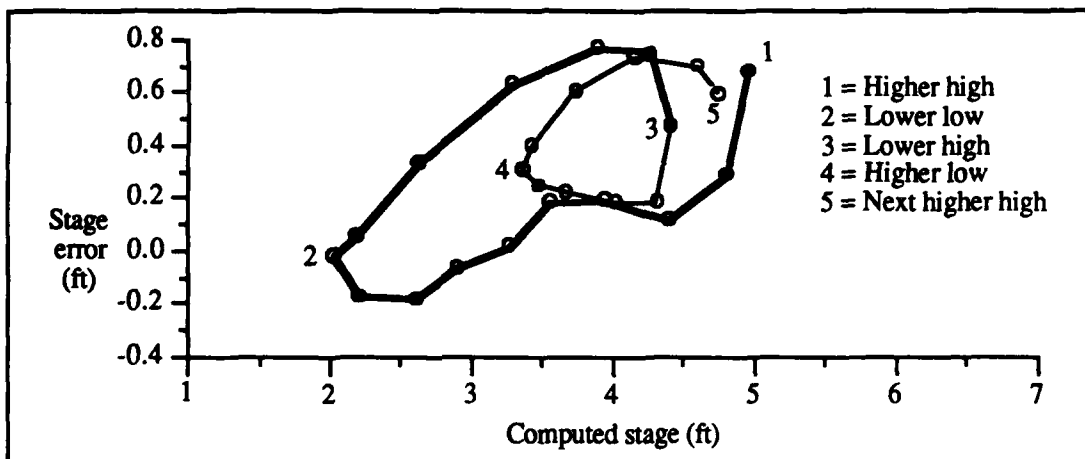


Figure 35. Longview Looped Error Curve

## IV. MODEL APPLICATIONS

The main use of the DWOPER program has been to forecast stages on the Lower Columbia River to support the shipping interests in the area. As outlined earlier in this paper, the Port of Portland and other port authorities on the lower Columbia River have used these forecasts to increase the amount of commerce on the river. However, other possible uses of the DWOPER model are discussed below.

### A. FLOOD ROUTING

A primary reason the National Weather Service produced DWOPER was for the routing of streamflow on major river systems. As such, this model has been used by the U.S. Army Corps of Engineers to simulate downstream river stages with various Bonneville dam regulation schedules. By doing so, downstream effects can be approximated and potential flood damage avoided. On the other hand, estimated dam regulation for maintaining minimum river flow conditions during the dry summer and autumn periods has been simulated by using the DWOPER program.

Also important is the link the Northwest River Forecast Center (NWRFC) has to the Corps of Engineers Columbia River Operational Hydromet Management System (CROHMS) regional database. By using the Corps' Synthetic Streamflow and Reservoir Regulation (SSARR) watershed and river routing model as input to the DWOPER program, the latest and best estimates of lateral inflows and ocean tides can be used to produce simulations on an operational basis (Orwig et al., 1986).

## B. CONFIDENCE INTERVALS

The NWRFC has successfully used the DWOPER model in producing three day Lower Columbia stage forecasts since May, 1986. This office has now been asked by the local merchants exchange to extend the river stage forecasts from three days to six days (personal conversation with Orwig). Questions of input data quality control and output adjustments must be addressed when forecasting stages so far into the future. Errors are a function of model exactness and input data accuracy.

The sensitivity analysis of DWOPER described earlier in this study can be used by forecasters to quantify how boundary condition variations will affect model output. By plotting computed stages made under varying Bonneville discharges, a type of "confidence interval" for any station can be produced. Such an approach would help estimate the possible river stages cause by unexpected changes in Bonneville dam releases.

In Figure 36, the effects of changing Bonneville dam outflows by  $\pm 25\%$  at Portland are shown. The low river flow during this time had a large tidal influence. The lower estimated flow (-25%) produced lower stages overall with the low tide having a slightly larger offset than the high water tide from the comparison stages. The higher estimated flow (+25%) produced higher overall stages with low water depths being modified the most. The differences over this tide cycle are depicted in Figure 37. This graph indicates that as Bonneville dam discharges are overestimated, forecasts will have a greater error at the low waters. When dam discharges are underestimated, an offset of approximately 0.6 feet is produced for all stages.

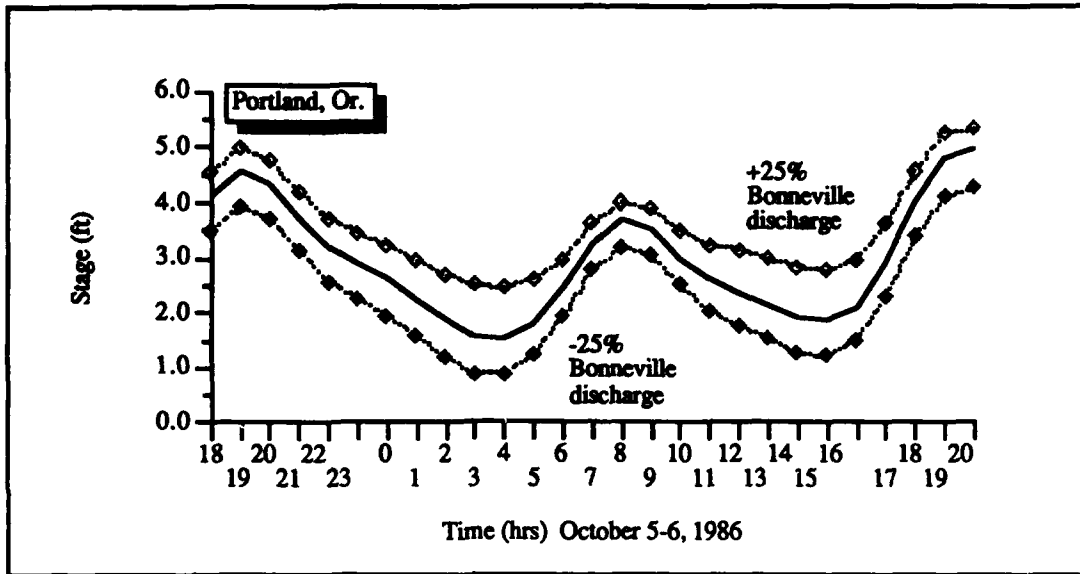


Figure 36. Bonneville Discharge Variation Effects at Portland, Oregon (October 5-6, 1986)

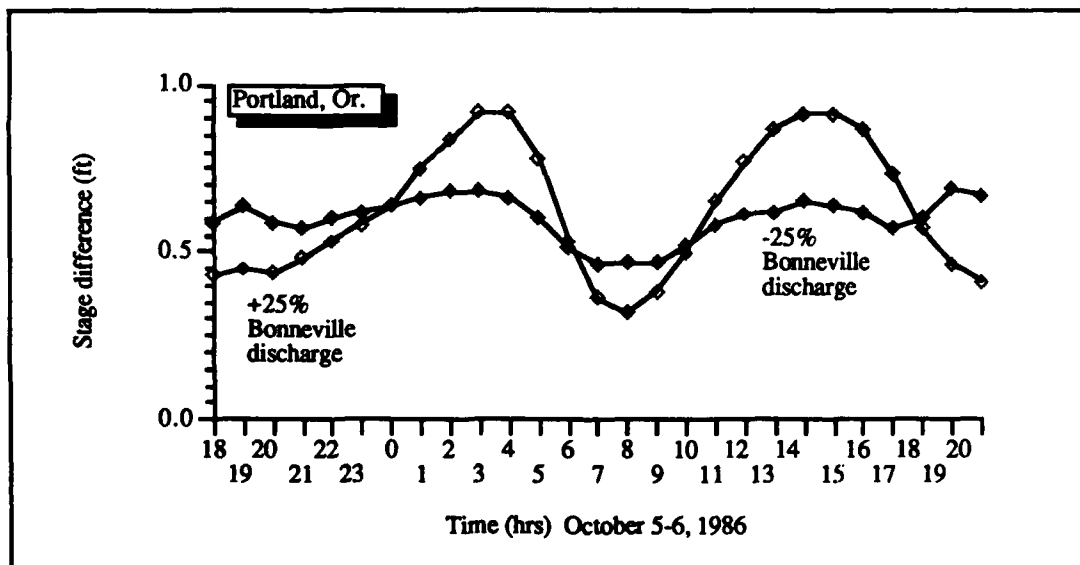
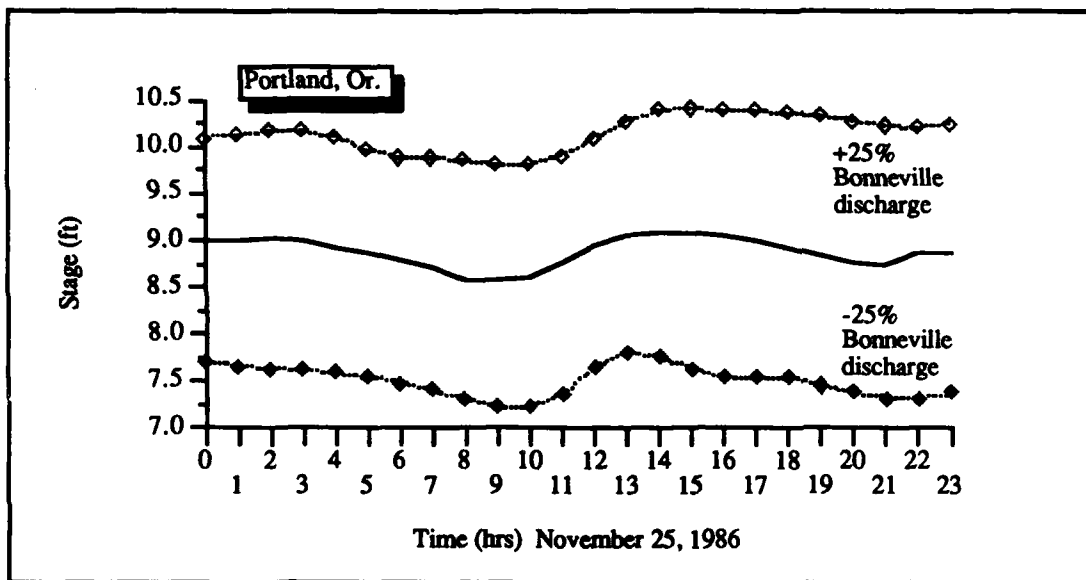
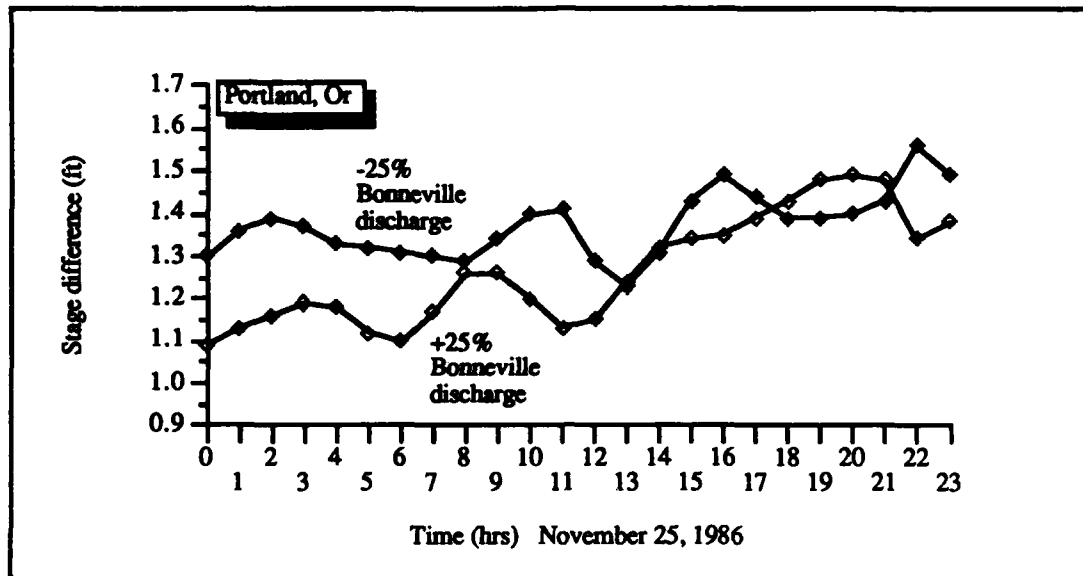


Figure 37. Stages Differences From Bonneville Discharge Variation at Portland, Oregon (October 5-6, 1986)

In Figure 38, the effects of changing Bonneville dam outflows by  $\pm 25\%$  at Portland during a high river flow period is shown. During this time, there was less tidal influence than in the October period. The lower estimated flow (-25%) again produced lower stages overall. The higher estimated flow (+25%) produced higher overall stages with the second low water almost eliminated (Figure 39). The graph indicates that as Bonneville dam discharges are overestimated, forecasted stages become very uniform with tidal influences almost entirely removed. The result will be greater errors occurring at the low waters. When dam discharges are underestimated, a false tidal influence could be introduced where none exists. In the high river flow case, departures from the comparison stages are much less tied to the tide cycle and are more complex.



**Figure 38. Bonneville Discharge Variation Effects at Portland, Oregon (November 25, 1986)**



**Figure 39. Stages Differences From Bonneville Discharge Variation at Portland, Oregon (November 25, 1986)**

The concept of over and underestimated boundary conditions can be applied to the Tongue Point tide gauge as well. When a forecaster estimates a storm surge influence on the ocean tides, the forecaster will know how far upstream the surge will be felt. Figures 39 and 41 show how Skamokawa stages change by adjusting the Astoria tide values by  $\pm 2$  feet. During this period, tidal influence on the Skamokawa stages is very evident. In both periods, the higher estimated tide (+ 2 ft) produced higher overall stages while the lower estimates produced lower overall stages. Differences over the tide cycles are depicted in Figures 40 and 42. Figure 41 indicates that the departures from the comparison stages are not just a constant offset and that the departures are associated with the tide cycle.

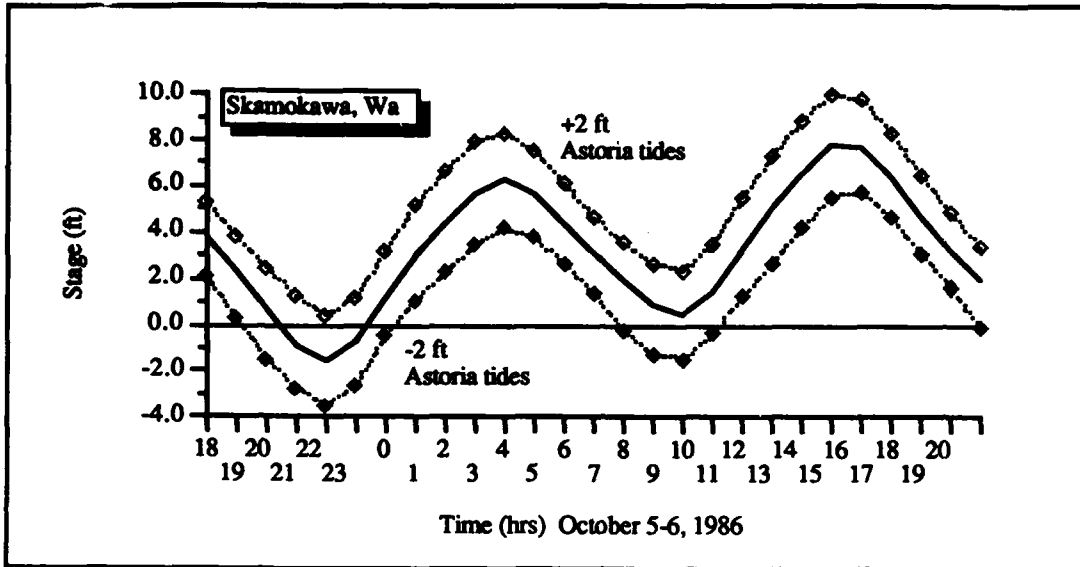


Figure 40. Astoria Tide Variation Effects at Skamokawa, Washington (October 5-6, 1986)

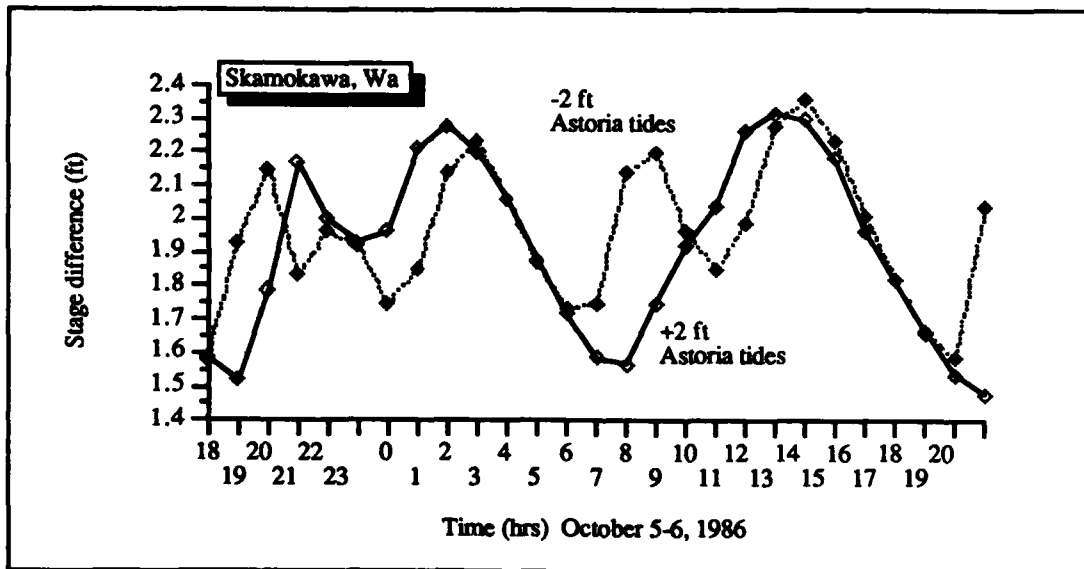
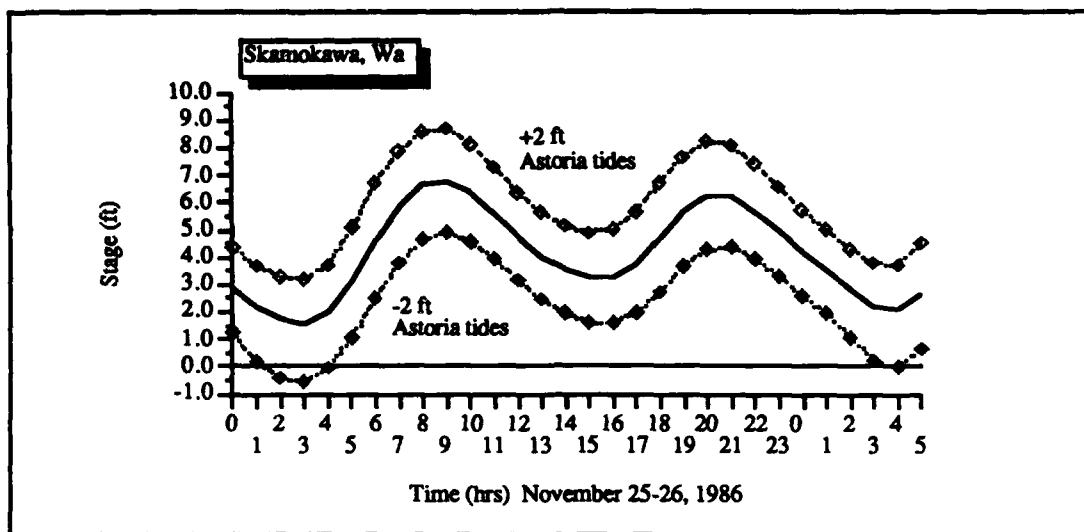
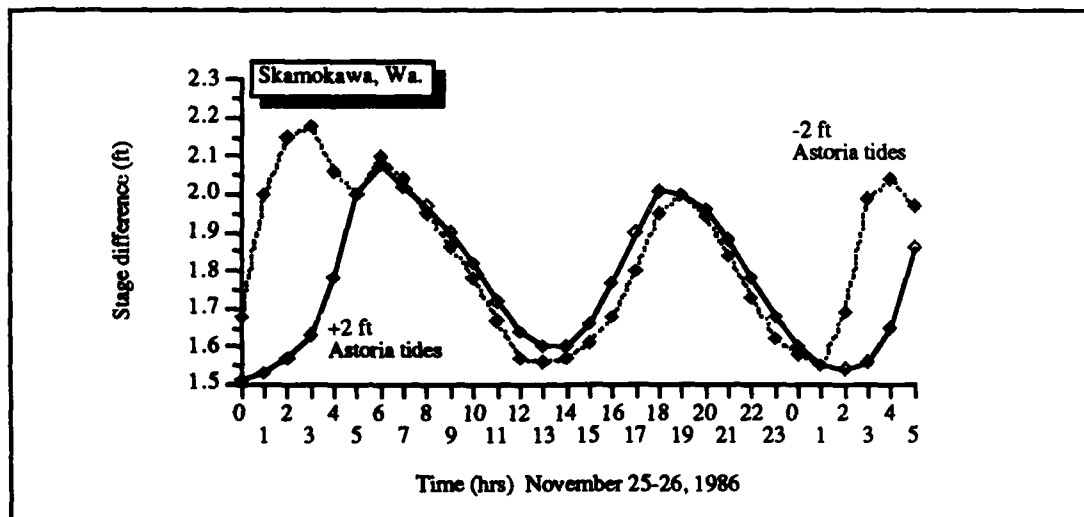


Figure 41. Stages Differences From Astoria Tide Variation at Skamokawa, Washington (October 5-6, 1986)



**Figure 42. Astoria Tide Variation Effects at Skamokawa, Washington (November 25-26, 1986)**



**Figure 43. Stages Differences From Astoria Tide Variation at Skamokawa, Washington (November 25-26, 1986)**

### **C. OTHER MAJOR USES**

Normally the DWOPER program provides output such as graphs displaying computed and observed values at gauging stations, a list of numbers plotted and the statistics of bias and root mean square. Additionally, the following three special tabular printouts can be specified.

- Computed stages, discharges and velocities, and observed stages for all stations on one river can be displayed for one time step.
- Computed and observed stages at a single station can be displayed for a period up to 25 hours.
- Computed discharges and velocities for a single station can be displayed for a period up to 25 hours.

The ability to compute average cross sectional velocities can be helpful in predicting average river currents. Applications could include modelling oil spill spreading, special boat operations, fish migration or location of objects lost in the river.

Tidal range variations influenced by dredging can be studied by varying cross sectional areas. By decreasing the cross sectional areas, debris or obstructions influences in the river can be studied.

The DWOPER model can be used as an error check since the program does not use observed stages in computing stages for a station. With the calibration of the model, residuals were kept to within 1.5 feet of the observed stages. By using the model output as a estimator for observed stages, stage data could be screened before entry into the Columbia River Operational Hydromet Management System (CROHMS) regional database. This database is a key link between several federal agencies and other public and private users in the Pacific Northwest (Pasteris, 1984).

## V. SUMMARY AND CONCLUSIONS

The DWOPER model uses a weighted four-point implicit finite difference method to solve the Saint Venant flow equations of conservation of mass and momentum. Both upstream and downstream boundary conditions of river flow or stage are specified to obtain solutions to  $h$  and  $Q$  along intermediate points on the river. The Dynamic Wave model is a mathematical model well suited for a variety of rivers and flow conditions. As applied to the Columbia River, this model can provide essential information on hourly river stages critical for navigational safety and ship commerce on this economically important river.

A sensitivity analysis was performed by varying the roughness, boundary conditions, and cross-sectional areas. The computed values from 1 October 1986, 0000 hours to 1 January 1987, 0000 hours were used as comparisons. These dates represent 2209 simulation hours. The calibration procedure examined the distinct flow conditions of a low river flow period, a winter storm event, and a spring snowmelt season.

In testing the Manning  $n$  roughness coefficients, each set of Manning  $n$  values were varied by  $\pm 0.01$  for each of the river reaches. In the case of decreased  $n$  values, the computed low water stages were much lower than the comparison stages; high waters changed very little. Increasing the roughness coefficients caused low water stages to be much higher with high water stages slightly increased. The conclusion from these simulations is that upstream stations are influenced by downstream Manning  $n$  changes but downstream stations are little affected by upstream changes. Any optimization procedure that is a trial and error adjustment of the roughness coefficients must begin at the most downstream station

and progress up the river system, with the dynamic tributaries the last to be calibrated.

Downstream boundary changes of  $\pm 0.5$  feet and  $\pm 2.0$  feet were made to the Astoria tide stages. Model simulations showed that 70% of the tide difference appears at Vancouver and Portland, 80% at St. Helens, 85% at Longview, 93% at Wauna and 95% at Skamokawa. These differences occurred as a constant offset and did not appear to be associated with the tide cycle.

The effects of varying the upstream boundary condition (Bonneville Dam discharges) by  $\pm 10\%$  and  $\pm 25\%$  were markedly different from the downstream boundary changes. Upstream, where the tide influence is weakest, the tidal cycle is more likely to be "washed out" by the higher flows of the Columbia. Also these changes fluctuated with the tide cycle. Downstream stations did not show such differences because of the larger cross section areas of the Columbia River nearer the mouth and the proximity to the downstream boundary condition.

To observe the effects of varying cross-sectional areas, the five most downstream cross-section locations were changed by  $\pm 50,000$  ft<sup>2</sup>. Low waters were changed the most by the increased areas with high waters less influenced. At the Skamokawa gauge site, the station is within the part of the river with altered areas and so did not show the "dam" affect seen at the Vancouver gauge site and other upstream sites. Because the tide wave could not easily propagate upstream with the restricted cross section areas, a decreased tide amplitude for upstream stations occurred. Fresh water runoff encountered the decreased areas as a "dam" and backed up causing an overall increase in upstream stages.

Most stations needing calibration involved a simple matter of adjusting the appropriate Manning n or roughness coefficient. The adjusted cross-sectional areas

areas reflect a general constriction of the river at this point, although no data were available to verify that this is the actual situation. However, it was at this point, just upstream of Longview at the confluence of the Cowlitz River, that debris from the 1980 eruption of Mount Saint Helens entered the Columbia River. As part of the Longview calibration two extra cross sections, one just above and another just below the Cowlitz River confluence, were added. After the Longview adjustments, the river system was calibrated from a downstream to upstream direction. Comparisons of observed data to post-calibration simulations showed that for the October and January periods, five of six stations had consistently lower mean stage errors while in the May period, three of six stations had consistently lower mean stage errors.

Regression analyses of the computed residual values for each of the stations gave correlation coefficients ( $r^2$ ) less than 0.360. However, cross correlations between residual and computed stages showed that the two series were highly sinusoidally correlated for all stations. A spectral estimation of the residuals exhibited very strong peaks at frequencies of  $0.081 \text{ hr}^{-1}$  (12.3 hrs),  $0.042 \text{ hr}^{-1}$  (24.0 hrs) and subsequent harmonics of these frequencies. The residual components are strongly associated with the tidal cycle.

The spectral values of the residuals changed between calibration periods and indicates that river flow condition influences the residual distribution. A one to three hour phase shift between the observed stages and the stage error is present in each of the calibration periods for all stations. All stations had larger correlations (either negative or positive) at the 12 to 13 hour and 24 to 25 hour forward and back-lag times from the cross correlation peak.

The DWOPER program can be used in the areas of forecasting stages, estimating average velocities, effects of dredging or constrictions due to added debris. As this type of information is used by the public, greater demands for longer, more accurate forecasts will be made. The NWRFC project to extend forecasts to six days is such an example.

The study presented here can be used for a basis in understanding how the DWOPER program reacts to changes in boundary conditions and how to approach predicting stage errors. Necessary adjustment of model results must be made to provide even higher quality, useful data. Ideas like confidence intervals for stages based on estimations of dam discharge variability can be addressed by simulations made beforehand.

Also demonstrated by this study is the need for dependable, high quality stage measurements. Any type of short term corrections to model output will need recent accurate hourly records for all stations of interest. Accurate hourly records will allow a correction algorithm to be used based on the dominant frequencies. As these frequencies change with flow conditions, so must the analysis of the residuals. This study looked briefly at three period of about 200 hours (606 hours out of a possible 8760 hours for a year) and can provide only some historical reference data. Further research is needed to completely understand how residuals change over time and over transitional flow conditions. The 1987 Water Year did provide an excellent chance to study DWOPER over a period when low stages are critical.

DWOPER handles the timing of the extreme parts of the tide cycle well but intermediate stages are not being modeled as well. Further studies of these residuals is needed to develop adjustment procedures to improve DWOPER model forecasts.

# APPENDIX

## DWOOPER PARAMETER LISTING

The following DWOOPER program output is a listing of the carryover file parameters. These values describe such things as bed roughness coefficients and cross section area locations. For a detailed explanation of each of the variables, the reader should refer to the model documentation (Fread, 1978).

DWOOPER PROGRAM VERSION--4/11/79

PROGRAM DEVELOPED BY D. L. FREAD  
NWS HYDROLOGIC RESEARCH LABORATORY, 8060 13 TH ST., SILVER SPRING

K1    K2    K3    K4    K5    K6    K7    K8    K9    K10    K11    K12    K13    K14    K15    K16  
2    24 9000    6    9    1    5    16    16    5    5    5    14    8    48    6

\*\*\* THE SPACE REQUIRED FOR THESE DIMENSIONS IS    1428188    THE SPACE AVAILABLE IN ARRAY A IS

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      COLI
      COLI
      CARRY-OVER FILE FOR RIVER SYSTEM COLC.
      9    MSTART    0    NSAVE    TECST
         25 1986    0    0    9000.0
      DNLABL    UPLABL(J), J = 1, 2
      ASTO    BONG GLAO
      BLABL(K,1)    FHT(K,1)    FOR RIVER 1
      VANB    16.    K = 1
      SBNQ    17.    K = 1
      LCFW    12.    K = 4
      WADO    14.    K = 4
      SRAW    14.    K = 5
      BLABL(K,2)    FHT(K,2)    FOR RIVER 2
      PRTD    18.    K = 1
      OLABL(K,J)
      RIVER 1    VANB    WDLW    CASW
      EPSY    EPSQ    EPSQJ    THETA    F1    XFACT    DHE    CFNAME
      0.0100    1000.0000    1000.0000    0.7000    0.5500    5280.0000    1.0000    COLC
      JN    NU    NCT    ICD    NYCD    ITMAX    NCM   
      2    128    1    1    0    10    15
      NCS    NCSS    NP    KTERM    KPL    KPL2    JNK    NPEND
      8    3    0    0    1    1    0    0
      NQL(J), J = 1, JN
      0
      NWJ(J), J = 1, JN
      0
      NUMLAD(J), J = 1, JN
      0
      NCSS1(J), J = 1, JN
      0
      NB(1)
      24
      NB(J)    NJUN(J)    ATE(J), J = 2, JN
      5    9    90.00
  
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5. Prof. C. S. Wu, Code 68 Department of Oceanography Naval Postgraduate School Monterey, California 93943-5000	1
6. Charles E. Orwig Northwest River Forecast Center National Weather Service, Rm 121 220 N.W. 8th Ave. Portland, Oregon 97209	1
7. Dr. Danny L. Fread National Weather Service Hydrologic Research Laboratory 8060 13th St. Silver Spring, Maryland 20910	1

8. Dr. Malcolm J. Zwolinski 1  
School of Renewable Natural Resources  
University Of Arizona  
Tucson, Arizona 85721
9. Rear Admiral F. D. Moran, NOAA 1  
Director, NOAA Corps  
NOAA, Code NC  
Rockville, Maryland 20852
10. Captain Arthur N. Fior, NOAA 1  
Chief, Program Planning, Liaison, and Training  
NOAA, code NC2  
Rockville, Maryland 20852
11. Commander Kurt J. Schneble, Code 68 1  
NOAA Liaison Officer  
P.O. Box 8688  
Monterey, California 93943-0688
12. Lieutenant Richard B. Koehler 2  
NOAA Ship Whiting  
229 York Street  
Norfolk, Virginia 23510
13. Mr. Sky Miller 1  
Snohomish County Public Works  
5th Floor, Administration Building  
Everett, Washington 98201
14. Oceanographer of the Navy 1  
200 Stovall Street  
Alexandria, Virginia 22332
15. Commander 1  
Naval Oceanography Command  
NSTL Station  
Bay St. Louis, Mississippi 39522
16. Defense Mapping Agency Hydrographic/Topographic Center 1  
6500 Brooks Lane  
Washington, DC 20315