Washington Public Power Supply System
P.O. Box 968 3000 George Washington Way Richland, Washington 99352 (509) 372-5000

Docket No. 50-508

January 18, 1983
G03-83-52

Director of Nuclear Reactor Regulation
Attention: Mr. George W. Knighton, Chief
Licensing Branch No. 3
Division of Licensing
U. S. Nuclear Regulatory Commission
Washington, D. C. 20555

Subject: SUPPLY SYSTEM NUCLEAR PROJECT NO.3
ENVIRONMENTAL REPORT - OPERATING STAGE
RESPONSE TO NRC REQUEST FOR INFORMATION

References: 1) Letter, JD Kerrigan (NRC) to RL Ferguson (Supply System), dated October 13, 1982
2) Letter, GD Bouchey (Supply System) to GW Knighton (NRC), dated November 17, 1982

A Set of questions addressing hydrologic aspects and resulting from the NRC review of the WNP-3 ER-OL was transmitted under Reference 1. In Reference 2 we provided an anticipated submittal date. Please find the Supply System's responses attached.

If you require additional information or clarification, please do not hesitate to contact KW Cook, Licensing Project Manager at WNP-3 (206/482-4428 - Ext: 5436).

Very truly yours,

C002

GD Bouchey

Apex. Dist

G. D. Bouchey, Manager
Nuclear Safety & Regulatory Programs

JPC/sm

Attachment

cc: WG Albert NRC
D Smithpeter BPA 762
A Vietti NRC

ATTACHMENT

RESPONSES TO NRC QUESTIONS OF OCTOBER 13, 1982 (Re: WNP-3 ER-OL)

240.08 Q. On a suitable scale map provide delineations of the one percent chance floodplains for watercourses altered or affected by construction and operation of the plant or appurtenant structures. Identify and describe the location of all facilities within the one percent chance floodplains. Include a floodplain delineation for conditions prior to initiation of plant construction and one for conditions expected when the plant is in operation.

A. In February 1978 the Corps of Engineers (COE), Seattle District, published the report, "Special Study, Suggested Hydraulic Floodway, Chehalis River, Aberdeen to Satsop and Vicinity, Grays Harbor County, Washington." This report (attached) contained estimates of the one-percent chance (100-year) flood elevation and floodplain in the vicinity of WNP-3. The COE report also included a 1"=1000' floodplain map without topography (Sheet 4, attached) and a profile of the 100-year flood elevation at a scale of 1"=5' vertical and 1"=0.2 miles horizontal (Sheet 6, attached). The conclusions of this study have been accepted as valid for pre-construction conditions, and copies of the report, Sheet 4, and Sheet 6 have been included in this submittal.

The only one-percent chance floodpain potentially affected by project construction is that portion of the Chehalis River adjacent the Ranney well intake structures (Subsection 3.4.5) and the associated bank protection. The calculated pre- and post-construction 100-year flood elevations (ft MSL) in the vicinity of the project are as follows:

River Mile	Pre-Construction	Post-Construction
17.28	19.0	19.1
17.40	19.2	19.3
17.67	19.2	19.4
18.34	19.9	20.0

As shown, the maximum increase in the 100-year flood elevation is only 0.2 feet, or about 2 inches. Topographic maps of adequate detail to show the lateral extent of such a small increase are not available.

The small increase in the 100-year flood elevation would not measurably alter or otherwise affect the Chehalis River 100-year floodplain. In fact, this increase is less than that commonly allowed by the COE for construction outside a floodway boundary. The COE's suggested hydraulic floodway would permit 0.5 to 0.6 feet of increase in the vicinity of the project.

Three features of the plant have been constructed within the Chehalis River floodplain. As noted above, the makeup water intake wells were constructed within the 100-year floodplain. Stabilization of the river bank at about RM 17.5 will raise the overbank area surrounding the wells to an elevation exceeding the 100-year flood level. Another facility within the floodplain is the barge unloading slip located at about RM 15.5. This facility was described in Subsection 4.1.2.3.5 of the ER-CP and was used upon receipt of the NSSS components in July 1981 and August 1982. The blowdown discharge diffuser is embedded in the Chehalis River at about RM 20.5 (Subsection 3.4.4). Only the above-mentioned bank stabilization has the potential for altering flood flows. All facilities within the floodplain were constructed in conformance with appropriate State and COE permits.

240.09 Q. Provide details of your methods of analyses for item 240.08. Include your assumptions of bases for pertinent parameters such as length and slope of drainage basins, times of concentration, infiltration rates, rainfall amounts and distribution, Manning's n" values, and any other assumptions or parameters used to determine the floodplains.

In some circumstances floodplain delineation by others may be acceptable. Specifically, if studies by FEMA or the Corps of Engineers are available for the site area, the details of analyses requested above need not be supplied; provide instead the reports from which you obtained the floodplain information.

A. Hydraulic calculations for both pre- and post-construction conditions were performed using the COE's HEC-2 Water Surface Profiles computer program. Major input requirements consist of channel and floodplain cross-sections, Manning's "n" coefficients for channel and floodplain reaches, a 100-year discharge rate, and a 100-year water surface elevation downstream of the proposed bank stabilization. The published 100-year flood elevation at RM 16.00 was selected as the starting elevation in the HEC-2 model.

Surveys of the existing channel and floodplain were supplied by the COE. These COE cross-sections at RM 16.00, 16.84, 17.40, and 18.34 were input to the HEC-2 model along with four additional cross-sections obtained from Supply System surveys at RM 16.97, 17.18, 17.28, and 17.67. These additional cross-sections provided data for an accurate hydraulic model in the vicinity of the project.

The HEC-2 model was calibrated to the COE's exact 100-year water-surface elevation at RM 16.00 and 17.40. It was within 0.2 feet of the published elevation at all other cross-sections in the project vicinity.

The 100-year discharge of 78,000 cfs was obtained from the COE report for the Chehalis River immediately below the Satsop River confluence (about RM 21).

Channel and right overbank "n" values of 0.03 and 0.06 were selected for existing conditions. A left overbank "n" value of 0.02 was chosen for the pre-construction project site and 0.08 for other reaches of the river. The post-construction left channel "n" value of 0.035 was used to simulate the resistance to flow of the riprap bank lining. A slide slope of 2.5 horizontal to 1 vertical was used for the left bank cross-section, and the entire left overbank was raised above the 100-year flood level so that no left overbank remained in the post-construction model. Distances between cross-sections along the left overbank, right overbank, and channel were adjusted to reflect the proposed alignment.

240.10 Q. Discuss the hydrologic effects of all items identified in 240.08 above. Discuss the potential for altered flood flows and levels, both upstream and downstream. Include the potential effect of debris accumulating on the plant structures. Additionally, discuss the effects of debris generated from the site on downstream facilities.

A. The left bank stabilization is designed to prevent flood flows from affecting the operation and integrity of the Ranney collectors and buried pipes. It will not alter flow rates nor will debris accumulate on the plant structures because of their new location above the 100-year flood level. Average flow velocity in the channel for the 100-year storm (78,000 cfs) will range between averages of 3 to 5 fps throughout the affected reach. These velocities are high enough to prevent excessive deposition and low enough to prevent scour of the riprap bank cover. Debris from upstream may accumulate on the riprap surface due to the increased roughness over existing conditions.

No groundwater hydrology effects were studied for the design, but the fill material used for the new left bank will probably have a minor local effect on both rainfall infiltration and groundwater levels. The areal extent of this change will be insignificant compared to the total recharge area.

Debris generated at the project site will not enter the river since the site will be above the 100-year flood elevation. Erosion and silt control measures will be employed throughout construction.

240.11 Q. Provide the details of your analysis used in response to 240.10 above. The level of detail is similar to that identified in item 240.09 above.

A. Computations used to develop the response to Q240.10 are provided with the response to Q240.09.

240.12 Q. Describe the effect on river flow of the bank protection constructed in the vicinity of the Ranney Well collectors.

A. According to the results of the HEC-2 tests, the left bank stabilization will have a very small effect on the stage, velocity, and distribution of flows across the channel and floodplain. The 100-year flood levels will rise slightly above conditions prior to construction, as noted in the response to Q240.08.

Channel velocities for the 100-year flood will average 0.2 feet per second (fps) higher for the planned modification than under existing conditions (an increase from 3.8 to 4.0 fps). Velocities in the right overbank will average less than 0.1 fps higher for project conditions (an increase from 1.0 to 1.1 fps). All of these changes will occur only between RM 17.09 and 17.60, and the model developed for design along the left bank shows that velocities elsewhere would not be increased.

Because the left overbank flows will be obstructed by the new bank, slightly higher proportions of the total flow will occur in the channel and on the right overbank. Approximately 4 percent more of the 100-year flow will be conveyed by the channel (a change from 42 to 46 percent), while 3 percent more of the 100-year discharge will occur on the right overbank (a change from 51 to 54 percent). About 7 percent of the total 100-year discharge is currently conveyed by the left overbank, and this amount will be redistributed almost equally to the channel and right overbank after construction.

240.13 Q. Calculate the radiological consequences of a liquid pathway release from a postulated core melt accident. The analysis should assume, unless otherwise justified, that there has been a penetration of the reactor basemat by the molten core mass, and that a substantial portion of radioactively contaminated sump water was released to the ground. Doses should be compared to those calculated in the Liquid Pathway Generic Study

(NUREG-0440, 1978). Provide a summary of your analysis procedures and the values of parameters used (such as permeabilities, gradients, populations effected, water use).

A. The scope and magnitude of potential radiological consequences of a liquid pathway release can be described by comparison of the WNP-3 site characteristics with those of the "small river" site considered in the Liquid Pathway Generic Study (LPGS). The LPGS compared the risk of accidents involving the liquid pathway (drinking water, irrigation, aquatic food, swimming and shoreline usage) for four conventional, generic land-based nuclear plants and a floating nuclear plant. Parameters for the land-based sites were chosen to represent averages for a wide range of real sites and are thus "typical," but represent no particular site. This response compares key parameters which characterize the WNP-3 site and the LPGS land-based small river site to determine if the WNP-3 liquid pathway consequences would be unique or present greater risks than identified in the LPGS. The parameters which are compared include groundwater travel time, sorption on geologic media, surface water transport, aquatic food consumption, shoreline and drinking water usage and irrigation.

The WNP-3 Reactor Building is located about 7000 ft south of the Chehalis River about 21 river miles from the mouth of the Chehalis in Grays Harbor (see Figures 2.4-1 and 2.4-2). Plant grade is at 390 ft MSL and the Reactor Building basemat is at 326 ft MSL. At the mean annual flow of 6600 cfs the river water surface in the vicinity of the plant is at about 8 ft MSL. The nearest water supply well is about 5000 ft NNW and draws from about 160 ft below the basemat elevation.

At the site groundwater is found confined in the Astoria sandstone formation, in the Pleistocene terrace deposits, and in the recent alluvial materials in the Chehalis River flood plain. The Category I structures, are founded on a common mat on the fresh sandstone of the Astoria formation. This formation is approximately 3000 ft thick, contains predominantly marine sandstone, and makes up the most extensive geologic unit at the site. The Astoria formation is then the relevant stratum in the evaluation.

The fresh sandstone has a premeability of 2×10^{-5} cm/sec (0.057 ft/day) and a porosity of about 35 percent (FSAR Table 2.5-16). With hydraulic gradients of 0.045 to the river and 0.032 to the nearest well, the groundwater travel time from the plant to both locations is about 2600 years. This may be compared with a travel time of 0.6 years for the 1500-ft distance to the river used in the LPGS.

The effective travel time of radionuclides which may contaminate the aquifer following a base mat penetration would be considerably greater due to adsorption and ion exchange on the sandstone. The distribution coefficients, K_d, for cesium and strontium, the critical radionuclides, are assumed to be 20 and 2, respectively. These values were taken from Table VII 3-7 of Appendix VII of WASH-1400 and are conservative when compared to values reported in the literature (e.g. NUREG/CR-0912, Vol. 1, Table 4-3). The calculated retention factors using these values for K_d, a porosity of 0.35 and a bulk dry weight density of 1.7 g/cm^3, are 98 for cesium and 10.7 for strontium. Using these retention factors, the travel time for Cs-137 and Sr-90 for transport to either the well or the Chehalis River are given in the table below. Comparable values used in the LPGS are also listed.

Parameter	LPGS	WNP-3
Retention Factor		
Sr-90	9.2	10.7
Cs-137	83	98
Time to Water (yrs)		
Sr-90	5.7	2.78×10^4
Cs-137	51	2.55×10^5
Number of Half Lives in Transit		
Sr-90	0.2	960
Cs-137	1.7	8470

From the above comparison it can be seen that the radionuclide travel times are considerably greater than those which characterize the small river site in the LPGS.

Once the contaminated groundwater reaches the Chehalis River, the initial dilution that occurs will be greater than that employed in the LPGS. The annual average water flow of the river at WNP-3 is approximately 6600 cfs, whereas, river flow at the LPGS site was taken as about 4500 cfs.

Contaminated river water could be used as drinking water by individuals and municipalities along the river reach. The LPGS assumed 32,300 water consumers. Presently there are no users that withdraw drinking water directly from the Chehalis River downstream of WNP-3.

The next comparison is the fishery catches. The annual average commercial finfish catch (in round pounds) from the Chehalis River and Grays Harbor is about 2.8×10^5 lbs, and the annual commercial invertebrate catch is about 4.0×10^4 lbs (Table 2.1-10). The recreational finfish catch was estimated to be about 5.0×10^4 lbs/yr. The LPGS round weight estimates employed for a land-based small river site were 8.5×10^5 lbs/yr and 1.7×10^6 lbs/yr for commercial and recreational catch, respectively. LPGS did not consider the invertebrate catch in the calculations for a small river site. The total of both the Chehalis River finfish and invertebrate weights is still less than half the finfish weights used in the LPGS.

Direct recreation exposure was considered in the LPGS with 2.2×10^7 user-hr swimming and 8.8×10^7 user-hr shoreline activity. To obtain comparable usage factors for the Chehalis River, the same user rates per surface area of river were employed as used in the LPGS. Multiplying these rates by the estimated area of the Chehalis River, the annual swimming usage totals 7.9×10^4 user-hr and shoreline activity would be 2.8×10^5 user-hr. Such a comparison ignores the fact that recreational usage of the Chehalis downstream of WNP-3 is mostly limited to seasonal shoreline activity and boating by hunters and fishermen. Each resident of Grays Harbor County would have to spend about 1650 hrs/yr on the river for total usage to approach the numbers used in the LPGS.

The irrigation pathway was not considered in the LPGS and is not a major consideration with WNP-3 because of the limited number of withdrawals (approximately 12) and the lead time available to implement mitigation measures.

The minimum groundwater travel time from WNP-3 to the Chehalis River was estimated to be roughly 2600 years, and because of the filtering properties of soil, the holdup of much of the radioactivity would be even greater. This would allow ample time for engineering measures to isolate the radioactive contamination near the source.

Several means are available for isolating contaminated groundwater. The construction of an impermeable membrane or sheet pilings to surround the site are two measures that could be employed to stop the flow of groundwater. Alternately, slurry trenches could be built to collect groundwater downgrade of the plant and to divert the water to a treatment or holdup facility. Wells could also be dug to remove the contaminated groundwater, however, pumping large volumes of water would impose unreasonable treatment requirements.

Doses to individuals and populations were calculated in the LPGS without consideration of interdiction methods such as isolating the contaminated groundwater or denying use of the water. In the event of surface water contamination, commercial and sports fishing, as well as many other water-related activities would be restricted. The consequences would therefore be largely economic or social, rather than radiological. In any event, based on the above comparison of radionuclide travel time, surface water dilution, and water usage, the radiological consequences for a liquid pathway release from WNP-3 would be small fractions of those postulated for the small river site in the LPGS.

WPPSS

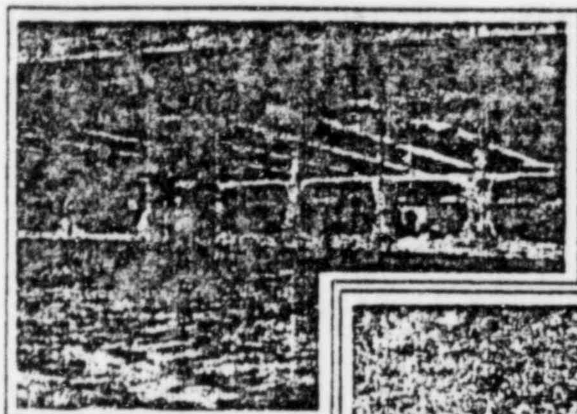
Rec'd 6/17/82

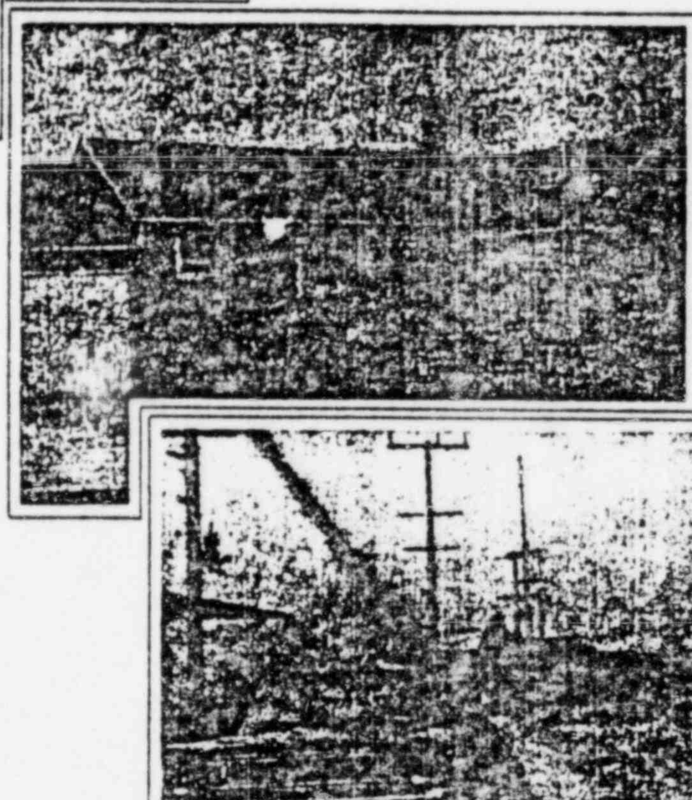
G.Y.

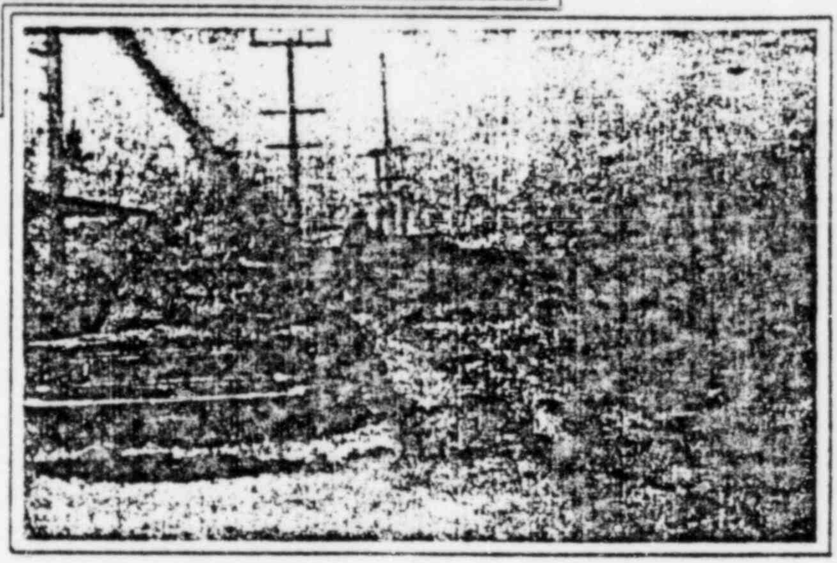
SPECIAL STUDY
SUGGESTED HYDRAULIC FLOODWAY

CHEHALIS RIVER

ABERDEEN TO SATSOP AND VICINITY
GRAYS HARBOR COUNTY, WASHINGTON







Prepared for
Washington State
Department of Ecology

February 1978

SEATTLE DISTRICT

UNITED STATES
ARMY CORPS OF ENGINEERS

CHEHALIS RIVER
ABERDEEN TO SATSOP AND VICINITY
CONTENTS

AUTHORITY

This report was prepared under authority granted by Section 206 of the 1970 Flood Control Act, Public Law 89-789, as amended. If additional studies are required, such as hydraulic evaluations of additional floodway alternatives, the U.S. Army Corps of Engineers can perform such studies upon request through the Washington State Department of Ecology.

ACKNOWLEDGMENTS

DISTRICT ENGINEER (Colonel John A. Poteat)

ENGINEERING DIVISION (Richard P. Sellevold, Chief)

Planning Branch (Dwain F. Hogan, Chief)

Flood Plain Management Section (William J. Spurlock, Chief)
Gerald M. Gardner - Assistant Chief
Horace Foxall - Study Manager
Edna Mae Yankosky - Secretary

Hydrology and Hydraulics Branch (Norman J. MacDonald, Chief)
Hydraulics Section (Richard Regan, Chief)
Lester Soule, Hydraulics
Water Management Section (Richard McLaughlin, Chief)
Robert Brown, Hydrology

Survey Branch (John Erlandson, Chief)
Mapping Section (Richard Hensen, Chief)
Computing Section (Lawrence Signani, Chief)
Field Survey Section (Jon Williams, Chief)
Photogrammetry Section (Cecil Hansel, Chief)

1.0 Introduction. This special study report was prepared by Seattle District, U.S. Army Corps of Engineers, for the State of Washington, Department of Ecology, on behalf of Grays Harbor County. The purpose of the study is to assist state and local governments in identifying flood hazard areas and to provide a basis for planning and regulating land use in the flood plain. This study delineates the 100-year flood plain boundary and suggested hydraulic floodway for the Chehalis River from the city of Aberdeen upstream to the vicinity of Satsop, and also includes lower portions of the Wynoochee and Satsop Rivers. This report supersedes the Chehalis River portions of previously published Flood Plain Information Study entitled CHEHALIS, WISHKAH AND ABERDEEN-HOQUIAM-COSMOPOLIS, June 1971, prepared by the Seattle District.

2.0 Scope. The report shows the extent and depth of flooding and a suggested hydraulic floodway for a 100-year frequency flood, for approximately 23.0 miles of the Chehalis River from river mile (R.M.) -2.0 near the mouth, to the confluence with the Satsop River, R.M. 20.2, and includes the Wynoochee River from the mouth to R.M. 1.55 (Devonshire Road Bridge) and Satsop River from the mouth to R.M. 2.1 (U.S. Highway 12 Bridge).

Additional field and aerial surveys were conducted for this study to augment survey data from the previous flood plain information study. Field surveyed cross-sections were taken on the Chehalis, Satsop, and Wynoochee Rivers. In addition, a 5-foot contour topographic map of the study area was developed by photogrammetric methods, using aerial photographs taken on 30 July 1974. Other data used in the study include high water marks for the 22 January 1972 flood, topographic maps from the Washington State Highway Department, streamflow records prepared by the U.S. Geological Survey (USGS) and rainfall records prepared by the weather service.

3.0 Past Floods. Major floods on the Chehalis River occur from October to March, caused by heavy precipitation sometimes accompanied by snowmelt. The tributary rivers along the Chehalis River within the study area rise rapidly during heavy rainfall because of the relatively quick runoff caused by steep terrain and channel slopes. Crest stage usually is reached within a few hours. Within the study area, flood crest stages on the main stem are usually within 2-3 days of heavy rain in the upper Chehalis basin, and within a few hours to a day on most tributary rivers and streams. In the immediate vicinity of the confluence of the Chehalis and Satsop Rivers, backwater effects may prolong high stages on either river for several hours.

The January 1972 flood was the largest recorded flood in the Chehalis River basin. The flood was most severe in upper portions of the Chehalis River basin, with comparatively moderate runoff from the tributary system downstream from Grand Mound.

A potential for extreme floods exists in the lower Chehalis River, downstream from Satsop, due to coincident timing of flood peaks from the main stem Chehalis River and the local tributary system. Intense maritime storm systems, following 2 to 3 days apart, could produce such coincident peaking of both main and tributary systems. Tributaries like the Satsop, Wynoochee and Wishkah Rivers rise rapidly from periods of heavy rainfall because of the steep basin terrain [illegible] the southern Olympic Mountains. Based on the available information, the Satsop and Wynoochee Rivers usually crest at about the same time. The Wishkah River is also assumed to crest at about the same time as the Satsop and Wynoochee because of the proximity and similarity of the drainage basins. The Satsop and Wynoochee Rivers crest approximately 2 days earlier than the Chehalis at Porter during storms that include the entire basin. However, records indicate that occasionally the Chehalis and its tributaries crest at about the same time when a second storm system causes the rivers of the lower Chehalis basin to peak coincidentally with arrival of a main stem flood crest developed in the upper basin. A combination of storm systems similar to that described above occurred in December 1933 and produced the most extreme flooding experienced in the lower portions of the Chehalis River basin.

4.0 <u>Hydrologic Analysis</u>. Hydrologic investigations were made to establish the 100-year frequency flood discharge for each study reach, using USGS streamflow records for the Chehalis River watershed streamgages. See table 1 for streamgage description. This streamgage data aided in developing hydrographs and simulation models for the various streams in the watershed. The investigations also include studies of drainage area characteristics, climatological records, flood discharge magnitudes and frequencies, regional flood relationships, and computerized streamflow flood routings. Table 2 shows discharges for the 1972 flood and the 100-year flood at various locations in the study area.

5.0 <u>Hydraulic Analysis</u>. The water surface profiles for the 100-year frequency flood were calculated utilizing two computer programs to compute water surface profiles by mathematical models. The two programs were "Backwater Curve - Method II," developed by Seattle District Army Corps of Engineers, and the "Gradually Varied Unsteady Flow" Hydrologic Engineering Center (HEC) model.

The backwater curve - Method II, a standard-step, steady-flow computer program performs an energy balance based on Manning's friction formula. The gradually varied unsteady-flow computer program is a flood routing procedure using hydraulic methods. It simulates movement of hydraulic transients by use of the St. Venant equations, the basic equations of unsteady flow. In general, this computer program will permit using either a time-varying stage or discharge at two known locations and calculating the resulting hydrographs of discharge, elevation, and velocity throughout the reach between the two locations.

Hydraulic calculations were made to develop water surface elevations for the 100-year frequency flood under natural conditions, and to develop the boundaries of a suggested hydraulic floodway. A hydraulic floodway is a portion of the flood plain needed to pass a regulatory flood without a significant rise in water surface elevation. For this study, a regulatory flood is defined as the 100-year frequency flood, and a significant

TABLE 1

Streamgage Data

Location	USGS No.	Drainage Area Sq. Mi.	Period of Record 1/	Maximum Recorded Peak Flow in CFS/Date
Chehalis River near Grand Mound	1202750O(	895	October 1928 - Present	49,200/January 1972
Chehalis River at Porter	12031000	1294	January 1952 - September 1972 October 1972 - September 1975 2/ October 1975 - Present	55,600/January 1972
Cloquallum Creek at Elma	1232500	64.9	July 1942 - October 1943 3/ July 1944 - September 1972 October 1972 - Present 2/	5,080/ ecember 1959
Satsop River near Satsop	12035000	299	March 1929 - Present	44,600/January 1935
Wynoochee River above Black Creek	12037400	155	October 1956 - Present	25,500/January 1968

Footnotes:
1/ Continuous Recorder except as noted
2/ Maximum annual crest stage recorder
3/ Fragmentary

TABLE 2

FLOOD DISCHARGES – 100 year

LOCATION	RIVER MILE	DRAINAGE AREA SQ-MI	CASE A 1/ Chehalis R. DISCHARGE CFS	CASE B 1/ Satsop R. DISCHARGE CFS	1972 DISCHARGE CFS
Chehalis River near Grand Mound	59.90	895	58,000		[illegible]3,200
Chehalis River at Porter	33.30	1,294	62,500	35,000	55,600
Chehalis River at South Elma	25.75	1,409	60,800		55,000
Chehalis River Above Satsop River	20	1,455	60,900	22,500	56,500
Satsop River near Satsop	0 - 2.1	299	18,700 (9,500)2/	55,000	31,000 (13,500)2/
Chehalis River-Satsop River Confluence	20.20	1,754	70,400	78,000	70,000
Wynoochee River	0 - 1.55	155	6,500	18,000 3/ 23,000 4/	16,800
Above Black Creek Ungaged Local 5/	--	---	4,000	15,000	------

Footnotes:

1/ Case A: 100-year flood on Chehalis River at Porter with coincidental flows on Satsop, Wynoochee and ungaged tributaries in the study area.

Case B: 100-year flood on Satsop River with coincidental flows on Chehalis, Wynoochee, and ungaged tributaries in the study area.

2/ Satsop River discharge coincident with peak discharge on Chehalis River.

3/ Regulated by Wynoochee Dam (100-year natural peak discharge at reference gage station is 38,000 CFS)

4/ 100-year regulated discharge, Wynoochee River at mouth, used for IRF profile for Wynoochee River.

5/ Ungaged local area between Chehalis River mouth and Satsop River = 220 sq. mi.

rise in water surface elevation is defined as 1 foot. The remaining portion of the flood plain is called the floodway fringe. The fringe area is not required for conveyance (flood carrying capacity) of floodflows and may be filled, diked, or otherwise obstructed without causing a significant rise in water surface elevation. Tables 3 and 4 show predicted water surface elevations for both natural and floodway conditions.

6.0 Chehalis River. To determine the maximum 100-year frequency flood conditions within the study area, two hydrologic investigations were conducted as follows:

Case A - A flood comprised of discharges of approximately 100-year intensity at the streamgage, Chehalis River near Porter, accompanied by reasonable coincident flows from the river system of the lower Chehalis basin; e.g., Satsop and Wynoochee Rivers and ungaged local streams.

Case B - A flood comprised of approximately 100-year intensity on the Satsop River accompanied by reasonable coincident discharges on the mainstem Chehalis River above the Satsop River, the Wynoochee River, and ungaged local streams.

Case B produced the higher stages downstream of the Satsop River. Flood discharges on the Chehalis River between Chehalis at Porter, river mile 33.3, and the mouth of the Satsop, river mile 20.2, were established using North Pacific Division's computer program "Streamflow Synthesis and Reservoir Regulation," (SSARR). Flood hydrographs for Chehalis River at Satsop River and hydrographs for the tributaries downstream from the Satsop River were used as boundary conditions in the "Gradually Varied Unsteady Flow" computer program.

The suggested hydraulic floodway limits for the Chehalis River were determined by using the 100-year natural water surface profile defined by the unsteady flow model. Then the steady-flow backwater model was adjusted to produce the same water surface profile throughout the reach. This enabled the hydraulic floodway option of the steady-flow model to be used. Discussions were held with Grays Harbor County officials to determine their needs with respect to future development and water management. Their input was incorporated into a hydraulic floodway determination using the equal-conveyance-reduction theory but predetermining where channel-only or full-valley flow would be allowed. Upon determination of a floodway based on steady-flow conditions, the unsteady-flow model was used to analyze the effects of lost storage upon the water surface profiles. Additional floodway computer calculations were made on adjusted floodways until the unsteady model water surface floodway profile was within the acceptable range of 1-foot maximum increase in stage.

TABLE 3

WATER SURFACE ELEVATIONS - 100-YEAR FREQUENCY FLOOD

CHEHALIS RIVER

CROSS SECTION 1/	MEAN VELOCITY (F.P.S.)	WITH FLOODWAY (M.S.L.)	WITHOUT FLOODWAY (M.S.L.)	DIFFERENCE (FT)
- 1 + 89	1.9	10.0	10.0	0.0
- 0 + 96	2.5	10.0	10.0	0.0
- 0 + 68	2.9	10.0	10.0	0.0
- 0 + 36	3.1	10.0	10.0	0.0
0 + 08	3.0	10.1	10.1	0.0
0 + 33	3.4	10.1	10.1	0.0
1 + 03	4.2	10.1	10.1	0.0
1 + 34	3.7	10.1	10.1	0.0
1 + 87	4.1	10.2	10.2	0.0
2 + 19	3.2	10.2	10.2	0.0
2 + 93	3.3	10.2	10.2	0.0
3 + 27	3.8	10.3	10.3	0.0
3 + 97	4.3	10.3	10.3	0.0
6 + 27	2.2	10.6	10.6	0.0
7 + 30	2.1	10.8	10.8	0.0
8 + 36	2.5	11.2	11.2	0.1
9 + 17	2.1	11.6	11.6	0.0
9 + 96	2.0	12.0	12.0	0.0
10 + 76	2.3	12.9	12.9	0.0
11 + 87	1.9	13.9	13.8	0.1
12 + 27	3.3	14.6	14.4	0.2
13 + 00	3.1	15.6	15.3	0.3
13 + 11	2.7	15.6	15.4	0.2
14 + 12	2.6	16.6	16.3	0.3
15 + 00	1.	17.7	17.2	0.5
15 + 78 RM 16.00	2.0	18.4	17.9	0.5
16 + 76 16.84	2.3	19.0	18.5	0.5
17 + 98 17.46	1.6	19.6	19.1	0.5
19 + 00 18.34	2.4	21.2	20.6	0.6
20 + 04	2.5	24.5	23.7	0.8
20 + 70	7.1	27.3	26.5	0.8
21 + 22	1.2	30.0	29.2	0.8
22 + 00	1.0	30.2	29.3	0.9

1/ Station numbers correspond to cross sections shown on Plates 2,3 & 4.

TABLE 4

WATER SURFACE ELEVATIONS-- 100-YEAR FREQUENCY FLOOD

WYNOOCHEE RIVER

CROSS SECTION 1/	MEAN VELOCITY (F.P.S.)	NATURAL FLOODWAY (S.L.D.)	HYDRAULIC FLOODWAY (S.L.D.)	DIFFERENCE (FT)
0 + 02	2.0	15.8	15.1	0.7
0 + 22	1.7	15.9	15.3	0.6
0 + 02	4.2	15.3	15.7	0.6
2 + 02	3.7	16.9	16.2	0.7
3 + 02	4.2	17.5	16.8	0.7

SATSOP RIVER

CROSS SECTION 1/	MEAN VELOCITY (F.P.S.)	NATURAL FLOODWAY (S.L.D.)	HYDRAULIC FLOODWAY (S.L.D.)	DIFFERENCE (FT)
6 + 66	1.4	29.5	28.5	1.0
5 + 55	1.1	29.5	28.5	1.0
4 + 44	1.7	29.6	28.6	1.0
3 + 33	[illegible]	29.8	28.8	0.9
2 + 22	2.6	30.2	29.3	0.9
1 + 11	3.7	31.3	30.4	0.9
1 + 01	3.8	32.2	31.4	0.8
2 + 01	8.2	34.3	33.9	0.5
2 + 51	7.5	36.8	36.8	6.0
3 + 01	10.2	37.8	37.6	0.2

1/ Station numbers correspond to cross sections shown on Plates 3 & 4

7.0 Satsop and Wynoochee Rivers. The 100-year flood discharge for the lower 2.1 mile reach of the Satsop River was estimated from flood frequency studies based on the 29-year record of streamflows at the USGS streamgage, Satsop River near Satsop, located at river mile 2.0. The 100-year flood discharge for the lower 1.55 mile reach of the Wynoochee River was estimated from the flood history at USGS streamgage, Wynoochee River above Black Creek, near Montesano, together with investigations of the flood control operation at Wynoochee Dam project which began in August 1972.

The natural conditions and the suggested hydraulic floodway limits for the Wynoochee and Satsop Rivers were determined by using the starting elevations of the water surface profile that resulted from the Chehalis River unsteady model floodway analysis (Case B type flood).

With this starting elevation the 100-year frequency flood water surface profile was computed using the steady-flow model. Steady-flow equal percentage conveyance reduction was used to determine the floodway for the Wynoochee River. The floodway for the Satsop River incorporated equal percentage conveyance reduction and engineering judgment. There is a high water channel in the right overbank of the Satsop flood plain. If this is allowed to pass water as part of the floodway, then a greater area of floodway fringe would result. This was incorporated into the floodway for the Satsop River area.

8.0 Tidal Analysis. The "Gradually Varied Unsteady Flow" computer program was used to simulate tidal and river discharge conditions in the Chehalis estuary to establish combined tidal and streamflow effects that produce the water surface of the 100-year frequency flood elevation. Two tidal conditions were examined: (1) A 100-year tidal cycle and a mean annual peak flood on the main stem, and (2) 100-year Chehalis River flood (Case A flood type) and a mean higher high water tidal cycle. The mean higher high tide and 100-year tide (4.72-feet mean sea level and 10-feet mean sea level, respectively) were estimated from a frequency curve. This curve was prepared by the Seattle District office in February 1970, revised August 1973, based on historical tide observations at a Port of Grays Harbor staff gage at Aberdeen.

A water surface profile was developed using the 100-year tidal cycle as the downstream boundary with the mean annual peak hydrograph for the Chehalis River as the upstream boundary and another profile with the mean higher high water tidal cycle and 100-year Chehalis River hydrograph as the respective boundaries. The 100-year Chehalis River water surface profile, shown in this report is the combination of the highest individual portion of these two profiles. The tidal-influenced flooding extended upstream to approximately river mile 8.5.

9.0 Regulatory Floodway. A regulatory floodway is needed for land-use regulation to assure that sufficient area is preserved in the flood plain to safely pass a major flood, such as the 100-year frequency flood. The floodway developed in this study is called a suggested hydraulic floodway because it is based primarily on hydraulic factors and is not intended for use as a regulatory instrument until implemented or revised by cognizant state or local authorities, after consideration of local political, social, economic, and environmental factors.

10.0 Flood Proofing. The fringe areas of the flood plain can be obstructed without causing a significant rise in flood depths. These areas are generally suitable for development, provided that structures are properly flood proofed by filling, diking, or other protective construction. Minimum floor elevations for flood proofing can be determined from the elevation of the site relative to the 100-year frequency flood profile, and adding a freeboard of at least 2 feet. Freeboard is needed because of possible increases in flood depths that might be caused by unpredictable debris accumulations, sediment deposition, or channel shifts.

11.0 Obstructions. During floods, debris collecting on bridges and culverts could decrease their carrying capacity and cause greater water depths (backwater effect) upstream of these structures. Since the occurrence and amount of debris are indeterminate factors, only the physical characteristics of the structures were considered in preparing profiles of the 100-year frequency flood. Similarly, the maps of flooded areas show the backwater effect of bridges, but do not reflect increased water surface elevation that could be caused by debris collecting against the structures, or by deposition of silt in the stream channel under structures.

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