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BAKER DAM
FAILURE INUNDATION STUDY

Prepared for

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by

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Note: The results of this study do not in any way reflect upon the structural integrity of any part of the White River Project. The "failures" are worst case hypothetical conditions assumed solely to comply with Federal Regulations.

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1.0 DEFINITIONS AND ABBREVIATIONS

Base Flow: The flow rates in the Baker/Skagit system that were used for the initial flow rates in the dam break simulations. Based on typical spring flows for the area.

Base Flow Stage: The water surface elevation resulting from an FEQ simulation running base flow only through the river.

Arrival of the Wave Front: Defined as a one foot increase in stage above the base flow stage.

Zero-damage Flood Flow: Defined as the flow where damage is just beginning to occur. Zero damage flood flow varies from place to place depending on the diking system, or obstructions to flooding. This study assumes a zero damage flood flow rate of 67,800 cfs which is estimated at Mt. Vernon at a stage of 28 ft.

Peak Stage: The maximum stage achieved at a particular point during the passage of the flood.

Peak Discharge: The maximum discharge achieved at a particular point during the passage of the flood.

Critical Flow: A term used in open channel flow theory for the flow rate that occurs at minimum specific energy. If the water depth is known at a point where critical flow occurs, then the flow rate can be determined mathematically. Critical flow occurs at free outfalls, such as the dam breaches.

FEQ: The Full EQuations, unsteady, open channel flow simulation program used to model the Skagit/Baker dam break.

FERC: Federal Energy Regulatory Commission.

EAP: Emergency Action Plan

cfs: cubic feet per second

ft: feet

2.0 SUMMARY

The Federal Energy Regulatory Commission (FERC), through the Hydropower Licensing Branch, requires licensed hydroelectric power projects to complete and file "Emergency Action Plans" with the Commission (1). FERC provides guidelines for the preparation of Emergency Action Plans. These plans promote public safety in the event of accidents or natural disasters that would cause a sudden release of water from reservoirs used for hydroelectric power. Emergency Action Plans contain three major topics:

Assumed Modes of Failure for Projects
Inundation Mapping
Warning Systems

This report includes assumed modes of failure and inundation analysis for the Upper Baker Development (Upper Baker) and for the Lower Baker Development (Lower Baker) which are owned by Puget Sound Power and Light Company (PSLP), and comprise the Baker River Project (FERC Project 2150). *The causes of failure assumed in this study are "worst case" hypothetical conditions that are considered solely for the purpose of emergency action planning.* The consequences of an actual failure would very likely be less severe than the consequences of the worst failure used in this study.

Hydrocomp, Inc. has previously investigated the inundation that would be caused by failure of the Baker River Dams (Hydrocomp, 1981). The 1981 study was reviewed and additional analyses were made in 1989 in response to updated FERC guidelines (Appendix B). This report describes the updated analyses and supersedes the 1981 report and inundation maps. The investigation included analyses of the inundations that result from the following three failure cases:

Failure of Upper Baker Dam only
Failure of Lower Baker Dam only
Failure of Upper Baker Dam followed by the
induced failure of Lower Baker Dam

These failures are assumed to be caused by natural catastrophes, such as landslides and earthquakes of a size never before experienced in this area. For the inundations that result from these cases, the maximum discharge and the elapsed times from the failure to flood stage and to the maximum discharge are given. Maximum discharges are shown in tables and on figures for several points in the Skagit River Valley.

The technical analysis used to study the inundation is "full equations routing" which predicts flood wave movement using the complete energy and momentum equations. Full equations routing, rather than conventional kinematic wave routing, is needed for dam break studies because the simplifying assumptions used for conventional routing are not valid when the river stage is changing rapidly. The results indicate that very large flows would result in the Skagit Valley from all of the assumed failure cases. The maximum discharges occurring at Concrete, Sedro-Wooley, and Mount Vernon are shown for each of the failure modes in Table 1.1.

The flow produced at Sedro-Woolley by the failure of both Upper and Lower Baker Dams is nearly as large as the maximum observed historic flood of 220,000 cfs on November 30, 1909 (2). While specific local differences may be significant, the flooding between Sedro-Woolley and Puget Sound from all of the inundation cases would be similar to that mapped by the Corps of Engineers for a Standard Project Flood (3). The peak flow from any of the failure modes creates a maximum water surface elevation at Sedro-Woolley of between 51.7 and 61.9 ft, a range of 10.2 ft. At Mt. Vernon, the range of maximum water surface elevations is reduced to 4.4 ft, from 27.9 to 32.3 ft. Levees protecting urban settlements and farm lands between Sedro-Woolley and Puget Sound would be expected to fail, and low lying urban centers like Burlington would be flooded.

In the lower Skagit Valley the flood is not a sudden, high velocity "wall of water". It is more like a flood caused by rainfall or snowmelt, and it occurs 8 to 15 hours after the hypothetical catastrophe at the Baker River project. For the cases where Lower Baker Dam does not fail immediately, there is a significant length of time before flooding occurs. For example, in the case of failure of Upper Baker Dam only, the elapsed time before the Skagit River reaches flood stage at Concrete is 30 minutes. Flood stage is reached at Sedro-Woolley 11.3 hours after the formation of the breach. This "warning-time" would be very important in an actual emergency. Inundation maps and elapsed times for the floods to reach points in the Skagit Valley are relatively independent of failure modes. For example if a landslide displaces the contents of Baker Lake in 60 seconds, or an earthquake causes a breach in the dam over ten minutes, the timing and peak of the flood at Sedro-Woolley are essentially unchanged.

TABLE 2.1 SUMMARY OF STUDY RESULTS AT SELECTED LOCATIONS

FAILURE MODE	ARRIVAL OF THE WAVE FRONT		ARRIVAL OF ZERO DAMAGE FLOOD FLOW		ARRIVAL OF THE PEAK FLOW		
	Water Surface Elevation (ft)	Time of Arrival (hrs)	Water Surface Elevation (ft)	Time of Arrival (hrs)	Peak Flow (cfs)	Water Surface Elevation (ft)	Time of Arrival (hrs)
Upper Baker Only							
Concrete	171.2	0.3	176.7	0.5	403,000	211.4	3.4
Sedro-Wooley	36.3	9.8	40.9	11.3	217,000	53.1	15.8
Mt. Vernon	22.3	13.8	23.9	15.3	183,000	28.7	22.3
Lower Baker Only							
Concrete	171.2	0.1	174.1	0.1	878,000	224.9	0.4
Sedro-Wooley	36.3	8.0	40.8	9.5	192,000	51.7	13.2
Mt. Vernon	22.3	12.0	23.9	13.7	155,000	27.9	19.7
Induced Failure of Lower Baker							
Concrete	171.2	0.3	176.7	0.5	1,263,000	239.4	3.0
Sedro-Wooley	36.3	7.6	40.4	8.1	446,000	61.9	12.0
Mt. Vernon	22.3	10.5	23.7	11.5	363,000	32.3	17.2

3.0 INTRODUCTION

This report summarizes Hydrocomp's study of the Baker River Project (FERC Project #2150), conducted for Puget Sound Power and Light Company (PSPL) as part of their Emergency Action Plan (EAP) required by the Federal Energy Regulatory Commission (FERC). The study is an analysis of the movement of flood waters in the Baker/Skagit river system caused by failure of the two Baker River dams. The report is an updated version of Hydrocomp's 1981 study of the same area with revisions based on the new FERC specifications for dam break studies (4). Results of the study do not in any way reflect upon the structural integrity of the dams, and are not meant to be construed as such. The effects of failure of the dams are investigated only to conform with FERC regulations.

3.1 The Baker River Project

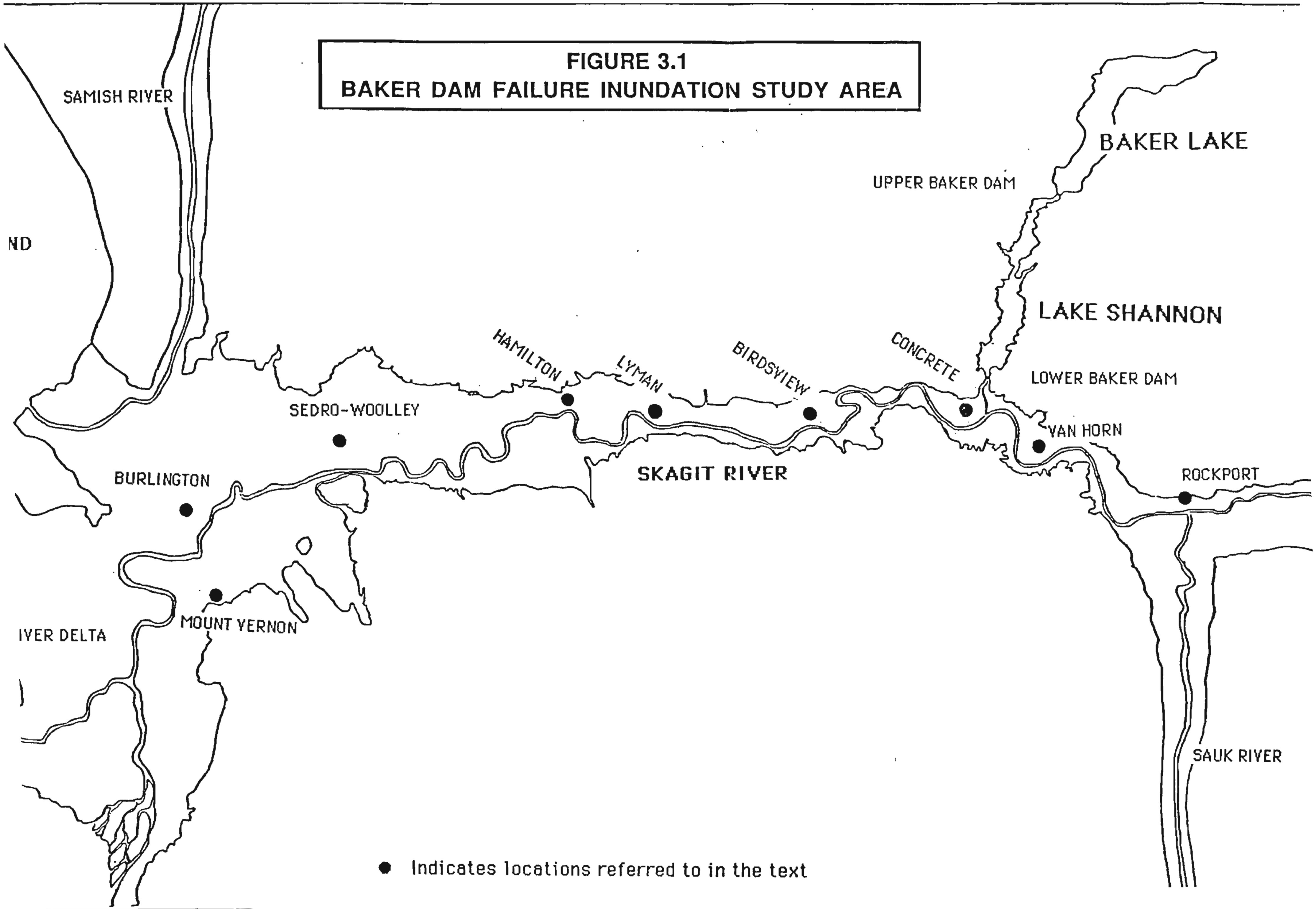
The Baker River Project consists of two dams: the Upper Baker dam which is a 280 foot concrete gravity dam, and the Lower Baker dam which is a 270 foot concrete arch dam. Both dams stand on the Baker River. The reservoir behind Upper Baker Dam is Baker Lake. The reservoir behind Lower Baker Dam is Lake Shannon. One mile below Lower Baker Dam, Baker River discharges into the Skagit River. The Skagit River (below the Baker confluence) flows through a one to three mile wide alluvial valley for 34 miles, before reaching the wide, flat Skagit and Samish Delta, and ultimately Puget Sound. Figure 3.1 delineates the Baker/Skagit System and shows locations referred to in this report for discharges and water surface elevations.

3.2 FERC Guidelines and Criteria

The FERC requirements for Emergency Action Plans are the result of an awareness by the Commission of the fact that every dam runs at least some risk of failure. As a result, the owner of any water impounding structure whose failure could endanger life or property, is required by CFR 18 (1) to prepare an EAP. Emergency Action Plans for hydroelectric projects must include the following:

1. Analysis of probable types of failures and their consequences
2. Identification of areas which would be affected by failures
3. Provision for maximum early warning to affected areas
4. Preplan of actions to prevent failures or to minimize impacts
5. Documentation of pre-emergency planning

FIGURE 3.1
BAKER DAM FAILURE INUNDATION STUDY AREA



This study deals with the first two of the questions, and with developing conclusions from which PSPL might prepare the EAP and answer the remaining questions. The revised FERC *Emergency Action Plan* guidelines issued February 22, 1988 were used to develop dam failure scenarios. The floods that would be caused by the dam failures were computed, and the area affected by the largest of these floods was delineated on an inundation map. Times indicating the arrival of the front and peak of the flood wave are also indicated on the inundation map.

Among the guidelines and criteria developed by FERC for inundation studies are "suggested breach parameters" (Table 1, Appendix B). Both the geometry of the breach in the dam and the time span over which it occurs are specified for several categories of dam construction types. Upper Baker Dam is a concrete gravity dam, with an upper crest width of approximately 1260 feet. The breach that was assumed to occur in Upper Baker Dam was at the top of the range suggested by FERC for gravity dams, with an average width equal to one half the crest width and side slope of zero. The simulated breach is 630 feet wide, and extends to the base of the dam (Figure 3.2). The FERC guidelines suggest a time to failure between 0.1 and 0.3 hours for gravity dams. The simulated breach for this study occurred over 0.17 hours (10.2 minutes).

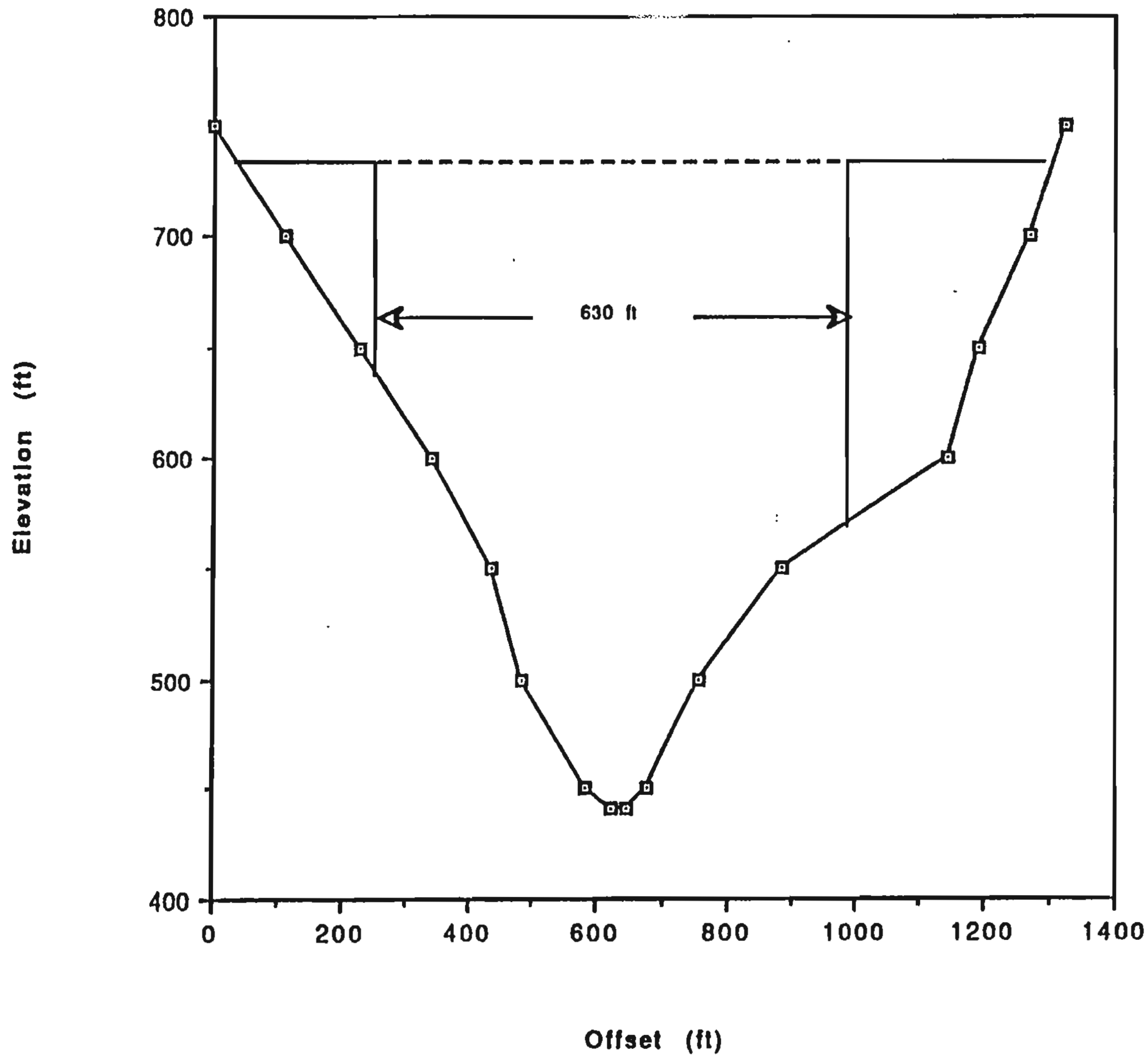
Lower Baker Dam is a concrete arch dam. The FERC guidelines suggest an average breach width for arch dams equal to the crest length, and a side slope between zero and the slope of the valley walls. The modeled breach in Lower Baker Dam conforms to these guidelines. The breach is the same geometry as the dam; that is, the entire dam is removed when failure occurs. The time span of failure suggested by FERC for arch dams is less than 0.1 hours. For Lower Baker Dam the failure was assumed to occur in 0.07 hours (4.2 minutes).

3.3 Failure Causes

This study is not sensitive to the specific cause of dam failure. However, possible causes of failure have been identified in order to illustrate the type of event that would be required to cause the inundations described in this report. Possible causes of failure include:

1. A sudden loss of structural integrity, most likely due to a massive earthquake in Baker Valley.

FIG. 3.2 UPPER BAKER DAM
Cross Section of Assumed Breach



2. Massive landslides into the reservoirs sufficient to cause excessive overflow depths (50 or more feet) at the dam sites, which are assumed to cause failure of the structure.

3. Failure at Upper Baker Dam, which results in excessive overflow depths at Lower Baker Dam, inducing the failure of Lower Baker Dam.

Earthquakes and landslides on a massive scale sufficient to cause project failures are highly improbable. They are simply judged to be more likely than other catastrophes such as meteor strikes.

Piping or erosive failures assumed for safety studies of earth and rock dams are not pertinent for the two Baker Dams.

3.4 West Pass Dike

West Pass Dike, located near Upper Baker Dam on Baker Lake is an earthen dam. The flood caused by the dike failure would flow over a distance of approximately 4250 feet before entering the Baker River near its confluence with Sulphur Creek, endangering structures located below the dike (5). The embankment failure was simulated according to FERC guidelines for earthen dams. The embankment is approximately 1200 feet long at the crest and 34 feet high. An average breach width of four times the height of the dam, developing over a failure time of 0.5 hours, was used. A trapezoidal shaped breach assumed with 45 ° side slopes.

The breach in West Pass Dike created a peak discharge of 40,000 cfs into Lake Shannon. The reservoir water surface level rose eight feet to an elevation of 445 ft. A peak discharge of 25,000 cfs entered Baker River en route to Skagit River. Because this peak is much smaller than the flood resulting from any of the other failure scenarios analyzed in this study, no further analysis of the effect on the Skagit River Valley was made.

3.5 Contents of the Report

Section 4.0 describes the FEQ software system used in this study and recounts the specific techniques and assumptions used to define the dam failure problem. The use of topographic data and the description of the system for use by the computer program are detailed. A brief discussion of full equations theory and input requirements are presented in Appendix A. Section 5.0 presents the results of the computer simulations. The movement of the flood waves are described as they pass through the channel system. Section 6.0 contains the study conclusions.

4.0 FULL EQUATIONS MODELING OF THE BAKER/SKAGIT SYSTEM

The movement of the flood wave caused by outflow through a breach in a dam must be calculated using the full equations of motion, which are based on the principles of conservation of mass and conservation of momentum. A computer program called FEQ was executed on Hydrocomp's Sun 4 workstation to solve the full equations of motion for the Baker/Skagit System for the three dam failure cases. FEQ was developed by Dr. Delbert Franz of Linsley-Kraeger Associates (6). FEQ is the most recent version of the FULEQ software that was used for the 1981 Baker River Project Inundation Study. Appendix A of this report describes full equations theory. This chapter explains basic modeling terminology and describes how the Baker/Skagit System was modeled with FEQ.

4.1 Model Concepts and Terminology

The rivers and reservoirs that are being represented by the computer model must be defined as a channel system. A channel system is a network of channels along which water flows forming junctions where two or more channels meet. The objective of streamflow modeling is to determine the depth of flow and the flow rate at points of interest in the channel network. The depth of flow is taken as the depth perpendicular to the modeled channel bottom profile. Each channel is defined by cross-sections taken at points along the river. The reliability of the simulation relies largely on the accuracy and detail of the channel description. Local irregularities unaccounted for in the cross-sections can be simulated numerically by a parameter representing roughness.

Naturally occurring lakes, as well as man-made lakes, may be present in the network. There may be a variety of structures such as roads and railways, underpasses, weirs, dams, points where water is taken out of the channel, and points where water is discharged into the channel. The network of channels must have clearly established boundaries or limits.

The channel system, shown in Figure 3.1, includes 67 miles of the Skagit River, from several miles upstream of the Sauk river to Puget Sound. Inflow from the Sauk River and the Baker River were accounted for but inflow from all other small tributaries was ignored in this study.

Detailed cross-sections were obtained for the river and flood plain area from Concrete to Mount Vernon from the U.S. Army Corps of Engineers 1972 *Flood Insurance Study for Skagit County, Washington (2).* These cross-sections were taken approximately every 3/4 mile. The remaining river cross-sections were scaled off U.S. Geological Survey topographic maps (7). The accuracy of the cross-sections depends on the scale of the topographic maps used. The USGS map contours range from 20 ft to 100 ft intervals.

Baker Lake cross-sections were scaled from 1"=400' Baker River Project Topographic maps (8) with a 10 ft contour interval. Contour mapping for Lake Shannon could not be obtained. Cross-sectional shapes were estimated using the top width at normal pool elevation and assuming a uniformly sloping bottom profile and a roughly parabolic cross-sectional shape. The resultant cross-sections yielded an elevation vs. storage curve which did not appreciably differ from the total capacity as reported in "Lake Shannon Usable Storage Table" (9).

4.2 Initial Conditions and Boundary Conditions

The specification of boundary and initial conditions required a number of assumptions. A constant inflow to Baker Lake of 6000 cfs was assumed. The initial elevation of Baker Lake was assumed to be 724 ft, the normal full pool elevation (10). The initial elevation of Lake Shannon was assumed to be 436.8 ft, also at full pool (10). The outflow through each dam site was supplied to the model as a discharge vs. elevation table. In calculating the tables, *critical flow* was assumed to occur at each dam site. The upstream inflow to the Skagit River was assumed to be a constant 12,000 cfs, above the Sauk. The flow from the Sauk River was also 12,000 cfs. The sum of the flows of the Upper Skagit, Sauk, and the Baker Rivers represent a typical May/June flow at Concrete (11). Table 4.1 lists the water surface elevations resulting from a simulation of the Baker/Skagit system **without** a dam break. The table is provided for comparison to flood flow elevations described in later sections of this report.

TABLE 4.1

WATER SURFACE ELEVATIONS AT BASEFLOW FOR SELECTED LOCATIONS ALONG THE SKAGIT RIVER

Station	Approximate Location	Water Surface Elevation (ft)
67.5	Rockport	226.5
58.5	Van Horn	179.4
54.9	Concrete	170.2
45.9	Birdsview	118.4
40.5	Hamilton	94.5
34.8	Lyman	70.1
23.2	Sedro-Woolley	35.3
18.6	Burlington	25.3
16.0	Mt. Vernon	21.3
2.0	Conway	4.8
0.0	La Conner	4.1
0.0	Edison	8.1

The mean-higher-high tide level of 4.1 ft at Bellingham, Washington was assumed at the mouth of the Skagit River. A flow vs head relationship was developed for use by the program to take into account the effect of the 8 ft sea dikes at Puget Sound.

The most severe failure, the induced failure of Lower Baker Dam, was simulated using a higher initial base flow in the Saulk and Skagit Rivers. The inflow to the Skagit River was set at 31,800 cfs, inflow to the Sauk River was set at 30,000, and inflow to the Baker River remained at 6000 cfs. The total of the flows through the Skagit River equal to the zero damage flow rate of 67,800 cfs. This simulation was made to evaluate the effect of a higher base flow on the water surface elevations and timing of the flood e wave resulting from the assumed failure of Upper Baker Dam which in turn causes the assumed failure of Lower Baker Dam by overtopping. Water surface elevation increases, in the vicinity of the confluence of the Baker and Skagit Rivers, ranged from 0.5 to 0.8 ft over those resulting from the simulation using the lower base flow of 30,000 cfs. Downstream of the confluence with the Baker River, water surface elevation increases ranged from 0.14 at the mouth of the Skagit River to 1.6 ft at Burlington.

4.3 Failure Modes Studied

For the Baker River Project, which has two dams in series, it is useful to consider not only the failure of each of the dams separately as well as both of the dams. If both dams fail, they might fail simultaneously, or the Upper or Lower dam might fail first. The failure of both dams simultaneously might be caused by a severe earthquake. Hydrocomp determined in 1981 that the flood caused by the simultaneous failure of both dams would not be as severe as the flood caused by the failure of the upper dam followed by the failure of the lower dam. The simultaneous failure of both dams was not reconsidered for this study. The failure of the upper dam is likely to induce the failure of the lower dam by overtopping and is therefore the most probable scenario for the failure of both dams.

Each of the failure modes were simulated using the same failure characteristics, i.e., a level pool reservoir with 6000 cfs base discharge flowing through the dam site while the breach in the dam develops rapidly. Flow through the breach is assumed to be *critical flow* throughout the simulation. The depth of the flow at the dam site controls the discharge.

Upper Baker Dam is simulated to fail in 0.17 hours (10.2 minutes). The simulated failure of Lower Baker Dam occurs in 0.7 hours (4.2 minutes). The simulated failure times were determined based on FERC guidelines, as described in Section 3.2. For the case where both dams fail, the failure of Lower Baker Dam begins when it is overtopped by 50 feet.

5.0 RESULTS

The three failure scenarios described in Chapter 4 were simulated using the full equations routing program, FEQ. Each simulation included a failure of one or both dams. The peak discharge occurs almost immediately at the location of the breach. As the water surface elevation lowers, the discharge is reduced. The discharge through the breach is controlled by a discharge vs. head relationship governed by *critical flow* conditions.

5.1 Inundation Mapping

Peak elevations are plotted on the inundation map showing the maximum flooding that would occur at all locations in the channel system resulting from the induced failure of Lower Baker Dam. Time of arrival of the peaks are indicated by a dotted black line perpendicular to the flow for every hour during the flood. Also shown is the time of arrival of the wave front. It is defined as the flow rate at which the flood has caused the river to rise one foot above base flow conditions. Throughout the channel system, the arrival of the wave front precedes the arrival of the zero damage flow level by less than an hour. The wave front and the zero damage level could be assumed to arrive concurrently because base conditions at the time of failure may be greater than was assumed in this study.

5.2 Failure of Upper Baker Dam Only

At normal full pool, Upper Baker Dam controls approximately 285,000 acre-feet of storage (10). The assumed discharge of 6000 cfs flowing into the reservoir was maintained throughout the failure simulation. The breach in Upper Baker Dam developed over a period of 10.2 minutes with the maximum discharge of 3.15 million cfs occurring approximately ten minutes after the failure begins. The level of the reservoir immediately upstream of the dam falls 77 feet in the first ten minutes reaching an elevation of 647 feet. However, it takes four hours for the reservoir to fall to the same elevation five miles upstream of the dam. As the water level at the dam site falls, the discharge also decreases, though it is still greater than one million cfs after 60 minutes.

A peak water surface elevation at Lower Baker Dam of 496 ft is reached 90 minutes after the formation of the breach in Upper Baker Dam. The maximum discharge into the Skagit River is nearly 560,000 cfs, resulting in a water surface level increase of 41.2 ft above base flow stage at Concrete to an elevation of 211.4 ft 3.5 hours after the formation of the breach. As the flood wave enters the Skagit River at the Baker River confluence, it moves in both upstream and downstream directions. At Van Horn, approximately 3.5 miles upstream on the Skagit, the water surface level is increased by

33.6 ft to an elevation of 213 ft 2.3 hours after formation of the breach. The peak upstream flow is 118,000 cfs 1.5 hours after formation of the breach. After the Baker River peak flow has entered the Skagit River, the upstream flow in the Skagit River reverses and resumes its flow toward the delta. The water that has been stored in the Skagit River upstream of the confluence with the Baker River creates a peak downstream flow at Van Horn of 65,300 cfs four hours after formation of the breach. The flow directions are illustrated in Figure 5.1. The Skagit River above its confluence with the Sauk River is not affected by the flood wave in this case.

Using the zero damage flood flow of 67,800 cfs (5) as an indicator of the onset of flood damage, flooding near Concrete begins about 20 minutes after the formation of the breach. Approximately 32 miles downstream at Sedro-Woolley, the front arrives after 9.8 hours with a water surface elevation of 36.3 feet. Flooding begins eleven hours after formation of the breach. The maximum flow of 217,000 cfs occurs at Sedro-Woolley 15.8 hours after formation of the breach causing a flood elevation of 17.9 ft above the base elevation.

It is assumed that river levees have failed in the Skagit River delta but that sea dikes remain intact to create the probable worst case scenario. The *Army Corps of Engineers Flood Plain Information Study* reported that the maximum flow that river levees will withstand is 143,000 cfs in Diking District 17, between Burlington and Mt. Vernon. The wave front arrives at Mt. Vernon 13.8 hours after formation of the breach. Flood damage begins 15.3 hours after formation of the breach and the maximum flow of 183,000 cfs through Mt. Vernon occurs 22.3 hours after formation of the breach. The resulting maximum water surface elevation is 28.7 ft, an increase of 6.4 ft over base flow elevation.

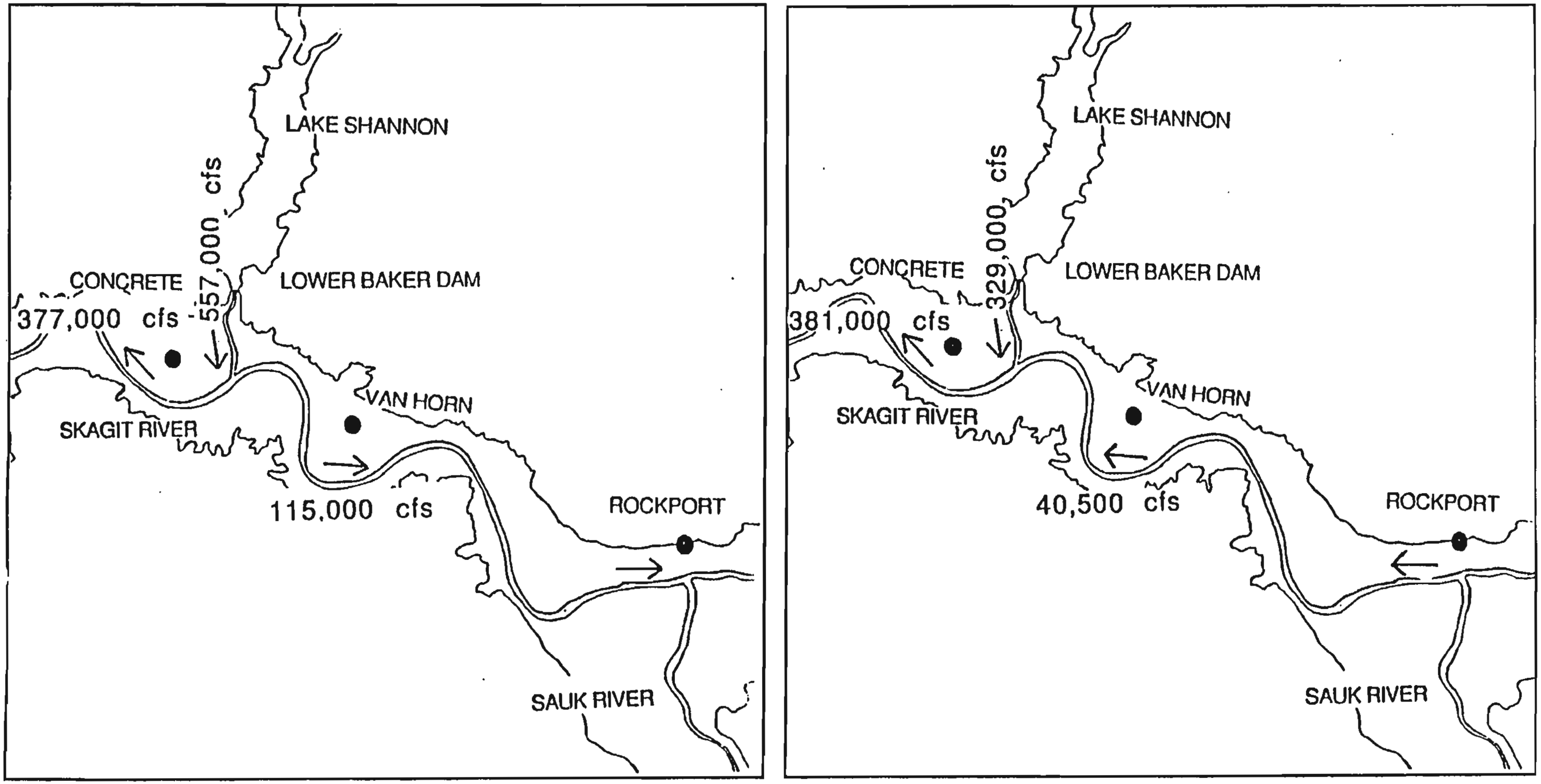
Figure 5.2 presents the hydrographs from the Upper Baker Dam breach and overtopping of Lower Baker Dam. Figure 5.3 shows hydrographs at both Concrete and Mt. Vernon. The attenuation of the peak resulting from flow through the Skagit River is demonstrated clearly in Figure 5.3.

5.3 Lower Baker Dam Failure Only

Failure of Lower Baker Dam is simulated according to FERC guidelines by removing it completely from the dam site over a period of 0.07 hours (4.2 minutes). A constant flow of 6,000 cfs was assumed for the Baker River. The water surface elevation in Lake Shannon is assumed to be 436.8 ft (normal full pool) at the time of failure. Lower Baker Dam controls a volume of 159,000 acre-feet at normal full pool elevation.

FIGURE 5.1 FAILURE OF UPPER BAKER DAM ONLY

REVERSAL OF WAVE MOVING UPSTREAM



1.75 HOURS AFTER BREACH BEGINS TO FORM

4.1 HOURS AFTER BREACH BEGINS TO FORM

FIG. 5.2 FAILURE OF UPPER BAKER DAM ONLY
Discharge at Upper and Lower Baker Dam

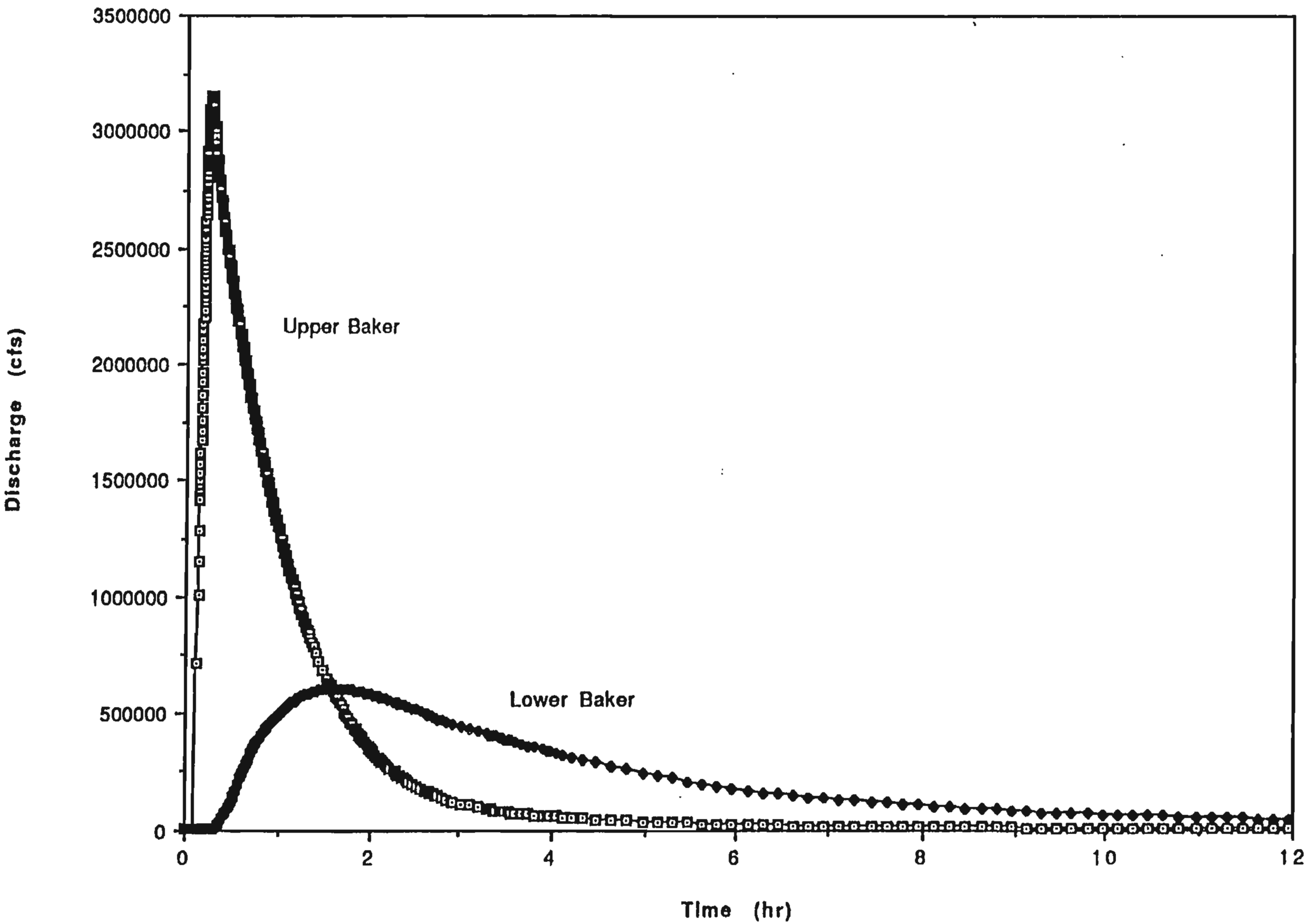
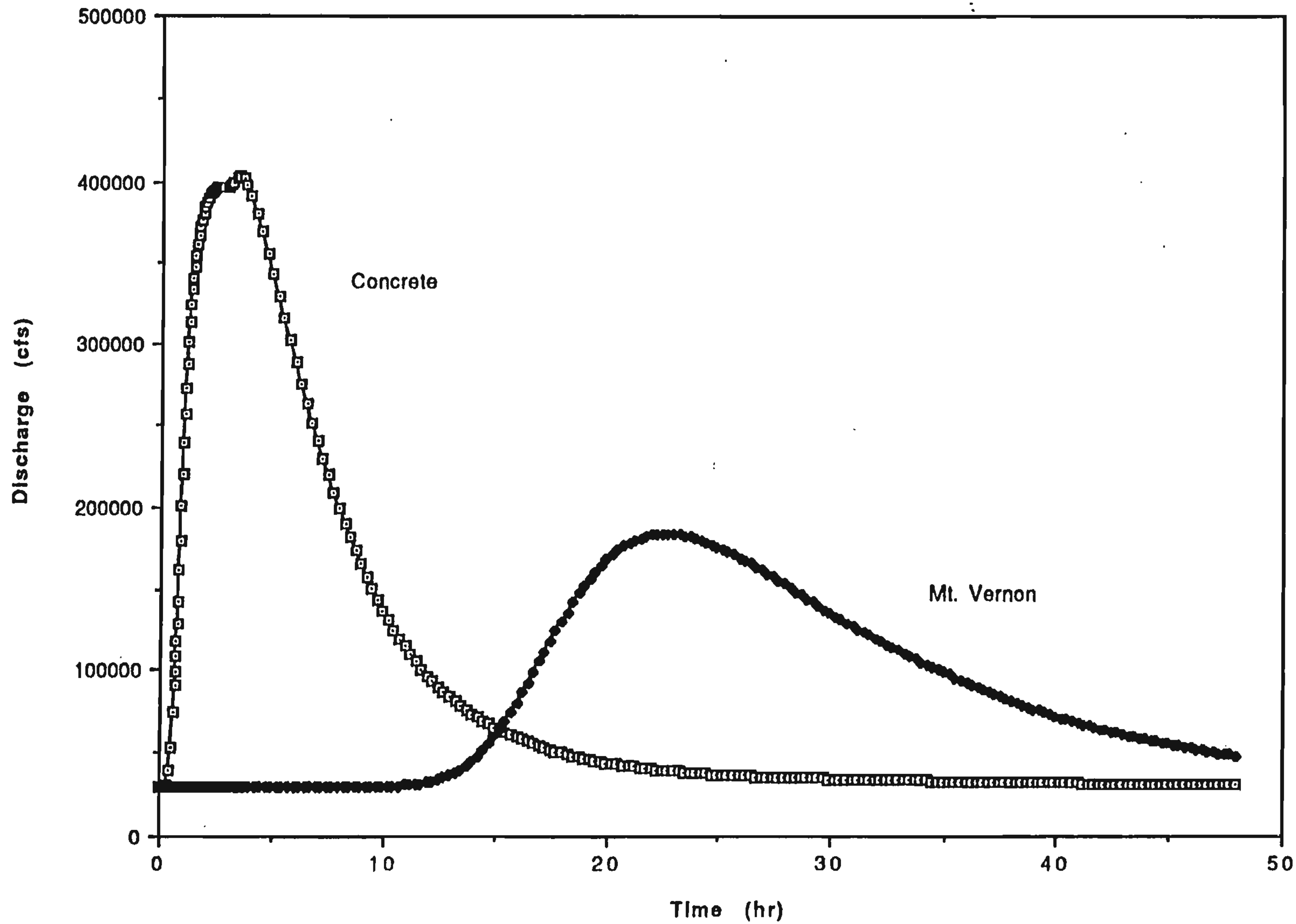


FIG. 5.3 FAILURE OF UPPER BAKER DAM ONLY
Discharge at Concrete and Mt. Vernon



Discharge from Lower Baker Dam site reaches a maximum of 1.9 million cfs six minutes after formation of the breach. The water surface elevation at this time is 369 ft at the site of the breach, however only one-half mile above the dam site, the water surface elevation is above 400 ft. Discharge at the dam site remains above one million cfs for nearly one hour after the failure. Two hours after formation of the breach the water surface elevation one-half mile above the dam site is 330 ft, and the flow out of Lake Shannon has been reduced to 130,000 cfs. Flow through the Lower Baker dam site has returned to the base flow of 6000 cfs three hours after formation of the breach and the reservoir is essentially empty.

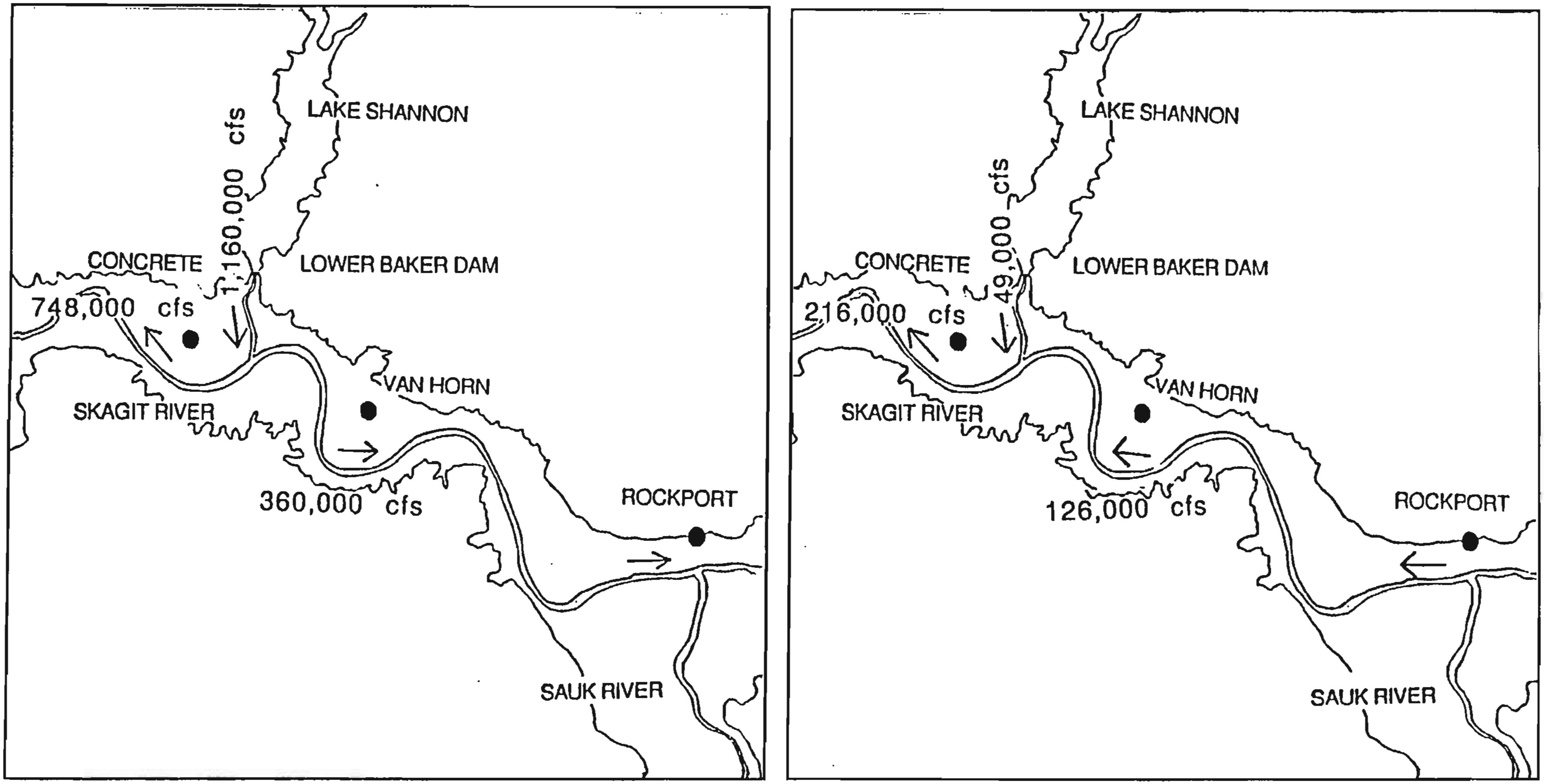
The maximum discharge to the Skagit River is nearly 900,000 cfs, resulting in an increase in water surface level of 54.8 ft at Concrete to an elevation of 225 ft 54 minutes after formation of the breach. As the flood wave enters the Skagit River at its confluence with the Baker River, the flow moves in both upstream and downstream directions. At Van Horn the water surface elevation is increased by 45.1 ft to a maximum of 224.5 ft 90 minutes after formation of the breach. The upstream flow reaches a peak of 384,000 cfs two hours after formation of the breach. The water stored upstream of Concrete creates a downstream peak flow at Van Horn of 146,000 cfs approximately three hours after formation of the breach. The stage at Rockport is unaffected by the reverse flow. Please refer to Figure 5.4 for a graphical representation of this phenomenon.

Flooding near Concrete begins about three minutes after the assumed failure of Lower Baker Dam. At Sedro-Woolley the front arrives eight hours after formation of the breach with a water surface elevation of 36.4 feet. Flooding begins 9.5 hours after formation of the breach and the maximum flow of nearly 192,000 cfs occurs at Sedro-Woolley 13.2 hours after formation of the breach. The resulting maximum water surface elevation is 51.7 ft, an increase of 16.4 ft above the base elevation. The wave front arrives at Mt. Vernon 12 hours after formation of the breach and flood damage begins at 13.7 hours after formation of the breach. The maximum flow of 155,000 cfs occurs at Mt. Vernon, 19.7 hours after formation of the breach.

Figure 5.5 represents the hydrograph at the Lower Baker Dam site and Figure 5.6 shows hydrographs at Concrete and Mt. Vernon, resulting from the flood wave created by the failure of Lower Baker Dam.

FIGURE 5.4 FAILURE OF LOWER BAKER DAM ONLY

REVERSAL OF WAVE MOVING UPSTREAM



0.8 HOURS AFTER BREACH BEGINS TO FORM

3.5 HOURS AFTER BREACH BEGINS TO FORM

FIG. 5.5 FAILURE OF LOWER BAKER DAM ONLY
Discharge from Lower Baker Dam

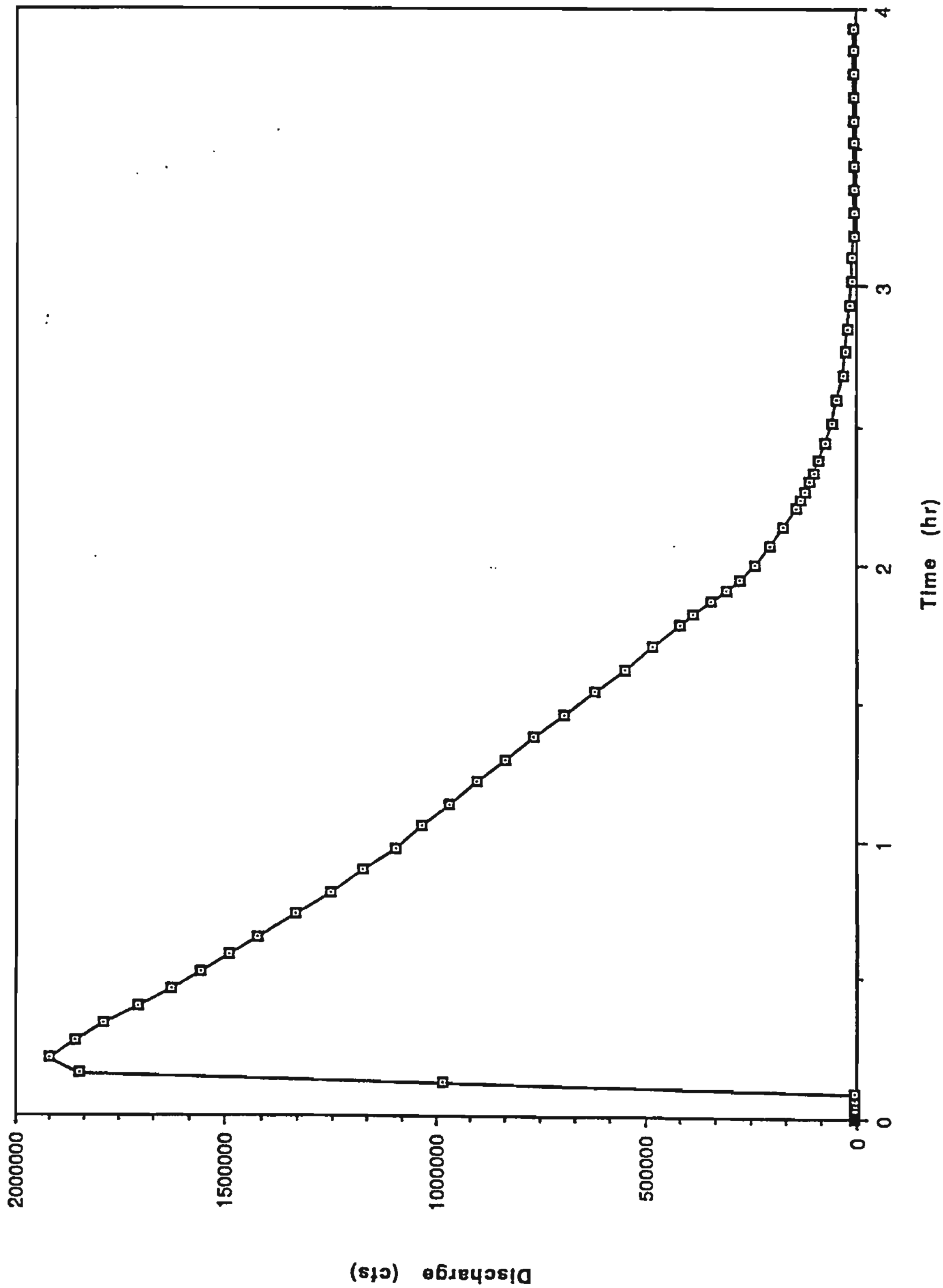
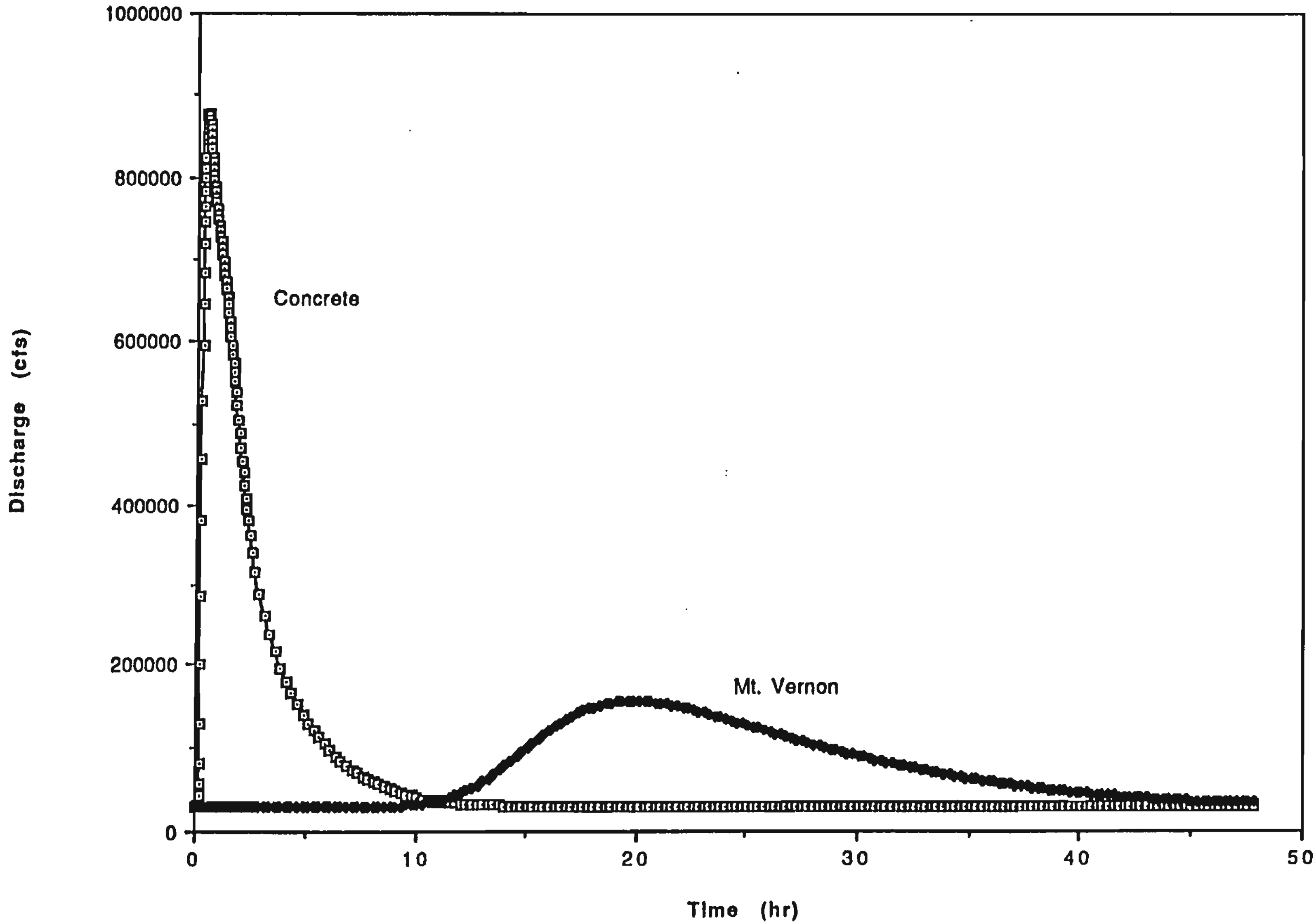


FIG. 5.6 FAILURE OF LOWER BAKER DAM ONLY
Discharge at Concrete and Mt. Vernon



5.4 Induced Failure of Lower Baker Dam Resulting from Failure of Upper Baker Dam

Lower Baker Dam is assumed to fail approximately 52 minutes after the formation of the breach in Upper Baker Dam. Lower Baker Dam is removed completely over a period of 4.2 minutes when the depth of water flowing over the crest due to the failure of Upper Baker Dam exceeds 50 ft. Within two minutes after the assumed failure of Upper Baker Dam, the discharge through the breach is over one million cfs and the water surface elevation in Lake Shannon at Lower Baker Dam is unchanged.

As described in Section 5.1, the flow through the breach in Upper Baker Dam reaches a maximum of 3.15 million cfs ten minutes after its assumed failure. At this time, the effect on Lake Shannon has not been felt at the dam site. However, three miles below Upper Baker Dam the water surface in Lake Shannon has been increased by over 13 ft. The flow through the Lower Baker Dam site reaches a maximum of 2.6 million cfs less than one hour after the assumed failure of Upper Baker Dam.

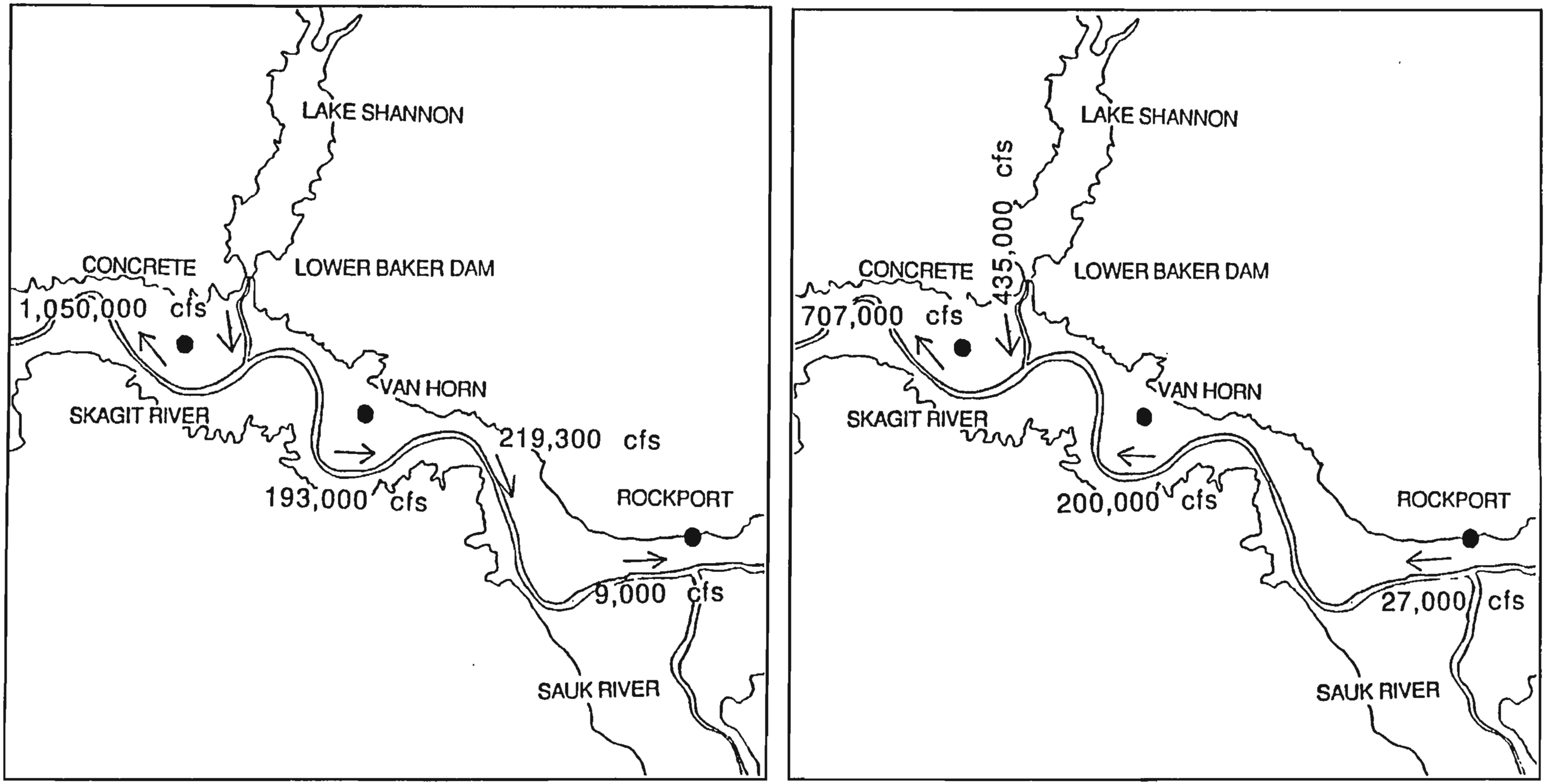
At Concrete, the wave arrives 20 minutes after the formation of the breach in Upper Baker Dam, more than 30 minutes before the induced failure of Lower Baker Dam. Flood damage begins to occur after 30 minutes and the maximum flow on the Skagit through Concrete of 1.26 million cfs occurs 40 minutes later. The water surface elevation at Concrete is increased by 69.2 ft to a maximum stage of 239.4 feet.

A maximum flow of 2.2 million cfs enters the Skagit River from the Baker River, again causing reverse flow in the Skagit River upstream toward Rockport. An upstream flow of 626,000 cfs is experienced at Van Horn after 1.5 hours. The water surface elevation is increased by 61 ft to a maximum elevation of 240.4 ft at Van Horn after 2.4 hours. The wave front arrives in Rockport only 2.7 hours after the assumed failure of Upper Baker Dam. The maximum upstream flow at Rockport of 44,800 cfs occurs 3.5 hours after formation of the breach in Upper Baker Dam. Ninety minutes later the flow has reversed, releasing water stored upstream. A peak downstream flow of 30,400 cfs results at Rockport. A graphical presentation of the flow reversal is shown in Figure 5.7. Figure 5.8 shows a hydrograph of the flood wave at Van Horn. The initial negative flow is upstream flow. When the flow crosses the "zero line", it reverses and then flows downstream.

The maximum flow at Birdsview is nearly one million cfs and occurs 3.5 hours after the assumed failure of Upper Baker Dam. The arrival of the front occurs at Birdsview 36 minutes after formation of the breach in Upper Baker Dam and flood damage begins to

FIGURE 5.7 INDUCED FAILURE OF LOWER BAKER DAM

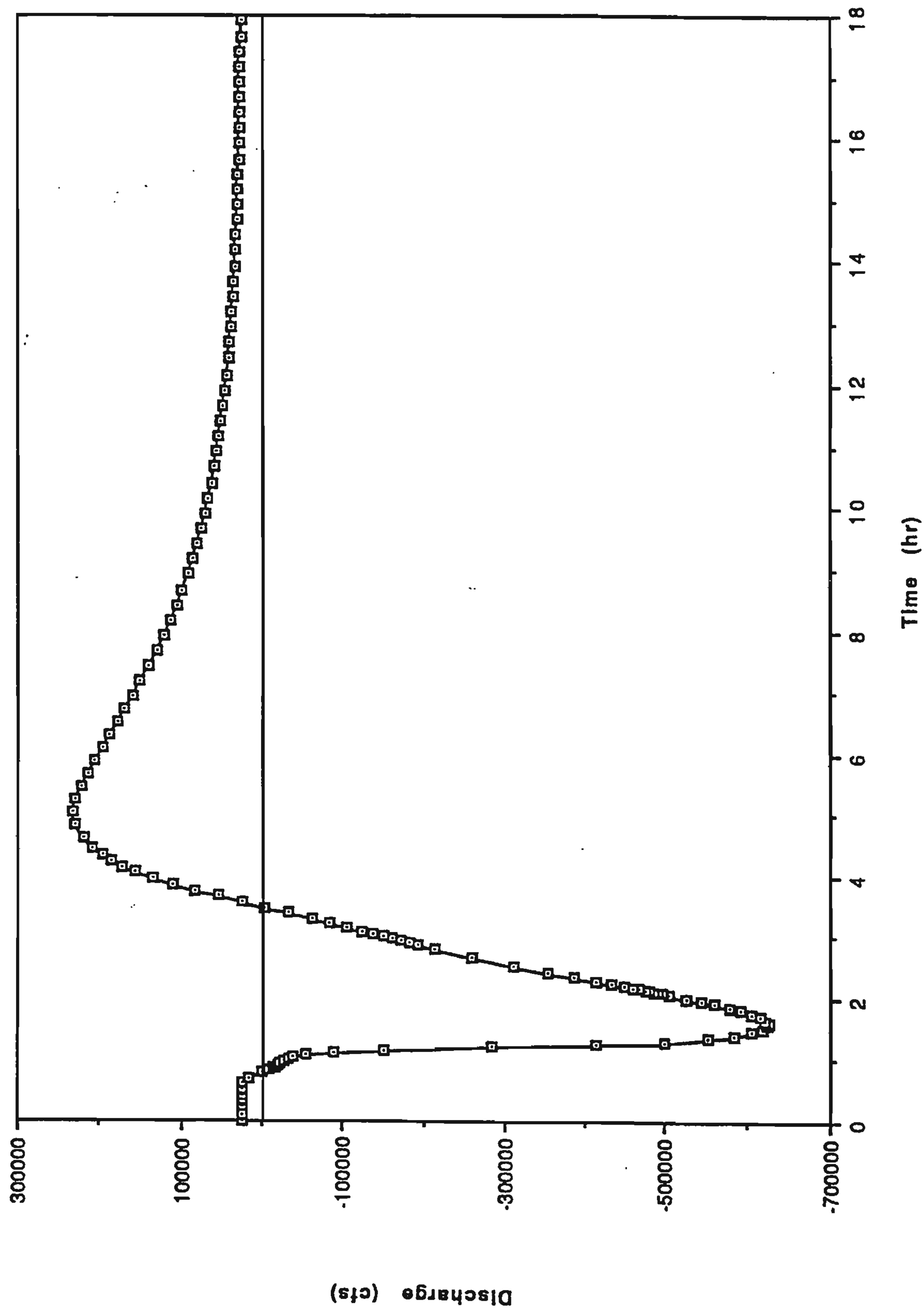
REVERSAL OF WAVE MOVING UPSTREAM



2.8 HOURS AFTER BREACH BEGINS TO FORM

4.3 HOURS AFTER BREACH BEGINS TO FORM

FIG. 5.8 INDUCED FAILURE OF LOWER BAKER DAM
Floodwave at Van Horn



occur 72 minutes later. The maximum flow at Hamilton is 900,000 cfs and it occurs nearly five hours after the formation of the breach in Upper Baker Dam.

The wave front arrives at Sedro-Woolley 7.5 hours after formation of the breach inUpper Baker Dam. Flood damage at Sedro-Woolley starts about 30 minutes later and the maximum flow of 446,000 cfs arrives 4 hours after damage begins. The water surface level is increases by 26.6 ft above base flow stage to an elevation of 61.9 ft.

The wave front arrives at Mt. Vernon 10.5 hours after formation of the breach in Upper Baker Dam failure followed by flood damage flow one hour later. The peak of 362,000 cfs occurs in Mt. Vernon 17.3 hours after formation of the breach in Upper Baker Dam. The maximum water surface elevation at Mt. Vernon of 32.3 ft indicates a water surface level of over 11 feet above base elevation. Complete inundation of the Skagit delta results. The inundation of the Skagit River Valley from Rockport to Puget Sound is indicated on the inundation map.

Figure 5.9 shows hydrographs for the Upper Baker Dam breach and the subsequent failure of Lower Baker Dam. Figure 5.10 shows hydrographs for Skagit River flow through Concrete and Mt. Vernon.

FIG. 5.9 INDUCED FAILURE OF LOWER BAKER DAM
Discharge at Upper and Lower Baker Dam

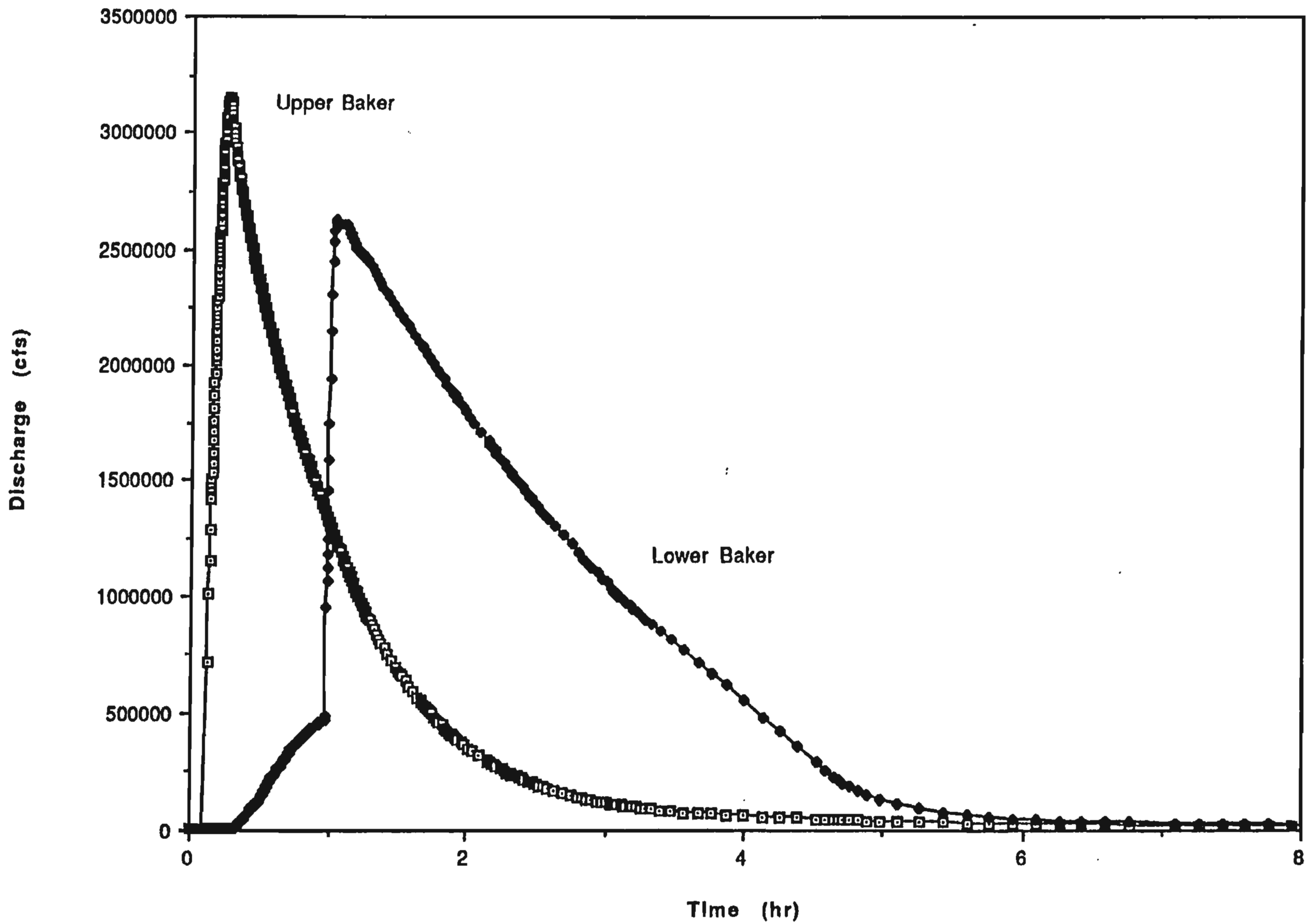
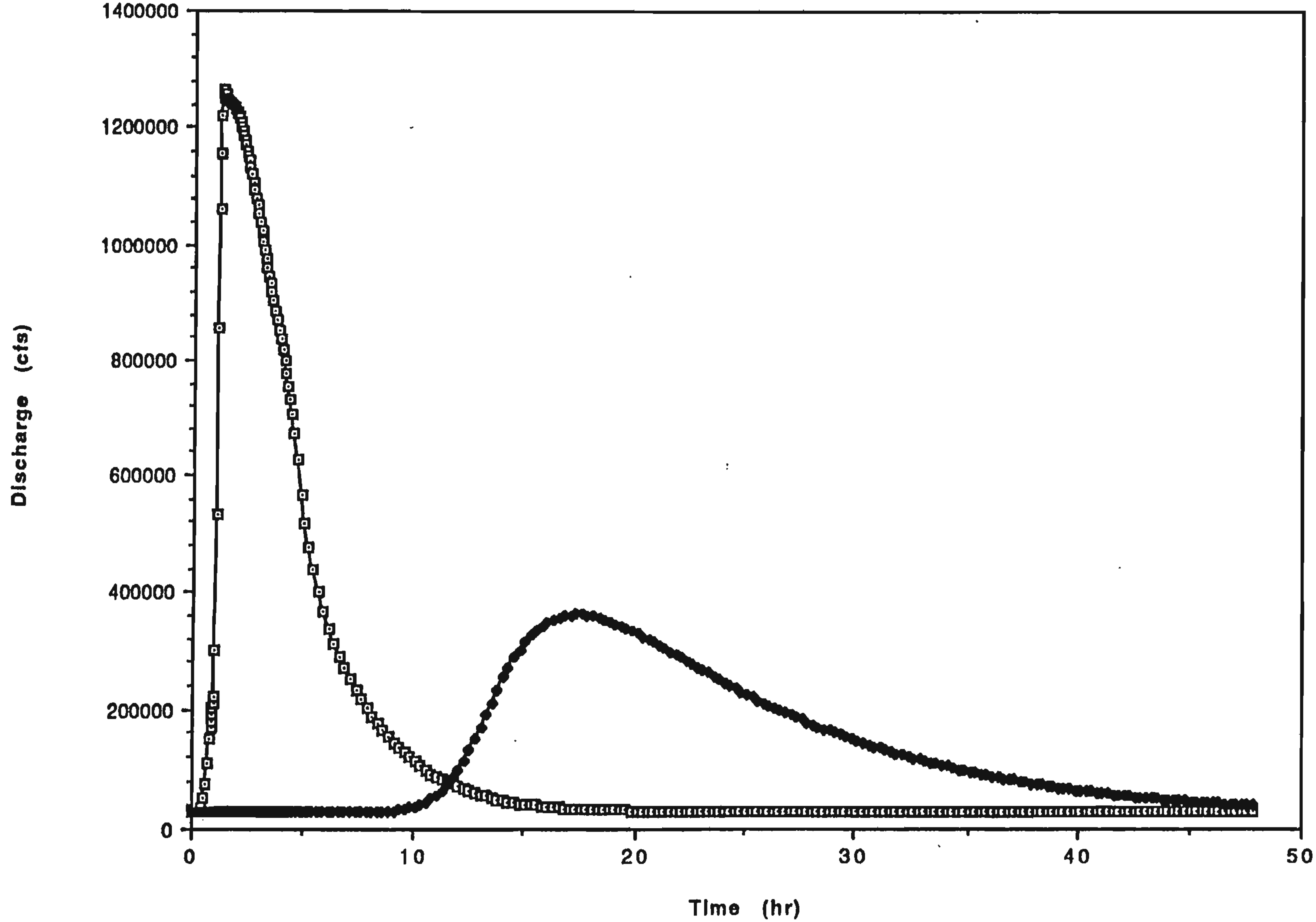


FIG. 5.10 INDUCED FAILURE OF LOWER BAKER DAM
Discharge at Concrete and Mt. Vernon



6.0 CONCLUSIONS

The three failure modes simulated produced peak discharges and water surface elevations high enough to cause major damage to life and property all along the Skagit River. Peak flows, elevations, and times of arrival for three locations are tabulated in Table 6.1. Also included in table 6.1 are water surface elevations and times of arrival for the wave front and zero damage flow. A flow of 67,800 cfs was used to define zero damage flood flow at all points on the Skagit River.

The worst flooding was produced by the failure of Lower Baker Dam by overtopping induced by the failure of Upper Baker Dam. The results of this failure mode were used to prepare the inundation map.

TABLE 6.1 SUMMARY OF STUDY RESULTS AT SELECTED LOCATIONS

FAILURE MODE	ARRIVAL OF THE WAVE FRONT: Water Surface Elevation (ft)	ARRIVAL OF THE WAVE FRONT: Time of Arrival (hrs)	ARRIVAL OF ZERO DAMAGE FLOOD FLOW: Water Surface Elevation (ft)	ARRIVAL OF ZERO DAMAGE FLOOD FLOW: Time of Arrival (hrs)	ARRIVAL OF THE PEAK FLOW: Peak Flow (cfs)	ARRIVAL OF THE PEAK FLOW: Water Surface Elevation (ft)	ARRIVAL OF THE PEAK FLOW: Time of Arrival (hrs)
Upper Baker Only							
Concrete	171.2	0.3	176.7	0.5	403,000	211.4	3.4
Sedro-Wooley	36.3	9.8	40.9	11.3	217,000	53.1	15.8
Mt. Vernon	22.3	13.8	23.9	15.3	183,000	28.7	22.3
Lower Baker Only							
Concrete	171.2	0.1	174.1	0.1	878,000	224.9	0.4
Sedro-Wooley	36.3	8.0	40.8	9.5	192,000	51.7	13.2
Mt. Vernon	22.3	12.0	23.9	13.7	155,000	27.9	19.7
Induced Failure of Lower Baker							
Concrete	171.2	0.3	176.7	0.5	1,263,000	239.4	3.0
Sedro-Wooley	36.3	7.6	40.4	8.1	446,000	61.9	12.0
Mt. Vernon	22.3	10.5	23.7	11.5	363,000	32.3	17.2

7.0 REFERENCES

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APPENDIX A

INTRODUCTION TO UNSTEADY FLOW

An unsteady flow is any flow in which some aspect of the flow is changing with time. We restrict our attention to flows with a free surface taking place in open channels such as canals, rivers, streams, brooks, sewers, pipes, etc. Many of these flows can be analyzed successfully by treating the flow as if it were one dimensional. If the flow is one dimensional, then only acceleration along the channel length(longitudinal acceleration) is important. Vertical and lateral accelerations are small and are ignored. The movement of water in this case is often described as a shallow-water wave meaning that the wave length of a disturbance is much larger than the depth of water. In contrast a deep-water wave has a wave length much smaller than the depth. In the case of a shallow-water wave, the disturbance reaches the bottom of the channel, whereas in the deep-water wave the disturbance is restricted to the near surface region. Waves of either type may occur in water of any reasonable depth.

Most real flows are unsteady. However, most flow analyses are made assuming steady flow. Practically, many real flows can be approximated by steady flow especially in those cases where the goal is to maintain essentially steady flow for long periods of time such as in irrigation canals. However, the principle reason for the predominance of steady flow analysis is that it is simpler to understand and it requires less computational power. As a practical matter, the computations for unsteady flow were too expensive or impossible to do by hand computations except for some simplified cases. Clearly widespread use of unsteady flow analysis could not begin until the power of the modern digital computer became available.

A classification scheme for both steady and unsteady flow is useful in describing the flows of interest to us. The simplest steady flow is uniform flow in which no flow variable changes with distance. In a uniform steady flow every flow variable is a constant with respect to both distance and time. If the flow is not uniform then it is non-uniform. In this case the flow is further divided into gradually varied or rapidly varied flow. Gradually varied steady flow may have variations in the flow variables with distance but all variables are constant in time. Furthermore the variations with distance are gradual as the name implies. The series of backwater profiles discussed in the typical open channel hydraulics course are all gradually varied flows. Finally the flow may be rapidly varied which means that there are significant variations in flow in other than the longitudinal direction. An extreme example is a hydraulic jump below a dam. This flow can still be analyzed using the assumption of one dimensional flow but such an analysis must recognize and isolate the rapidly varied flow portion. Additional examples of rapidly varied flow include flows through culverts and bridges as well as flow over weirs and spillways.

If the flow is unsteady, uniform flow is impossible so that only non-uniform flow exists for unsteady flow. Both gradually varied and rapidly varied flow exist and the same general rules for analysis apply. We must isolate the regions of rapidly varied flow before we can analyze then using the one-dimensional flow assumption. As a result the method of analysis of steady and unsteady flow is the same in this respect. There are many other similarities between the analyzes of these flows. Because steady flow is much more widely used and because I assume you are familiar with steady flow, I will use it as a springboard to introduce some aspects of the analysis of unsteady flow.

Comparison of Steady and Unsteady Flow Analysis

In steady flow we have a governing equation which describes how the flow varies. This is most often written as an energy equation but a momentum equation can be used as well. In either case, this equation is a differential or integral equation which has a solution but that solution cannot be found without recourse to numerical methods. We can only obtain an approximate solution to the governing equation as a result. To find this solution we subdivide the channel into short pieces. Then for each short piece we approximate the differential or integral terms in the governing equations algebraically and obtain an algebraic equation which approximates the governing equation in each of the short channel pieces. The channel pieces I will call computational elements. They are called computational because they have been introduced for purely computational reasons and they are called elements because from them the whole solution scheme proceeds. Therefore they are truly elemental.

The ends of the computational element, called nodes, are defined by cross sections either measured or estimated from measurements. The cross section is taken at right angles to the direction of flow in so far as we can predict the direction of flow. One way to visualize a computational element is as a slice of the channel taken at right angles to the longitudinal axis of the channel. Adjacent computational elements have a cross section and therefore a node in common. As a result a length of channel divided into 10 computational elements will have 11 nodes. In general there will always be one more node(cross section) than the number of computational elements.

The values of interest in the cross sections at the end of the computational element, also called elements, in this case cross sectional elements, are the the width of the water surface, the flow area, the first moment of area, the conveyance, and perhaps an energy or momentum flux coefficient all computed at any given elevation. Please note that the governing equation does not explicitly know anything of the shape of the cross section beyond the information on shape reflected in these cross sectional elements. This is why they are called cross sectional elements because they convey all the information that the governing equation can comprehend about the cross section. Therefore they are truly elemental.

In steady flows, except for the case of flow over a side weir, the flow is known at all points in the channel. Now we have been careful to write the algebraic approximations such that they involve values at the ends of the computational element only. Consequently we have two unknowns in each computational element: the elevation of the water surface at each end. If we are given an initial elevation we can compute the unknown elevation along the channel in a sequential manner and we never have to solve for more than one unknown at a time. The direction of solution must be from a point of known or assumed elevation to points of unknown elevation. In general if the flow is subcritical the direction of solution is upstream and if the flow is supercritical the direction of solution is downstream.

An important factor in a steady flow analysis is determining points where the elevation can be computed once the steady flow is selected. These points, of known or knowable relationship between flow and elevation, are called control points. Consequently in a steady flow analysis one of the first things done is to determine the location and nature of the control points. Additionally the algebraic governing equation for steady flow does not apply to rapidly varied flow which occurs at bridges, culverts, falls, rapids, dams, etc. Furthermore it does not apply to junctions of two or more channels or to abrupt expansions or contractions in channel size or shape. All of these features must be isolated and analyzed using equations other than the equations used for each computational element.

Another feature of steady flow analysis which is important is that our first selection of the length of one or more computational elements may be too long. We may find that the non-linear equation which we must solve in each computational element has no solution. The only recourse is to subdivide the computational element into two or more shorter computational elements. We also may find that occasionally our solution process may find an elevation such that the flow is supercritical at the upstream end of the computational element when we started with an elevation at the downstream end at which the flow was subcritical. This is incorrect! In steady flow, such a pattern indicates that there is a hydraulic jump present at some point in the computational element. Several possibilities present themselves to the analyst at this point in a steady flow analysis. First, the incorrect result may be purely a computational artifact and results only from the failure of the solution process to find the subcritical solution at the upstream end of the computational element. The solution process should be changed to always seek for a subcritical solution so that this case should not arise frequently. Second, there may not be a subcritical solution to the unknown in the computational element. This can also be a computational artifact because the computational element is too long so that the errors in our approximation of the differential or integral terms are falsifying the solution. We must then subdivide the computational element and try the solution again. The final possibility is that the flow is physically supercritical near the computational element yielding the invalid solution. If this is the case, then we must find the control point for the supercritical flow and compute that profile downstream from the control point and locate the hydraulic jump by computing another profile upstream from the subcritical control point until the conditions for a jump are found. This process normally requires several trial and error attempts.

A final feature of steady flow is that we must provide the hydraulics of the special features such as junctions, bridges, etc. These are not adequately described by the algebraic governing equation used for the computational elements along the channel. Appropriate assumptions must be made to develop equations relating flows and elevations at junctions, bridges, dams, etc. It is then possible for us to compute the hydraulic effects so that we can find the initial conditions for the channels upstream of these special features. Of course, there must be at least one point at which we know the elevation to start the computations. This point, in subcritical flow will be at the downstream end of the stream system we are analyzing. It is at the downstream boundary of the region of interest to us and is called an initial condition.

This outline of a steady flow analysis provides the background for introducing some concepts of unsteady flow analysis. We can expect some similarities because steady flow is a special case of unsteady flow. There will be some differences also because unsteady flow must describe conditions not comprehended by the steady flow governing equations.

In unsteady flow we need two governing equations because both the flow and the elevation of the water surface are unknown. One of the governing equations is based on the conservation of water volume and the other is based on the conservation of water momentum. In steady flow the conservation of water volume was trivial because the flows were constant and we used it to solve for the flows everywhere *without* having to know the elevations *anywhere*! We can no longer do this in unsteady flow. Therefore the conservation of water volume becomes a governing equation involving both flows and elevations.

We again must use computational elements in the unsteady flow analysis and we use algebraic approximations to the differential or integral terms involved in the governing equations to develop two algebraic equations for each computational element written in terms of elevations and flows at the ends of the element. But these governing equations are not only more in number they are more complex. In the unsteady flow case we must also introduce a "computational element" with respect to time. This was not needed in steady flow because no value changed with time. Fortunately, the "computational element" with respect to time is very simple. We merely divide the time axis into finite time increments which we hope will be short enough so that the algebraic approximations to the differential and integral terms will be sufficiently accurate. As a result of this dependence on time the algebraic governing equation involve not only the unknown flow and elevation at two points along the channel but also at two points in time.

We need to identify control points of known relationship between elevation and flow as well as all the other points of rapidly varied flow or of interaction between channels not described by the algebraic governing equations. Just as in steady flow, these points establish the limits of applicability of the governing equations with respect to distance along the channel as well as providing known values for the analysis. However, in unsteady flow we have an additional requirement in that we must have some starting time to begin the computations. We must start at some point in time and we must know all the flow values at this point in time! That at first sight sounds impossible.

How will we ever start if we need to know the flows and elevations at a time point and that is just the purpose of the unsteady flow analysis. We need to make some assumption to break the circle so that we can start. The way we do this is that we assume that the flow is steady at the starting time everywhere in the system. This is the first major difference between steady flow and unsteady flow: we must complete a steady flow analysis to establish the initial condition for the unsteady flow analysis.

A second major difference between unsteady flow analysis and steady flow is the nature of the information we need at the boundaries of the stream system. In steady flow we need only know one elevation at the downstream boundary to start the computations. However, a cursory analysis of the number of equations we have available so far in unsteady flow shows that we will need more information for unsteady flow analysis. Let us assume that we have a single channel with no special features. We divide this channel into 9 computational elements which yields 10 nodes. With 2 unknowns at each node we have 20 unknowns but only 18 equations(2 per computational element). Thus we cannot possibly solve for the unknowns unless we have some additional information. We need additional information at the boundaries of the system we are analyzing. When the flow is subcritical, we need information at both the upstream and downstream boundary of the system. This information can be in one of three forms: flow known as a function of time, elevation known as a function of time, or a relationship between flow and elevation. The upstream boundary will often be flow known as a function of time and the downstream boundary will often be a known relationship between flow and elevation. The information we supply at a boundary is called a boundary condition.

The information supplied at special features internal to the system is often called an internal boundary condition. In unsteady flow these are approximated as steady flow relationships because the special features are usually sufficiently short so that the changes in momentum and volume of the water within the special features are very small. We will discover as we continue in the study of unsteady flow analysis that the isolation and description of the special features is a major component of the analysis.

Major differences between the two analyses are not in the hydraulic geometry because that can be identical but are in the number and complexity of the governing equations, the initial conditions required, and the addition of boundary conditions. The same problems can arise computationally because both use algebraic approximations to the differential and integral terms. These approximations are developed for the computational element of finite length. The computational element may be too long. The difference is that in unsteady flow analysis the computational problems are both more complex and more frequent. The increased frequency comes about primarily because unsteady flow analysis involves computations over a wide range of elevation whereas most steady flow analysis involves computations over a narrow range of elevations. Furthermore, the time dimension has been added and this brings its own set of problems. We will discuss some of these problems and their solution as we go through the course.

Major Assumptions in Unsteady Flow Analysis

In order to complete an analysis of flow in open channels we must make many assumptions. Some of the major ones are:

1. The wave length of the disturbance is very long relative to the depth of the flow. This is the shallow-water wave assumption. This implies that the pressure distribution does not deviate significantly from hydrostatic. This also implies that lateral and vertical accelerations are insignificant compared to the longitudinal accelerations.
2. The channel geometry is fixed so that effects of deposition or scour of sediment is insignificant.
3. The bed of the channel has only a small slope so that the tangent and sine of an angle and the angle are nearly the same magnitude.

4. The effect of friction can be estimated by using a relationship derived from steady uniform flow. We assume that non-uniformity and unsteadiness have an insignificant effect on the friction losses.
5. The water surface in any cross section of the stream is assumed to be horizontal. This cannot be true in many cases so we assume again that the deviations from horizontal have only a small effect on the results.
6. We assume that the effects of variations in velocity across a cross section can be approximated by using flux correction coefficients which are functions of water surface elevation only.

Using these assumptions we can develop formal statements of the conservation of water volume(mass) and conservation of water momentum. We will develop these mathematical statements later in the course. For now I want to concentrate on some basic concepts so that the mathematics presented later will be able to build on concepts with a minimum of confusion.

The conservation of volume(mass) principle relates to flows and changes in the amount of water stored in the channels and reservoirs. It is not concerned with forces of any kind. On the other hand the conservation of momentum principle relates forces, momentum fluxes, and the momentum of water in storage. The factors involved in this equation are:

1. Gravity force on the water in the channel.
2. Friction force on the wetted perimeter of the channel.
3. Pressure force on the boundaries.
4. Inertia of the water.

Some of these factors can be or are ignored to simplify the unsteady flow computations. If all these factors are include we have what are called the complete, full, dynamic, St. Venant, or shallow-water equations. Various names are used but they refer to equations in which all of these factors are included in the analysis. If the inertia of the water is ignored we have what is called the zero-inertia form of the governing equation. If the variations of pressure force along the channel are ignored because they are thought to be small we obtain what is called the kinematic form of the governing equation. Reservoir routing is also a form of unsteady flow analysis in which the governing equation degenerates to a simple relationship between water surface elevation and the flow. In a certain sense reservoir routing ignores all four factors even though some or all are implicit in the relationship between flow and water surface elevation. In each case one of the factors is dropped from the momentum equation and to be precise we should call the governing equation a motion equation. The unsteady flow analysis program to be discussed here, called FEQ(Full EQuations), includes three of the four forms of governing equation for unsteady flow: (1) the so-called full equation form involving all four factors, (2) the zero-inertia form in which the inertia of the water is ignored, and (3) the reservoir routing form in which the motion equation is reduced to a relationship between water surface elevation and flow.

Unsteady Flow Analysis Examples

There are many possible examples of unsteady flow analysis. I mention only a few here.

1. Passage of a Flood Wave. In flood insurance studies the presumed maximum elevation envelope caused by a flood wave is computed assuming steady flow. Of course the flow is really unsteady and this is only an approximation and little work has been done to evaluate the accuracy of the approximation. The effect of flood plain filling and obstruction is also often analyzed using steady flow. Steady flow analysis can only evaluate the effects of changes in the ability of the stream to convey water and does not reflect the effects of changes in the ability of the stream to store water. This latter change may be large in some cases.
2. Operation of Irrigation and Power Canals. Unsteady flow analysis is required to properly design these canals because the flow variations can often be abrupt. Allowance must be made

for the wave heights which might result. Furthermore, the time of travel of transients becomes important in the design and operation of structures intended to reduce or control transients.

3. Tidal Influence. Analysis of the influence of tides on streams requires unsteady flow analysis. Steady flow analysis is often used to approximate the envelope of maximum elevations but again little work has been reported on the accuracy of this approximation.
4. Junctions. The complex interactions at junctions among streams often requires unsteady flow analysis. For example, a large flood or failure of a dam on a tributary stream to a second larger stream can sometimes result in upstream flow at the junction in the receiving stream. This can lead to a very rapid rise in water surface elevation because not only does the influx of water serve as a temporary dam it also provides another source of inflow.

World View of FEQ

We have already introduced part of the world view of FEQ without giving it a name. What is a world view and why use the term? A world view is simply an explicit description of how the software looks at the stream system and how it describes it. The concept of a world view is important because unless we know the world view of a piece of software or of an analysis we do not know what the software can do and if it is able to give us answers to our questions. The world view tries to define which systems can be described meaningfully in general terms without describing any particular stream system. As a result some of the concepts are a bit abstract without examples. Examples will be presented as soon as enough of the world view is presented to start doing so.

We are interested in describing stream channels, reservoirs, lakes, bridges, weirs, and any other aspect of an open channel flow system which has an influence on the flow and its elevation. We must pick names for things so we can talk about them. We must also be able to describe these things quantitatively. We have already talked about the stream channels and the special features where the stream channels interact with other channels or with other physical structure.

Branches

I call the length of channel between special features a branch. The key idea of a branch is that its flow is described by the governing equations and in this sense every branch is identical. The branch has many related concepts which we must define. A branch is subdivided into computational elements for purposes of developing the approximate algebraic governing equations. A branch also has nodes at the boundaries of these computational elements and each node on the branch has a cross section associated with it. The nodes at the two ends of the branch are called exterior nodes because they are used to describe interactions between the branch and the world exterior to the branch. The nodes on the branch not on the ends are called interior nodes. The branch has an upstream end and a downstream end which the user must assign. For example, the exterior node at the upstream end is called the upstream exterior node. The nodes on a branch are numbered for identification with the numbers increasing and consecutive from the upstream end to the downstream end. Each node on the branch has a station associated with it where the station is the distance measured along the stream from some convenient reference point. Each node on a branch also has associated with it the elevation of the minimum point in the cross section at that node. The station-elevation pair for each node defines the bottom profile of the stream channel. The absolute value of the difference in the stations of two consecutive nodes on a branch gives the length of the computational element between the two nodes.

There are only three ways for water to enter branch: inflow at the ends or inflow from the area tributary to the branch. Thus a branch and also a computational element may have a tributary area associated with it also. The tributary area for a computational element is that area which will contribute inflow to the computational element which enters by some means other than at the ends of the channel. Generically this inflow is referred to as lateral inflow but it might enter at only selected inflow points if a storm sewer is involved. Most often the lateral inflow from a tributary area is estimated using a hydrologic model which produces unit values of runoff for one or more land uses. For example, the area tributary to a computational element might consist of agricultural land, forested land, and part of a city. Each of these land uses or cover types would have a different rainfall-runoff relationship in the hydrologic model. It is therefore convenient to allow the subdivision of the tributary area into the different cover types used in the hydrologic model.

An additional concept related to the estimation of lateral inflow to the computational element is the concept of a gage. The gage refers to the precipitation gage used to define the rainfall from which the runoff was computed. More than one rainfall gage may be available in a watershed being analyzed. In order to allow this variability in the operation of FEQ the tributary area for each computational element must be associated with the gage used to compute the unit area runoff for that tributary area.

Special Features

Recall that a branch was defined as the length of stream between special features. We now must come to grips with special features. They can be grouped into several classes:

1. Junctions. These are locations where two or more channels meet and combine to form a single channel. This could also be the case were a single channel splits and forms two or more channels.
2. Boundary conditions at the boundaries of the system under analysis.
3. Control structures. This is any physical feature which exerts a measure of control or influence on the flow. The control might be complete so that a unique relationship between flow and water surface elevation is established by the structure. In that case we call the structure a one-node control structure because only the value of flow or elevation need be known at one exterior node to fully define the other value. The control might be incomplete in that we need to know the water surface elevation at two exterior nodes before the flow is defined. Such control structures are called two-node control structures. As you will see a major challenge of unsteady flow is often the identification of the control structures and their description.
4. Level-Pool Reservoirs. A level-pool reservoir is a reservoir of such size and shape relative to the flows through it that its surface departs insignificantly from the horizontal. Level-pool reservoirs are often configured such that meaningful cross sections for one-dimensional flow analysis cannot be defined. These reservoirs are considered special features because they can be analyzed using governing equations far simpler than those used for branches. Long and narrow reservoirs or lakes are often not level-pool and should be treated as a branch.

Functions and Function Tables

One more general concept in the world view of FEQ is the concept of a function and of a function table. A function is a mapping from one set of numbers called the domain of the function to another set of numbers called the range of the function. The mapping must be such that for any number in the domain there is only one number in the range. This assures that the function will be single valued. This definition is rather abstract but it is the basis for the traditional function idea that most engineers and scientists have. We tend to think of some mathematical expression with some variable, say x denoting the domain of the function, and some other variable, say y, denoting the range of the function. The variable denoting the domain is called the argument of the function. An example is the square root function defined for all positive real numbers. If we are given a positive number we can compute the square root which is either a positive or negative number. To make this function follow the rule we must always specify in advance which of the two values we have in mind. The square root function on calculators and in computer software gives the positive value. An even simpler function is $y = 3x$ which indicates that each value in the domain, x, is to be multiplied by 3 to yield the corresponding value in the range, y.

Enough of formal details and on to examples from open channel flow. An important example is the top width function for a cross section. The top width for a cross section is the width of the water surface at any elevation in the cross section from the minimum point to some user established maximum point. Given a valid elevation the top width function will return a single value of topwidth. There are many other functions associated with a cross section. These include the area function, conveyance function, etc. In fact, we view any element of the cross section as a function because these are the aspects of a cross section used by the governing equations.

There are other functions of interest to us. These include stage-discharge relationships at gaging stations, head-discharge relationships for a wide variety of physical features, elevation-area-storage relationships for reservoirs, inflow hydrographs, etc. A major effort in any analysis is to define these functions for the stream system. Hundreds of functions may be involved in even a modest sized stream system.

Most of these functions are not known as a simple mathematical expression and we are faced with describing a wide variety of functions in some consistent manner which is both flexible and convenient. To do this I decided to use function tables for nearly all functions encountered in unsteady flow analysis. A function table consists of a set of selected argument values(the tabulated argument set) and the corresponding set of function values, and a rule for defining the function values for arguments not in the tabulated argument set. This approach was taken because most functions of interest to us are only known approximately, and we can allow some error in the function value in the rule used to compute the values not found in the table. Consequently the elements of the cross sections needed by FEQ are computed by a utility program called FEQUTL and placed in specially designed function tables called cross section tables. Thus FEQ does not know anything about the cross section unless it is reflected in the cross section function table.

This is another major difference between steady and unsteady flow analysis. Many steady flow programs compute the elements of the cross section from the definition of the cross section. The cross section is normally defined by giving the location of selected points on the periphery of the cross section in some convenient co-ordinate system and then assuming that adjacent points may be connected with straight lines without introducing significant error. In fact the points should be measured in the field or taken from a topographic with this assumption in mind. The cross section may be subdivided by vertical frictionless fictional walls to account for the problems that the hydraulic radius concept encounters in describing the shape of the cross section when computing the conveyance. Each subdivision may also have a separate value of Manning's n to account for variations in roughness along the periphery of the cross section.

The approach of computing the elements as required from the fundamental or raw cross section is no longer efficient for unsteady flow. The cross sectional element values need be computed only a few times for each steady flow analysis. This is in stark contrast to the needs of unsteady flow analysis. The cross sectional element values may be needed many thousands of times for each analysis. Therefore it is economical of comuputer time to compute the cross sectional elements and place them in a well designed cross section table for later access.

Exterior Nodes

Exterior nodes have already be introduced as the nodes on a branch at the ends of the branch. In simple stream systems these nodes are all that are needed to specify the system completely. However, in more complex systems we need to introduce additional nodes which are not on a branch. The first such node is a level-pool reservoir node or more concisely a reservoir node. We do not associate a cross section with this node because no cross section is defined for level-pool reservoirs. However, we do associate an analogous entity, the elevation-area-storage relationship for the reservoir with the reservoir node. We visualize the reservoir node as being at the downstream end of a degenerate branch of zero length. As a result the reservoir node inherits the characteristics of the downstream exterior node on a branch when it is involved in a special feature such as a junction.

The second kind of exterior node not on a branch which we need in some cases is a free node. This is an exterior node not associated with either a cross section or an elevation-area-storage relationship but is associated with a flow path for which we wish to identify the flow and we either do not wish to or cannot define the flow path as a branch or a level-pool reservoir. Examples for free nodes cannot be given until we have discussed simpler examples of flow systems. The best way to visualize a free node is as a branch of zero length with an undefined cross section but yet having an upstream and a downstream end or face. The user must decide on which end of the virtual branch the free node is to be by default. The user must then also indicate in context when the free node should be treated as being on the other end of the virtual branch. These concepts lead naturally into the very important concept of the flow sign convention used in FEQ.

Flow Sign Convention

A precise and general flow sign convention is essential for an efficient and comprehensible solution of the complex flow systems for which FEQ was designed. We normally think of the flow in a stream as a positive variable just as we think of most variables we use as positive. However, when we do think of flow as positive we have an implicit knowledge or assumption about its direction. Flow in FEQ always has a direction. There are two possibilities: upstream or downstream. If the flow is zero it makes no difference what direction we agree to say the flow has! The basis for the entire sign convention is the choice of defining flow moving in the direction from the upstream node to the downstream node of a branch as positive. Flow movement in the other direction is then negative. Be advised that in a program like FEQ the designations of upstream and downstream flow for a branch are purely nominal. The natural downstream direction need not be the downstream direction defined in FEQ and the user should pick the downstream direction to be that direction for which FEQ is supposed to print positive flow values.

With this basic designation we move to the concept of a sign associated with each exterior node in the system. First we begin with nodes on a branch because they are the next step in the convention. We would like to be able to tell, at least inside the program, when flow is leaving a branch and when it is entering a branch. The usage in FEQ is arbitrary but it is used consistently. We say that the sign of the downstream node on the branch is positive and that the sign of the upstream node on a branch is negative. Then the product of the sign of the exterior node and the sign of the flow at the exterior node will determine whether the flow is leaving or entering the branch. If the product(we take the sign to have an implicit absolute value of unity) is positive then the flow is leaving the branch and if it is negative flow is entering the branch. For a node not on a branch the same convention is used but in this case the current conceptual position and role of the node must be kept in mind because free nodes can shift from one end of a virtual branch to the other at the discretion of the user.

An Analogy

An analogy is useful in putting these concepts into context. As a child you may have had or been aware of what are called tinker toys; a building toy composed of slender pencil-like sticks and round knobs with holes on their periphery and a hole in the center of the knob. A wide variety of stick structures could be built by inserting the sticks into the holes of the knobs. In a sense, FEQ has a tinker toy view of the world of open channel hydraulics. The branches are like the sticks and the special features are like the knobs. We are able to build a model of an open channel system using these parts. Consequently there are few predefined limits in FEQ. The limit to the complexity is usually set by the memory and computer time requirements rather than the structure of the program. As a result, a wide variety of stream systems and conditions can be described without requiring changes to the program.

In keeping with the tinker toy analogy, FEQ describes the parts of the system more or less separately. For example, the branches are completely described in terms of nodes, stations, elevation, and cross sections before any description of the special features are given. Furthermore, the cross sections are only described in terms of their table number and the contents of the table are given later. This is a general rule: all references to functions are given by the table number containing the description of the table. This is done because we can focus on how the various pieces are connected without concern for the location, size, or shape of the cross sections. This layered approach to describing the system attempts to reduce the number of details which must be comprehended simultaneously in order to make management of the details simpler.

The Utility Program: FEQUTL

This program is essential in the description of the stream system. It is used primarily to define cross section tables and tables associated with the bridge loss routines. The program also computes tables describing the elements of circular conduits and collections of circular conduits. In general the cross section is defined by giving the coordinates of the points along the boundary of the section. The section can be divided into subsections to account for the variation of shape as well as roughness. The user must supply a unique table number for each cross section table to be used by FEQUTL as an identifying number. The table numbers must be unique throughout the stream system so they should be picked carefully. It is generally wise to devise some meaningfull categories for table numbers. The range of table numbers can be made quite large. Currently the maximum table number allowed in FEQ(the determining value for FEQUTL) is 10000. For example, all measured cross sections tables might be required to have numbers between 101 and 600. The range for cross section tables to be interpolated by FEQ might then be 601 through 1000. The table numbers for other functions, such as head loss, outflow from reservoirs, storage versus elevation in reservoirs and so forth could also be assigned a range of table numbers.

APPENDIX B

TABLE 1
SUGGESTED BREACH PARAMETERS
(Definition Sketch Shown in Figure 1)

Parameter	Value	Type of Dam
Average width of Breach ($\overline{BR}$) (See Comment No. 1)	$\overline{BR}$ = Crest Length	Arch
	$\overline{BR}$ = Width of 1 or More Monoliths, usually $\overline{BR} \leq 0.5$ W	Masonry, Gravity
	$\overline{HD} \leq \overline{BR} \leq 5HD$ (usually between 2HD & 4HD)	Earthen, Rockfill, Timber Crib
	$\overline{BR} \geq 0.8$ x Crest Length	Slag, Refuse
Horizontal Component of Side Slope of Breach (Z) (See Comment No. 2)	$0 \leq Z \leq$ slope of valley walls	Arch
	Z = 0	Masonry, Gravity, Timber Crib
	$1/4 \leq Z \leq 1$	Earthen (Engineered, Compacted)
	$1 \leq Z \leq 2$	Slag, Refuse (Non-Engineered)
Time to Failure (TFH) (in hours (See Comment No. 3)	$TFH \leq 0.1$	Arch
	$0.1 \leq TFH \leq 0.3$	Masonry, Gravity
	$0.1 \leq TFH \leq 1.0$	Earthen (Engineered, Compacted) Timber Crib
	$0.1 \leq TFH \leq 0.5$	Earthen (Non Engineered, Poor Construction)
	$0.1 \leq TFH \leq 0.3$	Slag, Refuse

Definition: HD - Height of Dam
Z - Horizontal Component of Side Slope of Breach
$\overline{BR}$ - Average Width of Breach
TFH - Time to Fully Form the Breach
W - Crest Length

Note: See page 29 for definition sketch.

Comments: See Page 27-28

Comments:

1. $\overline{BR}$ is the average breach width, which is not necessarily the bottom width. $\overline{BR}$ is the bottom width for a rectangle, but $\overline{BR}$ is not the bottom width for a trapezoid.

2. Whether the shape is rectangular, trapezoidal, or triangular is not generally critical if the average breach width for each shape is the same. What is critical is the assumed average width of the breach.

3. Time to failure is a function of height of dam and location of breach. Therefore, the longer the time to failure, the wider the breach should be. Also, the greater the height of the dam and the storage volume, the greater the time to failure and average breach will probably be.

4. The bottom of the breach should be at the foundation elevation.

5. Breach width assumptions should be based on the height of the dam, the volume of the reservoir, and the type of failure, (e.g. piping, sustained overtopping, etc.).

6. For a worst-case scenario, the average breach width should be in the upper portion of the recommended range, the time to failure should be in the lower portion of recommended range, and the manning's "n" value should be in the upper portion of the recommended range. If a worst-case scenario is not used, a sensitivity analysis should be performed to fully investigate the impacts of a failure on downstream areas since the actual breach parameters will not be known. The sensitivity analysis will provide an estimate of the confidence limits and relative differences resulting from varying failure assumptions.

    a. To compare relative differences in peak elevation based on variations in breach widths, the sensitivity analysis should be based on the following assumptions:

        1. Assume a probable (reasonable) maximum breach width, a probable minimum time to failure, and a probable maximum manning's "n" value. Manning's "n" values in the vicinity of the dam (up to several thousand feet or more downstream) should be assumed to be larger than the maximum value suggested by field investigations in order to account for uncertainities of high energy losses, velocities, turbulence, etc., resulting from the initial failure.

2. Assume a probable minimum breach width, a probable maximum time to failure, and a probable minimum manning's "n" value.

Plot the results of both runs on the same graph showing changes in elevation with respect to distance downstream from the dam.

b. To compare differences in travel time of the flood wave, the sensitivity analysis should be based on the following assumptions:

1. Use criteria in a. 1.

2. Assume a probable maximum breach width, a probable minimum time to failure, and a probable minimum manning's "n" value.

Plot the results of both runs on the same graph showing the changes in travel time with respect to distance downstream from the dam.

c. To compare differences in elevation between natural flood conditions and natural flood conditions plus dambreak, the sensitivity analysis should be based on the following assumptions:

1. Route the natural flood without dambreak assuming a maximum probable manning's "n" value.

2. Use criteria in a. 1.

Plot the results of both runs on the same graph showing changes in elevation with respect to distance downstream from the dam.

d. Investigations under both normal and flood flow conditions should be considered, as appropriate.

7. When dams are assumed to fail from overtopping, wider breach widths than those suggested om Table 1 should be considered if overtopping is sustained for a long period of time.

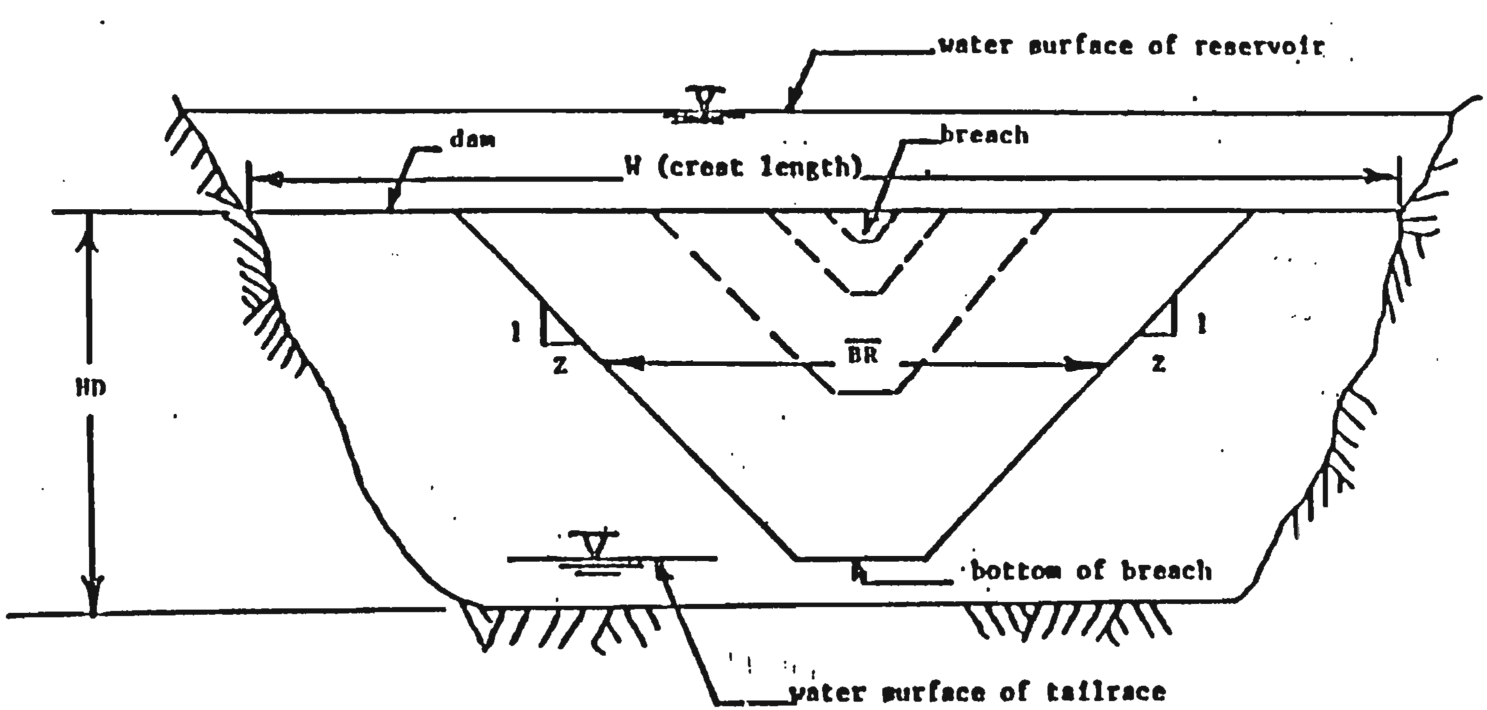

FIGURE 1. DEFINITION SKETCH OF BREACH PARAMETERS