West Regional
Dam Safety
Technical Seminar

DETERMINATION OF THE PROBABLE MAXIMUM FLOOD

Volume II

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Phoenix, AZ

UNIT HYDROGRAPHS FOR SITES WITH LIMITED DATA

(SEC. 8-9 of FERC Guidelines on the Determination of Probable Maximum Flood)

Materials under TAB 19 in Volume 2 of the Notebook

This is part of the Guidelines related to the development of unit hydrograph for basins with limited data - Sec 8.9 of the FERC Guidelines

GENERAL APPROACH

- conduct a search for regional studies which have developed synthetic unit hydrograph parameters applicable to the basin of interest

- perform a regional study to develop synthetic unit hydrograph parameters

- If there are no suitable data available for a regional study, use one of the existing approaches such as those developed by Snyder, Clark, SCS, or others

- for drainage areas smaller than 20 square miles, it is acceptable to use the SCS dimensionless unit hydrograph

APPLICABLE UNIT HYDROGRAPH PARAMETERS

- source of data: local, state and federal agencies

- U.S. Army Corps of Engineers - Chief Engineer's Office in Washington, D.C. or the District Offices

  - U.S. Army Corps of Engineers Report on Civil Works Investigation Project No. CW-153; Unit Hydrograph Compilations, Volumes 1 through 4. Volumes 1, 2 and 3 were published in 1949 and Volume 4 in 1954 published by the Office of the District Engineer, Washington District

  - This publication contains the Snyder's C_t and C_p of 146 watersheds primarily in the areas East ~~west~~ of the Mississippi Valley

  - Los Angeles District has developed dimensionless unit hydrographs for parts of Arizona, California, Colorado, Nevada and New Mexico which are under its jurisdiction

  - The U.S. Army Corps of Engineers Hydrologic Engineering Center at Davis, California also has selected study results for many regions

- U.S Bureau of Reclamation Flood Hydrology Manual (1989) - It contains dimensionless unit hydrographs for regions in the Rocky Mountains; the Great Plains; Southwest Desert, Great Basin and Colorado Plateau; Sierra Nevada, Coast and Cascade Ranges; and urban basins

- U.S. Geological Survey published a number statewide regional studies in cooperation with the state departments of transportation. Data are available for Alabama, Georgia, Illinois, South Carolina and Tennessee

- Illinois Water Survey has data for Illinois

- Some state universities may also have regional study results funded by state agencies. Pennsylvania State University has data for Pennsylvania

CAUTION:

- must assess if the basin of interest is hydro-meteorologically similar to the those used in the regional study

- lag time and channel or basin slopes are often defined differently in the various methodologies. They must be consistent with the methodology used

- the developed relationships must be verified using data available in the basin of interest, or in the region, if applicable

REGIONAL STUDY

at least 10 gaging station data

- must be performed if no applicable unit hydrograph parameters are available and the watershed is larger than 100 square miles

- this is an expensive undertaking

- it involves the development of unit hydrographs for gaged basins in the region, if they have not already been developed

- need continuous streamflow records of major floods in the region and their corresponding hyetographs

GENERAL APPROACH

- "hydro-meteorologically similar" gaged basins in the region need to be identified

- unit hydrographs for these gaged basins are developed using observed hyetographs and corresponding flood hydrographs

- unit hydrograph parameters, such as lag time, T_c, storage coefficient (R), Snyder's C_t and C_p are derived from these unit hydrographs

- develop generalized regional relationships between the unit hydrograph parameters of these gaged basins and their physical characteristics using regression and correlation analyses

- the basin characteristics used in the regression analyses should be those which can be defined easily, such as drainage area, length of principal watercourse, average channel slope, percentage of impervious area, percent of area covered by forest and or lakes. etc

- the general rule in selecting the appropriate regression relationship is to use the coefficient of determination (R^2) and the standard error of estimate (S_e) as the guide and select the relationship with the fewest independent variables and the largest R^2 and smallest S_e values.

- need to assure that the selected regression relationship meet at least a 90% confidence level

- the physical characteristics of the basin of interest are then developed
- unit hydrograph parameters of the basin of interest are developed using the established regional relationships
- the value of $R/(T_c+R)$ estimated using the Clark unit hydrograph parameters, T_c and R, was found to be near a constant for a hydro-meteorologically similar region
- the developed relationships must be verified using data available in the basin of interest, if possible, or in the region

REGIONAL STUDY
STATISTICAL CORRELATION

Suppose:

$$Q_p = C_p \frac{640A}{T_p}$$

$$T_p = C_T \, (LL_{CA})^{0.3}$$

1) Construct unit hydrographs for each available storm and flood.
2) Calculate C_p and C_T for each unit hydrograph.
3) Identify physical parameters for each basin

   S, L, L_{CA}

   Drainage Density
4) $C_p = C_1 A^{n_1} + C_2 L^{n_2} + C_3 L_{CA}^{n_3} + \ldots$

REGIONAL STUDY FOR PENNSYLVANIA BY MILLER

$$C_P = 0.907 + 0.0020(L \cdot L_{CA}) - 0.130(DD) - 0.0613(SCE) - 0.0352(LEXT)$$

$$C_T = 18.6 + 0.0108(L \cdot L_{CA}) - 1.29(DD) - 0.464(SCE) - 0.468(LMAX) - 0.150(CN)$$

DD = drainage density (1/mi)

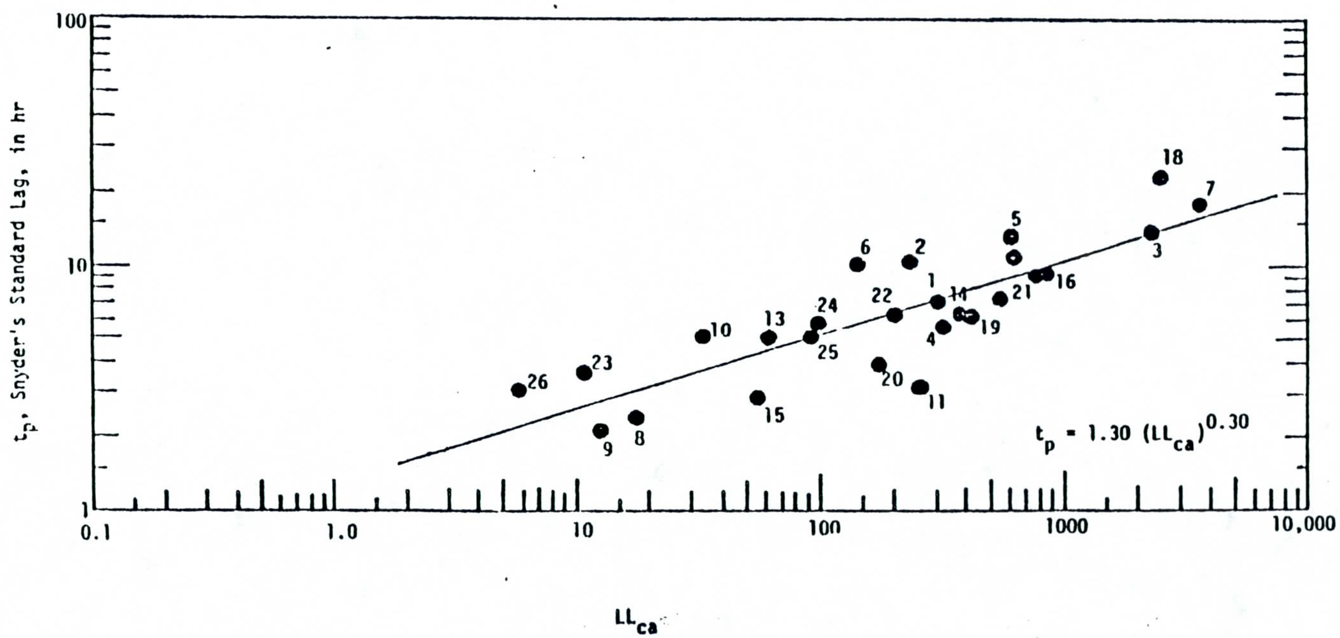
$$SCE = \frac{\text{Maximum Elev Diff}}{\text{Maximum Stream Length in a St. Line top to bottom}}$$

Table 2.--Summary of lagtime estimating equations.

Area	Equation	Standard error of regression (percent)	Coefficient of determination, R^2
North of the Fall Line (rural)	$T_L = 4.64A^{.49}S^{-.21}$	$\pm$ 31	0.94
South of the Fall Line (rural)	$T_L = 13.6A^{.43}S^{-.31}$	$\pm$ 25	.96
Metropolitan Atlanta (urban)	$T_L = 161A^{.22}S^{-.66}IA^{-.67}$	$\pm$ 19	.94

	Gage Location	t_p (hours)	DA (sq mi)
1	Pohopoco Creek near Parryville	7.1	109.0
2	Aquashicola Creek at Palmerton	10.2	76.7
3	Lehigh River at Walnutport	13.4	889.0
4	Little Lehigh River near Allentown	5.5	80.8
5	Jordan Creek at Allentown	12.3	75.8
6	Monocacy Creek at Bethlehem	10.2	44.5
7	Lehigh River at Bethlehem	17.0	1,279.0
8	Saucon Creek at Lanark	2.4	12.0
9	So. Branch Saucon Cr. at Friedensville	2.0	10.6
10	Saucon Creek at Friedensville	5.0	26.6
11	Tohickon Creek near Pipersville	3.1	97.4
12	Neshaminy Creek near Langhorne	10.2	210.0
13	Schuylkill River at Pottsville	5.0	53.4

	Gage Location	t_p (hours)	DA (sq mi)
14	Schuylkill River at Landingville	6.0	133.0
15	Little Schuylkill River at Tamaqua	2.9	42.9
16	Schuylkill River at Berne	9.2	355.0
17	Tulpehocken Creek at Reading	7.1	211.0
18	Schuylkill River at Pottstown	22.8	1,147.0
19	Perkiomen Creek at Graterford	6.0	279.0
20	Ridley Creek at Moylan	3.9	31.9
21	Brandywine Creek at Chadds Ford	9.2	287.0
22	Chester Creek at U.S.G.S. gage	6.4	61.1
23	Maiden Creek Tributary at Lenhartsville	3.7	7.5
24	French Creek near Phoenixville	5.7	59.1
25	Skippack Creek near Collegeville	5.1	53.7
26	Pickering Creek near Chester Springs	3.1	6.0



Graphical Correlation - t_p as a Function of LL_{ca}

Rancocas Ck, NJ

SUBAREA PHYSICAL AND UNITGRAPH CHARACTERISTICS

Subarea Index Number	Drainage Area D.A.	1970 Population	Population Density Per sq mi D	Imperviousness I	Length L	Length to Center LL_{ca}	Slope S_{10-85}	Lakes & Swamps St	TC+R	$\frac{R}{TC+R}$	TC	R
	(sq mi)			(%)	(mi)	(mi)	(ft/mi)	(%)				
36	3.58	1340	374	7.0	2.4	1.2	9.0	2	16	.75	4	12
37	1.87	930	479	8.4	2.9	1.7	15.8	4	15	.75	4	11
38	5.17	7200	1392	15	4.0	1.8	14.8	10	22	.75	5	17
39	5.41	640	118	3.4	3.6	1.6	12.1	.5	12	.75	3	9
40	4.04	21750	5380	30	3.9	2.6	17.4	.5	6.4	.75	1.6	4.8
41	1.91	15200	7960	38	2.9	1.7	16.9	7	12	.75	3	9
42	6.06	14000	2310	19	4.4	3.0	8.5	17	29	.75	7	22
Total	351.86											

$$I = 0.117D^{0.792 - 0.039 \log D}$$

$$TC+R = 21\ (DA/S)^{.22}\ (St)^{.33}\ (1+.3I)^{-.28}$$

*These values reflect a 50 percent increase over the regional values as required to reconstitute observed hydrographs at the Pemberton gage.

**Philadelphia District February 1978 revised population estimates.

SUMMARY OF REGRESSION RESULTS

Equation No.	Dependent Variable	Independent Variables	Function Type	R^2	$\bar{R}^2$	Standard Error of Estimate in % of Mean of Dependent Variable
1	TC	St,L,S,A	Linear	0.226	0.019	67.6
2	TC	L,A,St,(S)	Log	0.327	0.200	37.6
3	R	St,A,L,S	Linear	0.581	0.469	55.4
4	R	St,A,(S)	Linear	0.570	0.520	52.7
5*	R	St,L,A,S	Log	0.466	0.324	28.2
6*	R	St,A,(S)	Log	0.402	0.331	28.1
7	LAG	St,L,S,A	Linear	0.462	0.319	51.6
8	LAG	St,A,S	Linear	0.394	0.281	53.0
9*	LAG	St,A,L,(S)	Log	0.467	0.367	27.6
10*	LAG	St,A,(S)	Log	0.460	0.397	27.0
11	CP	S,(A,L,St)	Linear	0.028	0.	43.9
12	CP	St,S,(A,L)	Log	0.045	0.	84.9
13*	(TC+R)	St,L,S,(A)	Linear	0.689	0.631	31.6
14*	(TC+R)	St,A,S	Linear	0.636	0.567	34.2
15*	(TC+R)	St,A	Linear	0.619	0.574	33.9
16*	(TC+R)	St,A/S	Linear	0.652	0.611	32.4
17*	(TC+R)	A,St,S,L	Log	0.713	0.594	10.1
18*	(TC+R)	A,St,S	Log	0.692	0.597	10.0
19*	(TC+R)	A,St	Log	0.679	0.606	9.9
20*	(TC+R)	St,A/S	Log	0.582	0.533	10.8
21	R/(TC+R)	S,(A,L,St)	Linear	0.046	0.	53.3
22	R/(TC+R)	St,A,L,(S)	Log	0.091	0.	83.8
23	R/(TC+R)	St,A,(S)	Log	0.054	0.	83.0

*Indicates equations selected for detailed analysis.

( )Indicates independent variables that were not included in the equation by the stepwise regression program because they did not significantly improve the results as described in the text.

The final regression equations were then:

$$(TC+R) = 7.52 * A^{.215} * St^{.425}$$

$$R = 3.30 * A^{.155} * St^{.775}$$

When regression analysis is used to determine equations relating the various model parameters, the standard error of estimate and the coefficient of determination are computed for each equation. Typical results of such an analysis are tabulated in the following Tables, where values of S_e and R^2 are given for each equation. The general rule is to use R^2 and S_e as a guide and select the equation with the fewest independent variables and the best values of R^2 and S_e.

TYPICAL RESULTS OF MULTIPLE REGRESSION ANALYSIS
FOR REGIONALIZATION OF MODEL PARAMETERS

(Several Basins in New Jersey and Pennsylvania)

	Standard Error of Estimate S_e	Correlation Coefficient R	Coefficient of Determination R^2
$TC = 26.19\ I^{-0.53} S^{-0.29}\ (DA)^{0.23}$	0.0495	0.9710	0.9428
$TC = 19.84\ I^{-0.50}\ (DA/S)^{0.26}$	0.0358	0.9849	0.9701
$TC = 8.29\ K^{-1.28}\ (DA/S)^{0.28}$ *	0.0269	0.9915	0.9831
$TC = 4.14\ (DA/S)^{0.39}$	0.1296	0.7800	0.6084
$(TC + R) = 122.64\ I^{0.42}\ S^{-0.55} (DA)^{0.09}$	0.1442	0.6844	0.4684
$(TC + R) = 15.69\ I^{-0.21} (DA/S)^{0.34}$	0.1161	0.8094	0.6552
$(TC + R) = 11.52\ K^{-0.67}\ (DA/S)^{0.33}$ *	0.1054	0.8461	0.7159
$(TC + R) = 7.98\ (DA/S)^{0.39}$	0.1093	0.8333	0.6944

* $K = 1.0 + 0.03I$

SEMINAR AGENDA

DETERMINATION OF PROBABLE MAXIMUM FLOOD

ASSOCIATION OF STATE DAM SAFETY OFFICIALS

1. Seminar Agenda
2. General Information
3. Instructor Resumes

Monday

	Time	Description
4.	8:00	Welcome - Introduction - Announcements An Overview Objective - To provide a reasonable approach for uniform application in determining the probable maximum flood (PMF) hydrograph PMF Guidelines Section 8-1
5.	8:30	Introduction to Runoff Analysis - The Hydrologic Cycle (1) Nature of Runoff Hydrographs (2) Basin Rainfall (3) Effective Rainfall (4) Loss Analysis
6.	9:00	Preliminary Review of Hydrologic Data PMF Guidelines Section 8-2
7.	9:20	Development of Hydrologic Criteria of the PMF PMF Guidelines Section 8-3
	9:40	COFFEE BREAK
8.	10:00	Data Acquisition PMF Guidelines Section 8-4

	Time	Description
9.	10:20	Review and Assessment of Data PMF Guidelines Section 8-5
10.	10:40	Subdivision and Drainage Area PMF Guidelines Section 8-6
11.	11:00	Approach to Tasks for PMF Development PMF Guidelines Section 8-7
12.	11:20	Unit Hydrograph Theory - Theory of Unit Hydrograph for Gaged Watersheds - Assumptions and Limitations PMF Guidelines Section 8-8 (1) Definition (2) Base Flow Separation (3) Duration of the Unit Hydrograph (4) Computation Time Increments - How Important
	12:00	LUNCH
13.	1:00	Methods of Calculating Infiltration (1) Uniform Loss Function - Time Index (2) Soil Conservation Service's Curve Number Method (3) Horton Equation (4) Green and Ampt Infiltration Equation (5) Physically Based Methodology
14.	1:45	Time of Concentration (1) Regression Methods (2) Hydraulic Methods (3) Hydrograph Method
15.	2:05	Clark Method for Deriving Unit Hydrographs (1) Conceptual Models of the Unit Hydrographs (2) Concept of the Instantaneous Unit Hydrograph (IUH)
	2:45	COFFEE BREAK
16.	3:05	Flood Hydrograph for Gaged Watershed - Sabrina Example
	5:00	Adjourn

Tuesday

	Time	Description
17.	8:00	Review and Questions
18.	8:30	Synthetic Unit Hydrography Theory for Ungaged Watersheds (1) Snyder Synthetic Unit Hydrograph (2) Soil Conservation Service Dimensionless Unit Hydrograph Developing Watershed Parameters for Ungaged Watersheds (1) Clark Instantaneous Unit Hydrograph (2) Snyder's Synthetic Unit Hydrograph (3) SCS Dimensionless Unit Graph
	9:30	COFFEE BREAK
	9:45	Continuation of Previous Lecture
19.	10:30	Unit Hydrographs for Sites with Limited Data PMF Guidelines Section 8-9 (1) Search for Applicable Unit Hydrographs (2) Regional Analysis (3) Data Required (4) Rainfall Analysis (5) Development of Generalized Regional Relationships
20.	11:00	Introduction to Flood Routing PMF Guidelines Section 8-11 (1) Hydraulic (2) Hydrologic (a) Muskingum (b) Muskingum Cunge (c) Reservoir Routing
	12:00	LUNCH
21.	1:00	Probable Maximum Flood (PMF) Development PMF Guidelines Section 8-10
22.	1:30	Data Collection for Ungaged Watersheds - Sensitivity
23.	2:00	Example: Corsorona Rapids
	3:00	COFFEE BREAK
	3:20	Continuation of Example Discussion
	5:00	Adjourn

Wednesday

	Time	Description
24.	8:00	Review and Questions
25.	8:30	Ungaged Watersheds - No Data (Bishopsville Example)
	9:30	COFFEE BREAK
26.	9:50	Glossary, Terms, and Report Formats
27.	10:10	Review and Questions
28.	11:00	Limitations of Unit Hydrograph Theory
29.	11:20	Hydrology (1) Future Models (2) GIS Databases (3) Kinematic Wave (4) New Research Being Developed
	12:00	LUNCH
30.	1:00	Example: Austen
	3:00	COFFEE BREAK
31.	3:20	Special Considerations (1) Dam Break Parameters (2) Antecedent Conditions (3) Start Q at Beginning of Flow (4) Reservoir Levels (5) Gate Operations (6) Sediment
	4:00	Summary
	4:30	Evaluations
	5:00	Adjourn

UNIT HYDROGRAPHS FOR SITES WITH LIMITED DATA

PMF GUIDELINES SECTION 8-9

Tuesday 10:30 a.m.

8-9 Unit Hydrographs for Sites with Limited Data

For this chapter an "ungaged" site is one for which there is either no data available from gages within the basin, or the available streamflow and rainfall data are insufficient in either quality or quantity to provide confidence in developing applicable unit hydrographs. When such a site is encountered, a unit hydrograph must be developed synthetically. One of the following approaches should be followed.

- Conduct a search for regional studies that have developed synthetic unit-hydrograph procedures applicable to the basin.

- Perform a regional study to develop synthetic unit-hydrograph procedures. The study could develop either a new approach or coefficients for an existing one.

- If there are no suitable data available for a regional study, use one of the existing approaches such as those developed by Snyder, Clark, the SCS, or others. In this situation, the required coefficients must be selected empirically based on coefficients developed for other regions. The applicability of the adopted coefficients must be justified and documented.

- For drainage areas smaller than 20 square miles, it is acceptable to use the SCS dimensionless unit hydrograph; however, adjustments may be necessary depending on basin characteristics (e.g., steep slopes). For basins larger than 20 square miles, an aggregate method can be used.

In a regional analysis, unit hydrographs are developed for gaged drainage basins in the region. A unit-hydrograph model is adopted. Relationships between the parameters of the unit-hydrograph model and the physical characteristics of the basin are developed. Synthetic unit hydrographs are estimated for ungaged basins by means of the established relationship between parameters of the unit-hydrograph model and the physical characteristics of the basin.

Caution: The applicability of any method to an ungaged site is always subject to question because of the fundamental uncertainty in predicting basin response in terms of defined physical characteristics. In general, any synthetic unit hydrograph should not be used unless:

- The parameters for the unit hydrograph are well defined and correlated with quantifiable basin characteristics.

- The unit hydrographs used in developing the relationships have been verified by reproducing the largest floods of record in the database.

Use any historic rainfall or peak flow data from within the basin to verify regional synthetic hydrographs and determine their applicability to the basin. Thus, it is always important to use all data available from stations within the basin when developing an inflow PMF hydrograph.

8-9.1 Applicable Unit Hydrograph for Each Basin/Subbasin Procedures

Many general studies have been performed by local, state, and federal agencies to develop synthetic unit-hydrograph procedures, or coefficients for existing ones, applicable to a particular region. The following are a few examples of regional studies available from federal, state, and local agencies for developing synthetic unit-hydrograph procedures for ungaged sites.

The COE has developed coefficients for use in computing Snyder and Clark unit hydrographs for many areas in the United States. There is no single source for the COE-developed information, but district offices of the COE can provide information on the results of any studies conducted in the region.

The USBR has developed a set of lag-time equations, dimensionless unit hydrographs, and S-graphs for different parts of the western states [Cudworth 1989].

The USGS has performed a number of statewide regional studies for the development of unit hydrographs in cooperation with state departments of transportation. These are published as USGS water resources investigation reports. Several, but not all, are referenced in Section 8-12 [USGS 1982, 1986, 1988, 1990].

Caution: Any information obtained must be carefully reviewed to determine if it is applicable to the project basin.

- A first check is to assess whether the basin of interest is hydrologically similar to those used in the regional study. If the available regional study was developed for basins in a rural setting, the study's applicability to watersheds in an urban environment would be questionable, or vice versa.

  Caution: The reviewer must keep in mind that adjoining basins are often not hydrologically similar even though they may adjoin. Any differences in drainage area, cover, soil type, orientation, or geology should be identified.

- Storm and flood data used in the regional study should meet the same quality requirements as set forth in Section 8-8 for the development of unit hydrographs for "gaged" sites, including the consideration of adjusting unit hydrographs for possible nonlinearity.

- In addition, the terminology used to define the various unit hydrograph and basin parameters in the regional study should be clearly understood—particularly the definitions of lag time and channel slope, since a misunderstanding could lead to development of an invalid unit hydrograph.

  Caution: Lag time and channel or basin slope are often defined differently in the various methodologies. The definition of the parameter must be consistent with the methodology used.

- In the Snyder unit hydrograph (Equation 8-9.3), the lag time is defined as the elapsed time from the centroid of the rainfall to the unit-hydrograph peak, which is the same definition used by the SCS.

- The USBR defines the lag time as the time from the center of the unit rainfall excess to the time that 50 percent of the volume of the unit runoff from the basin has passed the concentration point.

Caution: The hydrologic engineer must have a clear understanding of the definitions of all parameters involved, if using methodologies or studies developed by others.

The capability of a developed unit hydrograph to reconstitute major historic flood hydrographs must be assessed. If reconstitutions were successfully performed in the available study, the unit hydrograph may be acceptable for application to the basin of interest. It will also be desirable to use the unit hydrograph to reconstitute a major historic flood hydrograph if data are available. If the results of that reconstitution are satisfactory, the unit hydrograph may be acceptable.

Upon obtaining parameters from an acceptable regional study, unit hydrographs for each subbasin should be developed in accordance with the application of the regional study or, in the absence of specific directions, according to common unit-hydrograph theory.

8-9.2 Regional Analysis

If the search for applicable synthetic unit-hydrograph procedures proves fruitless, and the drainage area is larger than 100 miles, a regional analysis will be required.

A regional study could be either relatively easy or require a substantial effort, depending on available regional data. For regions where systematic records of both rainfall and streamflow have been carefully kept and are readily available, the effort may be as simple as plotting graphs of peak-flow rate and lag time against drainage area; otherwise, the effort can involve significant time and expenditure.

Regional unit-hydrograph studies are generally performed by developing unit hydrographs for historic storms on "gaged" basins within the region. The process of developing unit hydrographs for gaged basins is described in Section 8-8 for basins with adequate data. In the final analysis, the parameters defining the developed unit hydrographs are correlated with measurable basin characteristics to determine if an analytical relationship can be formulated. If the hydrograph parameters correlate well with basin characteristics, the results can then be used to generate unit hydrographs for the ungaged basin of interest.

To conduct a regional study, "gaged" basins in the region need to be identified. The needs for, and sources of, data for development of unit hydrographs for such basins in the region are the same as given in Section 8-3. Data review should follow the procedures given in Section 8-4.

8-9.2.1 Data Required

To evaluate the hydrograph parameters needed for input to HEC-1, an analysis of data for "gaged" basins in the region is required. Rainfall and flood records for all basins in the region should be obtained and examined.

Caution: It is desirable to limit the basins examined to those with gaged areas about the same size and slope as the basin of interest, since the effects of the various parameters cannot be accurately quantified. In practice, it will be necessary to consider both larger and smaller basins. Data and basin selection should be justified.

Since the objective is to develop a unit hydrograph that can be used to determine the inflow PMF hydrograph, the data obtained should include:

- Available topographic, soil, and geologic maps for each basin.
- Drainage area.
- Location and history of all stream gages in each of the basins.
- Location and history of all rain gages in each of the basins.
- Location, history, and data available for snow courses in the basins, if the PMF is apt to be influenced by snowmelt.
- Continuous streamflow records for major floods of interest. It is desirable to have records for at least three or four floods and concurrent rainfall data for each basin to provide confidence in the representative unit hydrograph for each basin. However, since the analysis is being done regionally, all large floods for which data are available should be analyzed.
- Rainfall records for storms that produced the historic floods for which flood-flow data have been obtained.
- Aerial photographs of the basins.

The basins should be visited to obtain information on land use, cover, and the physical characteristics of any dams and reservoirs. If there are dams in any of the basins, information on reservoir area and volume, spillway and outlet works capacity, and operation during historic floods should be obtained.

The following parameters have been found to be useful for correlation of unit-hydrograph parameters in regional analyses:

- Drainage area (A).

- Length of the longest watercourse in miles from the basin outlet to the upper limit of the basin (L).

- Length of the main watercourse in miles from the basin outlet to the point nearest the centroid of the basin area (L_{ca}).

- Channel slope (S).

- Percent impervious area (A_I).

- Percent of area covered by forest.

- Percent of area covered by lakes or marshes.

For each basin analyzed, the following parameters should be computed.

- An estimate of lag time T_L and time of concentration T_c for each basin based on applicable equations obtained from the local flood-control agencies, or calculated as described in Section 8-8.

- The maximum time increment of rainfall to be used in the unit-hydrograph analysis is $T_L/4$ rounded to the next lower even number.

- Infiltration rates for each basin/subbasin using methods described in Sections 8-8.3.2 and 8-8.7.

Caution: Subdivision to areas smaller than that represented by a recording stream gage cannot be done, because the object of the study is to develop unit hydrographs.

8-9.2.2 Rainfall Analysis

Basin average rainfall should be computed using the procedures described in Section 8-8.2.

Temporal distribution of rainfall for each storm should be developed for each basin using the procedures described in Section 8-8.6.

8-9.2.3 Development of Generalized Regional Relationships

HEC-1 and the Clark unit-hydrograph method should be used to develop representative unit hydrographs for the selected basins with available data. The selection of the basins should be justified. In general, it is desirable to have gage data for at least four basins in the region. Parameters for use with the Clark unit hydrograph should be developed from the basin data, including Clark's storage coefficient R, and the time of concentration T_c. In addition, it will be necessary to evaluate the HEC-1 baseflow separation parameters STRTQ, RTIOR, and QRCSN. Procedures for calculating these parameters are given in Sections 8-8.4 and 8-8.5. Once all input information has been entered, HEC-1 should be used to optimize a unit hydrograph for each selected basin. The HEC-1 runs for each basin should be programmed to optimize the hydrograph parameters while allowing $R/(T_c + R)$ to vary. A representative unit hydrograph must be developed for each basin analyzed.

Once a representative unit hydrograph has been developed for each basin analyzed, the values of $R/(T_c + R)$ for all of the basins should be used in a regression analysis against basin parameters. A very simple regression analysis could be performed by plotting values of peak flow and lag time against drainage area on semi-log or log-log paper. If a well-defined relationship is found, the results can be used to develop a representative unit hydrograph for the project basin.

If a well-defined relationship is not found in the simple regression analysis, it may be that parameters other than drainage area have a strong influence in determining the peak flow rate and lag time for basins in the region. In that case, it will be necessary to perform a multiple linear regression of T_c and $R/(T_c + R)$ against identifiable basin parameters, such as S, L, L_{ca}, and A, or combinations of these parameters. If a portion of the basin is impervious, a measure of that parameter—such as the basin's percentage of impervious drainage area—should be included in the regression analysis. If lakes or marshes exist in the basins, it may also be necessary to include the percent of drainage area occupied and controlled by lakes and marshes as an independent parameter.

A multiple linear regression program will yield values of the coefficient of determination, which provides a measure of the degree to which the independent variables influence the value of the dependent variable. The regression analysis should be started using all independent parameters and then eliminating those with little influence on the value of the dependent parameter. For basins where impervious areas are small enough to be considered insignificant, the resulting equation for T_c or $(T_c + R)$ may have the form

$$T_c = C_1 \left(\frac{A}{S}\right)^{C_2} \tag{8-9.1}$$

where C_1 and C_2 are constants determined in the regression. Ideally, the value of the coefficient of determination will be equal to or greater than 0.9; a perfect correlation would yield a value of 1.

Caution: In actuality, the value of the coefficient of determination will often range from 0.6 to 0.8. Different values of the regression constants will be determined for each set of independent variables included in the regression.

The hydrologic engineer should review the derived relationships for consistency and use the equation that yields the smallest value of standard error of estimate and the largest value of the coefficient of determination.

Caution: Since $R/(T_c + R)$ tends to be constant for a region, it may not be statistically significant in a regression analysis. In that case, an average value for the region should be computed from the regional results and used for the analysis of the project basin. In either event, the selected values should be justified.

Once the regression analysis has been completed, the values of T_c, R, and $R/(T_c + R)$ can be computed for the project basin in terms of the computed basin parameters identified as important in the regression analysis. All parameters are then available for use in the Clark unit-hydrograph option in HEC-1 and can be used to develop the inflow PMF hydrograph.

8-9.3 Empirical Coefficients for Synthetic Unit-Hydrograph Procedures

Failing to find applicable procedures or data to perform a regional analysis, consideration should be given to using empirical coefficients for one of the existing procedures. Empirical coefficients for computing a synthetic unit hydrograph are often presented in technical literature as being applicable to basins described only in general terms, such as rolling hills or coastal plains. These unit hydrographs are often used to design minor civil works projects. However, synthetic unit hydrographs and empirical equations for lag time and time to peak are not acceptable for use in PMF-hydrograph computations, unless there is documented evidence of their applicability, or proof that applicability can be developed. Such justification may exist in the form of special regional studies.

- In this chapter the Clark, Snyder, and SCS unit hydrographs are the only ones recommended, but only because the HEC-1 program includes these methods.

- Other synthetic unit hydrographs may be available from other studies or technical references and may be applicable to the project. If they are used, full documentation must be provided and their use justified.

- Always check and explain regional results by comparison to TR 55 calculated time of concentration [SCS 1986].

- Most synthetic unit hydrographs have been developed for a particular storm duration in keeping with unit-hydrograph theory. It will be necessary to know the duration for any unit-hydrograph considered and to adjust that unit hydrograph to fit the duration required for the basin being considered (required duration must not be more than the lag time divided by 5). Methods for making such adjustments, such as use of the S-Curve, are covered in standard hydrology textbooks. The Snyder parameters employed by HEC-1 are the "standard" lag, t_p, and peaking coefficient, C_p. HEC-1 sets the unit duration of a developed unit hydrograph equal to the computation interval (Δ t) using equations based on the Snyder "standard" parameters.

8-9.3.1 Snyder Unit Hydrograph

Many regional studies performed in the United States have concentrated on computing coefficients for the Snyder unit hydrograph in terms of measurable basin parameters. The equations used for the Snyder unit hydrograph are [HEC 1990a]:

$$t_p = C_t(L * L_{ca})^{0.3} \tag{8-9.2}$$

$$C_p = \frac{Q_p * t_p}{(640 * A)} \tag{8-9.3}$$

where:

	t_p	=	Time to peak measured from the onset of precipitation excess (hours)
	L	=	Length of the main watercourse (miles)
	L_{ca}	=	Length along the main watercourse measured upstream to the point opposite the centroid of the basin (miles)
	Q_p	=	Peak flow rate of the unit hydrograph (cfs)
	A	=	Drainage area (square miles)

The coefficients C_t and C_p are strictly empirical values often recommended as applicable to specific regions. C_t accounts for storage and slope of the watershed, and C_p is a function of flood wave velocity and storage.

Caution: Snyder's original development was performed for large basins in the Appalachian region [Snyder 1938]. If information from detailed regional studies give values of C_t and C_p in terms of definable parameters for regional drainage basins, use of the Snyder equations may provide satisfactory results. The acceptability of the Snyder method and parameters, or any other method, must be documented and justified.

8-9.3.2 Clark Unit Hydrograph

The Clark unit hydrograph uses a time-area curve for the basin. Since the unit hydrographs appear to be relatively insensitive to the shape of this time-area curve unless the basin is one with little storage, the automatic generalized curve in HEC-1 can be used. Values for T_c and R should be estimated as described in Section 8-8. The calculated value of $R/(T_c + R)$ should be fixed in HEC-1 for the development of the unit hydrograph.

Caution: The means of estimating T_c and R are by no means infallible; it is extremely important that the hydrologic engineer doing this estimation have substantial experience so as to understand the hydrologic behavior of the basin. Although analytical techniques are indispensable when working on ungaged basins, the judgment of the experienced hydrologic engineer is of extreme importance. The values selected for T_c and R should be justified.

Snyder unit-hydrograph parameters may be entered in the HEC-1 program if acceptable generalized values are available for the region. The Snyder unit-hydrograph relationships define only the unit-hydrograph peak discharge and the time to peak t_p. Recommended widths of the unit hydrograph at 50 percent and 75 percent of the peak flow can be computed in terms of estimated values of C_t and C_p for the basin [COE 1946]. However, when using HEC-1, this is not required since the program computes a Clark unit hydrograph by estimating T_c and R from the t_p and C_p values of the Snyder unit hydrograph.

Caution: Unless a regional study has been performed for the selection of appropriate t_p and C_p values as a function of definable basin characteristics, their selection would be entirely judgmental based on the hydrologic engineer's personal impression of basin conditions—a procedure which is not recommended. Selected values for t_p and C_p should be documented and justified.

8-9.4 SCS Dimensionless Unit Hydrograph

If applicable methods from regional studies are not available, the SCS unit-hydrograph method for ungaged sites—which is described fully in the SCS National Engineering Handbook [SCS 1985]—may be used for basins with total areas not exceeding 100 square miles. (This upper limit on total area size only applies to ungaged sites.) However, subbasins should not exceed 20 square miles if the SCS method is used. The only analytical requirement for application of this method is estimation of the lag time for the basin. In HEC-1, the SCS dimensionless unit hydrograph is fully defined by one parameter—the SCS lag time—and is assumed equal to 0.6 T_c.

Caution: Many empirical equations have been published for estimating T_c, but all are subject to large uncertainties; the hydraulic method of calculating T_c, as recommended in Section 8-8, should be used. The value, method, and equation selected for computation of T_c must be justified and consistent with the respective methodologies.

APPLICATION OF REGIONALIZATION PROCEDURE
TO UNIT HYDROGRAPH PARAMETERS AND FREQUENCY STATISTICS

1. Application of Regionalization Procedure to Unit Hydrograph and Loss Rate Parameters

   a. Select study area.

      (1) Should be large enough to have several gaged basins

      (2) Gaged basins need not be in the same watershed as the ungaged sites for which parameters are desired

      (3) Area should be as nearly hydrologically and meteorologically homogeneous as possible.

   b. Derive unit hydrograph and loss rate parameters

      (1) The Flood Hydrograph Package (HEC-1) computer program can be used to develop the best parameters for historic floods.

      (2) Derived results should be review for consistency and an appropriate value adoped for each parameter for each basin.

   c. Select basin variables for regression analysis

      (1) Time of concentration (TC) and Clark's storage relation (R) are interrelated; therefore, best to correlate TC + R

      (2) Possible basin variables are drainage area (DA), slope (S), and length (L). Various investigators have proposed different combinations of these: $L/\sqrt{S}$ (NAD), $LL_{ca}/\sqrt{S}$ (SPD), $L\sqrt{DA/S}$ (Linsley) , and there are others.

      (3) Usually the logarithmic transformation is appropriate

d. Make multiple regression analyses.

(1) A graphical analysis could be used for one or two independent variables.

(2) Select the best equation with the fewest possible independent variables.

(3) The residuals can be obtained from most computer regression packages.

e. Map the residuals

(1) Linear relations

(a) Residual is observed value minus computed.

(b) Plot residuals at centroid of basin area and draw lines of equal residual.

(c) Prediction equation becomes: $Y = C_m + b_1X_1 + b_2X_2 + \ldots + b_nX_n$ where: C_m is the map coefficient, note that "a" has been added to the residual to reduce the number of constants.

(2) Logarithmic relations

(a) Regression equation is $\log Y = a + b_1 \log X_1 + b_2 \log X_2 + \ldots + b_n \log X_n$ or transforms to

$$Y = \text{antilog}\ (a) \cdot X_1^{b1} \cdot X_2^{b2} \cdot \ldots \cdot X_n^{b_n}$$

(b) Residual is observed divided by computed

(c) Plot residuals

(d) Prediction equation becomes $Y = C_m \cdot X_1^{b_1} \cdot X_2^{b_2} \cdot \ldots \cdot X_n^{b_n}$

where C_m is map coefficient, note antilog (a) has been multipled by the ratio of observed to computed to reduce the number of constants.

f. Ratio of R/(TC + R)

If TC + R has been the parameter, an average R/(TC + R) may be adopted or repeat steps c thur e.

g. Loss Rate Parameters

(1) HEC-1 loss rate coefficients ERAIN and RTIOL can generally be assumed constant for a given study area; however, the coefficients STRKR AND DKTKR will vary with each storm event.

(2) STRKR and DLTKR can be generalized for synthetic events.

(3) Initial and constant loss rate values could be used, but the optimum values <u>cannot</u> be automatically derived by HEC-1.

2. Application of Regionalization Procedure to Flood Frequency Relations

a. Select study area

Same criteria as in Sec 4a.

b. Process information

(1) Logarithmic transformation usually appropriate for annual flood peaks

(2) The log Pearson Type III distribution requires, computation of the mean, standard deviation, and the skew coefficient of the logs.

(3) The Regional Frequency Computation computer program facilitates the required computations.

c. Select appropriate basin variables.

(1) The logarithmic transformation is usually appropriate for the independent variables, elevation is an example of of an exception.

(2) Do not transform the mean log or standard deviation.

(3) Regression techiques are <u>not</u> appropriate for the skew coefficient (see Sec 5f)

d. Make multiple regression analyses

Same comments as in Sec 4d.

e. Map the residuals

Same comments as in Sec 4e.

f. Regional skew values.

(1) If the area is sufficiently small, an adopted skew coefficient may be reasonable.

(2) For large areas, the individual frequency curves should be screened for reasonable skew values. The values can then be plotted on a map and lines of equal skew determined. Skew coefficients for ungaged sites can be estimated from the map.

Chapter 16
Ungaged Basin Analysis

16-1. General

a. Problem Definition. Earlier chapters of this manual describe various flood-runoff analysis models. Some of the models are *causal*: They are based on the laws of thermodynamics and laws of conservation of mass, momentum, and energy. The St. Venant equations described in chapter 9 are an example. Other models are *empirical*: They represent only the numerical relationship of observed output to observed input data. A linear regression model that relates runoff volume to rainfall depth is an empirical model.

To use either a causal or empirical flood-runoff analysis model, the analyst must identify model parameters for the catchment or channel in question. Section 7.3.e describes a method for finding rainfall-runoff parameters for existing conditions in a gaged catchment. Through systematic search, parameter values are found to yield computed runoff hydrographs that best match observed hydrographs caused by observed rainfall. With these parameter values, runoff from other rainfall events can be estimated with the model. A similar search can be conducted for routing model parameters, given channel inflow and outflow hydrographs.

Unfortunately, as Loague and Freeze (1985) point out, "...when it comes to models and data sets, there is a surprisingly small intersecting set." The rainfall and runoff data necessary to search for the existing-condition calibration parameters often are not available. Streamflow data may be missing, rainfall data may be sparse, or the available data may be unreliable. Furthermore, for USACE civil-works project evaluation, runoff estimates are required for the forecasted future and for with-project conditions. Rainfall and runoff data never are available for these conditions. In the absence of data required for parameter estimation, for either existing or future conditions, the stream and contributing catchment are declared ungaged. This chapter presents alternatives for parameter estimation for such catchments.

b. Summary of Solutions. To estimate runoff from an ungaged catchment, for existing or forecasted-future conditions, the analyst can

1. Use a model that includes only parameters that can be observed or inferred from measurements;

2. Extrapolate parameters from parameters found for gaged catchments within the same region.

In practice, some combination of these solutions typically is employed, because most models include both physically-based and calibration parameters.

c. Using Models With Physically-Based Parameters. Model parameters may be classified as physically-based parameters or as calibration parameters. Physically-based parameters are those that can be observed or estimated directly from measurements of catchment or channel characteristics. Calibration parameters, on the other hand, are lumped, single-valued parameters that have no direct physical significance. They must be estimated from rainfall and runoff data.

If data necessary for estimating the calibration parameters are not available, one solution is to use a flood-runoff analysis model that has only physically-based parameters. For example, the parameters of the Muskingum-Cunge routing model described in section 9.3.e are channel geometry, reach length, roughness coefficient, and slope. These parameters may be estimated with topographic maps, field surveys, photographs, and site visits. Therefore, that model may be used for analysis of an ungaged catchment.

d. Extrapolating Calibration Parameters. If the necessary rainfall or runoff data are not available to estimate calibration parameters using a search procedure such as that described in Section 7.3.e, the parameters may be estimated indirectly through extrapolation of gaged-catchment results. This extrapolation is accomplished by developing equations that predict the calibration parameters for the gaged catchments as a function of measurable catchment characteristics. The assumption is that the resulting predictive equations apply for catchments other than those from which data are drawn for development of the equations.

The steps in developing predictive relationships for calibration parameters for a rainfall-runoff model are as follows:

(1). *Collect rainfall and discharge data for gaged catchments in the region.* The catchments selected should have hydrological characteristics similar to the ungaged catchment of interest. For example, the gaged and ungaged catchments should have similar geomorphological and topographical characteristics. They should have similar land use, vegetative cover, and agricultural practices. The catchments should be of similar size. Rainfall distribution and magnitude and factors affecting rainfall losses should be similar. If possible, data should be collected for several flood events. These rainfall and discharge data should represent, if possible, events consistent with the intended use of the model of the ungaged catchment. If the rainfall-runoff model will be used to predict runoff from large design storms, data from large historical storms should be used to estimate the calibration parameters.

(2). *For each gaged catchment individually, use the data to estimate the calibration parameters for the selected rainfall-runoff model.* The procedure is described in chapter 7, and guidelines for application of the procedure are presented in chapter 13 of this document.

(3). *Select and measure or estimate physiographic characteristics of the gaged catchments to which the rainfall-runoff model parameters may be related.* Table 16-1 lists candidate catchment characteristics. Some of these characteristics, such as the catchment area, are directly measured. Others, such as the Horton ratios, are computed from measured characteristics.

(4). *Develop predictive equations that relate the calibration parameters found in step 2 with characteristics measured or estimated in step 3.* In a simple case, the results of steps 2 and 3 may be plotted, with the ordinate a rainfall-runoff model parameter and the abscissa a catchment characteristic selected in step 3. Each point of the plot will represent the value of the parameter and the selected characteristic for one gaged catchment. With such a plot, a relationship can be "fitted by eye" and sketched on the plot. Regression analysis is an alternative to the subjective graphical approach to defining a predictive relationship. Regression procedures determine numerically the optimal predictive equation. Details of regression analysis are presented in EM 1110-2-1415 and in most statistics texts, including those by Haan (1977) and McCuen and Snyder (1986).

To apply a parameter-predictive equation for an ungaged catchment, the independent variables in the equation are measured or estimated for the ungaged catchment. Solution of the equation with these values yields the desired flood runoff model parameter. This parameter is used with the same model to predict runoff from the ungaged catchment.

TABLE 16.1
Catchment Characteristics for Regression Models

Total catchment area
Area below lowest detention storage

Stream length
Steam length to catchment centroid

Average catchment slope
Average conveyance slope
Conveyance slope measured at 10% and 85% of stream length (from mouth)
Height differential
Elevation of catchment centroid
Average of elevation of points at 10% and 85% of stream length

Permeability of soil profile
Soil-moisture capacity average over soil profile
Hydrologic soil group

Population density
Street density
Impervious area
Directly-connected impervious area
Area drained by storm sewer system
Land use
Detention storage

Rainfall depth for specified frequency, duration
Rainfall intensity for specified frequency, duration

Horton's ratios (Horton, 1945)
Drainage density (Smart, 1972)
Length of overland flow (Smart, 1972)

16-2. Loss-Model Parameter Estimates

a. Options. Two of the rainfall loss models described in chapter 6 of this document are particularly useful for ungaged catchment analysis: the Green-Ampt model and the Soil Conservation Service (SCS) model. The Green-Ampt model is a causal model with quasi-physically-based parameters. The SCS loss model is an empirical model with parameters that have been related to catchment characteristics.

Other loss models may be used if parameter-predictive equations are developed from gaged catchment data.

b. Physically-Based Parameter Estimates for Green-Ampt Model. The Green-Ampt model is derived from Darcy's law for flow in porous media. The model predicts infiltration as a function of time with three parameters: volumetric moisture deficit, wetting-front suction, and hydraulic conductivity. In application, an initial loss may be included to represent interception and depression storage. Additional details of the Green-Ampt model are presented in chapter 6.

Brakensiek and Onstad (1977), McCuen et al. (1981), and Rawls, et al. (1982, 1983, 1985) propose relationships of the Green-Ampt model parameters to observable catchment characteristics, thus permitting application of the model to an ungaged catchment. The relationships define model parameters as a function of soil texture class. Texture class, in turn, is a function of soil particle size distribution. This distribution can be estimated from a sample of catchment soil. For example, a soil that is 80% sand, 5% clay, and 10% silt is classified as a loamy sand. For this texture class, Rawls, et al. (1982) suggest that the average saturated hydraulic conductivity is 6.11 cm/hr. The other parameters can be estimated similarly from the soil sample.

c. Predictive Equations for SCS Model Parameters. The SCS loss model, described in detail in chapter 6, is an empirical model with two parameters: initial abstraction and maximum watershed retention (maximum loss). Often both parameters are related to a single parameter, the curve number (CN). Using data from gaged catchments in the U.S, the SCS developed a tabular relationship that predicts CN as a function of catchment soil type, land use/ground cover, and antecedent moisture. Table 16.2 is an excerpt from this table (USDA, 1986).

To apply the SCS loss model to an ungaged catchment, the analyst determines soil type from a catchment soil survey. For many locations in the U.S., the SCS has conducted such surveys and published soil maps. The analyst determines existing-condition land use/ground cover from on-site inspection or through remote sensing. In the case of forecasted future condition, the land use/ground cover may be determined from development plans. The analyst selects an appropriate antecedent moisture condition for catchment conditions to be modeled (wet, dry, or average). With these three catchment characteristics estimated, the tabular relationship may be used to estimate CN. For example, for a residential catchment with 2-acre lots on hydrologic soil group C, the curve number found in Table 16.2 for average antecedent moisture is 77. With this curve number, the initial abstraction and maximum watershed retention can be estimated, and the loss from any storm can be predicted.

Publications from the SCS provide additional details for estimating the curve number for more complex cases.

16-3. Runoff-Model Parameter Estimates

a. Options. Chapter 7 presents a variety of models for estimating runoff due to excess rainfall. For an ungaged catchment, the analyst may use:

1. The kinematic-wave model;
2. A UH model with physically-based parameters; or
3. A UH model with predictive equations for the calibration parameters.

TABLE 16.2
Excerpt from SCS Curve Number Relationship

Cover description: Cover type and hydrologic condition	Average percent impervious area[2]	Curve numbers for hydrologic soil group— A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.)[3]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only)[4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation)[5]		77	86	91	94
Idle lands (CN's are determined using cover types similar to those in table 2-2c).					

[1]Average runoff condition, and $I_a = 0.2S$.
[2]The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition. CN's for other combinations of conditions may be computed using figure 2-3 or 2-4.
[3]CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.
[4]Composite CN's for natural desert landscaping should be computed using figures 2-3 or 2-4 based on the impervious area percentage (CN = 98) and the pervious area CN. The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.
[5]Composite CN's to use for the design of temporary measures during grading and construction should be computed using figure 2-3 or 2-4, based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

(210-VI-TR-55, Second Ed., June 1986) 2-5 *b. Physically-Based Parameter Estimates for Kinematic Wave Model.* The kinematic-wave model described in chapter 7 is particularly well suited to analysis of an ungaged urban catchment. This causal model, which is described in further detail in HEC documents (USACE, 1979, 1982, 1990), represents the catchment rainfall-runoff process by solving theoretical equations for flow over planes. Catchment runoff is estimated by accumulating the flow from many such planes.

Application of the model requires identification of the following parameters: catchment area, flow length, slope, and overland-flow roughness factor. The area, length, and slope are physically-based and are estimated for existing catchment conditions from maps, photographs, or inspection. For forecasted-future condition, these parameters are forecasted from development plans. The overland-flow roughness factor is a quasi-physically-based parameter that describes resistance to flow as a function of surface characteristics. Published relationships, based on hydraulic experimentation, are used to select this coefficient for existing or forecasted conditions. Thus all parameters of the kinematic wave model can be estimated without gaged data.

c. Physically-Based Parameter Estimates for Clark's IUH and SCS UH. Parameters of Clark's and the SCS empirical UH models have a strong link to the physical processes and thus can be estimated from observation or measurement of catchment characteristics. Clark's IUH accounts for translation and attenuation of overland and channel flow. Translation is described with the time-discharge histogram. To develop this histogram, the time of concentration is estimated and contributing areas are measured. Likewise, the SCS UH hydrograph peak and time to peak are estimated as a function of the time of concentration.

The time of concentration, t_c, can be estimated for an ungaged catchment with principles of hydraulics. The Soil Conservation Service (SCS) suggests that t_c is the sum of travel times for all consecutive components of the drainage conveyance system (U.S. Dept. of Agriculture, 1986). That is,

$$t_c = t_1 + t_2 + \dots t_m \tag{16.1}$$

in which t_i = travel time for component i; and m = number of components. Each component is categorized by the type of flow. In the headwaters of streams, the flow is sheet flow across a plane. Sheet-flow travel time is estimated via solution of the kinematic-wave equations. The SCS suggests a simplified solution. When flow from several planes combines, the result is shallow concentrated flow. The travel time for shallow concentrated flow is estimated with an open-channel flow model, such as Manning's equation. Shallow concentrated flow ultimately enters a channel. The travel time for channel flow is estimated also with Manning's equation or an equivalent model.

d. Predictive Equations for UH Calibration Parameters. The procedure described in section 16.1.d can be used to develop predictive equations for UH model calibration parameters for ungaged catchments. For example, Snyder (1938) related unit hydrograph lag, t_p, to a catchment shape factor using the following equation:

$$t_p = C_t\ (L\ L_{ca})^{0.3} \tag{16.2}$$

in which t_p = basin lag, hr; C_t = predictive-equation parameter; L = length of main stream, mi; L_{ca} =

length from outlet to point on stream nearest centroid of catchment, mi. T
he value of C_t is found via linear regression analysis with data from gaged catchments.

A wide variety of predictive equations for UH model calibration parameters have been developed by analysts. Table 16.3 shows example equations for Snyder's and Clark's UH parameters. In general, these equations should not be used in regions other than those for which they were developed. If they are, the analyst must be especially cautious. He or she should review derivation of the equations. Conditions under which the equations were derived should be examined and compared with conditions of the catchments of interest.

TABLE 16.3
Example UH ParameterPrediction Equations

Equation	Reference
$C_t = 7.81 / I^{0.78}$	Wright-McLaughlin Engineers (1969)
$C_p = 0.89\ C_t^{0.46}$	Wright-McLaughlin Engineers (1969)
$R = c\ T_c$	Russell, Kenning, Sunnell (1979)
$T_c / R = 1.46 - 0.0867\ L^2/A$	Sabol (1988)
$T_c = 8.29\ (1.00 + I)^{-1.28}\ (A/S)^{0.28}$	USACE (1982)

Note: In the above equations, C_t = calibration coefficient for Snyder's UH (see Section 7.3.c); C_p = calibration coefficient for Snyder's UH (see Section 7.3.c); T_c = time of concentration, hr; R = Clark's IUH storage coefficient, hr; I = impervious area, %; L = length of channel/ditch from headwater to outlet, mi; S = average watershed slope, ft/ft; c = calibration parameter, for forested catchments = 8-12, for rural catchments = 1.5-2.8, and for developed catchments = 1.1-2.1; A = catchment area, sq mi.

16-4. Routing-Model Parameter Estimates

a. Candidate Models. The routing models described in chapter 9 account for flood flow in channels. Of the models presented, the Muskingum-Cunge, modified Puls, and kinematic-wave are most easily applied in ungaged catchments. Parameters of each of these models are quasi-physically based and can be estimated from channel characteristics.

b. Physically-Based Parameter Estimates for Modified Puls Routing Model. The modified Puls (level-pool) routing model is described in detail in sections 9.3.a and 9.3.b. The parameters of this model, as it is applied to a river channel, include the channel storage v. outflow relationship and the number of steps (subreaches). The former is considered a physically-based parameter, while the latter is a calibration parameter.

For an ungaged catchment, the channel storage v. outflow relationship can be developed with normal depth calculations or steady-flow profile computations. In either case, channel cross sections are required. These may be measured in the field, or they may be determined from previous mapping or aerial photography. Both procedures require also estimates of the channel roughness. Again, this may be estimated from field inspection or from photographs. With principles of hydraulics, water-surface elevations are estimated for selected discharges. From the elevations, the storage volume is estimated with solid geometry. Repetition yields the necessary storage v. outflow relationship. These computations can be accomplished conveniently with a water-surface profile computer program, such as HEC-2 (USACE, 1990).

The second parameter, the number of steps, is, in fact, a calibration parameter. Section 9.3.b suggests estimating the number of steps as channel reach length / velocity of the flood wave / time interval (see Eq. 9.13). Strelkoff (1980) suggests that if the flow is controlled heavily from downstream, one step should be used. For locally-controlled flow typical of steeper channels, he suggests the more steps, the better. He reports that in numerical experiments with such a channel, the best peak reproduction was observed with:

$$NSTPS = 2\,L\,\frac{S_o}{Y_o} \qquad (16.3)$$

in which *NSTPS* = number of steps; L = entire reach length, in mi; S_0 = bottom slope, in ft/mi; and Y_0 = baseflow normal depth, in ft. So, for example, for a 12.4 mi reach with slope 2.4 ft/mi and Y_0 = 4 ft, the number of steps would be estimated as 15.

c. Physically-Based Parameter Estimates for Kinematic Wave Model. The physical basis of the kinematic-wave model parameters makes that model useful for some ungaged channels. In particular, if the channels are steep, well-defined channels, with insignificant backwater effects, the kinematic-wave model works well. These limitations are met most frequently in channels in urban catchments.

The parameters of the kinematic-wave channel routing model include the channel geometry and channel roughness factor. The necessary channel geometry parameters include channel cross section and slope data. As these are physically-based, they may be estimated for existing conditions from topographic maps or field survey. For modified channel conditions, the geometry data are specified by the proposed design. The roughness generally is expressed in terms of Manning's *n*.

This is a quasi-physically-based parameter that describes resistance to flow as a function of surface characteristics. Published relationships predict this coefficient for existing or modified conditions.

d. Physically-Based Parameter Estimates for Muskingum-Cunge Model. If the channel of interest is not a steep, well-defined channel, as required for application of the kinematic-wave channel routing model, a diffusion model may be used instead. In the case of an ungaged channel, the Muskingum-Cunge model is a convenient choice, as the parameters are physically-based.

Parameters of the Muskingum-Cunge channel routing model include the channel geometry and channel roughness factor. The necessary channel geometry parameters include channel cross section and slope data, which may be estimated for existing conditions from topographic maps or field survey. For modified channel conditions, the geometry data are specified by the proposed design. The roughness is expressed in terms of Manning's *n*.

16-5. Statistical-Model Parameter Estimates

In some hydrologic-engineering studies, the goal is limited to definition of discharge-frequency relationships. EM 1110-2-1415 describes procedures for USACE flood-frequency studies. Chapter 12 of this document summarizes those procedures and describes the statistical models used. All the models described are empirical. Observed data are necessary for calibration. Consequently, these statistical models cannot be applied directly to an ungaged catchment.

Options available to the analyst requiring frequency estimates for an ungaged stream include

1. Develop frequency-distribution parameter predictive equations; or

2. Develop distribution quantile predictive equations.

a. Parameter Predictive Equations. The log-Pearson type III distribution (model) is used for USACE annual maximum discharge frequency studies. As described in chapter 12, this model has three parameters. These are estimated from the mean, standard deviation, and skew coefficient of the logarithms of observed peak discharges.

In the absence of flow data, regional frequency analysis procedures described in section 12.5.c may be applied to develop distribution parameter predictive equations. As with the equations for rainfall-runoff model parameters, these equations relate model parameters to catchment characteristics. For example, for the Shellpot creek catchment, Delaware, the following predictive equation was developed (USACE, 1982):

$$S = 0.311 - 0.05 \log A \tag{16.4}$$

in which S = standard deviation of logarithms; and A = catchment drainage area, in sq mi. With similar equations, other parameters can be estimated.

To apply a distribution parameter-predictive equation for an ungaged catchment, the independent variables in the equation are measured or estimated for the ungaged catchment. Solution of the equation with these values yields the desired statistical distribution parameter. The frequency curve is then computed as described in EM 1110-2-1415 and chapter 12.

c. Quantile Predictive Equations. The frequency distribution quantiles for an ungaged catchment also may be defined with predictive equations. Such a predictive equation is developed by defining the frequency distributions for streams with gaged data, identifying from the distributions specified quantiles, and using regression analysis procedures to derive a predictive equation. For example, for the Red Lion creek catchment, Delaware, the following quantile predictive equation was developed (USACE, 1982):

$$Q_{100} = 1040\, A^{0.91} \tag{16.5}$$

in which Q_{100} = 100-year (0.01 probability) discharge.

16-6. Reliability of Estimates

The reliability of a runoff estimate made for an ungaged catchment is a function of the following:

1. The reliability of the flood-runoff model.

2. The form of and coefficients found for the predictive equations;

3. The talents and experience of the analyst.

a. Model Reliability. Linsley (1986) relates the results of a 1981 pilot test by the Hydrology Committee of the U.S. Water Resources Council that found that all runoff models tested were subject to very large errors and exhibited a pronounced bias to overestimate. He shows that errors of plus or minus 10% in estimating discharge for a desired 100-year (0.01 probability) event may, in fact, yield an event as small as a 30-year event or as large as a 190-year event for design. Lettenmaier (1984) categorizes the sources of error as model error, input error, and parameter error. Model error is the inability of a model to predict runoff accurately, even given the correct parameters and input. Input error is the result of error in specifying rainfall for predicting runoff, or in specifying rainfall and runoff for estimating the model parameters. This input error may be due to measurement errors or timing errors. Parameter error is the result of inability to measure properly physically-based parameters or to estimate properly calibration parameters. The net impact of these errors is impossible to quantify. They are identified here only to indicate sources of uncertainty in discharge prediction.

b. Predictive Equation Reliability. Predictive equations are subject to the same errors as runoff models. The form of and parameters of the equations are not known and must be found by trial and error. The sample size upon which the decision must be based is very small by statistical standards, because data are available for relatively few gaged catchments. Overton and Meadows (1976) go so far as to suggest that the reliability of a regionalized model can always be improved by incorporating a larger data base into the analysis. Predictive equations are subject also to input error. Many of the catchment characteristics used in predictive equations have considerable uncertainty in their measured values. For example, the accuracy of stream length and slope estimates are a function of map scale (Pilgrim, 1986). Furthermore, many of the characteristics are strongly correlated, thus increasing the risk of invalid and illogical relationships.

c. Role of Hydrologic Engineer. Loague and Freeze (1985) suggest that hydrologic modeling is more an art than a science. Consequently the usefulness of the results depends in large measure on the talents and experience of the hydrologic engineer and her or his understanding of the mathematical nuances of a particular model and the hydrologic nuances of a particular catchment. This is especially true in estimation of runoff from an ungaged catchment. The hydrologic engineer must exercise wisdom in selecting data for gaged catchments, in estimating flood-runoff model parameters for these catchments, in establishing predictive relationships, and finally, in applying the relationships.

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SIMULATION OF FLOOD HYDROGRAPHS FOR GEORGIA STREAMS

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SIMULATION OF FLOOD HYDROGRAPHS FOR GEORGIA STREAMS

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ABSTRACT

Flood hydrographs are needed for the design of many highway drainage structures and embankments. A method for simulating these flood hydrographs at urban and rural ungaged sites in Georgia is presented in this report.

The O'Donnell method was used to compute unit hydrographs from 355 flood events from 80 stations. An average unit hydrograph and an average lagtime were computed for each station. These average unit hydrographs were transformed to unit hydrographs having durations of one-fourth, one-third, one-half, and three-fourths lagtime, then reduced to dimensionless terms by dividing the time by lagtime and the discharge by peak discharge. Hydrographs were simulated for these 355 flood events and their widths were compared with the widths of the observed hydrographs at 50 and 75 percent of peak flow. The dimensionless hydrograph based on one-half-lagtime duration provided the best fit of the observed data.

Multiple-regression analysis was used to define relations between lagtime and certain physical basin characteristics, of which drainage area and slope were significant for the rural equations, with impervious area being added for the Atlanta urban equation.

A hydrograph can be simulated from the dimensionless hydrograph, peak discharge of a specific recurrence interval, and lagtime obtained from regression equations for any site of less than 500 mi^2 in Georgia.

For simulating hydrographs at sites larger than 500 mi^2, the U.S. Geological Survey computer model CONROUT, can be used. CONROUT produces a simulated outflow discharge hydrograph with a peak discharge of a specific recurrence interval. The diffusion analogy routing method with single linearization was used in this study.

INTRODUCTION

The design of many highway drainage structures and embankments requires an evaluation of the flood-related risk to the structures and to the surrounding property. Risk analyses of alternate designs are necessary to determine the design with the least total expected cost. In order to fully evaluate these risks, a runoff hydrograph with a peak discharge of specific recurrence interval may be necessary to estimate the length of time of inundation of specific features, for example, roads and bridges. For ungaged streams, this information is difficult to obtain; therefore, there is a need for a method based on Georgia hydrologic data to estimate the flood hydrograph associated with a design discharge. The objective of this study was to define techniques for simulating flood hydrographs for specific design discharges at ungaged sites in Georgia. The scope of this study was statewide for rural basins, and the Atlanta metropolitan area for urban basins up to 25 mi^2.

HYDROGRAPH SIMULATION PROCEDURE

Several traditional methods for simulating a hydrograph for a flood of selected recurrence interval at an ungaged watershed were considered for this study. However, a new procedure based on observed streamflow data was developed for this study and is presented in this section.

Basins less than 500 square miles

A dimensionless hydrograph was developed for use in basins up to 500 mi^2. Peak discharge of a selected recurrence interval and lagtime are necessary parameters to convert the dimensionless hydrograph to a simulated hydrograph for a given basin. Price (1) presents a technique for estimating the peak discharge of a selected recurrence interval for rural streams in Georgia. Inman (2) presents a technique for estimating the peak discharge of a selected recurrence interval for basins less than 25 mi^2 in the Atlanta urban area. Lagtime-estimating equations were developed for Georgia streams as part of the present study and will be presented in a later section.

The dimensionless hydrograph was developed from observed flood hydrographs. Using data from 80 basins having drainage areas less than 20 mi^2, the method is as follows:

(1) Compute a unit hydrograph and lagtime for three to five storms for each of the 80 gaging stations. All unit hydrographs should be for the same time interval (duration) at a station. Lagtime is computed as the time at the centroid of the unit hydrograph minus one-half the time of the computation interval (duration). The unit hydrograph computation method is by O'Donnell (3).

(2) Eliminate the unit hydrographs with inconsistent shapes and compute additional unit hydrographs if needed.

(3) Compute an average unit hydrograph for each station by aligning the peaks and averaging each ordinate of discharge for the final selection of unit hydrographs. The correct timing of the average unit hydrograph is obtained by averaging the time of the center of mass of the individual unit hydrographs and plotting the average center of mass at this average time. The time of the center of mass of the discharge hydrograph is obtained by adding one-half the unit hydrograph computation interval (duration) to that hydrograph's lagtime.

(4) Transform the average unit hydrographs computed in step 3 to hydrographs having durations of one-fourth, one-third, one-half, and three-fourths lagtime. These durations must be to the nearest multiple of the original duration (computation interval). These transformed unit hydrographs will have durations of 2-times, 3-times, 4-times, and 6-times the duration of the original unit hydrograph. The transformation of a short duration unit hydrograph to a long duration unit hydrograph (for instance, a 5-minute duration to a 20-minute duration) can be accomplished through the use of the following equations:

$D/\Delta t$	EQUATION	
2	TUHD(t)=1/2[TUH(t)+TUH(t-1)]	(1)
3	TUHD(t)=1/3[TUH(t)+TUH(t-1)+TUH(t-2)]	(2)
4	TUHD(t)=1/4[TUH(t)+TUH(t-1)+TUH(t-2)+ TUH(t-3)]	(3)
n	TUHD(t)=1/n[TUH(t)+TUH(t-1) ... TUH(t-n+1)],	(4)

where Δt = computation interval, (the original unit hydrograph has an actual duration equal to Δt),

D = design duration of the unit hydrograph, (this must be a multiple of Δt),

TUHD(t) = ordinates of the desired unit hydrograph at time t,

and

TUH(t), TUH(t-1), etc. = ordinates of the original unit hydrograph at times t, t-1, t-2, etc.

Duration may be thought of as actual duration or design duration, so a distinction must be made between the two. Actual duration which is highly variable may be defined as the time during which precipitation falls at a rate greater than the existing infiltration capacity. It is the actual time during which rainfall excess is occurring. Design duration is that duration which is most convenient for use on any particular basin. The design duration is that for which the unit hydrograph is computed. For this report, design duration is expressed as a fractional part of lagtime, such as one-fourth, one-third, one-half, an three-fourths lagtime. It is later shown that the design duration of one-half lagtime provides the best fit of observed data.

(5) Reduce the one-fourth, one-third, one-half, and three-fourths lagtime hydrographs to dimensionless terms by dividing the time by lagtime and the discharge by peak discharge.

(6) For Hydrologic Regions 1, 2, and 3, as defined by Price (1) and the Atlanta urban area as reported by Inman (2), compute an average dimensionless hydrograph by using the dimensionless hydrographs at the stations within that area or region. The hydrographs were computed by aligning the peaks and averaging each ordinate of the discharge ratio, Q/Q_p.

Steps 1 through 5 were done for all stations having data in the U.S. Geological Survey WATSTORE unit-values file, which had hydrographs plotted from earlier studies. A total of 355 unit hydrographs from 80 stations, including 19 Atlanta urban sites, were used to develop the one-fourth, one-third, one-half, and three-fourths lagtime duration dimensionless hydrographs. A statistical analysis to select the best fitting design duration was done by comparing the widths of hydrographs estimated (or computed) from the one-fourth, one-third, one-half, and three-fourths lagtime duration dimensionless hydrographs from each region or area with the observed hydrograph widths from their respective regions or area. The one-half-lagtime duration was the best fit of width at 50 percent of peak flow and at 75 percent of peak flow. Figure 1 illustrates plots of the one-half-lagtime duration dimensionless hydrograph for Regions 1, 2, and 3, and for the Atlanta urban area. Based on these plots, one dimensionless hydrograph was selected for both rural and urban conditions for the entire State as shown in figure 2 and table 1.

Another statistical analysis to test the accuracy of the dimensionless hydrograph application technique was done by comparing the simulated hydrograph widths at 50 and 75 percent of peak flow from simulated hydrographs using the statewide one-half lagtime duration dimensionless hydrograph with the 355 observed hydrographs. Figure 3 illustrates one example of this comparison. The results were: The 50 percent of peak-flow width comparison had a standard error of estimate of ± 31.8 percent and the 75-percent comparison had a standard error of estimate of ± 35.9 percent. The standard error of estimate of the width comparisons is based on mean-square difference between observed and simulated widths. Based on verification and bias testing, which are presented in a later section, this dimensionless hydrograph can be used for flood-hydrograph simulation for ungaged basins up to 500 mi^2. Steps 3 through 6 of the dimensionless hydrograph development and the statistical analyses were programmed for computer use by S. E. Ryan (U.S. Geological Survey, written commun., 1985).

Basins Greater Than 500 Square Miles

The method for simulating a hydrograph at basins greater than 500 mi^2 uses the U.S. Geological Survey computer model, CONROUT. The model routes streamflow from an upstream channel location to a user-defined location downstream. CONROUT is described in detail by Doyle and others (4).

CONROUT provides the user with two methods of routing: diffusion analogy and storage-continuity. The diffusion analogy method with single linearization as recommended by Keefer (5), was used in this study.

TESTING OF DIMENSIONLESS HYDROGRAPHS

Four tests are generally required to establish the soundness of models. The first test is the standard error of estimate which has been explained and presented in prior sections of this report. The other tests are for verification, bias, and sensitivity.

Verification

For verification, the dimensionless hydrograph was applied to other hydrographs not used in its development. This test included the use of 138 flood events from 37 stations having drainage areas of 20-500 mi^2 located throughout the State. The average station lagtime and peak discharge for each flood event were used to simulate a theoretical flood hydrograph, which was compared to the observed hydrograph. At the 50 and 75 percent of peak flow widths the standard errors of estimate were $\pm$ 39.5 percent and $\pm$ 43.6 percent, respectively. Figure 4 illustrates an example of this comparison.

An additional verification, or test, of the entire simulation procedure was conducted on the highest peaks (simple or compound) having unit values available in the Georgia District and a station flood-frequency curve. Thirty-one stations having drainage areas of 20-500 mi^2 were tested as follows. The recurrence interval of this observed peak discharge (Q), was determined from the station-frequency curve. The appropriate regional frequency equation from Price (1) was used to compute the corresponding peak discharge for this recurrence interval. The lagtime (T_L) for this station was computed from the appropriate regional lagtime equation. The regression Q and regression T_L were then used to simulate a flood hydrograph. A comparison of the simulated and observed hydrograph widths at 50 and 75 percent of peak flow yielded standard errors of estimate of $\pm$ 51.7 percent and $\pm$ 57.1 percent, respectively. Figure 5 illustrates an example of this comparison.

Bias

Two tests for bias were conducted, one for simulated versus observed hydrograph width, and the other for geographical bias. The width-bias test was performed on the widths at 50 percent and 75 percent of peak flow at the 31 stations used in the additional verification step. As explained earlier, these were the highest available floods at these stations. The average recurrence interval was about 30 years. The mean error, $\bar{x}$, indicated that there was a positive error (simulated greater than observed) in the hydrograph widths at 50 percent of peak flow and a negative error (observed greater than simulated) in the hydrograph widths at 75 percent of peak flow. Also, there was a negative error (estimated less than observed) in the comparison of peak Q from regional regression equations and peak Q from station frequency curves. However, the students t-test indicated that these errors are not statistically significant at the 0.01 level of significance, and therefore, the simulated hydrograph widths are not biased.

The test for geographical bias was done by comparing the widths at 50 percent and 75 percent of the ratio, Q/Q_p, of the dimensionless hydrographs simulated for Regions 1, 2, and 3 as defined by Price (1), and shown in figure 6, and for the Atlanta metropolitan area with the widths of the statewide dimensionless hydrograph. Figure 1 illustrates these four dimensionless hydrographs. There was no significant bias. In fact, the mean error, $\bar{x}$, was very small in both the 50 percent and the 75 percent test, which further confirmed the decision to use one dimensionless hydrograph statewide for basins up to 500 mi^2.

Sensitivity

The fourth test was to analyze the sensitivity of the simulated hydrograph widths to errors in the two independent variables (Q and T_L) that are used to simulate the hydrograph. This test was done by holding one variable constant and varying the other by $\pm$ 10 percent, and $\pm$ 20 percent at the hydrograph widths corresponding to 50 percent and 75 percent of peak flow. When peak Q was varied, the test results indicated that the hydrograph width did not change at 50 percent or 75 percent of that varied peak Q. When lagtime was varied, the test results indicated that the hydrograph width varied by the same percentage.

REGRESSION ANALYSIS OF LAGTIME

So that lagtime could be estimated for ungaged sites, the average station lagtimes obtained from the stations used in the dimensionless hydrograph development were related to their basin characteristics. This was done by the linear, multiple-regression method described by Riggs (6). Lagtimes were computed for each flood event with the same program that computed the t-hour unit hydrographs. These storm-event lagtimes were then averaged to compute an average station lagtime, which was in turn used in the regression analyses. Lagtime is generally considered to be constant for a basin and is defined by Stricker and Sauer (7) as the time from the centroid of rainfall excess to the centroid of the runoff hydrograph. Lagtime for the 19 Atlanta urban stations was analyzed separately, owing to the effect of urbanization on lagtime.

The regression equations provide a mathematical relation between the dependent variable (lagtime) and the independent variables (the basin characteristics found to be statistically significant). All variables were transformed into logarithms before analysis to: (1) obtain a linear regression model, and (2) achieve equal variance about the regression line throughout the range. In the analyses performed, a 95-percent confidence limit was specified to select the significant independent variables.

The independent variables, or physical basin characteristics, are defined in the following paragraphs.

Lagtime (T_L).--The elapsed time, in hours, from the centroid of rainfall excess to the centroid of the resultant runoff hydrograph. Lagtime is computed from the unit hydrograph.

Drainage area (A).--Area of the basin, in square miles, planimetered from U.S. Geological Survey 7 1/2-minute topographic maps. Basin boundaries were all field checked.

Channel slope (S).--The main channel slope, in feet per mile, as determined from topographic maps. The main channel slope was computed as the difference in elevation, in feet, at the 10- and 85-percent points divided by the length, in miles, between the two points.

Channel length (L).--The length of the main channel, in miles, as measured from the gaging station upstream along the channel to the basin divide.

$L/S^{0.5}$.--A ratio, where L and S have been previously defined.

Measured total impervious area (IA).--The percentage of drainage area that is impervious to infiltration of rainfall. This parameter was determined by a grid-overlay method using aerial photography. According to Cochran (8) a minimum of 200 points, or grid intersections, per area or subbasin will provide a confidence level of 0.10. Three counts of at least 200 points per subbasin were obtained and the results averaged for the final value of measured total impervious area. On several of the larger basins where some development occurred during the period of data collection, this parameter was determined from aerial photographs made in 1972 (near the beginning of data collection), and then averaged with the values obtained from aerial photographs made in 1978 (near the end of data collection).

Measured effective impervious area (MEIA).--The percentage of impervious area which is directly connected to the channel drainage system. Noneffective impervious area, such as house rooftops that drain onto a lawn, are subtracted from this total. This parameter was obtained in conjunction with measured total impervious area. When the minimum of 200 points were counted, three totals per subbasin were obtained. The first total was pervious points, the second definite impervious points such as streets and parking lots, and the third rooftops. One building out of three was field checked to determine the percentage of effective impervious area of its roof and gutter system. An average percent effective impervious area was determined for the buildings field checked in the subbasin, and this factor was multiplied by the total number of building points. The resulting product was added to the definite impervious points, and this total of effective impervious area points was divided by the total number of points counted in the subbasins to determine the MEIA percentage.

Regionalization

The initial regression run utilized data from 91 rural stations, of less than 500 mi^2, located throughout the State. A geographical bias was detected. The area north of the Fall Line, consisting of Regions 1 and 2 as defined by Price (1), and shown in figure 6, tended to overpredict lagtime, whereas, the area south of the Fall Line, consisting of Regions 3, 4, and 5 as defined by Price (1), and shown in figure 6, tended to underpredict lagtime.

The next step was to make separate regression runs for each of the five regions. Region 1 had no equations with two or more variables significant at the 95-percent confidence limit. The standard error of estimate of the regression using only one variable ranged from 43 to 51 percent. Such large standard errors are not desirable. Region 2, also, had no equations with two or more variables significant at the 95-percent confidence limit. The standard error of estimate of the regression using only one variable ranged from 34 to 37 percent, with a tendency to overpredict on the lower end of the curve and underpredict on the upper end.

Regions 1 and 2 were combined and analyzed as one region. Two equations each have two variables significant at the 95-percent confidence limit. The equation selected was lagtime (T_L) = $4.64A^{0.49}\ S^{-0.21}$. Region 4 had (5) only five stations, and Region 5 only three. Therefore, neither region could be analyzed separately. Regions 3, 4, and 5 were combined and analyzed as one region. Only one equation had two variables significant at the 95-percent confidence limit. The equation was $T_L = 13.6A^{0.43}\ S^{-0.31}$. (6)

The Atlanta urban area was analyzed separately due to the effects of urbanization on lagtime. IA and MEIA were added as independent variables in the analysis. The equation that was selected, $T_L = 161A^{0.22}\ S^{-0.66}\ IA^{-0.67}$, (7) is similar to the rural equations, in that both rural and urban equations have area and slope as independent variables. Impervious area accounts for the urbanization effect. Drainage area, (A), had a significance level of 6.8 percent, but was retained in order to provide continuity with the rural equations. The Atlanta urban equation (7) should be considered preliminary, and subject to revision after more urban data are analyzed in the Rome, Athens, Augusta, and Columbus metropolitan areas. If these additional data show the same regionalization pattern as the rural data north of the Fall Line, then these data will be analyzed with the Atlanta data, which could possibly change the Atlanta urban equation.

The accuracy of regression equations can be expressed by two standard statistical measures: The coefficient of determination, R-square (the correlation coefficient squared); and the standard error of regression. R-square measures how much variation in the dependent variable can be accounted for by the independent variables. For example, an R-square of 0.94 would indicate that 94 percent of the variation is accounted for by the independent variables, and that 6 percent is due to other factors. The standard error of regression (or estimate) is, by definition, one standard deviation on each side of the regression line and contains about two-thirds of the data within this range. A summary of the lagtime equations and their related statistics are given in table 2.

Limits of Independent Variables

The effective usable range of basin characteristics for the rural equations are as follows:

North of the Fall Line

Variable	Minimum	Maximum	Units
A	0.3	500	square miles
S	5.0	200	feet per mile

South of the Fall Line

Variable	Minimum	Maximum	Units
A	0.2	500	square miles
S	1.3	60	feet per mile

The effective usable range of basin characteristics for the Atlanta urban equation is as follows:

Variable	Minimum	Maximum	Units
A	0.2	25	square miles
S	13	175	feet per mile
IA	14	50	percent

TESTING OF LAGTIME REGRESSION EQUATIONS

The lagtime regression equations were tested with the same four tests as the dimensionless hydrograph. The standard error of estimate has been explained and presented in a prior section of this report. Verification, bias, and sensitivity are the other tests.

Verification

Split-sample testing is the process by which part of a data set is used for calibration and the remaining part for verification or prediction. The standard error of estimate, obtained from the calibration phase, is a measure of how well the regression equations will estimate the dependent variable at the sites used to calibrate them. The standard error of prediction, on the other hand, is a measure of how well the regression equations will estimate the dependent variable at other than calibration sites according to Sauer and others (9). Split-sample testing was used for verification of the regression equations, both north and south of the Fall Line. It was also used to estimate the magnitude of the average prediction error, and to determine whether the same variables were significant. The stations from each region were divided into two groups of about equal size. The sites were arrayed in ascending order according to drainage-area magnitude. The odd-numbered events made up the first sample and the even-numbered events the second sample. Multiple-regression analyses were performed on both regions using only the sites in one of the samples, then recalibrated using the sites in the other sample. The results were all acceptable, as shown in table 3. The regression analyses yielded new regression equations similar to the equations originally developed using all the sites in each region.

The first set of equations tentatively selected had area (A) and $L/S^{0.5}$ as the two independent variables. The standard errors of regression were about the same as for the equations with A and slope (S) as independent variables for both regions. However, when split-sample testing was performed, $L/S^{0.5}$ was not significant at the 95-percent confidence limit for either odd or even sample above the Fall Line. The equation with A and $L/S^{0.5}$ was split-sample tested for the area south of the Fall Line with A not being significant at the 95-percent confidence limit for either the odd or even sample. No attempt was made to analyze the Atlanta urban equation with split-sample testing because of the limited number of stations available.

Bias

Two tests for bias were performed, one for variable bias and the other for geographical bias. The variable-bias tests were made by plotting the residuals (difference between observed and predicted lagtime) versus each of the independent variables for all stations. These plots were visually inspected to determine whether there was a consistent overprediction or underprediction within the range of any of the independent variables. These plots also verified the linearity assumptions of the equations. The equations were found to be free of variable bias throughout the range of all independent variables.

Geographical bias was tested by plotting the residuals of observed lagtimes minus predicted lagtimes on a State map. The plot was visually inspected to determine if any area of the State consistently overestimated or underestimated. Because this test indicated no consistent overestimation or underestimation in any part of the State, it can be concluded that no geographical bias exists.

The same bias analyses were perfomed on the Atlanta urban equation. There was no geographical or variable bias.

Sensitivity

The fourth test was to analyze the sensitivity of lagtime to errors in the two independent variables in the regression equations. The computation of these independent variables is subject to errors in measurement and judgement. To illustrate the effect of such errors, the equations were tested to determine how much error was introduced into the computed lagtime from specified percentage errors in the independent variables. The test results are shown in tables 4 and 5. These tables were computed by assuming that all independent variables were constant, except the one being tested for sensitivity.

The Atlanta urban equation was tested for sensitivity of lagtime to errors in the three independent variables in the same manner as the two rural equations. The test results are shown in table 6.

SUMMARY

A dimensionless hydrograph was developed for Georgia streams having drainage areas of less than 500 mi^2. This dimensionless hydrograph can be used to simulate flood hydrographs at ungaged sites for both rural and urban streams statewide. Over 350 observed flood hydrographs were used for its development. For verification, the dimensionless hydrograph was applied to 169 flood hydrographs not used in its development.

Multiple-regression analysis was used to define relations between lagtime and selected basin characteristics, of which drainage area and slope were significant for the rural basins, and drainage area, slope, and impervious area were significant for the Atlanta urban basins. The rural equation was regionalized into one equation for the area north of the Fall Line, and one equation for the area south of the Fall Line. Both rural equations were verified by split-sample testing. There was no variable or geographical bias in either the rural equation or the Atlanta urban equation. Sensitivity tests indicated drainage area as the most sensitive basin characteristic in the rural equations, and impervious area as the most sensitive in the Atlanta urban equation.

A simulated flood hydrograph may be computed by applying lagtime, obtained from the proper regression equation, and peak discharge of a specific recurrence interval, to the dimensionless hydrograph. The coordinates of the runoff hydrograph can be computed by multiplying lagtime by the time ratios and peak discharge by the discharge ratios in table 1.

For basins larger than 500 mi^2 the U.S. Geological Survey computer model CONROUT is used for simulating flood hydrographs. CONROUT routes streamflow from an upstream channel location to a user-defined location downstream. The product of CONROUT is a simulated outflow discharge hydrograph with a peak of a specific recurrence interval at the end of a reach.

ACKNOWLEDGMENTS

This study was carried out by the U.S. Geological Survey in cooperation with the Georgia Department of Transportation. Hourly rainfall records were obtained from monthly publications of the National Climatic Data Center.

The guidance and technical assistance of hydrologists in the U.S. Geological Survey and particularly Vernon B. Sauer are recognized and greatly appreciated. Also, the computer programming contributions of S. E. Ryan, Hydrologist, U.S. Geological Survey, have been invaluable to this study.

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LIST OF TABLES

Table 1.--Time and discharge ratios of the statewide dimensionless hydrograph.

Time ratio (t/T_1)	Discharge ratio (Q/Q_p)
0.25	0.12
.30	.16
.35	.21
.40	.26
.45	.33
.50	.40
.55	.49
.60	.58
.65	.67
.70	.76
.75	.84
.80	.90
.85	.95
.90	.98
.95	1.00
1.00	.99
1.05	.96
1.10	.92
1.15	.86
1.20	.80
1.25	.74
1.30	.68

Table 1.--Time and discharge ratios of the statewide dimensionless hydrograph--continued.

Time ratio (t/T_L)	Discharge ratio (Q/Q_p)
1.35	.62
1.40	.56
1.45	.51
1.50	.47
1.55	.43
1.60	.39
1.65	.36
1.70	.33
1.75	.30
1.80	.28
1.85	.26
1.90	.24
1.95	.22
2.00	.20
2.05	.19
2.10	.17
2.15	.16
2.20	.15
2.25	.14
2.30	.13
2.35	.12
2.40	.11

Table 2.--Summary of lagtime estimating equations.

Area	Equation	Standard error of regression (percent)	Coefficient of determination, R^2
North of the Fall Line (rural)	$T_L = 4.64A^{.49}S^{-.21}$	$\pm$ 31	0.94
South of the Fall Line (rural)	$T_L = 13.6A^{.43}S^{-.31}$	$\pm$ 25	.96
Metropolitan Atlanta (urban)	$T_L = 161A^{.22}S^{-.66}IA^{-.67}$	$\pm$ 19	.94

Table 3.--Lagtime equations split-sample test results.

Area	Number of stations	Equation	Standard error of regression (percent)	Standard error of prediction (percent)	Coefficient of determination, R^2
North of the Fall Line (odd)	25	$T_L = 4.88A^{0.48}S^{-0.22}$	$\pm$ 32	--	0.94
North of the Fall Line (even)	24	--	--	$\pm$ 32	.93
North of the Fall Line (even)	24	$T_L = 4.51A^{0.50}S^{-0.21}$	$\pm$ 31	--	.94
North of the Fall Line (odd)	25	--	--	$\pm$ 32	.94
South of the Fall Line (odd)	21	$T_L = 36.8A^{0.35}S^{-0.57}$	$\pm$ 18	--	.98
South of the Fall Line (even)	21	--	--	$\pm$ 41	.92
South of the Fall Line (even)	21	$T_L = 8.63A^{0.48}S^{-0.21}$	$\pm$ 26	--	.96
South of the Fall Line (odd)	21	--	--	$\pm$ 29	.96

--Data not applicable.

Table 4.--Sensitivity of computed lagtime to errors in independent variables with the north of the Fall Line equation.

Percent error in independent variable	INDEPENDENT VARIABLES (Percent error in computed lagtime)	
	Area	Slope
+50	+21.9	-8.2
+25	+11.5	-4.6
+10	+4.8	-2.0
-10	-5.0	+2.2
-25	-13.1	+6.2
-50	-28.5	+15.7

Table 5.--Sensitivity of computed lagtime to errors in independent variables with the south of the Fall Line equation.

Percent error in independent variable	INDEPENDENT VARIABLES (Percent error in computed lagtime)	
	Area	Slope
+50	+19.2	-11.8
+25	+10.1	-6.7
+10	+4.2	-2.9
-10	-4.5	+3.3
-25	-11.7	+9.4
-50	-25.9	+24.1

Table 6.--Sensitivity of computed lagtime to errors in independent variables with the Atlanta urban equation.

Percent error in independent variable	INDEPENDENT VARIABLES (Percent error in computed lagtime)		
	Area	Slope	Impervious area
+50	+9.9	-23.4	-23.9
+25	+5.4	-13.5	-14.0
+10	+2.7	-5.9	-6.3
-10	-2.2	+7.2	+7.2
-25	-5.9	+21.2	+21.2
-50	-14.0	+58.1	+59.0

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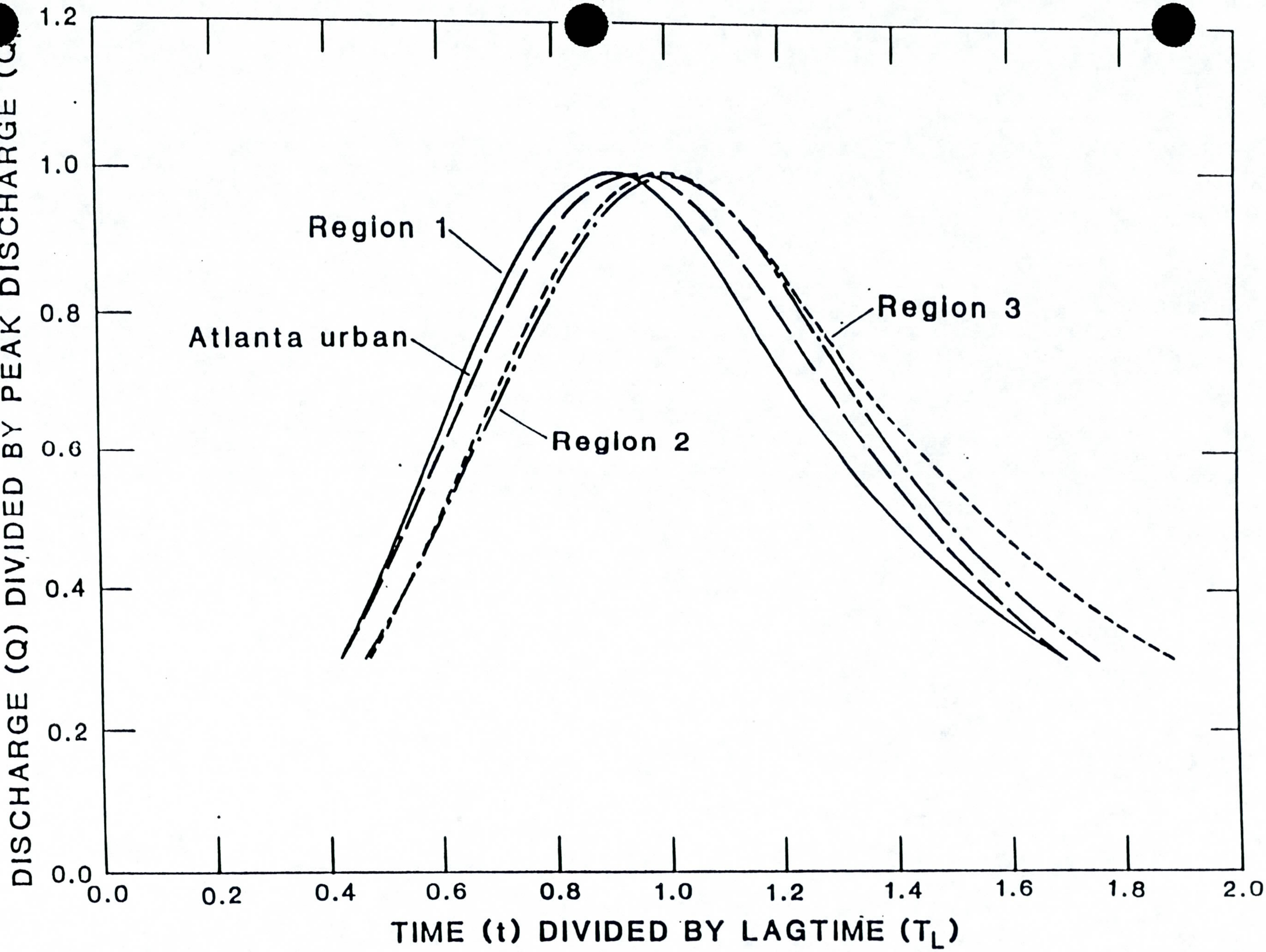

Figure 1.—Plot of the average, one-half-lagtime duration, dimensionless hydrographs for Regions 1, 2, 3, and the Atlanta urban area.

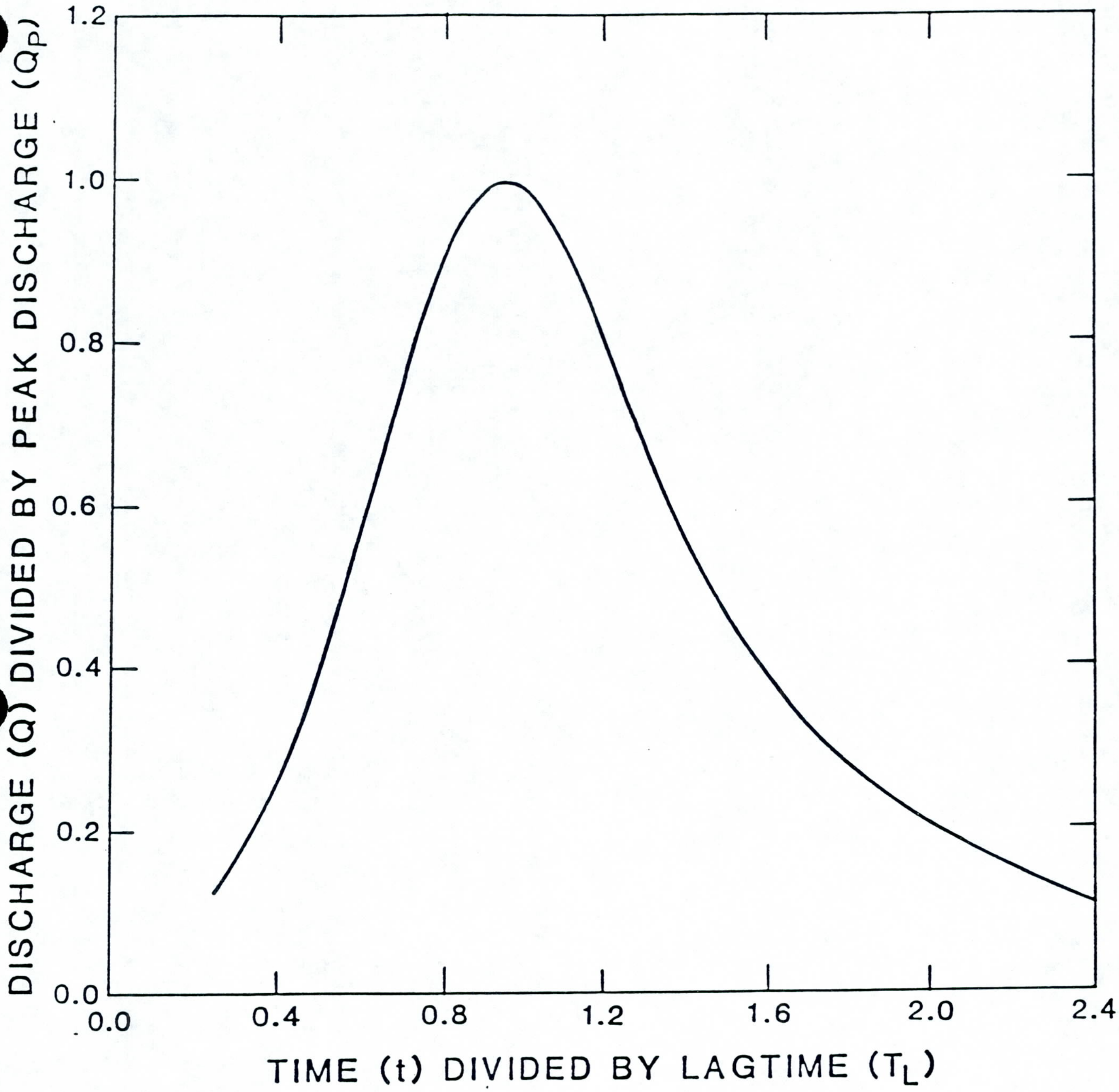

Figure 2.—Statewide dimensionless hydrograph.

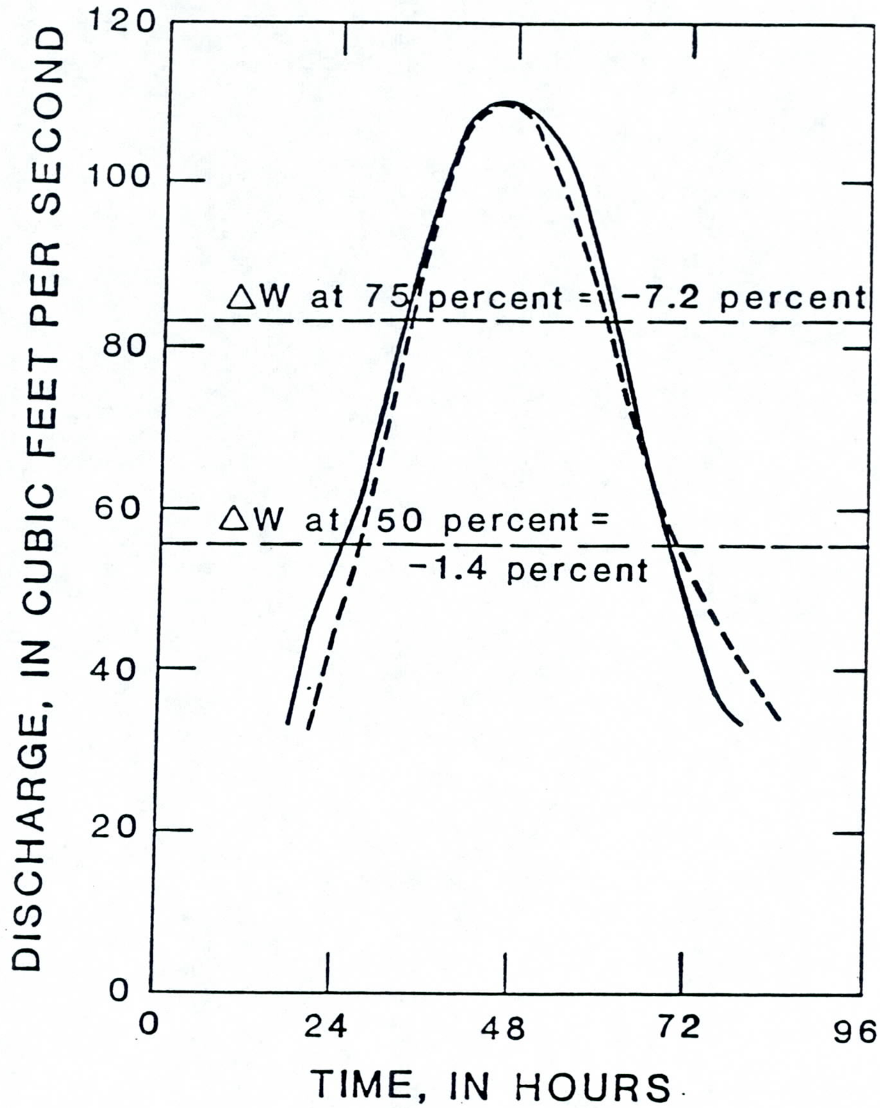

Figure 3.—Plot of observed and predicted hydrographs showing width comparisons at 50 and 75 percent of peak flow for an Atlanta urban station.

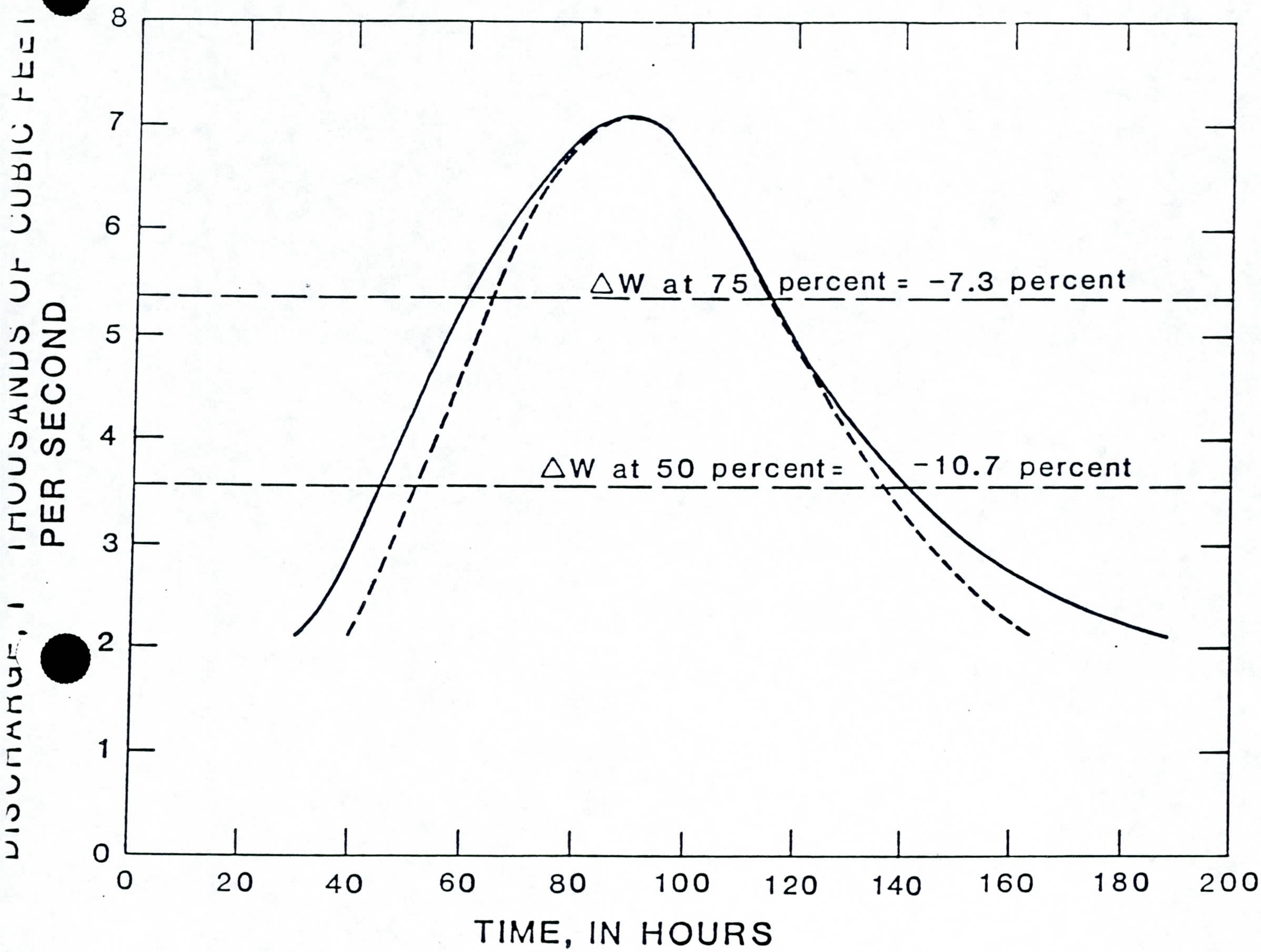

Figure 4.—Plot of observed and predicted hydrographs for width comparisons at 50 and 75 percent of peak flow for Spring Creek near Iron City.

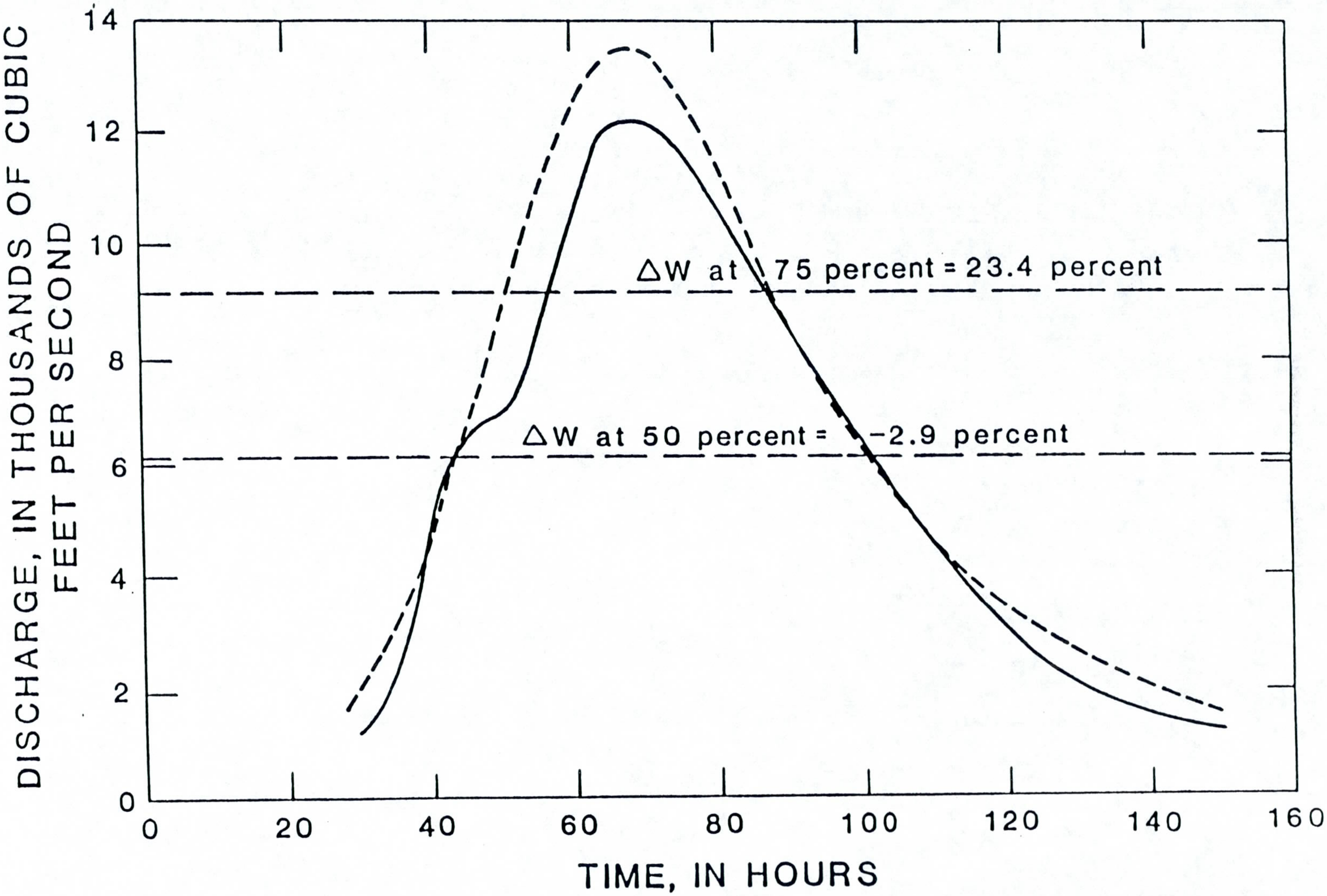

Figure 5.—Plot of observed and predicted hydrographs for width comparisons at 50 and 75 percent of peak flow for Flint River near Griffin.

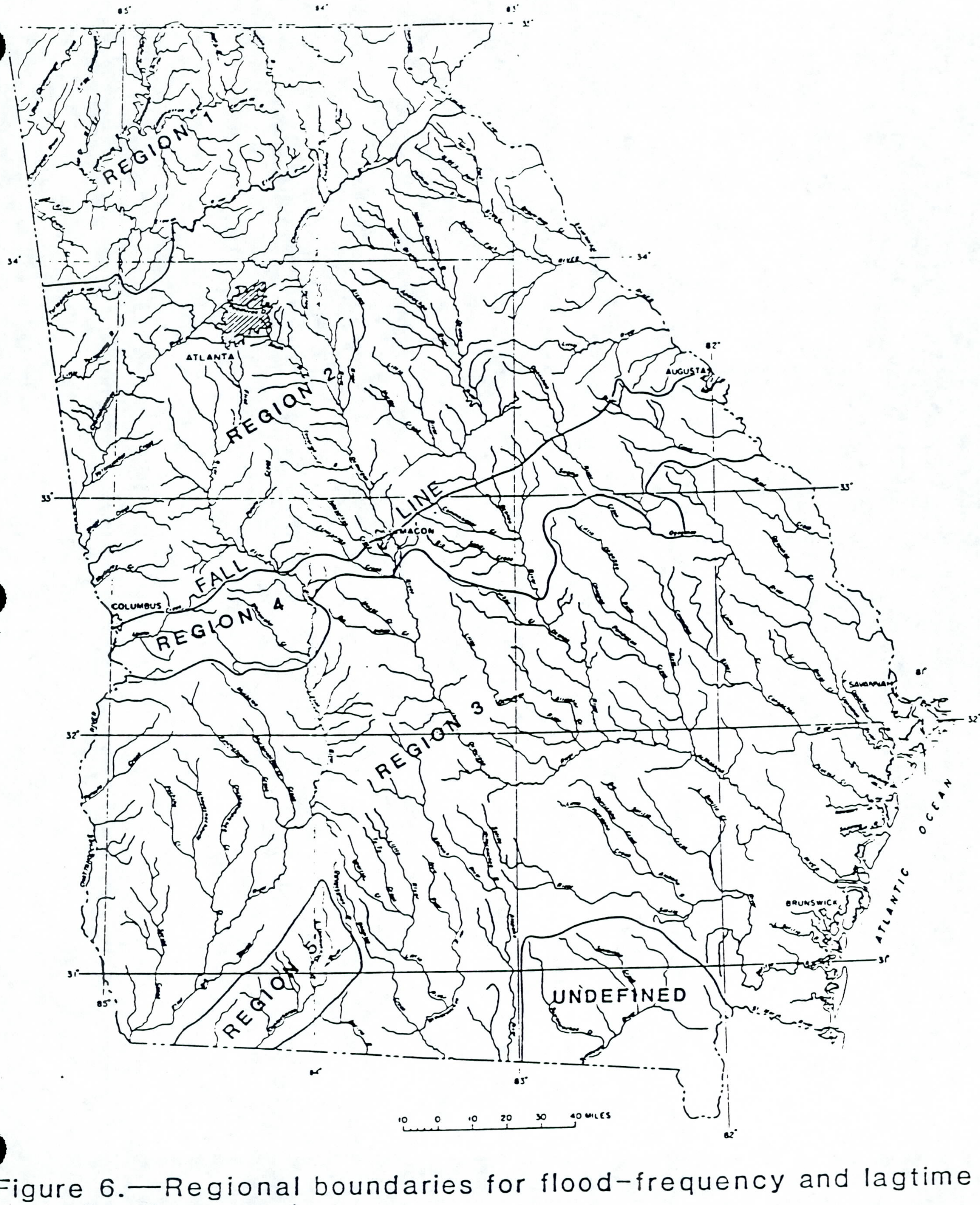

Figure 6.—Regional boundaries for flood-frequency and lagtime estimating equations.

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UNIT HYDROGRAPHS FOR DEVELOPING DESIGN FLOOD HYDROGRAPHS[1]

Krishan P. Singh[2]

ABSTRACT: In Illinois, a procedure has been developed to derive unit hydrographs for generating 100-year and probable maximum flood hydrographs, on the basis of 11 parameters that define the hydrograph shape very well. Regional regressions of these parameters with basin factors show very high correlation. Thus satisfactory values of parameters can be determined for ungaged areas or those with a few years' record. The nonlinearity in unit hydrographs derived from usual floods is largely attributed to mixing within-channel and overbank-flow flood events. To minimize the effects of nonlinearity and to derive unit hydrographs suitable for calculating spillway design floods, use of the proposed method of developing such hydrographs is recommended.
(KEY TERMS: unit hydrographs; 100-year flood; maximum probable flood; unit hydrograph peak; time to peak; time base; regional study; regional analyses.)

INTRODUCTION

Dam failure caused by overtopping during very high flood conditions results mainly from inadequate spillway capacity and insufficient freeboard. The Corps of Engineers and many state agencies have been preparing inspection reports or having them prepared by consultants to meet the goals of the National Dam Safety Program under PL 92-367 – The National Dam Inspection Act. These inspection reports contain hydraulic and hydrologic evaluations of the adequacy of the spillway and dam to handle floods of various frequencies without endangering the structure or causing dam failure due to overtopping. These evaluations require information on storms of various frequencies and probable maximum storms, their depth-area-duration relations, and the soil moisture conditions at the beginning of a design storm, as well as suitable unit hydrographs for converting design storms into flood hydrographs.

Most of the methods in use suffer from shortcomings such as 1) not enough data to satisfactorily delineate the unit hydrograph shape; 2) assumptions of unique, linear storage-discharge relationship for both in-channel and overbank flood flow; 3) use of only some of the explanatory variables; 4) lack of adjustments to make unit hydrographs suitable for simulation of floods needed for dam safety evaluations and dam design; and 5) adherence to functional relationships developed in one area for use in other areas with different climate, soils, and land topography.

Snyder (1938) analyzed a number of hydrographs from drainage areas in the Appalachian Mountain region and developed the following equations:

$$t_p = C_t (L L_c)^{0.3} \tag{1}$$

$$t_r = t_p / 5.5 \tag{2}$$

$$q_p = 640\, C_p / t_p \tag{3}$$

$$t_{pR} = t_p + 0.25\, (t_R - t_r) \tag{4}$$

$$q_{pR} = 640\, C_p / t_{pR} \tag{5}$$

in which t_p = lagtime from the midpoint of the effective rainfall of duration t_r to the peak of the unit hydrograph, hr; t_R = duration of effective rainfall other than standard t_r, hr; t_{pR} = time lag with effective rainfall duration t_R, hr; q_p = peak discharge for

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standard duration t_r, cfs/mi^2 or cfsm; q_{pR} =peak discharge for duration t_R; L = river length in miles from the given station to the upstream limit of the drainage area; L_c = river miles from the basin outlet to the center of gravity of the drainage area; and C_t and C_p are coefficients, depending on units and basin characteristics. The t_p (= t_p – 0.5 t_r in Figure 1 because Snyder's t_p is from center of effective rainfall to the hydrograph peak), t_r, and q_p (or U_p if effective rainfall is 1 inch) are shown in Figure 1. The average values of C_t and C_p have been found to be, respectively, 2.0 and 0.63 in the fairly mountainous Appalachian Highlands.

Snyder's equations give values only of t_{pR} and q_{pR} for a given t_R. The U.S. Army Corps of Engineers (COE, 1959) developed the following relations to help in sketching a unit hydrograph:

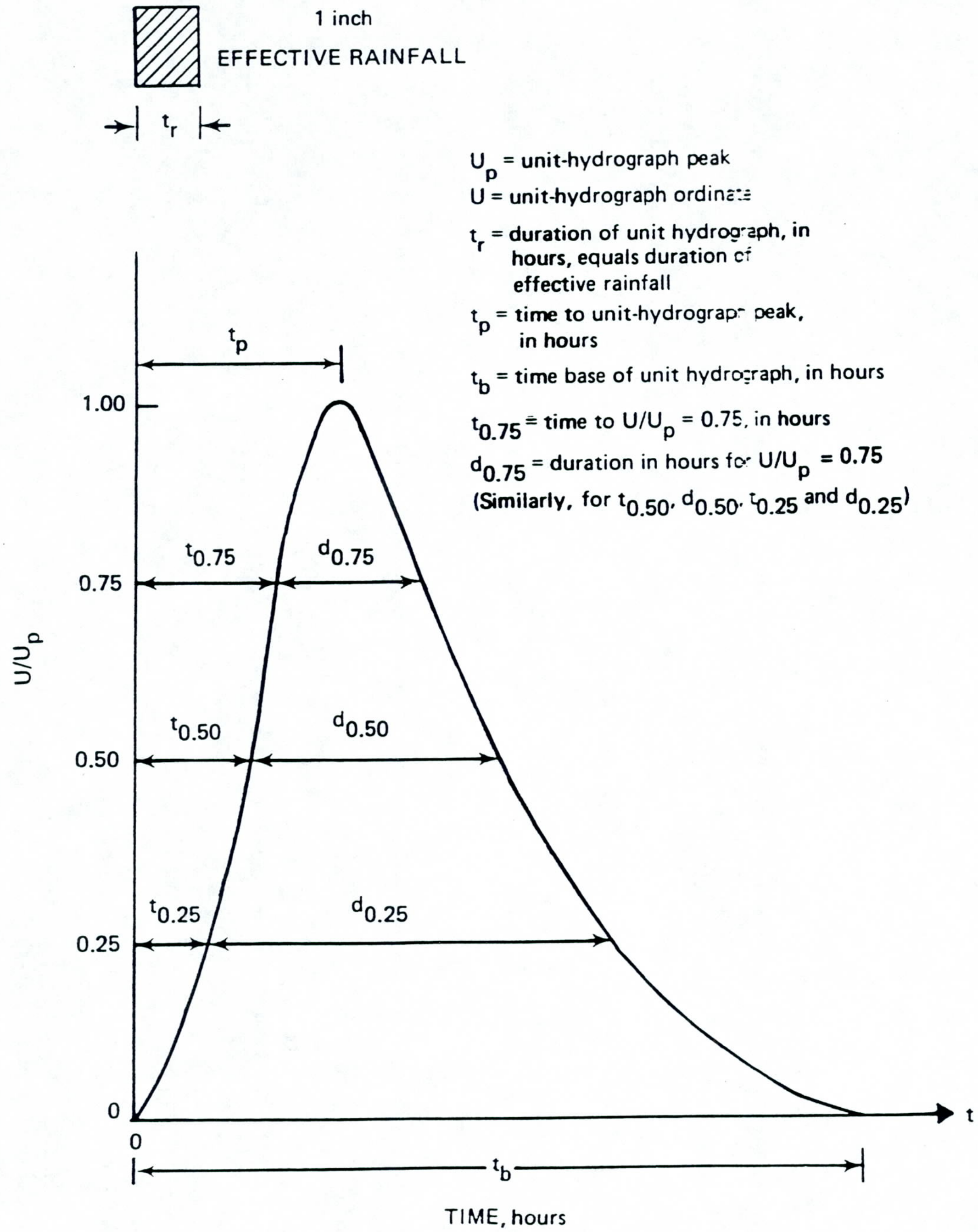

Figure 1. Unit Hydrograph Parameters – Definition Sketch.

$$W_{75} = 440 / q_{pR}^{1.08} \tag{6}$$

$$W_{50} = 770 / q_{pR}^{1.08} \tag{7}$$

in which W_{75} and W_{50} are widths of the unit hydrograph in hours at discharges 75 percent and 50 percent of the peak discharge. Widths are taken as functions of peak discharge and not of the time base because the latter is considerably affected by the method of baseflow separation used as well as by minor rainfall closely following a significant storm event. Snyder (1938) proposed the following equation to estimate the time base, t_b, in days of the unit hydrograph:

$$t_b = 3 + t_p / 8 \tag{8}$$

in which t_p is in hours. The shape of the unit hydrograph can only be roughly drawn from t_R, q_{pR}, W_{75}, W_{50}, t_{pR}, and t_b. Assumptions of a minimum t_b = 3 days and $W_{50} = 1.75\ W_{75}$ are open to question.

UNIT HYDROGRAPH FOR DAM DESIGN AND SAFETY STUDIES

Presently used unit hydrograph procedures may be suitable for deriving 1.1- to 5-year floods because of the averaging processes inherent in these procedures and the use of small to medium-sized flood events used for deriving unit hydrographs. For spillway and dam design and safety evaluations, unit hydrographs suitable for deriving 100-year flood and probable maximum flood (PMF) hydrographs are needed. Notwithstanding the principle of linearity of the unit hydrograph, it is common knowledge that unit hydrographs derived from very high floods generally yield higher peaks and shorter times to the peak than those derived from small- to medium-sized floods, although the degree of increase in peak and decrease in time to peak varies from basin to basin and region to region, depending on the physiographic, channel, and basin factors. It can be assumed that the unit hydrograph derived for developing a 100-year flood hydrograph will also be satisfactory for developing a PMF hydrograph, because the portion of flood discharge carried in bankfull channel section is rather small in comparison with the 100-year flood. This is true also for the PMF.

For satisfactory delineation of a unit hydrograph, information is needed not only on the unit hydrograph widths at 75, 50, and 25 percent of the peak discharge, but also on the time to reach these discharges from the beginning of the unit hydrograph. This provides coordinates for nine points in the discharge-time space for satisfactory delineation of the unit hydrograph. Obviously, the time base of the unit hydrograph given by Equation (8) is too long for small drainage basins and needs to be evaluated carefully. The ratio of t_p and t_r is given a constant value of 5.5 by Snyder (1938), but this value depends on basin factors.

The desirable parameters for delineating a unit hydrograph are shown in Figure 1. Effective rainfall is 1 inch, as is the runoff under the unit hydrograph. Conversion factors are: 1 inch = 25.4 mm; 1 ft^3/sec = 0.0283 m^3/sec; 1 cfs = 0.00155/A in/hr where A is drainage area in sq. mi.

UNIT-HYDROGRAPH PARAMETERS

Hickory Creek above Lake Bloomington, Illinois, is used here for illustrating the determination of unit-hydrograph parameters. Pertinent data for the basin above USGS (U.S. Geological Survey) gaging station 05565000 are:

Drainage Area	9.81 mi^2
Main Channel Length	6.74 mi
Main Channel Slope	11.88 ft/mi
Flow Record	1939-1959
Annual Maxima (top 8 values)	1690, 1460, 1050, 930, 890, 855, 820, and 680 cfs

The stage hydrographs and the storms associated with the top eight floods were examined to select four flood events such that their flood hydrographs (obtained by transforming the stage hydrographs with the rating tables) were well-defined and sharp-peaked, and had low baseflow. High flood events were chosen because suitable unit hydrographs for developing design flood hydrographs are needed.

A baseflow separation method (Singh and Stall, 1971) was then applied. This method considers the baseflow recession curve (at the end of the flood event) projected backwards to the time corresponding to the inflection point on the falling limb of the flood hydrograph. This corresponds to the peak of the baseflow hydrograph. This is joined by a smooth curve to the beginning point of the flood hydrograph. The overall curve defines the baseflow hydrograph from the beginning to the end of the flood hydrograph.

After the baseflow separation, the surface runoff hydrographs were derived for each of the four events. The duration of the effective rainfall was estimated from the basin hyetograph, and the rainfall intensity was assumed uniform over the duration because of the small-duration, intense storms. The rainfall excess was obtained from the surface runoff hydrograph (or the flood hydrograph minus the baseflow hydrograph).

A computer program calculated the unit hydrograph and the S-hydrograph (Chow, 1964) with duration of effective rainfall as well as with two durations somewhat higher and two durations somewhat lower than the effective rainfall duration. An S-hydrograph is constructed by summing a series of identical unit hydrographs spaced at intervals equal to the duration of the effective rainfall. It corresponds to a continuous effective rainfall at a constant rate of one inch per t_r hours for an indefinite period. A suitable unit-hydrograph duration was selected based primarily on closeness to the already estimated duration and the smoothness of S-curves derived by assuming shorter or longer durations. The derived unit hydrographs are given below (Table 1).

The date refers to the day the observed flood peak occurred; t_r, t_p, and t_b are in hours; SRO denotes surface runoff in inches; Q_s and U_p are the surface runoff hydrograph peak and unit hydrograph peak, respectively, in cfs; and T is the recurrence interval in years. The recurrence interval for the flood peak was derived from the annual peak series, with record length varying from about 25 to 65 years.

Unit hydrographs of the selected flood events were examined to determine a suitable duration for all four events by using the S-hydrograph method (Chow, 1964) from the unit hydrographs obtained earlier. An effective rainfall duration of 1.25 hours was selected from these analyses. The computed values of U_p and t_p with this duration were plotted with respect to T, and their values for T = 100 were determined by extrapolating the fitted curves (Figure 2). The final unit hydrograph with these expected values of U_p and t_p for T = 100 is shown in Figure 2. This unit hydrograph is considered suitable for deriving 100-year flood and PMF hydrographs because it reflects the fully developed floodplain flow conditions.

The rate of change in unit-hydrograph peak flow for a small change in the unit-hydrograph duration (say from t_r to t_R) can be written as:

$$-a\, dt_r = dU_p \tag{10}$$

integrating between t_r and t_R,

$$U_{pR} = U_p - a(t_R - t_r) \tag{11}$$

in which t_R refers to the new duration and a is positive. $U_{pR} < U_p$ if $t_R > t_r$ and vice versa. The values of a are obtained by deriving unit hydrographs for various values of t_R with the S-hydrograph method. The unit hydrograph parameters obtained from Figure 2 are t_r = 1.25 hours; U_p = 1200 cfs (from U_p vs. T curve); $t_{0.25}$ = 1.75 hours; $t_{0.50}$ = 2.15 hours; $t_{0.75}$ = 2.60 hours, t_p = 3.50 hours; $d_{0.75}$ = 2.50 hours, $d_{0.50}$ = 4.45 hours; $d_{0.25}$ = 7.60 hours, t_b = 18.5 hours; and a = 120 cfs/hr. The hydrograph shape in the recession for discharge 0.25 U_p to zero should be approximated by a curve asymptotic to the time axis and not by a straight line.

REGIONALIZATION OF UNIT-HYDROGRAPH PARAMETERS

Regionalization of unit-hydrograph parameters not only reduces bias and errors associated with a single station but also provides relationships for evaluating these parameters for an ungaged area in a hydrologically and meteorologically homogeneous region. An example is given here from a study (Singh, 1981) conducted for derivation and regionalization of unit hydrograph parameters for Illinois. As shown in Figure 3, the state was divided into eight hydrologically homogeneous regions on the basis of

TABLE 1. Unit Hydrographs for Four Flood Events.

Date	t_r (hr)	t_p (hr)	t_b (hr)	SRO (inch)	Q_s (cfs)	U_p (cfs)	T (yr)
April 22, 1944	1.00	4.5	21.0	1.08	1,037	960	7.3
April 25, 1950	2.00	5.0	29.0	0.95	783	824	3.7
July 9, 1951	1.25	2.5	27.0	1.55	1,667	1,075	22.0
July 5, 1953	1.25	3.0	30.0	1.08	923	855	5.5

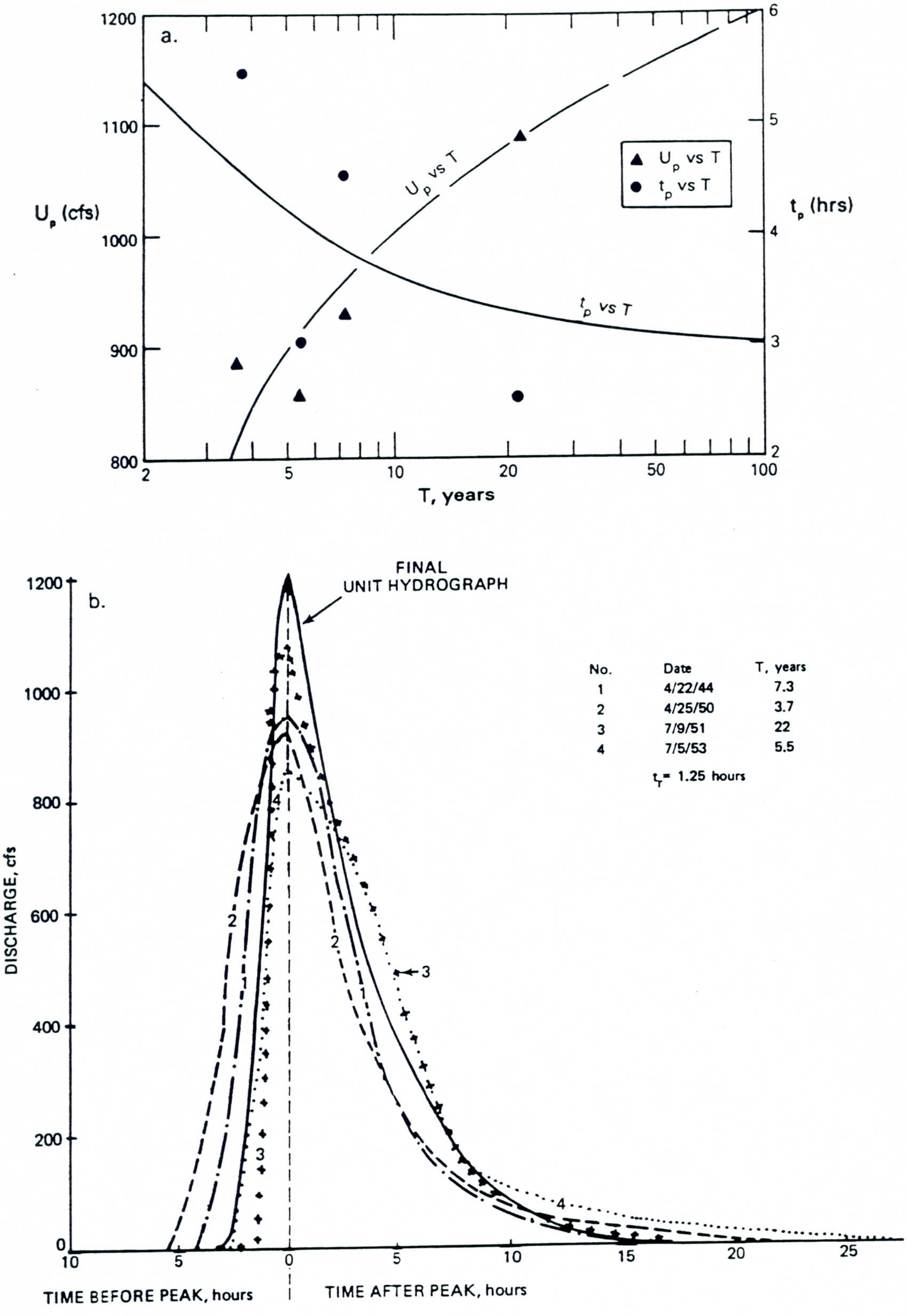

Figure 2. Unit Hydrographs for Hickory Creek Above Lake Bloomington.

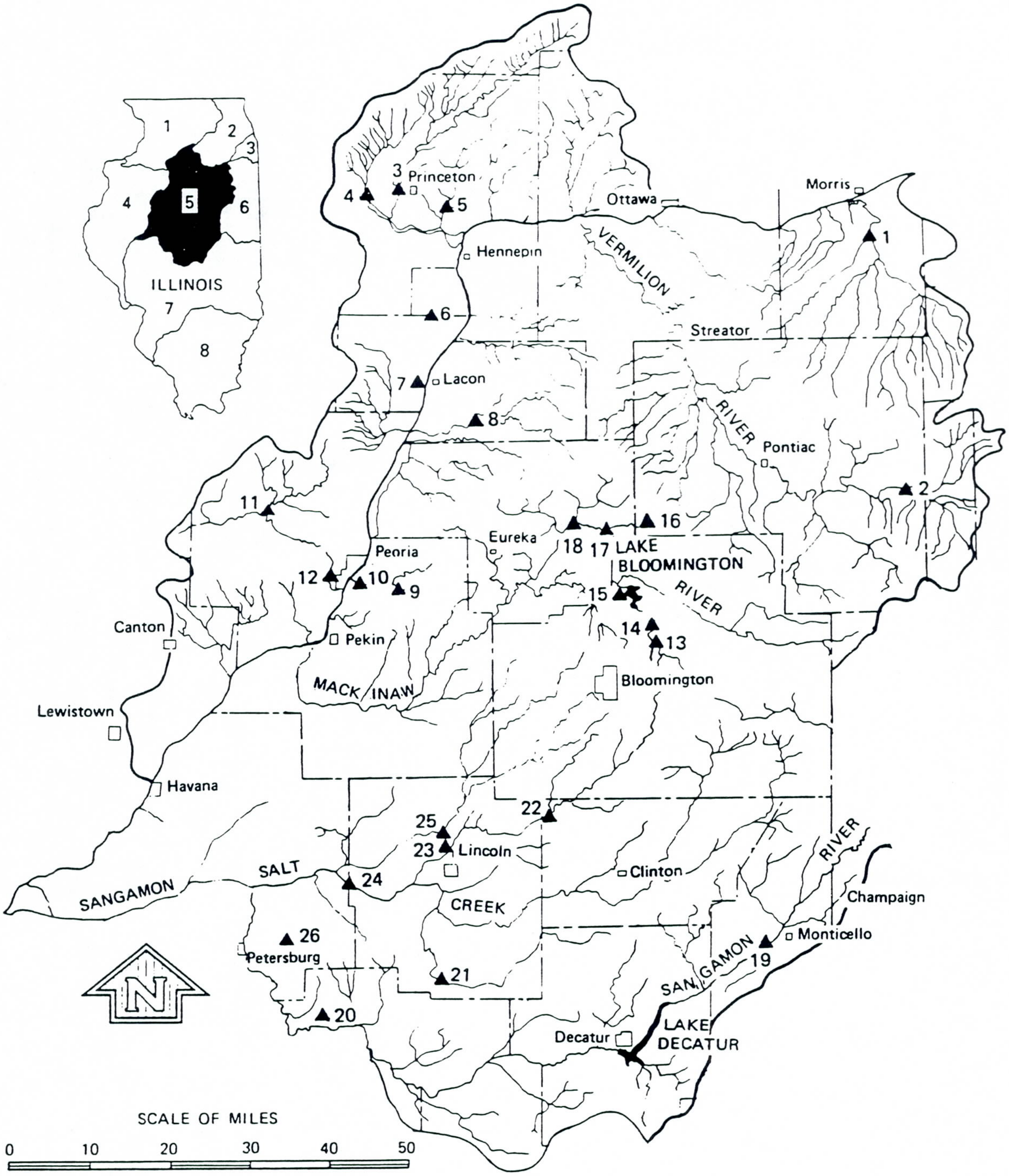

Figure 3. Study Basins in Region 5.

physiography, hydrology, and meteorology. The regionalization study for Region 5 is briefly described.

The rivers, streams, and tributaries included in this region are shown in Figure 3, together with the locations of 26 gaging stations used for deriving the unit-hydrograph parameters. These gaging stations; their USGS numbers; drainage area, A, above the gaging station, in mi^2; main channel length, L, in miles; and main channel slope, s, in ft/mi are given in Table 2.

Derived Unit-Hydrograph Parameters

The derived unit-hydrograph parameters were derived at each of the 26 gaging stations as described earlier. These are given in Table 3. The stepwise

TABLE 2. Unit-Hydrograph Parameters for Region 5: Basin Factors.

No.	Stream and Gaging Station	USGS No.	Area (mi^2)	Length (mi)	Slope (ft/mi)
1	Mazon River near Coal City	05542000	455.00	36.27	4.33
2	North Fork Vermilion River Near Charlotte	05554000	186.00	23.00	5.39
3	Big Bureau Creek at Princeton	05556500	196.00	54.59	6.07
4	West Bureau Creek at Wyanet	05557000	86.70	22.54	9.03
5	East Bureau Creek near Bureau	05557500	99.00	23.50	12.72
6	Crow Creek (west) Near Henry	05558500	56.20	27.49	10.24
7	Gimlet Creek at Sparland	05559000	5.66	4.81	53.86
8	Crow Creek near Washburn	05559500	115.00	27.68	6.07
9	Ackerman Creek at Farmdale	05561000	11.20	6.72	39.86
10	Farm Creek at East Peoria	05562000	61.20	18.60	18.90
11	Kickapoo Creek near Kickapoo	05563000	119.00	22.18	10.93
12	Kickapoo Creek near Peoria	05563500	297.00	39.36	7.50
13	Money Creek near Towanda	05564400	49.00	25.78	5.25
14	Money Creek above Lake Bloomington	05564500	53.10	29.20	4.91
15	Hickory Creek above Lake Bloomington	05565000	9.81	6.74	11.88
16	East Branch Panther Creek near Gridley	05566000	6.30	3.11	11.14
17	East Branch Panther Creek at El Paso	05566500	30.50	8.47	4.54
18	Panther Creek near El Paso	05567000	93.90	13.59	4.22
19	Wildcat Creek Tributary near Monticello	05572100	0.10	0.37	34.11
20	Sangamon River Tributary at Andrew	05577700	1.50	1.36	40.13
21	Lake Fork near Cornland	05579500	214.0	37.00	4.65
22	Kickapoo Creek at Waynesville	05580000	227.00	36.08	6.23
23	Kickapoo Creek at Lincoln	05580500	306.00	54.48	5.12
24	Salt Creek Tributary at Middletown	05580700	0.90	1.55	48.94
25	Sugar Creek near Hartsburg	05581500	333.00	42.77	5.76
26	Cabiness Creek Tributary near Petersburg	05582200	0.94	1.57	23.76

TABLE 3. Derived Unit-Hydrograph Parameters for Region 5.

No.	t_r (hr)	t_p (hr)	U_p (cfs)	$t_{0.75}$ (hr)	$d_{0.75}$ (hr)	$t_{0.50}$ (hr)	$d_{0.50}$ (hr)	$t_{0.25}$ (hr)	$d_{0.25}$ (hr)	t_b (hr)	a (cfs/hr)
1	8.00	24.00	10400	17.30	14.70	14.00	24.00	10.00	38.50	95.00	100
2	5.00	15.50	4300	12.00	12.20	9.00	25.05	6.30	39.40	85.50	100
3	5.00	14.00	7000	7.20	10.30	6.00	15.00	4.00	23.50	69.00	100
4	3.00	9.00	5000	6.30	5.70	5.00	9.00	3.50	13.70	41.00	160
5	3.00	7.50	6800	5.50	4.60	4.50	7.50	3.60	11.20	47.00	250
6	3.00	9.00	3350	6.50	6.30	5.20	9.30	3.50	14.80	39.00	150
7	1.00	1.50	2400	1.00	0.90	0.80	1.30	0.65	1.85	5.25	700
8	4.00	12.50	5000	9.20	7.50	7.60	11.90	5.50	19.70	56.00	100
9	1.00	2.33	3000	1.73	1.25	1.30	2.17	0.80	3.28	7.83	500
10	2.00	5.00	7000	4.00	3.25	3.50	5.00	2.50	7.60	21.00	350
11	4.00	8.00	10000	6.30	4.30	5.75	6.75	4.51	10.20	30.50	200
12	5.00	14.50	13000	11.20	8.80	9.20	12.80	6.50	20.60	47.50	160
13	3.00	9.00	2200	6.80	6.80	5.70	11.30	4.00	20.60	48.00	80
14	3.00	9.00	2200	7.00	6.90	5.75	13.25	4.50	25.70	50.00	80
15	1.25	3.50	1200	2.60	2.50	2.15	4.45	1.75	7.60	18.50	120
16	1.00	3.50	700	2.50	2.50	2.00	4.10	1.20	7.80	23.50	80
17	2.00	8.50	1600	6.00	5.50	4.90	8.60	3.50	16.80	43.00	50
18	4.00	13.00	3200	9.00	11.30	7.50	16.60	6.00	26.00	68.00	80
19	0.08	0.33	130	0.28	0.19	0.23	0.36	0.17	0.57	2.17	120
20	0.42	1.12	625	0.78	0.77	0.58	1.30	0.37	2.10	4.80	250
21	6.00	24.00	3600	17.20	20.40	14.20	31.60	10.00	51.00	112.00	80
22	5.00	20.00	6000	14.50	12.40	11.50	19.70	8.30	33.20	76.00	120
23	6.00	21.00	7600	15.50	13.80	12.10	22.70	7.30	32.80	86.00	100
24	0.33	0.82	600	0.61	0.48	0.50	0.77	0.35	1.33	3.40	300
25	6.00	23.00	8200	16.90	12.30	13.50	20.30	8.30	36.50	83.00	100
26	0.33	1.09	450	0.84	0.62	0.67	1.02	0.49	2.00	5.58	150

multiple correlation analyses yielded the following best regressions for t_r and a:

$$\log t_r = -0.324 + 0.482 \log A - 0.100 \log s \quad (12)$$
$$(S_e = 0.060; R = 0.993)$$

$$\log a = 0.577 + 0.256 \log A + 1.151 \log s \quad (13)$$
$$(S_e = 0.053; R = 0.983)$$

in which S_e = standard error estimate (same units as the dependent variable) and R = multiple correlation coefficient, both apply to regression on log-transformed variables.

Modified Unit-Hydrograph Parameters

With the fitted values of t_r and a (given as t_r' and a' in the following equation), the remaining nine unit-hydrograph parameters were modified for any difference between the derived and fitted values of these two parameters. The following equations are used in these modifications:

$$U_{p'} = U_p - a(t_{r'} - t_r) \quad (14a)$$

$$t_{0.25'} = t_{0.25} + 0.5(t_{r'} - t_r) \quad (14b)$$

$$t_{0.50'} = t_{0.50} + 0.5(t_{r'} - t_r) \quad (14c)$$

$$t_{0.75'} = t_{0.75} + 0.5(t_{r'} - t_r) \quad (14d)$$

$$t_{p'} = t_p + 0.5(t_{r'} - t_r) \quad (14e)$$

$$d_{0.75'} = d_{0.75} + 0.50(t_{r'} - t_r) \quad (14f)$$

$$d_{0.50'} = d_{0.50} + 0.75(t_{r'} - t_r) \quad (14g)$$

$$d_{0.25'} = d_{0.25} + (t_{r'} - t_r) \quad (14h)$$

$$t_{b'} = t_b + (t_{r'} - t_r) \quad (14i)$$

The significant regression equations obtained with the stepwise multiple correlation analyses using the log-transformed modified values of the parameters are given below.

$$\left.\begin{array}{l} t_r = 0.474 A^{0.482} s^{-0.100} \\ a = 3.777 A^{0.256} s^{-1.151} \end{array}\right\} \text{determined from Equations (12) and (13)}$$

$$t_p = 4.539\ A^{0.388} s^{-0.453}; \quad (15a)$$
$$(S_e = 0.054; R = 0.995)$$

$$U_p = 43.76\ A^{0.071} s^{0.709}; \quad (15b)$$
$$(S_e = 0.080; R = 0.988)$$

$$t_{0.75} = 3.600\ A^{0.374} s^{-0.469}; \quad (15c)$$
$$(S_e = 0.065; R = 0.992)$$

$$d_{0.75} = 4.561\ A^{0.360} s^{-0.593}; \quad (15d)$$
$$(S_e = 0.076; R = 0.991)$$

$$t_{0.50} = 3.005\ A^{0.374} s^{-0.481}; \quad (15e)$$
$$(S_e = 0.062; R = 0.993)$$

$$d_{0.50} = 9.184\ A^{0.334} s^{-0.645}; \quad (15f)$$
$$(S_e = 0.083; R = 0.989)$$

$$t_{0.25} = 2.223\ A^{0.369} s^{-0.493}; \quad (15g)$$
$$(S_e = 0.065; R = 0.992)$$

$$d_{0.25} = 23.40\ A^{0.289} s^{-0.763}; \quad (15h)$$
$$(S_e = 0.088; R = 0.987)$$

$$t_b = 55.73\ A^{0.281} s^{-0.709}; \quad (15i)$$
$$(S_e = 0.070; R = 0.991)$$

Fitted unit hydrograph parameters using Equations (12), (13), and (15a) through (15i) are given in Table 4.

Similar analyses were conducted for all eight regions shown in Figure 3. The results of these analyses (Singh, 1981) show that the methodology developed and presented for Region 5 is equally applicable over all the regions in Illinois and can be used in other geographical regions and settings to develop unit hydrographs for use in dam safety and other studies requiring 100-year or higher flood hydrographs.

COMPARISON WITH SNYDER'S EQUATIONS

Snyder (1938) indicated that t_p/t_r = 5.5. For this study, t_p/t_r becomes 5.5 + 0.5, or 6 because t_p in Snyder's equation is from midpoint of excess rainfall duration t_r whereas in this study t_p is taken from the beginning of rainfall excess. The t_p/t_r ratios were calculated for all stations in each of the eight regions from tables similar to Table 4. The range of the ratios in each region as well as the median values are given in Table 5, which also contains the range of basin drainage areas and number of basins in each region. For all eight regions combined, the ratios range from 1.9 to 4.7 instead of 6 as per Snyder's formulation. The median values for all the regions vary from 2.9 to 3.9. The ratios generally increase with increase in drainage area and/or decrease in channel slope.

Snyder (1938) gave q_p as inversely proportional to t_p. Thus, U_p/A will be inversely proportional to t_p, given a constant value of C_p for a homogeneous region. In other words, the product of U_p/A and t_p (comparable to $q_p \times t_p = 640\ C_p$ in Snyder's formulation) should be the same for any drainage area in a homogeneous region. The regional range of the products as well as the regional median values are given in Table 5. The product varies considerably within each region. Generally, the product decreases with increase in drainage area and/or decrease in slope. For all eight regions combined, the product varies from 280 to 873. The median values for all the regions vary from 395 to 552.

According to U.S. Army Corps of Engineers (1959), the ratio of W_{50} to W_{75} is 1.75. This ratio equals $d_{0.50}/d_{0.75}$. The range of this ratio as well as its median value for the eight study regions are given in Table 5. The regional median values vary from 1.57 to 1.74. High values of the ratio are usually associated with drainage basins having large areas and/or less slopes.

TABLE 4. Fitted Unit-Hydrograph Parameters for Region 5.

Basin No.	t_r (hr)	t_p (hr)	U_p (cfs)	$t_{0.75}$ (hr)	$d_{0.75}$ (hr)	$t_{0.50}$ (hr)	$d_{0.50}$ (hr)	$t_{0.25}$ (hr)	$d_{0.25}$ (hr)	t_b (hr)	a (cfs/hr)
1	7.84	25.05	9026	17.88	17.29	14.65	27.56	10.34	44.81	110.01	94
2	4.98	16.04	5632	11.55	11.00	9.44	17.75	6.67	29.28	73.27	97
3	5.05	15.51	6356	11.14	10.45	9.09	16.73	6.42	27.15	68.36	121
4	3.27	9.45	4757	6.82	6.16	5.53	9.86	3.90	15.85	41.03	150
5	3.37	8.52	6657	6.10	5.27	4.93	8.26	3.46	12.68	33.40	229
6	2.62	7.55	3838	5.46	4.89	4.43	7.87	3.13	12.70	33.23	161
7	0.73	1.46	2494	1.06	0.80	0.84	1.25	0.59	1.85	5.38	584
8	3.90	12.62	4374	9.13	8.63	7.45	14.00	5.27	23.28	58.85	102
9	1.05	2.18	3250	1.58	1.22	1.26	1.91	0.88	2.83	8.06	489
10	2.57	5.91	6293	4.23	3.50	3.41	5.45	2.39	8.16	22.04	320
11	3.74	9.80	6800	7.02	6.16	5.68	9.69	3.99	15.01	39.16	199
12	6.04	16.56	9883	11.79	10.70	9.59	16.77	6.74	26.05	66.12	163
13	2.62	9.68	2170	7.10	6.92	5.80	11.56	4.13	20.33	51.33	72
14	2.75	10.29	2189	7.55	7.41	6.18	12.40	4.40	21.89	55.05	69
15	1.11	3.59	1254	2.65	2.39	2.15	3.99	1.53	6.85	18.32	118
16	0.91	3.11	879	2.32	2.12	1.88	3.59	1.34	6.33	16.93	94
17	2.12	8.60	1404	6.36	6.36	5.21	10.84	3.73	19.80	49.81	51
18	3.67	13.75	2932	10.03	9.95	8.22	16.54	5.85	28.97	71.93	61
19	0.11	0.38	106	0.29	0.25	0.23	0.44	0.17	0.82	2.39	122
20	0.40	1.00	798	0.74	0.59	0.59	0.97	0.42	1.57	4.56	285
21	5.41	18.11	5595	13.04	12.63	10.68	20.46	7.56	34.13	84.63	88
22	5.40	16.23	7177	11.63	10.85	9.48	17.28	6.69	27.77	69.93	124
23	6.36	19.91	7698	14.25	13.57	11.65	21.67	8.22	35.16	87.40	109
24	0.31	0.75	642	0.56	0.44	0.44	0.72	0.31	1.17	3.43	326
25	6.55	19.51	8879	13.92	13.04	11.37	20.66	8.01	32.93	82.33	124
26	0.34	1.06	396	0.80	0.68	0.64	1.17	0.46	2.05	5.80	144

TABLE 5. Results From Regionalization of Unit-Hydrograph Parameters.

Region	Range of A	n	Ratio t_p/t_r Range	Ratio t_p/t_r Median	Product $U_p t_p/A$ Range	Product $U_p t_p/A$ Median	Ratio $d_{0.50}/d_{0.75}$ Range	Ratio $d_{0.50}/d_{0.75}$ Median	t_b, days Range
1	1.15-387	15	1.9-4.7	3.2	302-873	482	1.49-1.76	1.67	0.1-4.9
2	0.07-324	20	3.4-4.2	3.9	280-677	416	1.60-1.86	1.72	0.1-7.7
3	8.84-107	11	2.4-3.7	2.9	354-585	395	1.56-2.03	1.72	2.1-3.5
4	0.24-445	12	2.6-4.5	3.2	493-658	552	1.55-1.73	1.57	0.1-5.4
5	0.10-455	26	2.0-4.1	3.1	396-643	502	1.56-1.76	1.61	0.1-4.6
6	1.05-446	10	2.3-4.5	3.4	426-504	432	1.67-1.85	1.74	0.4-6.4
7	0.08-319	19	2.1-4.2	3.3	363-662	461	1.54-1.76	1.59	0.1-6.9
8	0.08-464	17	2.5-4.7	3.9	393-851	483	1.50-1.71	1.67	0.1-5.9

Note: A = basin drainage area, mi^2.
n = number of basins in a region.
$U_p t_p/A$ = product, cfs hr/mi^2.

The t_b was given by Snyder (1938) as $3 + t_p/8$ days where t_p is in hours. Thus the lowest value for t_b is 3 days. However, the range for t_b values and number of basins with $t_b < 3$ days for each region, as given in Table 5, show that Snyder's t_b values are not useful in delineating unit-hydrograph shapes.

CONCLUSIONS

Satisfactory delineation of unit hydrographs suitable for developing 100-year and up to PMF hydrographs is essential for avoiding underestimation or overestimation of design spillway capacity and freeboard, as well as for evaluating the hydrologic safety of existing dams and reservoirs. Procedures for developing such unit hydrographs have been developed. Eleven unit hydrograph parameters have been identified, which will lead to practically the same unit hydrograph for a given basin no matter who delineates it. Regionalization of these parameters in terms of basin factors provides an easy and relatively accurate determination of unit hydrograph for ungaged areas (without reservoirs and other flow regulations) in the region. The derived unit hydrographs do not suffer from deficiencies inherent in those derived by using many other methods in use. However, the following cautions should be exercised wherever applicable.

1. In finalizing a unit hydrograph for an ungaged area, the variation of the derived unit hydrographs from the fitted unit hydrographs with regionalized parameters for basins in the nearby area may be considered to refine the unit hydrograph.

2. Generally, a longer length and/or milder slope of the main channel than that characterized by log L – Log A and log s – log A regional regressions increases the magnitude of time parameters and reduces the peak, whereas a shorter length and/or steeper slope decreases the magnitude of time parameters and increases the peak. Any effect on the unit hydrograph parameters caused by considerable variation in L and s from values expected from regional relations can be evaluated to some extent from the information developed for basins in a given region.

3. Parameter a, one of the 11 unit hydrograph parameters, serves the purpose of modifying the unit hydrograph peak for a small change in the value of t_r. If t_r is to be changed significantly, a minor change may be effected with a, and then the S-hydrograph method may be used to determine the unit hydrograph of the desired duration.

4. If the basin for which a unit hydrograph is needed has two major and distinct streams joining a relatively small distance upstream of the point under consideration, the unit hydrograph may be determined for each branch separately and then routed through the main stem downstream of the junction to obtain the desired unit hydrograph.

5. If the basin for which the unit hydrograph is to be determined is near the boundary of a region, the unit hydrographs may be determined from the equations of that basin and also from those for the adjacent basin. The supplementary information, together with any physical or other data, may be considered in deriving the desired unit hydrographs.

ACKNOWLEDGMENTS

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INTRODUCTION TO FLOOD ROUTING

PMF GUIDELINES SECTION 8-11

Tuesday 11:00 a.m.

8-11 Reservoir Routing to Obtain Outflow Probable Maximum Flood

The preceding sections led to the development of the inflow PMF hydrograph, which must be routed through the reservoir to determine the maximum reservoir elevation and peak discharge that must pass the dam. Assumptions of reservoir starting elevation and initial flow must be made. This section provides guidelines for making the necessary assumptions and for performing the routing.

8-11.1 Initial Assumptions

The following assumptions should be made in performing routing of the PMF:

- Use the reservoir area-volume-elevation information as obtained and reviewed in Sections 8-4 and 8-5, respectively.
- Use spillway and outlet-works capacities established in Section 8-5.5.2.
- Use the gate operating policy as established in Section 8-5.5.3.

8-11.2 Reservoir Starting Elevations

Considerations regarding reservoir starting elevations were given in Section 8-3.1 and should be considered simultaneously with the gate and flashboard operations established in Section 8-5.5.3 to determine the critical reservoir starting elevation.

- If the considerations with regard to operation of gates or failure or removal of flashboards indicate a higher reservoir starting elevation than would be given by the considerations in Section 8-3.1, the higher elevation should be used.

8-11.3 Initial Flow

The flow rate of the river at the time the PMP begins should be consistent with the antecedent approach selected from Section 8-3.1. Average monthly flow should be obtained for the months during the season when the critical PMP would occur. Tabulated monthly average data are available in USGS water data reports. The average monthly flow for the month of the critical PMP should be added to the inflow PMF hydrograph before routing through the reservoir. When using HEC-1, this initial flow is the parameter STRTQ. For the particular case when the basin has been subdivided, the initial flow will already have been added as described in Section 8-10.5. For "ungaged" basins, the average monthly flow per square mile of drainage area, obtained from records for nearby "gaged" basins, should be used to compute the required initial flow.

8-11.4 Routing Procedures

Level-pool-routing procedures can generally be used. Whether or not level-pool-routing procedures are satisfactory will depend on the unit hydrograph used to develop the PMF inflow hydrograph and the dynamic effect of the reservoir on flood flows.

Caution: If reverse-reservoir routing was used to develop inflow hydrographs to the reservoir during passage of the historic floods used in the unit-hydrograph analysis, some of the dynamic effects will have already been implicitly included in the developed inflow PMF hydrograph. Although dynamic effects during passage of a PMF may be more dramatic than during the analyzed historic floods, they are satisfactorily approximated in the reverse-reservoir routing process. Level-pool-routing procedures can be used in these situations. Problems with data will often make it impossible to derive an accurate inflow hydrograph by reverse routing.

If the unit hydrograph used to develop the PMF inflow hydrograph at the dam site is based on natural upstream channel conditions, a method may be needed to adjust for the dynamic effect of the reservoir and the lost channel storage. Level-pool-routing procedures can lead to errors. These procedures should be used with caution and must be justified.

An alternative is to use a distributed inflow procedure where all inflows to the reservoir at its rim are estimated. This requires developing PMF inflow hydrographs at all major tributaries and the direct rainfall on the reservoir. The flows are then routed through the reservoir using dynamic routing procedures or simple translation with timing based on wave celerity calculations. Dynamic routing procedures—although mathematically complex and sometimes difficult because of numerical instability—can be accomplished using the NWS unsteady routing program DAMBRK (Fread 1989).

The flood-passage operations should be reviewed after the initial routing of the inflow PMF to assess sensitivity of resulting maximum outflow rate and reservoir elevation to the reservoir starting elevation.

Introduction to Flood Routing

Time

Wednesday

11:00 a.m. → 12.00 Noon

FLOOD ROUTING IN STREAMS

A. INTRODUCTION TO FLOOD ROUTING

1. Purpose
2. Nature of Flood Movement

B. DERIVATION OF THE CONTINUITY AND MOMENTUM EQUATIONS

1. Methods of Flood Routing
   a. Hydraulic Routing
   b. Hydrologic Routing

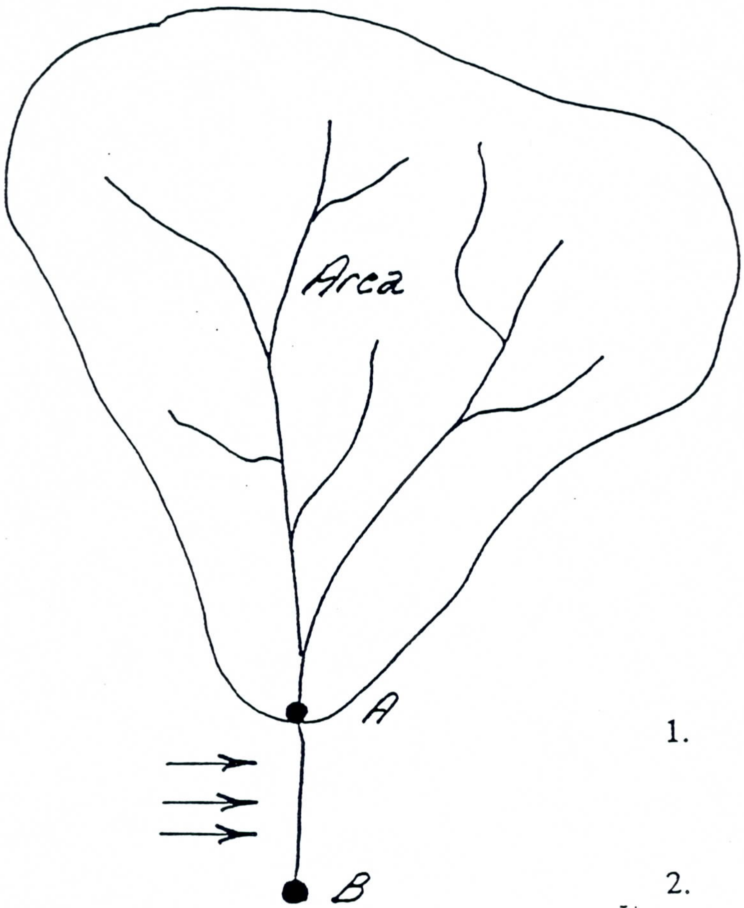
C. HYDROLOGIC TECHNIQUES FOR FLOOD ROUTING

1. Muskingum Method
2. Muskingum Cunge Method
3. Kinematic Wave Method for Channel Routing
4. Attenuated Kinematic Wave Method

INTRODUCTION TO FLOOD ROUTING

1. Purpose

Determine hydrograph at one location on a stream from known hydrograph at upstream location.



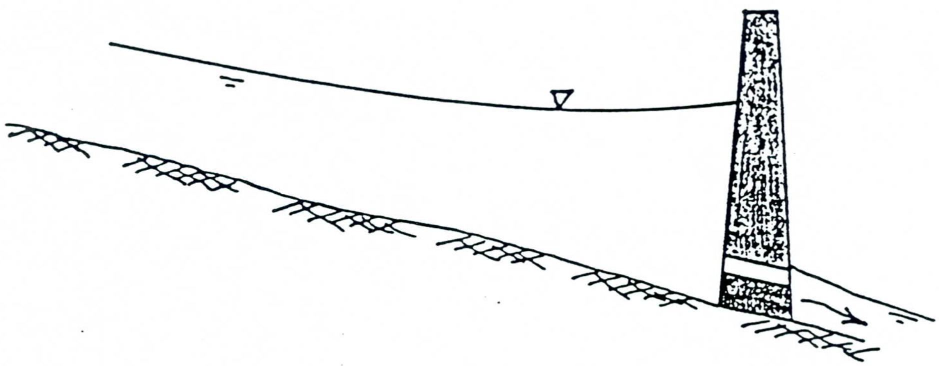
1. Apply rainfall to area and develop hydrograph at point "A" from unit graph.

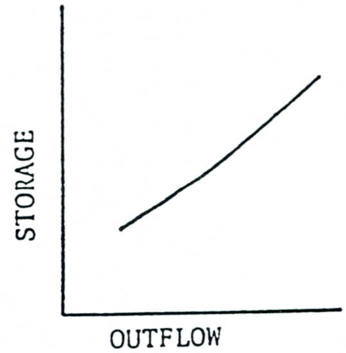
2. Route hydrograph from point "A" to point "B."

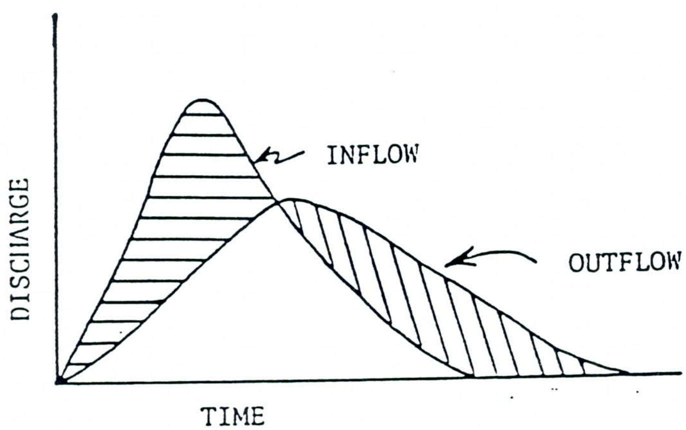
Flood routing is a method for accounting for the change in hydraulic characteristics of a food wave as it passes through a river.

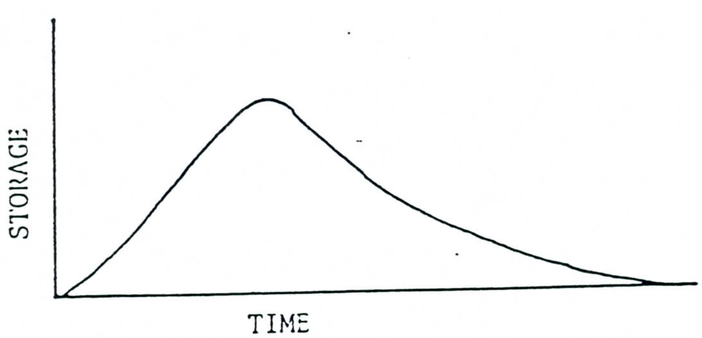
2. Nature of Flood Wave Movement

a. Effect of reservoir-type storage

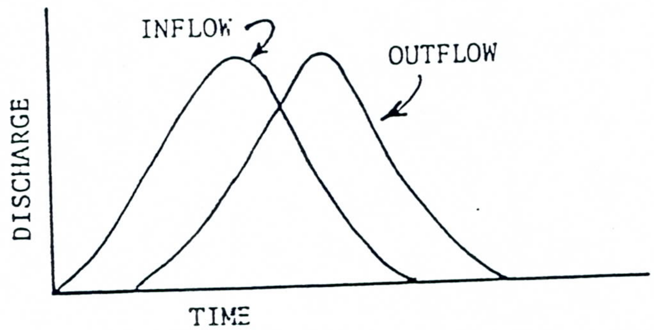


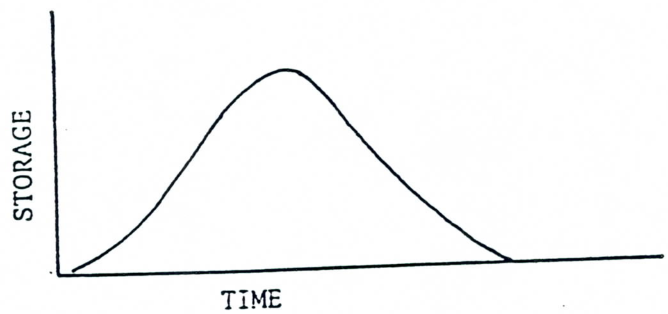






b. Uniformly progressive wave





3. Methods of Flood Routing

a. Hydraulic methods

These methods are based on solving the basic differential equations that describe unsteady flow.

$$S_f = S_o - \frac{\partial v}{\partial x} - \frac{\partial (v^2/2g)}{\partial x} - \frac{1}{g}\frac{\partial v}{\partial t} \qquad \text{Energy} \qquad (1)$$

Steady

Uniform Flow

Steady

Gradually Varied Flow

Unsteady

Gradually Varied Flow

$$A\frac{\partial v}{\partial x} + vB\frac{\partial v}{\partial x} + B\frac{\partial v}{\partial t} = q \qquad \text{Continuity} \qquad (2)$$

These are often called the <u>Saint Venant Equations</u>.

A typical set of input requirements for a computer program that solves the Saint Venant equations numerically is the following:

(1) River cross sections
(2) Manning's 'n' values
(3) Water surface profile at $t = 0$
(4) Inflow hydrograph
(5) Stage-discharge relation at downstream end

b. Hydrologic Methods

Hydrologic methods of flood routing do not attempt a direct, complete solution of the differential equations that describe unsteady flow. These methods solve the continuity equation and a much simplified version of the energy equation. The methods generally employ semi-empirical coefficients that must be calibrated. Some hydrologic routing methods are the Modified Puls, Working R&D, and Muskingum methods. Also to be included are the simpler averaging and lagging methods such as the Successive Average-Lag (Tatum) and Progressive Average-Lag (Straddle-Stagger) methods.

4. Modified-Puls Method

a. The Modified Puls method is a technique for solving the continuity equation, given a unique relationship between outflow and storage. The procedure for the Modified Puls method is illustrated in Handout H-41-5 and is summarized below:

I Given: Inflow hydrograph, routing interval, initial storage

II Determine: Storage-outflow curve, outflow hydrograph

III Procedure:

(1) Determine discharge rating curve at downstream end

(2) Determine storage that goes with each discharge on the rating curve and each elevation

(3) Determine $\frac{S}{\Delta t} + \frac{O}{2}$ vs. O curve.

(4) Route inflow hydrograph through reach

(5) Compare result with historical events to verify your model

(6) Perform required study

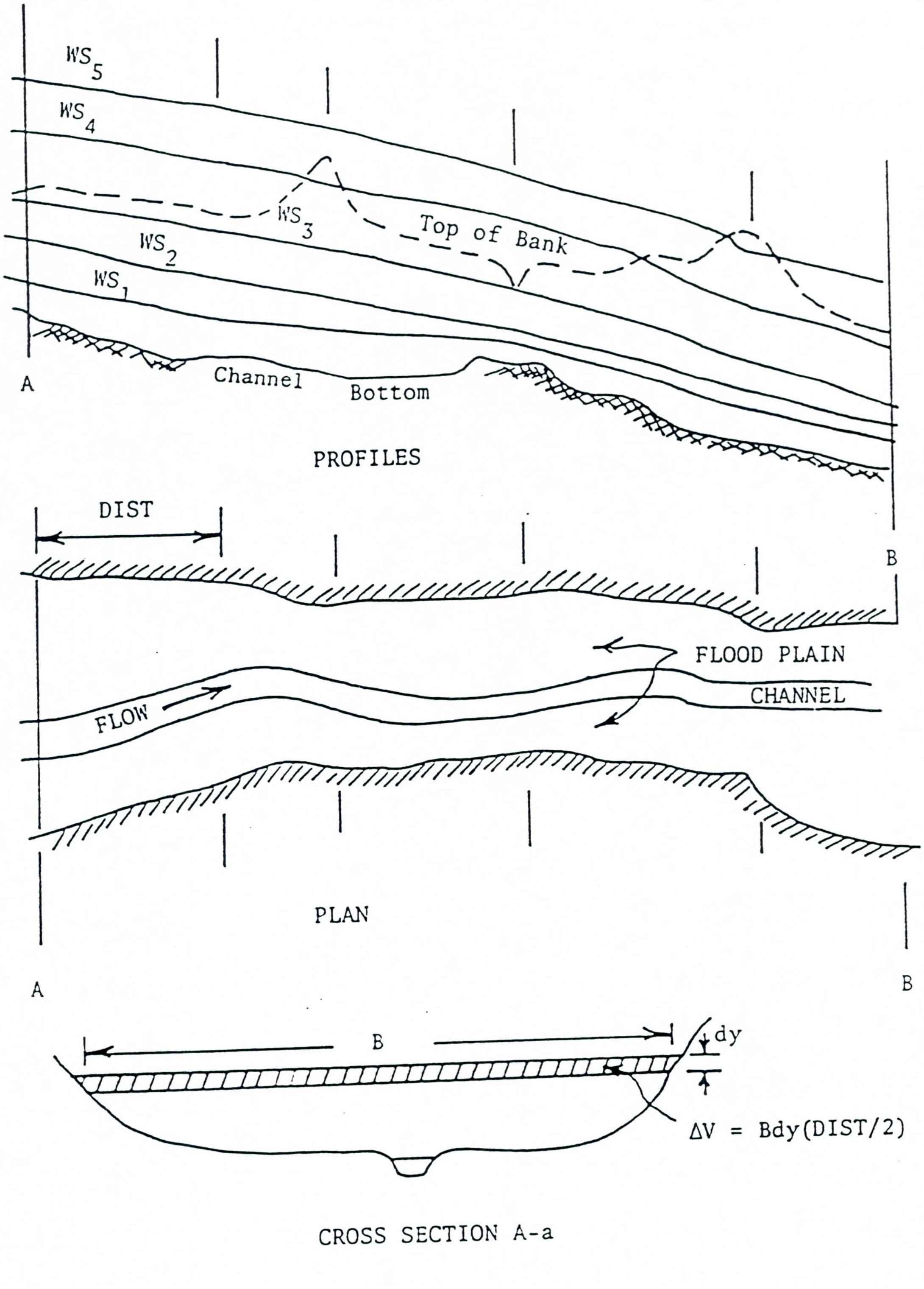
b. Application of Modified Puls Method to Rivers

(1) Determine storage-outflow relation by compiling steady-flow water surface profiles. This is illustrated on the following page.

(2) Determine the number of routing steps

$$k \text{ (i.e., Travel Time)} = \frac{\text{Total Distance Between Gages}}{V_w \text{ (i.e., Velocity of Flood Wave)}}$$

$$NSTPS = \frac{K}{\Delta t}$$



Determination of Storage in A Routing Reach From
Geometry of Channel and Flood Plain

(3) Apply the Modified Puls method as for reservoir routing.

c. Problems observed in using Modified Puls method in rivers

(1) The amount of attenuation is affected by the number of routing steps used.

(2) Storage is related only to outflow and when calculated from steady flow water surface profile, the "wedge" is not included.

REFERENCES

a. EM 1110-2-1408, "Routing of Floods Through River Channels," U.S. Army Corps of Engineers, March 1960.
b. Chow, Ven Te, Open Channel Hydraulics, Chapter 20, pp. 604-613.
c. Henderson, F. M., Open Channel Flow, Chapter 9, MacMillan, 1966.

HYDROLOGIC TECHNIQUES FOR FLOOD ROUTING

1. Definition of Hydrologic Techniques

Hydrologic techniques are techniques that do not attempt a direct, complete solution of the basic differential equations of continuity and energy or momentum. They include storage routing methods such as the Modified Puls, Working R&D, and Muskingum methods. They also include the simpler averaging and lagging methods such as the straddle-stagger and Tatum methods.

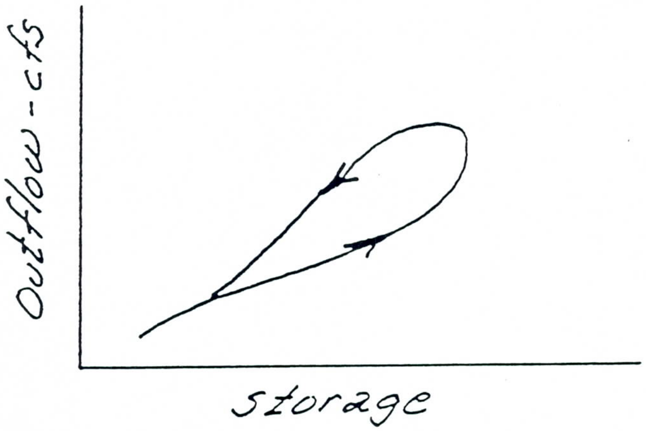
2. Focus of Lecture

Discuss the basis of the Muskingum Method, how it is applied, and how required routing parameters can be determined.

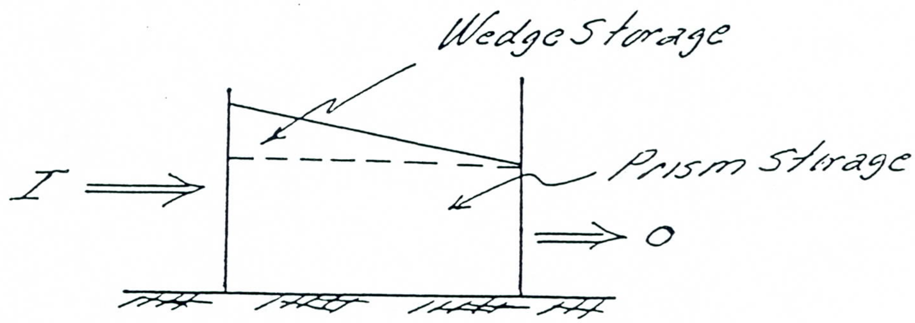
3. The Muskingum Method

a. Basis of Method

The relationship of storage in a river reach vs. discharge leaving the reach (outflow) corresponding to the passage of a flood wave is typically a "loop" relationship, as illustrated below.



The loop reflects the influence of wedge storage. In the Muskingum method, wedge storage is accounted for as follows:



S = total storage in reach = prism storage + wedge storage

$$S = KO + KX\,(I - O)$$

$$S = K\,[XI + (1 - X)\,O]$$

where

O = rate of outflow from routing reach

I = rate of inflow to routing reach

K = travel time through routing reach

X = dimensionless constant that ranges between 0 and .5

If the above equation for total reach storage, S, is substituted in the continuity equation, the following Muskingum routing equation results:

$$O_2 = C_1\,I_2 + C_2\,I_1 + C_3\,O_1$$

The subscripts 1 and 2 in this equation indicate the beginning and end, respectively, of a time interval Δt.

The routing coefficients -- C_1, C_2, and C_3 -- are defined as follows:

$$C_1 = \frac{\Delta t - 2K\,X}{2K\,(1 - X) + \Delta t}$$

$$C_2 = \frac{\Delta t + 2K\,X}{2K\,(1 - X) + \Delta t}$$

$$C_3 = \frac{2K\,(1 - X) - \Delta t}{2K\,(1 - X) + \Delta t}$$

d. Application of Muskingum Method

See Example 1.

REFERENCES

a. EM 1110-2-1408, "Routing of Floods Through River Channels," U.S. Army Corps of Engineers, March 1960.

b. Storage and Flood Routing, U.S. Geological Survey Water Supply Paper 1543-B, 1960.

c. Henderson, F. M., Open Channel Flow, Chapter 9, MacMillan, 1966.

Muskingum Conge

Muskingum-Cunge Channel Routing

I. INTRODUCTION

The Muskingum-Cunge channel routing technique is a non-linear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The advantages of this method over other hydrologic techniques are: (1) the parameters of the model are physically based; (2) the method has been shown to compare well against the full unsteady flow equations over a wide range of flow situations (Ponce, 1983); and (3) the solution is independent of the user specified computation interval. The major limitations of the Muskingum-Cunge technique are that (1) it cannot account for backwater effects; and (2) the method begins to diverge from the full unsteady flow solution when very rapidly rising hydrographs are routed through flat channel sections (i.e., channel slopes less than 1 ft/mile).

II. DEVELOPMENT OF EQUATIONS

The basic formulation of the equations is derived from the continuity equation and the diffusion form of the momentum equation:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_L \qquad \text{(continuity)} \quad (1)$$

$$S_f = S_o - \frac{\partial Y}{\partial x} \qquad \text{(diffusion form of Momentum equation)} \quad (2)$$

By combining equations (1) and (2) and linearizing, the following convective diffusion equation is formulated (Miller and Cunge, 1975):

$$\frac{\partial Q}{\partial t} + c\,\frac{\partial Q}{\partial x} = \mu\,\frac{\partial^2 Q}{\partial x^2} + c q_L \quad (3)$$

Where:

Q	=	Discharge in cfs
A	=	Flow area in ft^2
t	=	Time in seconds
x	=	Distance along the channel in feet
Y	=	Depth of flow in feet
q_l	=	Lateral inflow per unit of channel length
S_f	=	Friction slope
S_c	=	Bed Slope
c	=	The wave celerity in the x direction as defined below.

$$c = \left.\frac{dQ}{dA}\right|_x \quad (4)$$

The hydraulic diffusivity (μ) is expressed as follows:

$$\mu = \frac{Q}{2BS_o} \qquad (5)$$

where B is the top width of the water surface.

Following a Muskingum-type formulation, with lateral inflow, the continuity equation (1) is discretized on the x-t plane (Figure 1) to yield:

$$Q_{j+1}^{n+1} = C_1Q_j^n + C^2Q_j^{n+1} + C_3Q_{j+1}^n + C_4Q_L \qquad (6)$$

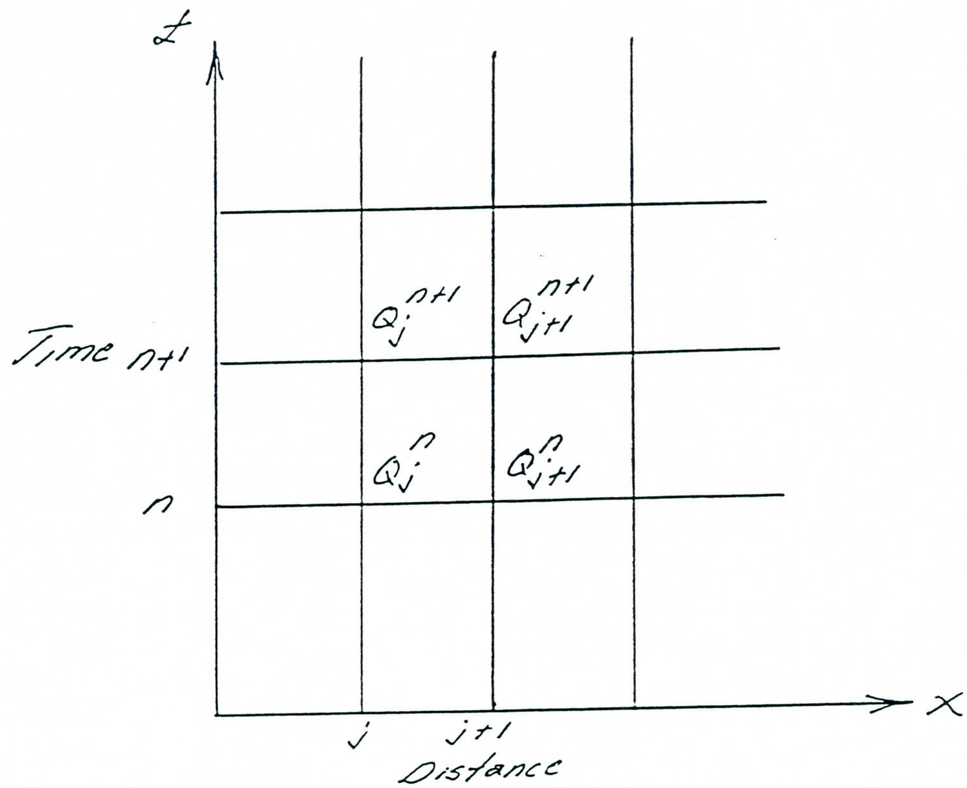


Figure 1: Discretization on x-t plane of the variable parameter Muskingum-Cunge Model.

It is assumed that the storage in the reach is expressed as the classical Muskingum storage:

$$S = K\,[X\,I + (1 - X)\,O] \qquad (7)$$

where:

where:	S	=	Channel storage
	K	=	Cell travel time (seconds)
	X	=	Weighing factor
	I	=	Inflow
	O	=	Outflow

Therefore, the coefficients can be expressed as follows:

$$C_1 = \frac{\frac{\Delta t}{K} + 2X}{\frac{\Delta t}{K} + 2\ (1 - X)}$$

$$C_2 = \frac{\frac{\Delta t}{K} - 2X}{\frac{\Delta t}{K} + 2\ (1 - X)}$$

$$C_3 = \frac{2(1 - X) - \frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2\ (1-X)}$$

$$C_4 = \frac{2K - \frac{\Delta t}{K}}{\frac{\Delta t}{K} + 2\ (1-X)}$$

$$Q_L = q_L \Delta x$$

In the Muskingum equation, the amount of diffusion is based on the value of X, which varies between 0.0 and 0.5. The Muskingum X parameter is not directly related to physical channel properties. The diffusion obtained with the Muskingum technique is a function of how the equation is solved, and is therefore considered numerical diffusion rather than physical. In the Muskingum-Cunge formulation, the amount of diffusion is controlled by forcing the numerical diffusion to match the physical diffusion (μ) from equations (3) and (5). The Muskingum-Cunge equation is therefore considered an approximation of the convective diffusion equation (3). As a result, the parameters K and X are expressed as follows (Cunge, 1969, and Ponce, 1981):

$$K = \frac{\Delta x}{c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (8)$$

$$X = \frac{1}{2}\left(1 - \frac{Q}{BX_o c \Delta x}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (9)$$

Then, the Courant (C) and cell Reynolds (D) numbers can be defined as:

$$C = c\,\frac{\Delta t}{\Delta x} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (10)$$

and

$$D = \frac{Q}{BS_o c\Delta x} \quad \text{(11)}$$

The routing coefficients for the non-linear diffusion method (Muskingum-Cunge) are then expressed as follows:

$$C_1 = \frac{1 + C - D}{1 + C + D}$$

$$C_2 = \frac{-1 + C + D}{1 + C + D}$$

$$C_3 = \frac{1 - C + D}{1 + C + D}$$

$$C_4 = \frac{2C}{1 + C + D}$$

in which the dimensionless numbers C and D are expressed in terms of physical quantities (Q, B, S_o, and c) and the grid dimensions (Δx and Δt).

III. SOLUTION OF THE EQUATIONS

The method is non-linear in that the flow hydraulics (Q, B, c), and therefore the routing coefficients (C_1, C_2, C_3, and C_4) are re-calculated for every Δdistance step and Δt time step. An iterative four-point averaging scheme is used to solve for c, B, and Q. This process has been described in detail by Ponce (1986).

Values for Δt and Δx are chosen internally by the model for accuracy and stability. First, Δt is evaluated by looking at the following three criteria and selecting the smallest value:

1. The user-defined computation interval, NMIN, from the first field of the IT record.

2. The time of rise of the inflow hydrograph divided by 20 (Tr/20).

3. The travel time of the channel reach.

Once Δt is chosen, Δx is evaluated as follows:

$$\Delta x = c\Delta t \qquad (12)$$

but Δx must also meet the following criteria to preserve consistency in the method (Ponce, 1983):

$$\Delta x < \frac{1}{2}\left(\frac{Q_o}{BS_oC}\right) \qquad (13)$$

where Q_o is the reference flow and Q_B is the baseflow taken from the inflow hydrograph as:

$$Q_o = Q_B + 0.05\,(Q_{peak} - Q_b)$$

Δx is chosen as the smaller value from the two criteria. The values chosen by the program for Δx and Δt are printed in the output, along with the computed peak flow. Before the hydrograph is used in subsequent operations, or printed in the hydrograph tables, it is converted back to the user-specified computation interval. The user should always check to see if the interpolation back to the user-specified computation interval has reduced the peak flow significantly. If the peak flow computed from the internal computation interval is markedly greater than the hydrograph interpolated back to the user-specified computation interval, the user-specified computation interval should be reduced and the model should be executed again.

IV. DATA REQUIREMENTS

Data for the Muskingum-Cunge method consist of the following:

1. Representative channel cross section.

2. Reach length, L

3. Manning roughness coefficients, n (for main channel and overbanks).

4. Channel bed slope, S_o.

The method can be used with a simple cross section (i.e., trapezoid, rectangle, square, triangle, or circular pipe), or a more detailed 8-point cross section can be provided. If one of the simple channel configurations is used, Muskingum-Cunge routing can be accomplished through the use of a single Rd record, as follows:

KK ..... Station Computation Identifier
RD ..... Muskingum-Cunge Data

If the more detailed 8-point cross section is used, enter the following sequence of records:

KK ..... Station Computation Identifier
RD ..... Blank record to indicate Muskingum-Cunge routing
RC

RX ..... 8-point Cross-Section Data
RY

When using the 8-point cross section, it is not necessary to fill out the data for the Rd record. All of the necessary information is taken from the RC, RX, and RY records.

V. INPUT AND OUTPUT EXAMPLE

The use of Muskingum-Cunge channel routing is demonstrated here in the development of a rainfall-runoff model for Kempton Creek. The watershed has been subdivided into three separate catchments, as shown in Figure 2. Clark's unit hydrograph and the SCS Curve Number method were used to evaluate local runoff from each of the subbasins. Channel routing from control point CP1 to CP2 and from CP2 to CP3 was accomplished with Muskingum-Cunge routing.

KEMPTON CREEK WATERSHED

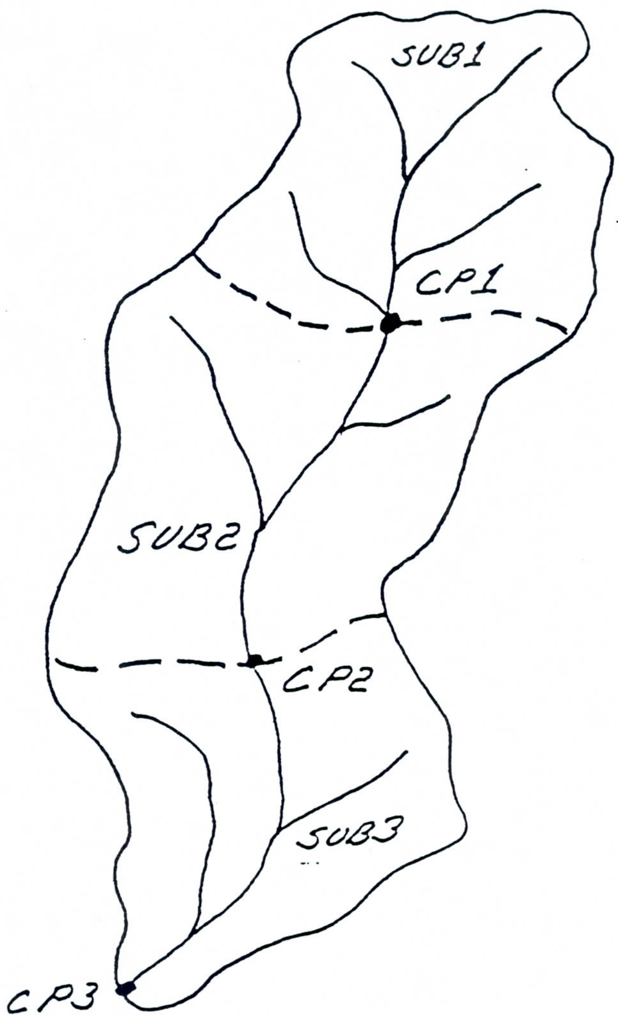


Figure 2. Kempton Creek Watershed for Muskingum-Cunge channel routing example.

Subbasin 2 (SUB2) is heavily urbanized with commercial and residential land use. The channel from CP1 to CP2 is a concrete lined trapezoidal channel with the following dimensions:

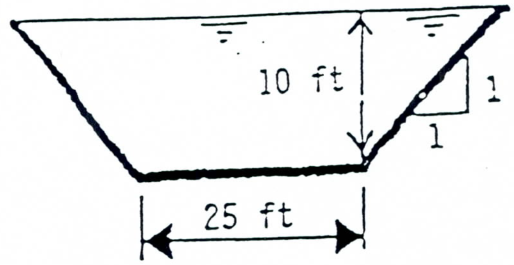


Figure 3. Trapezoidal channel.

Both subbasins 1 and 3 are completely undeveloped. The channel between CP2 and CP3 is in its natural state. A representative 8-point cross section has been fit to match the main channel and overbank flows through the reach as shown below:

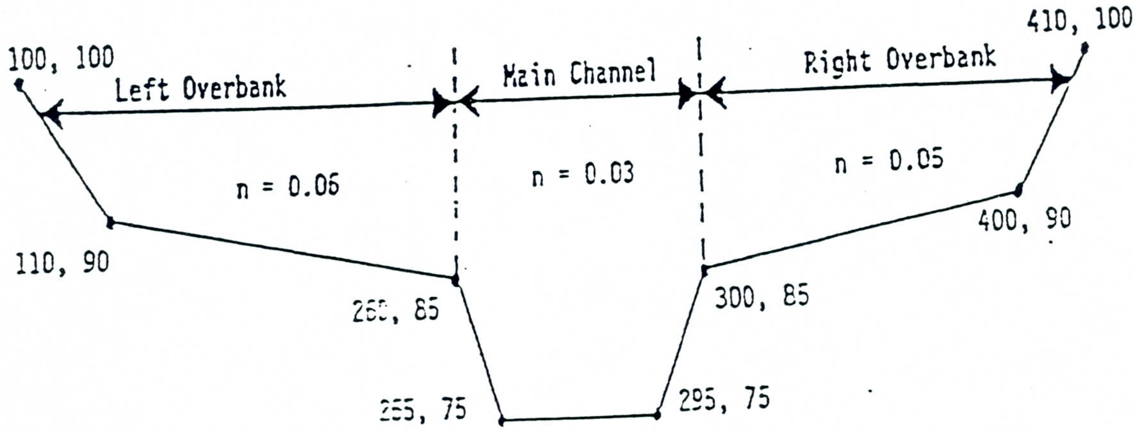


Figure 4. 8-point Cross Section

Listings of the required input data and the resulting output are shown in table 1. For the channel routing from CP1 to CP2, it is only necessary to have an RD record. Use of the RD record by itself means that the channel geometry can be described with a simple geometric element, such as a trapezoid. For the routing reach between CP2 and CP3, it is necessary to also include RC, RX, and RY records to describe the geometry through this reach. When using the 8-point cross-section option, the RD record only serves to indicate a Muskingum-Cunge channel routing is being performed. All of the necessary information is obtained from the RC, RX, and RY records.

TABLE 1

Example Problem : Input and Output

HEC-1 INPUT

```
LINE      ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1       ID   TEST EXAMPLE NO. 15.  MUSKINGUM-CUNGE CHANNEL ROUTING EXAMPLE
  2       ID   GARY W. BRUNNER  APRIL 18, 1989
  3       IT      15 18APR89    1100      60
  4       IO       5
          *

  5       KK    SUB1
  6       KM    RUNOFF CALCULATION FOR SUB1
  7       BA    25.0
  8       PB     3.5
  9       PI     0.2     0.3     0.5     0.8     1.0     0.8     0.6     0.4     0.2     0.1
 10       BF    -1.0    -.05    1.02
 11       LS     0.5      65
 12       UC     3.5     3.0
          *

 13       KK   ROUT1
 14       KM   ROUTE SUB1 HYDROGRAPH FROM CP1 TO CP2
 15       KO       1
 16       RD   31680  0.0008   0.015            TRAP      25     1.0
          *

 17       KK    SUB2
 18       KM    LOCAL RUNOFF FROM SUBBASIN SUB2
 19       BA    35.0
 20       PB     3.0
 21       LS     0.5      75      35
 22       UC     2.8     2.1
          *

 23       KK    SUB2
 24       KM    COMBINE LOCAL SUB2 AND ROUTED SUB1 HYDROGRAPHS
 25       HC       2
          *

 26       KK   ROUT2
 27       KM   ROUTE TOTAL FLOW AT SUB2 FROM CP2 TO CP3
 28       KO       1
 29       RD
 30       RC    0.06    0.03    0.05   29040  0.0007      96
 31       RX     100     110     260     265     295     300     400     410
 32       RY     100      90      85      75      75      85      90     100
          *

 33       KK    SUB3
 34       KM    LOCAL RUNOFF FROM SUBBASIN SUB3
 35       BA    32.5
 36       PB     2.9
 37       LS     0.5      70
 38       UC     4.0     3.5
          *

 39       KK    SUB3
 40       KM    COMBINE LOCAL SUB3 WITH ROUTED FROM SUB2
 41       HC       2
          *

 42       ZZ
```

```
*****************************************                    *****************************************
*                                       *                    *                                       *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                    *   U.S. ARMY CORPS OF ENGINEERS        *
*            FEBRUARY 1981              *                    *   THE HYDROLOGIC ENGINEERING CENTER   *
*         REVISED 05 DEC 88             *                    *        609 SECOND STREET              *
*                                       *                    *     DAVIS, CALIFORNIA 95616           *
*  RUN DATE  05/01/1989  TIME  13:12:37 *                    *        (916) 551-1748                 *
*                                       *                    *                                       *
*****************************************                    *****************************************


                  TEST EXAMPLE NO. 15.  MUSKINGUM-CUNGE CHANNEL ROUTING EXAMPLE
                  GARY W. BRUNNER  APRIL 18, 1989

  4 IO          OUTPUT CONTROL VARIABLES
                      IPRNT          5  PRINT CONTROL
                      IPLOT          0  PLOT CONTROL
                      QSCAL         0.  HYDROGRAPH PLOT SCALE

    IT          HYDROGRAPH TIME DATA
                       NMIN         15  MINUTES IN COMPUTATION INTERVAL
                      IDATE    18APR89  STARTING DATE
                      ITIME       1100  STARTING TIME
                         NQ         60  NUMBER OF HYDROGRAPH ORDINATES
                     NDDATE    19APR89  ENDING DATE
                     NDTIME       0145  ENDING TIME
                      ICENT         19  CENTURY MARK

                COMPUTATION INTERVAL    .25 HOURS
                     TOTAL TIME BASE  14.75 HOURS

        ENGLISH UNITS
            DRAINAGE AREA           SQUARE MILES
            PRECIPITATION DEPTH     INCHES
            LENGTH, ELEVATION       FEET
            FLOW                    CUBIC FEET PER SECOND
            STORAGE VOLUME          ACRE-FEET
            SURFACE AREA            ACRES
            TEMPERATURE             DEGREES FAHRENHEIT

*** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** *** ***
```

```
                ***************
                *             *
   KK           *    ROUT1    *
                *             *
                ***************

15 KO        OUTPUT CONTROL VARIABLES
                    IPRNT          1  PRINT CONTROL
                    IPLOT          0  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE

           HYDROGRAPH ROUTING DATA

16 RD        MUSKINGUM-CUNGE CHANNEL ROUTING
                        L     31680.  CHANNEL LENGTH
                        S      .0008  SLOPE
                        N       .015  CHANNEL ROUGHNESS COEFFICIENT
                       CA        .00  CONTRIBUTING AREA
                    SHAPE       TRAP  CHANNEL SHAPE
                       WD      25.00  BOTTOM WIDTH OR DIAMETER
                        Z       1.00  SIDE SLOPE
                    DXMIN          2  MINIMUM NUMBER OF DX INTERVALS

                                                               ***

PROGRAM COMPUTED DELTA-T =      12.00 MIN.
PROGRAM COMPUTED DELTA-X =      2880.  FT.

INFLOW VOLUME + BASEFLOW IN CHANNEL      =      1435.133 ACRE-FEET
OUTFLOW VOLUME + VOLUME LEFT IN CHANNEL  =      1434.975 ACRE-FEET

COMPUTED PEAK FLOW =       3330.35   CFS
```

HYDROGRAPH AT STATION ROUT1

DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW	DA MON HRMN	ORD	FLOW
18 APR 1100	1	25.	18 APR 1445	16	2065.	18 APR 1830	31	1858.	18 APR 2215	46	607.
18 APR 1115	2	25.	18 APR 1500	17	2481.	18 APR 1845	32	1721.	18 APR 2230	47	565.
18 APR 1130	3	25.	18 APR 1515	18	2818.	18 APR 1900	33	1594.	18 APR 2245	48	526.
18 APR 1145	4	25.	18 APR 1530	19	3077.	18 APR 1915	34	1479.	18 APR 2300	49	489.
18 APR 1200	5	25.	18 APR 1545	20	3248.	18 APR 1930	35	1371.	18 APR 2315	50	456.
18 APR 1215	6	25.	18 APR 1600	21	3330.	18 APR 1945	36	1271.	18 APR 2330	51	425.
18 APR 1230	7	25.	18 APR 1615	22	3315.	18 APR 2000	37	1179.	18 APR 2345	52	396.
18 APR 1245	8	25.	18 APR 1630	23	3228.	18 APR 2015	38	1094.	19 APR 0000	53	369.
18 APR 1300	9	25.	18 APR 1645	24	3085.	18 APR 2030	39	1016.	19 APR 0015	54	344.
18 APR 1315	10	25.	18 APR 1700	25	2907.	18 APR 2045	40	943.	19 APR 0030	55	321.
18 APR 1330	11	25.	18 APR 1715	26	2714.	18 APR 2100	41	875.	19 APR 0045	56	300.
18 APR 1345	12	34.	18 APR 1730	27	2522.	18 APR 2115	42	813.	19 APR 0100	57	280.
18 APR 1400	13	346.	18 APR 1745	28	2337.	18 APR 2130	43	756.	19 APR 0115	58	262.
18 APR 1415	14	1043.	18 APR 1800	29	2164.	18 APR 2145	44	702.	19 APR 0130	59	245.
18 APR 1430	15	1593.	18 APR 1815	30	2005.	18 APR 2200	45	653.	19 APR 0145	60	238.

PEAK FLOW	TIME		MAXIMUM AVERAGE FLOW			
			6-HR	24-HR	72-HR	14.75-HR
+ (CFS)	(HR)					
		(CFS)				
+ 3330.	5.00		2268.	1146.	1146.	1146.
		(INCHES)	.843	1.048	1.048	1.048
		(AC-FT)	1125.	1397.	1397.	1397.

CUMULATIVE AREA = 25.00 SQ MI

```
              ***************
              *             *
KK            *    ROUT2    *
              *             *
              ***************
```

28 KO OUTPUT CONTROL VARIABLES

	IPRNT	1	PRINT CONTROL
	IPLOT	0	PLOT CONTROL
	QSCAL	0.	HYDROGRAPH PLOT SCALE

HYDROGRAPH ROUTING DATA

29 RD MUSKINGUM-CUNGE CHANNEL ROUTING

30 RC NORMAL DEPTH CHANNEL

	ANL	.060	LEFT OVERBANK N-VALUE
	ANCH	.030	MAIN CHANNEL N-VALUE
	ANR	.050	RIGHT OVERBANK N-VALUE
	RLNTH	29040.	REACH LENGTH
	SEL	.0007	ENERGY SLOPE
	ELMAX	96.0	MAX. ELEV. FOR STORAGE/OUTFLOW CALCULATION

CROSS-SECTION DATA

		--- LEFT OVERBANK ---		+ ------ MAIN CHANNEL		------- +	--- RIGHT OVERBANK ---		
32 RY	ELEVATION	100.00	90.00	85.00	75.00	75.00	85.00	90.00	100.00
31 RX	DISTANCE	100.00	110.00	260.00	265.00	295.00	300.00	400.00	410.00

***

COMPUTED STORAGE-OUTFLOW-ELEVATION DATA

STORAGE	.00	22.51	45.84	69.98	94.94	120.71	147.29	174.69	202.90	231.93
OUTFLOW	.00	45.55	141.88	274.39	437.13	626.66	840.79	1078.04	1337.39	1618.12
ELEVATION	75.00	76.11	77.21	78.32	79.42	80.53	81.63	82.74	83.84	84.95
STORAGE	279.87	368.49	497.82	667.88	873.06	1090.26	1307.08	1525.54	1745.62	1967.33
OUTFLOW	1984.99	2443.41	3029.47	3774.70	4754.58	5986.67	7389.58	8951.15	10662.29	12515.79
ELEVATION	86.05	87.16	88.26	89.37	90.47	91.58	92.68	93.79	94.89	96.00

PROGRAM COMPUTED DELTA-T = 12.00 MIN.
PROGRAM COMPUTED DELTA-X = 3630. FT.

INFLOW VOLUME + BASEFLOW IN CHANNEL = 4788.254 ACRE-FEET
OUTFLOW VOLUME + VOLUME LEFT IN CHANNEL = 4747.717 ACRE-FEET

COMPUTED PEAK FLOW = 9998.17 CFS

HYDROGRAPH AT STATION ROUT2

DA MON HRMN	ORD	FLOW
18 APR 1100	1	60.
18 APR 1115	2	60.
18 APR 1130	3	60.
18 APR 1145	4	60.
18 APR 1200	5	60.
18 APR 1215	6	60.
18 APR 1230	7	60.
18 APR 1245	8	60.
18 APR 1300	9	60.
18 APR 1315	10	122.
18 APR 1330	11	519.
18 APR 1345	12	1112.
18 APR 1400	13	1660.
18 APR 1415	14	2077.
18 APR 1430	15	2503.
18 APR 1445	16	3055.
18 APR 1500	17	3809.
18 APR 1515	18	4866.
18 APR 1530	19	6114.
18 APR 1545	20	7359.
18 APR 1600	21	8441.
18 APR 1615	22	9221.
18 APR 1630	23	9721.
18 APR 1645	24	9965.
18 APR 1700	25	9993.
18 APR 1715	26	9829.
18 APR 1730	27	9547.
18 APR 1745	28	9184.
18 APR 1800	29	8769.
18 APR 1815	30	8324.
18 APR 1830	31	7873.
18 APR 1845	32	7428.
18 APR 1900	33	6997.
18 APR 1915	34	6590.
18 APR 1930	35	6206.
18 APR 1945	36	5845.
18 APR 2000	37	5506.
18 APR 2015	38	5186.
18 APR 2030	39	4876.
18 APR 2045	40	4569.
18 APR 2100	41	4259.
18 APR 2115	42	3937.
18 APR 2130	43	3591.
18 APR 2145	44	3211.
18 APR 2200	45	2777.
18 APR 2215	46	2296.
18 APR 2230	47	1861.
18 APR 2245	48	1569.
18 APR 2300	49	1385.
18 APR 2315	50	1267.
18 APR 2330	51	1180.
18 APR 2345	52	1113.
19 APR 0000	53	1059.
19 APR 0015	54	1014.
19 APR 0030	55	974.
19 APR 0045	56	938.
19 APR 0100	57	905.
19 APR 0115	58	875.
19 APR 0130	59	847.
19 APR 0145	60	836.

PEAK FLOW (CFS)	TIME (HR)		MAXIMUM AVERAGE FLOW 6-HR	24-HR	72-HR	14.75-HR
+ 9993.	6.00	(CFS)	7352.	3784.	3784.	3784.
		(INCHES)	1.139	1.441	1.441	1.441
		(AC-FT)	3646.	4613.	4613.	4613.

CUMULATIVE AREA = 60.00 SQ MI

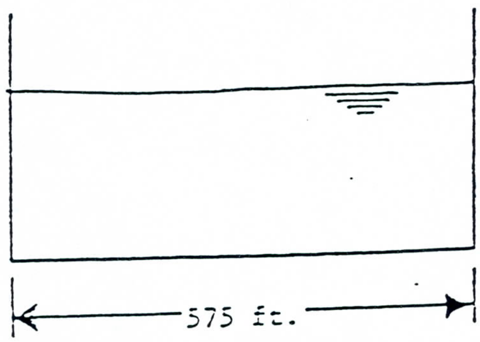
RUNOFF SUMMARY
FLOW IN CUBIC FEET PER SECOND
TIME IN HOURS, AREA IN SQUARE MILES

OPERATION	STATION	PEAK FLOW	TIME OF PEAK	AVERAGE FLOW FOR MAXIMUM PERIOD 6-HOUR	24-HOUR	72-HOUR	BASIN AREA	MAXIMUM STAGE	TIME OF MAX STAGE
HYDROGRAPH AT	SUB1	3381.	4.50	2285.	1169.	1169.	25.00		
ROUTED TO	ROUT1	3330.	5.00	2268.	1146.	1146.	25.00		
HYDROGRAPH AT	SUB2	9862.	3.75	5816.	2763.	2763.	35.00		
2 COMBINED AT	SUB2	12131.	4.00	7807.	3909.	3909.	60.00		
ROUTED TO	ROUT2	9993.	6.00	7352.	3784.	3784.	60.00		
HYDROGRAPH AT	SUB3	3091.	5.00	2225.	1202.	1202.	32.50		
2 COMBINED AT	SUB3	12794.	5.75	9422.	4986.	4986.	92.50		

VI. COMPARISON WITH THE COMPLETE UNSTEADY FLOW EQUATIONS

In an effort to quantify the applicability and limitations of the Muskingum-Cunge routing technique, a comparison with the complete unsteady flow equations was undertaken. This analysis consisted of comparisons for prismatic channels of rectangular cross section, as well as more detailed compound cross sections (8 point cross sections). The analysis encompassed a wide range of channel slopes, varying from 42 ft/mi to 1 ft/mi. Rapidly rising hydrographs as well as slow rising hydrographs were routed through long channel sections with no lateral inflow. This analysis represents a very controlled routing situation, which is necessary to make a clear comparison between the variable coefficient Muckingum-Cunge method and the complete unsteady flow equations.

The first set of tests were for a rectangular channel with the following dimensions:



Channel Length - 95040 ft.

Manning's n - 0.03

Channel Slopes - 1 to 10 ft/mi.

Figure 5. Rectangular channel section with varying channel slopes.

Hydrographs were routed with the Muskingum-Cunge routing technique in HEC-1. The same channels and hydrographs were then analyzed with the National Weather Service DAMBRK model. This model was chosen as the standard for comparison because it has been nationally accepted and is considered one of the most accurate tools available for one dimensional channel flow. Extreme care was taken to ensure that the best possible answer was obtained with the DAMBRK model. Plots of the inflow and respective outflow hydrographs are shown in Figures 6 through 10. As shown in the plots, the Muskingum-Cunge method compares very well with the complete unsteady flow equations (DAMBRK model). The Muskingum-Cunge method begins to diverge from the DAMBRK answer when the channel slope is reduced to 1 ft/mi or less. The divergence is due to the fact that the inertial terms in the complete unsteady flow equations are becoming more dominant, compared to the bed slope, as the channel slope is decreased. The Muskingum-Cunge method does not account for the inertial effects, and consequently the method tends to show more diffusion than what may actually occur.

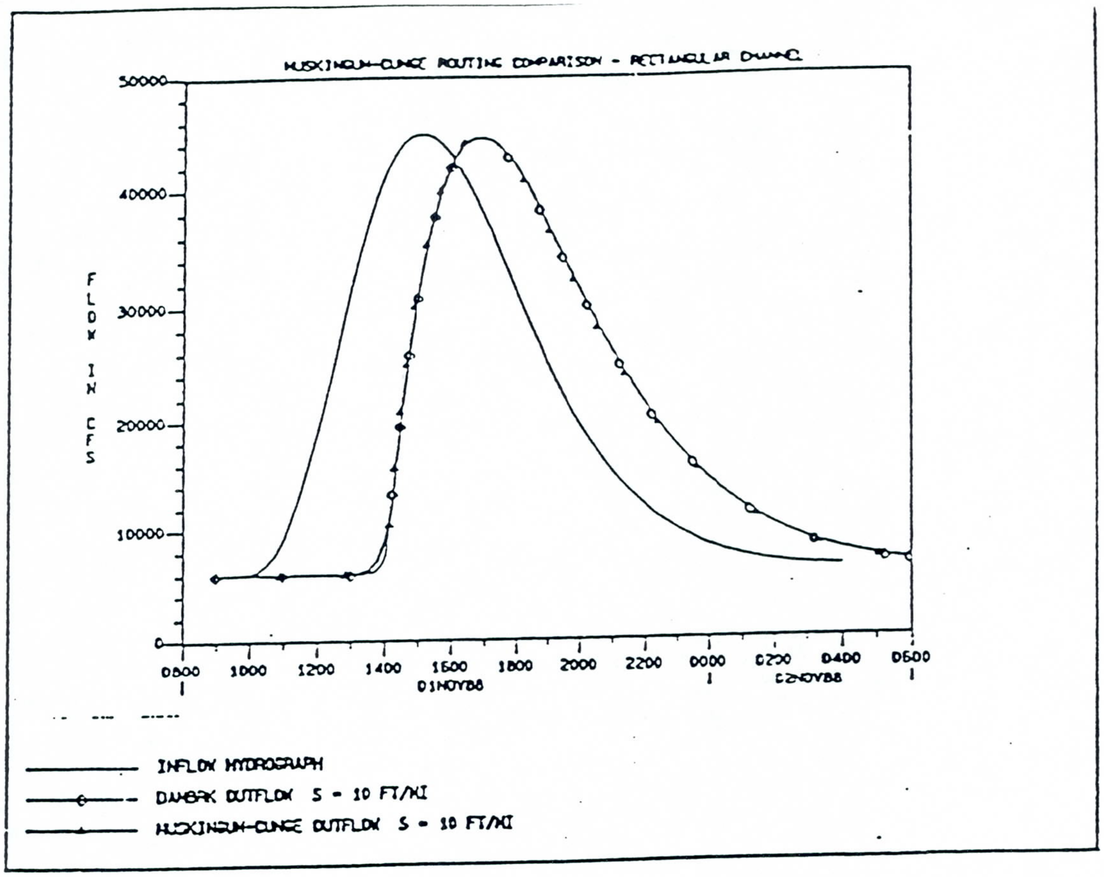


Figure 6. Rectangular channel with S=10 ft/mi (0.0019)

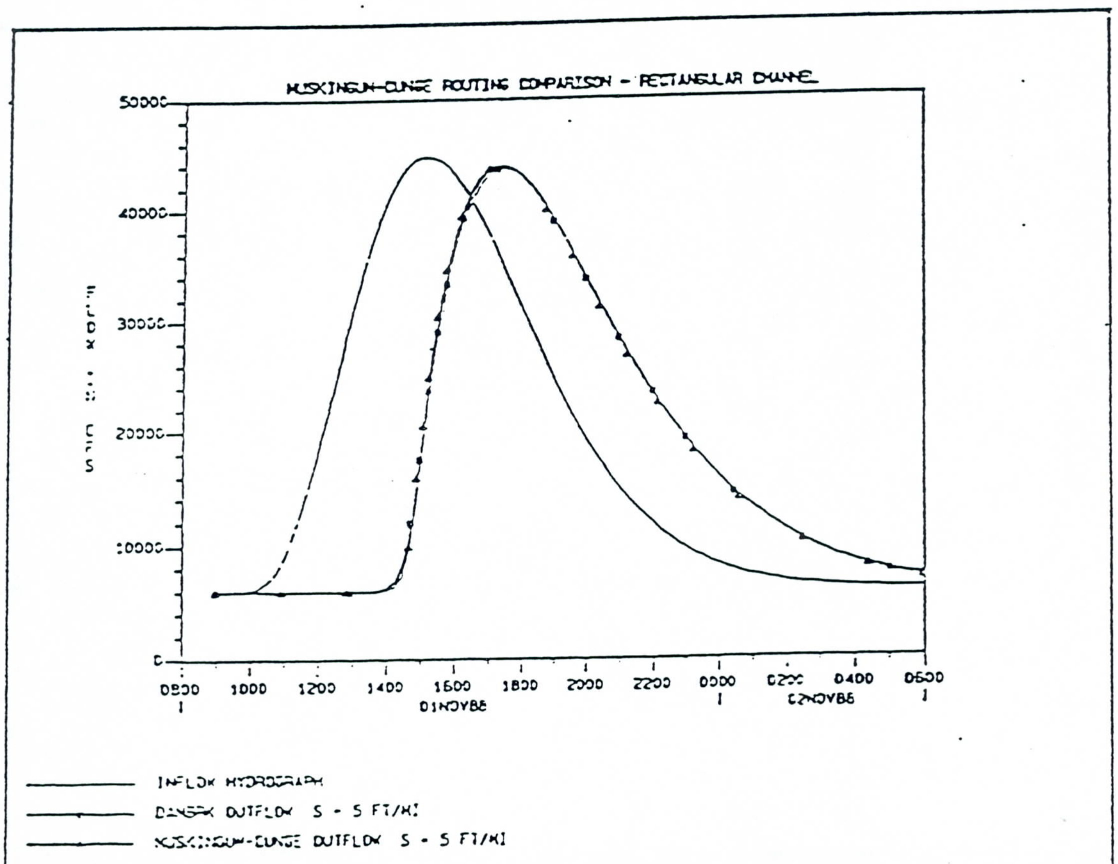


Figure 7. Rectangular channel with S=5 ft/mi (0.00095)

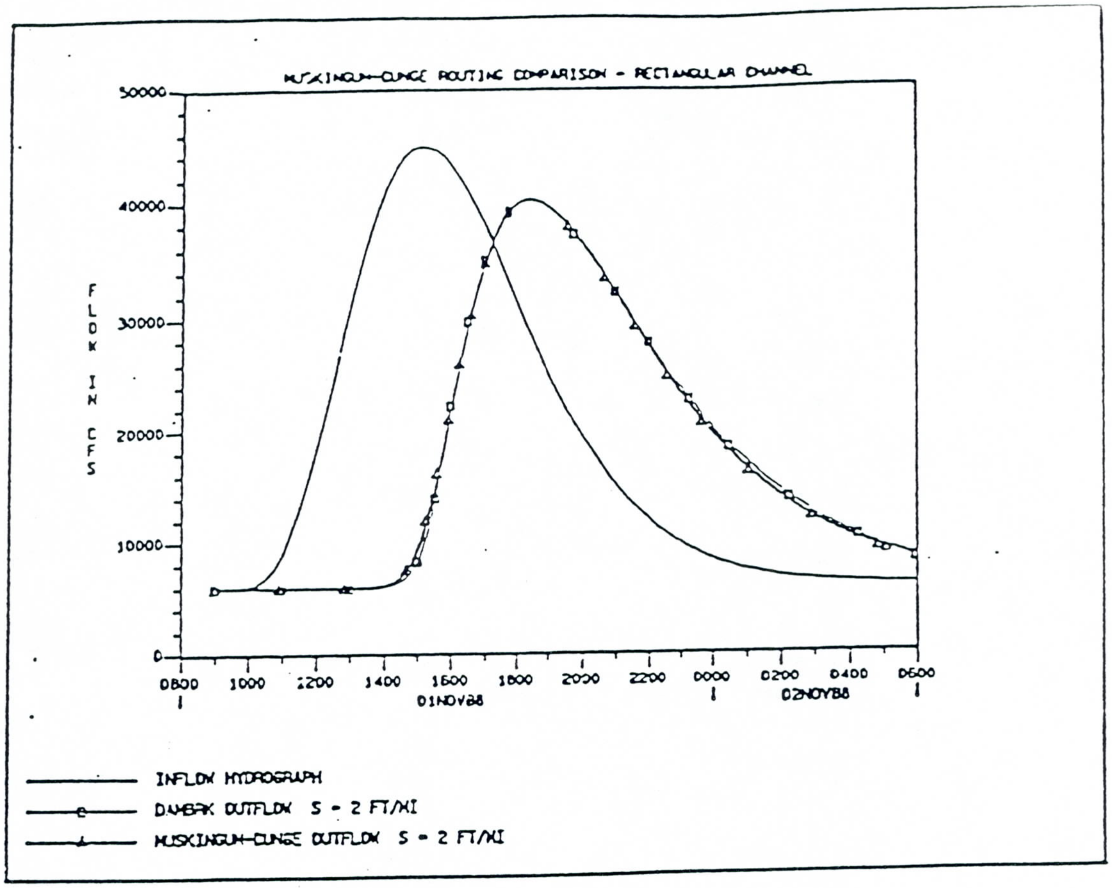


Figure 8. Rectangular channel with S=2 ft/mi (0.00038)

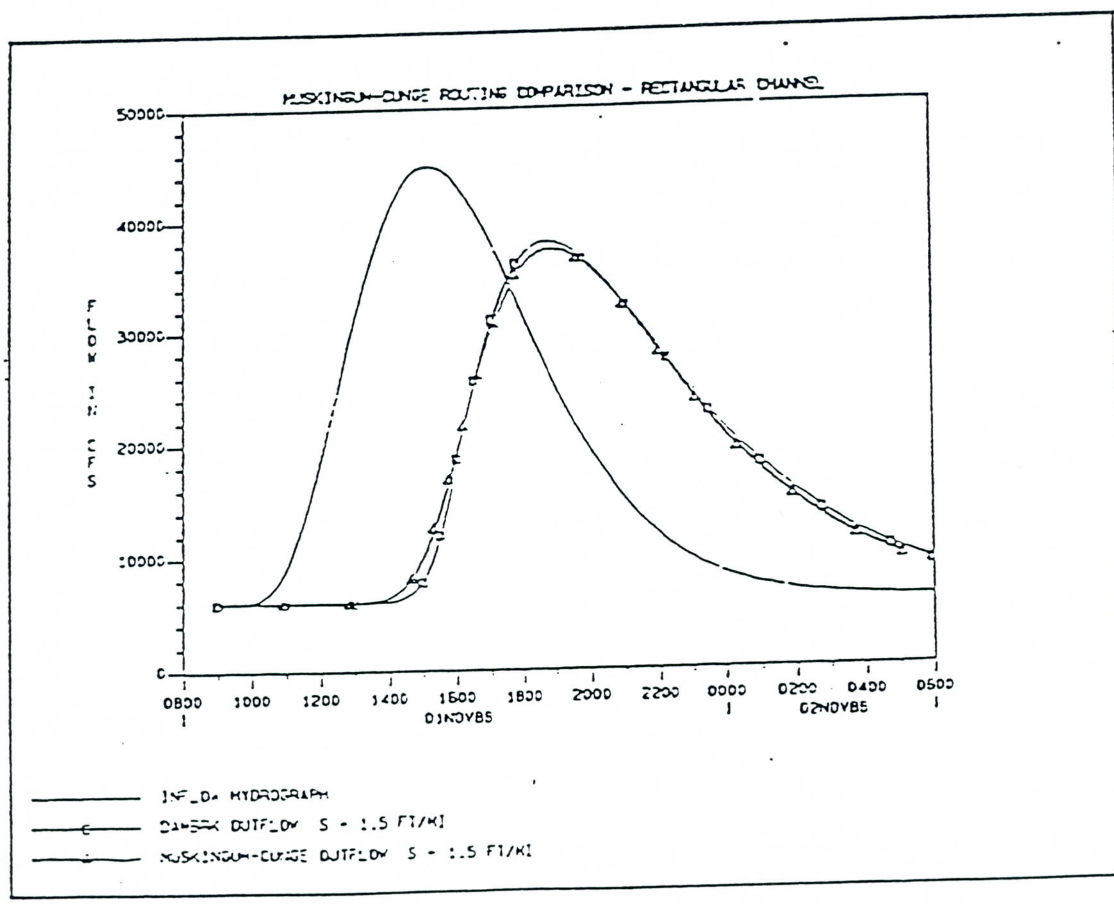


Figure 9. Rectangular channel with S=1.5 ft/mi (0.00028)

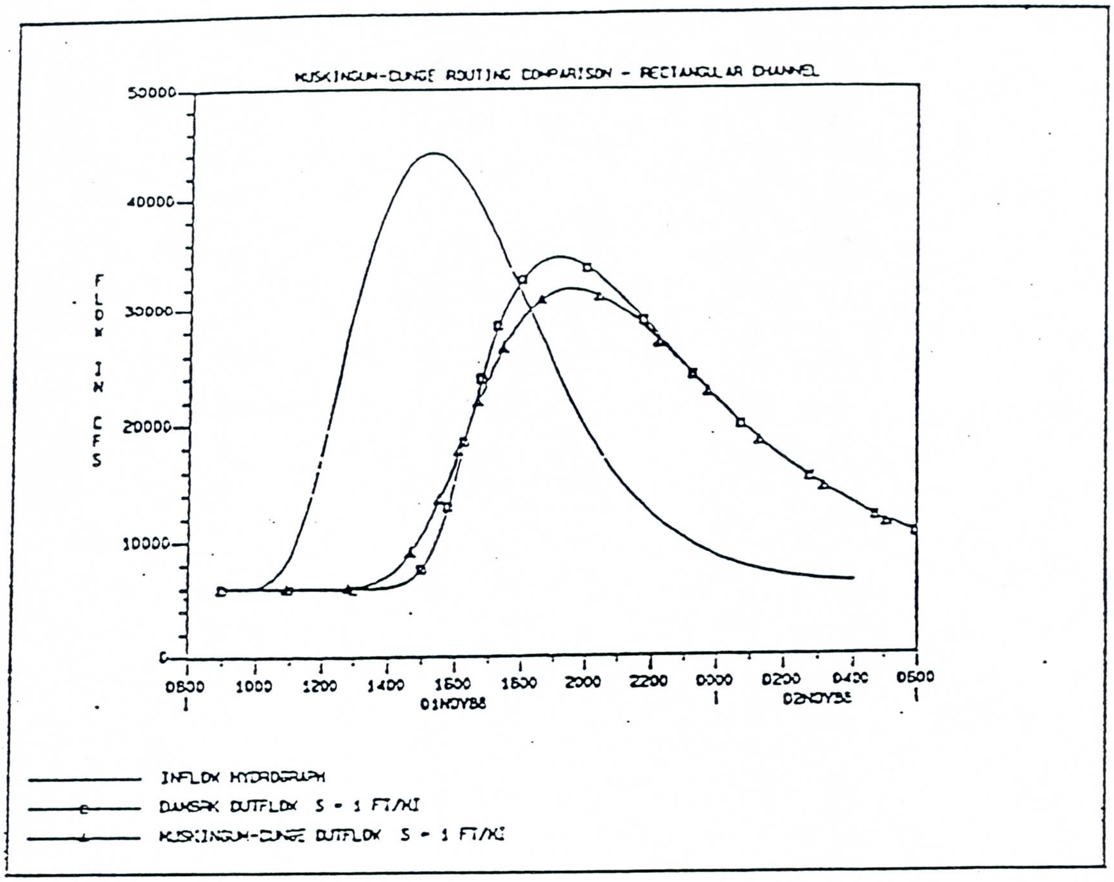


Figure 10. Rectangular channel with S=1 ft/mi (0.00019)

In the second series of tests, the effects of varying the rise time of the inflow hydrograph, as well as channel slope, were analyzed. In this analysis two different inflow hydrographs were used. The first inflow hydrograph has a time of rise of 45 minutes, peak flow of 70,622 cfs, and a time base of runoff equal to 2 hours. The second inflow hydrograph has a time of rise of 2 hours, peak flow of 70,622 cfs, and a time base of runoff equal to 6 hours. Channel slopes for this example were varied from 42 ft/mi to 1 ft/mi. The channel section was rectangular with the following hydrualic characteristics:

Channel length = 82,025 ft.

Manning's n = 0.04

Channel slopes = 1 to 42 ft/mi

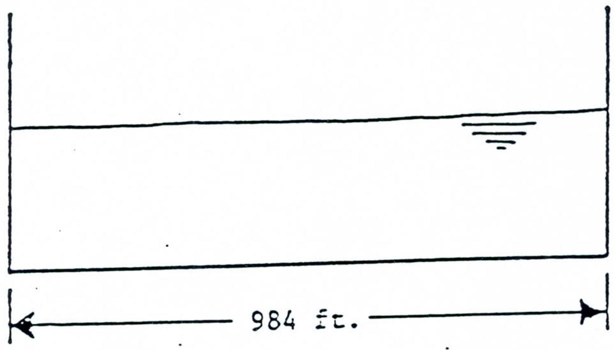


Figure 11. Rectangular channel with slopes from 42 ft/mi to 1 ft/mi.

Hydrographs were routed with the Muskingum-Cunge method as well as the NWS DAMBRK program. The resulting hydrographs are shown in figures 12 through 19. In general, the Muskingum-Cunge method compared very well for this series of tests. From review of the hydrograph plots, it is evident that the model performs better for slow rising hydrographs through steep channel sections. For rapidly rising hydrographs routed through flat river reaches, the Muskingum-Cunge method will tend to over predict the amount of diffusion. Although, the answers produced by the Muskingum-Cunge method may be within practical engineering limits. Also, these tests were performed for very long routing reaches with no lateral inflow, which is more of a dam breach type of analysis. For natural flood events, where lateral inflow will be added to the stream, the model will perform better over a wider range of channel slopes.

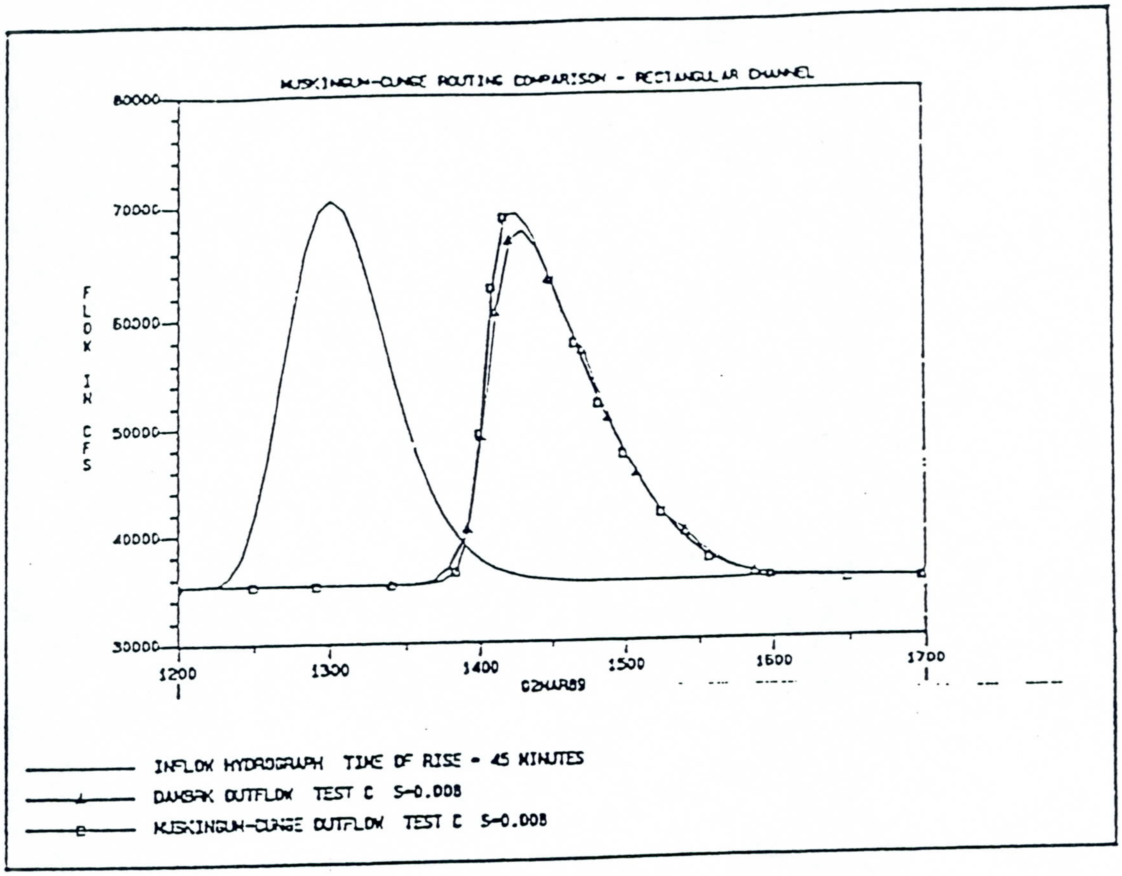


Figure 12. Rectangular channel, time of rise = 45 min, S = 0.008

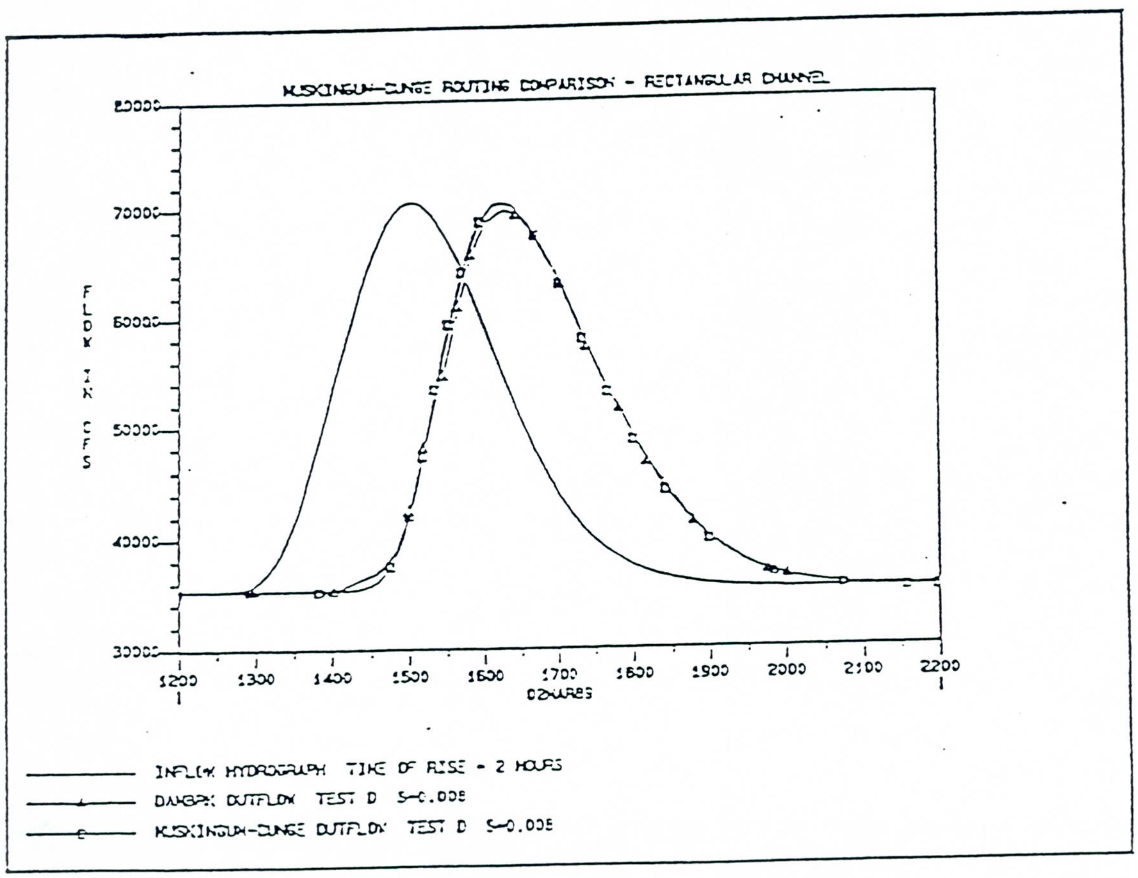


Figure 13. Rectangular channel, time of rise = 2 hrs, S = 0.008

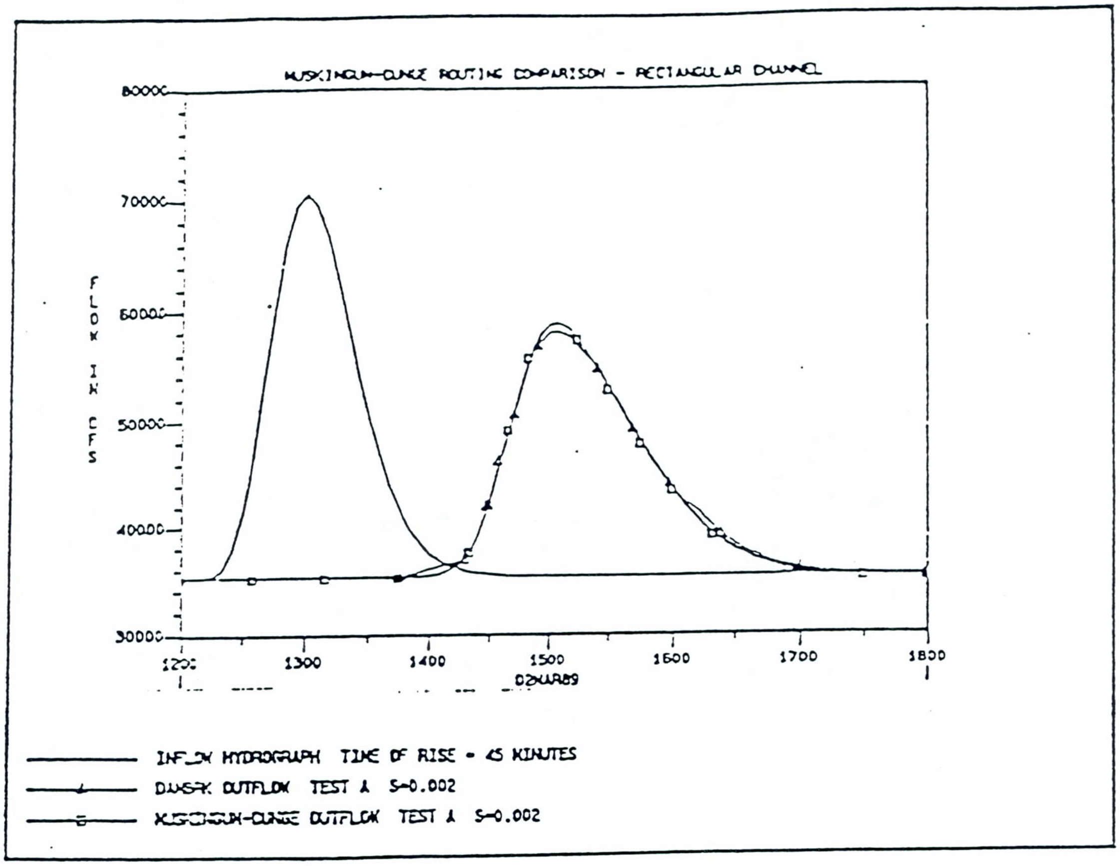


Figure 14. Rectangular channel, time of rise = 45 min, S = 0.002

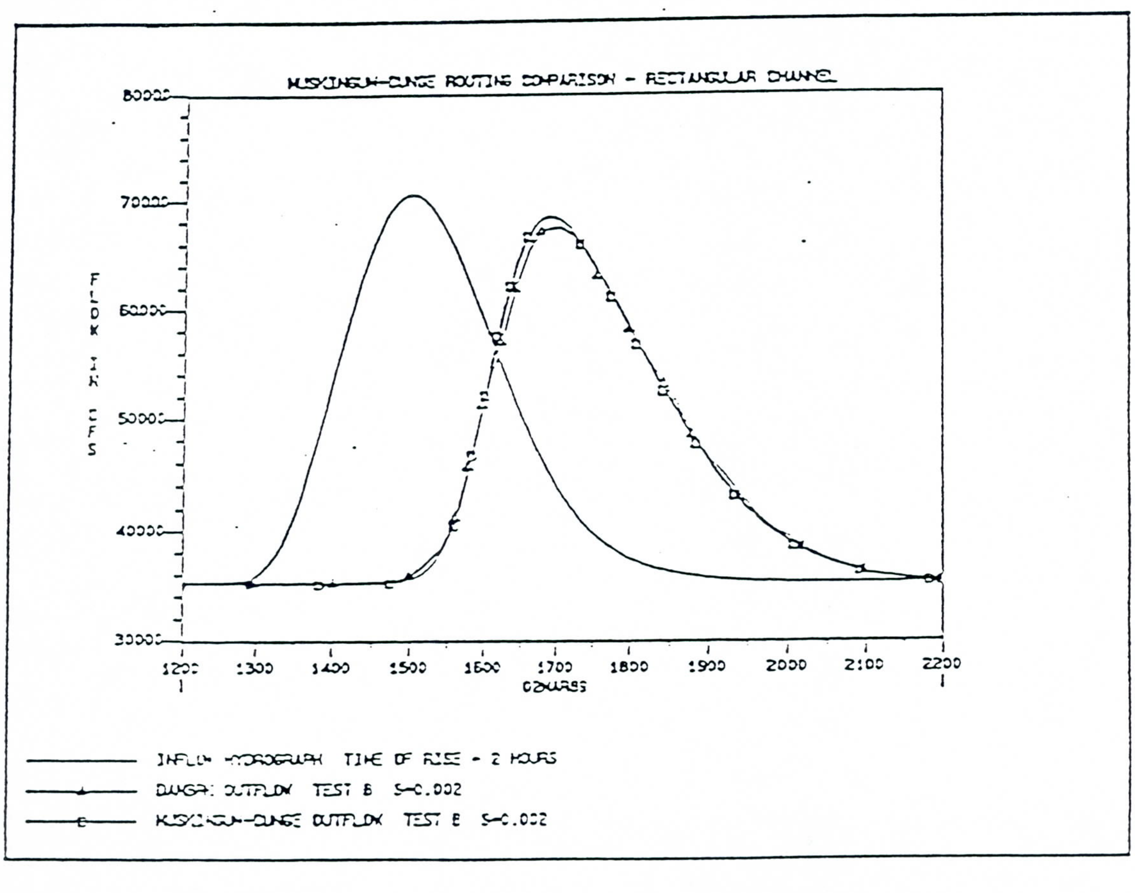


Figure 15. Rectangular channel, time of rise = 2 hrs, S = 0.002

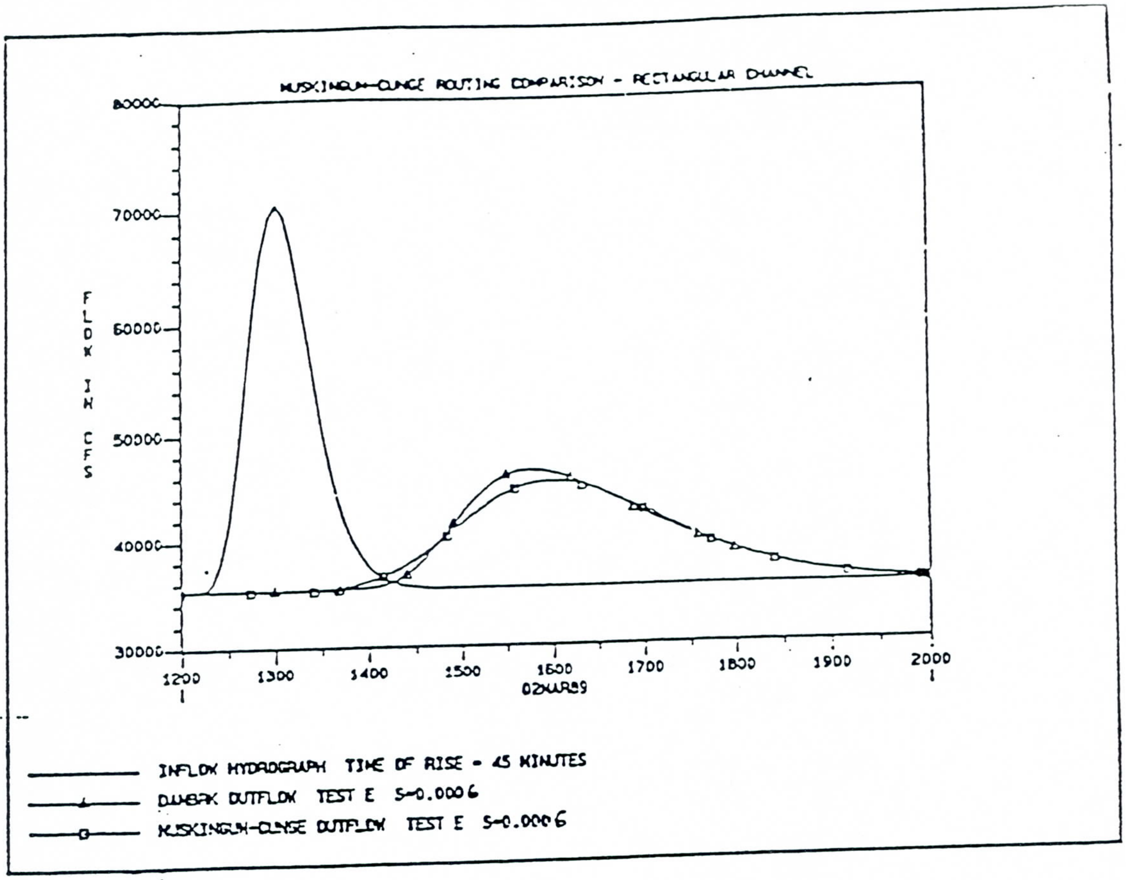


Figure 16. Rectangular channel, time of rise = 45 min, S = 0.0006

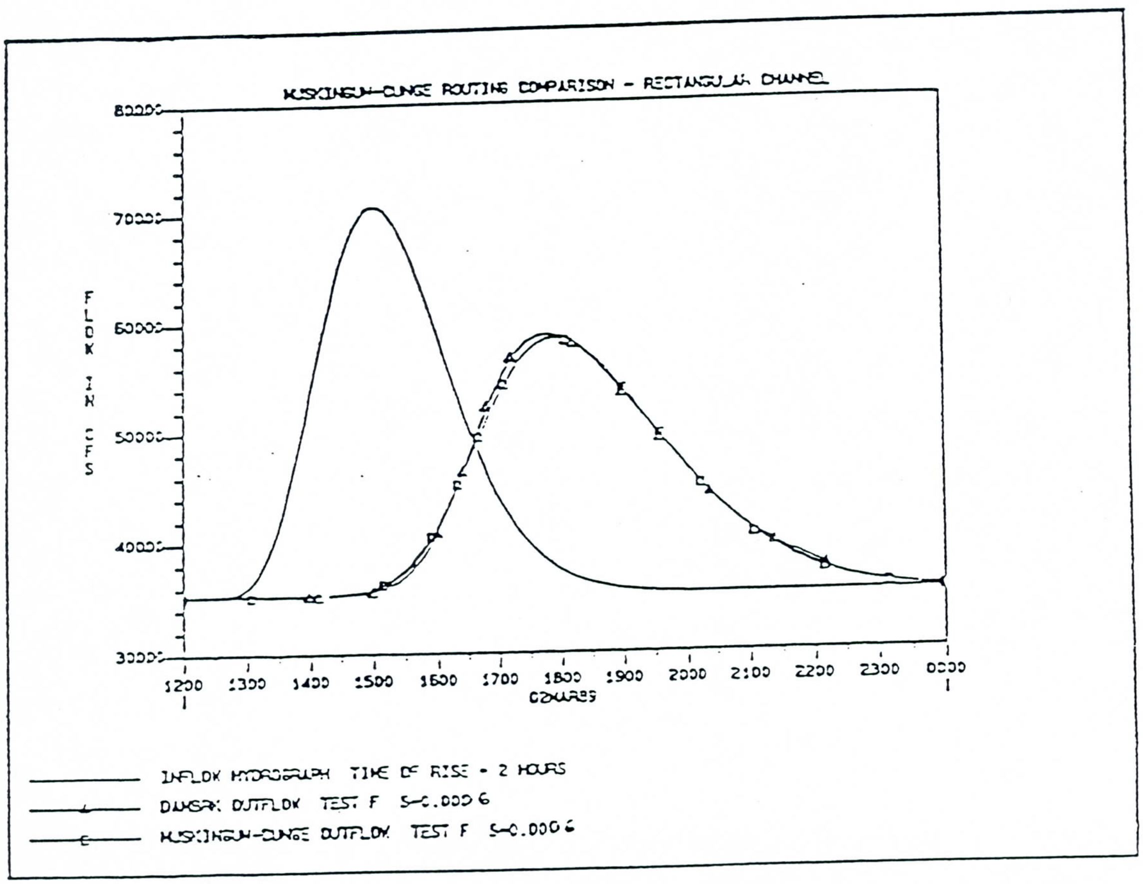


Figure 17. Rectangular channel, time of rise = 2 hrs, S = 0.0006

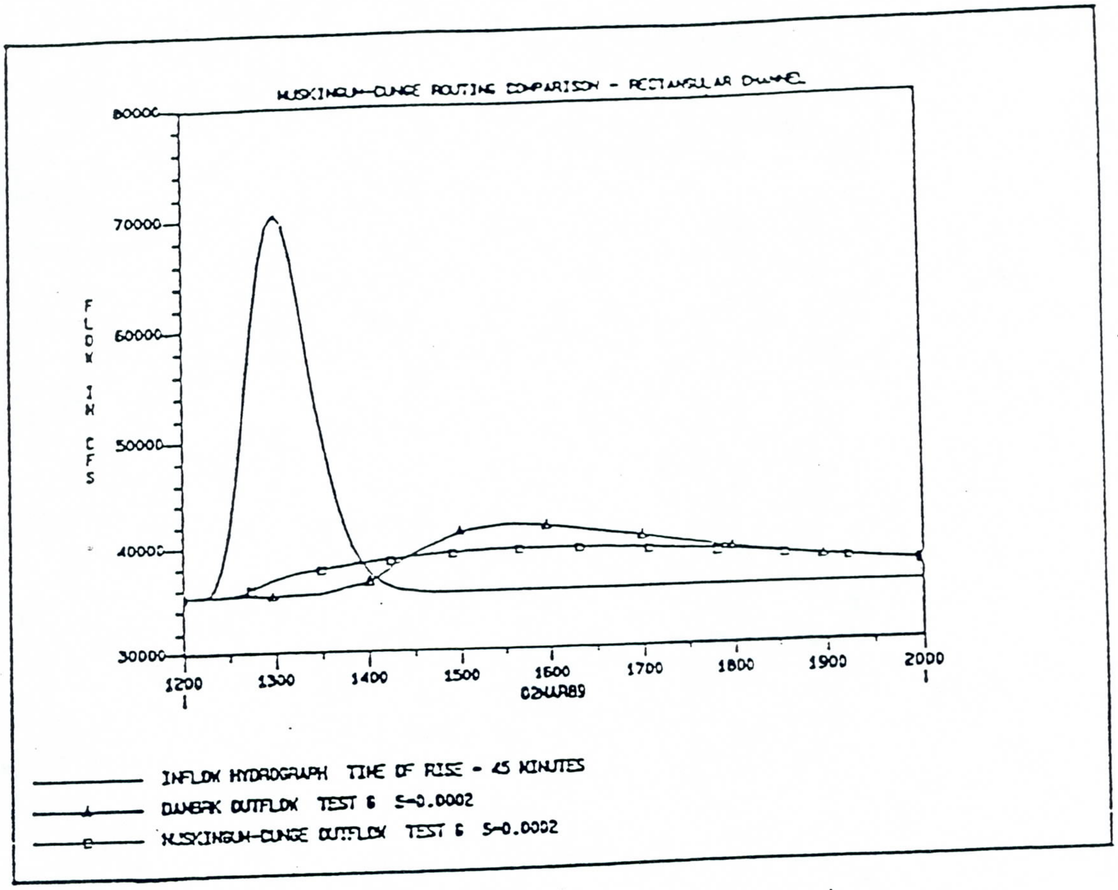


Figure 18. Rectangular channel, time of rise = 45 min, S = 0.0002

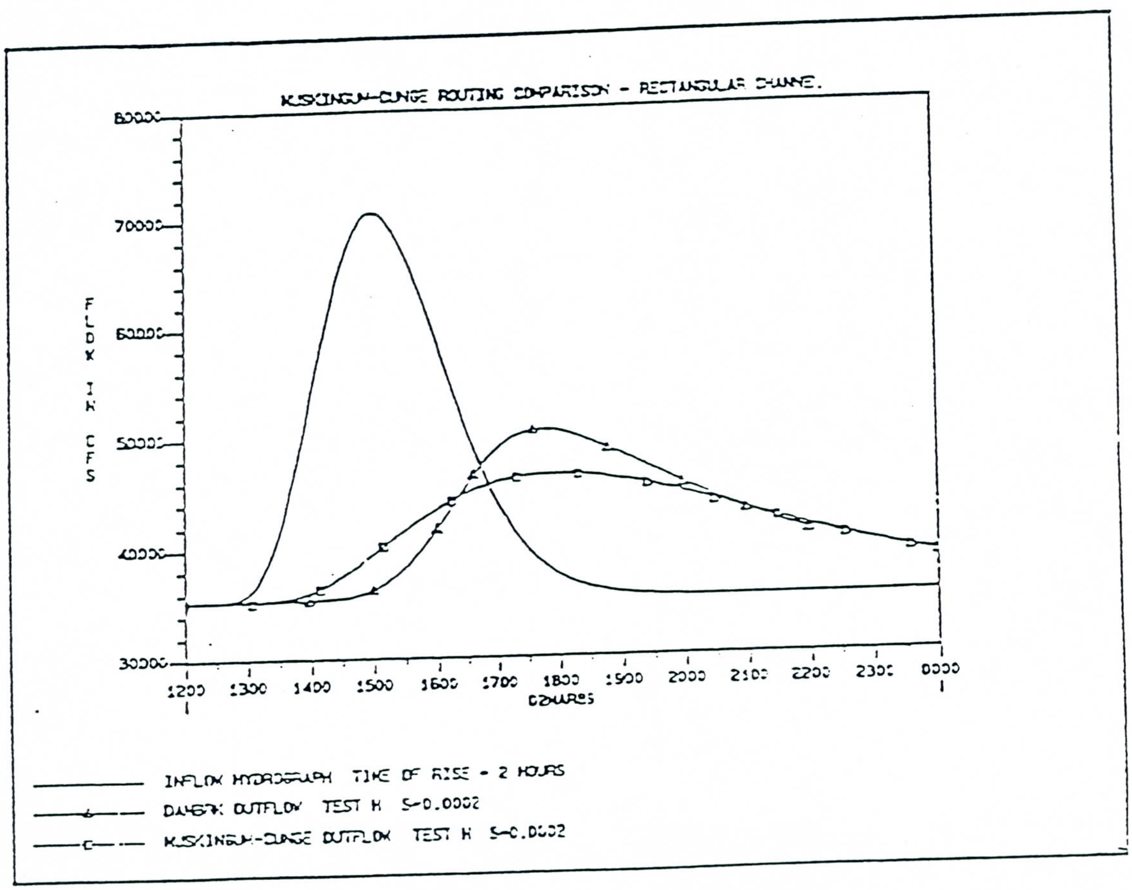


Figure 19. Rectangular channel, time of rise = 2 hrs, S = 0.0002

A third set of tests was performed for compound channel cross sections. A limited number of tests were run to analyze how well the Muskingum-Cunge method would compare to the full unsteady flow equations for channels with overbank flows. Shown in figures 20 through 25 are three different compound cross sections and the respective hydrographs from DAMBRK and the Muskingum-Cunge method. As shown in the plots, the Muskingum-Cunge method matches the DAMBRK hydrographs extremely well. However, for compound cross sections with very flat overbanks, the variable parameter Muskingum-Cunge method tends to lose volume. In general, variable coefficient methods have a tendency not to conserve mass. The error in mass conservation tends to be small (0 to 4 percent) and is not considered a significant problem.

The final set of tests compare the Muskingum-Cunge method with the traditional Muskingum method and the Normal Depth routing technique in HEC-1. The rectangular channel from the first series of tests (Figure 5) was used in this analysis. The resulting hydrographs are shown in figures 26 and 27. Both the Muskingum method and the Normal Depth routing technique had to be calibrated in order to match the results from DAMBRK. With the Muskingum method, it is necessary to calibrate all three parameters, K (travel time of the channel), X (weighting factor), and NSTPS (number of routing steps). The Muskingum method is considered a linear routing technique in that the parameters remain constant during the routing computations. Because of the linear nature of the traditional Muskingum method, it was not possible to match the shape of the DAMBRK hydrograph. This is evident in figure 25, where the traditional Muskingum method begins to rise much sooner than the DAMBRK and the Muskingum-Cunge hydrographs. This is typical of linear coefficient models.

The Normal Depth routing technique was able to match the DAMBRK hydrograph extremely well. The only draw back of this method is that the parameter NSTPS had to be calibrated. An equation for estimating NSTPS is provided in the HEC-1 manual. Unfortunately, this equation only ensures numerical stability during the computation, and does not guaranty accuracy.

VII. SUMMARY AND CONCLUSIONS

The numerical and physical basis for the Muskingum-Cunge channel routing technique were presented herein. This routing technique is considered a non-linear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The advantages of this method over other hydrologic techniques are: (1) the parameters of the model are physically based, and therefore this method will make for a good ungaged routing technique; (2) the method has been shown to compare well against the complete unsteady flow equations for one dimensional flow; and (3) the solution is independent of the user specified computation interval. The major limitations of the Muskingum-Cunge technique are that: (1) the method can not account for backwater effects; and (2) the method begins to diverge from the complete unsteady flow solution when very rapidly rising hydrographs are routed through flat channel sections (i.e. channel slopes less than 1 ft/mi).

Channel length = 95,040 ft
Channel slope = 10 ft/mi

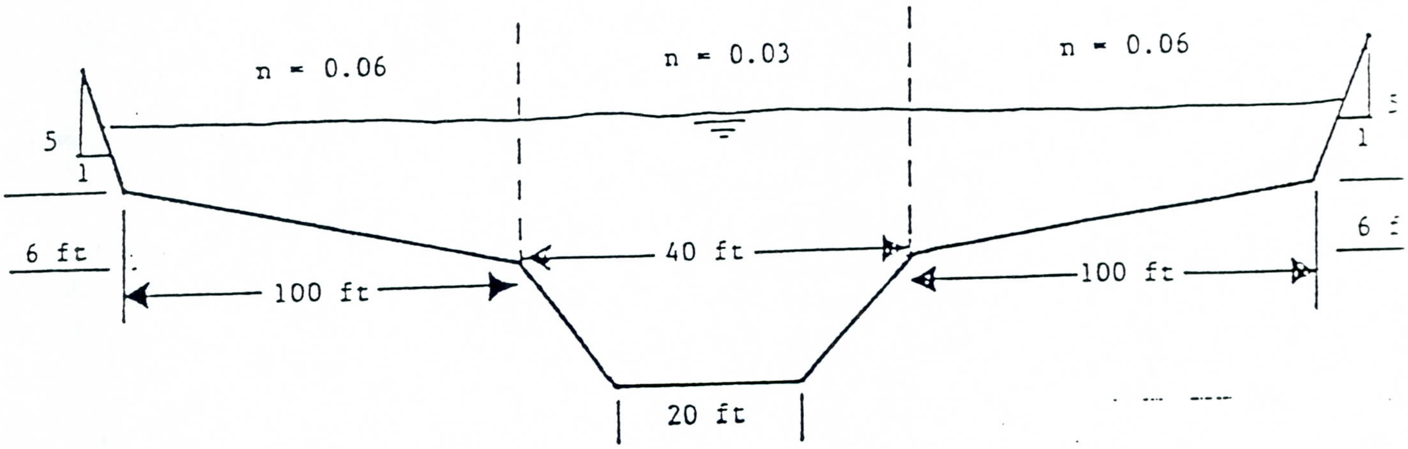


Figure 20. Compound cross section No. 1

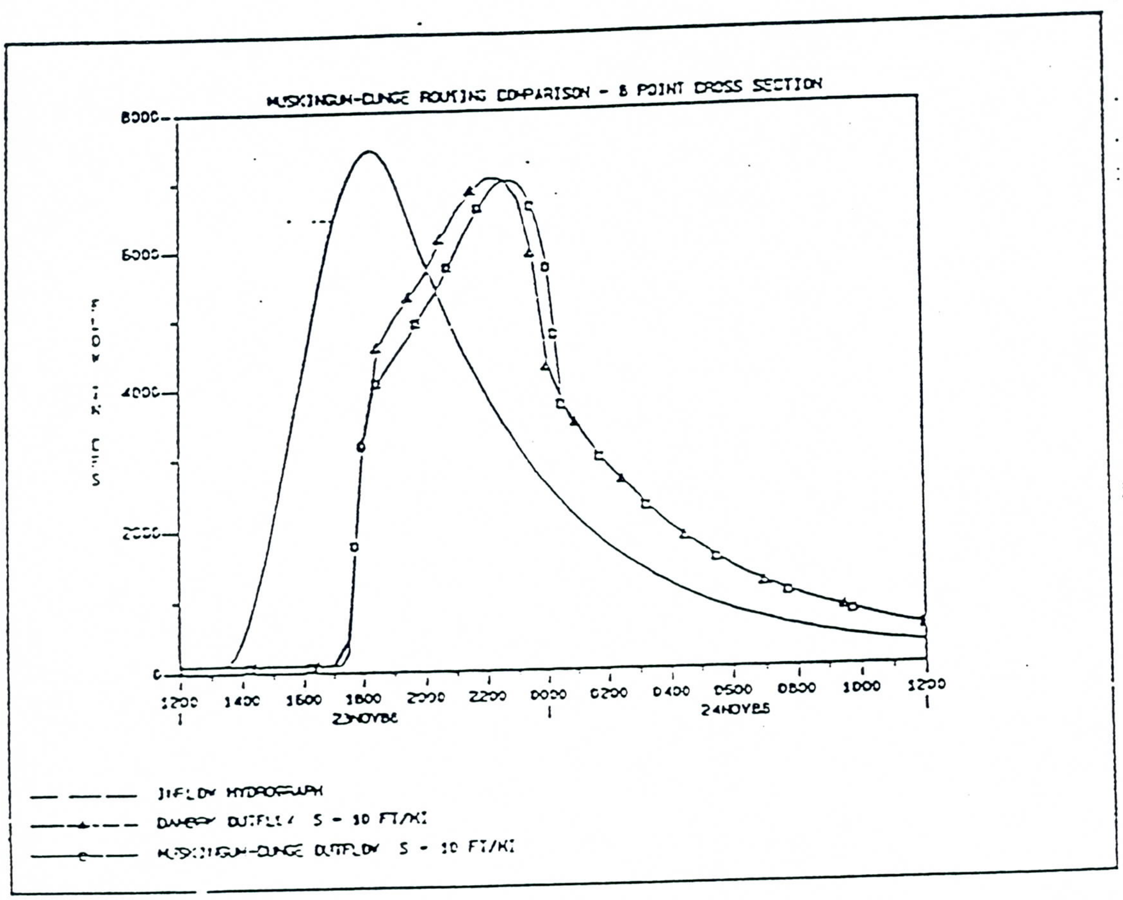


Figure 21. Resulting hydrographs from compound cross section No. 1

Channel length = 95,040 ft
Channel slope = 10 ft/mi

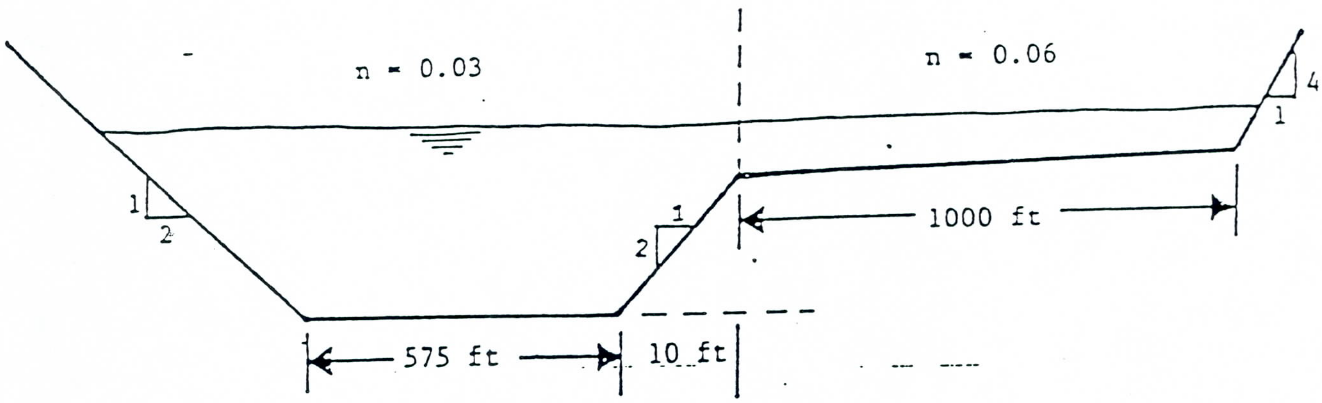


Figure 22. Compound cross section No. 2

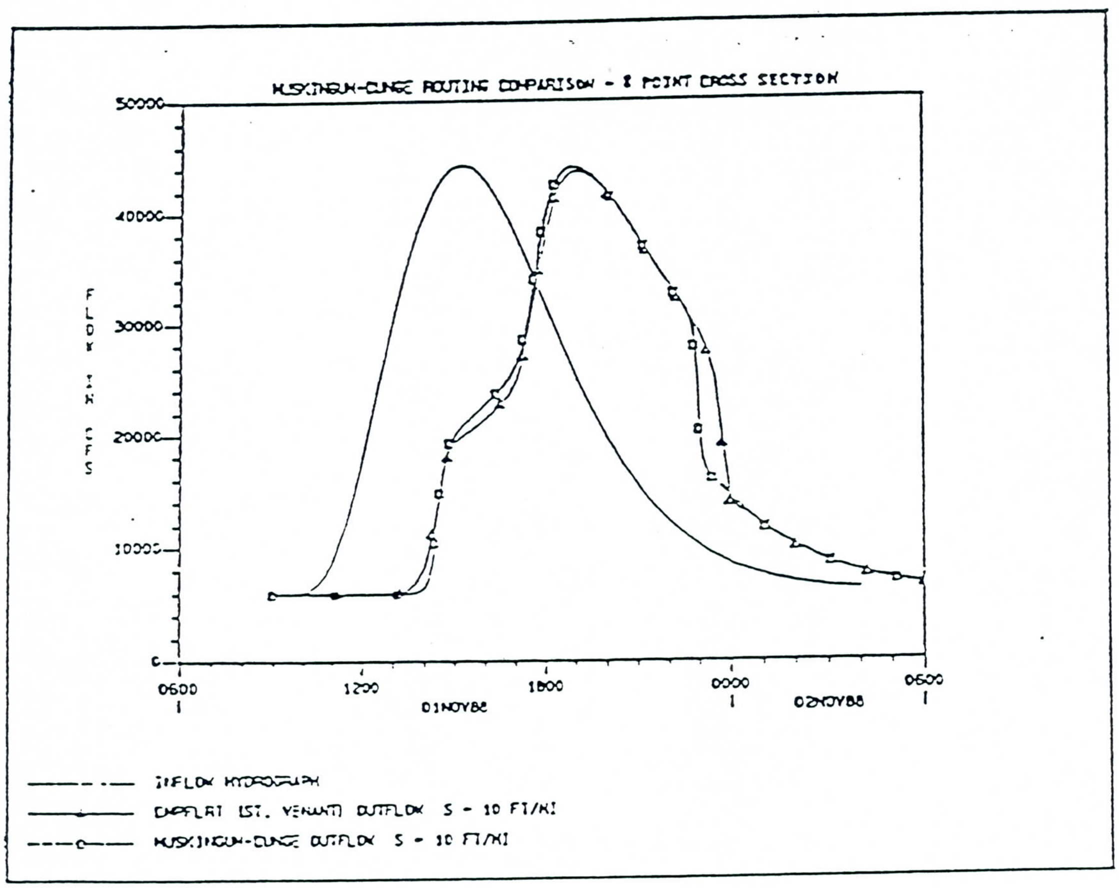


Figure 23. Resulting hydrpgraphs from compound cross section No. 2

Channel length = 82,025 ft
Channel slope = 10 ft/mi

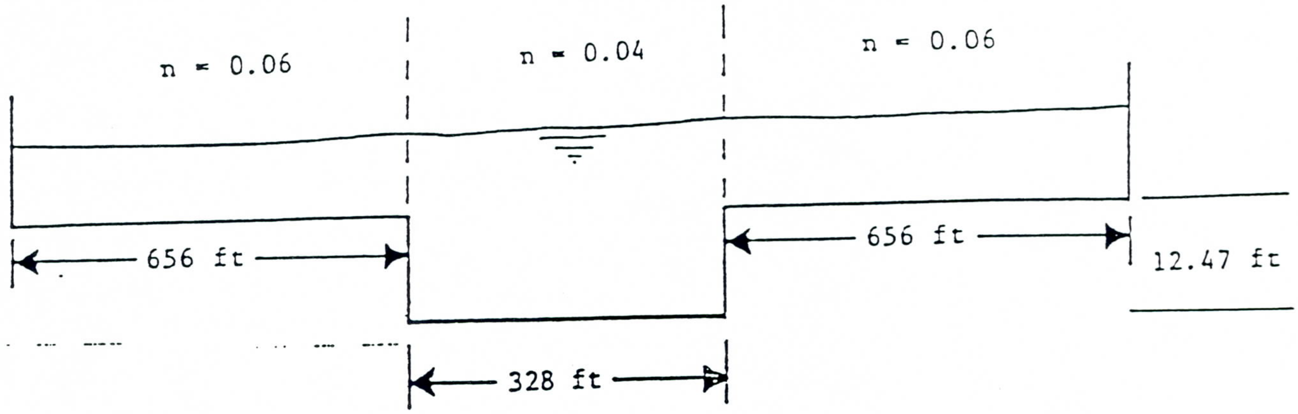


Figure 24. Compound cross section No. 3

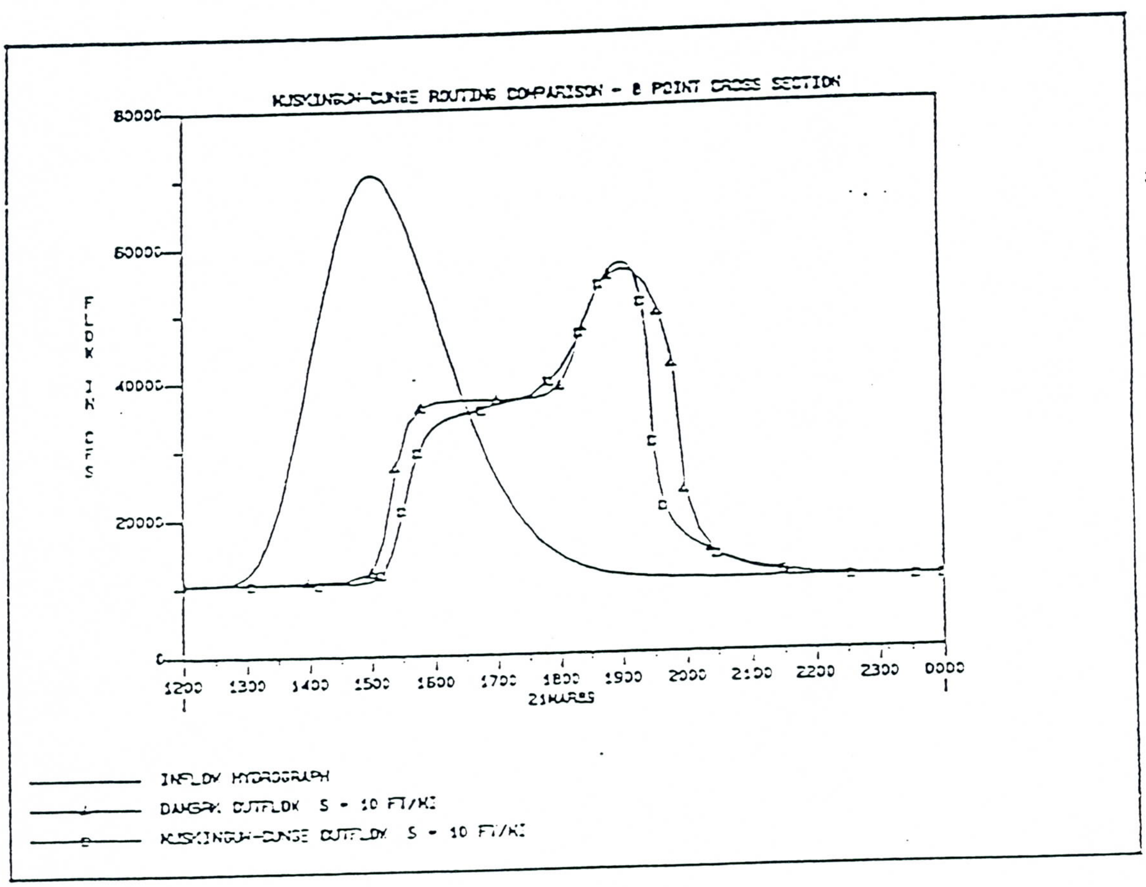


Figure 25. Resulting hydrographs from compound cross section No. 3

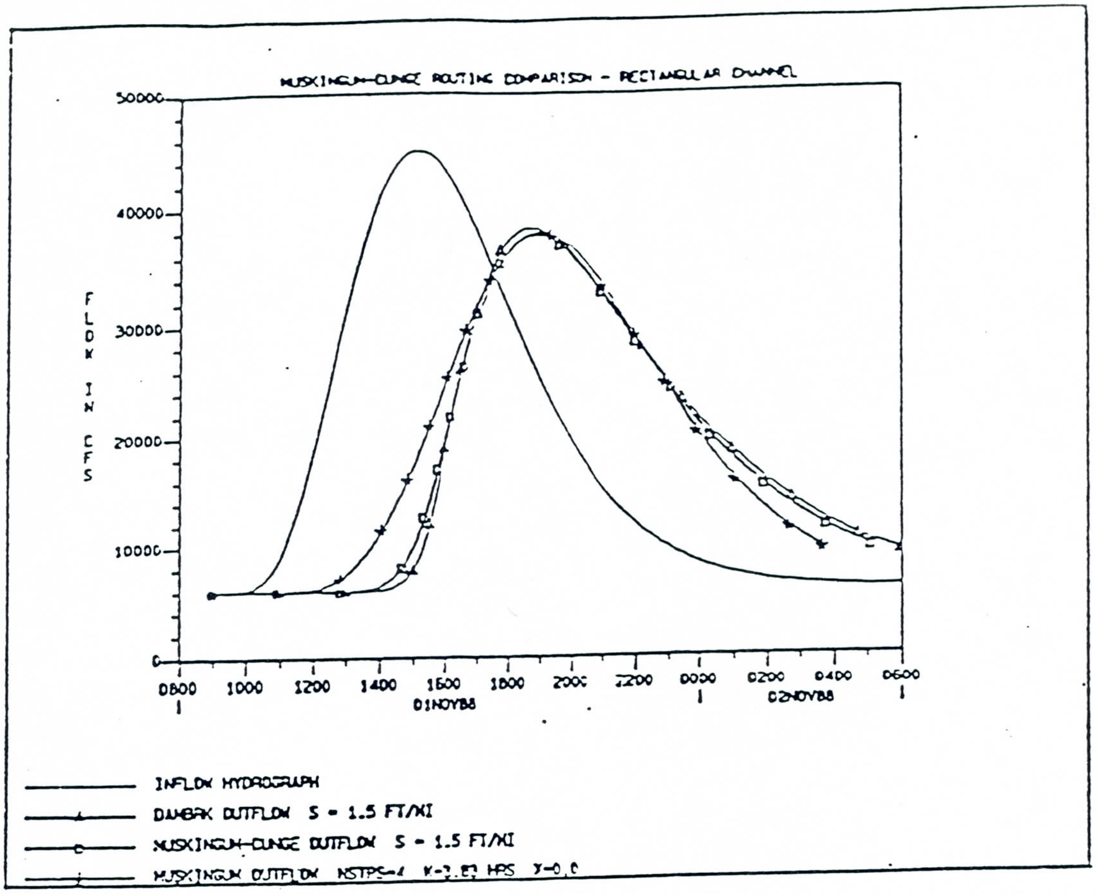


Figure 26. Comparison with traditional Muskingum method.

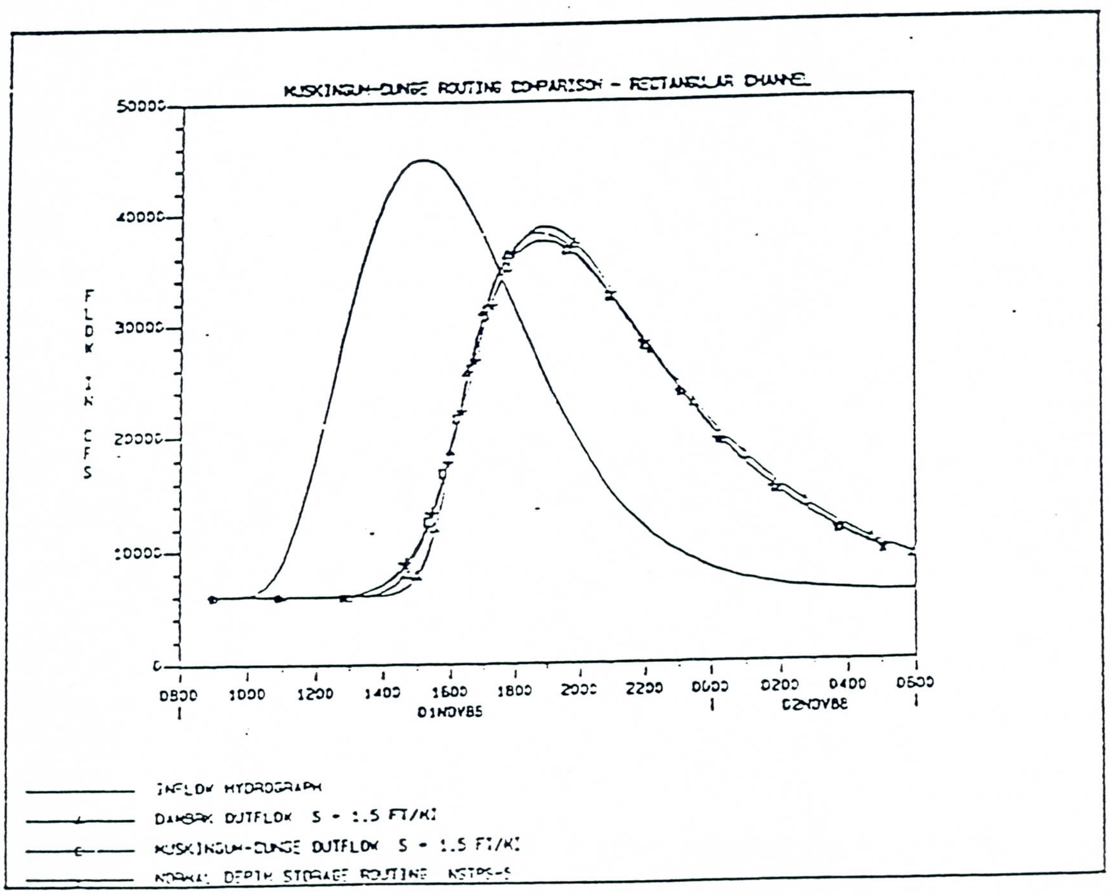


Figure 27. Comparison with Normal Depth storage routing.

VIII. REFERENCES

1. Fread, D.L., "The NWS DAMBRK model," Theoretical Background/User Documentation, Office of Hydrology, The National Weather Service, Silver Springs, Maryland, June 1988.

2. "HEC-1, Flood Hydrograph Package, User's Manual, The Hydrologic Engineering Center, U.S. Army Corps of Engineers, Davis, CA, September 1981, revised March 1987.

3. Miller, W. A. and J. A. Cunge, "Simplified Equations of Unsteady Flow," Unsteady Flow in Open Channels, Volume I, edited by K. Mahmood and V. Yevjevich, 1975, pp. 183-249.

4. Ponce, V. M., "Development of Physically Based Coefficients for the Diffusion Method of Flood Routing," Final report to the USDA, Soil Conservation Service, Lanham, Maryland, September 1983.

5. Ponce, V. M., "Diffusion Wave Modeling of Catchment Dynamics," Journal of Hydraulics Division, ASCE, Vol 112, No. 8, August 1986, pp. 716-727.

Comparisons of Flood Routing

COMPARISON OF DIFFERENT ROUTING METHODS

REQUIREMENTS AND RESULTS

I. Introduction to Unsteady Flow Models

A. Solve the complete St. Venant equations for one-dimensional free surface flow. These equations are generally considered appropriate governing equations for dam break floods (even instantaneous failures). If bores are present, special equations in addition to the St. Venant equations are necessary.

1. The equations are generally written:

Continuity: $$\frac{\partial Q}{\partial X} + B\frac{\partial h}{\partial T} - q = 0 \qquad (1)$$

and

Energy or

Momentum: $$q\frac{\partial h}{\partial X} + V\frac{\partial V}{\partial X} + \frac{\partial V}{\partial T} + gS_f + \frac{q}{A}V = 0 \qquad (2)$$

where:
- Q = Discharge
- X = Distance
- B = Surface width
- h = Surface elevation
- T = Time
- q = Lateral inflow or outflow
- g = Gravitational acceleration
- A = Cross sectional area
- S_f = Friction slope, given by

$$S_f = \frac{n^2 V|V|}{2.21 R^{4/3}}$$

where:
- n = Manning's n
- R = Hydraulic radius
- V = Velocity

2. The principal assumptions made in deriving the St. Venant equations are:

   a. the pressure distribution is hydrostatic

   b. the velocity at each section is uniform

   c. the water surface is horizontal across each cross-section

   d. the slope of the channel bottom is small

   e. n values for steady flow are applicable

B. Why use an unsteady flow model?

   1. Theoretical soundness lends credibility to the answers.

   2. Obtain complete hydrograph of both stage and discharge at all computation points in a single simulation.

   3. Avoid extrapolation of empirical coefficients (except Manning's) to outside range of calibration.

   4. The event and stream geometry may be such that none of the terms in the St. Venant equations can be ignored.

C. Why <u>not</u> use an unsteady flow model?

   1. Currently available, generalized, unsteady flow models are not "robust" enough to handle dam break floods reliably (i.e., without program aborts and stops).

   2. Substantial experience with, or support of, an unsteady flow model is required to guide the user when problems do arise.

   3. Computer costs may become significant, particularly if many runs are required to overcome difficulties.

D. Data requirements for unsteady flow models - similar to requirements for any procedure that routes a hydrograph and determines stages at multiple downstream points.

   1. Outflow hydrograph from the structure (= inflow hydrograph to study reach).

2. Geometric description of reach (cross-sections, distances between sections.

3. Roughness coefficients

4. Downstream boundary condition (usually a rating curve).

5. Tributary or local inflow hydrographs, if any.

II. Available Unsteady Flow Models

A. "Gradually Varied Unsteady Flow Profiles"[(1)]

1. A generalized unsteady flow model.

2. Can be applied to dam break flow analysis though was not developed for such.

3. Explicit solution - computational time step less than 1 minute for dam break flood simulation.

4. Geometric data input in HEC-2 format.

5. Supported by HEC.

6. Guidance for application of this program to dam-break floodwaves may be found in references (2) and (3).

B. National Weather Service (NWS) Dam Break Model[(4)]

1. A specialized unsteady flow model developed for analyzing dam break floods exclusively.

2. Specific features which are attractive for dam break modeling:

   a. User provides inflow hydrograph to reservoir - failure of structure begins at preselected pool elevation - breach develops in time (as specified by user).

   b. Can handle structures in tandem (domino effect).

   c. Can handle supercritical flow (user must determine where and if, however).

3. Implicit solution - computational time step several minutes for dam break flood solution.

4. Internal adjustment of time step.

5. Lack of experience, training, and support of this model within the Corps.

C. Hydraulics Module of "Water Quality for Rivers and Reservoirs (HEC Stream Hydraulics Package)"

1. A generalized unsteady flow program originally developed for water quality simulation.

2. User has choice of four routing methods:

   a. Muskingum

   b. Modified Puls

   c. Kinematic Wave

   d. St. Venant equations

3. Geometric data input in HEC-2 format

4. Implicit (finite element) solution, computational time step of several minutes for dam-break flood simulation using St. Venant equations.

5. HEC has recently incorporated the dam breach outflow hydrograph generator portion of the NWS model into this package.

6. Supported by HEC.

7. Documentation is being prepared.

8. Internal adjustment of time step.

9. Allows usage of nonuniform longitudinal (ΔX) element lengths.

III. When to Use Unsteady Flow Models

A. Strelkoff(5)

1. Used WES flume tests (6) to evaluate accuracy of various techniques.

2. Found that accuracy of simplified techniques can be related to a characteristic Froude number:

$$F_n^2 = \frac{Q^2}{gA^2y}$$

where: A, y, and Q are the area, depth and normal discharge at that depth, at a location just behind the dam prior to failure. Kinematic wave provided good solutions for $F_n > 1.6$ and poor solutions for $F_n < 0.3$.

3. The results were based on prismatic flume data. The Modified Puls technique with backwater-developed storage outflow relationships was not evaluated.

B. When using triangular hydrographs with Modified Puls routing (either normal depth or backwater developed storage-outflow functions), in some cases the water level immediately downstream of the dam associated with the computed peak discharge may be higher than the lake level. At some distance downstream, however, this initial error may not impact calculated water levels. Such conditions may indicate the need for an unsteady flow model.

C. Submergence of the breach by tailwater.

IV. Comparison of Dam-Break Flood Routing Procedures

A. Reconstitution of the Teton event (2) using an unsteady flow model (1) and Modified Puls (HEC-1) yielded comparable results.

B. Analysis of Oak Dam

1. Models used:

a. HEC-1

b. "Gradually Varied Unsteady Flow Profiles"(1)

c. HEC Stream Hydraulics Package

d. NWS(4)

A COMPARISON OF FLOOD ROUTING METHODS

METHOD	ADVANTAGES	DISADVANTAGES
a. Complete solution of basic equations of energy and continuity (St. Venant Eq.)	- A complete analysis of the hydraulics of flow and includes all energy components (potential, pressure, kinetic, inertial) plus continuity integrated in both time and space - Measures the impact of changes in flood plain storage directly in terms of the response of discharge and water surface elevation - Measures the impact of changes to the size or efficiency of conveyance channels directly in terms of response of discharge and water surface elevation - No coefficients required other than hydraulic roughness values	- Requires lots of computer time - Works from a detailed description of geometry (x-sections and reach lengths) and hydraulic roughness values in the channel and overbanks
b. Simplified versions of St. Venant Eq.		
c. Storage routing	- Measures changes in flood plain storage in terms of water discharge - Can be done by hand calculations - Faster than A	- Does not consider the energy equation directly but infers knowledge about it - Does not consider hydraulics of wave itself
Muskingum	- Purely analytical and does not require curves or table look-up in its solution	- Requires two empirical coefficients, one of which comes from reproducing known events - Requires a linear relationship between storage and discharge
Modified Puls	- Does not require a linear relationship between storage and discharge - Can relate storage to inflow and outflow	- Does not provide a coefficient for manipulating the impact of complicated hydraulics on the energy equation

METHOD	ADVANTAGES	DISADVANTAGES
Storage Routing (cont'd) Working R & D	- Has the advantage of two above. It is a Modified Puls technique that has a coefficient to better relate storage to energy of the flow	- Requires the determination of one empirical coefficient - Has limitations when storage varies with inflow and outflow
d. Averaging and Lagging	- Simple and fast	- Does not consider storage - Implies knowledge of both energy and storage - Implies knowledge of hydraulics of wave itself

Table 4b
DAMBRK - Teton
Peak Water Surface Elevations

Distance From Dam (Miles)	Maximum Water Surface Elevations in Feet				
	Manning N Increased		Breach Time		
	by 50%	by 100%	0.5 hr.	2.0 hr.	5.0 hr.
0	5119.5	5131.0	5106.7	5096.7	5083.3
2.75	5073.9	5082.2	5064.2	5056.0	5045.2
4.17	5044.2	5050.5	5037.2	5031.2	5022.8
6.63	4992.9	4995.2	4990.0	4988.9	4985.2
9.47	4948.1	4949.5	4946.0	4945.6	4943.4
13.26	4897.2	4898.1	4895.7	4895.5	4893.8
18.27	4850.2	4850.9	4848.9	4848.6	4847.1
25.57	4831.4	4832.0	4830.6	4830.4	4829.8
29.55	4822.9	4823.5	4822.1	4821.9	4821.4
35.98	4790.1	4790.6	4789.6	4789.4	4789.0
41.10	4779.5	4780.0	4778.9	4778.7	4778.3
48.86	4766.9	4767.4	4766.4	4766.0	4765.4
53.79	4746.1	4745.8	4745.5	4744.9	4743.9
58.71	4711.6	4712.1	4711.0	4710.3	4709.1
68.65	4628.6	4629.1	4628.2	4627.3	4625.6
79.17	4561.4	4562.7	4561.2	4560.2	4559.7
84.09	4530.4	4531.5	4529.5	4529.2	4529.2
89.02	4500.5	4502.0	4497.9	4497.8	4497.8
94.32	4462.7	4463.7	4461.4	4461.4	4461.4
98.48	4435.0	4436.0	4433.8	4433.8	4433.8
101.89	4421.2	4422.5	4419.5	4419.5	4419.5

Table 5b
DAMBRK - Teton
Maximum Flood Depths

Distance From Dam (Miles)	Maximum Depth in Feet				
	Manning N Increased		Breach Time		
	by 50%	by 100%	0.5 hr.	2.0 hr.	5.0 hr.
0	89.5	101.0	76.7	66.7	53.3
2.75	68.9	77.2	59.2	51.0	40.2
4.17	59.2	65.5	52.2	46.2	37.8
6.63	29.9	32.2	27.0	25.9	22.2
9.47	23.1	24.5	21.0	20.6	18.4
13.26	17.2	18.1	15.7	15.5	13.8
18.27	25.2	25.9	23.9	23.6	22.1
25.57	23.4	24.0	22.6	24.0	21.8
29.55	20.9	21.5	20.1	19.9	19.4
35.98	17.1	17.6	16.6	16.4	16.0
41.10	17.5	18.0	16.9	16.7	16.3
48.86	16.9	17.3	16.4	16.0	15.4
53.79	28.1	27.8	27.5	26.9	25.9
58.71	28.6	29.1	28.0	27.3	26.1
68.65	28.6	29.1	28.2	27.3	25.6
79.17	9.4	10.7	9.2	8.2	7.7
84.09	8.4	9.5	7.5	7.3	7.2
89.02	10.5	12.0	7.9	7.8	7.8
94.32	7.7	8.7	6.4	6.4	6.4
98.48	10.0	11.0	8.8	8.8	8.8
101.89	6.2	7.5	4.5	4.5	4.5

Table 6
DAMBRK - Teton
Time to Crest Elevation

Distance From Dam (Miles)	Time to Maximum Elevation in Hours								
					Manning N Increased		Breach Time		
	Measured	Base Run	Without Volume Losses	Without Inactive Areas	By 50%	By 100%	0.5 hr.	2.0 hr.	5.0 hr.
0.00	---	1.00	1.00	1.00	1.00	1.05	0.52	2.00	5.00
2.54	2	1.10	1.10	1.10	1.15	1.15	0.60	2.00	5.00
8.86	2.5	2.65	2.65	1.60	3.25	3.75	2.30	3.50	6.25
20.43	---	6.13	6.13	4.53	8.25	10.33	5.76	6.90	9.25
30.09	--	10.25	10.25	7.73	14.30	18.02	9.90	11.10	13.25
53.79	31	24.98	24.66	18.85	35.22	45.58	24.61	25.69	28.25
67.54	36	27.72	27.55	21.76	39.54	50.78	27.27	28.50	31.26

Table 3
DAMBRK - Teton
Peak Discharges

Distance From Dam (Miles)	Maximum Discharges in 1,000 cfs								
	Measured	Base Run	Without Volume Losses	Without Inactive Areas	Manning N Increased		Breach Time		
					By 50%	By 100%	0.5 hr.	2.0 hr.	5.0 hr.
0.00	---	2,004	2,004	2,006	2,004	2,004	2,231	1,428	830
2.54	2,300	1,890	1,894	1,892	1,847	1,795	2,031	1,404	798
8.86	1,060	1,011	1,020	1,587	887	793	1,023	956	692
20.43	---	696	713	827	540	439	698	678	561
30.09	---	344	359	412	258	210	346	337	313
53.79	90.5	168	203	213	127	105	171	156	131
67.54	67.3	117	200	192	90.9	76.9	123	97	64

Notes:

1. Time to maximum breach size is 1.0 hour for base run.

2. Runs with time to maximum breach size of 0.05 hour and 0.20 hour resulted in nonconvergence.

3. Runs with breach bottom widths increased from 50 feet to 500 feet and 300 feet resulted in nonconvergence.

4. Runs decreasing Manning N by 50% and 205 resulted in nonconvergence.

5. Cross-section at mile 6.63 was removed to achieve convergence for the without inactive areas case.

Reservoir Routing

RESERVOIR ROUTING BY THE

STORAGE INDICATION (MODIFIED-PULS) METHOD

BASIC EQUATIONS

- The storage indication method consists of the repetitive solution of the continuity equation and is based on the assumption that the reservoir water surface remains horizontal and that outflow from the reservoir is a unique function of storage.

The continuity equation may be expressed as:

$$\bar{I} - \bar{O} = \frac{\Delta S}{\Delta t} \qquad (1)$$

where

$\bar{I}$ = mean inflow into reservoir during routing period Δt,

$\bar{O}$ = mean outflow from reservoir during routing period Δt,

ΔS = change in reservoir storage during routing period Δt.

Equation (1) may be approximated by:

$$\frac{(I_1 + I_2)}{2} - \frac{(O_1 + O_2)}{2} = \frac{(S_2 - S_1)}{\Delta t} \qquad (2)$$

where subscripts 1 and 2 denote the beginning and end, respectively, of a routing period, Δt.

The assumption implicit in Equation (2) is that discharge varies linearly with time during a routing period Δt. This assumption must be borne in mind when selecting a routing period.

Equation (2) may be restated as follows:

$$(I_1 + I_2) + \left[\frac{2S_1}{\Delta t} - O_1\right] = \left[\frac{2S_2}{\Delta t} + O_2\right] \tag{3}$$

In Equation (3), all terms on the left-hand side are known from preceding routing computations. The terms on the right-hand side involving S_2 and O_2 are unknown and must be determined by storage routing.

ROUTING PROCEDURE

- given hydrograph of pre-development conditions from which the maximum basin outflow is determined
- given the post-development hydrograph which is to be routed through the proposed detention basin
- assume a size and shape for the first trial basin and outlet works
- compute a table and/or curve of water depth versus storage (a function of basin geometry), water depth is measured above the spillway or outflow pipe invert.
- compute a table and/or curve of water depth versus outflow (stage discharge relationships are a function of the outlet structure)
- select a routing period Δt such that there are five or six points on the rising side of the inflow hydrograph, one of which coincides with the inflow peak
- construct a graph of $\left(\frac{2S}{\Delta t} + O\right)$ versus O

  S = storage volume

  Δt = routing period

  O = outflow rate

- the routing procedure is now accomplished, using a tabular method for the solution

  Modified-Puls equation:

$$(I_1 + I_2) + \left[\frac{2S_1}{\Delta t} - O_1\right] = \left[\frac{2S_2}{\Delta t} + O_2\right]$$

- compare the maximum outflow rate with the allowable rate of discharge from the drainage area
- adjust size, shape, and/or outlet structure if the maximum outflow rate is greater than the allowable
- repeat the design procedure for alternative design solutions

EXAMPLE - RESERVOIR ROUTING BY STORAGE INDICATION METHOD

(1) Given the pre- and post-development hydrographs - Figure 1. The problem is to size the detention basin and outlet works to reduce the development flood peak (249 cfs) back to the pre-development level of 100 cfs.

(2) Given the storage-elevation curve for the proposed site of the detention facility - Figure 2. The outlet invert is assumed to be at elevation 1060.0 feet.

(3) Given the hydraulic performance curve for a 48-inch C.M.P. culvert with projecting inlet flowing under inlet control - Figure 3. This is an assumed size and flow condition.

(4) Select the routing interval Δt such that there are five or six points on the rising limb of the inflow hydrograph, one of which coincides with the inflow peak. From Figure 1, $\Delta t = 10$ min.

(5) Construct a graph of $\left(\frac{2S}{\Delta t} + O\right)$ versus O using Figures 2 and 3 and $\Delta t = 10$ min. - Figure 4.

(6) Organize a routing table to solve Equation (3), using Figures 1 and 4 - Table 1.

(7) Compare the maximum outflow rate, 128 cfs, with the allowable rate, 100 cfs. Since this design results in maximum outflow rate greater than the pre-development flow rate, an alternative design must be investigated, i.e., modify the outlet structure.

(8) Repeat the design procedure for alternative designs.

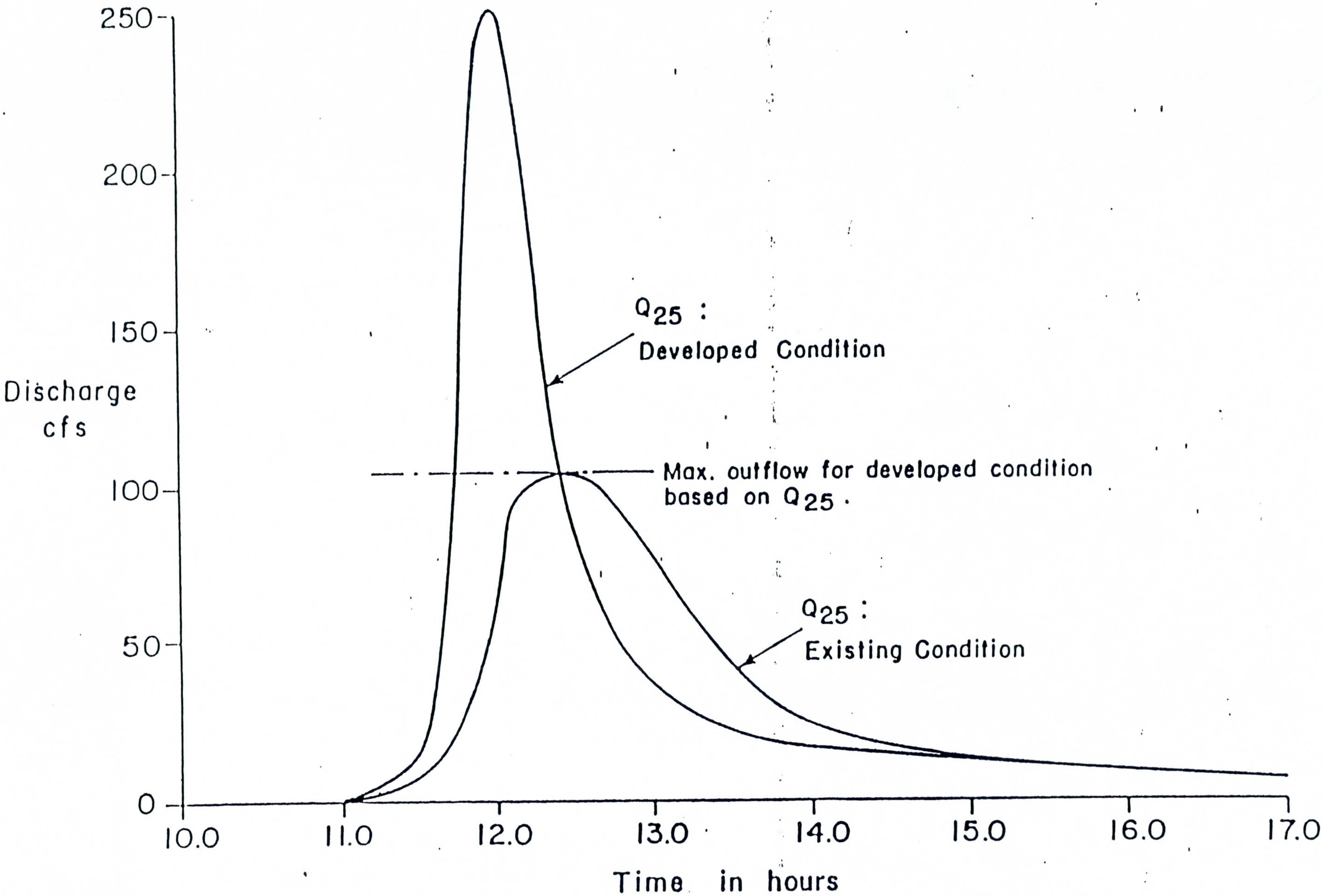


Figure 1. Comparison of outflow hydrographs by SCS Tabular Method for existing and developed conditions.

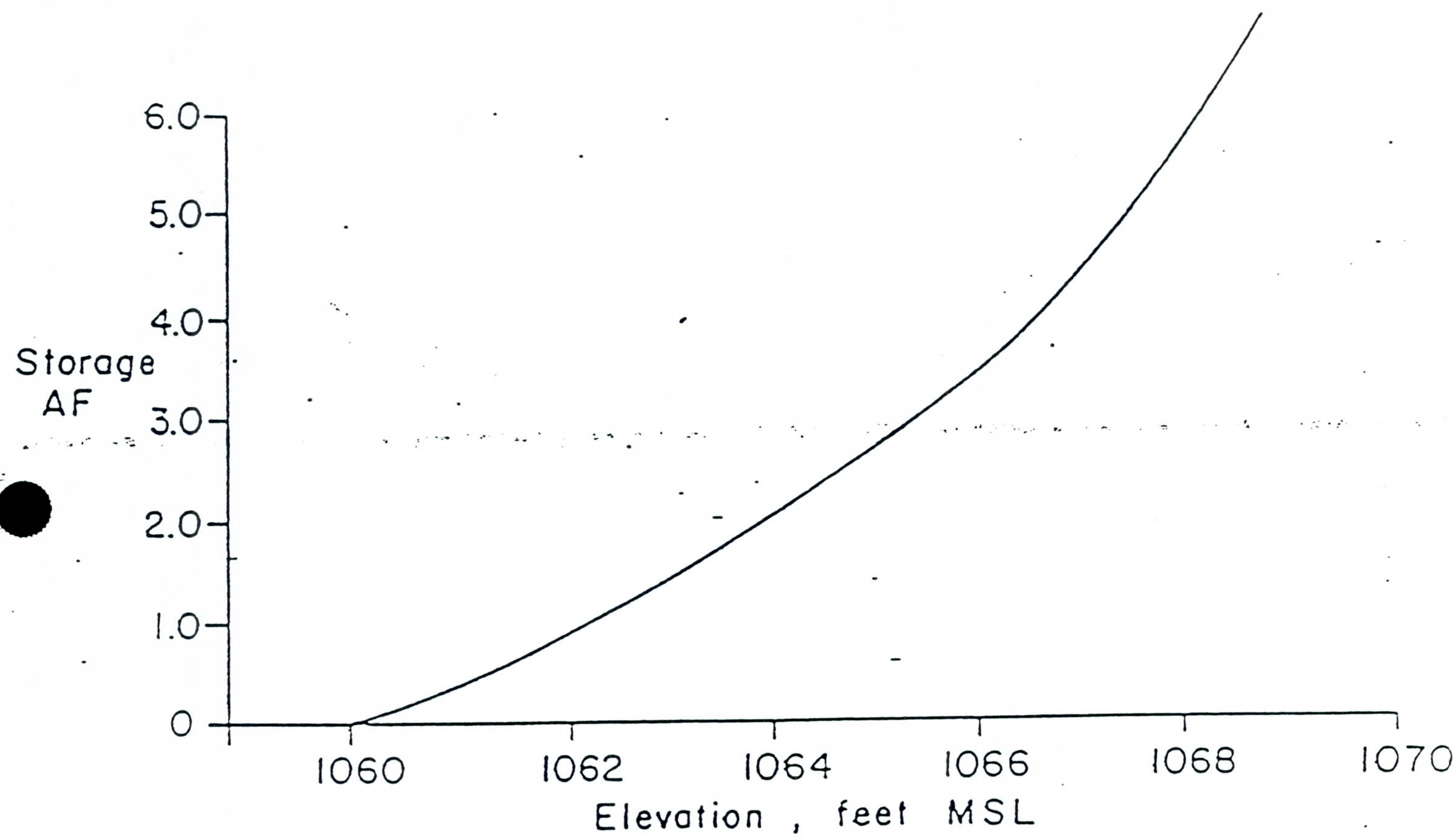


Figure 2. Storage-evaluation curve for proposed site.

HYDRAULIC PERFORMANCE CURVES
FOR
48-INCH C.M. PIPE CULVERT
WITH
PROJECTING INLET

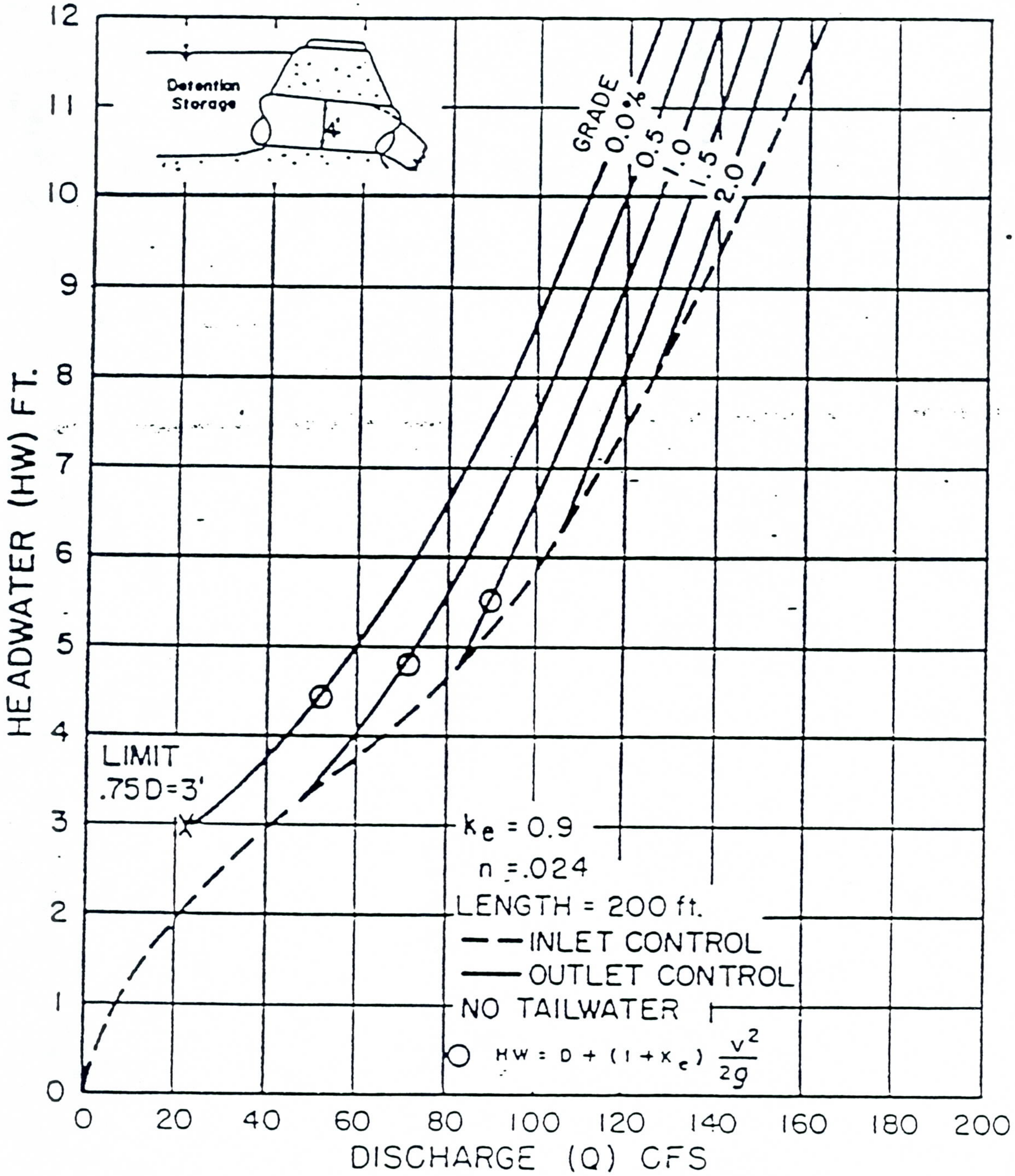


Figure 3

Fig. 4. Develop $\frac{2S}{\Delta t} + O$ vs. O for 48-inch CM pipe culvert with Δt = 10 minutes.

Elev	Storage, AF	Storage, cfs.hrs	O cfs	$\frac{2S}{\Delta t} + O$
1060	0	0	0	0
1061	0.40	4.8	6	63
1062	1.00	12.0	21	165
1063	1.50	18.0	40	256
1064	2.00	24.0	65	353
1065	2.60	31.2	87	461
1066	3.40	40.8	103	593
1067	4.20	50.4	115	720
1068	5.40	64.8	125	903
1069	7.20	86.4	135	1172
1070	10.00	120.0	147	1587

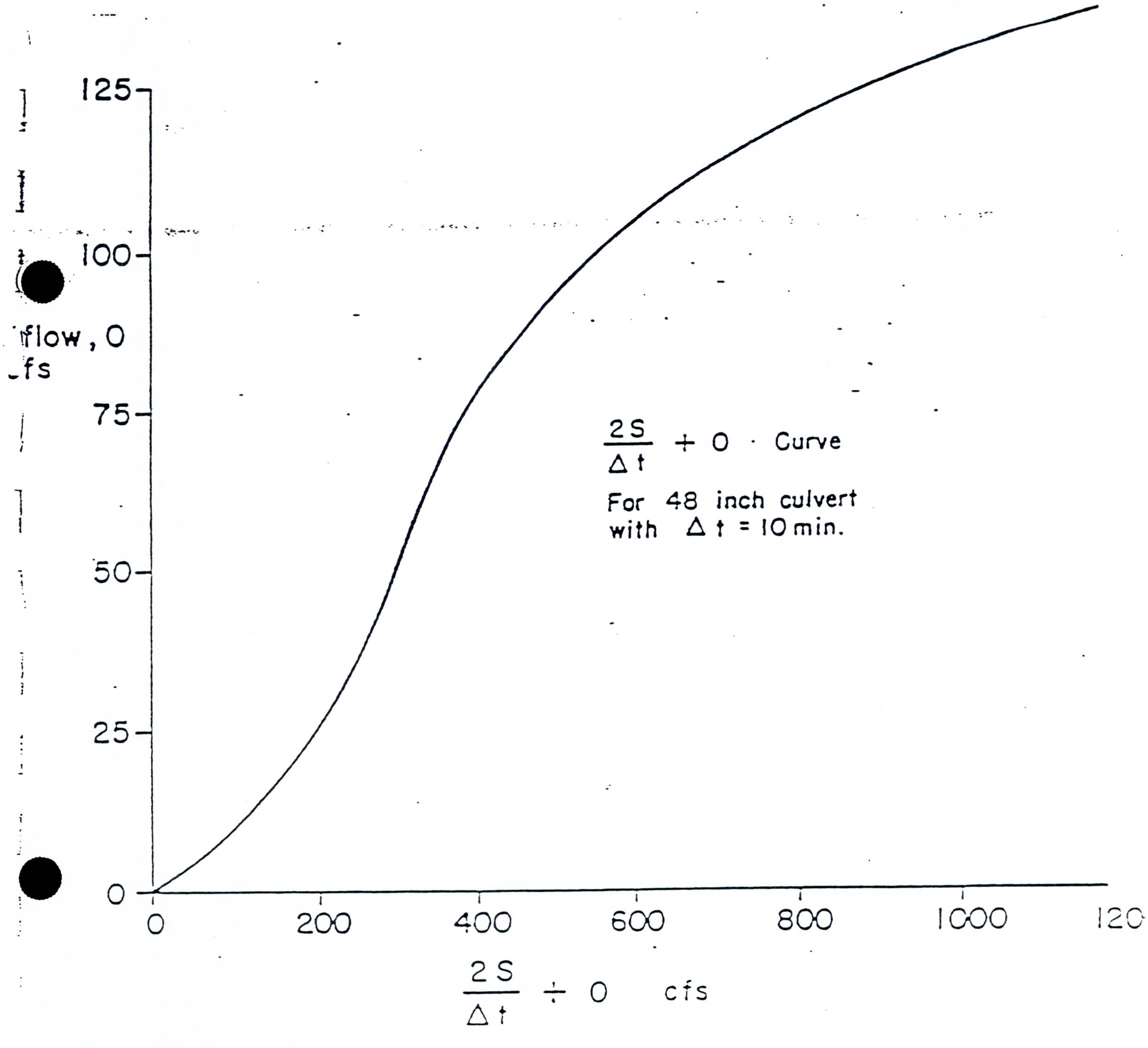


Table 1. Routing Table and Detention Storage Analysis.

(1) Time Hrs:Min	(2) I_1 (cfs)	(3) $I_1 + I_2$ (cfs)	(4) $\frac{2S}{\Delta t} - O$ (cfs)	(5) $\frac{2S}{\Delta t} + O$ (cfs)	(6) O_2 (cfs)
11:00	0	5	0	0	0
11:10	5	15	3	5	1
11:20	10	26	14	18	2
11:30	16	66	34	40	3
11:40	50	230	82	100	9
11:50	180	429	208	312	52
12:00	249	429	425	637	106
12:10	180	320	608	854	123
12:20	140	222	672	928	128
12:30	82	142	644	894	125
12:40	60	105	550	786	118
12:50	45	79	439	655	108
13:00	34	64	340	518	89
13:10	30	57	252	404	76
13:20	27	50	203	309	53
13:30	23	44	173	253	40
13:40	21	40	161	217	28
13:50	19	37	149	201	26
14:00	18	36	140	186	23
14:10	18	35	132	176	22
14:20	17	34	125	167	21
14:30	17	33	121	159	19
14:40	16	32	120	154	17
14:50	16	31	120	152	16
15:00	15	30	119	151	16

NOTES:

(1) Max outflow of 128 exceeds max allowable of ~ 100 cfs for pre-development Q_{25}. Requires modified outlet structure.

(2) Max storage occurs at max outflow at time 12:20. Since $2S/\Delta t + O$ = 928 cfs at this time, S = 66.80 cfs.hrs = 5.56 AF. From storage-elevation curve, this produces a max depth of 8.4 feet at elevation 1068.4.

WORKSHOP I SOLUTION

DETENTION BASIN ANALYSIS FOR 36-INCH OUTLET PIPE

Step (1): Develop $2S/\Delta t + O$ vs. O table for $\Delta t = 10$ minutes using 36-inch culvert performance curve.

ELEV	STORAGE, AF	STORAGE, cfs.hrs	O cfs	$\frac{2S}{\Delta t} + O$
1060	0	0	0	0
1061	0.40	4.8	10	68
1062	1.00	12.0	21	165
1063	1.50	18.0	35	251
1064	2.00	24.0	51	339
1065	2.60	31.2	64	438
1066	3.40	40.8	73	563
1067	4.20	50.4	82	687
1068	5.40	64.8	88	866
1069	7.20	86.4	94	1131
1070	10.00	120.0	100	1540

Step (2): Routing Table and Detention Storage Analysis for 36-inch Culvert Pipe

(1) Time Hrs:Min	(2) I_1 (cfs)	(3) $I_1 + I_2$ (cfs)	(4) $\frac{2S}{\Delta t} - O$ (cfs)	(5) $\frac{2S}{\Delta t} + O$ (cfs)	(6) O_2 (cfs)
11:00	0	5	0	0	0
11:10	5	15	3	5	1
11:20	10	26	14	18	2
11:30	16	66	34	40	3
11:40	50	230	82	100	9
11:50	180	429	214	312	49
12:00	249	429	487	643	78
12:10	180	320	738	916	89
12:20	140	222	874	1058	92
12:30	82	142	910	1096	93
12:40	60	105	868	1052	92
12:50	45	79	793	973	90
13:00	34	64	696	872	88
13:10	30	57	590	760	85
13:20	27	50	491	647	78
13:30	23	44	395	541	73
13:40	21	40			
13:50	19	37			
14:00	18	36			
14:10	18	35			
14:20	17	34			
14:30	17	33			
14:40	16	32			
14:50	16	31			
15:00	15	30			

NOTES: (1) Max storage occurs at max outflow at time 12:30. Since $2S/\Delta t + O$ = 1096 cfs at this time, S = 83.6 cfs.hrs = 6.97 AF. From storage-elevation curve, this produces a max depth of 8.9 feet at elevation 1068.9.

(2) From TR.55 (Chapter 7), we can also estimate the storage required to reduce outflow peak to 93 cfs:

$V_r = \Sigma$ inflows = 245 cfs.hrs = 20.39 AF

Q_i = 249 cfs; Q_o = 93 cfs; $Q_o/D{=}Q_i$ = 0.37

V_s/V_r = 0.38 from Fig. 7-2

V_s = 0.38 x 20.39 = 7.75 AF

PROBABLE MAXIMUM FLOOD DEVELOPMENT

PMF GUIDELINES SECTION 8-10

Tuesday 1:00 p.m.

8-10 Probable Maximum Flood Development

Sections 8-7, 8-8, and 8-9 described the process of developing the necessary runoff model for use in computing the inflow PMF hydrograph. For simple basins, this runoff model will consist of a single representative unit hydrograph. For more complex basins, the runoff model will consist of a combination of unit hydrographs for subbasins and a streamflow-routing process. The runoff model is used to calculate the inflow PMF hydrograph. This section provides guidelines for calculating the PMF including parameters related to the PMP, basin losses, antecedent hydrologic conditions, snowmelt, base flow, and channel routing. In addition, guidelines for sensitivity analysis of the calculated inflow PMF are provided in Section 8-10.7.

8-10.1 Spatial Distribution and Disaggregation of the Probable Maximum Precipitation

To compute the inflow PMF, it is necessary to arrange both a temporal and spatial distribution of the PMP on the project basin.

8-10.1.1 Storm Duration

A primary assumption on which this chapter is based is that complete depth-duration information is available for the PMP for both general and local storms, so that the necessary design storms can be constructed. A local storm is one with a relatively small area of influence such as a thunderstorm. General storms can be assumed to cover over 1,000 square miles at any particular instant. In general, local storms will be of short duration and high intensity and, hence, may produce larger rates of peak runoff and smaller total runoff volumes than general storms. However, for some combinations of reservoir volume and spillway capacity, the inflow PMF produced by a long-duration general storm, when routed through the reservoir, will result in higher reservoir levels and may produce the largest rate of outflow. Thus, it is necessary to develop inflow hydrographs for both general and local seasonal PMPs to establish the PMF event.

8-10.1.2 Storm Spatial Distribution

Basin-average rainfall must be developed for the PMF. This will require establishment of a spatial distribution for PMP within the basin. Rainfall data are seldom available from a large enough number of rain gages to allow construction of an accurate isohyetal map for each historic storm. If a historic storm has been studied by the COE, USBR, or NWS, isohyetal maps may have been developed from rainfall depth information obtained during "bucket surveys." If isohyetal maps are available for any of the historic extreme storms that have occurred in the area, or if they can be constructed from data available, they could be used in defining the spatial distribution of storm rainfall for the PMP.

However, individual storm distribution may be biased because of a singular feature of the storm. For that reason, this chapter recommends that the elliptical isohyetal map produced by the NWS in Hydrometeorological Report No. 52 should be used [NWS 1982] in the region east of the 105th meridian. For other areas, refer to the appropriate HMR or site specific study. The isohyetal pattern covers areas from 10 to 60,000 square miles. Orientation of the isohyetal map should be with its major axis parallel to the direction of moisture flow, but rotations up to 40° are permitted without reduction of PMP depths.

The storm pattern on the basin should be adjusted so that the maximum rainfall volume falls on the drainage area. In general, this will require that the area of greatest rainfall depth be approximately centered on the basin and that the storm pattern be rotated (within the 40° limits) so that the basin is covered to the greatest extent possible by the isohyets of greatest rainfall depth. If, however, the basin is subdivided, the peak runoff rate might be produced by different centering. A sensitivity analysis is required.

A computer program developed by the Hydrologic Engineering Center of the COE can be used to apply the procedures contained in HMR 52 [COE 1984]; it is available through some private software vendors. In Wisconsin and Michigan, the computer program WMPMS is available through Electric Power Research Institute (EPRI). These programs automatically produce a 72-hour storm. However, the storm totals are balanced so that lesser durations are also PMP values for the storm size.

For locations where the areal distribution of the storm cannot be generalized as readily due to orographic influences or unique storm patterns, such as in the western states, dependence must be placed on the patterns produced by the historic storm and annual rainfall depths in the region. If insufficient data exist to provide for development of an isohyetal pattern, a uniform distribution over the basin may be assumed. The method used by the USBR, known as successive substraction, can be used to advantage [Cudworth 1989].

8-10.1.3 Temporal Distribution of the Probable Maximum Precipitation

The depth–duration relationship for the PMP should be taken from the envelope–curve included in the PMP data. Time distribution of severe rainfall has been shown to follow no particular pattern. In general, if the peak period of rainfall is placed at the beginning of the storm, the peak rate of runoff will be minimized because the largest rates of infiltration and initial abstraction will act to reduce the peak rate of rainfall. If, however, the peak period of rainfall is placed at the end of the storm, the peak rate of runoff will be maximized. For this chapter, it is recommended that the peak 6–hour period of rainfall be placed between the half and two–thirds point of the storm and that the remaining 6–hour increments be arranged in alternating descending order on each side of the peak, beginning with the time period that precedes the peak 6–hour period. Hourly increments of rainfall should also be taken from the PMP envelope curve and distributed so as to provide a smooth temporal curve. Reference should be made to the appropriate HMR or site specific study.

8-10.2 Coincident Snowmelt Conditions

For basins and seasons where the PMF will have a snowmelt contribution, it is necessary to adopt temperature and snowpack criteria for use in developing the PMF. The following steps should be followed:

- Identify the area that may be covered by snowpack at the time the PMP begins by considering the data on historic snowpack coverage obtained in Section 8-4.
- Assume a 100–year snowpack water equivalent and snowpack areal distribution.
- Develop the coincident temperature sequence and temperature–elevation distribution from data analyzed in Section 8-5. In California and the Northwestern states, the temperature sequence coincident with PMP can be found in NWS HMR Nos. 36 and 43, respectively. For other areas, the maximum temperature sequence observed in the area for the season of the critical PMP is recommended.
- In areas east of the 103rd Meridian, seasonal PMP values can be obtained from HMR 33 where an updated site-specific study of seasonal PMP values is not available.

8-10.2.1 Snowmelt Estimates

Three items of data are required as follows:

- Temperature sequence
- Depth of snow on the ground
- Water-equivalent of the snow on the ground

Each of the above parameters is season-dependent. The temperature sequence is selected from historic temperature sequence data, with the qualification that the sequence was associated with simultaneous occurrence of rainfall and snow on the ground. The maximum temperature sequence is obtained by comparing average daily temperatures above 34°F during periods of rainfall. This temperature sequence is assumed to optimally coincide with the probable maximum storm (PMS).

Establish combinations of temperature sequence, snowpack depth, and rainfall intensity for time periods under consideration (e.g., monthly).

Determine for each time period (e.g., monthly) the availability of snowpack depth data from climatological data stations. Snowpack water equivalent is generally not recorded, but data should be compiled and used, if available. If data is not available for seasonally appropriate water equivalent values, regional references may be used with water equivalent values doubled to provide a conservative estimate [Gray and Prowse 1992].

The degree-day method is then used to develop the snowmelt-runoff component. Where climatological stations are located in the basin, temperature data records will usually be available. Snowpack depths may not be as readily available. In that case, assume an unlimited snowpack and melt as much depth as the temperature sequence will allow, then convert to water equivalent using references as discussed above.

Absent temperature sequence data and snowpack depths, in non-mountainous regions, seasonal, 100-year 3-day flood peak discharge may be used in lieu of the snowmelt component. This uniform flow should be added in with normal base flow covering the entire time base of the hydrograph. Combine this value with seasonal rain on seasonal, frost-conditioned soils.

8-10.3 Loss Rates for Subbasins

It will be necessary to assume a saturated infiltration rate be used in the PMF computation. The infiltration rate should be assumed in accordance with recognizable characteristics of the drainage area. The initial abstraction obtained from analysis of historical floods can be used; however, in most cases, this is not a significant parameter in developing the PMF. If the SCS loss function is used, Antecedent Moisture Condition (AMC) II must be assumed when establishing the runoff curve number.

Caution: Use of nonsaturated infiltration rates may be appropriate in arid and semi-arid regions, but must be justified.

8-10.3.1 Approximate Method

For PMF runoff computations, the soil should be assumed to be saturated with infiltration occurring at the minimum rate applicable to the average soil type covering each subbasin. Soil data for the basin should be examined, and the major soil classifications in the basin should be delineated on the drainage area. An average soil classification should be established for each subbasin that can be identified with an SCS Hydrologic Soil Classification (A, B, C, or D). Minimum infiltration rates for the average hydrologic soil classification should be selected from the information provided in the 1955 Yearbook of Agriculture [USDA 1955]. Table 8-10.1 provides the general soil characteristics and minimum infiltration rates taken from the USDA reference. The value of uniform infiltration calculated by HEC-1 for the historical floods will be a guide to assessing the suitability of the chosen infiltration rates.

Caution: Application of this approach can lead to overly conservative results when soils in the area in question have permeabilities in excess of the values listed in Table 8-10.1. For comparison on specific soil, check SCS National Engineering Handbook (NEH-4) Chapter 7, page 7.7, 1985, and specific soil descriptions.

Caution: Infiltration rates, as determined by use of HEC-1 in analyzing historic floods, can only be used as a guide since they can be quite variable depending upon the rainfall intensity and the accuracy with which other input to HEC-1 (particularly rainfall distribution) is known. In addition, antecedent conditions will be different prior to the PMF than for historic storms.

8-10.3.2 Detailed Method

The SCS STATSGO database can be used to give a more detailed estimate of the infiltration rate for a basin or subbasin.[2] This procedure is of particular use when soil types and their associated infiltration rates vary widely within the basin. For each soil series in the SCS STATSGO database, the geometric mean permeability of the limiting (least permeable) soil layer should be used as the representative infiltration rate. The following steps provide a means to estimate excess precipitation while taking into account the variation of infiltration within the basin:

(1) Calculate PMP rainfall in hourly increments.

(2) Use a basin (subbasin) delineation to identify the area, the STATSGO database to determine the percentage of the basin covered by each soil association identified within the basin, and Land Use and Land Cover maps to identify forested and wetland areas.

(3) Using the STATSGO database to determine, for each soil association, the soil series percentage composition of each soil unit.

(4) Use the STATSGO database to identify the soil profile layer in each soil series with the minimum geometric mean value (i.e., the limiting layer), and use that layer's range geometric mean permeability to represent that soil series' infiltration rate.

(5) Use the results of steps (2), (3), and (4) to calculate the total area of the basin represented by each limiting geometric mean permeability, and formulate values of percent of total basin area with limiting geometric mean permeability values.

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2 See Appendix 8-C for a detailed explanation of applying STATSGO data to determine infiltration rates.

(6) For each hour of the PMP, calculate the depth of excess rainfall for each limiting geometric mean infiltration rate category separately, multiply by the appropriate percentage of basin area, and sum the volumes by hour—thus calculating basin runoff for each storm hour from each soil series' limiting layer infiltration rate.

(7) Use the results of step (6) as the rainfall input, and set the loss function to zero in HEC-1.

Caution: This method assumes that the overlying layer of soils controls the rate at which infiltration takes place in terms of soil permeability. Cases exist for which this may not be true, such as for areas underlain with shallow, impermeable bedrock or areas having a groundwater table very near the surface.

8-10.3.3 Infiltration Characteristics of Potentially Frozen Soils

It is well understood that the structure type of soil frost has a strong influence on the rate of infiltration of soil [Trimble, et al. 1987]. Because of different vegetation cover and surface soil characteristics, soils will respond differently to freezing, producing different types of soil frost structures. These structures are most commonly classified as either concrete or granular frost. Soils with concrete frost are identified by dense thin ice lenses and ice crystals. Soils with concrete frost allow very little infiltration. Granular frost, typically found in woodland soils, consists of small frost particles intermingled with soil particles. Typically, soils classified as having granular frost have higher infiltration rates than the same soil unfrozen [Blackburn and Wood 1990].

Frost structures are related to the moisture content of the frozen soil [Post and Dreibelbis 1942]. Soils frozen at low moisture content may become granulated and provide little impediment to infiltration. Conversely, soils frozen at high moisture contents often freeze into massive, dense, concrete-like structures that are nearly impermeable to water [Zuzel and Pikul 1987].

Reduced levels of moisture content are found in forested areas because of interception and evapotranspiration [Kane and Stein 1983]. These low moisture contents result in granular frost structures in the winter.

Many researchers have identified the effects of soil freezing on the infiltration capability of soils. Type of frost, soil structure, and antecedent soil moisture content have all been noted as factors influencing frozen soil infiltration.

In Engelmark's set of laboratory experiments [Engelmark 1987], infiltration rates were measured in a fine sand. The grain-size curve of the fine sand indicated 84 percent passing a #40 sieve and 5 percent passing a #200 sieve.

Infiltration rates obtained for this soil in the frozen state were between 1-2 mm/min. (2.4-4.7 in/hr).

Another experiment executed by Blackburn and Wood [Blackburn and Wood 1990] provided a range of infiltration rates of 0.42-1.08 mm/min (1-2.4 in/hr), depending on the type of frost that existed. This experiment was performed on a sandy soil of the Larimer series.

When the soil type is combined with the vegetation, a low soil moisture content can be predicted. Even during the PMP, the rainfall rate may not exceed the rate of infiltration in soils and they will not be saturated. With these conditions, a granular soil frost will predominate in the winter. Granular soil frost is far from impervious; it typically has infiltration rates the same as, or higher than, the soil in an unfrozen condition [Blackburn and Wood 1990].

- Wetlands should be modeled as impervious elements. These soils, although sandy, may intersect the seasonal high water table and thus have a higher potential to produce a concrete type of frost.

- Infiltration rates for granular soils, such as sand and sandy loam, should be assumed equal to the unfrozen condition.

- Soils with high silt content associated with high groundwater tables should be assumed to be impervious.

- Clays should also be assumed to be impervious.

- Forested soils or soils with a minimum 4-inch humus depth should have unfrozen condition infiltration rates applied [Kane and Stein 1983].

- Nonforested soils, other than sands or sandy loams, should be considered impervious when they occur within the historical maximum frost depth.

8-10.4 Reservoir and Channel-Routing Approach

This section provides guidance for routing the flood hydrographs from subbasins to the dam site. This routing will generally be through natural channels, but it may also involve routing inflow hydrographs through upstream reservoirs. The following procedures should be used:

- Assume a level pool when routing the flood hydrograph through any upstream reservoirs. (Use dynamic routing, if appropriate.)

- Use the Muskingum–Cunge method, as incorporated in HEC–1 to perform any channel routing from subbasins to the basin outlet. Cross sections of the channels, along with Manning's roughness coefficients, will be required to use the Muskingum-Cunge routing method. For most cases, cross sections sufficiently accurate for routing the PMF can be obtained from 7½-minute USGS quadrangle maps. HEC-1 has the capability to compute and combine hydrographs from side areas with the routed channel hydrographs.

Caution: Muskingum-Cunge uses a single (representative) cross section defined by eight coordinate points for each routing reach. The method cannot accommodate for backwater effects and should not be used when attenuation of the hydrograph is expected. An example of where this technique might be used is when translating a hydrograph from an upstream location to a downstream point where off-channel storage is insignificant. Where the intention is to properly model the attenuation of the hydrograph, the dynamic wave routing is the preferred method (e.g., when the river is expanding or contracting or where there is natural storage).

- If evidence is available with regard to channel loss rates occurring during passage of floods, those rates may be used in the routing process. However, their effect is usually small compared to PMF flow and often can be neglected.
- Consider large natural constrictions as control points for channel routing.

8-10.5 Base Flow Coincident with Probable Maximum Flood

The flow rate is the river for basins or subbasins at the time the PMP begins should be consistent with the antecedent approach selected from Section 8-3.1. Average monthly flow should be obtained for the months during the season when the critical PMP would occur. Tabulated monthly average flow data are available in USGS water data reports. The average monthly flow for the month of the critical PMP should be used flow and added to the inflow PMF hydrograph before routing through the reservoir, or combining or routing subbasin hydrographs. When using HEC-1 this initial flow is the parameter STRTQ. For "ungaged" basins, the average monthly flow per square mile of drainage area, obtained from records for nearby "gaged" basins, should be used to compute the required base flow. If the 100-year, 3-day snowmelt option, as delineated in Section 8-10.2.1, is used, there is no need for an additional base flow component as that component is already included in the data record used for the statistical analysis.

8-10.6 Inflow PMF Hydrograph

Use the input developed in Sections 8-10.1 through 8-10.4 and run HEC-1 for computation of the inflow PMF hydrograph.

8-10.7 Review and Sensitivity Analysis of Representative PMF Hydrograph

8-10.7.1 General Considerations

The first computed inflow PMF hydrograph should be considered as preliminary. Reviews of the assumptions seen to have a significant effect on the PMF should be made to assess their sensitivity.

- If uncertainty exists with regard to the assumptions, a sensitivity analysis should be made to determine the degree to which key parameters affect the PMF.

- If the PMF is particularly sensitive to the magnitude of a parameter, the source of the parameter determination should be reviewed to ensure that the value chosen is reasonable.

- The results of the sensitivity analysis and the selection of the parameter should be documented and justified.

8-10.7.2 Nonlinear Effects and the Representative Unit Hydrograph

The predicted peak flow of the inflow PMF may be too low (or too high) as a result of nonlinear effects in the runoff and channel-flow process that violate the unit-hydrograph assumption of linearity between streamflow and excess rainfall. Studies related to these nonlinear effects have been inconclusive [Pilgrim 1988]. However, if the historic floods used in developing the representative unit hydrographs are large, nonlinear effects may not be significant; if the historic floods used are small. Those effects can be important. To provide guidance for adjustments to compensate for possible nonlinear effects, the following recommendations are provided:

- Where historic floods used in developing unit hydrographs were large and clearly overbank throughout the channels in the basin, no correction is necessary.

- If the historic floods, used in developing unit hydrographs, were small and clearly not out of banks, the following adjustments should be considered:

  — Where valleys in the basin are V-shaped with little overbank storage, the unit-hydrograph peak should be increased by 20 percent [Pilgrim 1988]. This requires adjustment to unit-hydrograph ordinates to obtain 1 inch of runoff over the drainage area in volume under the unit hydrograph.

  — Where valleys in the basin have overbank storage, the unit-hydrograph peak should be increased by 15 percent [Pilgrim 1988]. This requires adjustment to unit-hydrograph ordinates to obtain 1 inch of runoff over the drainage area in volume under the unit hydrograph.

— If the spillway flood flow is volume-dependent, a peak adjustment will probably not be important.

— If the peak of the routed PMF will depend on peak inflow, an adjustment will be important. The effect of this correction for nonlinear effects should be considered during sensitivity analyses described in Section 8-10.1.

The above does not generate an inflow PMF hydrograph for a reservoir! The hydrologic engineer must apply PMP (PMS), initial loss, uniform loss, baseflow, snowmelt, unit hydrograph, initial reservoir level, spillway rating curves, and turbine flow rating (if appropriate) with the HEC-1 program to obtain the inflow PMF hydrograph.

Hydrologic Group	Minimum* Infiltration Rate (in/hr)	Soil Description
A	0.30 to 0.45	Deep sand, deep loess, aggregated silts
B	0.15 to 0.30	Shallow loess, sandy loam
C	0.05 to 0.15	Clay loams, shallow loam, soils low in organic content, soils usually high in clay
D	0 to 0.05	Soils that swell significantly when wet, heavy plastic clays, certain saline soils
* Within the Approximate Method, use low values unless values up to the maximum within the hydrologic group can be justified.		

Table 8-10.1 Minimum Infiltration Rates for Hydrologic Soil Groups [USDA 1955].

MEAD & HUNT, INC.
FERC Chapter VIII Engineering Guideline Applications

Project	FERC Project No.	Client	River(s)	Drainage Area (sq. mi.)	No. of Subbasins	Previous PMF	PMF New Guidelines	FERC Status
DONE:								
Mio	2448	Consumers Power Co.	Au Sable River	1300	1	45,000	20,570	Pending
Alcona	2447	Consumers Power Co.	Au Sable River	1520	2	53,200	22,100	Approved
Lord	2449	Consumers Power Co.	Au Sable River	1620	2	56,000	23,270	Approved
Five Channels	2453	Consumers Power Co.	Au Sable River	1630	2	56,000	23,270	Approved
Cooke	2450	Consumers Power Co.	Au Sable River	1650	2	56,000	23,270	Approved
Foote	2436	Consumers Power Co.	Au Sable River	1675	2	56,000	23,270	Approved
Webber	2566	Consumers Power Co.	Grand River	1737	6	87,000	62,500	Sent to client
Croton	2468	Consumers Power Co.	Muskegon	2300	2	54,500	46,400	Pending
Hardy	2452	Consumers Power Co.	Muskegon	1910	1	54,000	38,200	Pending
Rogers	2451	Consumers Power Co.	Muskegon	1800	1	65,100	39,400	Pending
Tippy	2589	Consumers Power Co.	Manistee	1400	2	39,700	18,700	Pending
Hodenpyl	2599	Consumers Power Co.	Manistee	1000	1	33,000	12,700	Pending
Dairyland Flambeau	1960	Dairyland Power Coop.	Flambeau	1840	4	100,000	75,700	Pending
Big Falls	2390	Northern States Power	Flambeau	1790	3	125,000	77,800	Pending
Upper Park Falls	2640	Flambeau Paper Co.	Flambeau	750	3	90,850	54,400	Pending
Lower Park Falls	2421	Flambeau Paper Co.	Flambeau	758	3	90,850	54,400	Pending
Pixley	2395	Flambeau Paper Co.	Flambeau	786	3	90,800	54,600	Pending
Crowley	2473	Flambeau Paper Co.	Flambeau	820	3	90,000	51,400	Pending
Rainbow	2113	Wis. Valley Improvement Co.	Wisconsin	740	1	56,600	33,000	Pending
Otter Rapids	1957	Wisconsin Public Service Corp.	Wisconsin	535	1	55,000	35,600	Sent to client
Jersey	2476	Wis. Valley Improvement Co.	Tomahawk	554	2	31,700	18,000	Sent to client
Rice	2113	Wis. Valley Improvement Co.	Wisconsin	544	2	32,900	17,200	Pending

Project	FERC Project No.	Client	River(s)	Drainage Area (sq. mi.)	No. of Subbasins	Previous PMF	PMF New Guidelines	FERC Status
Spirit	2113	Wis. Valley Improvement Co.	Wisconsin	153	1	20,000	17,500	Pending
Willow	2113	Wis. Valley Improvement Co.	Wisconsin	310	1	38,300	12,300	Pending
Alexander	1979	Wisconsin Public Service Corp.	Wisconsin	2484	8	188,000	68,500	Sent to client
Hat Rapids	1968	Wisconsin Public Service Corp.	Wisconsin	1153	3		42,600	Sent to client
Grandmother	2180	Packaging Corp. of America	Wisconsin	2246	8	140,050	70,900	Sent to client
Grandfather	1966	Wisconsin Public Service Corp.	Wisconsin	2269	8	140,050	71,200	Sent to client
Tomahawk	1994	Wisconsin Public Service Corp.	Wisconsin	2020	6	130,000	63,600	Sent to client
Kings	2239	Tomahawk Power & Pulp	Wisconsin	1320	3	64,000	40,300	Pending
Merrill	1989	Wisconsin Public Service Corp.	Wisconsin	2720	9	193,000	80,500	Sent to client
Wausau	1999	Wisconsin Public Service Corp.	Wisconsin	3056	10	162,600	115,200	Sent to client
Castle Rock	1984b	Wisconsin River Power Co.	Wisconsin	7036	22	292,000	221,000	Sent to client
Petenwell	1984a	Wisconsin River Power Co.	Wisconsin	5964	19	328,000	214,000	Sent to client
Brown Bridge	2978	Traverse City Light & Power	Boardman	151	1	40,800	7,000	Draft sent to client
Boardman	2979	Traverse City Light & Power	Boardman	237	2	58,100	10,000	Draft sent to client
Sabin	2980	Traverse City Light & Power	Boardman	239	2	58,100	10,000	Draft sent to client
Hatfield	10805	Midwest Hydraulics	Black River	1280	1	180,000	117,000	Pending
Upper	2589	Marquette Light & Power	Dead	153	8	42,300	30,300	Pending
Lower	2589	Marquette Light & Power	Dead	159	9	43,300	30,300	Pending
High Falls	2595	Wisconsin Public Service Corp.	Peshtigo	537	2	44,000	21,800	Pending
Caldron Falls	2525	Wisconsin Public Service Corp.	Peshtigo	465	1	40,000	20,500	Pending
Oconto Falls	2523	North American Hydro	Oconto	739	1	35,700	17,800	Pending since 5/93
Sturgeon	2471	Wisconsin Electric Power Co.	Sturgeon	305	1	16,800	9,900	Pending since 2/93
IN PROGRESS:								
Chippewa Reservoir	8286	Northern States Power	Chippewa	763	*	75,000	*	*
Cornell	2639	Northern States Power	Chippewa	3400	*	323,000	*	*

Project	FERC Project No.	Client	River(s)	Drainage Area (sq. mi.)	No. of Subbasins	Previous PMF	PMF New Guidelines	FERC Status
Jim Falls	2491	Northern States Power	Chippewa	3450	*	330,000	*	*
Holcombe	1982	Northern States Power	Chippewa	3300	*	329,000	*	*
Wissota	2567	Northern States Power	Chippewa	5548	*	367,000	*	*
Edenville	10808	Wolverine Power Coop.	Tittabawassee	933	7		73,600	*
Sanford	2785	Wolverine Power Coop.	Tittabawassee	968	8	131,400	73,200	*
Secord	10809	Wolverine Power Coop.	Tittabawassee	190	2		27,800	*
Smallwood	10810	Wolverine Power Coop.	Tittabawassee	308	4		41,300	*
Big Eau Pleine	2113	Wis. Valley Improvement Co.	Eau Pleine	377	1	115,000	82,900	FERC reviewing
Riverdale	9003	Northern States Power Co.	Apple River	300	25	32,000	*	*
Apple River Falls	9002	Northern States Power Co.	Apple River	303	26	32,700	14,400	*
Lake Nacimiento	*	Monterey Cnty Water Resources	Salinas River	*	*	*	*	*
San Antonio Lake	*	Monterey Cnty Water Resources	*	*	*	*	*	*

* *Project in progress; information is not yet available.*

DATA COLLECTION FOR UNGAGED WATERSHEDS SENSITIVITY

Tuesday 1:30 p.m.

8-10.7 Review and Sensitivity Analysis of Representative PMF Hydrograph

8-10.7.1 General Considerations

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  — Where valleys in the basin have overbank storage, the unit-hydrograph peak should be increased by 15 percent [Pilgrim 1988]. This requires adjustment to unit-hydrograph ordinates to obtain 1 inch of runoff over the drainage area in volume under the unit hydrograph.

— If the spillway flood flow is volume-dependent, a peak adjustment will probably not be important.

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The above does not generate an inflow PMF hydrograph for a reservoir! The hydrologic engineer must apply PMP (PMS), initial loss, uniform loss, baseflow, snowmelt, unit hydrograph, initial reservoir level, spillway rating curves, and turbine flow rating (if appropriate) with the HEC-1 program to obtain the inflow PMF hydrograph.

Hydrologic Group	**Minimum* Infiltration Rate (in/hr)**	**Soil Description**
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D	0 to 0.05	Soils that swell significantly when wet, heavy plastic clays, certain saline soils
* Within the Approximate Method, use low values unless values up to the maximum within the hydrologic group can be justified.		

Table 8-10.1 Minimum Infiltration Rates for Hydrologic Soil Groups [USDA 1955].

EXAMPLE: CORSORONA RAPIDS

Tuesday 2:00 p.m.

Probable Maximum Flood Studies

Example 3: Corsorona Rapids Hydroelectric Project

June 1994

Purpose: *To illustrate a multi-subbasin PMF study for an ungaged basin with a regional unit hydrograph study.*

Summary

Subbasin Division:	Eight subbasins
Routing:	COE UNET model
Unit Hydrograph Analysis:	Regional study
Loss Rates:	Detailed method using STATSGO data
Initial Reservoir Level	Annual maximum normal operating level at run-of-river project
Snowpack:	100-year snowpack
Snowmelt:	10-day melt sequence including 3-day record high temperatures
Sensitivity Analysis:	None

Example 3
Corsorona Rapids Hydroelectric Project
Probable Maximum Flood

June 1994

Table of Contents

Tables

Exhibits

Appendix

CASE STUDY

CORSORONA HYDRO PROJECT

to illustrate a multi-basin PMF study for an ungaged watershed that requires a regional unit hydrograph study

Materials in TAB 23 in Volume 2 of Notebook

Probable Maximum Flood Studies

Example 4: Corsorona Hydroelectric Project

March 1994

Purpose: *To illustrate a multi-subbasin PMF study for an ungaged basin with a regional unit hydrograph study.*

Summary

Subbasin Division:	Eight subbasins
Routing:	COE UNET model
Unit Hydrograph Analysis:	Regional study
Loss Rates:	Detailed method using STATSGO data
Initial Reservoir Level	Annual maximum normal operating level at run-of-river project
Snowpack:	100-year snowpack
Snowmelt:	10-day melt sequence including 3-day record high temperatures
Sensitivity Analysis:	None

PMF STUDY REPORT OUTLINE

UNGAGED BASINS

VII. LOSS RATES

VIII. COINCIDENT HYDROMETEOROLOGICAL AND HYDROLOGICAL CONDITIONS FOR THE PROBABLE MAXIMUM FLOOD

IX. PMF HYDROGRAPHS

CORSORONA HYDROELECTRIC PROJECT
PROBABLE MAXIMUM FLOOD

PROJECT DATA

- 195-foot long concrete overflow dam
- Tainter gate section with twelve 24-foot wide by 18-foot high gates
- Concrete and masonry powerhouse
- 40-foot high concrete gravity dam

CORSORONA PROJECT DATA (continued)

- Total spillway capacity of dam - 88,000 cfs when pool is at top of right gravity dam (elevation 1,282 feet)
- Reservoir surface area - 190 acres
- Total storage volume at max. normal operating level - 3,600 acre-feet
- Operated in run-of-river mode

CORSORONA PROJECT DATA (continued)

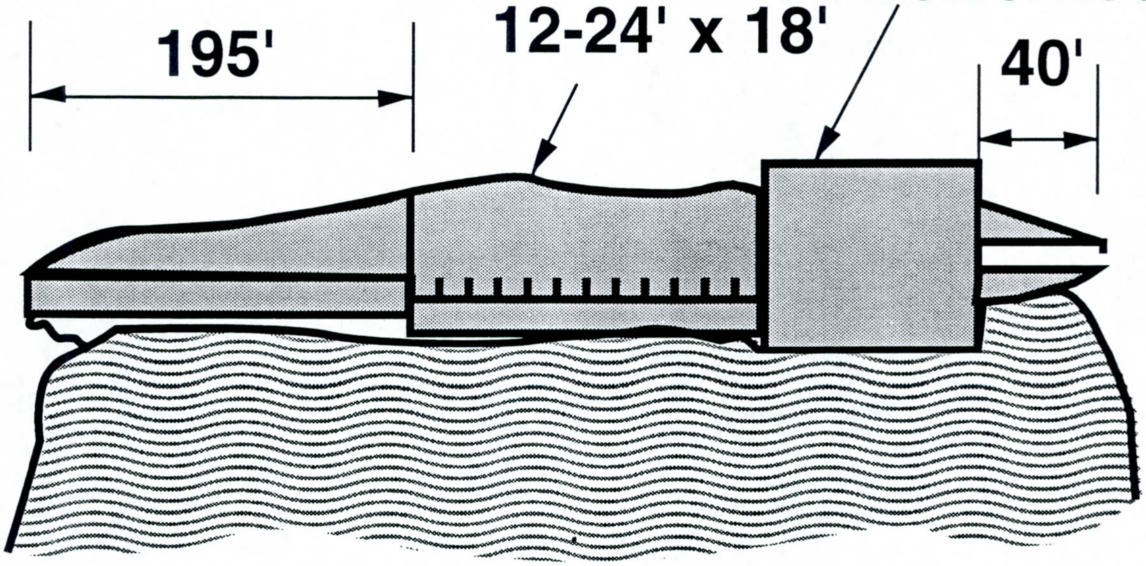
- Operating level - 1,278.5 feet
  - Tolerance of ±0.5 foot
- Visited twice daily by operator
- Headwater/tailwater levels monitored from Edwards City control center
- Spill gates operated by two moveable electric hoists

CORSORONA PROJECT DATA (continued)

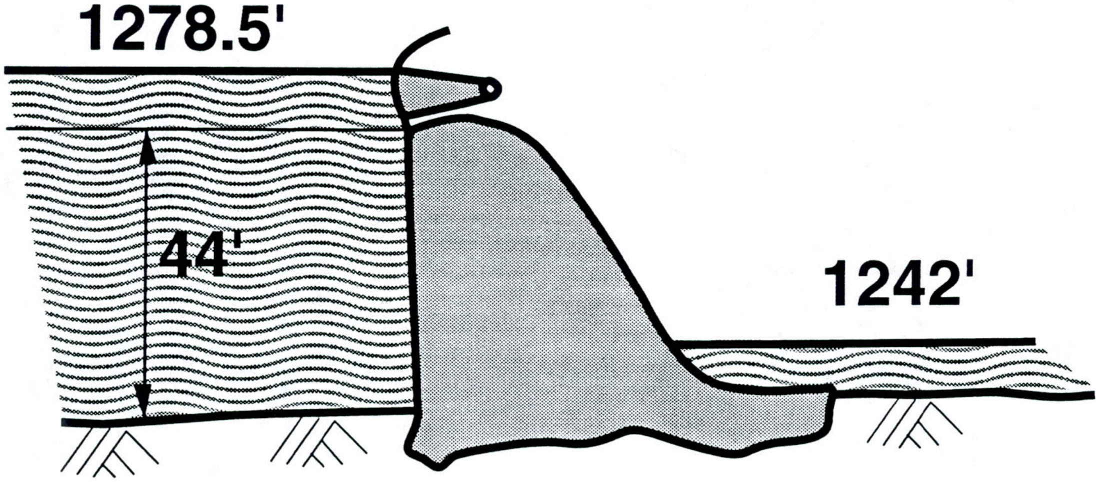
- Maximum height of dam above riverbed - 44 feet
- No earth dikes
- Annual max. normal headwater level - 1,278.5 feet
- Normal Tailwater - 1,242 feet

CORSORONA PROJECT DATA (continued)

- Backup generator on-site to power hoists in event of electric failure



PLAN



SECTION

Q Spillway = 88000 cfs
Reservoir Elevation = 1282 ft

GHES_GHES cassidy 22647-000_7plan/section of dam 8/29/94

CORSORONA BASIN HYDROLOGIC DATA

- Drainage area approx. 2,721 sq. miles
- Approx. 95 miles from headwaters of basin to Corsorona Rapids project
- Northern portion of basin - small lakes, wetlands, forested
- Southern 1/3 of basin used more for agriculture but 50% forested

BASIN HYDROLOGIC DATA (continued)

- Basin relief is moderate - elev. from 1,278.5 feet at project to 1,859 at upper basin divide
- U.S. Geological Survey steam gage (Blue River near Mercy Creek) approx. 7 miles downstream of project at drainage area of 2,823 sq. miles

FIELD VISIT

- the operator was interviewed to confirm project operation during flood conditions

- spillway gates are tested annually and are in good working order

- information on project works, reservoir storage capacity and operations of upstream dams was obtained through telephone interviewed with project owner and review of previous consultant's inspection reports

PREVIOUS FLOOD STUDIES

- No previous PMF studies for the Corsorona Hydro Project exist.

- There are two previous PMF studies on the Blue River Basin upstream of the Corsorona Rapids Project:a 1981 study for the Badger Butte Dam predicted a PMF peak of 45,000 cfs and a 1984 study for Reis Dam yielded a PMF of 16,700 cfs

- There are some reverse reservoir routing studies performed by the state geological survey for some of the hydro projects in the Blue River Basin

- Federal flood insurance study reports are also available from FEMA for some of the surrounding counties

CORSORONA STREAM & RAIN GAGES

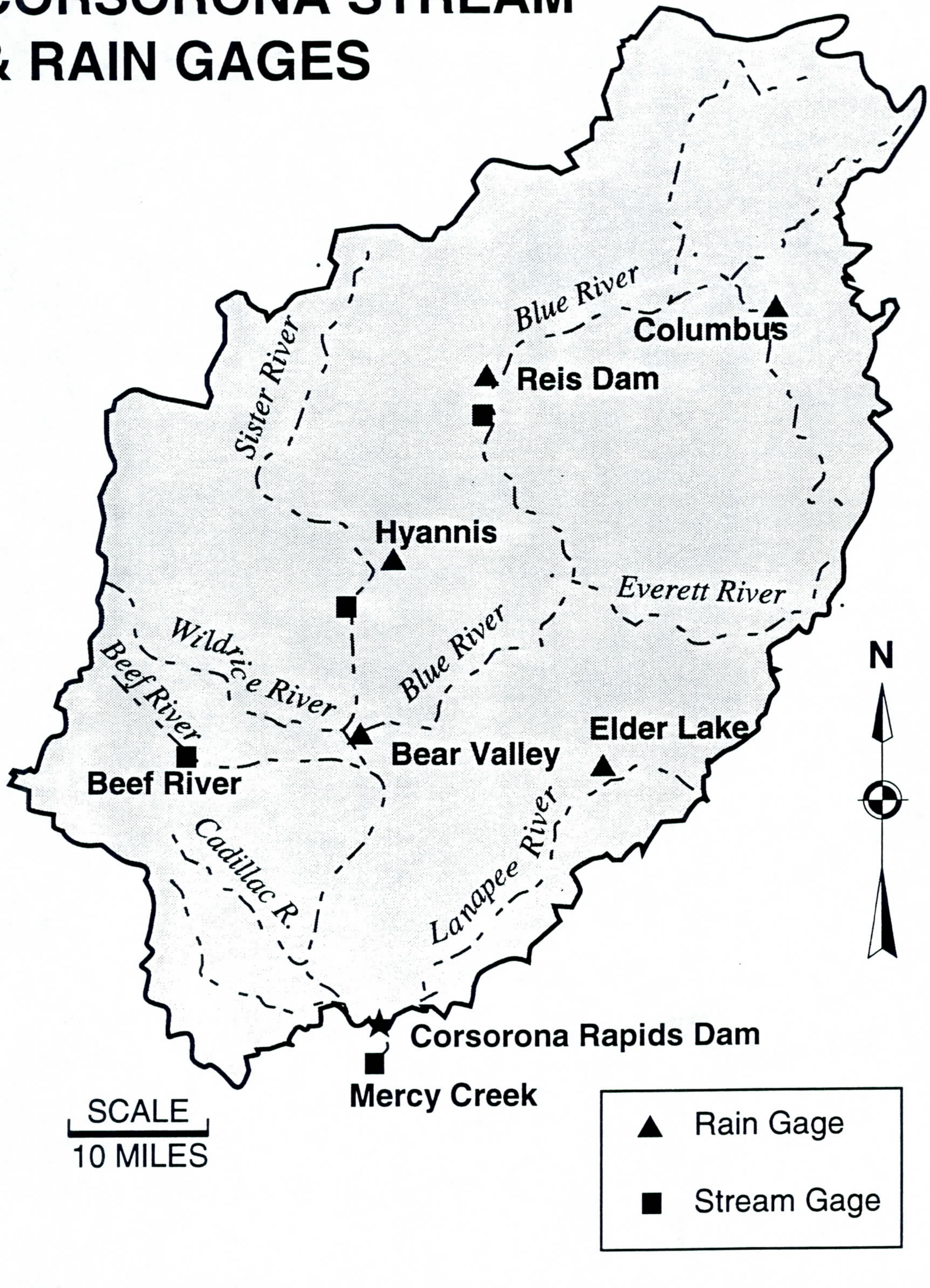


TABLE 4
Blue River at Corsorona Rapids
Stream Gages In and Near Blue River Basin

Gage Name and Number	Drainage Area (sq. mi.)	Period of Record
Blue River near Mercy Creek (No. 60982200)[1]	2,821	1938–1988
Sister River near Hyannis (No. 60873330)	545	1930–1974
Blue River near Reis (No. 60880000)	757	1936–present
Beef River at Beef Rapids (No. 60845600)	82	1979–present

1 *Outside basin, 7 miles downstream of Corsorona Rapids project.*

TABLE 1
Largest Floods Recorded at Blue River Near Mercy Creek Stream Gage

Date of Flood	Peak Flow (cfs)
April 23, 1963	32,000
September 29, 1943	29,900
March 21, 1948	29,300
May 3, 1985	28,400
May 1, 1977	27,500
October 15, 1950	25,500
October 11, 1957	25,500
April 27, 1958	24,900

TABLE 5
Historic Floods in Blue River Basin

Gage Name and Number	Peak Flow of Record (cfs)	Date of Peak Flow
Blue River near Mercy Creek (No. 60982200)	32,000	4/23/63
Sister River near Hyannis (No. 60873330)	5,200	10/5/84
Blue River near Reis (No. 60880000)	6,000	4/11/63
Beef River at Beef Rapids (No. 60845600)	4,200	6/3/87

TABLE 2
Upstream Dams and Reservoirs

Name of Dam	Owner	River	Drainage Area (sq. mi.)	Storage (acre-ft.)
Reis Dam	BRHC	Blue River	753	63,600
Frenchman Lake Dam	City of Victoria	Blue River	878	21,500
Elbow Rapids	Rivers of the North Corporation (RONCO)	Blue River	1,160	7,500
Badger Butte Dam	Western Hydro Co.	Blue River	1,320	14,000
Upper Sister Dam	BRHC	Sister River	544	38,000
Lower Sister Dam	RONCO	Sister River	553	2,200
Hyannis Dam	Western Hydro Co.	Blue River	2,030	19,000
Barnum Dam	Big Top Paper Co.	Blue River	2,250	6,700
Bailey Dam	Big Top Paper Co.	Blue River	2,270	3,200

WATERSHED MODEL METHODOLOGY

- use a program called TSPMP which is based on U.S. Army Corps of Engineers' HMR 52 approach to determine the Probable Maximum Storm (PMS) distribution for the watershed

- use HEC-1 for PMF development

- use U.S. Army Corps of Engineers' UNET dynamic routing program to route the floods from the mouths of the tributary basins to the project

SUBDIVISION OF WATERSHED INTO SUB-BASINS

- the 2,721-square mile watershed was subdivided into eight sub-basins ranging in size from 136 sq. mi. to 753 sq. mi. according to the locations of the existing dams and reservoirs

TABLE 3
Subbasins Used to Model PMF

Subbasin No.	Major Tributaries	Drainage Area (sq. mi.)
1	Blue River (above Reis)	753
2	Everett River	265
3	Blue River (Reis to Lake Somo)	313
4	Sister River	554
5	Wildrice River	136
6	Beef River	168
7	Cadillac River	295
8	Lenapee River	237

UNIT HYDROGRAPH DEVELOPMENT

- this is an ungaged basin larger than 100 sq. mi.

- only unit hydrograph study identified is the one performed by local state water resources department at Landro Levee, some 23 miles downstream of the Corsorona Rapids Project. This study was performed to estimate the 100- and 500-year flood hydrographs for proposed levee works

- must use regional approach to develop unit hydrograph

- eight basins in the region, including three upstream of the project in the Blue River Basin, were selected and used in the regional study

- they were selected for having drainage areas, topography, climate and geological characteristics similar to the Blue River sub-basins

  Note that the FERC guidelines recommend the use of at least 10 gaged basins in a regional study. Justifications must be given if using fewer than recommended

  In this case, other gaged basins in the region are suitable due to different land use pattern, topography and soil conditions

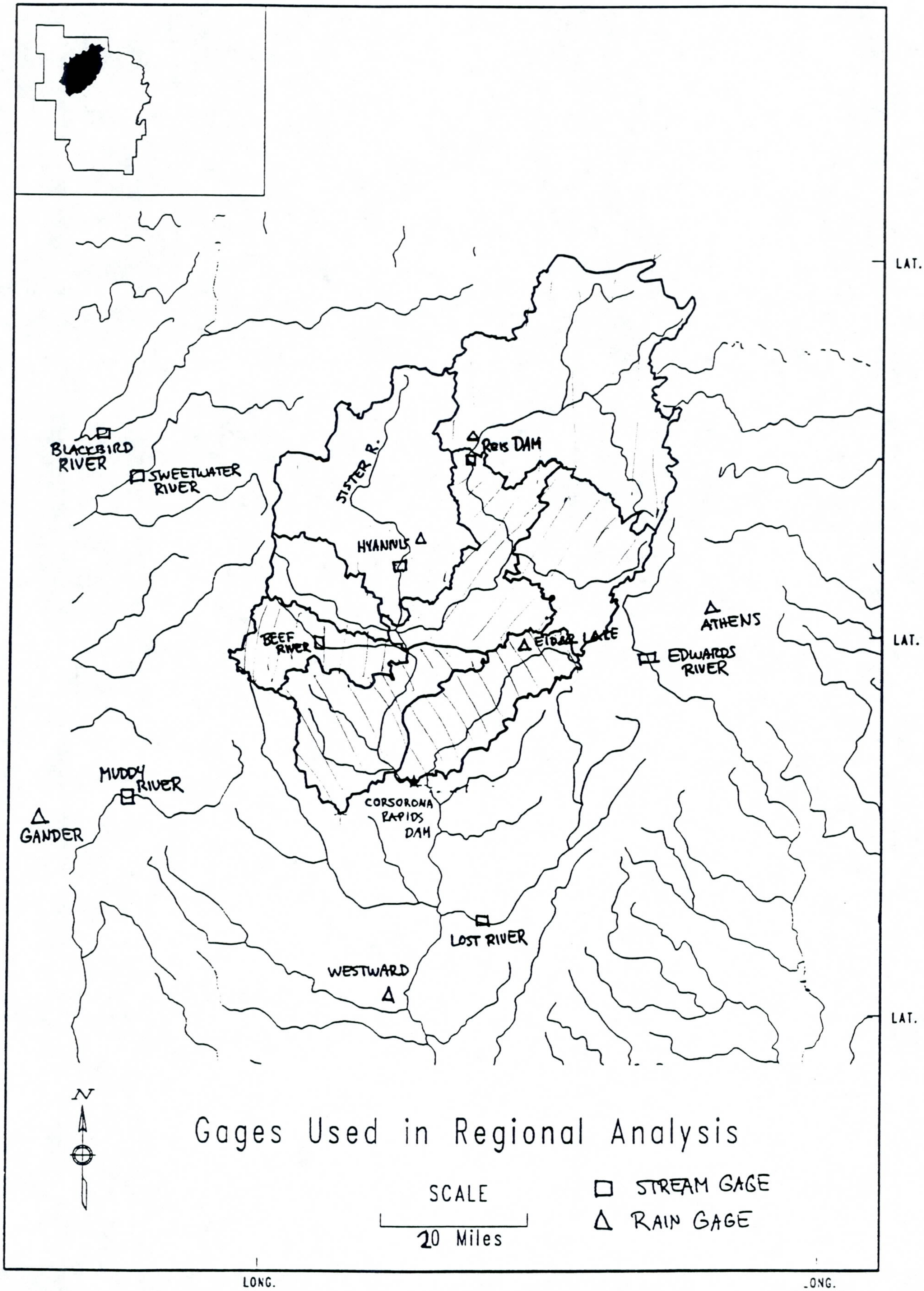
- the time-area curves for each gaged basin were developed from the 7.5-minute USGS quadrangle sheets

- the Clark's coefficients T_c and R were estimated for one to three floods on each basin with additional historical floods being used for verification

- the Clark coefficients T_c and R for all eight basins were regressed against their physical basin characteristics

TABLE A-1
Gaged Basins Used in Regional Analysis

Name of Basin	Drainage Area at Gage (sq. mi.)	Period of Record	Flood Dates Used in Analysis
Blue River near Reis (No. 60880000)	757	1936–present	October 1953
			September-October 1969
			June 1990
Sister River near Hyannis (No. 60873330)	545	1930–1974	June 1958
			October 1968
Beef River at Beef Rapids (No. 60845600)	82	1979–present	April 1980
Edwards River near Athens (No. 60896000)	184	1914–1928, 1938–present	April–May 1973
			June 1979
			June 1990
Lost River near Oak Prairie (No. 60967000)	375	1939–present	September 1959
			September 1969
			October 1986
			June 1990
Sweetwater River at Yale (No. 60411400)	224	1937–present	May 1968
			June 1972
			June 1986
Muddy River at Bradley Landing (No. 60499900)	215	1944–present	May 1968
			April-May 1973
			September 1980
Blackbird River near Jersey (No. 60675000)	749	1913–present	June 1968
			May 1973
			September 1980
			October 1986



Gages Used in Regional Analysis

TABLE A-2
Clark Parameters Estimated in Regional Study

Basin Name and Gate No.	Clark T_c (hrs.)	Clark R (hrs.)	$\frac{R}{T_c+R}$
Blue River (No. 60880000)	35	50	0.59
Sister River (No. 60873330)	54	72	0.57
Beef River (No. 608454600)	21.5	25	0.54
Edwards River (No. 6089600)	30.5	62	0.67
Lost River (No. 60967000)	46	45	0.49
Sweetwater River (No. 60411400)	11.5	11.5	0.50
Muddy River (No. 60499900)	28	32	0.53
Blackbird River (No. 60675000)	35	28	0.44

RESULTS OF REGRESSION ANALYSIS

$$Tc = 0.189\ L^{0.75}\ F^{0.58}$$

$$R = 1.95\ L^{0.44}\ ST^{0.5}$$

L = Length of Longest Flow Path

F = Percent Forested

ST = Percent Covered by Lakes and Wetlands

TABLE A-3
Regional Unit Hydrograph Analysis:
Regression Parameters and Predicted T_c and R

Basin	L	ST	F	T_c^1	T_c^2	R^1	R^2
Blue River	58.0	28.1	63.5	35	42.6	50	60.9
Sister River	64.4	28.6	64.7	54	46.5	72	64.3
Beef River	18.8	17.2	73.6	21.5	20.0	25	29.1
Edwards River	40.2	23.2	66.4	27.5	33.2	57	47.1
Lost River	63.5	11.4	49.8	46	39.6	45	40.4
Sweetwater River	50.2	4.9	27.5	28	23.6	32	23.9
Muddy River	32.3	1.9	16.1	11.5	12.5	11.5	12.3
Blackbird River	88.0	7.3	30.8	35	38.3	28	37.3

1 *By calibration.*
2 *By regression.*

TABLE 6
Estimated Clark Unit Hydrograph Parameters for Model Subbasins

No.	Subbasin Name	Independent Variables			Dependent Variables	
		L (mi.)	ST (%)	F (%)	T_c (hrs.)	R (hrs.)
1	Upper Blue River	58	28	64	43	61
2	Everett River	34	38	51	25	55
3	Blue River	58	23	62	42	55
4	Sister River	70	28	65	49	66
5	Wildrice River	26	22	68	24	38
6	Beef River	34	14	74	31	34
7	Cadillac River	22	12	72	22	26
8	Lenapee River	49	11	64	37	35

UNIT HYDROGRAPH VERIFICATION

- Cold-season Considerations

  - None of the calibrated or verified floods had a significant snowmelt component

  - snowmelt flood is an important element in PMF development for this project

  - must perform validity check of the estimated unit hydrograph parameters for snowmelt floods

  - three snowmelt floods were used: two at the Sister River basin and one in the Beef River Basin

TABLE 7
Estimates of T_c for Snowmelt Floods

River and Date	Starting Flow (cfs)	Peak Flow (cfs)	Est. End of Effective Snowmelt	Est. End of Runoff	Est. T_c from Snowmelt Flood (hrs.)	Regression T_c (hrs.)
Sister River 4/25/56	1,340	1,990	1800 hrs. April 23	0300 hrs. April 26	57	47
Sister River 3/11/89	1,080	1,520	1800 hrs. March 9	2100 hrs. March 11	51	47
Beef River 4/1/64	450	710	1800 hrs. March 31	1100 hrs. April 1	17	20

- the results shows that the T_cs for the snowmelt floods are lower than those for non-snowmelt floods

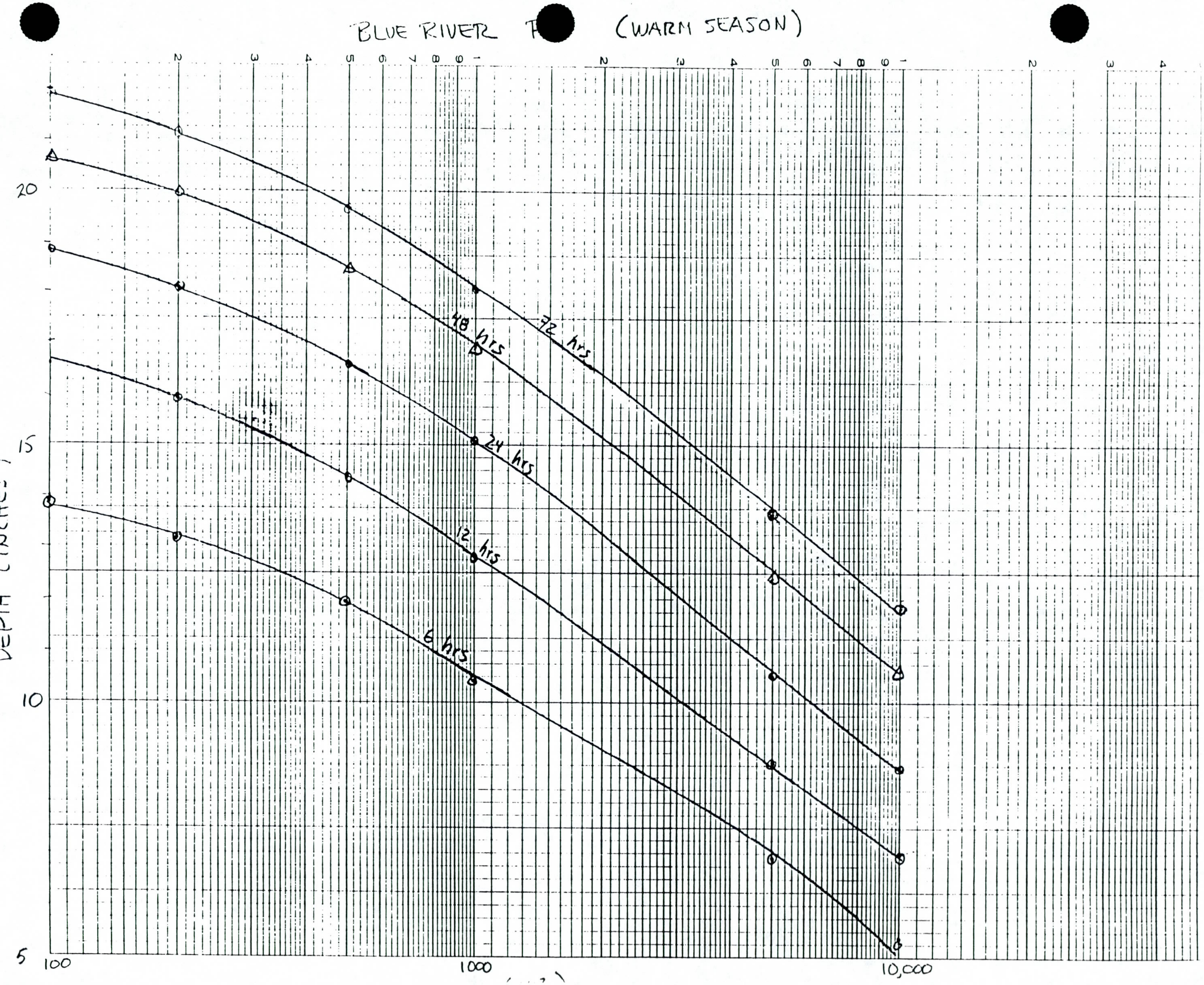
- using warm-season T_c for the development of cold-season floods would be conservative

TABLE 8
Probable Maximum Precipitation
Depth-Area-Duration Values (Warm Season)

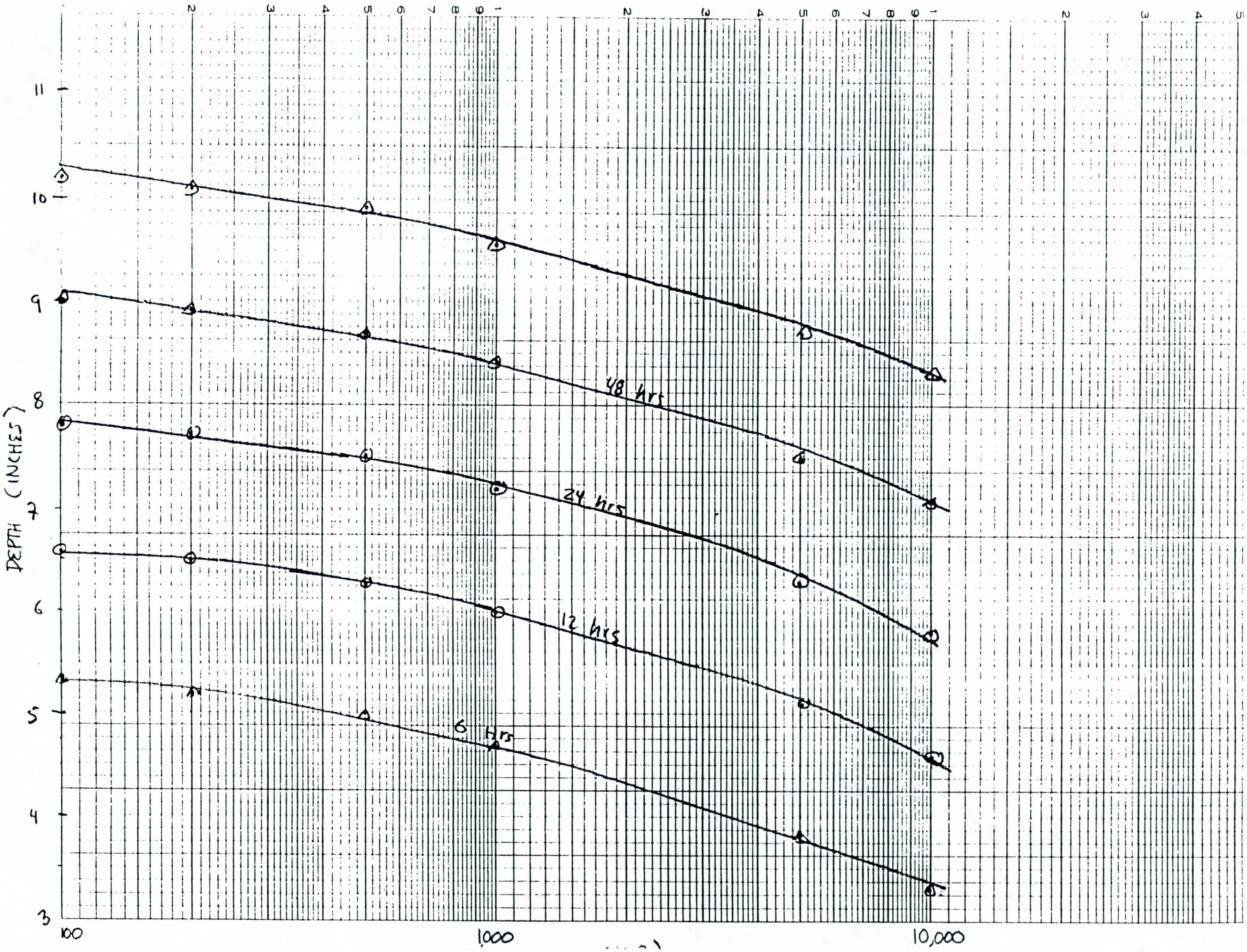
Area (sq. mi.)	6-hour PMP (in.)	12-hour PMP (in.)	24-hour PMP (in.)	48-hour PMP (in.)	72-hour PMP (in.)
100	13.9	16.6	18.8	20.7	21.9
200	13.2	15.9	18.1	20.0	21.2
500	11.9	14.4	16.6	18.5	19.7
1,000	10.4	12.8	15.0	16.9	18.1
5,000	6.9	8.8	10.5	12.4	13.6
10,000	5.3	7.0	8.7	10.6	11.8

TABLE 9
Probable Maximum Precipitation
Depth-Area-Duration Values (Cool Season)

Area (sq. mi.)	6-hour PMP (in.)	12-hour PMP (in.)	24-hour PMP (in.)	48-hour PMP (in.)	72-hour PMP (in.)
100	5.3	6.6	7.8	9.0	10.2
200	5.2	6.5	7.7	8.9	10.1
500	5.0	6.3	7.5	8.7	9.9
1,000	4.7	6.0	7.2	8.4	9.6
5,000	3.8	5.1	6.3	7.5	8.7
10,000	3.3	4.6	5.8	7.1	8.3


BLUE RIVER
(WARM SEASON)
DEPTH (INCHES)
20
15
10
5
100
1000
10,000
72 hrs
48 hrs
24 hrs
12 hrs
6 hrs

BLUE [illegible]VER PMP (COOL SEASON)



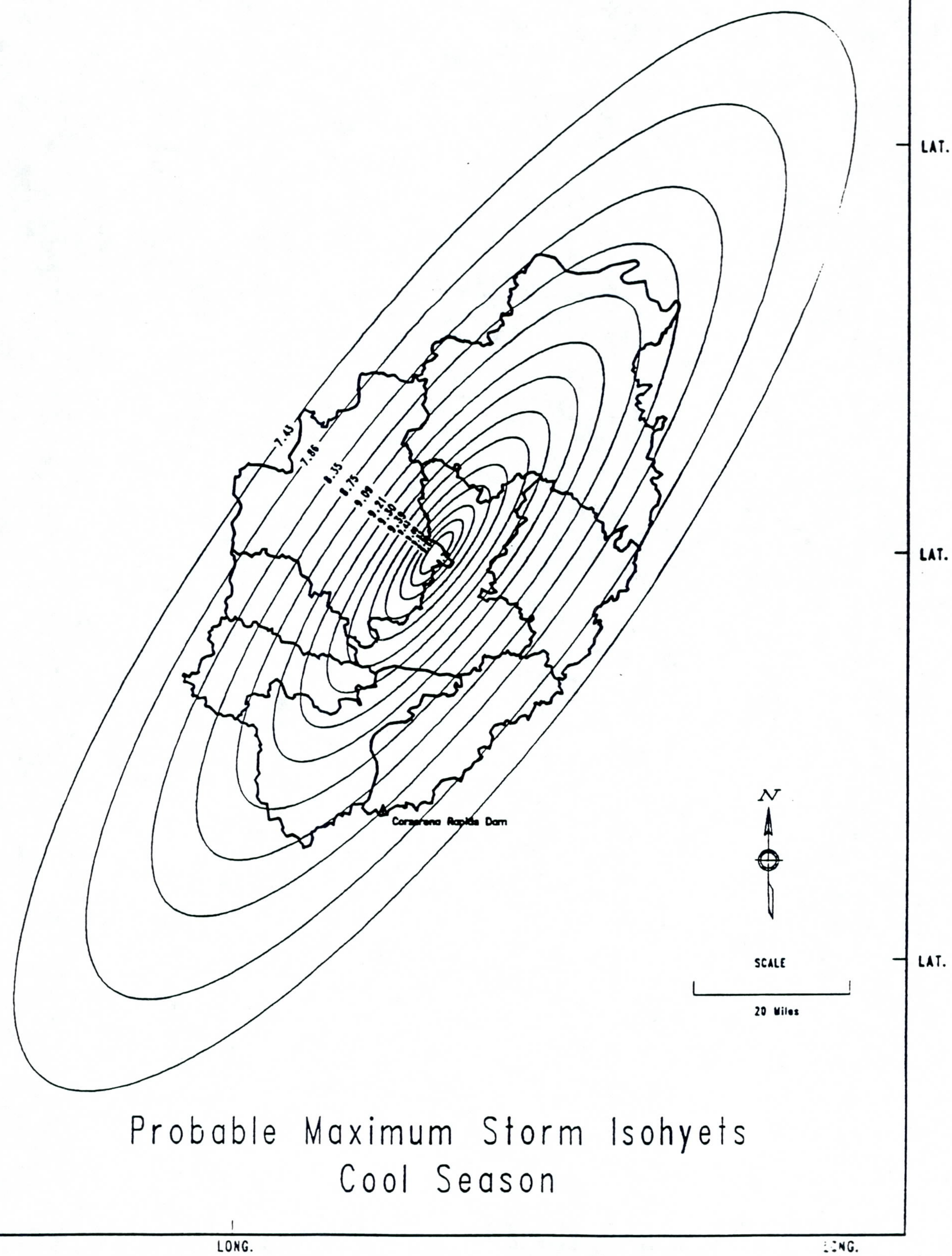
PROBABLE MAXIMUM STORM

- the isohyetal pattern should be moved around to center at different parts of the watershed to maximize the PMP and to affect the development of a critical PMF hydrograph for the project

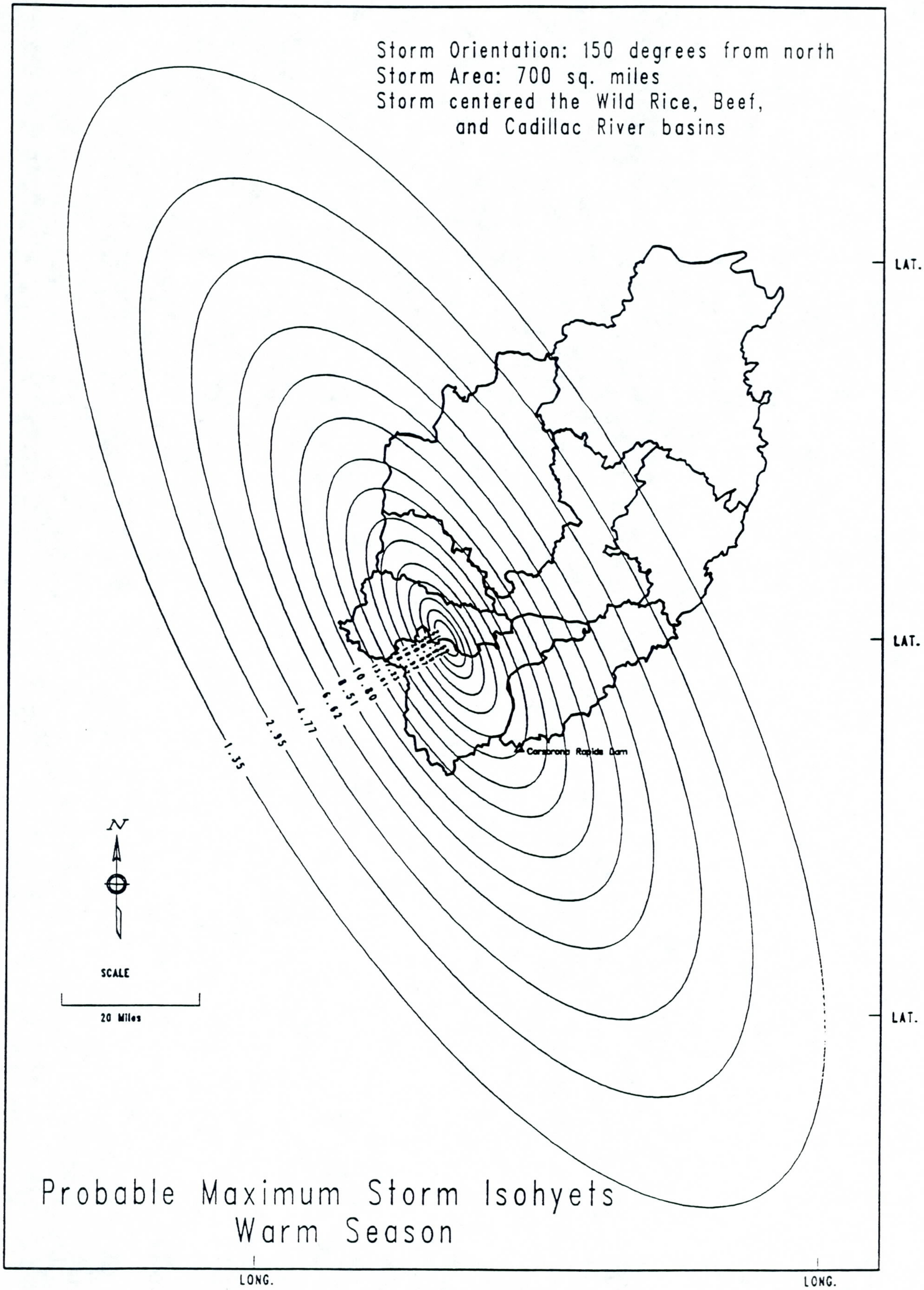
- the program TSPMP was used to develop the various PMPs for the sub-basins

- the critical pattern was found to be the warm-season PMP centering in the Lenapee, Beef and Cadillac sub-basins and oriented 150 degrees from the north

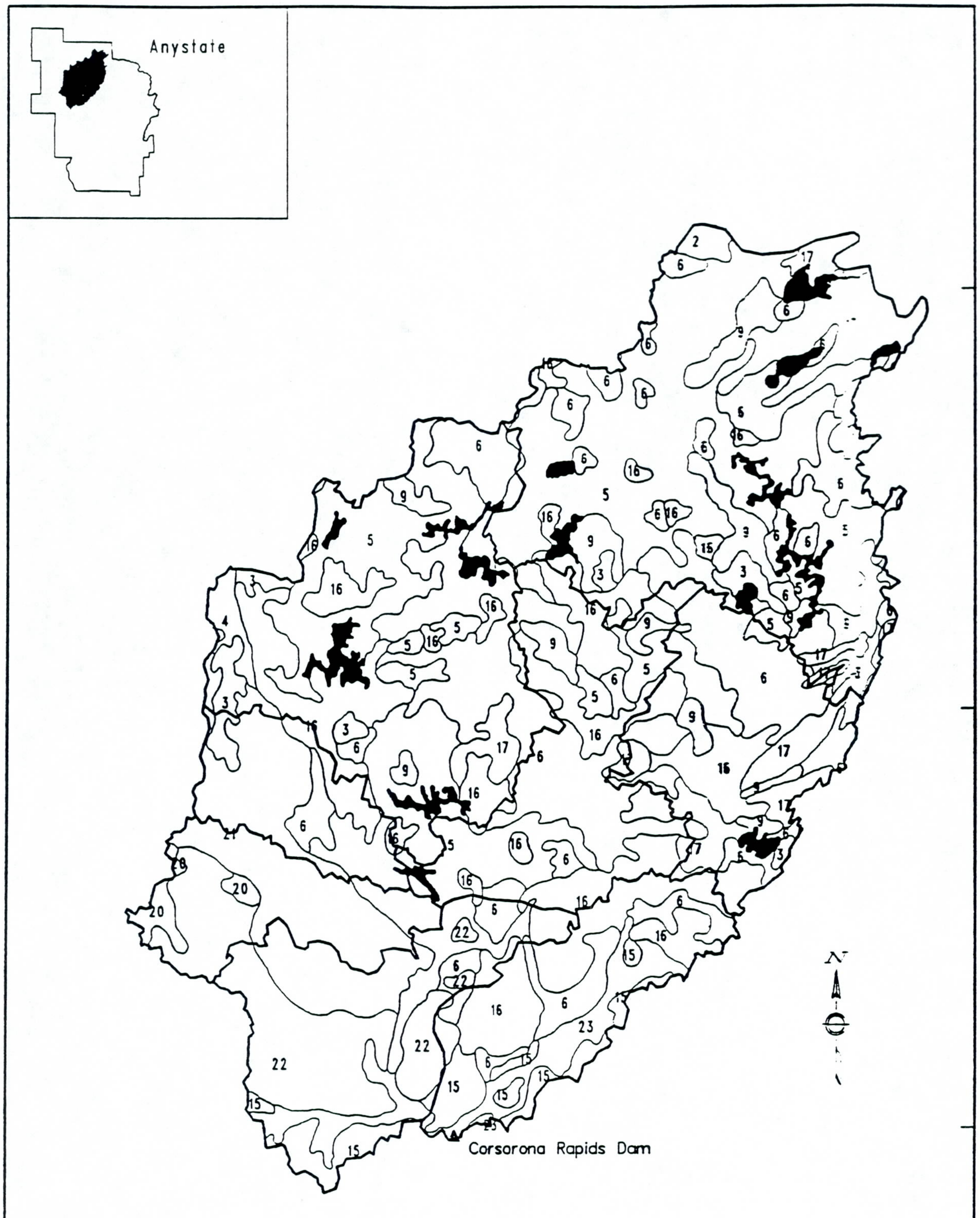
Storm Orientation: 216.7 degrees from north
Storm Area: 6500 sq. miles
Storm centered over whole basin



Probable Maximum Storm Isohyets
Cool Season



Probable Maximum Storm Isohyets
Warm Season



Corsorona Rapids STATSGO Soil Associations

LEGEND

Water

SCALE

10 Miles

LOSS RATES

- initial losses set at zero
- warm-season loss rates:
  - developed from the STATSGO data base
  - STATSGO gives the permeability rate for each soil layer of each soil association down to approximately 60 inches
  - the minimum average permeability among all these soil layers was identified and assumed to be the controlling rate
  - GIS was used to estimate the weighted average value for each basin
  - GIS was also used to identified land cover pattern, such as lake and wetlands
- cold-season loss rate:
  - all soils with the potential to be impervious frozen were assumed to be impervious
  - to be impervious, soils other than sand or loamy sand must occur in the top 24 inches of the soil profile
  - imperviously frozen soils will not occur in natural or managed forests where humus depth is adequate to prevent frost from forming

- all lakes and wetlands are assumed to be frozen

COINCIDENT HYDRO-METEOROLOGICAL CONDITIONS

- Reservoir level - annual maximum normal operating pool at El 1,278.5 feet

- Baseflow:

  - 1 cfs per sq. mi. for sub-basins with a storage area of 25% or more of their drainage areas

  - 0.5 cfs per sq. mi. for sub-basins with a storage area of less than 5% of their drainage areas

  - for storage areas between 5 and 25%, linearly interpolate between the 0.5 and 1.0 cfs per sq. mi.

- QRCSN value:

  - start recession flow at 15% of the peak flow for sub-basins with storage areas 20% or more

  - for sub-basins with storage areas equal to 10% or less, start recession flow at 7%

  - linearly interpolate the QRCSN between 7 and 15% for sub-basins with storage areas between 10 and 20%

- RTIOR used is 1.008, the average of all the hydrographs analyzed

- Snowpack

  - assume typical water equivalent being 25%

  - 100-year snowpack in mid-March is 28 inches, or 7 inches water equivalent

  - 100-year snowpack in mid-April is 11 inches, or 2.25 inches water equivalent

- Snowmelt Computations

  - Daily average temperature data at Hyannis, Reis Dam and Elder Lake were reviewed

  - a 10-day snowmelt period was used to coincide with the PMF duration

  - the critical period was assumed to be a 3-day period superimposed on the 72-hour PMP

  - the critical 3-day temperature sequence was taken as the maximum recorded 3-day sequence with rain on at least one day

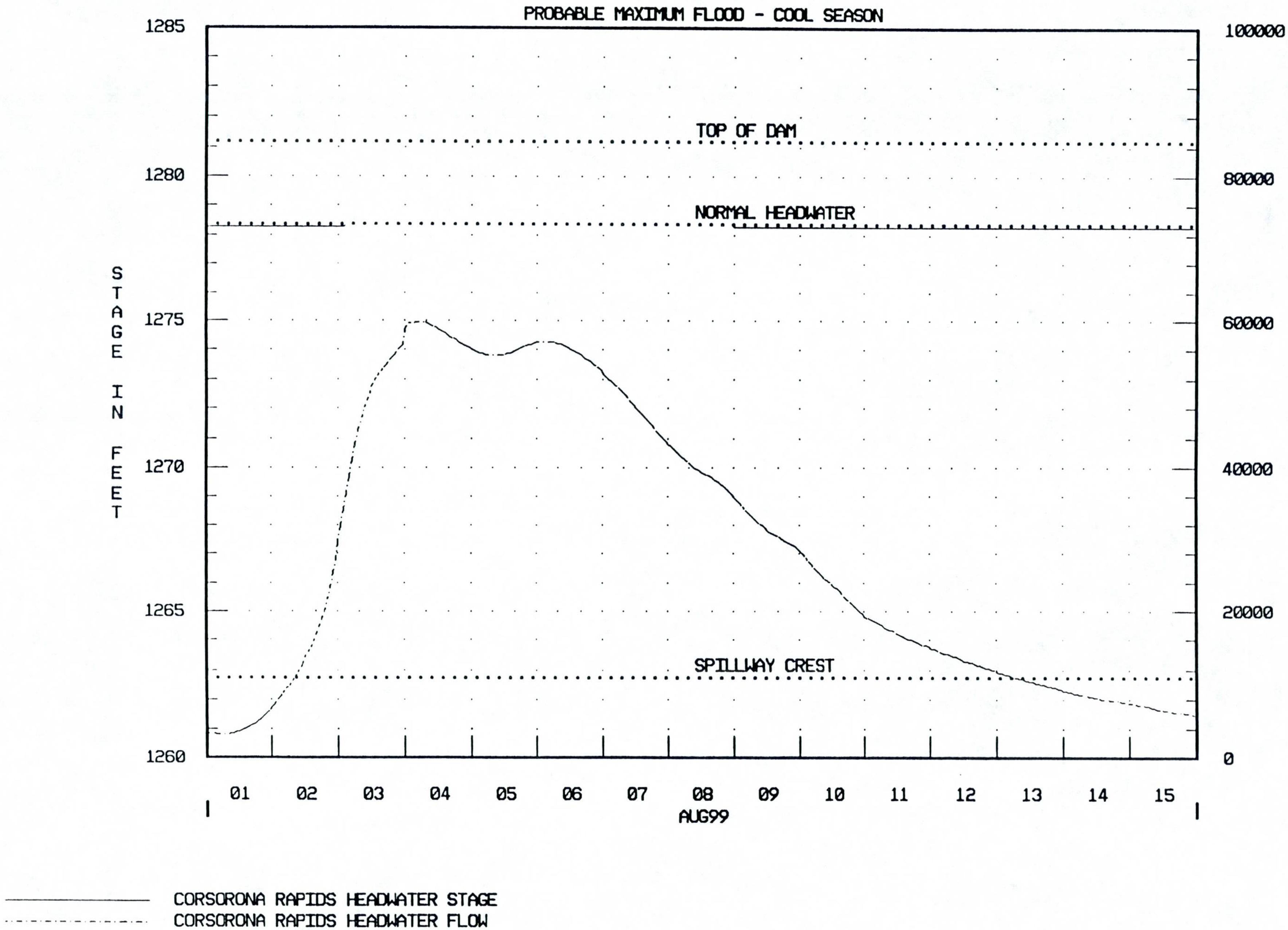
  - the temperature sequence for remaining 7 days was assumed to be average temperature equal to the 10-year 10-day high temperature for the month of interest

  - for March, 3-day sequence is 48°, 53° and 47° F and the remaining 7 days at 42° F

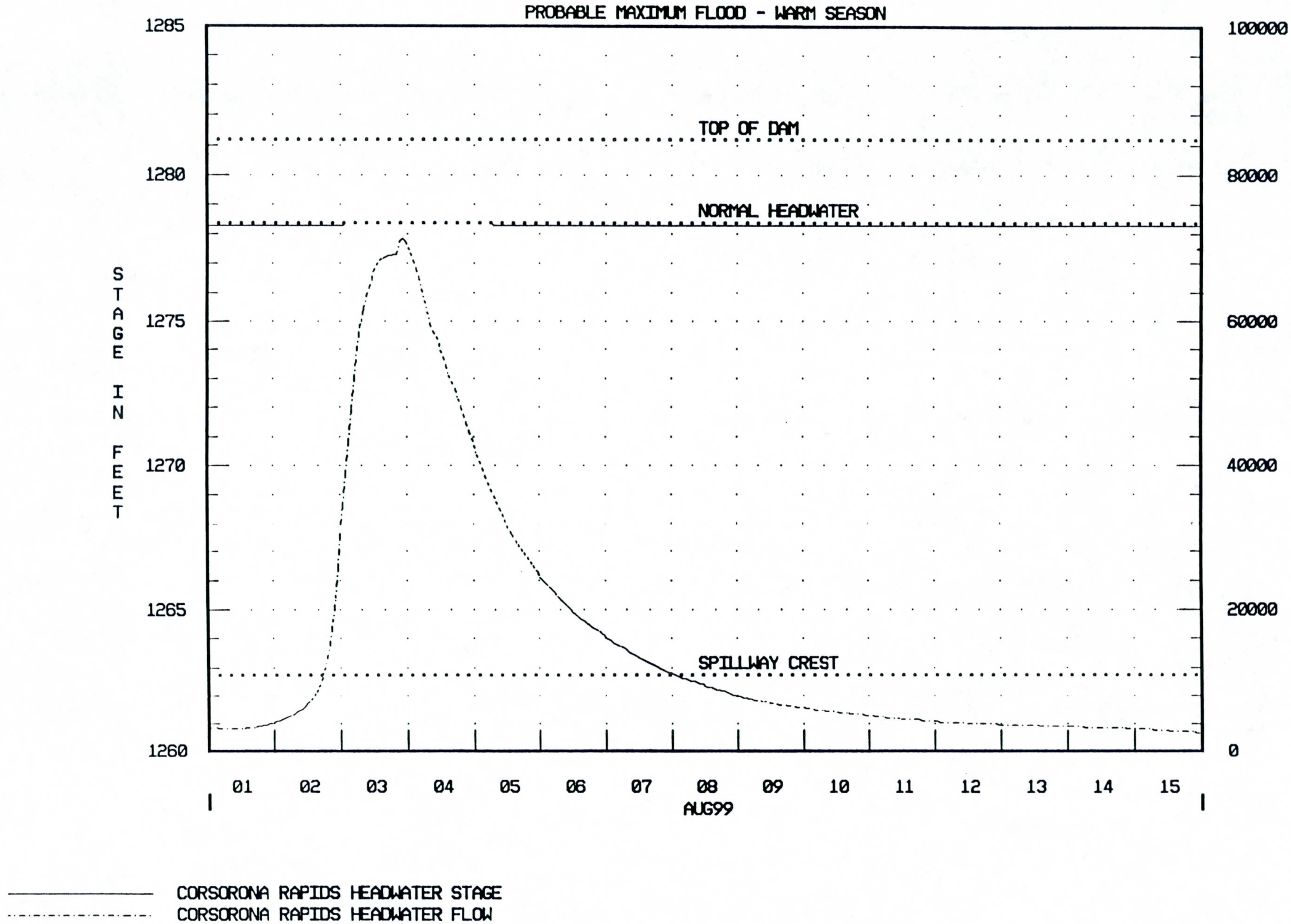
  - for April, 3-day sequence is 61°, 67° and 60° F and the remaining 7 days at 54° F

- degree-day method was used to estimate snowmelt

PMF HYDROGRAPHS



PROBABLE MAXIMUM FLOOD - COOL SEASON
STAGE IN FEET
FLOW IN CFS
1285
1280
1275
1270
1265
1260
100000
80000
60000
40000
20000
0
TOP OF DAM
NORMAL HEADWATER
SPILLWAY CREST
01 02 03 04 05 06 07 08 09 10 11 12 13 14 15
AUG99
CORSORONA RAPIDS HEADWATER STAGE
CORSORONA RAPIDS HEADWATER FLOW



PROBABLE MAXIMUM FLOOD - WARM SEASON
1285
1280
1275
1270
1265
1260
100000
80000
60000
40000
20000
0
STAGE IN FEET
FLOW IN CFS
TOP OF DAM
NORMAL HEADWATER
SPILLWAY CREST
01 02 03 04 05 06 07 08 09 10 11 12 13 14 15
AUG99
CORSORONA RAPIDS HEADWATER STAGE
CORSORONA RAPIDS HEADWATER FLOW

SENSITIVITY ANALYSIS

- No sensitivity analysis was performed
- used worst-case scenario for temperature sequence for snowmelt computation
- soils were assumed to be saturated at the inception of PMF
- unit hydrograph parameters in the regional study were developed from large floods - most of which having overbank flows

RESERVOIR FLOOD ROUTING

- starting reservoir water level at annual maximum normal operating pool of 1,278.5 feet

- no significant attenuation of peak flow because the reservoir is small

- outflow PMFs equal to inflow PMFs

  - Warm-season outflow PMF is 69,100 cfs

  - Cold-season outflow PMF is 59,700 cfs

Example 3
Corsorona Rapids Hydroelectric Project
Probable Maximum Flood

June 1994

Summary

This study was performed to estimate the Probable Maximum Flood (PMF) at the Corsorona Rapids Hydroelectric Project, Federal Energy Regulatory Commission (FERC) Project No. CCCC. The project is located on the Blue River in Anystate. The PMF study was conducted in accordance with Chapter VIII of the FERC *Engineering Guidelines for the Evaluation of Hydropower Projects* (Reference 1). The peak PMF inflow to the project is approximately 69,100 cubic feet per second (cfs), and peak outflow is also approximately 69,100 cfs. The peak stage at the project during the PMF would be 1,278.5 feet, which is the annual maximum normal operating level. The PMF is less than the spillway capacity of 88,000 cfs; therefore, no further hazard studies are needed.

I. Project Description

A. Project Data

Data on project structures and operations were obtained from the 1989 CFR Part 12 *Report on Inspection* for the Corsorona Rapids project (Reference 2). All elevations given in this report are referenced to the National Geodetic Vertical Datum (NGVD).

The Corsorona Rapids Hydroelectric Project is on the Blue River in northern Edwards County, Anystate. From left to right looking downstream, the project structures consist of a 195-foot concrete overflow dam; a tainter gate section with twelve 24-foot-wide by 18-foot-high gates; a concrete and masonry powerhouse; and a 40-foot concrete gravity dam. The maximum height of the dam above the riverbed is 44 feet. There are no earth dikes associated with the project. Annual maximum normal headwater level is 1,278.5 feet, and normal tailwater is 1,242 feet. The total spillway capacity of the dam is 88,000 cfs when the pool is at the top of the right gravity dam (elevation 1,282 feet). The reservoir surface area is 190 acres and the total storage volume at the maximum normal operating level is 3,600 acre-feet.

The project is operated in a run-of river mode, with an operating level of 1,278.5 feet and tolerance of ±0.5 foot. The project is visited twice daily by an operator. The hydroelectric units are controlled, and the headwater and tailwater levels monitored, from the licensee's control center in Edwards City. The spill gates are operated by two moveable electric hoists. A backup generator is kept on-site to power the hoists in the event of an electrical failure.

B. Basin Hydrologic Data

The project is located at a drainage area of approximately 2,721 square miles. The Blue River flows from north to south, a distance of approximately 95 miles from the headwaters of the basin to the Corsorona Rapids project. The northern portion of the basin contains many small lakes and wetlands and is mostly forested. The southern one-third of the basin is used more intensively for agriculture than the northern part, but is still 50 percent or more forested (Reference 3). The basin relief is moderate, with elevations ranging from 1,278.5 feet at the project to 1,859 feet at the upper basin divide.

There is a U.S. Geological Survey (USGS) stream gage (No. 60982200, Blue River near Mercy Creek) approximately 7 miles downstream of the project at a drainage area of 2,823 square miles. The eight largest floods recorded at this gage are shown in Table 1.

TABLE 1
Largest Floods Recorded at Blue River Near Mercy Creek Stream Gage

Date of Flood	Peak Flow (cfs)
April 23, 1963	32,000
September 29, 1943	29,900
March 21, 1948	29,300
May 3, 1985	28,400
May 1, 1977	27,500
October 15, 1950	25,500
October 11, 1957	25,500
April 27, 1958	24,900

Many major historical floods have occurred in mid- to late spring. Most of these were caused by extreme precipitation events combined with either snowmelt or saturated ground due to earlier snowmelt. During a normal spring, however, flooding is moderated by storage in the Blue River Hydropower Cooperative reservoirs in the basin headwaters (see *Section I.C.*).

Hourly recording rain gages are located at Badger Butte Dam and Columbus, and there are nonrecording rain gages at Hyannis, Rys Dam, and Eider Lake. Daily maximum and minimum temperatures are collected at these three stations, as well. Daily snowpack data are collected at the Hyannis and Columbus stations.

A basin map showing the project location, major tributaries, stream gages, and weather stations is shown in Exhibit 1.

C. Upstream Dams

In the headwaters of the Blue River basin, a drainage area of approximately 1,300 square miles is controlled by two storage reservoirs owned by the Blue River Hydropower Cooperative (BRHC). All hydropower project owners on the river are members of the cooperative, which operates the reservoirs for the benefit of power generation throughout the system. In addition to the storage reservoirs, there are seven other hydroelectric dams on the Blue River and its tributaries upstream of the Corsorona Rapids project. Table 2 lists upstream storage reservoirs and other dams.

TABLE 2
Upstream Dams and Reservoirs

Name of Dam	Owner	River	Drainage Area (sq. mi.)	Storage (acre-ft.)
Rys Dam	BRHC	Blue River	753	63,600
Frenchman Lake Dam	City of Victoria	Blue River	878	21,500
Elbow Rapids	Rivers of the North Corporation (RONCO)	Blue River	1,160	7,500
Badger Butte Dam	Western Hydro Co.	Blue River	1,320	14,000
Upper Sister Dam	BRHC	Sister River	544	38,000
Lower Sister Dam	RONCO	Sister River	553	2,200
Hyannis Dam	Western Hydro Co.	Blue River	2,030	19,000
Barnum Dam	Big Top Paper Co.	Blue River	2,250	6,700
Bailey Dam	Big Top Paper Co.	Blue River	2,270	3,200

The locations of the upstream dams and reservoirs are shown in Exhibit 1.

D. Field Visit

The project hydrologists familiarized themselves with the basin by an aerial and ground reconnaissance on September 11 and 12, 1993. The project itself was visited on September 11, 1993. The operator was interviewed to confirm project operation during flood conditions. All of the spill gates are tested annually and are in good working condition.

Information on the project works, storage capacity, and operation of upstream projects was obtained through telephone contacts with project owners and review of previous consultants' inspection reports.

E. Previous Studies

No previous PMF studies for the Corsorona Rapids project exist. However, there are two previous PMF studies on the Blue River in the basin above the Corsorona Rapids project. The first, an estimate of the PMF at the Badger Butte Dam, was prepared by Horris-Mampton Engineering in 1981 (Reference 4). A PMF study for the Rys Dam was also conducted in 1984 by the firm of Spotsmer & Gath (Reference 5). However, the only data taken directly from either of these two studies for this analysis were spillway rating curve data for the Badger Butte and Rys dams. The Horris-Mampton Engineering PMF analysis predicted a PMF of 45,000 cfs at the Badger Butte Dam. The Spotsmer & Gath study estimated a PMF of 16,700 cfs at Rys Dam.

Other studies were used in estimating unit hydrograph parameters and developing the river hydraulic model. These include a basinwide study by the Anystate Geological Survey (AGS), in which regulated flows measured at various gages were back-routed through the BRHC reservoirs to estimate flows that would occur for unregulated conditions (Reference 6). Also, flood routing models used in flood insurance studies for Blue Lake, Lenapee, Thomas, and Forest Counties were obtained from the Federal Emergency Management Agency (FEMA). The river cross sections used in those studies were adapted for the flood routing model used in this one.

II. Watershed Model and Subdivision

A. Watershed Model Methodology

Hydrologic simulation of runoff from the Corsorona Rapids watershed was performed with the U.S. Army Corps of Engineers (COE) HEC-1 computer model. Since there is significant floodplain storage upstream of the Corsorona Rapids project, potentially attenuating the PMF, the COE's UNET dynamic routing model was also used to route the flood from the mouths of tributary basins to the project. Input files for the HEC-1 and UNET models are shown in Exhibits 2 and 3, respectively.

In addition to the HEC-1 and UNET models, a program based on the COE's HMR52 program was used to calculate the Probable Maximum Storm (PMS) distribution. The program (called TSPMP, or Tri-State PMP) was developed in 1991 at the University of Otherstate. Since it is not in general circulation at this time, an executable copy of the

TSPMP program is included in Exhibit 4 on diskette.[1] The three models were linked in a job file so that output from one program was written directly into the input file of the next.

B. Subbasin Definition

The 2,721-square-mile watershed was divided into eight subbasins ranging in size from 136 square miles to 753 square miles. The subbasins were selected to capture the effects of storage in the BRHC reservoirs and differences in subbasin timing, soils, and topography. Because the UNET model accepts a distributed lateral inflow along a reach, in some cases a dam can be located in the middle of a UNET reach and need not be the downstream end of a subbasin.

The eight subbasins modeled are listed in Table 3.

TABLE 3
Subbasins Used to Model PMF

Subbasin No.	Major Tributaries	Drainage Area (sq. mi.)
1	Blue River (above Rys)	753
2	Everett River	265
3	Blue River (Rys to Lake Somo)	313
4	Sister River	554
5	Wildrice River	136
6	Beef River	168
7	Cadillac River	295
8	Lenapee River	237

The eight subbasins used in the HEC-1 model are mapped in Exhibit 5.

[1] Executable code on diskette should be included when a program is not in general use. No diskette is provided with this example.

C. Channel Routing Method

The COE UNET model was used to route flows through the 81-mile reach of river from the Rys Dam to the project. Cross-section data used in the UNET model were provided by the FEMA from input files used in the Blue Lake, Lenapee, Thomas, and Forest County Flood Insurance Studies. Information on dams and reservoirs in the routing reach was obtained from the BRHC and independent consultants' safety inspection reports submitted to the FERC (References 4, 7, 8, 9, 10, and 11).

III. Historic Flood Records

A. Stream Gages

Stream gages in the basin and their drainage areas and periods of record are listed in Table 4.

TABLE 4
Blue River at Corsorona Rapids
Stream Gages In and Near Blue River Basin

Gage Name and Number	Drainage Area (sq. mi.)	Period of Record
Blue River near Mercy Creek (No. 60982200)[1]	2,821	1938–1988
Sister River near Hyannis (No. 60873330)	545	1930–1974
Blue River near Rys (No. 60880000)	757	1936–present
Beef River at Beef Rapids (No. 60845600)	82	1979–present

[1] *Outside basin, 7 miles downstream of Corsorona Rapids project.*

B. Historic Floods

Table 5 lists the floods of record at each gage.

TABLE 5
Historic Floods in Blue River Basin

Gage Name and Number	Peak Flow of Record (cfs)	Date of Peak Flow
Blue River near Mercy Creek (No. 60982200)	32,000	4/23/63
Sister River near Hyannis (No. 60873330)	5,200	10/5/84
Blue River near Rys (No. 60880000)	6,000	4/11/63
Beef River at Beef Rapids (No. 60845600)	4,200	6/3/87

Only three of the eight subbasins used in the HEC-1 model were gaged. Since subdivision was necessary to account for storage in the BRHC reservoirs upstream from the project, unit hydrograph parameters were required for the remaining five subbasins. No gage data were available for these subbasins. Therefore, to develop unit hydrograph parameters for the ungaged subbasins, a regional study was conducted as described in the following section.

IV. Unit Hydrograph Development

A. Discussion of Approach and Tasks

The Blue River basin is considered "ungaged" for the purposes of unit hydrograph analyses, because only three of the eight subbasins have adequate hydrograph data for calibration of unit hydrograph parameters. Since these three basins vary considerably in hydrologic characteristics such as storage and forest cover, they were not considered to be adequate sources for unit hydrograph parameters for the rest of the basin. Therefore, a regional study was conducted as described in the following paragraphs and Appendix A. Parameters developed from the regional study were also applied to one of the gaged subbasins, for which the flow data were imprecise and did not yield a reliable calibration.

B. Existing Studies

Inquiries for existing unit hydrograph studies were made to the COE, the Anystate Water Regulation Department (AWRD), the USGS, and the National Weather Service. The only study identified in this search was a unit hydrograph study done by the AWRD at the Landro Levee, 23 miles downstream of the Corsorona Rapids project. This study was done to estimate the 100-year and 500-year flood hydrograph at the proposed levee location. However, the study does not provide information relevant to smaller subbasins and is incompletely documented.

C. Regional Analysis

A regional analysis of unit hydrograph parameters was conducted to obtain equations relating physical characteristics of a drainage basin to the Clark unit hydrograph parameters T_c (time of concentration) and R. The regional analysis, documented fully in Appendix A, considered eight gaged, unregulated basins. Three of these—the upper Blue, the Sister, and the Beef Rivers—are within the Corsorona Rapids drainage basin. Four of the remaining five are tributaries to the Blue River below the project. The eighth basin is in another drainage but is adjacent to the Blue River drainage.

The eight basins used in the analysis were selected for having drainage areas, topography, climate, and geological characteristics similar to the Blue River subbasins. Although the Guidelines recommend that 10 or more basins be included in a regional study (Reference 1, *Section 8-9.2*), these eight were the only basins in the region that could be considered similar to the study basin. To the south and east of the eight gaged basins selected for analysis, intensive agriculture replaces the forest and less intensive land use of the Blue River region. To the north and west, topography becomes considerably more mountainous and the soils thinner.

(Note: The Engineering Guidelines (Reference 1, Section 8-9.2) recommend the use of at least 10 gaged basins in a regional study. In this case, the use of fewer gaged basins is justified by the physical limits of the hydrologic region.)

The eight gaged basins and floods used in the analysis are summarized in Appendix A.

A time-area curve was constructed for each gaged basin from 7.5-minute quadrangle maps. The HEC-1 model was used to optimize Clark parameters for one to three floods on each basin, with an additional historic flood being used for verification.

Finally, the calibrated Clark parameters T_c and R for all eight basins were regressed against their physical parameters, such as drainage area, channel length, slope, storage, and soil permeability. Due to the small sample size, each final equation was limited to two independent variables. The final equations used to estimate T_c and R are:

$$T_c = 0.19L^{0.75} F^{0.58}$$

$$R = 1.95L^{0.44} ST^{0.50}$$

where:

L = Length of the longest flow path in miles
F = Percent of basin covered by forest
ST = Percent of basin covered by lake and wetland storage.

Appendix A gives details of the calibration and regression analysis for each gaged basin.

The regression equations were applied to each of the eight subbasins in the watershed model, including those with gages. Of the three gaged basins in the Corsorona Rapids basin, only one (the upper Blue River) is gaged at the downstream end of the subbasin defined for the model study (the Rys Dam gage). Therefore, only the upper Blue River could potentially be analyzed using gage calibrations directly; all of the other basins would require some transfer or synthesis of hydrograph parameters.

Of the eight gaged basins analyzed in the regional study, the calibrated parameters for the Blue and Sister Rivers were considered to be the most uncertain. Since flow data for these two gages were daily only, some subjectivity was unavoidable in selecting the "best" parameters for these basins. To minimize this subjectivity in the final PMF modeling, <u>all</u> subbasin unit hydrograph parameters were estimated from the regression equations. Table 6 summarizes the estimated Clark parameters for each subbasin. (Note that the Sister and Beef Rivers, as listed in Table 6, are defined at the mouth of the river instead of at the gage site and therefore have larger watersheds than the gaged basins discussed in Appendix A.)

TABLE 6
Estimated Clark Unit Hydrograph Parameters for Model Subbasins

No.	Subbasin Name	Independent Variables			Dependent Variables	
		L (mi.)	ST (%)	F (%)	T_c (hrs.)	R (hrs.)
1	Upper Blue River	58	28	64	43	61
2	Everett River	34	38	51	25	55
3	Blue River	58	23	62	42	55
4	Sister River	70	28	65	49	66
5	Wildrice River	26	22	68	24	38
6	Beef River	34	14	74	31	34
7	Cadillac River	22	12	72	22	26
8	Lenapee River	49	11	64	37	35

V. Unit Hydrograph Verification

All but one set of unit hydrograph parameters calibrated in the regional study were verified against another flood on the same basin, as discussed in Appendix A. The unit hydrograph parameters calibrated for the eight gaged basins in the regional analysis are also compared to those predicted by the regression equations in Appendix A. Since none of the subbasins in the study basin are gaged, the only option other than the use of regional parameters is the use of synthetic equations (Reference 1, *Section 8-9*). The regression equations, developed from similar basins in the immediate region, are preferable to synthetic equations for estimating unit hydrograph parameters in the Corsorona Rapids basin. This approach is consistent with the FERC Guidelines (Reference 1, *Section 8-9*).

Cold-Season Considerations—None of the calibrated or verified floods had a significant snowmelt component, because snowpack data are available only in one-day intervals and only as depth of snow (not water equivalent). Therefore, there is no reliable way to estimate the water equivalent melted, either on a daily or an hourly basis. Furthermore, almost all spring flood hydrographs are long and flat and have no distinct peak.

As a rough check on the validity of the estimated unit hydrograph parameters for snowmelt floods, T_c was estimated for two snowmelt events at the Sister River gage and one at the Beef River gage. Each of these events resulted from a one-day rise in temperatures with snowpack present. Nighttime temperatures before and after each day's melt were below freezing. The event on the Beef River also included intermittent precipitation throughout the day. The hydrograph peaks resulting from these events were small—less than two times the antecedent flow in the river. The end of effective rainfall and/or snowmelt was assumed to be 6:00 p.m. on the day of the melt (approximately the time of sunset in the late winter months). The end of direct runoff was estimated by plotting the logarithms of the hydrograph ordinates against time and identifying the beginning of the straight-line recession (Reference 1, *Section 8-8.4*). The resulting estimates of T_c associated with snowmelt are summarized in Table 7.

TABLE 7
Estimates of T_c for Snowmelt Floods

River and Date	Starting Flow (cfs)	Peak Flow (cfs)	Est. End of Effective Snowmelt	Est. End of Runoff	Est. T_c from Snowmelt Flood (hrs.)	Regression T_c (hrs.)
Sister River 4/25/56	1,340	1,990	1800 hrs. April 23	0300 hrs. April 26	57	47
Sister River 3/11/89	1,080	1,520	1800 hrs. March 9	2100 hrs. March 11	51	47
Beef River 4/1/64	450	710	1800 hrs. March 31	1100 hrs. April 1	17	20

These estimates show that the T_c estimated from nonsnowmelt floods is reasonable when applied to snowmelt floods. If anything, T_c is generally longer for snowmelt floods. Since the project's storage volume is relatively small, it is expected that peak flow rather than volume will control the PMF at the project. Using the warm-season T_c for cool-season floods is slightly conservative, because it simulates a higher peak for the same volume of runoff than a longer cold-season T_c.

VI. Probable Maximum Storm

A. Probable Maximum Precipitation Data

Probable Maximum Precipitation (PMP) data for the Corsorona Rapids basin were obtained from the TSPMP study (Reference 12), which supersedes HMR52 for Anystate, Otherstate, and Thirdstate. The study provides maps of the 6- through 72-hour PMP for storm areas from 100 to 10,000 square miles and for warm and cool seasons. The depth-area-duration values for the centroid of the entire Corsorona Rapids basin were selected from these maps. The depth-area-duration relationships for warm and cool seasons are shown in Tables 8 and 9, and depth-area-duration curves are plotted in Exhibit 6.

TABLE 8
Probable Maximum Precipitation
Depth-Area-Duration Values (Warm Season)

Area (sq. mi.)	6-hour PMP (in.)	12-hour PMP (in.)	24-hour PMP (in.)	48-hour PMP (in.)	72-hour PMP (in.)
100	13.9	16.6	18.8	20.7	21.9
200	13.2	15.9	18.1	20.0	21.2
500	11.9	14.4	16.6	18.5	19.7
1,000	10.4	12.8	15.0	16.9	18.1
5,000	6.9	8.8	10.5	12.4	13.6
10,000	5.3	7.0	8.7	10.6	11.8

TABLE 9
Probable Maximum Precipitation
Depth-Area-Duration Values (Cool Season)

Area (sq. mi.)	6-hour PMP (in.)	12-hour PMP (in.)	24-hour PMP (in.)	48-hour PMP (in.)	72-hour PMP (in.)
100	5.3	6.6	7.8	9.0	10.2
200	5.2	6.5	7.7	8.9	10.1
500	5.0	6.3	7.5	8.7	9.9
1,000	4.7	6.0	7.2	8.4	9.6
5,000	3.8	5.1	6.3	7.5	8.7
10,000	3.3	4.6	5.8	7.1	8.3

B. Candidate Storms for the PMF

Various PMS sizes, centerings, and orientations were analyzed to identify the storm configuration that would produce the greatest outflow from the Corsorona Rapids project. Analyses were conducted with TSPMP, a program similar to the COE's HMR52 program for optimizing PMS rainfall on a basin, or set of basins. The program requires as input the boundary coordinates for the basin and the regional depth-area-duration relationship and constructs various storms consistent with the depth-area-duration values. The precipitation distribution for each subbasin is computed for a given storm size, orientation, and centering, and the storm configuration that maximizes precipitation over a given basin, or group of basins, is chosen.

For this analysis, the program was run to maximize precipitation for the following:

- Each of the eight subbasins
- The entire basin at Corsorona Rapids
- Subbasins 5, 6, 7 and 8 (the four southernmost subbasins, chosen because they have less forest and less permeable soils than the northern ones)

- Subbasins 2, 3, 5, 6, 7, and 8 (excluding subbasins 1 and 4 because of the storage effects of the BRHC reservoirs).

The storm causing the PMF at the Corsorona Rapids project proved to be a warm-season, 700-square-mile PMP, centered on the Lenapee, Beef, and Cadillac subbasins and oriented 150 degrees from north. The isohyetal pattern for this storm is shown in Exhibit 7.

VII. Loss Rates

A. Discussion of Loss Rate Methodology

The detailed method of estimating losses from rainfall was adopted for this study (Reference 1, *Section 8-10.3.2*). This method involved identifying the infiltration rate of the least permeable layer in each soil unit in each subbasin. Areas occupied by soils with like infiltration rates were then aggregated for each subbasin. The hourly PMS distribution for the subbasin was then applied to the various soil classes and the rainfall excess determined for each hour. Each hourly increment of rainfall excess for each unit was then weighted by subbasin area occupied by the unit and summed over the basin area to produce the hourly increment of runoff for the entire subbasin.

All of the preceding computations were performed in a database calculation. The hourly runoff sequence was then applied, as precipitation, in the subbasin HEC-1 model. Infiltration losses in the HEC-1 model were set to zero, as losses had already been subtracted in the spreadsheet. Initial losses were assumed to be negligible and were set to zero.

B. Warm-Season

Warm-season loss rates were estimated for each soil unit by reference to the STATSGO database for Anystate. This database gives, for each soil association, the permeability range for each layer down to approximately 60 inches. Geographic Information System (GIS) software was used to calculate the subbasin areas occupied by each soil association and assign each soil association a percentage composition by soil unit. For each soil unit, the layer with the minimum average permeability was identified. Infiltration into the soil unit was assumed to be controlled by that minimum rate. Since

STATSGO gives a range of permeabilities for each layer, the representative permeability was defined as the geometric average of the range.

A separate layer of information in the GIS analysis identified lakes and wetlands from a state land cover map (Reference 3). All areas designated "lake" or "wetland" were assumed to be impervious, which is appropriate for lakes and conservative for wetlands. Many wetlands are usually less than saturated and, furthermore, have significant depression storage that must be satisfied before runoff begins. These factors were conservatively ignored in this analysis. Again, this corresponds to an assumption of basin saturation before the onset of the PMS.

The STATSGO database and state geological maps (Reference 13) indicate that the regional bedrock is generally overlain by 10 to 30 feet of soils and other unconsolidated glacial material. Therefore, it is not likely that bedrock near the surface would impede infiltration of precipitation, and the use of the STATSGO infiltration rates is justified.

The distribution of assumed warm-season infiltration rates in each subbasin is summarized in Exhibit 8.

C. Cool Season

The assignment of cool-season loss rates to each soil unit was similar to the warm-season procedure, except that it was assumed that all soils with the potential to be imperviously frozen were indeed impervious. For this study, these soils were identified by the following criteria:

- To be impervious, soils other than sand or loamy sand must occur in the top 24 inches of the soil profile. Twenty-four inches is a maximum expected frost depth, based on analyses by the Anystate Office of Climatology (Reference 14).

- Imperviously frozen soils will not occur in natural or managed forests, where the humus depth is adequate to prevent concrete frosts from forming (Reference 1, *Section 8-10.3.3*).

- All lakes and wetlands are imperviously frozen, whether or not they are forested (Reference 1, *Section 8-10.3.3*).

For this analysis, sands and loamy sands were defined to be those soils with an average infiltration rate of 10 inches per hour or more. The infiltration rates for sands and loamy sands were taken from properties listed for these soils in the STATSGO database.

The distribution of assumed cold-season infiltration rates in each subbasin is summarized in Exhibit 8.

VIII. Coincident Hydrometeorological and Hydrological Conditions for the Probable Maximum Flood

A. Reservoir Level

The Corsorona Rapids project is operated in a run-of-river mode with an operating tolerance of ±0.5 foot. Therefore, the annual maximum normal operating level is 0.5 foot above the target headwater elevation, or 1,278.5 feet.

B. Baseflow

The gage data used to develop the regional study indicate that the initial baseflow is related to the amount of lake and wetland storage in the subbasin, with higher baseflows occurring in basins with greater amounts of storage. An initial baseflow of 1 cfs per square mile was used for the subbasins in the Corsorona Rapids watershed with a storage area of 25 percent or more of the drainage area. For basins with storage areas under 5 percent, such as the Beef River, the baseflow was determined to be 0.5 cfs per square mile. For storage areas between 5 and 25 percent, the initial baseflow was linearly interpolated between 0.5 cfs per square mile and 1 cfs per square mile.

The flow hydrograph data for the gaged basins also indicate that the flow at the start of recession QRCSN is approximately 15 percent of the peak flow for subbasins with storage areas 20 percent or more of the basin area. The QRCSN parameter was found to be 7 percent for basins with storage areas equal to 10 percent or less. For other basins with storage areas between 10 and 20 percent, QRCSN was linearly interpolated.

The value of the recession constant, RTIOR, varied considerably for the observed flow hydrographs. A value of 1.008 (the average over all the hydrographs analyzed) was adopted for all the basins.

C. Snowpack

A frequency analysis of snowpack in March and April was conducted. These months were selected based on the flood records for the basin, which show that spring flooding typically begins in March and subsides by the end of April. Earlier in the season, temperatures rarely rise above freezing. Beyond April, snowpack is nonexistent.

The frequency analysis was conducted by the state climatologist (Reference 14) for snowpacks at the Hyannis weather station at mid-March and mid-April. Water equivalent was not recorded systematically, but data from other climatological stations in the area suggested that a late-winter water equivalent of 10 to 15 percent was common. As suggested in the *Engineering Guidelines* (Reference 1, *Section 8-10.2.1*), the typical water equivalent was doubled to 25 percent. The 100-year snowpack in mid-March is 28 inches snow depth, or 7 inches water equivalent. The 100-year snowpack in mid-April is 11 inches snow depth, or 2.25 inches water equivalent.

D. Snowmelt

An estimate of the highest probable temperature sequence was made for March and April. Daily average temperature data at the Hyannis, Rys Dam, and Eider Lake weather stations were reviewed. The critical period was assumed to be a 3-day period superimposed on the 72-hour PMS. However, a total snowmelt period of 10 days was modeled to coincide with the entire duration of the PMF hydrograph. The critical 3-day temperature sequence for each month was taken to be the maximum recorded 3-day sequence that coincided with rain on at least one day. The remaining 7 days were assumed to maintain an average temperature equal to the ten-year, ten-day high temperature for the month. These data were also obtained from analyses by the state climatologist (Reference 15). The snowmelt temperature sequences for each month are as follows:

March: 3-day sequence: average temperatures of 48, 53, and 47 degrees
Remaining 7-day average temperature: 42 degrees

April: 3-day sequence: average temperatures of 61, 67, and 60 degrees
Remaining 7-day average temperature: 54 degrees

The 10-day temperature sequence for each month was applied to the 100-year snowpack for the month. One of the following two daily melt equations given by Chow (Reference 16) was used for each subbasin, depending on whether the subbasin was mostly forested or mostly open:

(1) *For forested areas*:

$$M = 0.05\ (T_{mean} - 32)$$

and

(2) *For open areas*:

$$M = 0.06\ (t_{mean} - 24)$$

where:

M = daily melt rate (inches)
T_{mean} = daily mean temperature in degrees Fahrenheit

The April high temperatures were found to melt the April snowpack early in the second day. The March high temperatures melted approximately 6.1 inches, less than the 100-year snowpack. The snowmelt flood caused by the March temperatures and snowpack was found to be greater than that caused by the April snowpack-temperature combination. Therefore, March conditions were assumed to coincide with the cool-season PMS.

IX. PMF Hydrographs

A. Inflow PMF Hydrograph

The entire sequence of PMS analysis, HEC-1 modeling of each subbasin, and UNET routing was carried out for each storm optimization. The warm-season PMF at the project is caused by a 700-square-mile storm centered on the Lenapee, Beef, and Cadillac River subbasin group and oriented 150 degrees from north. Input and output files for HEC-1, UNET, and TSPMP are found in Exhibits 2, 3, and 4, respectively.

Since routing through the Corsorona Rapids reservoir was accomplished by dynamic routing in the UNET model, the inflow hydrograph is defined as the hydrograph at the most upstream cross section in the reservoir. The peak of the warm-season PMF inflow hydrograph is 69,100 cfs. The warm-season PMF inflow hydrograph is plotted in Exhibit 9.

The cool-season PMF at the project is caused by a 6,500-square-mile storm centered on the entire basin and oriented 217 degrees from north. The peak of the cool-season inflow PMF hydrograph is 59,700 cfs. The cool-season inflow hydrograph is plotted in Exhibit 10.

B. Sensitivity Analysis

In general, when assumptions were required in this study, they were made to maximize the estimate of the PMF. For example, a worst-case temperature sequence was assumed to coincide with a 100-year snowpack and a PMS. Soils were also conservatively assumed to be saturated, with no initial losses. Finally, unit hydrograph parameters in the regional study were estimated from large floods—most of which were overbank events—and further adjustment for linearity is not needed. Therefore, no sensitivity analyses were conducted.

C. Reservoir PMF Outflow

The PMF was routed through the Corsorona Rapids reservoir using dynamic routing in the UNET model. Due to the reservoir's small size, there would be no significant attenuation of the peak flow. The warm-season PMF outflow hydrograph would have a peak flow of 69,100 cfs, which could be passed at the annual maximum normal operation level of 1,278.5 feet. The cool-season PMF outflow would be 59,700 cfs and would also be passed at headwater elevation 1,278.5 feet.

The warm-season and cool-season outflow hydrographs are essentially equivalent to the inflow hydrographs, which are plotted in Exhibits 9 and 10, respectively.

References

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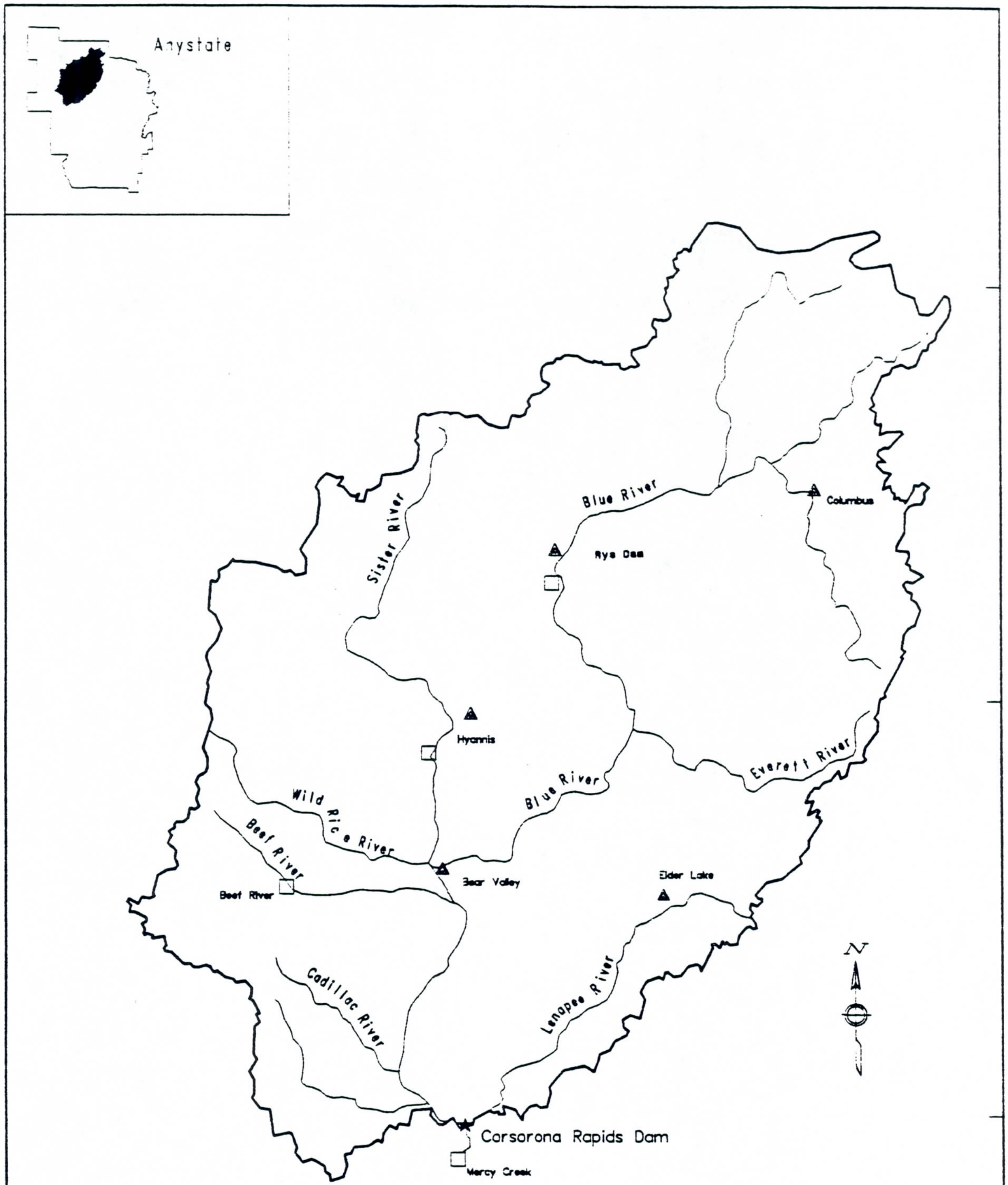
15. Anystate Office of Climatology, 1991. Weather Data for Experimental Farms in Anystate.

16. Chow, V.T., 1964. *Handbook of Applied Hydrology.* McGraw-Hill, New York.

EXHIBIT 1

Basin Map



Corsorona Rapids Stream and Rain Gages

EXHIBIT 2

HEC-1 Input/Output Data

Note: Complete HEC-1 input and output have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

EXHIBIT 3

UNET Input/Output Data

Note: Complete UNET input and output have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output

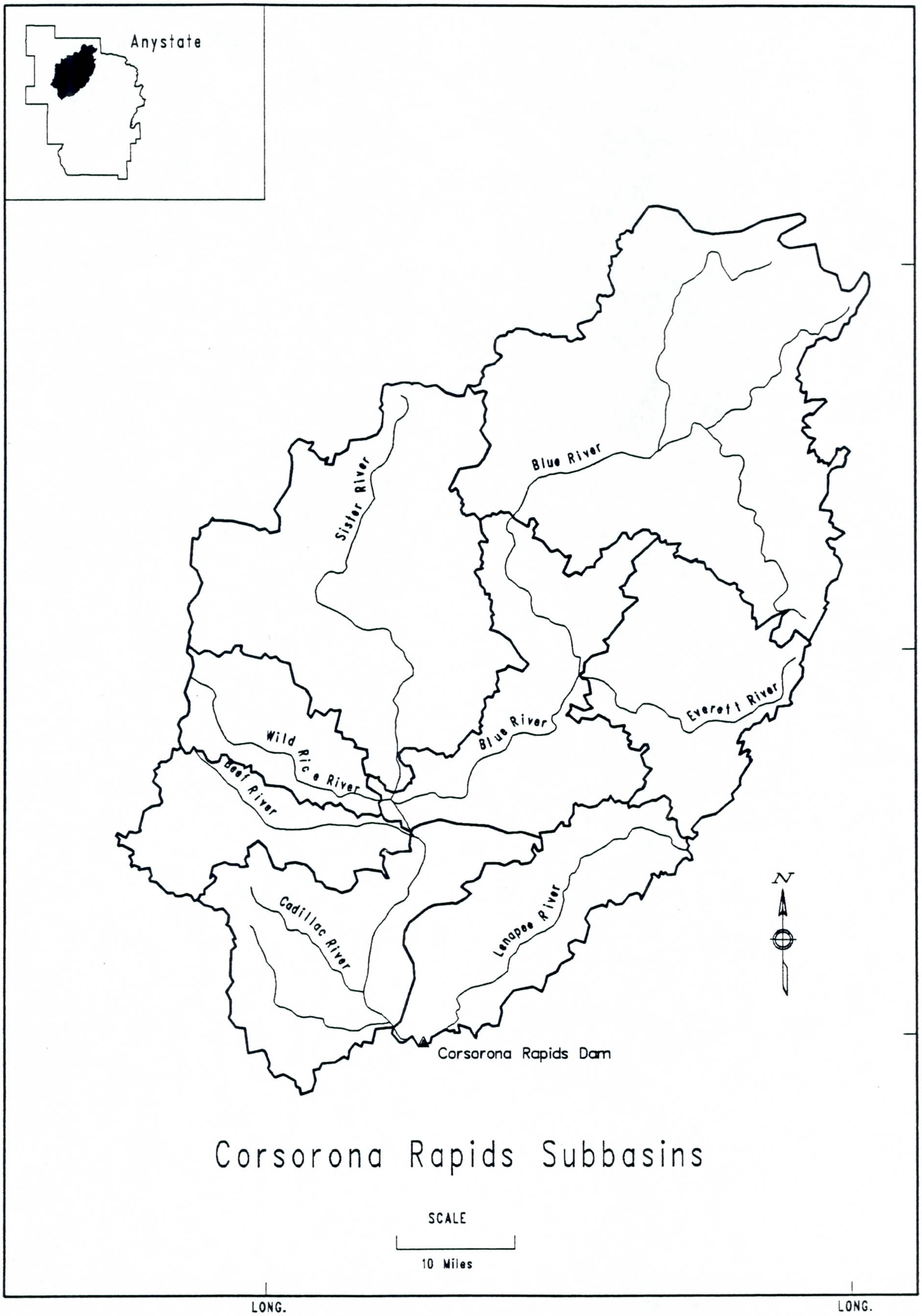
EXHIBIT 4

TSPMP Input and Output

Note: Complete TSPMP input and output have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

EXHIBIT 5

Subbasin Map



Corsorona Rapids Subbasins

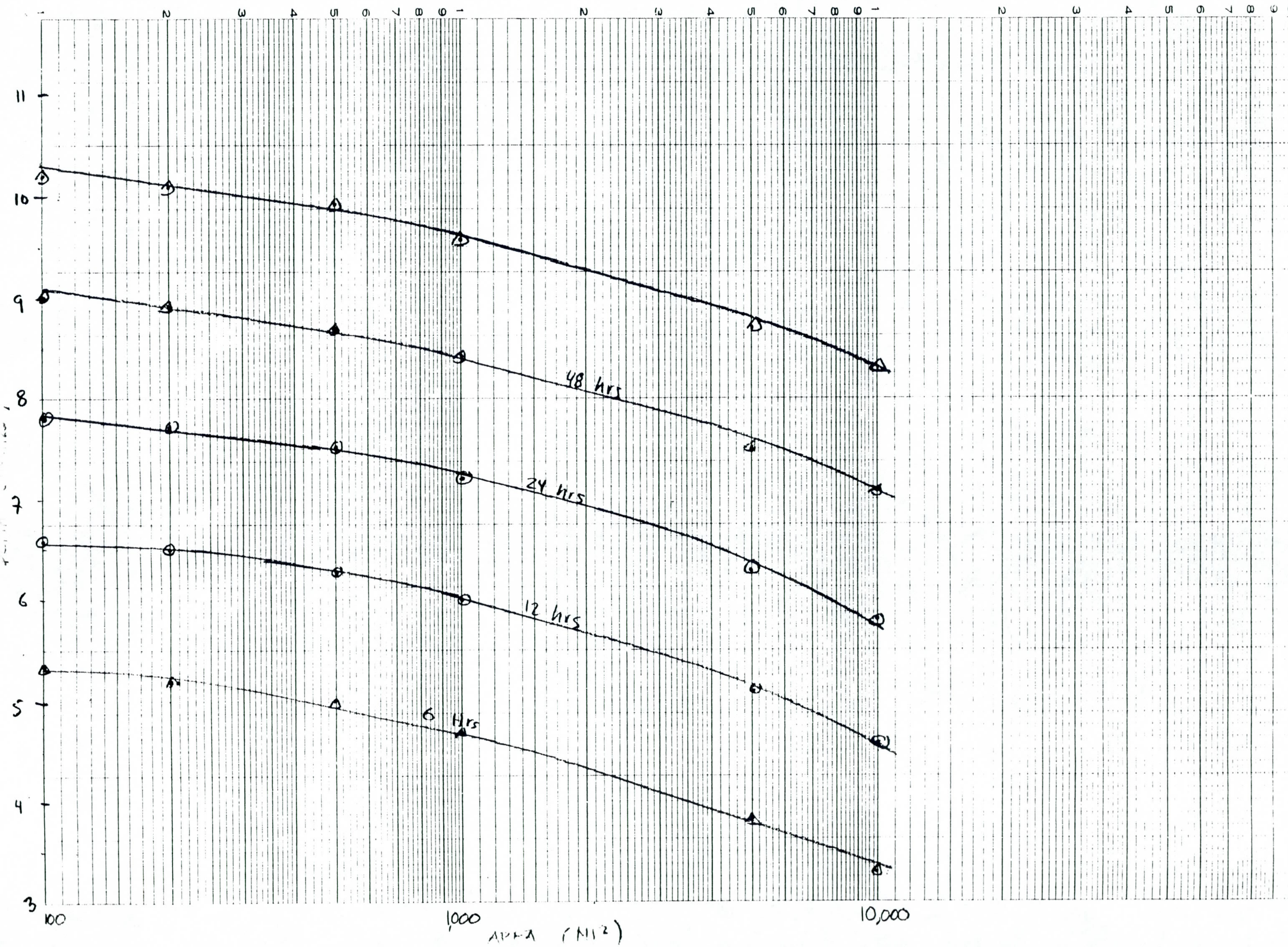
SCALE

10 Miles

EXHIBIT 6

Depth-Area-Duration Curves

BLUE RIVER PMP (COOL SEASON)



NO. 340-L310 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
3 CYCLES X 10 DIVISIONS PER INCH

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MADE IN U.S.A.

BLUE RIVER PMP (WARM SEASON)

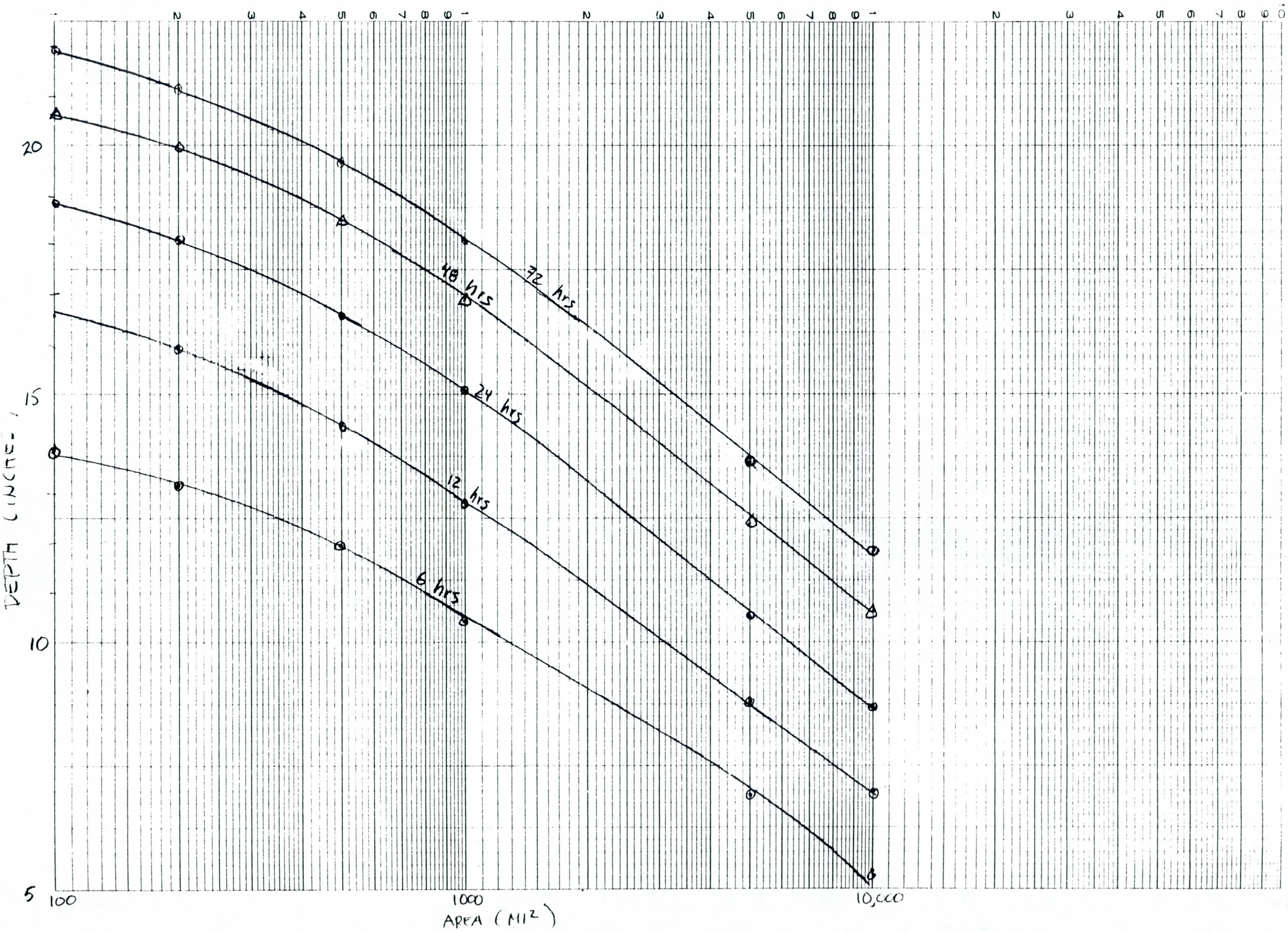
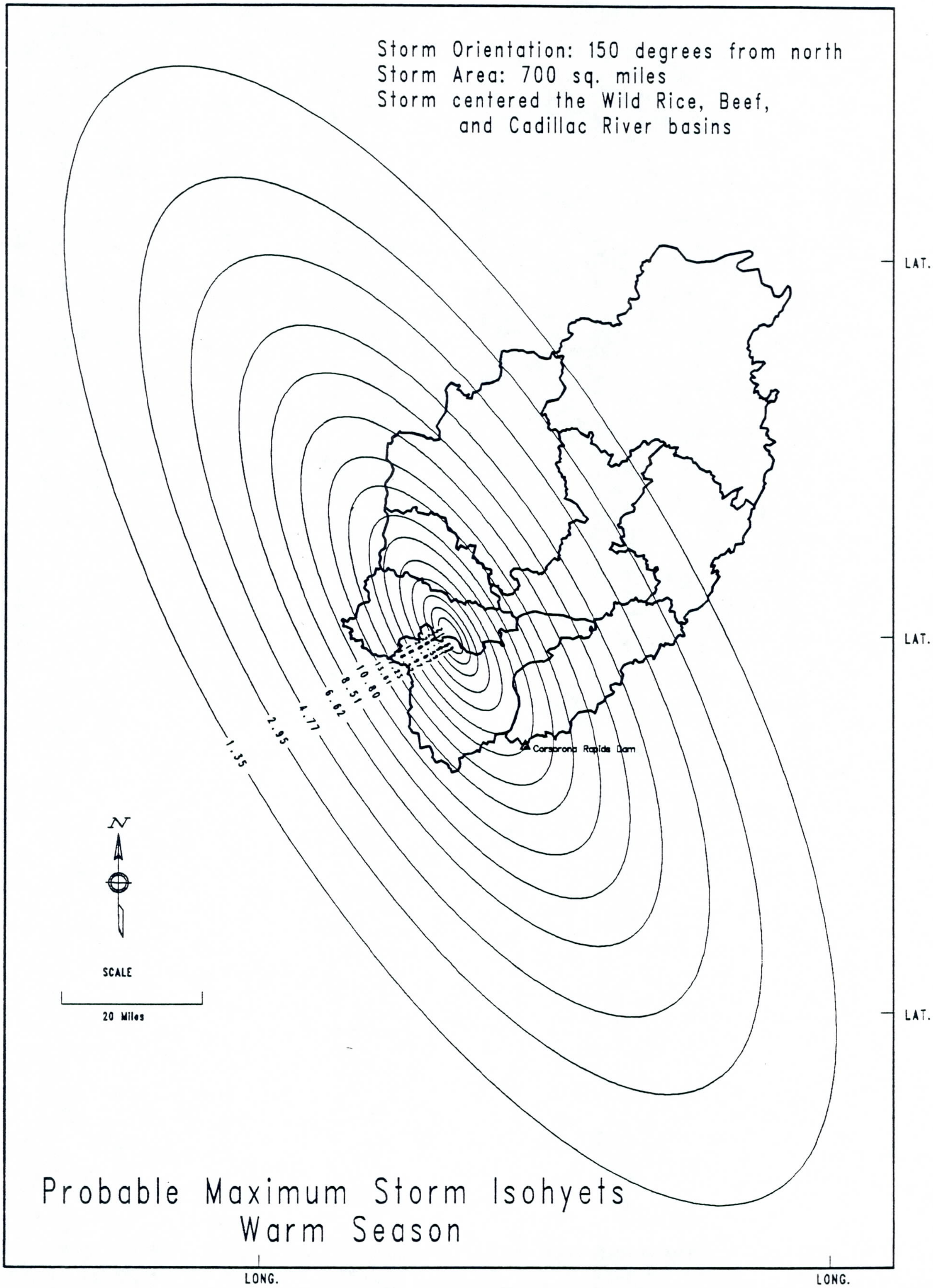


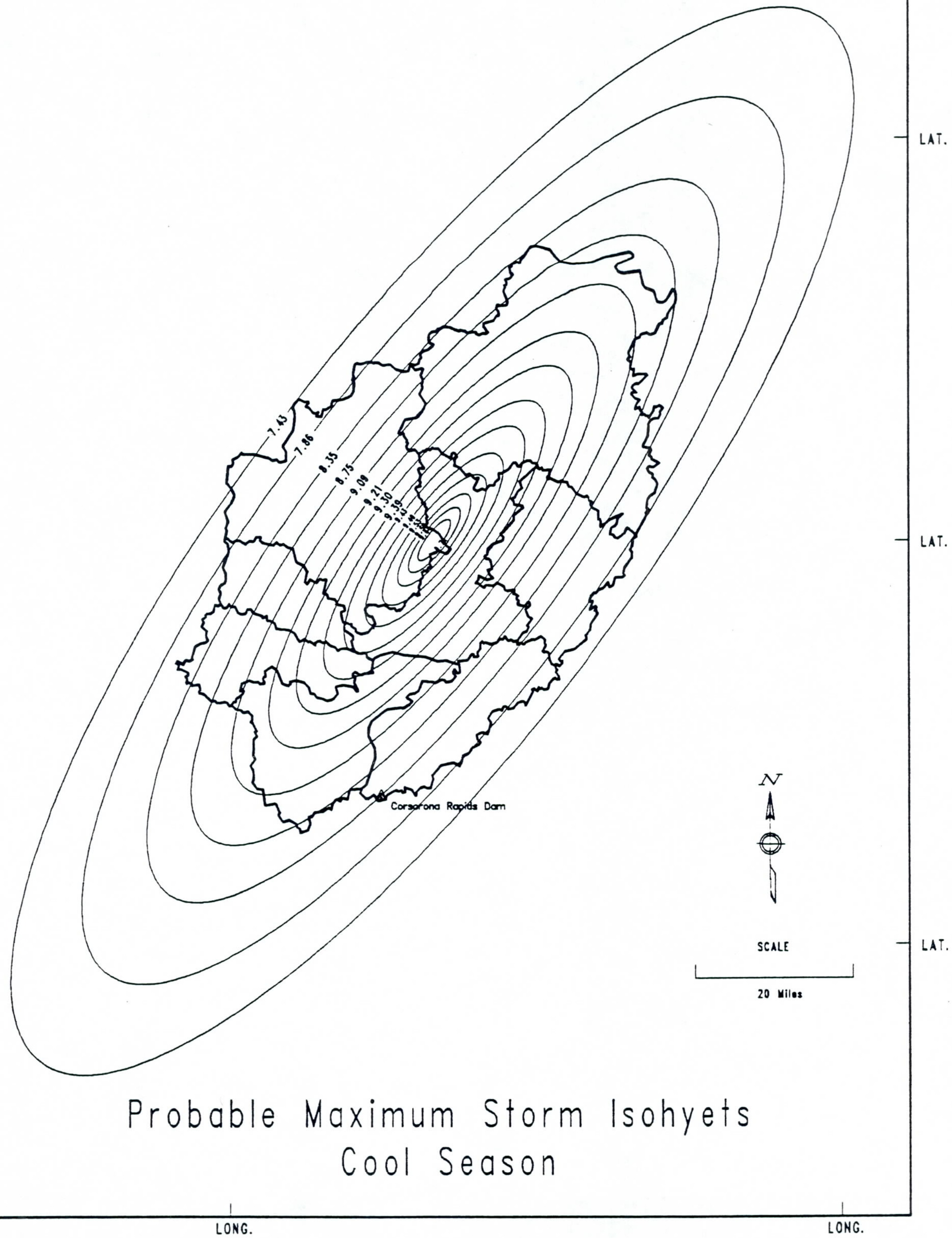
EXHIBIT 7

Probable Maximum Storm Isohyets



Probable Maximum Storm Isohyets
Warm Season

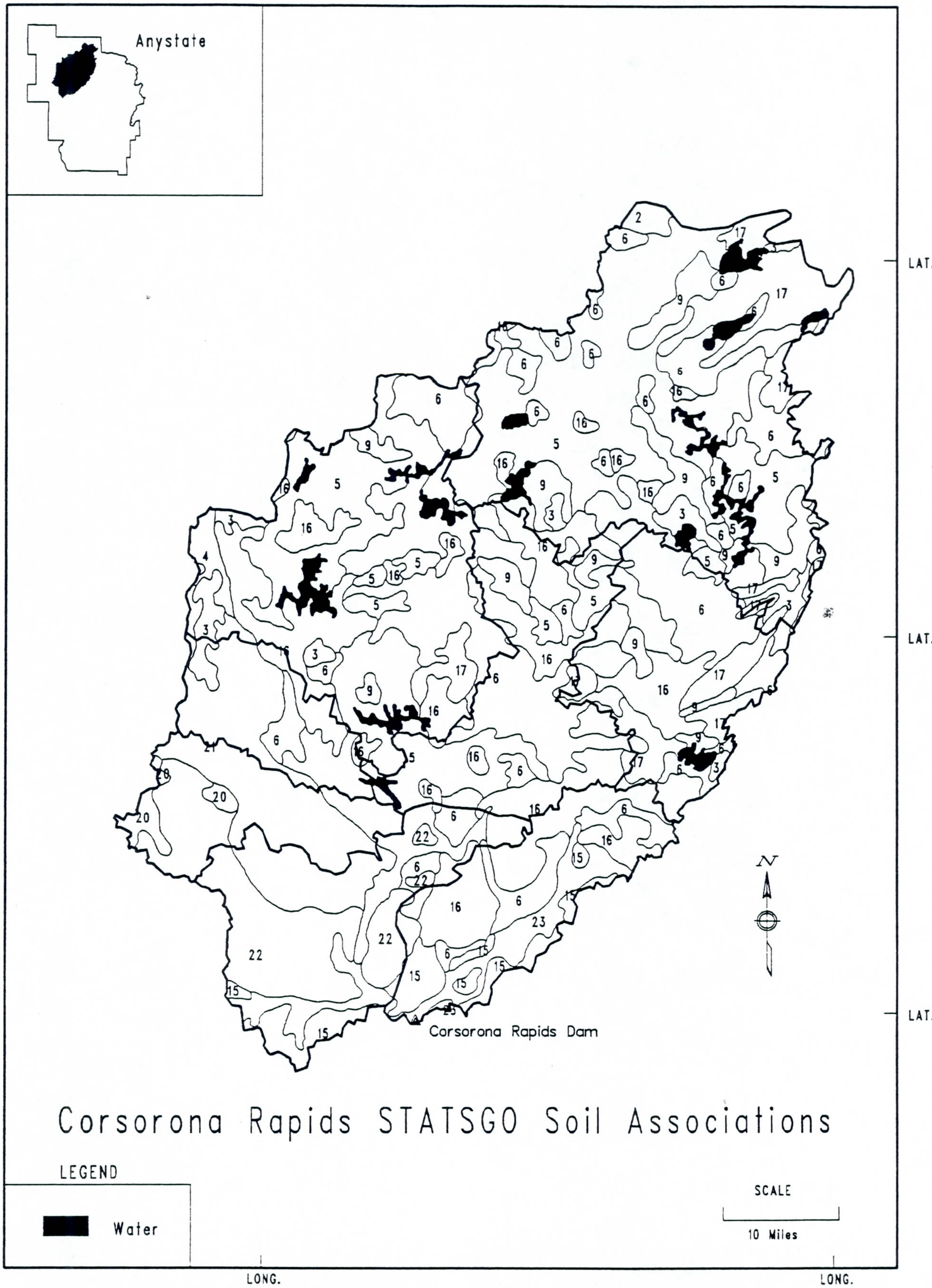
Storm Orientation: 216.7 degrees from north
Storm Area: 6500 sq. miles
Storm centered over whole basin



Probable Maximum Storm Isohyets
Cool Season

EXHIBIT 8

Subbasin Soil Infiltration (STATSGO) Map and Distributions


Anystate
Corsorona Rapids Dam
Corsorona Rapids STATSGO Soil Associations
LEGEND
Water
SCALE
10 Miles
LAT.
LAT.
LAT.
LONG.
LONG.

Map Unit	Sequence	% of Map Unit	60"	24"
2	1	10	0.03	0.03
2	2	19	0.03	0.03
2	3	6	0.03	0.03
2	4	9	1.30	1.30
2	5	4	1.30	1.30
2	6	10	1.10	1.10
2	7	7	1.10	1.10
2	8	6	0.00	0.00
2	9	5	3.10	3.10
2	10	5	3.10	3.10
2	11	4	1.30	1.30
2	12	2	0.40	0.40
2	13	1	0.40	0.40
2	14	1	4.00	4.00
2	15	2	4.00	4.00
2	16	3	1.10	1.10
2	17	3	3.30	3.30
2	18	2	0.13	0.13
2	19	1	0.04	0.04
3	1	17	3.10	3.10
3	2	13	3.10	3.10
3	3	12	1.10	1.10
3	4	12	3.10	3.10
3	5	9	1.30	3.10
3	6	8	0.40	0.40
3	7	7	13.00	13.00
3	8	6	3.30	3.30
3	9	5	4.00	4.00
3	10	4	13.00	13.00
3	11	4	0.40	0.40
3	12	3	13.00	13.00

Map Unit	Sequence	% of Map Unit	60"	24"
4	1	9	0.03	0.03
4	2	22	0.03	0.03
4	3	28	0.03	0.03
4	4	8	4.00	4.00
4	5	8	4.00	4.00
4	6	6	3.10	3.10
4	7	6	3.10	3.10
4	8	5	3.10	3.10
4	9	5	1.30	1.30
4	10	3	1.30	1.30
5	1	5	4.00	4.00
5	2	14	4.00	4.00
5	3	12	4.00	4.00
5	4	4	13.00	13.00
5	5	6	13.00	13.00
5	6	12	13.00	13.00
5	7	6	13.00	13.00
5	8	11	13.00	13.00
5	9	2	3.30	4.00
5	10	4	3.30	4.00
5	11	6	3.30	3.30
5	12	5	3.10	3.10
5	13	4	3.10	3.10
5	14	4	1.30	3.10
5	15	2	3.10	3.10
5	16	2	4.00	4.00
5	17	1	13.00	13.00
6	1	14	3.30	3.30
6	2	11	3.30	3.30
6	3	23	3.30	3.30
6	4	5	4.00	4.00

Map Unit	Sequence	% of Map Unit	60"	24"
6	5	7	4.00	4.00
6	6	10	4.00	4.00
6	7	8	3.30	3.30
6	8	7	3.10	3.10
6	9	5	4.00	4.00
6	10	2	1.30	3.10
6	11	2	1.10	1.10
6	12	2	1.30	1.30
6	13	2	0.40	1.10
6	14	1	3.10	3.10
6	15	1	0.40	0.40
9	1	16	13.00	13.00
9	2	14	13.00	13.00
9	3	14	13.00	13.00
9	4	11	3.10	3.10
9	5	9	3.10	3.10
9	6	7	1.30	3.10
9	7	5	4.00	4.00
9	8	4	3.30	3.30
9	9	4	3.10	3.10
9	10	4	13.00	13.00
9	11	4	13.00	13.00
9	12	3	4.00	4.00
9	13	1	3.30	3.30
9	14	4	0.00	0.00
15	1	36	0.03	0.13
15	2	12	0.13	0.13
15	3	6	0.13	0.13
15	4	3	1.30	1.30
15	5	5	1.30	1.30
15	6	6	1.10	1.10

Map Unit	Sequence	% of Map Unit	60"	24"
15	7	5	0.40	1.10
15	8	5	0.33	0.33
15	9	4	1.10	1.10
15	10	3	0.40	1.10
15	11	3	0.03	1.30
15	12	2	1.30	1.30
15	13	2	1.30	1.30
15	14	2	13.00	13.00
15	15	2	3.10	3.10
15	16	1	1.30	1.30
15	17	1	1.30	1.30
15	18	1	1.10	1.10
15	19	1	13.00	13.00
16	1	10	3.30	4.00
16	2	18	3.30	4.00
16	3	11	3.30	4.00
16	4	8	1.30	1.30
16	5	8	1.30	1.30
16	6	3	1.30	1.30
16	7	6	4.00	4.00
16	8	3	4.00	4.00
16	9	6	4.00	4.00
16	10	2	4.00	4.00
16	11	6	3.10	3.10
16	12	4	1.30	1.30
16	13	4	3.30	3.30
16	14	2	13.00	13.00
16	15	1	13.00	13.00
16	16	2	1.30	3.10
16	17	2	3.10	3.10
16	18	1	1.10	1.10

Map Unit	Sequence	% of Map Unit	60"	24"
16	19	1	3.30	3.30
16	20	1	1.30	1.30
16	21	1	0.04	1.30
17	1	8	0.03	0.03
17	2	10	0.03	0.03
17	3	12	1.30	1.30
17	4	6	0.03	0.03
17	5	4	0.03	0.03
17	6	8	0.40	1.10
17	7	5	3.10	3.10
17	8	9	0.40	0.40
17	9	12	0.40	0.40
17	10	3	1.30	1.30
17	11	2	1.30	1.30
17	12	4	3.30	3.30
17	13	4	0.40	0.40
17	14	3	1.30	1.30
17	15	3	3.10	3.10
17	16	2	3.10	3.10
17	17	2	1.10	1.10
17	18	2	1.10	1.10
17	19	1	0.04	0.04
20	1	1	0.03	1.30
20	2	55	0.03	1.30
20	3	12	4.00	4.00
20	4	4	4.00	4.00
20	5	8	3.30	3.30
20	6	6	3.10	3.10
20	7	5	1.10	1.10
20	8	5	3.10	3.10
20	9	4	1.10	1.10

Map Unit	Sequence	% of Map Unit	60"	24"
21	1	16	0.03	0.13
21	2	18	0.03	1.10
21	3	1	0.40	1.10
21	4	19	0.40	1.10
21	5	10	1.10	1.10
21	6	7	0.13	0.13
21	7	6	3.10	3.10
21	8	3	0.03	0.03
21	9	4	1.30	3.10
21	10	4	3.10	3.10
21	11	3	1.10	1.10
21	12	3	20.00	20.00
21	13	2	3.10	3.10
21	14	2	1.30	1.30
21	15	2	1.30	1.30
22	1	4	0.03	1.30
22	2	22	0.03	1.30
22	3	15	0.03	1.30
22	4	13	1.30	1.30
22	5	3	4.00	4.00
22	6	7	4.00	4.00
22	7	4	0.13	0.13
22	8	3	0.13	0.13
22	9	5	3.30	3.30
22	10	5	0.03	0.13
22	11	4	0.40	1.30
22	12	4	3.10	3.10
22	13	3	1.10	1.10
22	14	3	3.10	3.10
22	15	2	3.10	3.10
22	16	2	1.30	1.30

Map Unit	Sequence	% of Map Unit	60"	24"
22	17	1	0.40	1.10
23	1	33	1.30	1.30
23	2	28	1.30	1.30
23	3	10	1.30	1.30
23	4	5	1.30	1.30
23	5	4	1.30	1.30
23	6	10	1.30	1.30
23	7	6	0.40	1.10
23	8	1	1.10	1.10
23	9	1	3.10	3.10
23	10	1	4.00	4.00
23	11	1	3.10	3.10

EXHIBIT 9

PMF Inflow Hydrograph
Warm Season

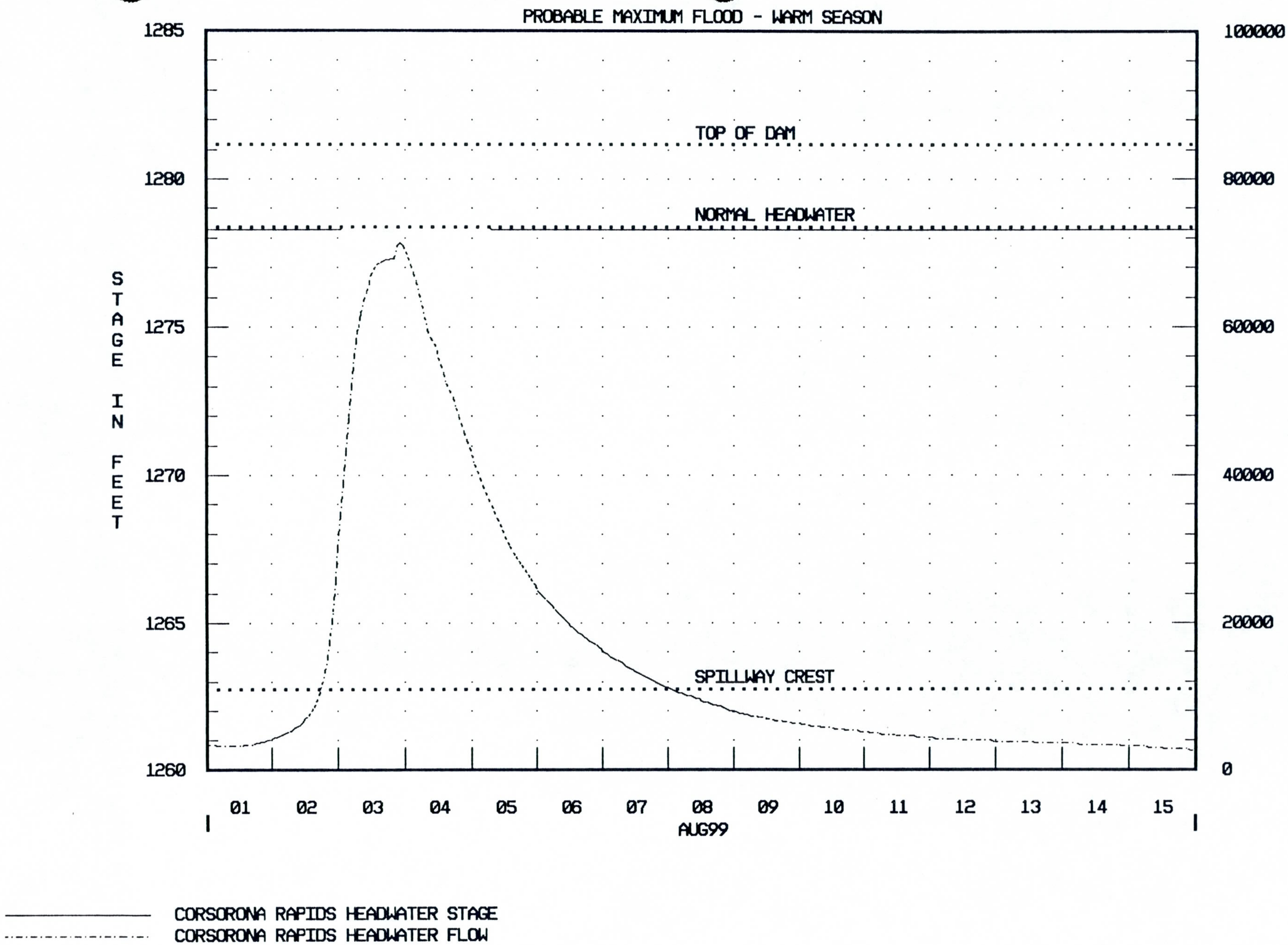
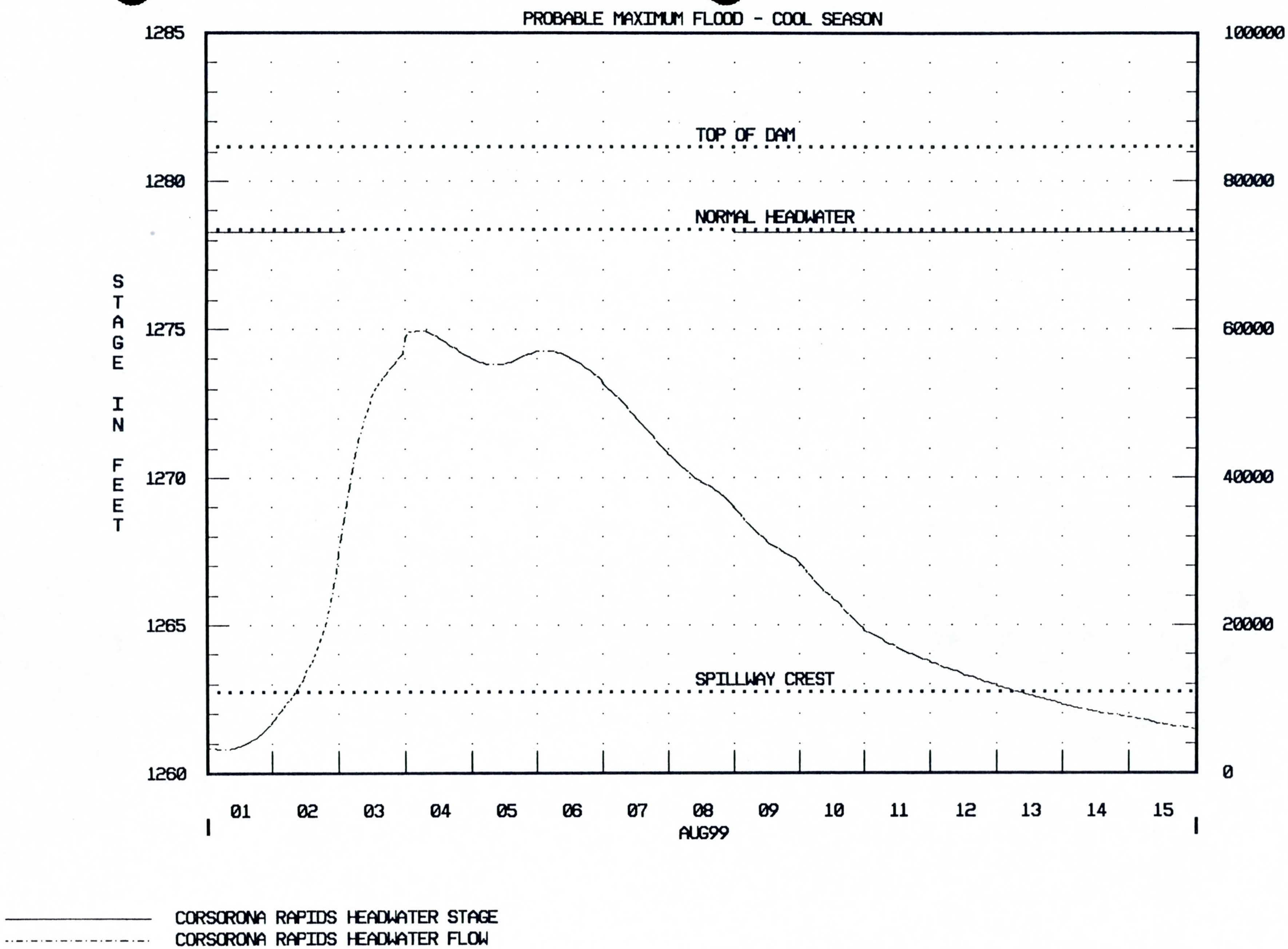

PROBABLE MAXIMUM FLOOD - WARM SEASON
STAGE IN FEET
FLOW IN CFS
1285
1280
1275
1270
1265
1260
100000
80000
60000
40000
20000
0
TOP OF DAM
NORMAL HEADWATER
SPILLWAY CREST
01 02 03 04 05 06 07 08 09 10 11 12 13 14 15
AUG99
CORSORONA RAPIDS HEADWATER STAGE
CORSORONA RAPIDS HEADWATER FLOW

EXHIBIT 10

PMF Inflow Hydrograph
Cool Season



PROBABLE MAXIMUM FLOOD - COOL SEASON
TOP OF DAM
NORMAL HEADWATER
SPILLWAY CREST
STAGE IN FEET
FLOW IN CFS
1285
1280
1275
1270
1265
1260
100000
80000
60000
40000
20000
0
01 02 03 04 05 06 07 08 09 10 11 12 13 14 15
AUG99
CORSORONA RAPIDS HEADWATER STAGE
CORSORONA RAPIDS HEADWATER FLOW

APPENDIX A

Regional Unit Hydrograph Analysis

APPENDIX A

Regional Unit Hydrograph Analysis

A. Basins and Events Used in Analysis

Eight gaged basins near the Corsorona Rapids project drainage basin were selected on the basis of their hydrologic similarity to that basin. The largest floods measured at each gage were investigated to identify those with adequate flow and rainfall data and distinct flood peaks clearly related to a precipitation event. None of the floods thus identified resulted from snowmelt. All large historic floods during the main snowmelt months (March and April) were found to have long, flat hydrographs. These were associated with prolonged and poorly defined snowmelt events and intermittent rainfall throughout the period.

The basins and events analyzed are shown in Table A-1, and locations of each gaged basin used in the analysis are shown in Exhibit A-1.

Historic hydrographs were requested from the U.S. Geological Survey (USGS). Hourly flow data were available for all of the gages except the Blue River gage at Rys and the Sister River near Hyannis. For these two gages, the only original flow data were daily flows recorded below storage reservoirs. However, these have been converted to unregulated flows by backrouting in a USGS report (Reference A-1). The backrouted daily hydrographs were used to estimate unit hydrograph parameters at these two gages.

Historical hydrographs are included in the HEC-1 model optimization runs in Exhibit A-2.

TABLE A-1
Gaged Basins Used in Regional Analysis

Name of Basin	Drainage Area at Gage (sq. mi.)	Period of Record	Flood Dates Used in Analysis
Blue River near Rys (No. 60880000)	757	1936–present	October 1953
			September-October 1969
			June 1990
Sister River near Hyannis (No. 60873330)	545	1930–1974	June 1958
			October 1968
Beef River at Beef Rapids (No. 60845600)	82	1979–present	April 1980
Edwards River near Athens (No. 60896000)	184	1914–1928, 1938–present	April–May 1973
			June 1979
			June 1990
Lost River near Oak Prairie (No. 60967000)	375	1939–present	September 1959
			September 1969
			October 1986
			June 1990
Sweetwater River at Yale (No. 60411400)	224	1937–present	May 1968
			June 1972
			June 1986
Muddy River at Bradley Landing (No. 60499900)	215	1944–present	May 1968
			April-May 1973
			September 1980
Blackbird River near Jersey (No. 60675000)	749	1913–present	June 1968
			May 1973
			September 1980
			October 1986

B. Rainfall Associated with Historic Floods

Rainfall for each historic flood was estimated from hourly and daily rain gages in and near each basin. For each gaged basin, the hourly gage closest to the basin centroid was selected to represent the hourly distribution of rainfall over the basin. The total depth of precipitation for the storm was estimated by area-weighting the storm total precipitation at the nearest daily or hourly gages. Locations of hourly and daily gages used in the analysis are shown in Exhibit A-1.

A summary of input data to all optimization runs (total precipitation, baseflow parameters, peak flow, runoff volume, and optimization statistics) is given on Page 1 of Exhibit A-2.

C. Baseflow Separation

For each historic flood, baseflow parameters used in the HEC-1 model were estimated by plotting the logarithms of the flows against time. The flow at which direct runoff ends and baseflow recession begins (QRCSN in the HEC-1 model) is the flow at the beginning of the straight recession limb on the semilog plot. The baseflow recession constant (input variable RTIOR in the HEC-1 model) is the slope of the recession line on a semilog graph. Baseflow parameters for each historical flood are summarized on Page 1 of Exhibit A-2.

D. Unit Hydrograph Parameter Estimation

For each basin, one large flood was set aside for unit hydrograph verification. The remaining two or three floods were used to estimate Clark unit hydrograph parameters, using the parameter optimization subroutine in the HEC-1 model. The only exception to this was the Beef River, where the short gage record includes only one flood suitable for calibration. For the Beef River, the unit hydrograph parameters derived from this flood were used without further verification.

A time-area curve was constructed for each basin by measuring stream distances on 7.5-minute quadrangle maps. An equal rate of travel was assumed in every stream reach, as recommended by the COE's Hydrologic Engineering Center (Reference A-2).

E. Unit Hydrograph Parameter Verification

The estimated unit hydrograph parameters were then applied in the HEC-1 model of each basin to predict the runoff hydrograph resulting from the storm reserved for verification. When the verification hydrographs and the previously estimated unit hydrograph parameters did not agree well, the data were reviewed and the parameters adjusted depending on the hydrologist's judgment of the reliability of the data from each storm. Agreement was good and only minor adjustments were made in all cases but one: the Lost River gage. For that gage, the three calibration runs yielded T_c values ranging from 16 to 46 hours. Initially, an average T_c of 29 hours was used in the verification run, but this underestimated the lag time by approximately 12 hours. A review of soils data for the Lost River basin showed that the soils in the lower basin closest to the gage are predominantly clays and other fine-grained soils. Farther upstream, soils are sandier and more permeable. It was concluded that the storm for which the apparent T_c was only 16 hours (the storm of September 1969) actually produced runoff only from the lower, relatively impermeable portion of the basin. This assumption was supported by a relatively high calibrated basin-averaged loss rate. The other storms analyzed, however, produced runoff from all of the basin. The latter condition was assumed to be most representative of the saturated conditions assumed to occur during the PMF. Therefore, the September 1969 storm was dropped from the analysis.

The final Clark parameters estimated for each of the eight gaged basins are summarized in Table A-2.

TABLE A-2
Clark Parameters Estimated in Regional Study

Basin Name and Gage No.	Clark T_c (hrs.)	Clark R (hrs.)
Blue River (No. 60880000)	35	50
Sister River (No. 60873330)	54	72
Beef River (No. 60845600)	21.5	25
Edwards River (No. 6089600)	30.5	62
Lost River (No. 60967000)	46	45
Sweetwater River (No. 60411400)	11.5	11.5
Muddy River (No. 60499900)	28	32
Blackbird River (No. 60675000)	35	28

F. Regression Analysis

Basin parameters including drainage area, flow length, channel slope, average soil permeability, forest cover, and wetland and lake storage were considered as possible predictors for T_c and R in a regional equation. Drainage area was determined from the USGS gage data. Channel length was measured on 7.5-minute quadrangle maps. Soil permeability, forest cover, and storage were measured from digital land cover and soil maps. A log-log multiple regression was performed for both T_c and R to derive regional equations for these parameters.

Due to the small number of data points in the analysis, each regression equation was limited to the two most significant parameters. The two most significant predictors for the dependent variable T_c were the length, in miles, of the longest flow path (L) and forest cover, expressed as percent (F). The standard error of estimate for the regression was 20 percent, expressed in real space. The R^2 value (coefficient of determination) for the logarithmic regression was 0.90. The two most significant predictors of R were the length of the longest flow path (L) and the percent of the basin in lake and wetland storage (ST). The standard error of estimate was 27 percent. The coefficient of determination for the logarithmic regression (which indicates the extent to which the dependent variables are explained by the independent variables) was 0.88. These coefficients of determination indicate a good correlation (Reference A-3, *Section 8-9.2.3*).

The regional equations for T_c and R are:

$$T_c = 0.19L^{0.75} F^{0.58}$$

$$R = 1.95L^{0.44} ST^{0.50}$$

where:

L = Length of the longest flow path in miles
F = Percent of basin covered by forest
ST = Percent of basin covered by lake and wetland storage.

Table A-3 shows the basin parameters for each basin, the actual estimated T_c and R, and the T_c and R predicted by the regression equation.

TABLE A-3
Regional Unit Hydrograph Analysis:
Regression Parameters and Predicted T_c and R

Basin	L	ST	F	T_c[1]	T_c[2]	R[1]	R[2]
Blue River	58.0	28.1	63.5	35	42.6	50	60.9
Sister River	64.4	28.6	64.7	54	46.5	72	64.3
Beef River	18.8	17.2	73.6	21.5	20.0	25	29.1
Edwards River	40.2	23.2	66.4	27.5	33.2	57	47.1
Lost River	63.5	11.4	49.8	46	39.6	45	40.4
Sweetwater River	50.2	4.9	27.5	28	23.6	32	23.9
Muddy River	32.3	1.9	16.1	11.5	12.5	11.5	12.3
Blackbird River	88.0	7.3	30.8	35	38.3	28	37.3

[1] *By calibration.*
[2] *By regression.*

The percent difference between the calibrated T_c and the predicted T_c varies from -16 percent to 22 percent. The percent difference between the calibrated R and predicted R varies from -25 percent to 33 percent. There does not appear to be a systematic bias in these estimates. The regression equations can be expected to overestimate or underestimate the true unit hydrograph parameters. However, these equations should be used on ungaged basins in the Blue River region, rather than using purely synthetic techniques, because they provide an objective method based on regional data and are thus the best available technique.

Appendix A References

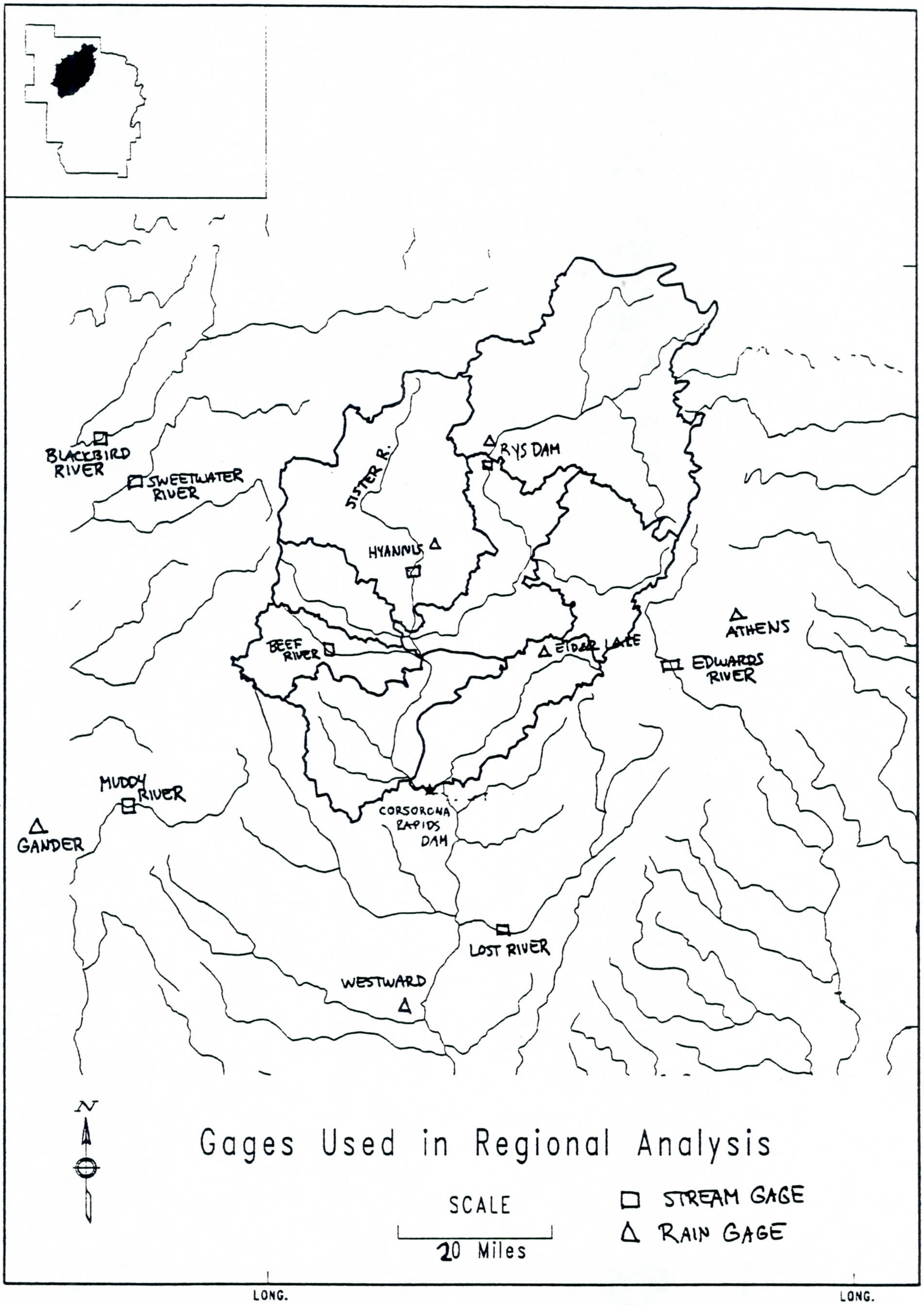
A-1. Anystate Geological Survey, 1987. *Effects of Regulation on Flood Frequencies and Magnitudes in the Blue River Basin*. Technical Report No. 87-092.

A-2. U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1982. "Hydrologic Analysis of Ungaged Watersheds Using HEC-1," Training Document No. 15.

A-3. Federal Energy Regulatory Commission, 1993. *Guidelines for the Evaluation of Hydropower Projects*. Chapter VIII, "Determination of the Probable Maximum Flood."

EXHIBIT A-1

Location of Gaged Basins Used in Analysis and Rain Gages



LAT.
LAT.
LAT.
BLACKBIRD RIVER
SWEETWATER RIVER
SISTER R.
RYS DAM
HYANNIS
BEEF RIVER
ELDER LAKE
ATHENS
EDWARDS RIVER
MUDDY RIVER
GANDER
CORSORONA RAPIDS DAM
LOST RIVER
WESTWARD
N
Gages Used in Regional Analysis
SCALE
20 Miles
STREAM GAGE
RAIN GAGE
LONG.
LONG.

EXHIBIT A-2

Unit Hydrograph Parameter Calibration and Verification

HEC-1 Model Input and Output

Note: Complete HEC-1 input and output have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

REVIEW AND QUESTIONS

Wednesday 8:00 a.m.

Probable Maximum Flood Studies

Example 2: Bishopsville Hydroelectric Project

June 1994

Purpose: *To illustrate a PMF calculation for an ungaged basin using synthetic unit hydrographs (SCS method) in the absence of local or regional streamflow information.*

Summary

Subbasin Division:	Six subbasins
Routing:	Reservoir routing using COE UNET model; hydrologic routing subroutines in HEC-1 for drainage basin upstream of reservoir; routing through Liberty Lake using Modified Puls in HEC-1 model; Muskingum routing for river reach between Liberty Lake and upstream end of reservoir
Unit Hydrograph Analysis:	"Ungaged" basin; no existing regional studies; SCS dimensionless unit hydrograph method used to determine PMF
Loss Rates:	Approximate method using hydrologic soil groups
Initial Reservoir Level:	Assumed equal to the early fall target elevation of 1,339 feet NGVD; 100-year rainfall distribution and runoff model
Snowpack:	Not applicable
Snowmelt:	Not applicable
Sensitivity Analysis:	Model sensitivity to subbasin lag time

Example 2
Bishopsville Hydroelectric Project
Probable Maximum Flood

June 1994

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Tables

Exhibits

Example 2
Bishopsville Hydroelectric Project
Probable Maximum Flood

June 1994

Summary

This study was performed to estimate the Probable Maximum Flood (PMF) at the Bishopsville Hydroelectric Project, Federal Energy Regulatory Commission (FERC) Project No. MMMM. The study follows the standards set forth in Chapter VIII of the FERC *Engineering Guidelines for the Evaluation of Hydropower Projects* (Reference 1). The PMF inflow calculated in this study is 116,880 cubic feet per second (cfs), and the PMF outflow is 32,330 cfs. During the PMF, the project dikes would overtop by 1.2 feet. Therefore, the hazard due to overtopping failure should be assessed and if found to be significant, remedial action should be taken to protect the dikes from overtopping during the PMF.

I. Project Description

A. Project Data

The Bishopsville Dam is located on the Liberty River in Yahquemon County, Anystate, approximately 2 miles upstream from the corporate limits of the City of Bishopsville. The reservoir is also used for municipal water supply and, as a result, normal headwater fluctuates between 1,339 and 1,343 feet National Geodetic Vertical Datum (NGVD), depending on season and water demand. (All elevations in this report are given in feet NGVD.) Normal tailwater elevation is 1,304 feet. The maximum height of the dam above the streambed is 55 feet. The maximum normal reservoir surface area is 2,600 acres, and the maximum normal volume is 35,100 acre-feet. Storage data for the project are shown in Table 1.

TABLE 1
Bishopsville Project Storage Data

Elevation (ft. NGVD)	Surface Area (acres)	Volume (acre-feet)
1,299	0	0
1,310	220	1,730
1,339 (minimum normal)	2,030	25,070
1,343 (maximum normal)	2,600	35,100
1,353 (top of dikes)	3,910	66,800

The dam was constructed in 1956 and the powerhouse and generating equipment added in 1988. From left to right looking downstream, the project structures consist of a 1,350-foot-long earth dike, a municipal water system intake, a 136-foot-long overflow spillway, an 1,880-foot-long earth dike, and a powerhouse. The crest elevation of the earth dikes is 1,353.0 feet, and the crest elevation of the overflow spillway is 1,343 feet. Maximum discharge over the spillway with the pool at the top of the dikes is 13,500 cfs (Reference 2).

The project is operated jointly by the Bishopsville Public Works Department (BPWD) and Ten-Mile Associates, the holder of the FERC license. According to the agreement between these two parties, flows are used first for municipal needs and secondarily for hydropower generation, with a minimum guaranteed percentage of average flow reserved for hydropower. The BPWD predicts inflows based on soil moisture, antecedent flow, and weather forecasts. An operator visits the plant twice daily. Headwater and tailwater are monitored electronically by the BPWD. There are no moveable spill gates, and all flood flows are passed over the overflow dam.

B. Basin Hydrologic Data

The drainage area of the Liberty River basin at the Bishopsville Dam is 71 square miles. The basin is moderately steep, with an elevation difference of 600 feet from the highest divide to the Bishopsville Reservoir. The average main channel slope is 23 feet per mile. The Bishopsville Reservoir is approximately 8 miles long and less than one-half mile wide. A basin map is included as Exhibit 1.

A nonrecording (daily totals only) rain gage is located approximately 10 miles east of the project at Murdoch Springs. According to the record from the Murdoch Springs rain gage, late fall and spring are generally the wettest times of the year in terms of monthly average precipitation. However, the three highest daily totals on record all occurred in September 4.8 inches in 1938, 4.4 inches in 1948, and 3.2 inches in 1984.

Snow occurs only rarely and does not remain on the ground throughout the winter. Winter daytime temperatures rise above freezing on most days. The critical season for a PMF was thus assumed to be nonwinter, and winter flooding conditions were not considered in this analysis.

Data used for hydrologic analyses were obtained from U.S. Geological Survey (USGS) topographic maps, county soil maps, and bathymetric maps of the Bishopsville Reservoir provided by the state Department of Fish and Game.

There are no operating or discontinued streamflow gages in the basin. The project owners and owners of an upstream dam were contacted for information on timing of flows at these dams, but no such data are available. Such records as do exist are daily only and are not useful for reconstructing flood hydrographs.

The Anystate Department of Natural Resources (ADNR) used flood frequency regression equations developed for the northwest region of the state to estimate a 100-year flood of 3,700 cfs at Bishopsville (Reference 3). The estimate of the 50-year flood is 3,200 cfs.

C. Upstream Dams

There is one dam, the Liberty Lake Dam, upstream of the project at a drainage area of 13 square miles. Owned by Yahquemon County Parks, the Liberty Lake Dam impounds a 1,280-acre reservoir. The dam has a 100-foot overflow spillway and a drop inlet structure for passing low flows. All flood flows are passed by the overflow spillway, and there is no other operating plan for flood conditions (Reference 3).

D. Field Visit

The project hydrologist visited the Bishopsville project on June 14, 1993, and discussed project operation, high and low operating levels, flow forecasting, and recent flooding with Mr. Marlon Spitzer of the BPWD. According to Mr. Spitzer, the long-term average reservoir level is approximately 1,341 feet. The reservoir is typically highest in early summer and late fall, following spring and fall rains. By early fall, the reservoir level is allowed to fall to a target elevation of 1,339 feet in anticipation of fall rains.

The largest flood event in the past 10 years caused the reservoir to rise approximately 3.4 feet over the crest of the overflow spillway. The estimated outflow associated with this event was 2,600 cfs.

The project hydrologist also visited the Liberty Lake Dam and interviewed Yahquemon County Parks Department staff regarding historical flooding. An informational display at the Liberty Lake Park described a flood event in 1948 in which a portion of the earth embankment was destroyed by overtopping. The estimated flood discharge associated with this event was 3,000 cfs.

A driving reconnaissance of the watershed was also conducted. The watershed's steeper slopes are forested, with flatter areas used for agriculture, grazing and low-density rural development. The Liberty River channel between Liberty Lake Dam and the Bishopsville Reservoir is cut into alluvial silts and sands. Upstream of Liberty Lake Dam, the main channel and tributaries are steeper and rockier than in the lower basin. The typical channel section is approximately 15 feet wide and 3 feet deep to the top of the bank.

E. Previous Studies

There have been no previous PMF studies for the Bishopsville Project. Before the installation of hydroelectric equipment, the project was regulated by the state, which requires a spillway capacity equal to the 500-year flood. This was estimated to be 4,200 cfs by the state regression equations discussed above.

II. Watershed Model and Subdivision

A. Watershed Model Methodology

The runoff hydrograph due to the Probable Maximum Precipitation (PMP) was simulated with the Corps of Engineers (COE) HEC-1 model. HEC-1 input and output data are included as Exhibit 2. The combined watershed hydrograph generated with the HEC-1 model was input to the COE UNET, a dynamic routing model, for reservoir routing. UNET input and output summaries are presented in Exhibit 3.

B. Subbasin Definition

The watershed was divided into six subbasins for input to the HEC-1 watershed model. The subbasins were selected to account for differences in timing, soils, and storage in Liberty Lake. Table 2 summarizes the subbasins, and subbasin division is mapped in Exhibit 4.

TABLE 2
Liberty River Subbasin Division

Subbasin	Area (Sq. miles)	Main Tributary
1	8	Tug Creek
2	20	Brooklyn-Deer-Sand Creek, & Bishopsville Reservoir
3	13	Liberty River headwaters and Liberty Lake
4	5	Miller Creek
5	12	Morrison Creek
6	13	Little Liberty River

C. Channel Routing Method

Hydrologic routing subroutines in the HEC-1 model were used for the drainage basin upstream of the Bishopsville Reservoir. In the reservoir itself, however, the COE's dynamic routing model, UNET, was used. The HEC-1 model was not used for this part of the analysis because neither level-pool nor hydrologic open-channel types of routing are considered appropriate for a long, narrow reservoir with significant backwater effects. UNET, in contrast to HEC-1, routes a flow hydrograph through a channel or reservoir using dynamic routing equations to define the water surface slope, storage, and travel time through the reach.

The runoff hydrograph from the Liberty Lake subbasin was generated with the HEC-1 watershed model and routed through Liberty Lake using the Modified Puls routing option in the HEC-1 model. The Muskingum-Cunge routing method incorporated in HEC-1 was used for the river reach between Liberty Lake and the upstream end of the Bishopsville Reservoir, combining with inflow hydrographs from Tug Creek and Miller Creek.

Bathymetric maps of the Bishopsville Reservoir were provided by the state Department of Fish and Game. Seventeen cross sections were measured from these maps and input to the UNET model. Cross-section data for elevations above the lake level were obtained from USGS quadrangle maps.

III. Historic Flood Records

A. Stream Gages

There are no operating or discontinued stream gages in the Liberty River watershed.

B. Historic Floods

Information regarding historic floods is anecdotal (see *Section I,D.*). The largest flood known to have occurred in the watershed had an estimated peak flow of 3,000 cfs at the Liberty Lake Dam, based on the estimated depth of spillway overtopping. It is not known what the corresponding flow at the Bishopsville project was, nor is the timing of this event well documented.

IV. Unit Hydrograph Development

A. Discussion of Approach and Tasks

Based on this review of flooding information, the data from the Liberty River are not considered adequate to develop a unit hydrograph. The basin is therefore considered "ungaged" (Reference 1, *Section 8-7.1.2*). The remaining options are to (1) use an existing regional study; (2) develop a regional study; or (3) use synthetic methods. As discussed below, existing data are inadequate to use or develop a regional study. Therefore, the SCS synthetic hydrograph method was used.

B. Existing Studies

The USGS, the COE, the ADNR, the Soil Conservation Service (SCS) and the National Weather Service (NWS) were all contacted in a search for regional unit hydrograph studies applicable to the Liberty River. Although the COE has conducted a unit hydrograph analysis on the Lower Fox River, into which the Liberty River flows, this study only covers drainage areas greater than 300 square miles and therefore was not useful at the scale of the Liberty River study. Similarly, almost all continuously recording stream gages in the region are located at drainage areas of 400 square miles or more and were not considered representative of the Liberty River above the Bishopsville project. Therefore, developing a regional unit hydrograph study for this analysis was not feasible.

C. SCS Dimensionless Unit Hydrograph

In the absence of local or regional unit hydrograph data, the SCS dimensionless unit hydrograph method was used to determine the PMF at the Bishopsville Project. The limitations of the SCS method (Reference 1, *Section 8-9,4*), as well as hydrologic considerations such as routing through Liberty Lake, influenced the selection of subbasins as discussed in *Section II, B*.

To apply the SCS method, the drainage basin was divided into the six subbasins shown in Table 2. The SCS unit hydrograph requires an estimate of the basin time of concentration (T_c) the time for the most distant point in the subbasin to begin to contribute to runoff at the point of interest. This is also the time for a kinematic wave to travel through the entire subbasin.

The T_c was estimated from 7½-minute USGS topographical maps. Each subbasin was divided into an overland section from the subbasin divide to the creek or river headwaters, and three reaches along the creek to the subbasin outlet. The channel was broken into reaches to reflect changes in slope. The overland distances were generally small and were measured with a scale. The channel distances were measured digitally using CAD software. Slopes were estimated between contour lines crossing the channel.

The average velocity for the overland flow was estimated as a function of percent slope (Reference 4). The average velocity for the channel reaches was determined by using the computed slope in the Manning's equation. A best hydraulic trapezoidal section with 3 feet water depth (the bank full depth) was assumed. An *n* value of 0.070 (Reference 5) was assumed to represent the resistance typical of the tributary streams at flood stage.

Kinematic wave velocity is approximately 1.5 times the average overland and channel velocity, varying somewhat as a function of channel geometry. For this study, the estimated velocities were multiplied by 1.5 to obtain kinematic wave velocity. Finally, T_c for each reach is the reach length divided by the kinematic wave velocity. The total T_c for the subbasin is the sum of the concentration times for each reach. Basin lag time is then estimated as 0.6 times the T_c. Estimated lag times and concentration times for each subbasin are summarized in Table 3, and calculations are summarized in Exhibit 5.

TABLE 3
Time of Concentration and Lag Time Estimates

Basin No.	Name	Drainage Area (sq. mi.)	Total Flow Length (mi.)	T_c (hrs.)	Lag time (hrs.)
1	Tug Creek	8	10.7	6.0	3.6
2	Bishopsville Reservoir Brooklyn-Deer-Sand Creek	20	7.6	4.5	2.7
3	Liberty Lake & Liberty River headwaters	13	6.1	3.3	2.0
4	Miller Creek	5	5.1	2.9	1.8
5	Morrison Creek	12	6.2	3.9	2.3
6	Little Liberty River	13	5.1	3.8	2.3

V. Unit Hydrograph Verification

Unit hydrographs could not be verified since gage data are not adequate, as stated previously. Daily flow and stage records at the project also could not be used to verify unit hydrographs, because the T_c for all the subbasins is much less than one day.

VI. Probable Maximum Storm

A. Probable Maximum Precipitation Data

Depth-area-duration values for the PMP were determined from maps in *Hydrometeorological Report No. 51* (Reference 6). The PMP depth-area-duration relationship for the Liberty River basin is shown in Table 4 and presented graphically in Exhibit 6.

TABLE 4
Probable Maximum Precipitation
Depth-Area-Duration Data (in.)

Area (sq. mi.)	Duration (hrs.)				
	6	12	24	48	72
10	22.5	25.7	28.0	30.4	32.6
200	16.5	19.2	21.2	23.4	25.4
1,000	12.4	14.7	16.6	18.6	20.4
5,000	7.8	9.7	11.5	13.4	15.2
10,000	5.7	7.5	9.2	11.2	12.8
20,000	3.8	5.5	7.2	9.0	10.7

B. Candidate Storms for the PMF

The spatial and temporal distributions of various candidate Probable Maximum Storms (PMS) were determined with the COE HMR52 computer program, which determines the storm pattern consistent with a given set of depth-area-duration values that maximizes precipitation depth over a specified subbasin or group of subbasins.

Several possible storm centerings were analyzed to identify the storm size and position that causes the most severe flood flow at the Bishopsville project. Because of the significant storage in the reservoir, it is possible that the flood with the highest peak outflow would not produce the highest stage in the reservoir. In other words, the project PMF may be controlled by volume rather than peak flow. For each storm centering considered, the entire model sequence (HMR52, HEC-1, and UNET) was run to generate the resulting inflow and outflow hydrographs at the project. HEC-1, UNET, and HMR52 input files and output data are included as Exhibits 2, 3, and 7, respectively.

PMS hyetographs were generated with HMR52 for storms centered over the entire basin, each subbasin, and one potentially critical combination of subbasins within the Liberty River watershed. In evaluating potentially critical combinations of subbasins, consideration was given to subbasin size and orientation, unit hydrograph timing, and basin permeability.

The potentially critical multi-subbasin storm was optimized for the lower three subbasins (Little Liberty River, Brooklyn-Deer-Sand Creek, and Morrison Creek). Taken together, these subbasins have less permeable soils than the upper three subbasins. Also, because this grouping consists of several small drainages entering the reservoir at about the same time, it is expected that the ratio of peak inflow to total runoff volume will be greatest for this combination of basins.

The critical storm proved to be a 50-square-mile storm, centered on the Brooklyn-Deer-Sand Creek basin. The PMS sequence for each subbasin is shown in Table 5.

TABLE 5
Probable Maximum Storm (PMS) Sequence
Liberty River Basin
PMS Incremental Depth (inches)

Hours	Subbasin Number/Name					
	1	2	3	4	5	6
	Tug Creek	Brooklyn Deer Sand Creek	Liberty Lake Liberty River Headwaters	Miller Creek	Morrison Creek	Little Liberty River
0-6	0.29	0.32	0.24	0.26	0.26	0.28
6-12	0.35	0.38	0.29	0.32	0.31	0.34
12-18	0.45	0.49	0.37	0.40	0.39	0.43
18-24	0.62	0.67	0.50	0.55	0.54	0.59
24-30	0.98	1.06	0.80	0.88	0.86	0.94
30-36	2.39	2.74	1.94	2.13	2.09	2.31
36-42	16.94	20.56	13.05	14.60	14.43	16.43
42-48	1.38	1.53	1.13	1.24	1.22	1.34
48-54	0.76	0.82	0.62	0.68	0.66	0.73
54-60	0.52	0.56	0.43	0.47	0.46	0.50
60-66	0.40	0.43	0.32	0.36	0.35	0.38
66-72	0.32	0.35	0.26	0.29	0.28	0.31

Hard copies of complete HMR52 input and output data and a 3.5-inch diskette containing the data are included in Exhibit 7. The isohyetal pattern for the critical storm is shown in Exhibit 8. *(Diskettes should accompany actual submittals; however, the diskette is omitted from this example.)*

VII. Loss Rates

A. Discussion of Loss Rate Methodology

Loss rates were determined by the approximate method using hydrologic soil groups (HSG) (Reference 1, *Section 8-10.3.1*). In this method, a minimum expected loss rate is assigned to each subbasin on the basis of HSGs in the basin. Rainfall losses are then modeled using the initial and constant loss rate method in the HEC-1 model. Hydrologic soil groups in each subbasin were identified from county soils surveys. The approximate percentage of each soil group was estimated by delineating the subbasins on the county soils maps and laying a sample grid over the subbasin.

B. Warm-Season

Initial losses were assumed to be zero, corresponding to an assumption of completely saturated conditions at the outset of the PMS. In addition to the minimum constant loss rate corresponding to HSG, the percentage of impervious surface (lakes, rivers, and paved surfaces) in each subbasin was also estimated from the soil survey maps. This fraction was entered separately in the HEC-1 model. Subbasin average loss rates were assigned based on an area weighted average of HSG loss rates, using the following classification (Reference 1, *Section 8-10.3.1 and Table 8-10.1*):

HSG A: 0.30 in./hr.
HSG B: 0.15 in./hr.
HSG C: 0.05 in./hr.
HSG D: 0.00 in./hr.

Table 6 shows the HSG composition of each subbasin, the minimum constant loss rate assigned to the subbasin, and the percent of subbasin area assumed impervious.

TABLE 6
Assumed Subbasin Loss Rates

Basin No.	Name	Hydrologic Soil Group Composition (%)	Assigned Loss Rate (in./hr.)	Percent Impervious
1	Tug Creek	50 A 40 B 10 D	0.21	5
2	Brooklyn-Deer-Sand Creek and Bishopsville Reservoir	20 A 25 B 45 C 10 D	0.12	24
3	Liberty River to Liberty Lake	40 A 40 B 20 C	0.19	9
4	Miller Creek	45 A 35 B 15 C 5 D	0.20	2
5	Morrison Creek	35 A 35 B 10 C 20 D	0.16	2
6	Little Liberty River	15 A 45 B 40 C	0.13	3

C. Cool-Season

The Liberty River basin is located in a temperate climate where significant freezing and snowpack do not occur. Therefore, separate cool-season calculations are not applicable.

VIII. Coincident Hydrometeorological and Hydrological Conditions for the Probable Maximum Flood

A. Reservoir Level

Rainfall data at the Murdoch Springs rain gage suggest that the PMS is most likely to occur in late summer or early fall. Therefore, the initial reservoir level was assumed to be equal to the early fall target elevation of 1,339 feet NGVD. Since this is not the annual maximum normal operating level, an antecedent 100-year storm was assumed to end three days before the PMS. The 24-hour, 100-year rainfall for the Liberty River basin is 5.3 inches (Reference 7). Using the loss rate assumptions documented above, the 100-year storm produces a peak inflow to the reservoir of 14,900 cfs. Three days after the end of this storm, when the PMS is assumed to begin, the reservoir level is at 1,343.4 feet. The 100-year rainfall distribution and runoff model are included in the HEC-1 output in Exhibit 2.

B. Baseflow

Baseflow was estimated on the basis of a comparison with nearby gaged basins. The four gaged basins nearest the Liberty River range in size from 398 square miles to 2,100 square miles. At these four sites, average September flow ranges from 0.7 to 1.2 cfs per square mile. The average of 1.0 cfs per square mile was assigned to the Liberty River. The SCS uses a baseflow recession constant of 1.01 for watersheds less than 100 square miles in this region (Reference 8). This value was adopted for the Liberty River.

C. Snowpack

Snow is uncommon in this region and permanent snowpacks do not develop. Therefore, snowpack analysis is not applicable to this basin.

D. Snowmelt

Snowmelt is not applicable.

IX. PMF Hydrographs

A number of different storm centerings and subbasin optimizations were evaluated, as discussed previously, to identify the conditions producing the PMF at the project. The critical conditions selected are those producing the highest peak stage (and hence highest outflow) at the project. Using this criterion, the PMF is produced by a 50-square-mile storm centered on the 20-square-mile Brooklyn-Deer-Sand Creek subbasin. Residual rainfall at isohyet areas greater than 50 square miles would fall on the remainder of the basin.

A. Inflow PMF Hydrograph

The PMF inflow hydrograph is plotted in Exhibit 9. The peak of the PMF inflow hydrograph is 116,880 cfs.

B. Sensitivity Analysis

Unit Hydrograph Parameters. Because the lag time used in the SCS dimensionless unit hydrograph could not be calibrated or verified against real data, a sensitivity analysis to this parameter was conducted by varying all the subbasin lag times by plus and minus 15 percent. The peak inflows and outflows estimated with these variations are shown in Table 7.

TABLE 7
Model Sensitivity to Lag Time

Change in Lag Time (All Subbasins)	Peak Inflow (cfs)	Percent Change in Peak Inflow from Base Case	Peak Outflow (cfs)	Percent Change in Peak Outflow from Base Case
Base Case	116,880	—	32,330	—
+ 15 percent	106,700	-8.7	30,910	-4.4
- 15 percent	126,380	8.1	34,000	5.2

Table 7 shows that the inflow peak is moderately sensitive to lag time and the outflow peak much less so. Since the outflow is largely volume-controlled and lag time does not affect total flood volume, the conclusions of this study are not sensitive to subbasin lag time.

Loss Rates. No sensitivity analyses for loss rate were performed, because the loss rates adopted in Table 5 are believed to be highly conservative. This conclusion is based on the difference between the peak discharge predicted to result from the 100-year rainfall and the estimate of the 100-year flood peak discharge derived from the Anystate regional regression equation (Reference 2). According to the PMF watershed model, the 100-year rainfall with the assumed loss rates would produce a flood four times greater than the estimated 100-year flood. Additional conservatism is unnecessary; in fact, this discrepancy warrants further investigation of the loss function as part of studies for design and construction of additional spillway capacity.

Precipitation. The worst-case precipitation pattern was determined by using the HMR52 computer model for several possible storm centerings. In addition, each run of the HMR52 model maximizes the subbasin average precipitation as a function of storm size and orientation.

C. Reservoir Outflow PMF

The PMF outflow hydrograph was determined by dynamically routing the combined hydrographs from all subbasins through the Bishopsville Reservoir. The outflow hydrograph is plotted in Exhibit 9. The peak PMF outflow is 32,330 cfs. The peak stage at the project is 1,354.2 feet, 1.2 feet over the top of the dikes.

References

1. Federal Energy Regulatory Commission, 1993. *Engineering Guidelines for the Evaluation of Hydropower Projects.* Chapter VIII, "Determination of the Probable Maximum Flood."

2. Grady & Grudy Engineers, 1991. *Report on Inspection, Bishopsville Hydroelectric Project.*

3. Anystate Department of Natural Resources, 1985. *Anystate Large Dam Inventory*, Technical Release ADNR-85003.

4. Barfield, Warner and Haan, 1983. *Applied Hydrology and Sedimentology for Disturbed Areas*, Figure 2.34, page 100.

5. Chow, Ven Te, 1959. *Open-Channel Hydraulics*, McGraw-Hill Book Company, page 161.

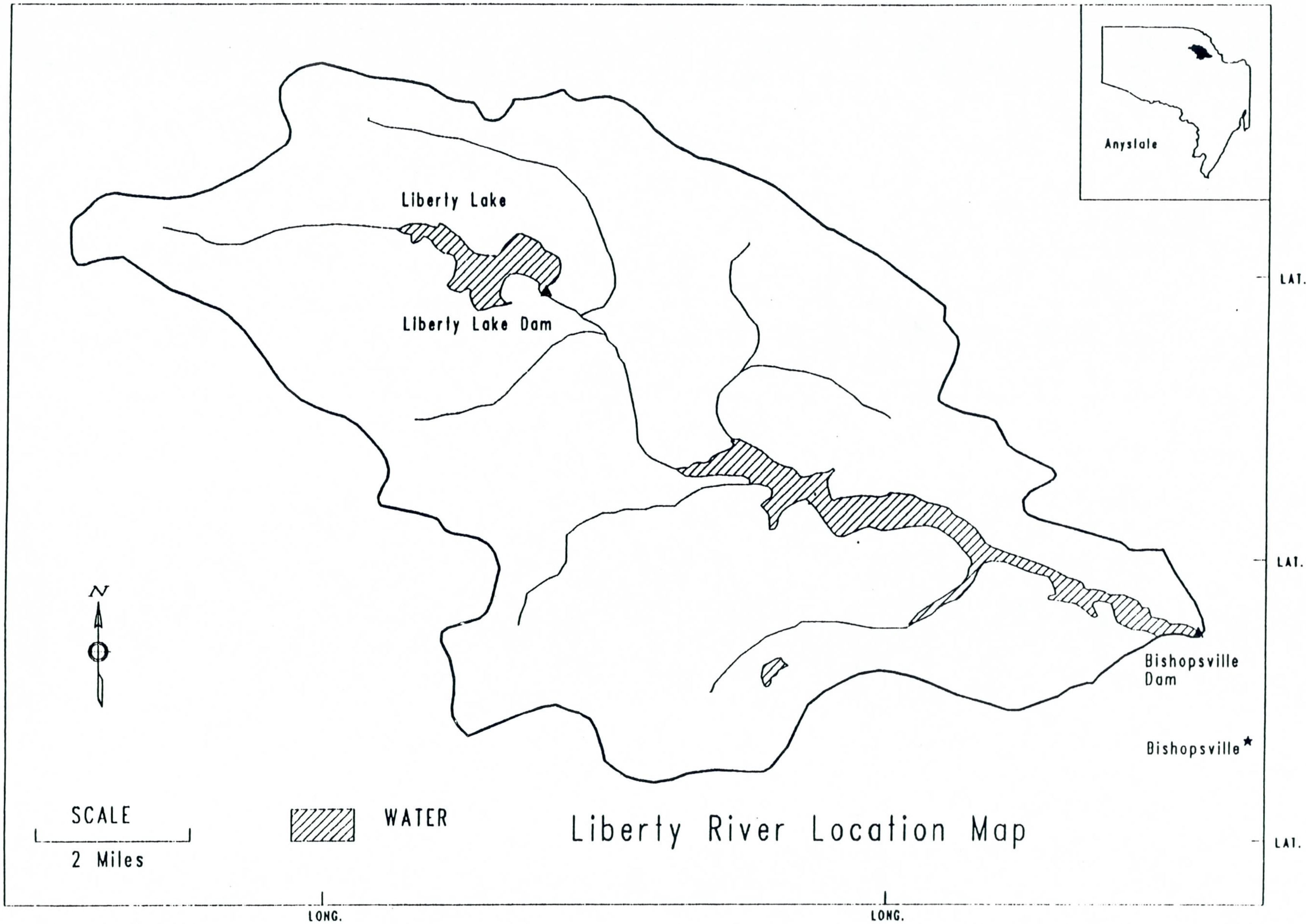
6. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, U.S. Army Corps of Engineers, 1978. *Hydrometeorological Report No. 51*.

7. U.S. Weather Bureau, 1963. *Rainfall Frequency Atlas of the United States*, Technical Paper 40.

8. Glossobor, Ralph, 1991. U.S. Department of Agriculture, Soil Conservation Service, personal communication.

EXHIBIT 1

Basin Map



Liberty River Location Map

EXHIBIT 2

HEC-1 Model Input Output

Note: Complete HEC-1 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

```
ID  WARM SEASON
ID  ******
ID  STORM CENTERED ON BROOKLYN, DEER, SAND CREEK SYSTEM SUBBASIN
ID  ******
ID  6/1994
ID  1993 PMF GUIDELINES
ID  SCS UNIT HYDROGRAPH METHOD
ID
ID  LIBERTYW.HC1
*FREE
IT 60    01AUG99   0100   300
IO  0
KK LIB
KM LIBERTY LAKE BASIN
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
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PI 0.12  0.12  0.13  0.14  0.14  0.15  0.24  0.24  0.27  0.32
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PI 0.05  0.05  0.04  0.04  0.04  0.04  0.04  0.04
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BF 13   -.1   1.01
LU  0.0  0.19  9.0
UD  2.0
KK  LIBDAM
KM STORAGE ROUTING THRU LIBERTY LAKE DAM
KM RATING BASED ON POWER INFO AND TOPO MAPS
RS 1  ELEV  1484.6
SV 25400  27900  30400  33310  40400  41490  46550
SE 1480    1481.9 1484    1486    1490.6  1491.2  1494
SQ 0       350    620     1255     1975     8800     33325
SE 1484.6  1486   1487    1488     1488.6   1490.65  1492.76
KK RT1
KM ROUTE HYDROGRAPH TO MILLER CREEK AND LIBERTY LAKE CONFLUENCE
RM  1  0.70  0.2
KK  TUG
KM  HYDROGRAPH FOR TUG CREEK
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
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PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
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PI 0.07  0.07  0.07  0.07  0.10  0.10  0.10  0.10  0.10  0.10
PI 0.14  0.15  0.16  0.17  0.18  0.19  0.29  0.31  0.34  0.40
PI 0.47  0.57  0.99  1.75  2.78  7.54  2.36  1.52  0.28  0.25
PI 0.23  0.22  0.20  0.19  0.13  0.13  0.13  0.13  0.13  0.13
PI 0.09  0.09  0.09  0.09  0.09  0.09  0.07  0.07  0.07  0.07
```

```
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BA  7.8
BF   8    -.1     1.01
LU  0.0  0.21     5.0
UD   3.6
KK  MILL
KM  HYDROGRAPH FOR MILLER CREEK
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
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PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
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BA  5.3
BF   5   -.1     1.01
LU  0.0  0.20   2.0
UD  1.7
KK  JN1
KM  COMBINE LIBERTY LAKE, TUG CREEK, MILLER CREEK HYDROGRAPHS
HC  3
KK  RT2
KM  ROUTE HYDROGRAPH TO BISHOPSVILLE RESERVOIR
RM  1  0.36  0.2
KK  MORR
KM  MORRISON CREEK HYDROGRAPH
PB
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PI 0.13  0.13  0.14  0.15  0.15  0.16  0.26  0.27  0.30  0.35
PI 0.42  0.51  0.94  1.72  2.59  5.40  2.27  1.50  0.24  0.22
PI 0.21  0.19  0.18  0.17  0.11  0.11  0.11  0.11  0.11  0.11
PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
PI 0.06  0.06  0.05  0.05  0.05  0.05  0.05  0.05
BA  11.8
BF  12  -.1    1.01
LU  0.0  0.16   2.0
UD  2.3
KK  BROO
KM  BROOKLYN-DEER-SAND CREEK SYSTEM AND BISHOPSVILLE RESERVOIR
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
```

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PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.06  0.06  0.06  0.06  0.06  0.06  0.08  0.08
PI 0.08  0.08  0.08  0.08  0.11  0.11  0.11  0.11  0.11  0.11
PI 0.16  0.16  0.17  0.18  0.19  0.20  0.33  0.36  0.40  0.46
PI 0.54  0.64  1.03  1.79  3.05 10.67  2.48  1.54  0.31  0.28
PI 0.26  0.24  0.22  0.21  0.14  0.14  0.14  0.14  0.14  0.14
PI 0.09  0.09  0.09  0.09  0.09  0.09  0.07  0.07  0.07  0.07
PI 0.07  0.07  0.06  0.06  0.06  0.06  0.06  0.06
BA 19.9
BF 20  -.1    1.01
LU  0.0  0.12 24.0
UD  2.8
KK LITT
KM LITTLE LIBERTY RIVER HYDROGRAPH
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.06  0.06  0.06  0.06  0.06  0.06  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.10  0.10  0.10  0.10  0.10  0.10
PI 0.14  0.15  0.15  0.16  0.17  0.18  0.28  0.30  0.33  0.39
PI 0.46  0.55  0.97  1.73  2.72  7.21  2.32  1.50  0.27  0.25
PI 0.23  0.21  0.20  0.19  0.12  0.12  0.12  0.12  0.12  0.12
PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
PI 0.06  0.06  0.05  0.05  0.05  0.05  0.05  0.05
BA 12.7
BF 13  -.1    1.01
LU  0.0  0.13 3.0
UD  2.3
KK  JN2
KM  COMBINE LIBERTY, BROOK(ET AL), MORRISON AND LITTLE LIBERTY
HC  4
ZW  A=BROO B=UPSW  C=FLOW  D=01AUG1999  E=1HOUR  F=WS
ZZ
```

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ID  WARM SEASON
ID  ******
ID  STORM CENTERED ON BROOKLYN, DEER, SAND CREEK SYSTEM SUBBASIN
ID  ******
ID  6/1994
ID  1993 PMF GUIDELINES
ID  SCS UNIT HYDROGRAPH METHOD
ID
ID  LIBERTYP.HC1
ID  LAG TIMES PLUS 15%
*FREE
IT 60    01AUG99   0100   300
IO  0
KK LIB
KM LIBERTY LAKE BASIN
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.04  0.04  0.04  0.04
PI 0.04  0.04  0.05  0.05  0.05  0.05  0.05  0.05  0.06  0.06
PI 0.06  0.06  0.06  0.06  0.08  0.08  0.08  0.08  0.08  0.08
PI 0.12  0.12  0.13  0.14  0.14  0.15  0.24  0.24  0.27  0.32
PI 0.39  0.48  0.94  1.75  2.54  4.00  2.28  1.53  0.23  0.21
PI 0.19  0.18  0.17  0.16  0.10  0.10  0.10  0.10  0.10  0.10
PI 0.07  0.07  0.07  0.07  0.07  0.07  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.04  0.04  0.04  0.04  0.04  0.04
BA 12.7
BF 13   -.1   1.01
LU  0.0  0.19  9.0
UD  2.3
KK  LIBDAM
KM STORAGE ROUTING THRU LIBERTY LAKE DAM
KM RATING BASED ON POWER INFO AND TOPO MAPS
RS 1  ELEV  1484.6
SV 25400  27900  30400  33310  40400  41490  46550
SE 1480   1481.9 1484   1486   1490.6  1491.2  1494
SQ 0      350    620    1255    1975    8800     33325
SE 1484.6  1486  1487   1488    1488.6  1490.65  1492.76
KK RT1
KM ROUTE HYDROGRAPH TO MILLER CREEK AND LIBERTY LAKE CONFLUENCE
RM  1  0.70  0.2
KK  TUG
KM  HYDROGRAPH FOR TUG CREEK
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.06  0.06  0.06  0.06  0.06  0.06  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.10  0.10  0.10  0.10  0.10  0.10
PI 0.14  0.15  0.16  0.17  0.18  0.19  0.29  0.31  0.34  0.40
PI 0.47  0.57  0.99  1.75  2.78  7.54  2.36  1.52  0.28  0.25
PI 0.23  0.22  0.20  0.19  0.13  0.13  0.13  0.13  0.13  0.13
```

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PI 0.09  0.09  0.09  0.09  0.09  0.09  0.07  0.07  0.07  0.07
PI 0.07  0.07  0.05  0.05  0.05  0.05  0.05  0.05
BA  7.8
BF   8    -.1    1.01
LU  0.0  0.21     5.0
UD   4.1
KK  MILL
KM  HYDROGRAPH FOR MILLER CREEK
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.04  0.04  0.04  0.04
PI 0.04  0.04  0.05  0.05  0.05  0.05  0.05  0.05  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.09  0.09  0.09  0.09  0.09  0.09
PI 0.13  0.14  0.14  0.15  0.16  0.17  0.26  0.27  0.30  0.35
PI 0.43  0.52  0.98  1.79  2.67  5.25  2.35  1.56  0.25  0.23
PI 0.21  0.20  0.18  0.17  0.11  0.11  0.11  0.11  0.11  0.11
PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
PI 0.06  0.06  0.05  0.05  0.05  0.05  0.05  0.05
BA  5.3
BF   5   -.1    1.01
LU  0.0  0.20   2.0
UD  2.0
KK  JN1
KM  COMBINE LIBERTY LAKE, TUG CREEK, MILLER CREEK HYDROGRAPHS
HC  3
KK  RT2
KM  ROUTE HYDROGRAPH TO BISHOPSVILLE RESERVOIR
RM  1  0.36  0.2
KK  MORR
KM  MORRISON CREEK HYDROGRAPH
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.04  0.04  0.04  0.04
PI 0.04  0.04  0.05  0.05  0.05  0.05  0.05  0.05  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.09  0.09  0.09  0.09  0.09  0.09
PI 0.13  0.13  0.14  0.15  0.15  0.16  0.26  0.27  0.30  0.35
PI 0.42  0.51  0.94  1.72  2.59  5.40  2.27  1.50  0.24  0.22
PI 0.21  0.19  0.18  0.17  0.11  0.11  0.11  0.11  0.11  0.11
PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
PI 0.06  0.06  0.05  0.05  0.05  0.05  0.05  0.05
BA  11.8
BF  12  -.1    1.01
LU  0.0  0.16   2.0
UD  2.6
KK  BROO
KM  BROOKLYN-DEER-SAND CREEK SYSTEM AND BISHOPSVILLE RESERVOIR
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
```

```
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.06  0.06  0.06  0.06  0.06  0.06  0.08  0.08
PI 0.08  0.08  0.08  0.08  0.11  0.11  0.11  0.11  0.11  0.11
PI 0.16  0.16  0.17  0.18  0.19  0.20  0.33  0.36  0.40  0.46
PI 0.54  0.64  1.03  1.79  3.05 10.67  2.48  1.54  0.31  0.28
PI 0.26  0.24  0.22  0.21  0.14  0.14  0.14  0.14  0.14  0.14
PI 0.09  0.09  0.09  0.09  0.09  0.09  0.07  0.07  0.07  0.07
PI 0.07  0.07  0.06  0.06  0.06  0.06  0.06  0.06
BA 19.9
BF 20  -.1   1.01
LU  0.0  0.12 24.0
UD  3.2
KK LITT
KM LITTLE LIBERTY RIVER HYDROGRAPH
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.06  0.06  0.06  0.06  0.06  0.06  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.10  0.10  0.10  0.10  0.10  0.10
PI 0.14  0.15  0.15  0.16  0.17  0.18  0.28  0.30  0.33  0.39
PI 0.46  0.55  0.97  1.73  2.72  7.21  2.32  1.50  0.27  0.25
PI 0.23  0.21  0.20  0.19  0.12  0.12  0.12  0.12  0.12  0.12
PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
PI 0.06  0.06  0.05  0.05  0.05  0.05  0.05  0.05
BA 12.7
BF 13  -.1   1.01
LU  0.0  0.13 3.0
UD  2.6
KK  JN2
KM  COMBINE LIBERTY, BROOK(ET AL), MORRISON AND LITTLE LIBERTY
HC  4
ZW  A=BROO B=UPSW  C=FLOW  D=01AUG1999  E=1HOUR  F=WS
ZZ
```

```
ID  WARM SEASON
ID  ******
ID  STORM CENTERED ON BROOKLYN, DEER, SAND CREEK SYSTEM SUBBASIN
ID  ******
ID  6/1994
ID  1993 PMF GUIDELINES
ID  SCS UNIT HYDROGRAPH METHOD
ID
ID
ID  LAG TIMES MINUS 15%
*FREE
IT 60    01AUG99   0100   300
IO  0
KK LIB
KM LIBERTY LAKE BASIN
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.04  0.04  0.04  0.04
PI 0.04  0.04  0.05  0.05  0.05  0.05  0.05  0.05  0.06  0.06
PI 0.06  0.06  0.06  0.06  0.08  0.08  0.08  0.08  0.08  0.08
PI 0.12  0.12  0.13  0.14  0.14  0.15  0.24  0.24  0.27  0.32
PI 0.39  0.48  0.94  1.75  2.54  4.00  2.28  1.53  0.23  0.21
PI 0.19  0.18  0.17  0.16  0.10  0.10  0.10  0.10  0.10  0.10
PI 0.07  0.07  0.07  0.07  0.07  0.07  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.04  0.04  0.04  0.04  0.04  0.04
BA 12.7
BF 13   -.1   1.01
LU  0.0  0.19  9.0
UD  1.7
KK  LIBDAM
KM STORAGE ROUTING THRU LIBERTY LAKE DAM
KM RATING BASED ON POWER INFO AND TOPO MAPS
RS 1  ELEV  1484.6
SV 25400  27900  30400  33310  40400  41490  46550
SE 1480    1481.9 1484    1486    1490.6  1491.2  1494
SQ 0       350    620     1255     1975     8800      33325
SE 1484.6  1486  1487    1488     1488.6  1490.65  1492.76
KK RT1
KM ROUTE HYDROGRAPH TO MILLER CREEK AND LIBERTY LAKE CONFLUENCE
RM  1  0.70  0.2
KK  TUG
KM  HYDROGRAPH FOR TUG CREEK
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.06  0.06  0.06  0.06  0.06  0.06  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.10  0.10  0.10  0.10  0.10  0.10
PI 0.14  0.15  0.16  0.17  0.18  0.19  0.29  0.31  0.34  0.40
PI 0.47  0.57  0.99  1.75  2.78  7.54  2.36  1.52  0.28  0.25
PI 0.23  0.22  0.20  0.19  0.13  0.13  0.13  0.13  0.13  0.13
```

```
PI 0.09  0.09  0.09  0.09  0.09  0.09  0.07  0.07  0.07  0.07
PI 0.07  0.07  0.05  0.05  0.05  0.05  0.05  0.05
BA  7.8
BF   8    -.1    1.01
LU  0.0  0.21     5.0
UD   3.1
KK  MILL
KM  HYDROGRAPH FOR MILLER CREEK
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.04  0.04  0.04  0.04
PI 0.04  0.04  0.05  0.05  0.05  0.05  0.05  0.05  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.09  0.09  0.09  0.09  0.09  0.09
PI 0.13  0.14  0.14  0.15  0.16  0.17  0.26  0.27  0.30  0.35
PI 0.43  0.52  0.98  1.79  2.67  5.25  2.35  1.56  0.25  0.23
PI 0.21  0.20  0.18  0.17  0.11  0.11  0.11  0.11  0.11  0.11
PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
PI 0.06  0.06  0.05  0.05  0.05  0.05  0.05  0.05
BA  5.3
BF   5   -.1    1.01
LU  0.0  0.20   2.0
UD  1.4
KK  JN1
KM  COMBINE LIBERTY LAKE, TUG CREEK, MILLER CREEK HYDROGRAPHS
HC  3
KK  RT2
KM  ROUTE HYDROGRAPH TO BISHOPSVILLE RESERVOIR
RM  1  0.36  0.2
KK  MORR
KM  MORRISON CREEK HYDROGRAPH
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.04  0.04  0.04  0.04
PI 0.04  0.04  0.05  0.05  0.05  0.05  0.05  0.05  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.09  0.09  0.09  0.09  0.09  0.09
PI 0.13  0.13  0.14  0.15  0.15  0.16  0.26  0.27  0.30  0.35
PI 0.42  0.51  0.94  1.72  2.59  5.40  2.27  1.50  0.24  0.22
PI 0.21  0.19  0.18  0.17  0.11  0.11  0.11  0.11  0.11  0.11
PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
PI 0.06  0.06  0.05  0.05  0.05  0.05  0.05  0.05
BA  11.8
BF  12  -.1   1.01
LU  0.0  0.16   2.0
UD  2.0
KK  BROO
KM  BROOKLYN-DEER-SAND CREEK SYSTEM AND BISHOPSVILLE RESERVOIR
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
```

```
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.06  0.06  0.06  0.06  0.06  0.06  0.08  0.08
PI 0.08  0.08  0.08  0.08  0.11  0.11  0.11  0.11  0.11  0.11
PI 0.16  0.16  0.17  0.18  0.19  0.20  0.33  0.36  0.40  0.46
PI 0.54  0.64  1.03  1.79  3.05 10.67  2.48  1.54  0.31  0.28
PI 0.26  0.24  0.22  0.21  0.14  0.14  0.14  0.14  0.14  0.14
PI 0.09  0.09  0.09  0.09  0.09  0.09  0.07  0.07  0.07  0.07
PI 0.07  0.07  0.06  0.06  0.06  0.06  0.06  0.06
BA 19.9
BF 20  -.1   1.01
LU  0.0  0.12 24.0
UD  2.4
KK LITT
KM LITTLE LIBERTY RIVER HYDROGRAPH
PB
PI 0.20  0.20  0.20  0.20  0.20  0.20  0.58  0.58  0.58  0.58
PI 0.58  0.58  0.07  0.07  0.07  0.07  0.07  0.07  0.03  0.03
PI 0.03  0.03  0.03  0.03  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00  0.00
PI 0.00  0.00  0.00  0.00  0.00  0.00  0.05  0.05  0.05  0.05
PI 0.05  0.05  0.06  0.06  0.06  0.06  0.06  0.06  0.07  0.07
PI 0.07  0.07  0.07  0.07  0.10  0.10  0.10  0.10  0.10  0.10
PI 0.14  0.15  0.15  0.16  0.17  0.18  0.28  0.30  0.33  0.39
PI 0.46  0.55  0.97  1.73  2.72  7.21  2.32  1.50  0.27  0.25
PI 0.23  0.21  0.20  0.19  0.12  0.12  0.12  0.12  0.12  0.12
PI 0.08  0.08  0.08  0.08  0.08  0.08  0.06  0.06  0.06  0.06
PI 0.06  0.06  0.05  0.05  0.05  0.05  0.05  0.05
BA 12.7
BF 13  -.1    1.01
LU  0.0  0.13 3.0
UD  2.0
KK  JN2
KM  COMBINE LIBERTY, BROOK(ET AL), MORRISON AND LITTLE LIBERTY
HC  4
ZW  A=BROO B=UPSW  C=FLOW  D=01AUG1999  E=1HOUR  F=WS
ZZ
```

EXHIBIT 3

UNET Input and Output Summaries

Note: Complete UNET outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

```
R ON
T1 LIBERTY
T2 PMF ANALYSIS
T3 UNET
*
*
* client:   BISHOPSVILLE PUBLIC WORKS DEPARTMENT
* location: BISHOPSVILLE RESERVOIR TO BISHOPSVILLE DAM
*           YAHQUEMON COUNTY, ANYSTATE
*
* special features of this model: Manning's "n" set to 0.110 in the overbank
*         areas.  Ref: USGS Water-Supply Paper 2339, Arcement and Schneider
*                 1989, Guide for Selecting Manning's Roughness Coefficients
*                 for Natural Channels and Flood Plains
*
* UPSTREAM BOUNDARY IS A HYDROGRAPH
UB
*
ZD LIBERTY CSECT
ZS ON
* Write Invert Profile to DSS, UNITS = MILES
ZI MILES
* Write Area and Conveyance Tables to DSS (ON/OFF)
ZA ON
*
* Limits on Cross-section Table Card
*  BELBK    RISE   ELSTRT  SLOPE   ELINC    XINC      FM   CMILE
XK   -2                              1.5     3.0     .67       0
*XK    1      15       0    .001      1.5     1.0       0       0
*
* Manning's "n" Values Card
*    XNL      XNR    XNCH   ALPHA
NC 0.085   0.085   0.030
******************************************************************************
*
*
* SURVEYED XSEC T-4, OCT 1992
NC  .110    .110    .035    .000
X1  0.00      12  9930.0 10070.0   5280.   5280.   5280.           -15.2
HY 1S0.0
GR1380.0  9160.0  1360.0  9800.0  1350.0  9862.5  1340.0  9930.0  1338.0  9972.5
GR1336.0  9990.0  1336.0 10010.0  1338.0 10027.5  1340.0 10070.0  1350.0 10137.5
GR1360.0 10200.0  1380.0 10840.0
* T-5
NC  .110    .110    .035    .000
X1  1.00      12  9510.0 10490.0   5808.   5808.   5808.           -13.7
GR1380.0  8050.0  1360.0  8270.0  1348.0  9400.0  1340.0  9510.0  1338.0  9830.0
GR1335.0  9977.5  1335.0 10022.5  1338.0 10170.0  1340.0 10490.0  1348.0 10600.0
GR1360.0 11730.0  1380.0 11950.0
* T-6
NC  .110    .110    .035    .000
X1  2.10      12  9262.5 10737.5   3168.   3168.   3168.           -12.2
HY 1S2.1
GR1380.0  5050.0  1360.0  6250.0  1342.0  9262.5  1338.0  9500.0  1335.0  9700.0
GR1330.0  9975.0  1330.0 10025.0  1335.0 10300.0  1338.0 10500.0  1342.0 10737.5
GR1360.0 13750.0  1380.0 14950.0
* T-7
NC  .110    .110    .035    .000
X1  2.70      12  7300.0 12700.0   3168.   3168.   3168.           -11.3
HY 1S2.7
```

```
GR1380.0  4820.0  1360.0  6400.0  1342.0  7300.0  1338.0  8687.5  1335.0  9162.5
GR1330.0  9725.0  1330.0 10275.0  1335.0 10837.5  1338.0 11312.5  1342.0 12700.0
GR1360.0 13600.0  1380.0 15180.0
* Limits on Cross-section Table Card
*  BELBK    RISE   ELSTRT  SLOPE   ELINC    XINC      FM   CMILE
XK    -4                             2.0     3.0     .67       0
* T-8
NC  .110    .110    .035    .000
X1  3.30      12  8387.5 11612.5   4224.   4224.   4224.             -10.5
HY 1S3.3
GR1380.0  7460.0  1360.0  7950.0  1344.0  8387.5  1335.0  9437.5  1330.0  9800.0
GR1325.0  9987.5  1325.0 10012.5  1330.0 10200.0  1335.0 10562.5  1344.0 11612.5
GR1360.0 12050.0  1380.0 12540.0
* T-9
NC  .110    .110    .035    .000
X1  4.10      12  8237.5 11762.5   4752.   4752.   4752.              -9.3
HY 1S4.1
GR1380.0  7380.0  1360.0  7550.0  1338.0  8237.5  1336.0  9150.0  1330.0  9400.0
GR1325.0  9900.0  1325.0 10100.0  1330.0 10600.0  1336.0 10850.0  1338.0 11762.5
GR1360.0 12450.0  1380.0 12620.0
* T-10
NC  .110    .110    .035    .000
X1  5.00      12  8375.0 11625.0   2640.   2640.   2640.              -8.0
HY 1S5.0
GR1380.0  8070.0  1360.0  8150.0  1340.0  8375.0  1335.0  8787.5  1325.0  9700.0
GR1320.0  9962.5  1320.0 10037.5  1325.0 10300.0  1335.0 11212.5  1340.0 11625.0
GR1360.0 11850.0  1380.0 11930.0
* T-11
NC  .110    .110    .035    .000
X1  5.50      12  8925.0 11075.0   4224.   4224.   4224.              -7.3
HY 1S5.5
GR1380.0  8670.0  1360.0  8790.0  1340.0  8925.0  1330.0  9200.0  1325.0  9837.5
GR1320.0  9900.0  1320.0 10100.0  1325.0 10162.5  1330.0 10800.0  1340.0 11075.0
GR1360.0 11210.0  1380.0 11330.0
* T-12
NC  .110    .110    .035    .000
X1  6.30      12  9012.5 10987.5   3696.   3696.   3696.              -6.2
HY 1S6.30
GR1380.0  6700.0  1360.0  8950.0  1340.0  9012.5  1325.0  9225.0  1315.0  9950.0
GR1310.0  9995.0  1310.0 10005.0  1315.0 10050.0  1325.0 10775.0  1340.0 10987.5
GR1360.0 11050.0  1380.0 13300.0
* T-13
NC  .110    .110    .035    .000
X1  7.00      12  9645.0 10355.0   4224.   4224.   4224.              -5.2
HY 1S7.0
GR1380.0  9130.0  1360.0  9370.0  1340.0  9645.0  1320.0  9745.0  1315.0  9915.0
GR1305.0  9960.0  1305.0 10040.0  1315.0 10085.0  1320.0 10255.0  1340.0 10355.0
GR1360.0 10630.0  1380.0 10870.0
* T-14
NC  .110    .110    .035    .000
X1  7.80      12  9550.0 10450.0   3168.   3168.   3168.              -4.0
HY 1S7.8
GR1380.0  9430.0  1360.0  9520.0  1345.0  9550.0  1320.0  9612.5  1310.0  9837.5
GR1300.0  9970.0  1300.0 10030.0  1310.0 10162.5  1320.0 10387.5  1345.0 10450.0
GR1360.0 10480.0  1380.0 10570.0
* T-15
NC  .110    .110    .035    .000
X1  8.40      12  9012.5 10987.5   2640.   2640.   2640.              -3.2
GR1380.0  7840.0  1360.0  8680.0  1340.0  9012.5  1320.0  9112.5  1310.0  9287.5
GR1300.0  9750.0  1300.0 10250.0  1310.0 10712.5  1320.0 10887.5  1340.0 10987.5
```

```
GR1360.0 11320.0  1380.0 12160.0
* T-16
NC  .110    .110    .035    .000
X1  8.90      12  9462.5 10537.5   4752.   4752.   4752.            -2.4
HY 1S8.9
GR1380.0  8840.0  1360.0  9360.0  1340.0  9462.5  1320.0  9750.0  1305.0  9862.5
GR1300.0  9962.5  1300.0 10037.5  1305.0 10137.5  1320.0 10250.0  1340.0 10537.5
GR1360.0 10640.0  1380.0 11160.0
* T-17
NC  .110    .110    .035    .000
X1  9.80      12  8712.5 11287.5   4224.   4224.   4224.            -1.1
HY 1S9.8
GR1380.0  7840.0  1360.0  8060.0  1340.0  8712.5  1320.0  9275.0  1300.0  9875.0
GR1295.0  9995.0  1295.0 10005.0  1300.0 10125.0  1320.0 10725.0  1340.0 11287.5
GR1360.0 11940.0  1380.0 12160.0
* U/S FACE OF BISHOPSVILLE DAM
NC  .200    .200    .200    .000
X1 10.60      12  8827.5 11172.5   4224.   4224.   4224.
HY BISHOPSVILLE DAM HW
GR1360.0  7800.0  1330.0  8827.5  1320.0  9107.5  1305.0  9625.0  1295.0  9900.0
GR1290.0  9987.5  1290.0 10012.5  1295.0 10100.0  1305.0 10375.0  1320.0 10892.5
GR1330.0 11172.5  1360.0 12200.0
*
* DOWNSTREAM BOUNDARY - SEE .BC FILE
DB
EJ
```

```
* FILE: EXAMPLE.BC
* Liberty River UNET model, MS season, centering
* on Brooklyn-Deer-Sand Creek subbasin
* PMF
* note: the third title line is used as the F-part of the DSS pathnames
*       for the output hydrographs
*
* JOB CONTROL PARAMETERS:
* IPRINT PZMX   DT   TSP   PT  TLEVEE  THETA  STORAGE  DPRINT  TWIC  DABINC
JOB CONTROL
       T    T  .1    24   -1      T   0.60        F       T    -1    -.5
*
*
QMULT= 1.0
*
*
DOWNSTREAM RATING CURVE
1 10
1338.9       0.
1339.0      90.
1343.0     340.
1345.0    1511.
1347.0    3754.
1349.0    6613.
1351.0    9997.
1353.0   13840.
1355.0   43662.
1357.0   95049.
1359.0  166865.
*
*
*----------------------------- HEC-1 generated input hydrographs in DSS format
OPEN DSS FILE INPUT
LIBERTYW.DSS 01AUG1999 0100 13AUG1999 1200 1
*
UPSTREAM FLOW HYDROGRAPH at the Cross section T-4
1
/BROO/UPSW/FLOW/01AUG1999/1HOUR/WS/
*
*
*
*
*
WRITE HYDROGRAPHS TO DSS OUTPUT
LIBERTYW.DSS
*
EJ
```

```
* FILE: EXAMPLE.BC
* Liberty River UNET model, MS season, centering
* on Brooklyn-Deer-Sand Creek subbasin
* PMF
* LAG TIMES INCREASED 15%
* note: the third title line is used as the F-part of the DSS pathnames
*       for the output hydrographs
*
* JOB CONTROL PARAMETERS:
* IPRINT PZMX   DT   TSP   PT  TLEVEE  THETA  STORAGE  DPRINT  TWIC  DABINC
JOB CONTROL
       T    T  .1    24   -1       T   0.60        F       T    -1    -.5
*
*
QMULT= 1.0
*
*
DOWNSTREAM RATING CURVE
1 10
1338.9        0.
1339.0       90.
1343.0      340.
1345.0     1511.
1347.0     3754.
1349.0     6613.
1351.0     9997.
1353.0    13840.
1355.0    43662.
1357.0    95049.
1359.0   166865.
*
*
*----------------------------- HEC-1 generated input hydrographs in DSS format
OPEN DSS FILE INPUT
LIBERTYP.DSS 01AUG1999 0100 13AUG1999 1200 1
*
UPSTREAM FLOW HYDROGRAPH at the Cross section T-4
1
/BROO/UPSW/FLOW/01AUG1999/1HOUR/WS/
*
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*
*
WRITE HYDROGRAPHS TO DSS OUTPUT
LIBERTYP.DSS
*
EJ
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```
* FILE: EXAMPLE.BC
* Liberty River UNET model, MS season, centering
* on Brooklyn-Deer-Sand Creek subbasin
* PMF
* LAG TIMES DECREASED 15%
* note: the third title line is used as the F-part of the DSS pathnames
*       for the output hydrographs
*
* JOB CONTROL PARAMETERS:
* IPRINT PZMX   DT   TSP   PT  TLEVEE  THETA  STORAGE  DPRINT  TWIC  DABINC
JOB CONTROL
      T     T  .1     24   -1       T   0.60        F        T    -1    -.5
*
*
QMULT= 1.0
*
*
DOWNSTREAM RATING CURVE
1 10
1338.9        0.
1339.0       90.
1343.0      340.
1345.0     1511.
1347.0     3754.
1349.0     6613.
1351.0     9997.
1353.0    13840.
1355.0    43662.
1357.0    95049.
1359.0   166865.
*
*
*------------------------------ HEC-1 generated input hydrographs in DSS format
OPEN DSS FILE INPUT
LIBERTYM.DSS 01AUG1999 0100 13AUG1999 1200 1
*
UPSTREAM FLOW HYDROGRAPH at the Cross section T-4
1
/BROO/UPSW/FLOW/01AUG1999/1HOUR/WS/
*
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*
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*
WRITE HYDROGRAPHS TO DSS OUTPUT
LIBERTYM.DSS
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EJ
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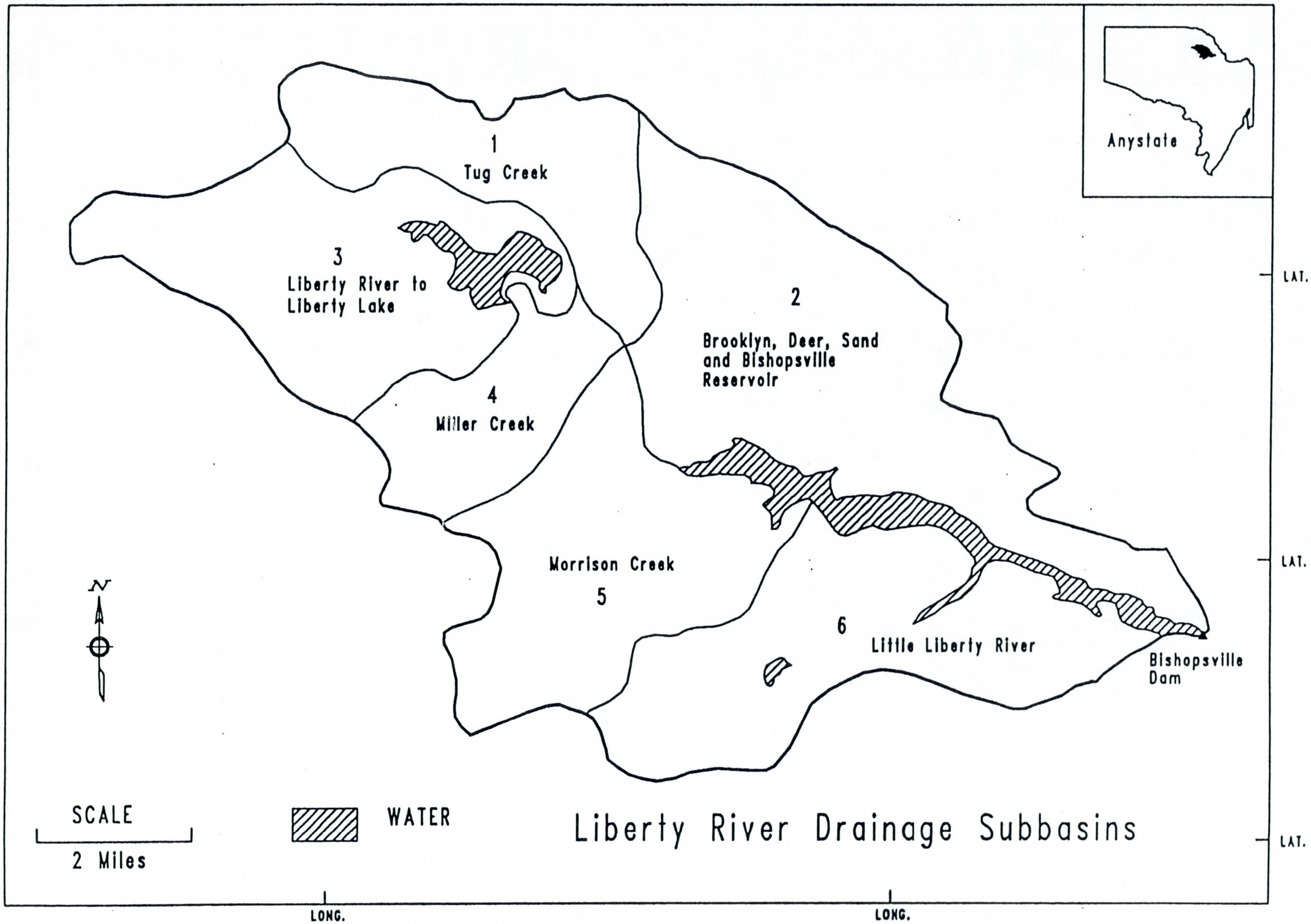
EXHIBIT 4

Subbasin Division Map



Liberty River Drainage Subbasins

EXHIBIT 5

Slope and Channel Velocity Calculations

Time of Concentration (T_c) Calculations[1]

1—Tug Creek

	Overland	1	2	3
Δx(ft)	1814	19998	15512	17895
Δy(ft)	59.5	107.1	166.6	47.6
V(ft/s)	0.5	2.0	2.9	1.4
t_c(hr)	0.7	1.9	1.0	2.4

2—Bishopsville Reservoir, Brooklyn-Deer-Sand Creek

	Overland	1	2	3
Δx(ft)	871	5770	15524	17938
Δy(ft)	28.6	173.7	59.5	47.8
V(ft/s)	0.5	4.8	1.7	4.8
t_c(hr)	0.3	0.3	1.7	2.3

3—Liberty Lake & Liberty River Headwaters

	Overland	1	2	3
Δx(ft)	1700	12642	8452	9305
Δy(ft)	52.4	54.7	130.9	52.6
V(ft/s)	0.4	1.8	3.5	2.1
t_c(hr)	0.7	1.3	.5	0.8

4—Miller Creek

	Overland	1	2	3
Δx(ft)	2539	3195	9327	11484
Δ(ft)	116.1	101.6	58	54.3
V(ft/s)	0.5	5.0	2.2	1.9
t_c(hr)	0.9	0.1	0.8	1.1

5—Morrison Creek

	Overland	1	2	3
Δx(ft)	1233	5572	11312	14669
Δ(ft)	89.2	99.4	72.5	25.4
V(ft/s)	0.7	3.7	2.2	1.1
t_c(hr)	0.3	0.3	0.9	2.3

6—Little Liberty River

	Overland	1	2	3
Δx(ft)	1741	4742	3524	16985
Δ(ft)	90.7	112.5	87.1	25.4
V(ft/s)	0.6	4.3	4.4	1.1
t_c(hr)	0.6	0.2	0.1	2.9

1 *Overland velocity from Applied Hydrology and Sedimentology for Disturbed Areas, Barfield, Warner, Haan, 1983, p. 100, Figure 2.34 Forested Land; section velocity - assume 3 ft. depth, hydraulic radius = wetted area/wetted perimeter, R = A/P = ½ y for best section, $S_f = S_o$. Mannings equation*

$$V = \frac{1.49 R^{2/3} S_f^{1/2}}{n}$$

S_f = bed grade
S_o = energy grade
$t_c = (L/v)/1.5$

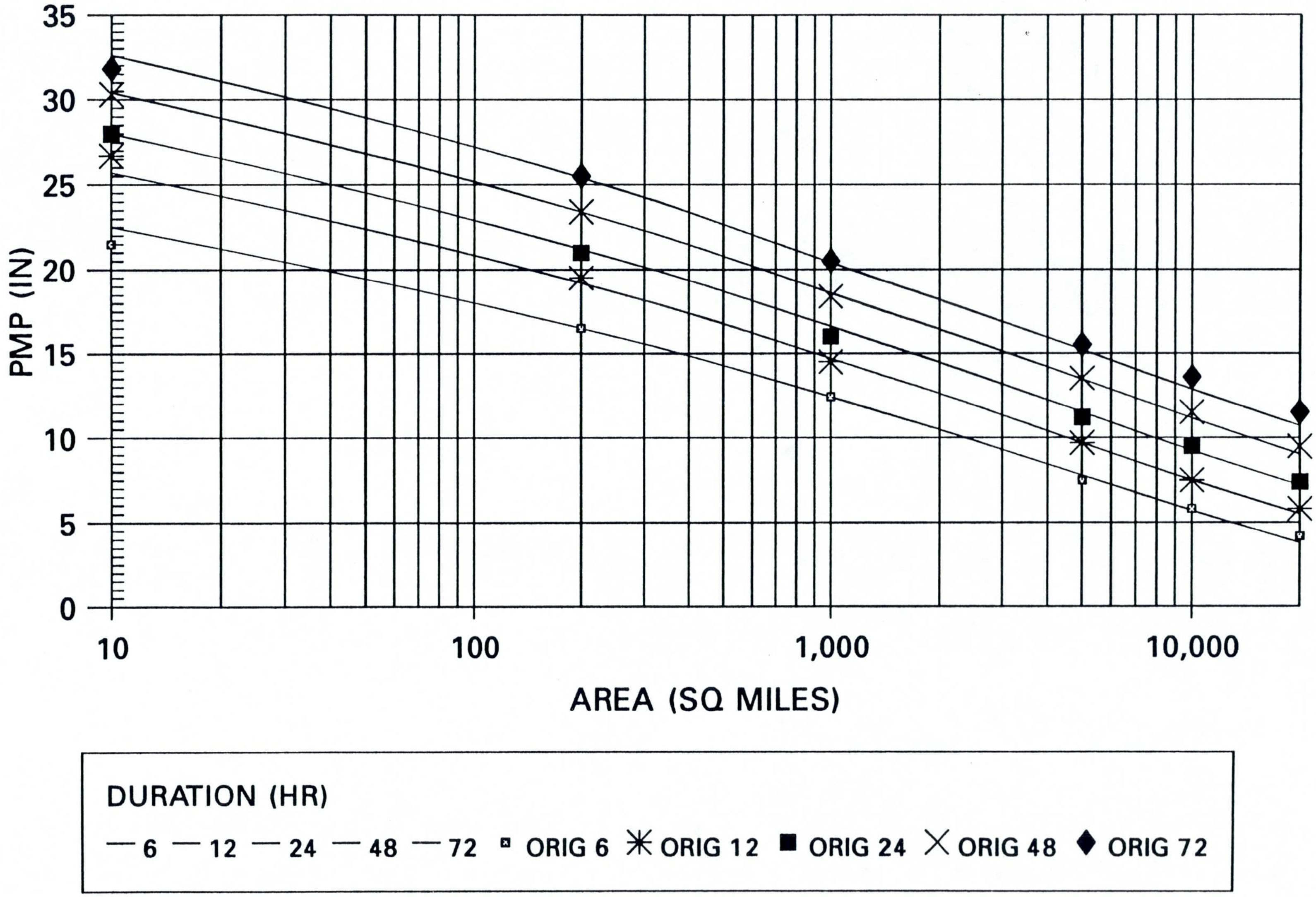
EXHIBIT 6

Liberty River Basin PMP
Depth-Area-Duration Relationship

Liberty River Basin to Bishopsville Dam

Yahquemon County, Anystate

Depth-Area-Duration Curves, Warm Season PMP



June 1994
Libertyw.ch3

EXHIBIT 7

HMR 52 Input and Output

Note: Complete HMR52 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

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ID WARM SEASON
ID CENTERING OVER BROOKLYN, DEER, SAND CREEKS SUBBASIN
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ID 6/1994
ID LIBERTYW.H52
*FREE
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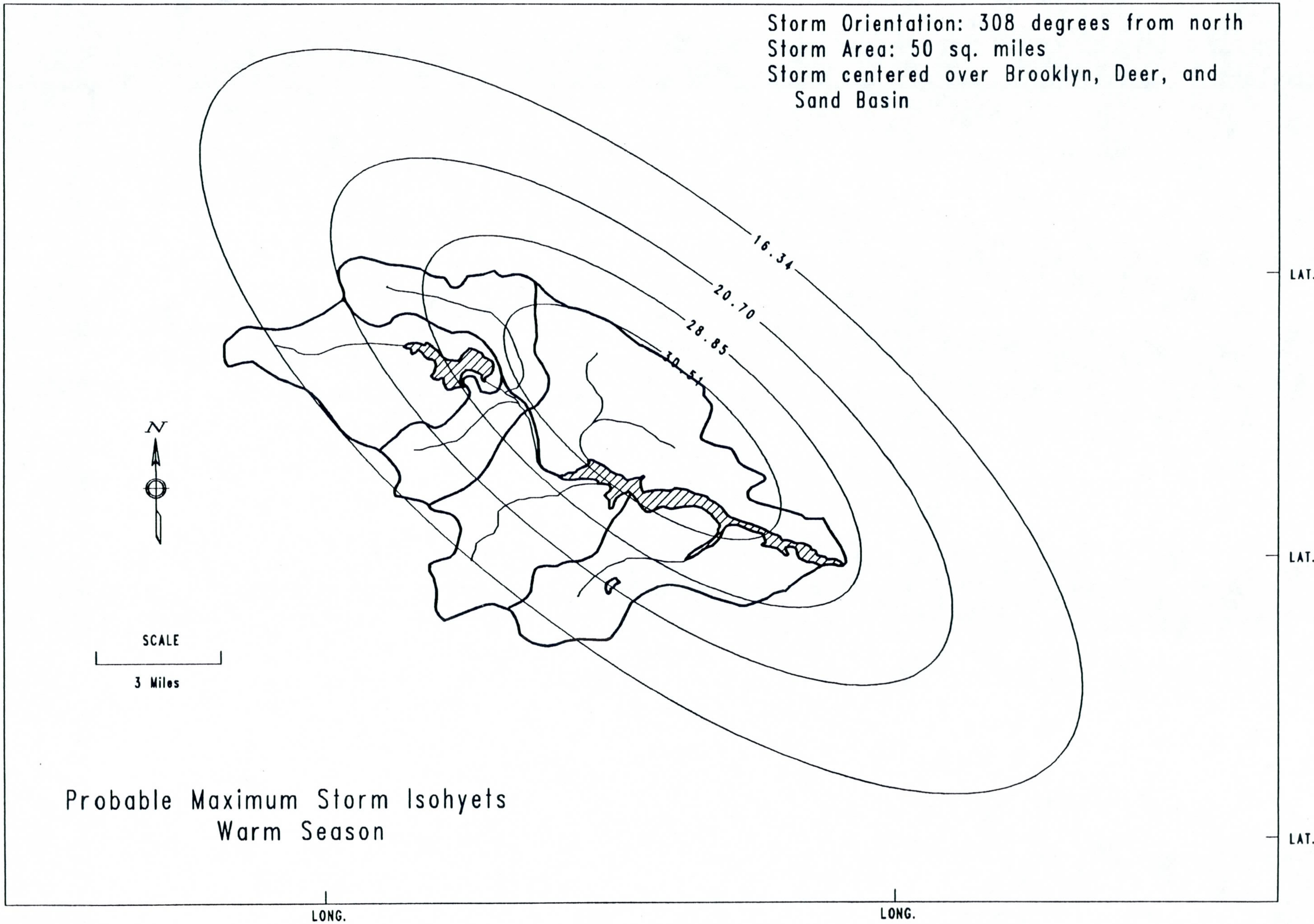
EXHIBIT 8

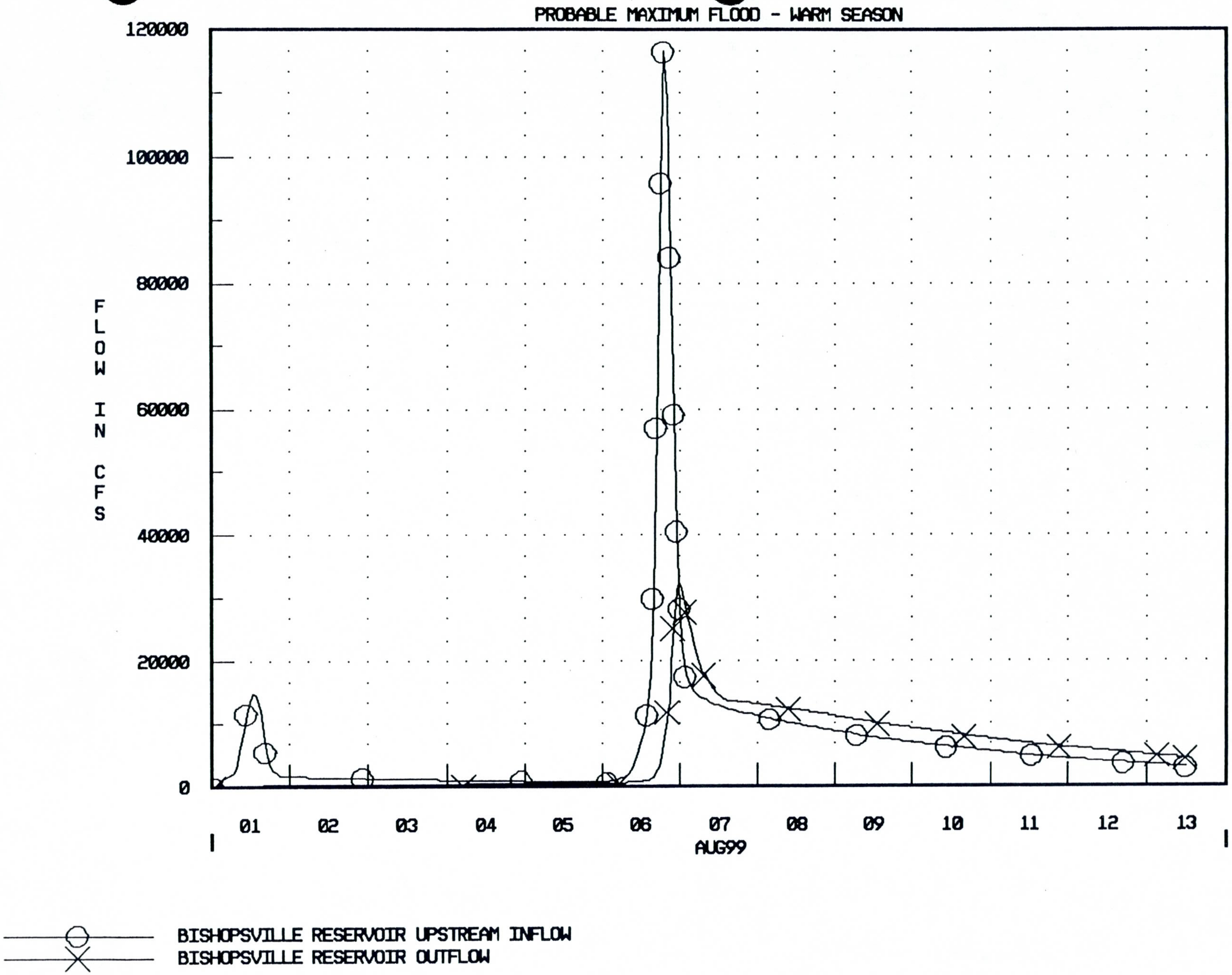
Probable Maximum Storm Isohyets



Probable Maximum Storm Isohyets
Warm Season

EXHIBIT 9

PMF Hydrographs


PROBABLE MAXIMUM FLOOD - WARM SEASON
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AUG99
BISHOPSVILLE RESERVOIR UPSTREAM INFLOW
BISHOPSVILLE RESERVOIR OUTFLOW

GLOSSARY, TERMS, AND REPORT FORMATS

Wednesday 9:50 a.m.

8-12 References

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8-13 Glossary

Some hydrologic terms have slightly different definitions depending upon the agency using them. These terms have been defined in terms of their meaning as used in these Guidelines.

Accuracy - Data are accurate if there are no errors. For example, clock records of rainfall and streamflow can be out of synchronization, implying that measured time is not accurate.

Active Storage - That portion of reservoir storage which is filled and emptied from year to year as the reservoir is operated.

Altitude-Depth Relationship - A relationship between snow pack water equivalent and elevation for a particular drainage area.

Antecedent Storm - A storm which precedes an extreme storm.

Baseflow - The streamflow rate occurring during recession of a hydrograph. Baseflow is separate from direct runoff.

Basin Average Rainfall - The spatially averaged rainfall depth within a drainage area for a particular total storm or time increment of that storm.

Basin Characteristics - The physical and meteorologic characteristics of a drainage area that control its hydrologic response in terms of runoff.

Channel Slope - The gradient measured by drop in elevation over channel distance, in foot per foot. The application should be consistent with the methodology.

Clark Unit Hydrograph - The unit hydrograph developed by C.O. Clark which accounts for storage in the basin as well as lag time.

Coefficient of Determination (r^2) - A measure of the degree to which a regression line explains the variance in the dependent variable.

Composite Unit Hydrograph - The unit hydrograph constructed from the unit hydrographs generated from historic storm and flood data. It is the unit hydrograph judged to be representative of the hydrologic response of the drainage area.

Consistency - Hydrologic data are consistent if no unusual changes or trends exist in the data.

Continuity - A record is continuous if the record contains no periods for which data is missing.

Continuous Streamflow Hydrograph - A hydrograph formed from the continuous stage recording at a streamgage.

Cover - The extent and type of vegetation covering the drainage area.

Cross Section - A vertical section taken across a stream channel or a reservoir used to determine flow area and hydraulic radius for flow routing.

Daily Flow Records - A record of average daily flows at a streamgage.

Degree-Day Method - A method to calculate snowmelt in terms of a degree-day factor [HEC 1990] determined from measured snowpack, runoff, and temperature for a historic storm.

Design Flood - The flood hydrograph for which a given project and its appurtenances are designed.

Dimensionless Unit Hydrograph - A unit hydrograph whose vertical and horizontal coordinates have been made dimensionless by dividing by the hydrograph peak flow and the time to peak respectively.

Disaggregating - The process of converting rainfall depths for one increment of time to the incremental depths for smaller increments of time.

Double-Mass Analysis - A plot of accumulated rainfall depth for one raingage against accumulated depth at another gage used to detect trends or inconsistencies within the data.

Drainage Area - The area above a particular point of interest from which surface drainage flows.

Emergency Gate Operation - The operation of gates on a controlled spillway when there is danger of the dam being overtopped if the gates are not opened sufficiently.

Extreme Flood - A flood whose peak flow is significantly larger than most historic floods.

Flashboards - Structures which temporarily raise the crest of an overflow spillway. Usually the flashboards are made from wooden planks supported by structural members.

Flood Hydrograph - A record of continuous streamflow versus time for a particular flood at a particular location on a stream.

Flood Storage - That portion of reservoir storage which is expressly reserved for storage of flood water.

Gaged Site - One for which available hydrologic data, recorded at stations within the basin, are sufficient in quantity and quality to provide confidence in development of an inflow PMF hydrograph.

General Storm - A storm caused by a frontal movement which generally covers a large area (ranging up to 60,000 square miles).

High-Water Mark - A mark which identifies the maximum stage which occurred at a particular location during a historic flood.

Homogeneous Data - Hydrologic data that all comes from the same phenomena and for the same time period.

Hydrograph - Rate of flow in a stream plotted against time for a particular section.

Hydrology - The science of the occurrence and movement of water on and within earth.

Hydrometeorology - The science of meteorology and hydrology related to the occurrence of extreme rainfall and extreme floods.

Hydrometeorological Report - Name given to a set of National Weather Service publications. They contain generalized studies of extreme rainfall for a particular region. Such reports provide generalized information for estimating probable maximum precipitation of a particular duration for given locations with the region.

Hyetograph - A graph of incremental rainfall depth versus time.

Infiltration Rate - The rate at which rainfall enters the surface of the soil in a given drainage area.

Inflow PMF Hydrograph - The hydrograph which represents PMF runoff entering a reservoir.

Initial Abstraction - That part of initial rainfall on a basin which is intercepted by vegetation, held in depressions, or evaporated.

Initial Flow - The streamflow at time, t, equals 0. Direct runoff does not necessarily start at this time. In HEC-1 initial flow is the parameter STRTQ.

Isohyet - A line along which rainfall depth is constant. Isohyets are used to develop an isohyetal map of rainfall for single storms or annual rainfall depth.

Isohyetal Pattern - Spatial distribution of rainfall represented by lines of equal rainfall depth (isohyets).

Kinematic Wave - The wave created by a change in flow rate in an open channel. The velocity of the wave is proportional to the change in depth and can be approximated as 1.5 times the average channel velocity.

Lag Time (T_L) - The time which locates the runoff hydrograph relative to the occurrence of a storm. It is generally determined as the difference in time between the centroid of rainfall excess and the peak of the runoff hydrograph, but definitions differ between methodologies.

Lapse Rate - The rate at which air temperature decreases with increasing altitude on a particular drainage area.

Large Dams - As defined by the International Commission on Large Dams, a Large Dam is one which is more than 50 meters in height.

Local Storms - A storm created by local convection which covers a limited area, generally not more than 500 square miles.

Manning Equation - The following equation for calculation of the average uniform velocity in an open channel $V = 1.48/n * R^{2/3} * S^{1/2}$ where V is the average velocity, R is the hydraulic radius for the section, S is the average slope of the channel, and "n" is a coefficient reflecting the roughness of the channel. The equation should be applied to segments of the channel that have constant slopes.

Manning's "n" - The coefficient used in the denominator of the Manning equation to represent the effect of channel roughness. It is roughly proportional to the one-sixth power of the relative roughness of the channel boundary.

Maximum Normal Operating Level - The maximum reservoir water-surface elevation which a hydroelectric project is normally operated during the year.

Maximum Possible Flood - An earlier term used to describe the Probable Maximum Flood.

Maximum Probable Precipitation - An earlier term used to describe the Probable Maximum Precipitation.

Minimum Infiltration Rate - The minimum rate at which infiltration occurs after the soil is saturated. This minimum rate is governed by the rate at which precipitation can enter the soil surface and percolate to the subsurface.

Nonlinear Effects - The tendency for a drainage area to yield peak flows for greater depths of storm rainfall which are larger than a linear proportion would indicate.

Operation Rules - The rules by which controlled spillways and outlet works are operated.

Orographic Effects - The effects of topographic variations on precipitation.

Overland Flow - Runoff flowing over the surface of a drainage area prior to reaching a channel.

Peak Flow - The maximum flow rate on a runoff hydrograph.

Permeability - The capacity of a soil to convey water. A high permeability occurs for soils having a large porosity, such as sands and gravels.

Preliminary Data - Physical and hydrologic data collected for a given project and its drainage area prior to making a visit to the site.

Probable Maximum Flood (PMF) - The flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the drainage basin under study.

Probable Maximum Precipitation (PMP) - The greatest depth of precipitation for a given duration that is physically possible for a given size storm area at a particular geographic location at a certain time of year.

Rainfall Sequence - The sequence of incremental rainfall depth used to develop a runoff hydrograph for the storm.

Rating Curve - A relationship between stage and flow rate developed for a particular streamgage location.

Reconstitution - The analytical process of using a developed unit hydrograph and historic storm rainfall to reproduce a historic flood hydrograph.

Redundant Operating System - An additional system for operating spillway and outlet works gates which is independent of all other systems.

Regional Studies - Studies of hydrologic data from drainage areas in the region to develop generalized information for calculation of a unit hydrograph for an ungaged area.

Regression - The mathematical analysis performed to assess the statistical correlation and relationship between a hydrologic parameter and physical or other hydrologic parameters for the drainage area.

Representative Unit Hydrograph - That unit hydrograph which represents the hydrologic response of the drainage area. It is the same as the composite unit hydrograph.

Reservoir Starting Level - The reservoir water-surface elevation assumed to exist at the beginning of the inflow PMF.

River Basin - The drainage area for a river above a particular point.

Routed Outflow - The downstream hydrograph which results from routing of a flood hydrograph through a reservoir using the relevant capacities of the spillway and outlet works.

Routing - The analytical process of computing the change in a flood wave as it passes through a reservoir or a channel.

Runoff Modeling - The analytical process of computing runoff from a particular storm. In these Guidelines the unit hydrograph is used as a component of the runoff model.

Safety Evaluation (As applied to a dam.) - The process of determining the ability of dam and its appurtenances to pass a given flood.

Snowmelt Calculation - Estimation of the snowmelt occurring for a particular snowpack and a given set of meteorologic conditions.

Snow Course - A defined line along which depths of snowpack and water content are measured and recorded on a regular basis.

Snow Cover - The portion of a drainage area which is covered by snow.

Snowpack - The depth of existing snow in a drainage area expressed in equivalent water content.

Snow Pillow - A device for the measurement of snow pack water equivalent through a process of weighing the overlying snow.

Snyder Unit Hydrograph - A synthetic unit-hydrograph method developed by F.M. Snyder for which the peak flow and time to peak are estimated in terms of regional coefficients.

Soil Map - A map identifying and showing the areal distribution of soil types.

Soil Moisture Content - The volume of moisture in the soil covering a drainage area. The volume existing at the beginning of a historical storm is of primary interest.

Spatial Rainfall Distribution - The location variation of rainfall on a drainage area.

Spillway - The structure provided to pass flows which are generally too large to be passed through the outlet works or the power plant. The spillway may be an overflow type or an orifice type.

Spurious Trend - A trend in hydrologic data with time that appears in the data but is actually the result of data errors or other anomalies rather than a real climatic effect.

Standard Error of Estimate - The square root of the variance between values of a given hydrologic data set and a set which is normally distributed.

Storage Coefficient (R) - A coefficient used with the Clark unit hydrograph which is identified with storage effects of the basin. For estimation of this parameter see Figure 8-8.2.

Storm Transposition - The analytical process of moving historic storm data from the location where it occurred to the location of interest.

Streamflow - The record of flow rate at a particular point in a stream.

Streamgage - A gage which measures and records the water-surface elevation (stage) in a stream. The recorded stage is converted to streamflow by use of a rating curve.

Subbasin - A subdivision of a drainage area.

Subdivision - The process of dividing a drainage area into subbasins.

Synthetic Unit Hydrograph - A unit hydrograph for an ungaged basin that has been developed based on unit hydrographs developed at gage sites within a region. Synthetic unit hydrographs are estimated for ungaged basins by means of relationships between parameters of the unit-hydrograph model and the physical characteristics of the basin.

Temporal rainfall distribution - The variation of rainfall depth with time for a particular storm.

Time of Concentration - The time of concentration is defined as the time required for runoff or water to travel from the most remote point in the watershed to the outlet or point of consideration.

Thiessen Polygon Method - The method of dividing a drainage area into polygons within which the average rainfall for a given storm is equal to that recorded at the nearest raingage.

Uncontrolled Spillway - A spillway where overflow is not controlled.

Ungaged Site - One for which available hydrologic data, recorded at stations within the basin, are insufficient in quantity and quality to provide confidence in development of an inflow PMF hydrograph.

Uniform Loss Rate - The constant rate of infiltration assumed to occur. It is calculated from average soil characteristics for each subbasin.

Unit Hydrograph - The direct runoff hydrograph from a given drainage area representing one inch of precipitation excess for a specified duration.

Urban Area - An area which has been developed for urban use.

Verification Hydrograph - A hydrograph of a historic flood which is regenerated using the corresponding rainfall data and the developed unit hydrograph as a means of checking the suitability of the unit hydrograph and/or the runoff model.

Watercourse - The path which runoff follows during passage from a drainage area.

Watershed - Another term meaning drainage basin.

ABBREVIATIONS AND SYMBOLS

The following abbreviations and symbols have been used in the text of these Guidelines.

A - Drainage area.

C_p - An empirical coefficient used in the Snyder synthetic unit hydrograph which accounts for flood-wave velocity and storage in the river channel.

C_t - An empirical coefficient used in the Snyder synthetic unit hydrograph which accounts for storage and slope of the basin.

R - Hydraulic radius of a channel.

S - Slope of a stream channel or a basin.

L_{ca} - The distance from the basin outlet to a point opposite the centroid of the drainage area, measured along the principal or main stream in the basin.

L - The distance from the basin outlet to the top of the drainage divide measured along the longest watercourse.

QRCSN - The flow rate on a flood hydrograph at which direct runoff ceases. This is one of three terms used in the optimization calculations performed by HEC-1.

RTIOR - Is equal to the ratio of a recession limb flow to the recession limb flow occurring one hour later. A recession characteristic used in the calculation of baseflow.

STRTQ - The flow rate in the river at the time hydrograph simulation begins. This is one of three terms used in base flow calculations performed by HEC-1.

T_c - Time of concentration for a drainage area.

T_L - Lag time for a drainage area.

t_p - Time to peak for a drainage area.

T_r - Duration of rainfall used in the Snyder unit hydrograph.

8-14 Appendices

Appendix VIII-A Determining the Probable Maximum Flood for Civil Works Flow Chart

Appendix VIII-B Probable Maximum Flood Study Report Outline

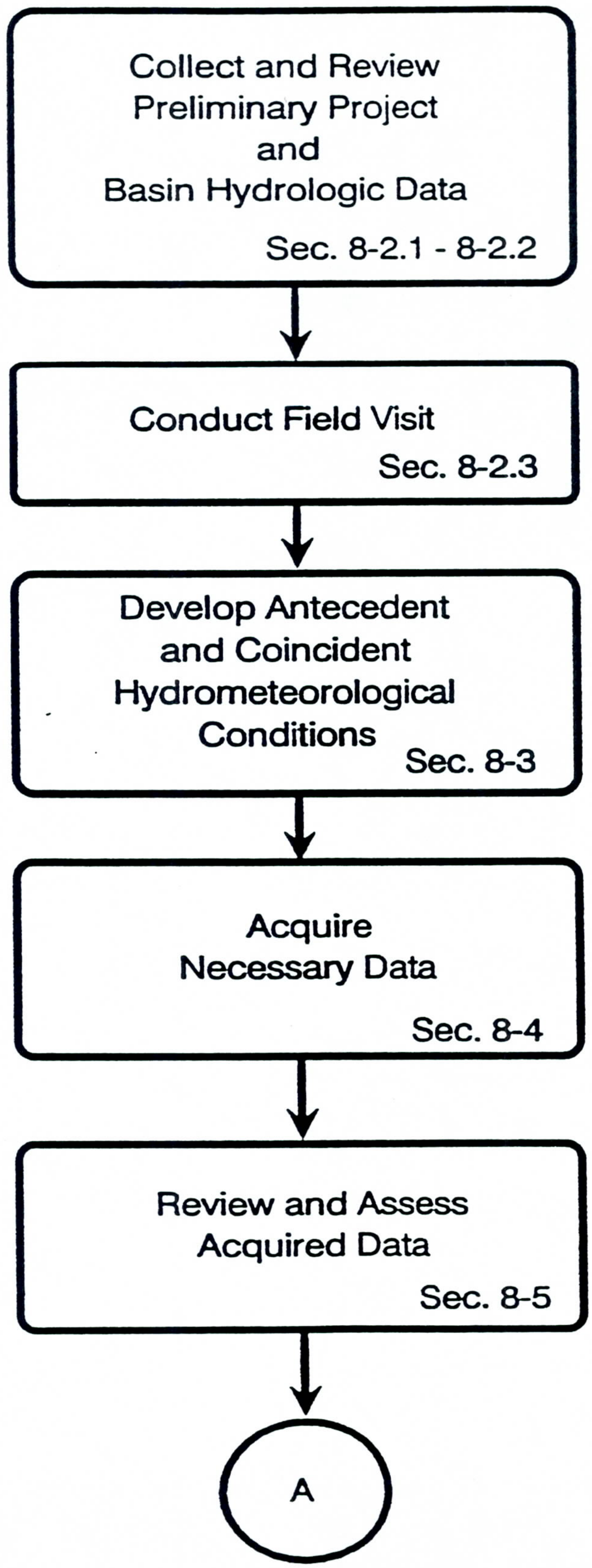
Appendix VIII-C Loss Rates for Subbasins - Detailed Method (8-10.3.2)

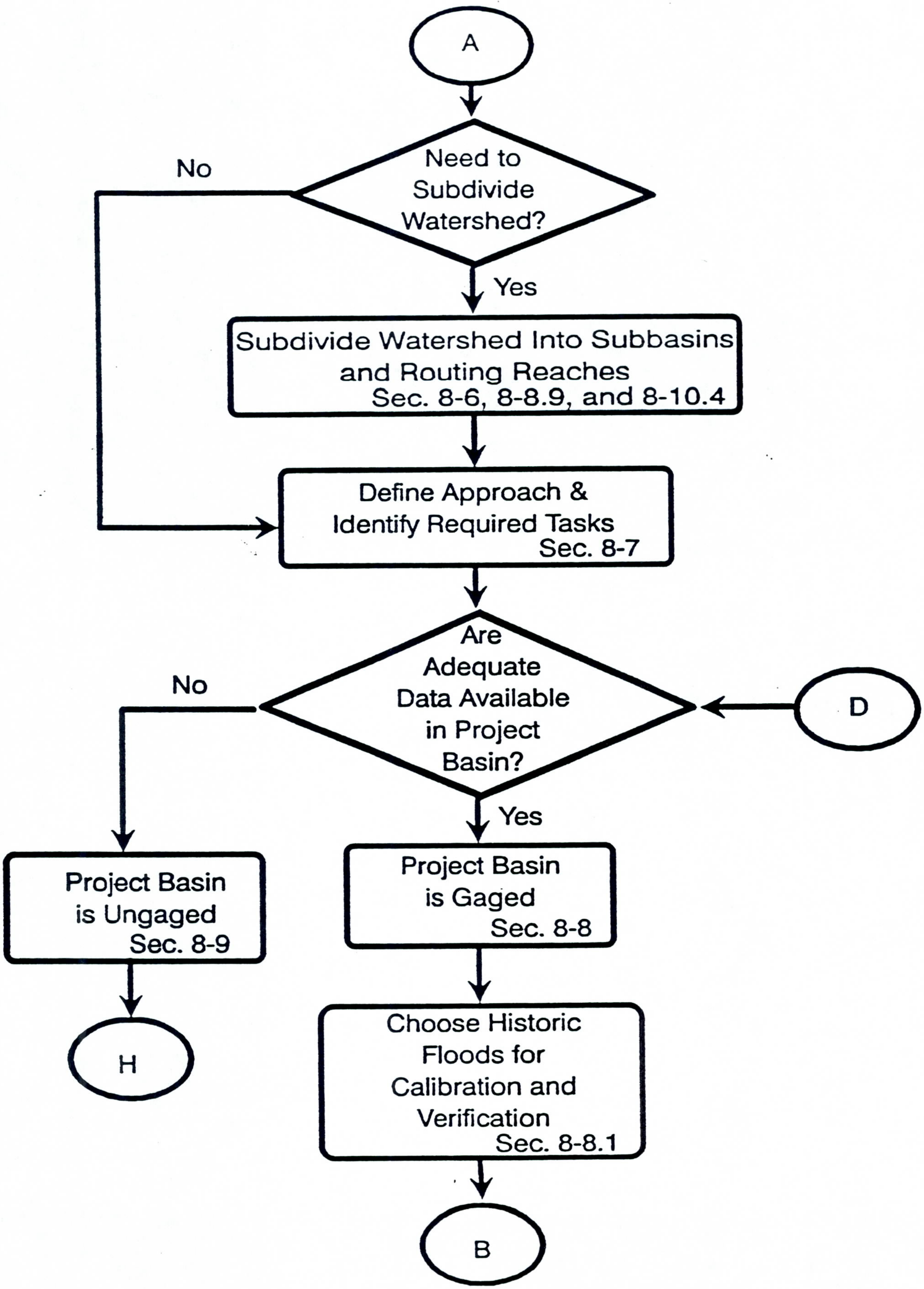
Appendix VIII-A

Determining the Probable Maximum Flood for Civil Works Flow Chart

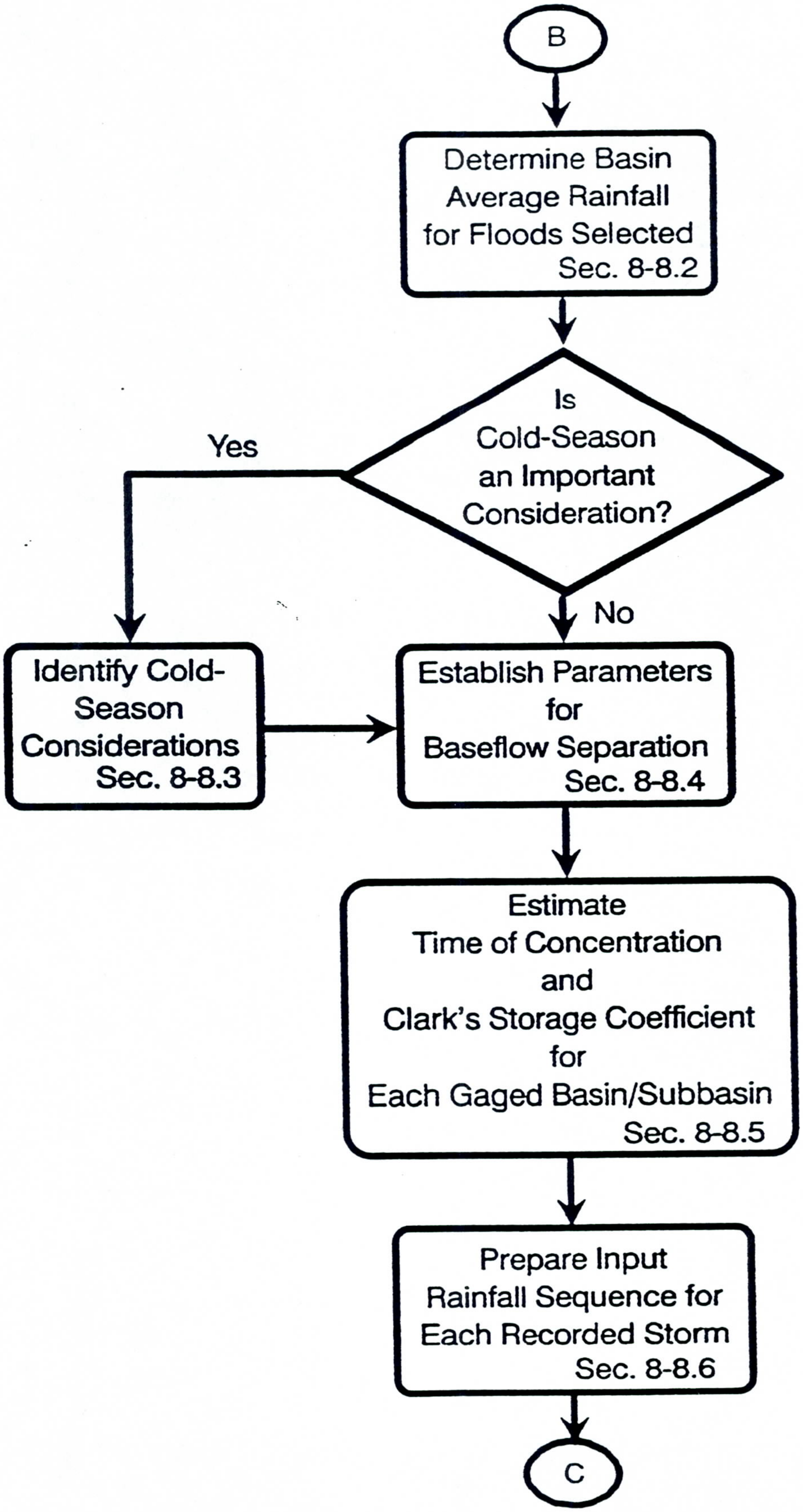
The flow chart shows the sequence of decisions and analyses required in determining the PMF for gaged and ungaged basins. PMF studies should follow the procedures specified in the flow chart, unless departures are justified in the study report. Chapter and section references are shown for each flow chart element.

DETERMINING THE PROBABLE MAXIMUM FLOOD FOR CIVIL WORKS
FLOW CHART

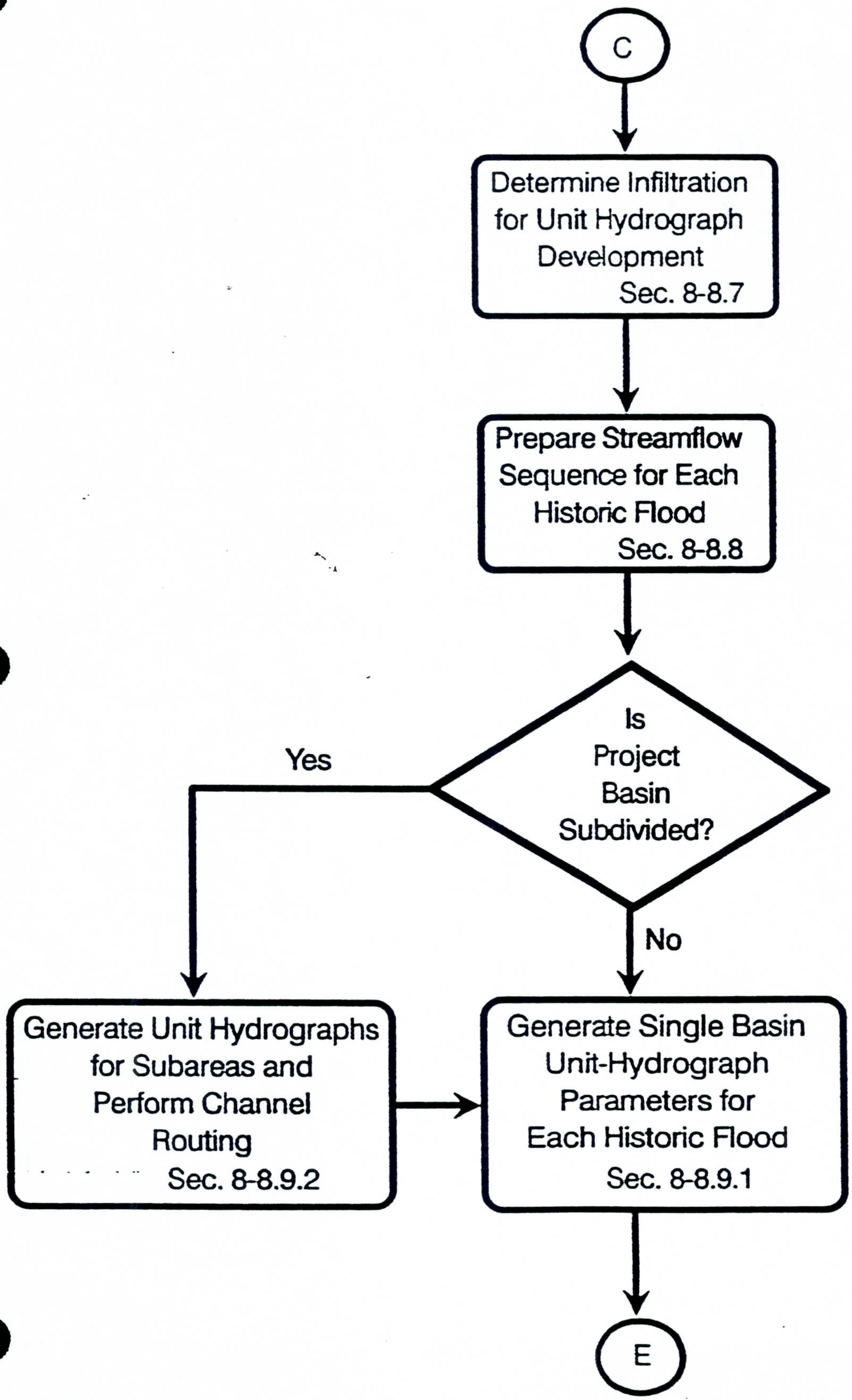



A
Need to Subdivide Watershed?
No
Yes
Subdivide Watershed Into Subbasins and Routing Reaches Sec. 8-6, 8-8.9, and 8-10.4
Define Approach & Identify Required Tasks Sec. 8-7
Are Adequate Data Available in Project Basin?
No
Yes
D
Project Basin is Ungaged Sec. 8-9
H
Project Basin is Gaged Sec. 8-8
Choose Historic Floods for Calibration and Verification Sec. 8-8.1
B

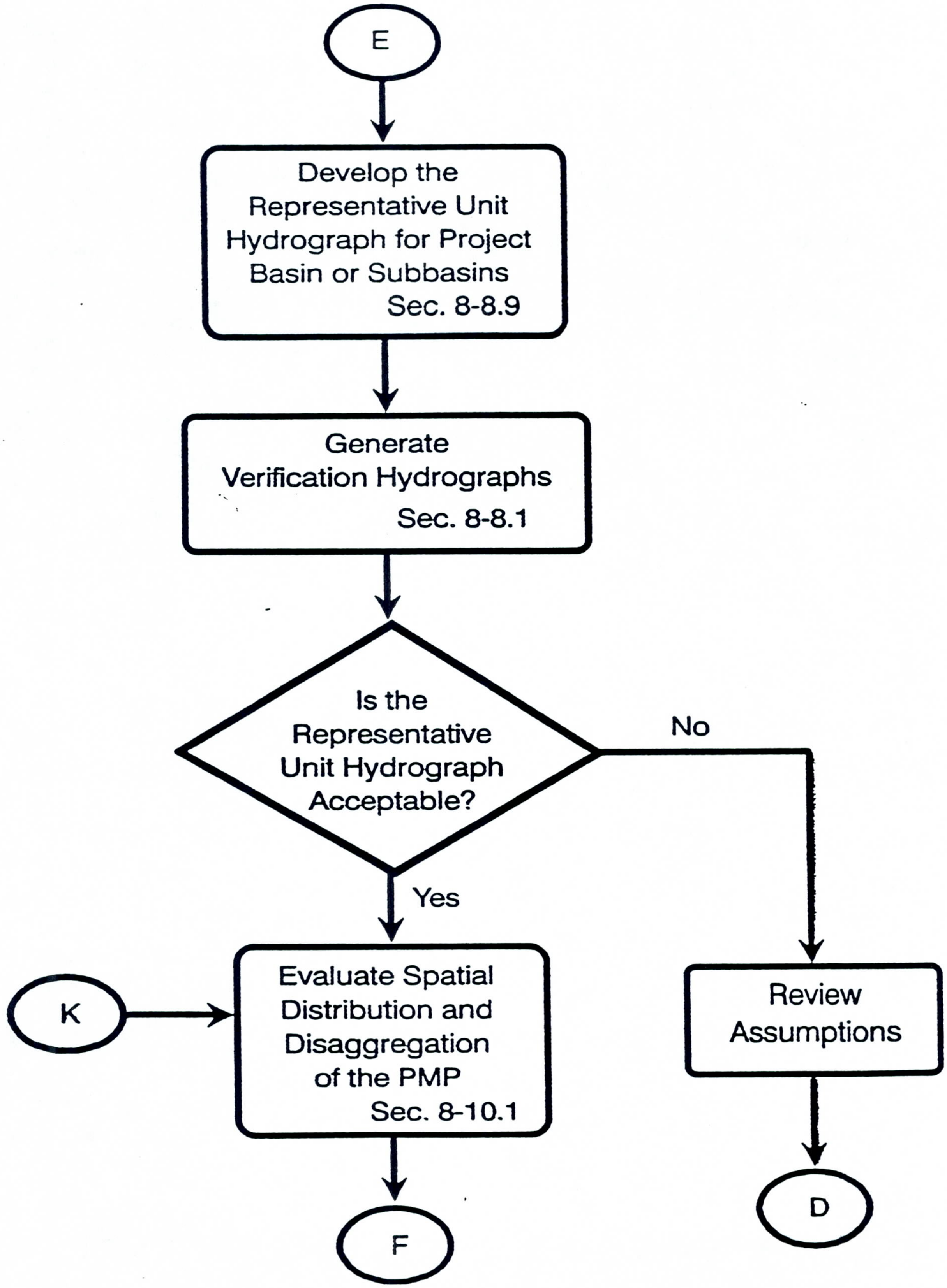
PMF DEVELOPMENT
(Gaged Basins)



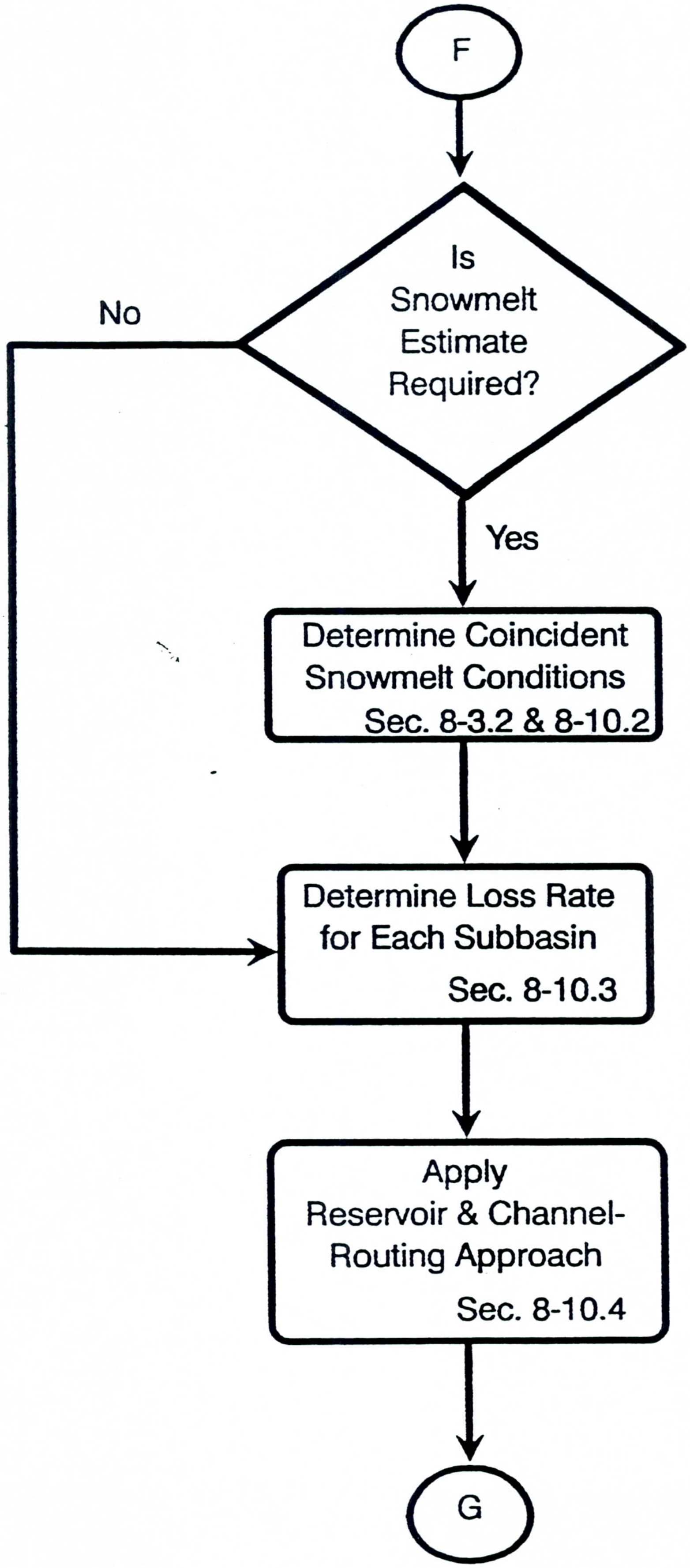
PMF DEVELOPMENT
(Gaged Basins)



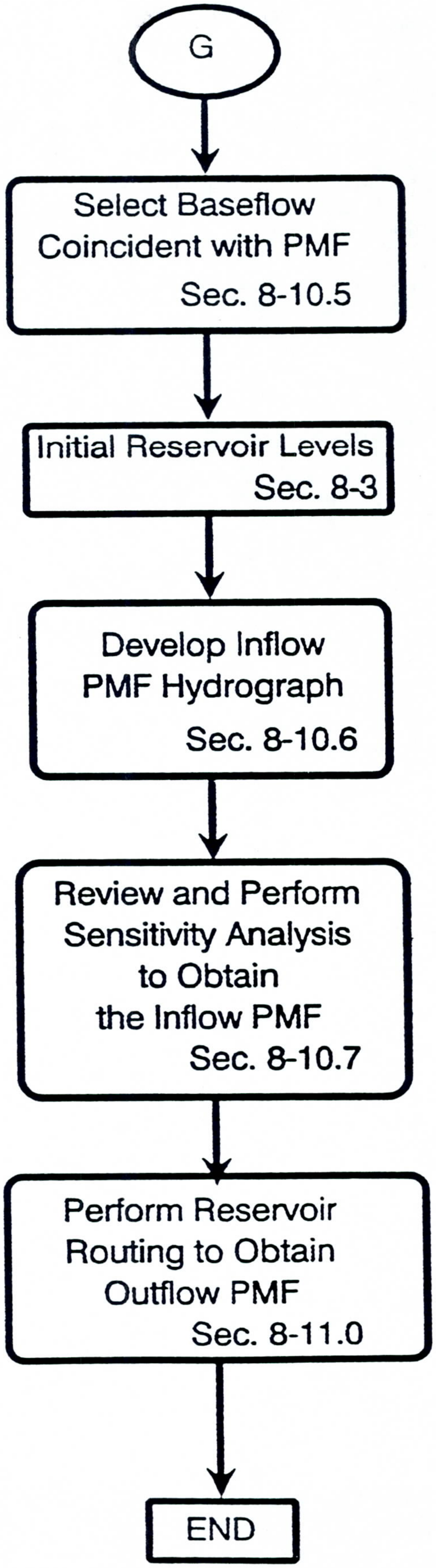
PMF DEVELOPMENT
(Gaged Basins)



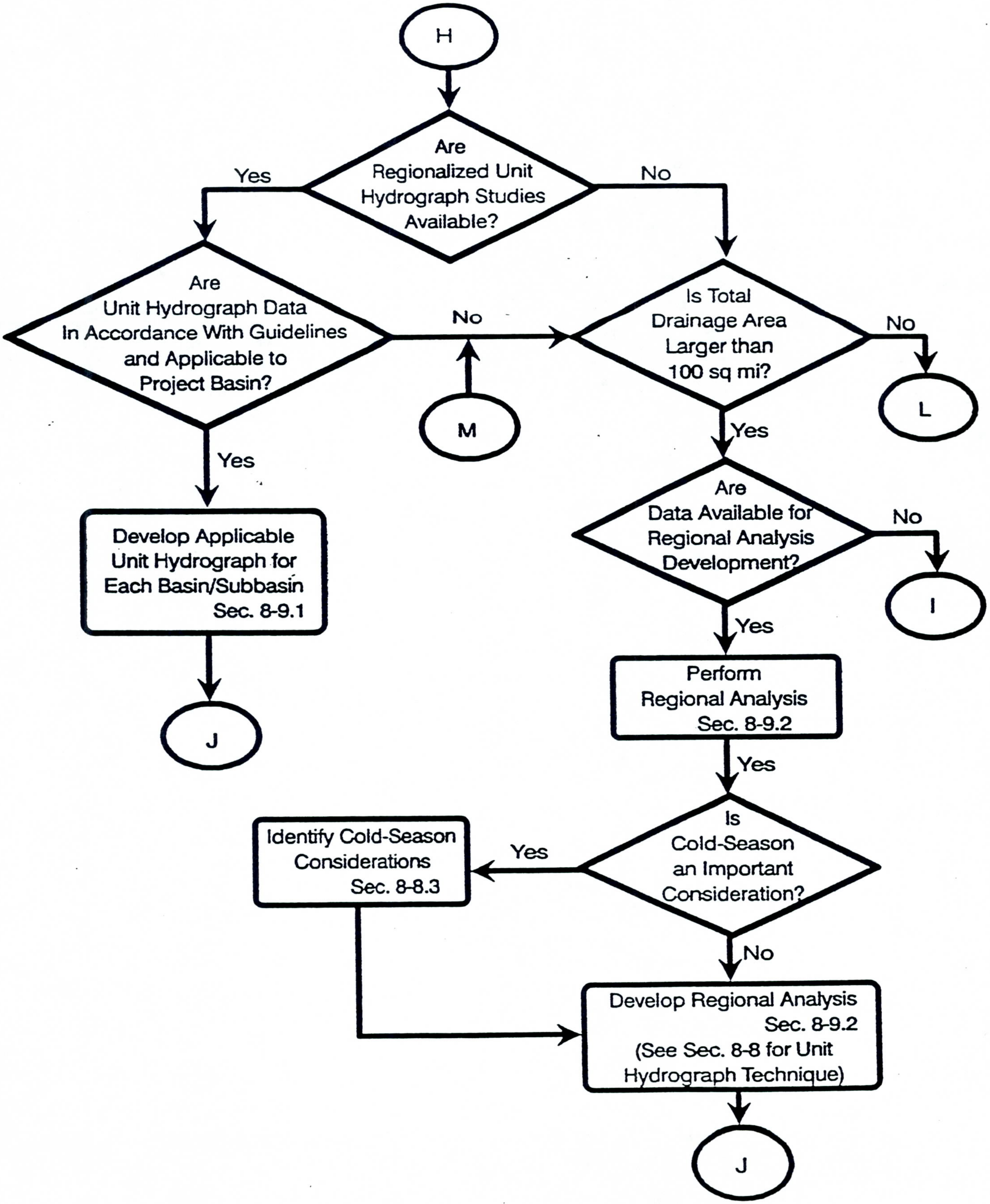
PMF DEVELOPMENT
(Gaged Basins)



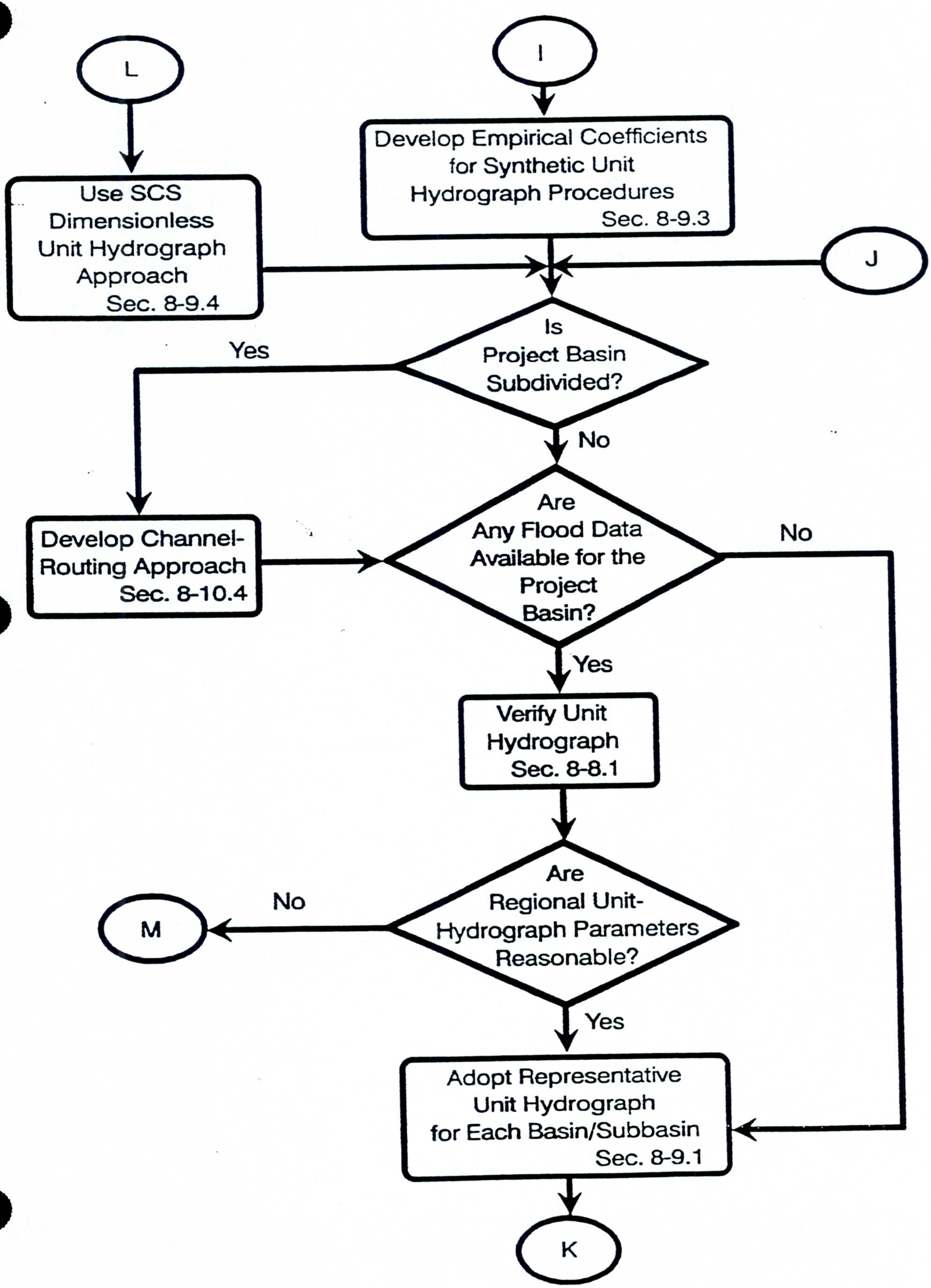
PMF DEVELOPMENT
(Gaged Basins)



PMF DEVELOPMENT
(Ungaged Basins)



PMF DEVELOPMENT
(Ungaged Basins)



Appendix VIII-B

Probable Maximum Flood Study Report Outline

The following study report outline should assist the analyst in documenting PMF studies. The outline parallels the reasoning in Chapter VII and the flow chart, except that some subject areas are consolidated to avoid repeating information in the written report. When subject headings are not applicable to the study, an explanation should be provided.

PMF STUDY REPORT OUTLINE

GAGED BASINS

PMF STUDY REPORT OUTLINE

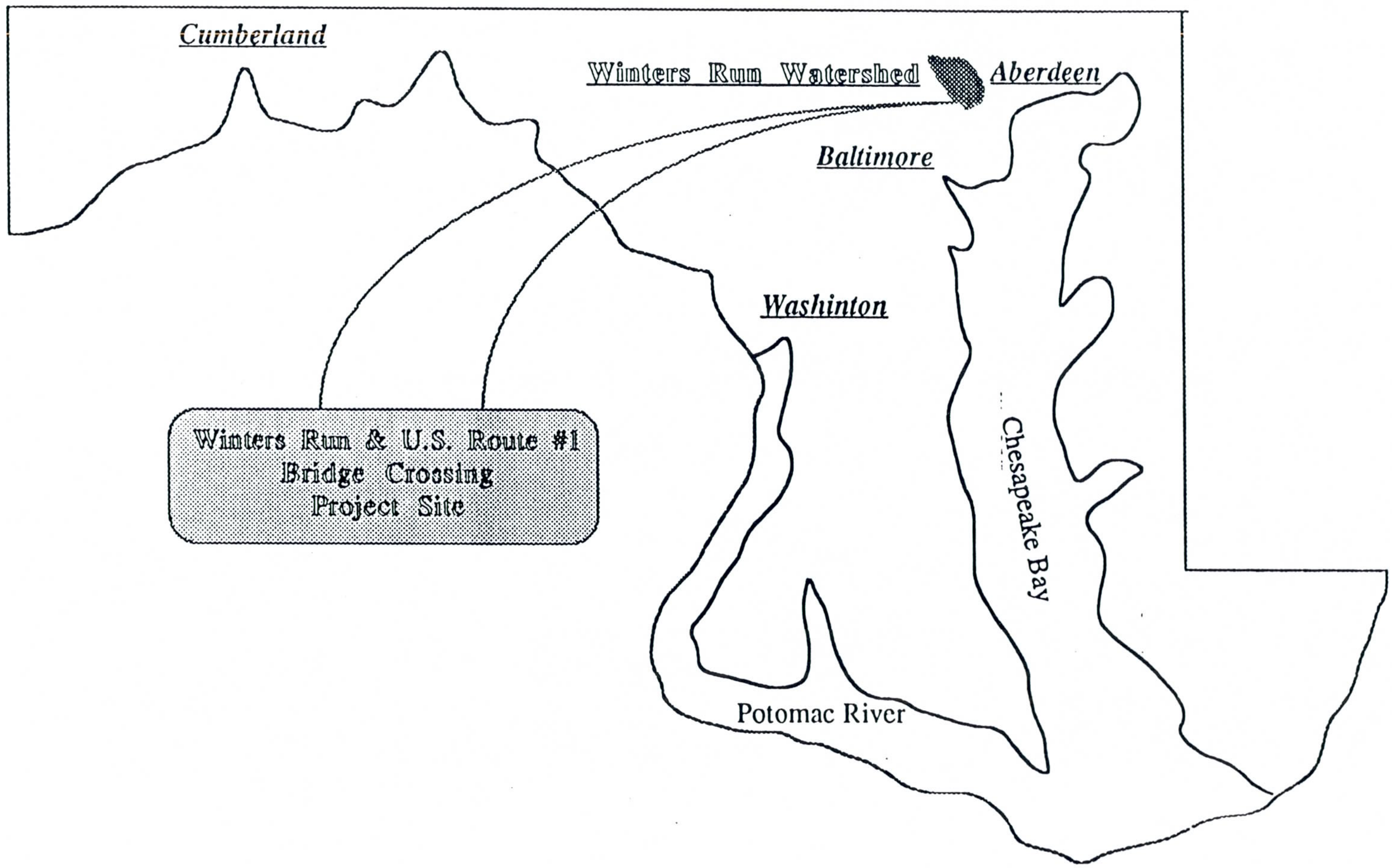
UNGAGED BASINS

REVIEW AND QUESTIONS

Wednesday 10:10 a.m.

LIMITATIONS OF UNIT HYDROGRAPH THEORY

Wednesday 11:00 a.m.


Cumberland
Winters Run Watershed
Aberdeen
Baltimore
Washinton
Winters Run & U.S. Route #1
Bridge Crossing
Project Site
Chesapeake Bay
Potomac River

Section 1: WATERSHED CHARACTERISTICS

The following sections provide a brief description of the topography and soil characteristics of the Winters Run watershed as well as the location of the USGS gaging station used for this study.

1.1 Physiography\Topography

The Winters Run watershed above the U.S. Route 1 crossing is located approximately 25 miles northeast of Baltimore, in the Piedmont region of Maryland. The terrain consists of gently rolling hills with elevations ranging from approximately 175 feet msl. at the U.S. Route 1 crossing to approximately 750 feet msl. in the upper northwest corner of the watershed near Madonna, MD. The watershed has a drainage area of 35.09 square miles. The main channel of Winters Run is roughly 13 miles in length and, overall, the watershed has approximately 64 miles of streams. For this study, the watershed was divided into 9 subareas (see figure 1). Slopes range from 0 to 10 percent with some isolated areas being higher. The mean annual precipitation is 44 inches (for the years 1931 through 1961 inclusive).

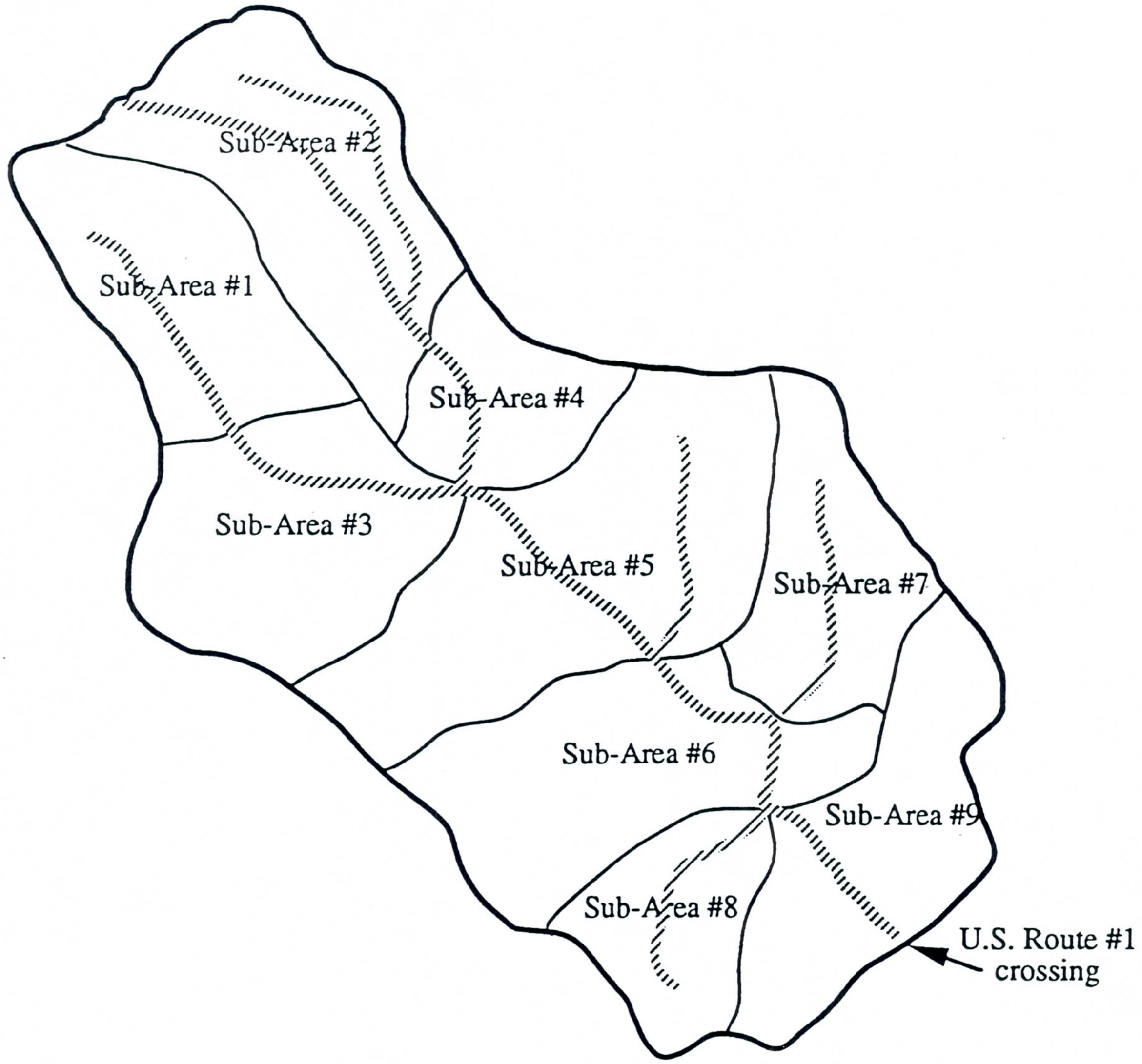


Figure 1 - Winters run watershed

1.2 Soils

A major portion of the watershed in the study area is comprised of soils of the Manor-Glenelg association. In addition, soils of the Chester-Glenelg-Manor, Glenelg Manor, Neshaminy-Aldino-Watchung, Montalto-Neshaminy-Aldino, and Codorus-Hatboro-Alluvial land associations are found in the watershed. All of the associations are of the Piedmont Plateau with the exception of the Codorus-Hatboro-Alluvial land association, which is considered to be a soil of the Floodplains and Low Terraces. The individual associations are characterized as follows:

* Manor-Glenelg - Deep, steep to gently sloping, somewhat excessively drained and well drained soils that are underlain by acid crystalline rock; on uplands.

* Chester-Glenelg-Manor - Deep, nearly level to steep, well drained and somewhat excessively drained soils that are underlain by acid crystalline rock; on uplands having broad ridge tops.

* Glenelg Manor - Deep, gently sloping to steep, well drained and somewhat excessively drained soils that are underlain by acid crystalline rock on uplands.

* Neshaminy-Aldino-Watchung - Deep, steep to nearly level, well drained to poorly drained soils that are underlain by basic, semibasic, or mixed basic and acidic rocks; on uplands having many broad flats.

* Montalto-Neshaminy-Aldino - Deep, steep to nearly level, well drained and moderately well drained soils that are underlain by basic, semibasic, or mixed basic and acidic rocks; on uplands.

* Codorus-Hatboro-Alluvial - Deep, nearly level, moderately well drained to poorly drained soils that are underlain by stratified alluvial sediment; on flood plains.

1.3 USGS Stream Gaging Station

The USGS stream gaging station (No. 01581700 and referred to as the Benson site) used in this study is located on Winters Run approximately 30 feet downstream U.S. Route 1 bridge crossing, 0.1 mile upstream of Heavenly Waters, and 10.5 miles upstream of the mouth at the Chesapeake Bay. Coordinates of the gaging station are Latitude 39°31'12" and Longitude 76°22'24". Data from this gaging site was used for the statistical analysis and unit hydrograph development portions of this report.

PEAK FLOW
and
CORRESPONDING YEAR

YEAR	FLOW (cfs)
1967	3350
1968	4300
1969	364
1970	1880
1971	5350
1972	7600
1973	1600
1974	1440
1975	3750
1976	5190
1977	1760
1978	4950
1979	5510
1980	2230
1981	632
1982	1230
1983	1480
1984	7280
1985	5230
1986	595
1987	5460
1988	2020
1989	4730
1990	2260

STORM DATA FROM USGS and NWS

STORM NUMBER	BEGINNING TIME AND DATE	ENDING TIME AND DATE
1	1200 hrs. - 05/03/84	2400 hrs. - 05/05/84
2	1900 hrs. - 11/28/84	2400 hrs. - 11/30/84
3	2000 hrs. - 09/26/85	2400 hrs. - 09/28/85
4	2100 hrs. - 02/11/88	0400 hrs. - 02/14/88
5	1900 hrs. - 04/27/88	1200 hrs. - 04/29/88
6	0100 hrs. - 03/23/89	2400 hrs. - 03/26/89

Criteria used in selecting the storms to be analyzed:

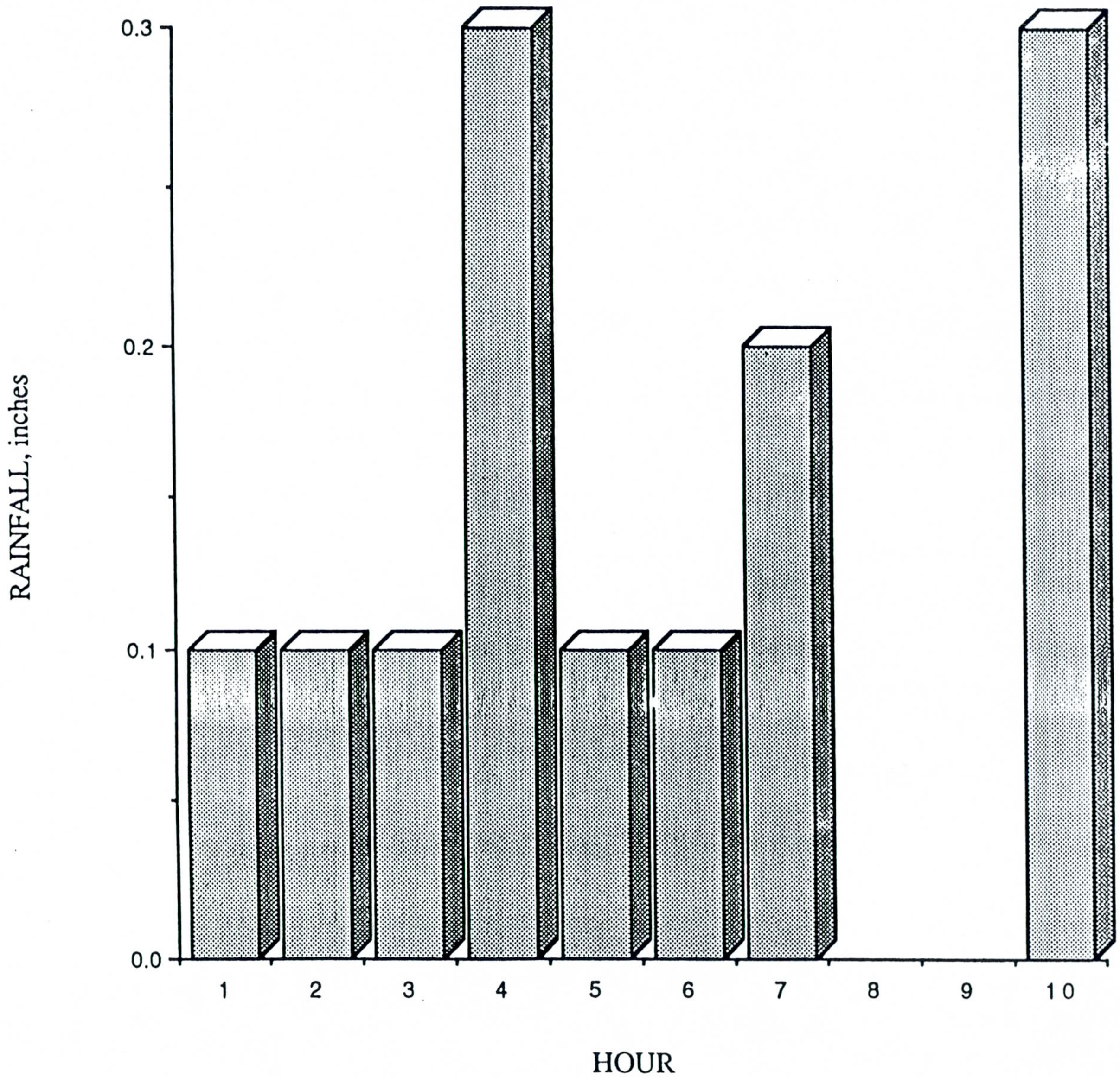
1> The runoff event had to be a single peak (i.e. complex runoff events were eliminated).

2> The rainfall excess should be continuous over some time interval.

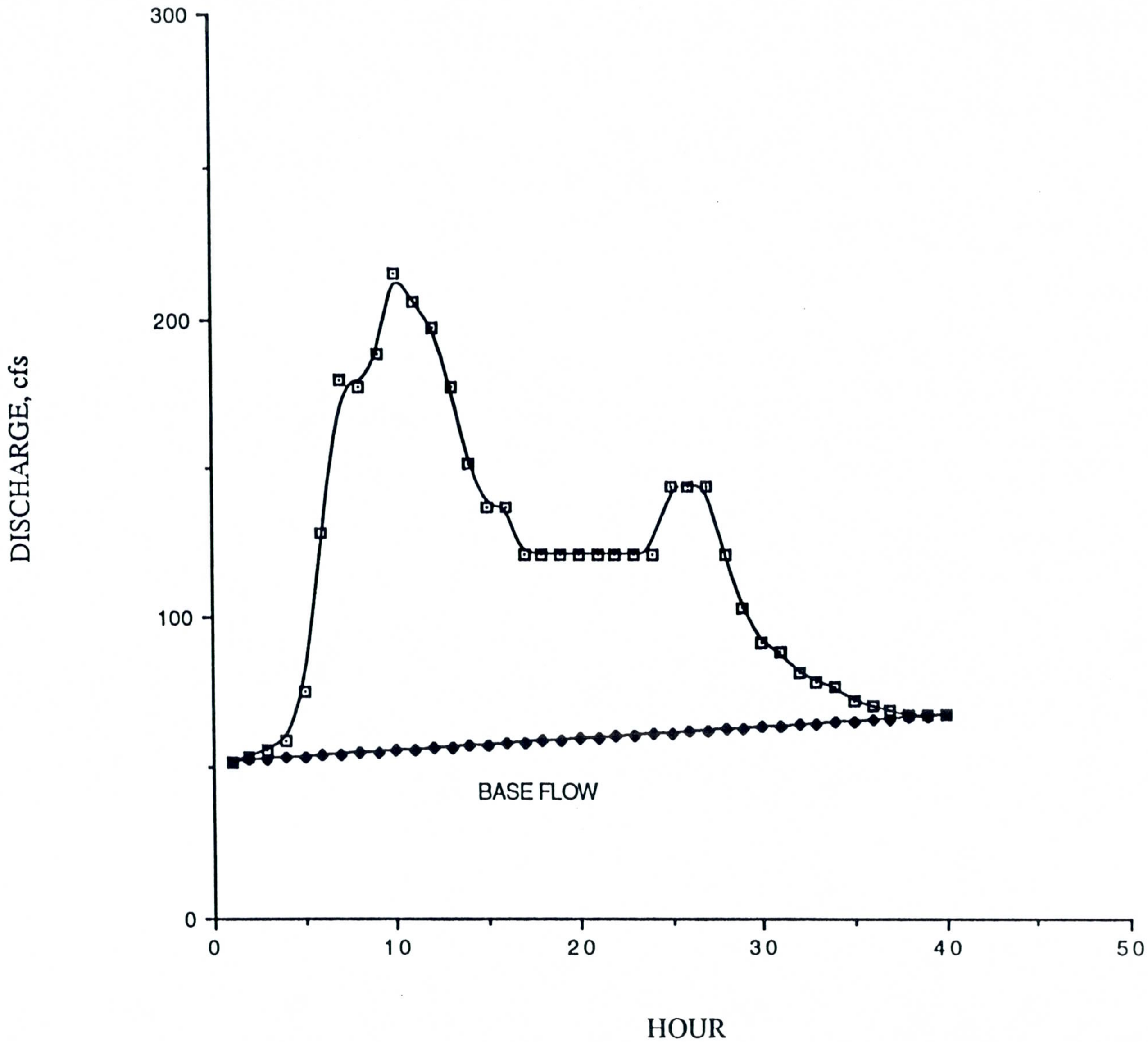
3> The storms were limited to the past ten years to ensure watershed homogeneity.

NOTE: Daily precipitation records are available from the Benson gaging site but with no hourly records. There is; however, a recording raingage station near Aberdeen, MD. with hourly data available. For the purposes of this study, it was assumed that the storm events would follow similar distribution patterns at Aberdeen and Benson. The daily precipitation from the Benson gaging station was, therefore, distributed according to the Aberdeen gaging station hourly distribution.

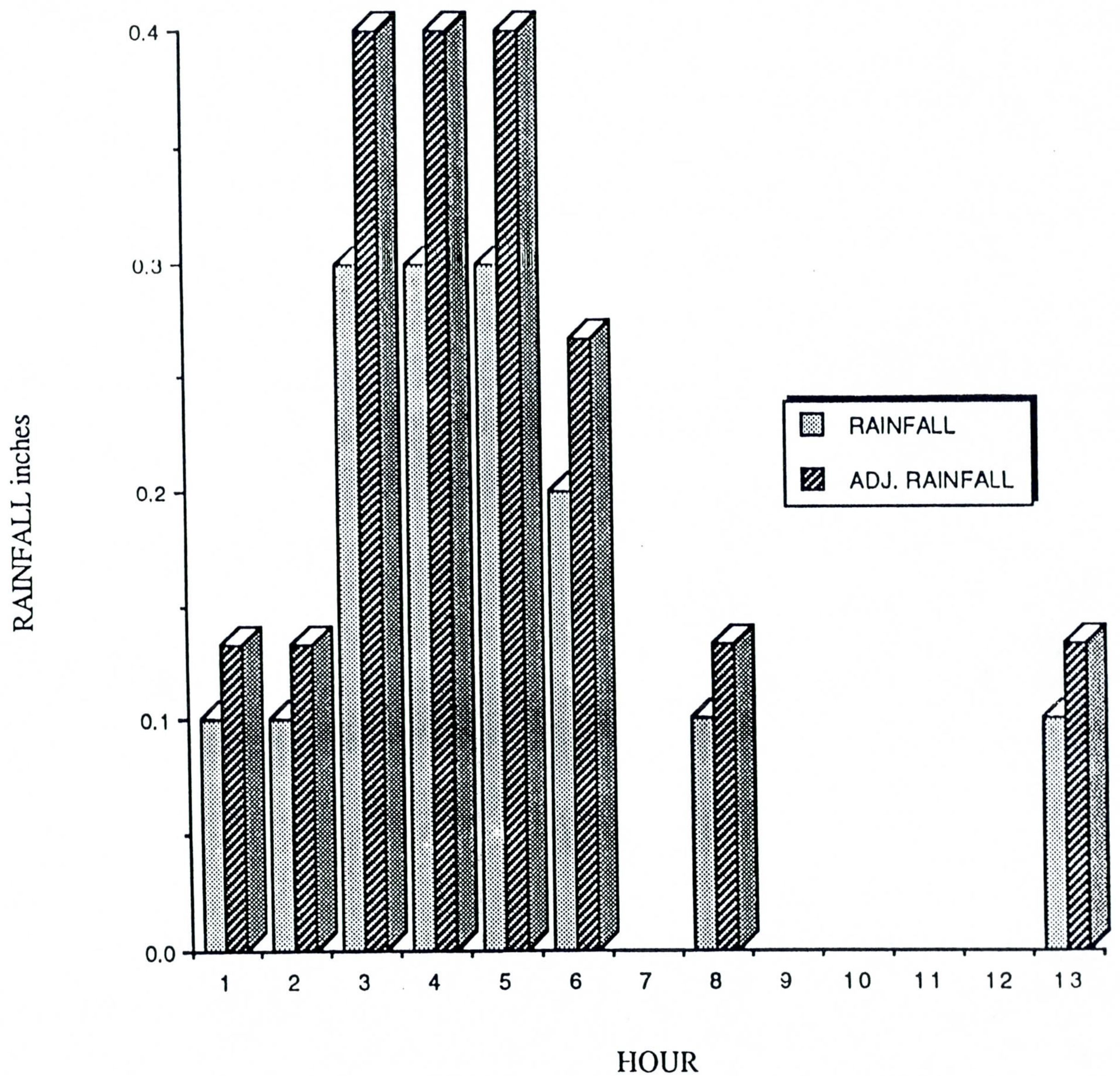
RAINFALL PATTERN - STORM #1



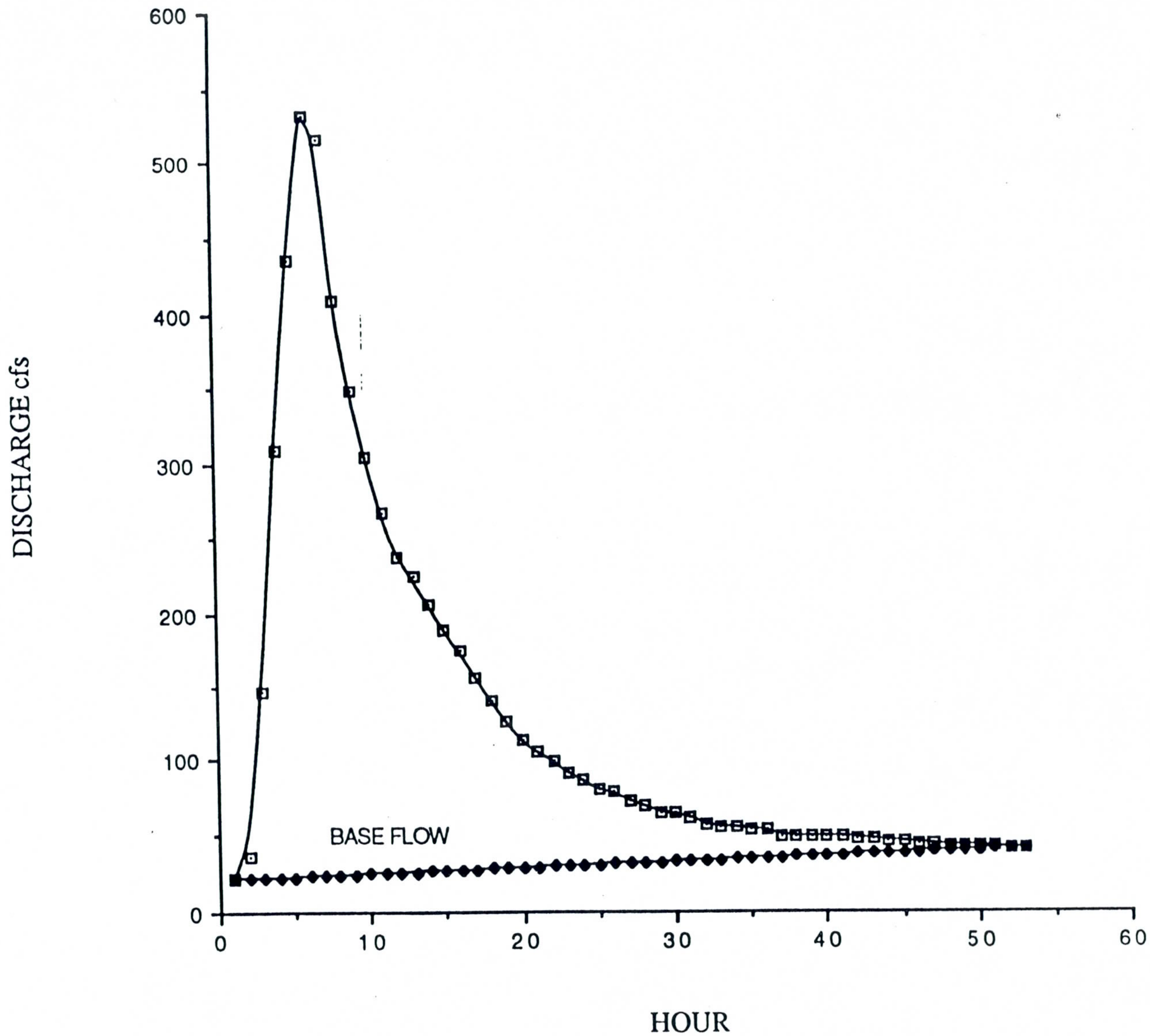
RUNOFF HYDROGRAPH - STORM #1



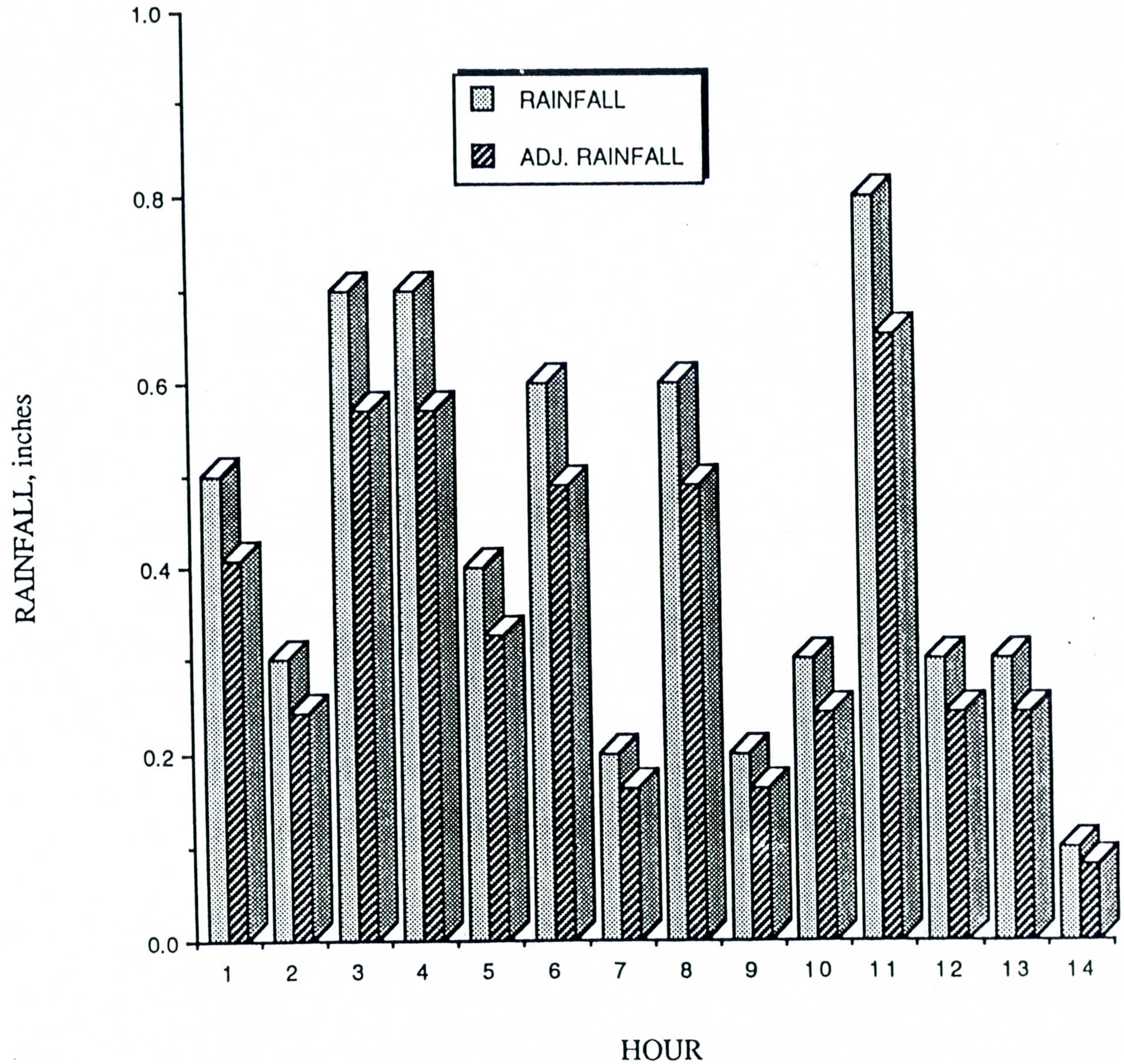
RAINFALL PATTERN - STORM #2



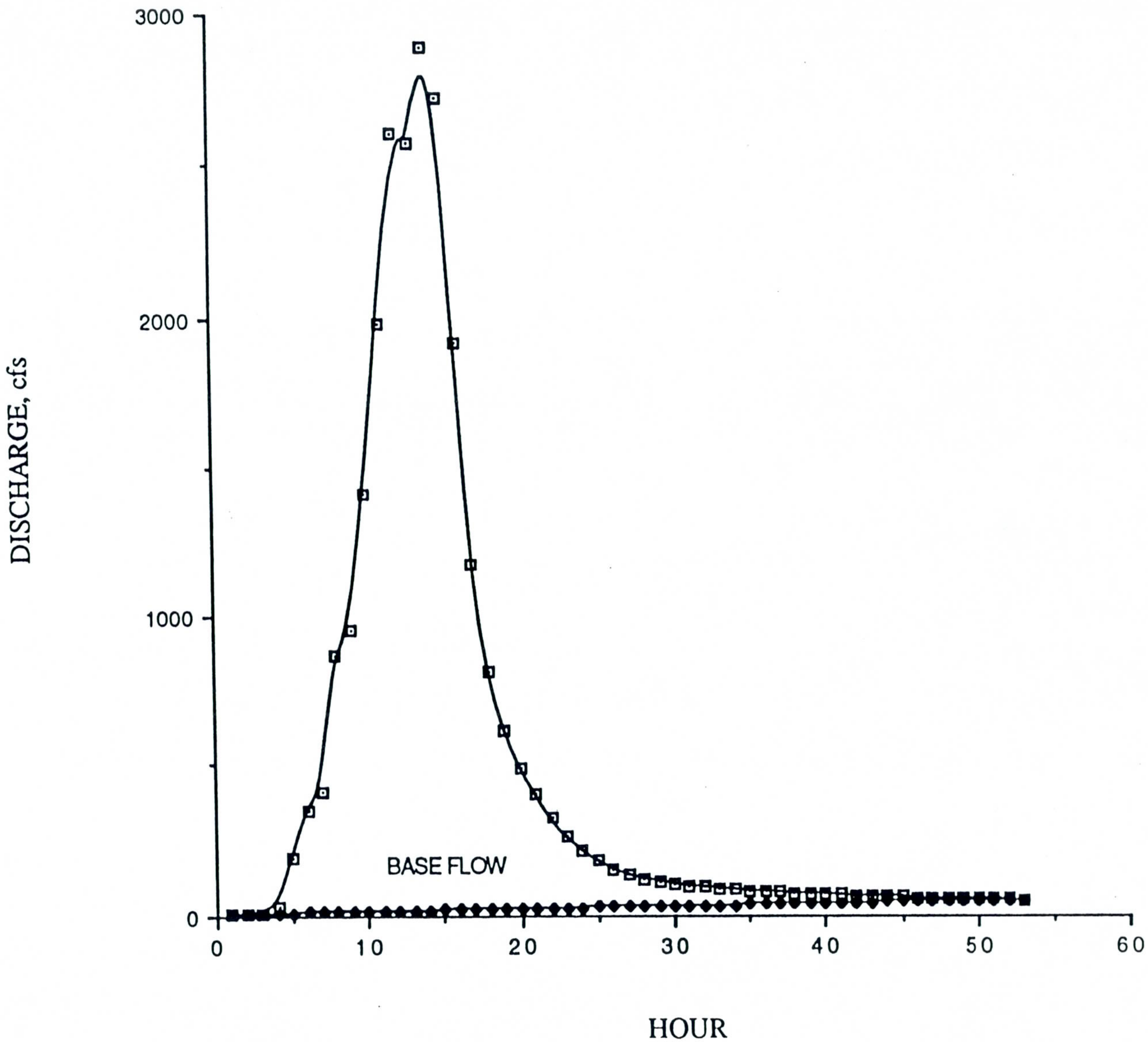
RUNOFF HYDROGRAPH - STORM #2

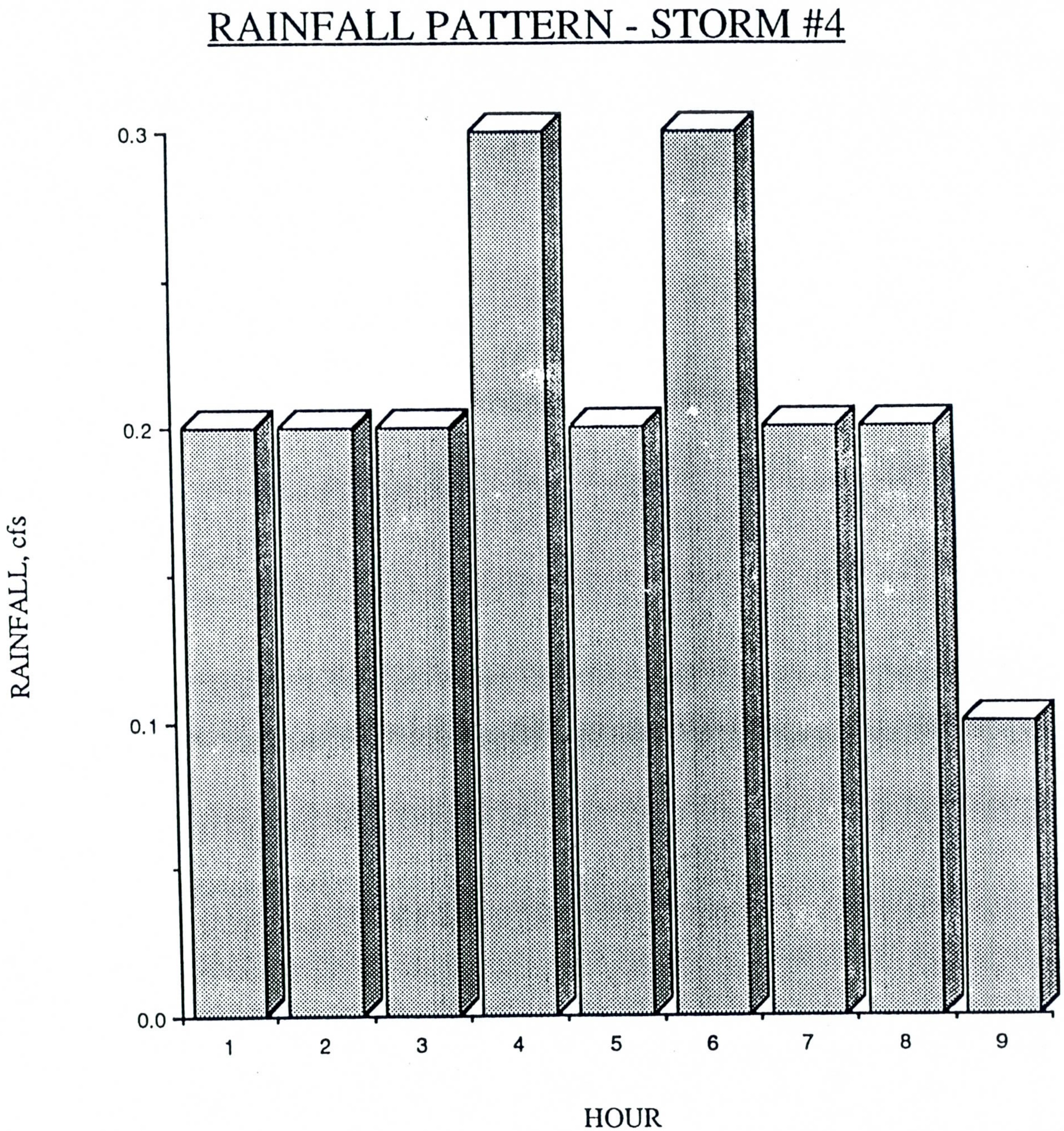


RAINFALL PATTERN - STORM #3

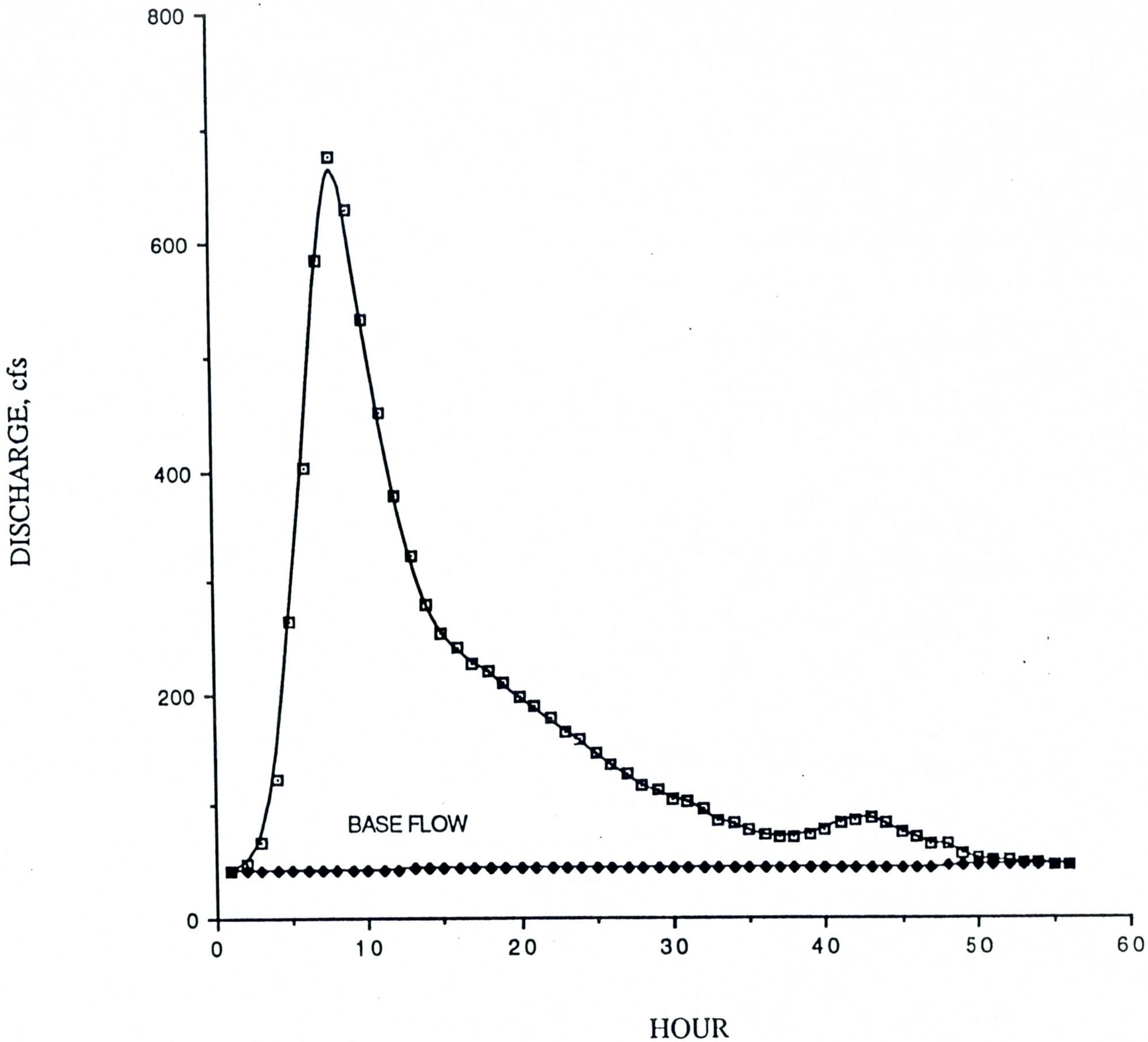


RUNOFF HYDROGRAPH - STORM #3

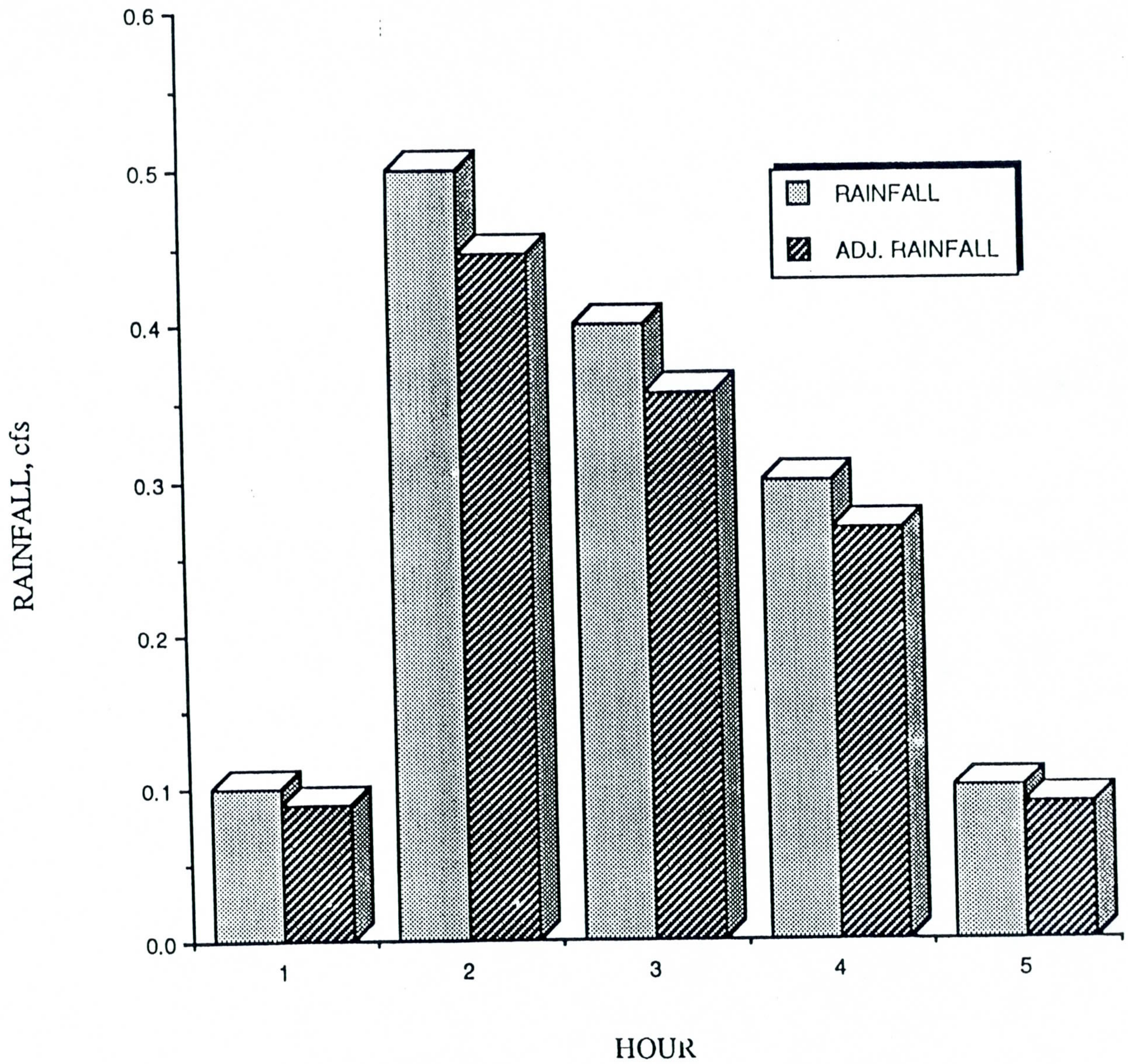



RAINFALL PATTERN - STORM #4
0.3
0.2
0.1
0.0
RAINFALL, cfs
1
2
3
4
5
6
7
8
9
HOUR

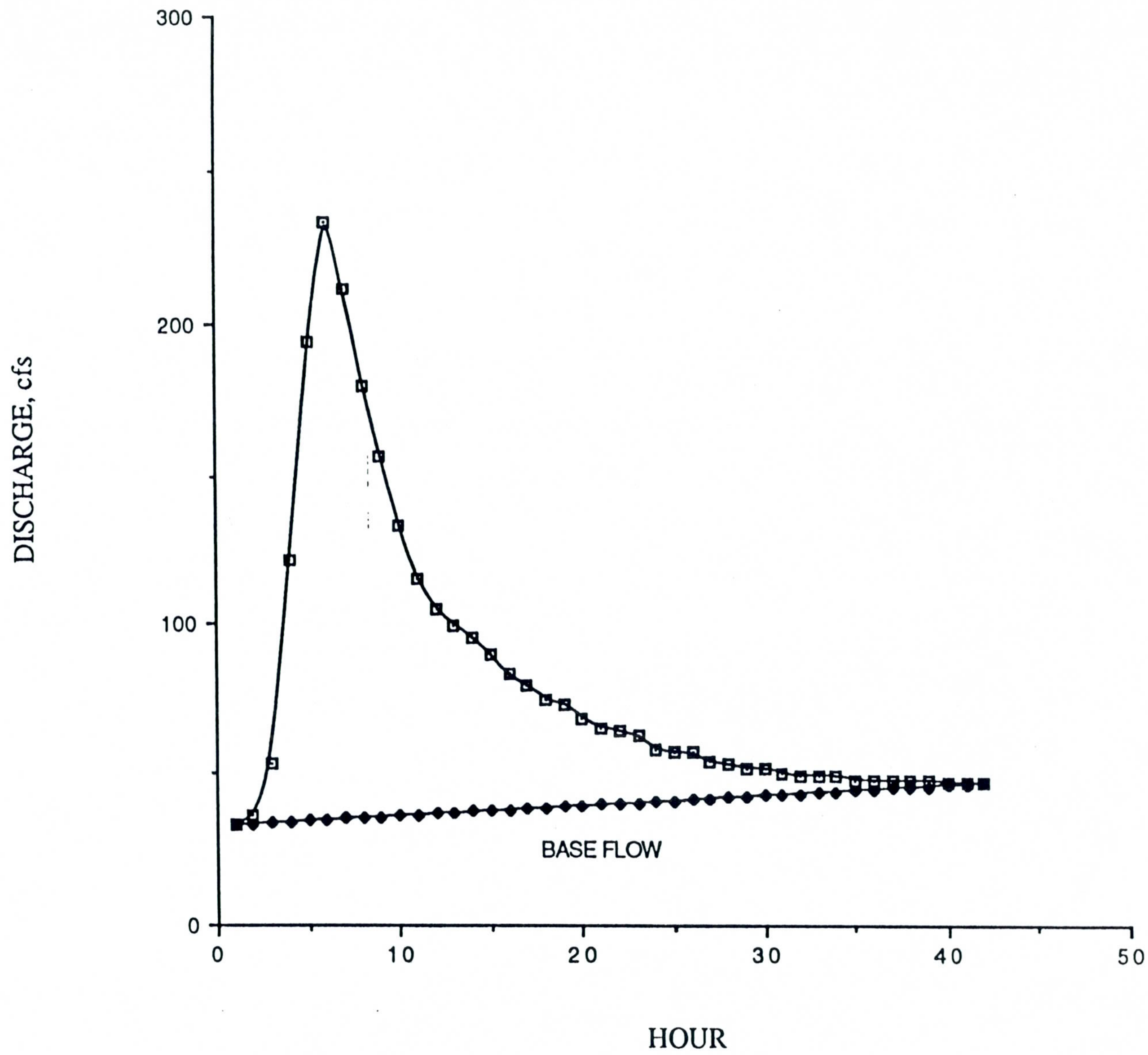
RUNOFF HYDROGRAPH - STORM #4



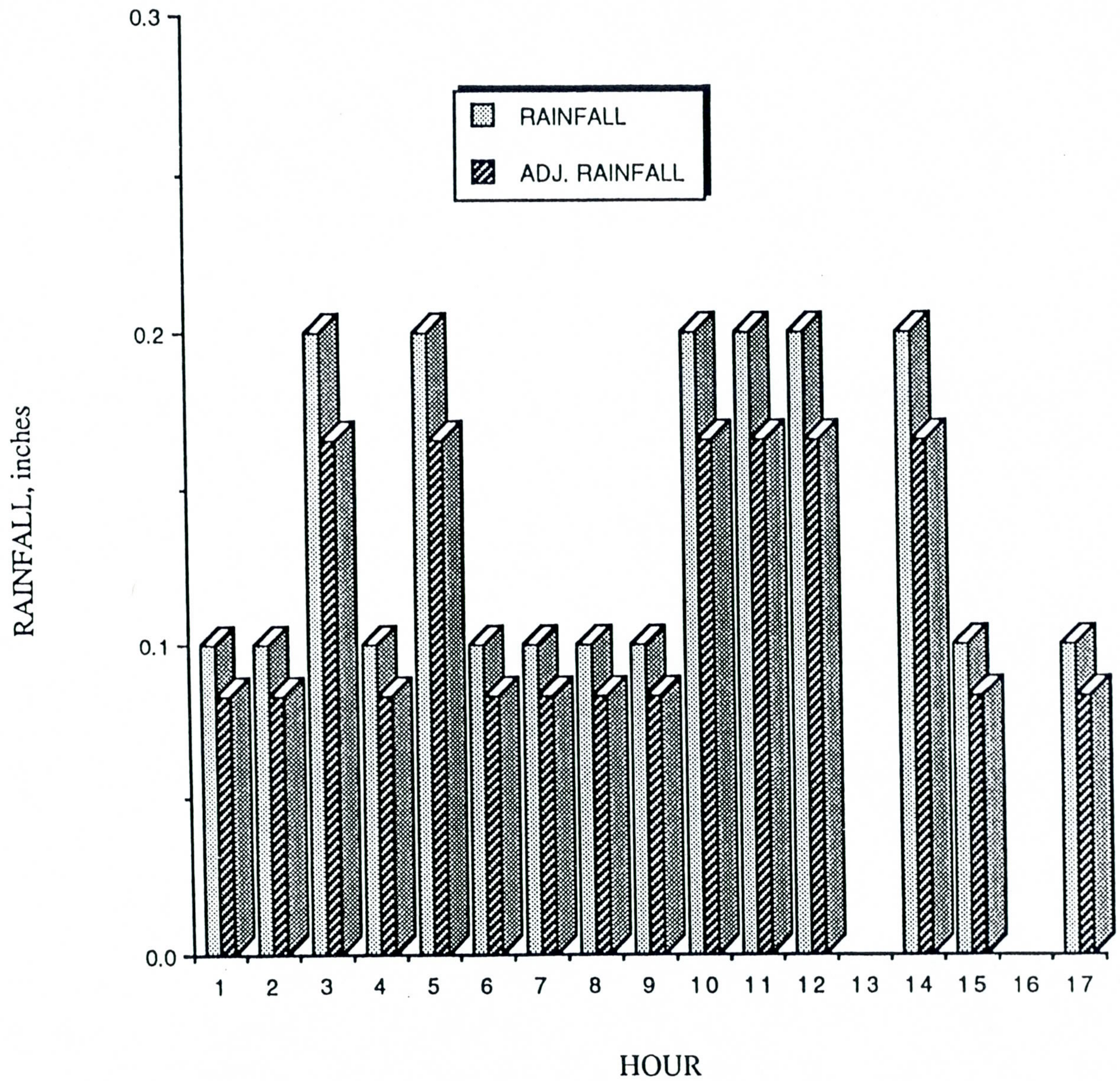
RAINFALL PATTERN - STORM #5



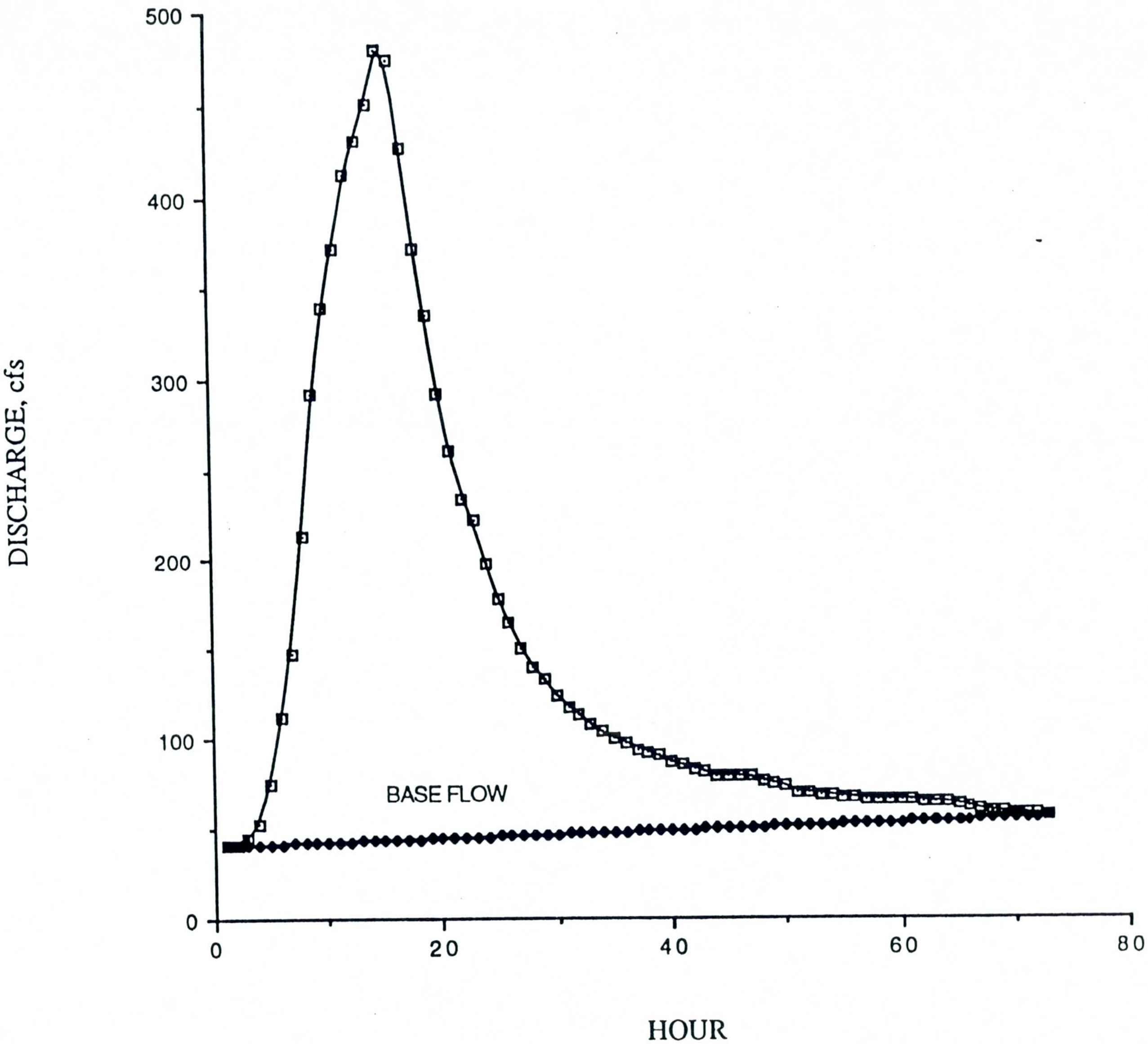
RUNOFF HYDROGRAPH - STORM #5



RAINFALL PATTERN - STORM #6



RUNOFF HYDROGRAPH - STORM #6



Snyder Synthetic Unit Hydrograph Method

The Snyder Synthetic unit hydrograph development relies upon a correlation of the dependent variables of lag time, peak discharge, and physiographic watershed characteristics. The following lag-time relationship was developed by Snyder for watersheds ranging from 10 to 10,000 sq. mi.

$$t_l = C_t (L\,L_{ca})^{0.3} \qquad (5)$$

where:

- t_l = lag time (hrs).
- C_t = coefficient representing variations of watershed slopes and storage.
- L = length of the main stream channel from the outlet to the divide (miles).
- L_{ca} = length of the main channel from the outlet to a point nearest the watershed centroid (miles).

Snyder's equation for peak discharge is as follows:

$$Q_p = \frac{640\, C_p\, A}{t_l} \qquad (6)$$

where:

- Q_p = peak discharge (cfs).
- C_p = coefficient accounting for flood wave and storage conditions. It is a function of lag time, duration of runoff producing rainfall, effective area contribution to peak flow, and drainage area.
- A = watershed area (sq. mi.)
- t_l = lag time (hrs).

WINTERS RUN DATA FOR SNYDER'S METHOD

SUB-AREA No.	AREA (mi^2)	LENGTH of MAIN CHANNEL (miles)	LENGTH TO CENTROID (miles)	CURVE NUMBER
1	3.44	2.86	1.36	75
2	5.63	7.12	4.68	72
3	6.11	3.94	1.99	72
4	4.43	3.31	1.97	72
5	5.09	2.61	1.12	72
6	3.06	2.65	3.43	80
7	3.55	3.79	1.89	75
8	1.68	2.50	1.42	80
9	2.10	1.03	0.28	80
TOTAL AREA	35.09	13.09	6.96	75

C_p and C_t ESTIMATES BY VARIOUS METHOD

METHOD	C_p	C_t
PSU Study	0.37	1.15
Optimization	0.10	0.30
Textbook Est.	0.40	0.60
FERC	?	?

SCS Dimensionless Unit Hydrograph Method

The Soil Conservation Service's unit hydrograph method is based on a dimensionless unit hydrograph. This hydrograph is the result of an analysis of a large number of natural unit hydrographs from a wide range of sizes and geographic locations. This method employs the following two equations for time-to-peak and peak discharge, respectively:

$$t_p = \frac{D}{2} + t_l \tag{7}$$

where:

t_p = time from beginning of rainfall to peak discharge (hrs).
D = duration of rainfall (hrs).
t_l = lag time from the centroid of the rainfall to the peak discharge (hrs).

and

$$q_p = \frac{484\ A}{t_p} \tag{8}$$

where:

q_p = peak discharge (cfs).
A = drainage area (mi^2).
t_p = the time to peak (hr).

Equation 9 is often used by the SCS to compute lag time and will be used as one of the methods for computing lag time in this study:

$$t_l = \frac{l^{0.8} (S + 1)^{0.7}}{1900\ Y^{0.5}} \tag{9}$$

where:

t_l = the lag time (hrs).
l = length to divide in feet
Y = average watershed slope in percent
S = the potential maximum retention (in) where:

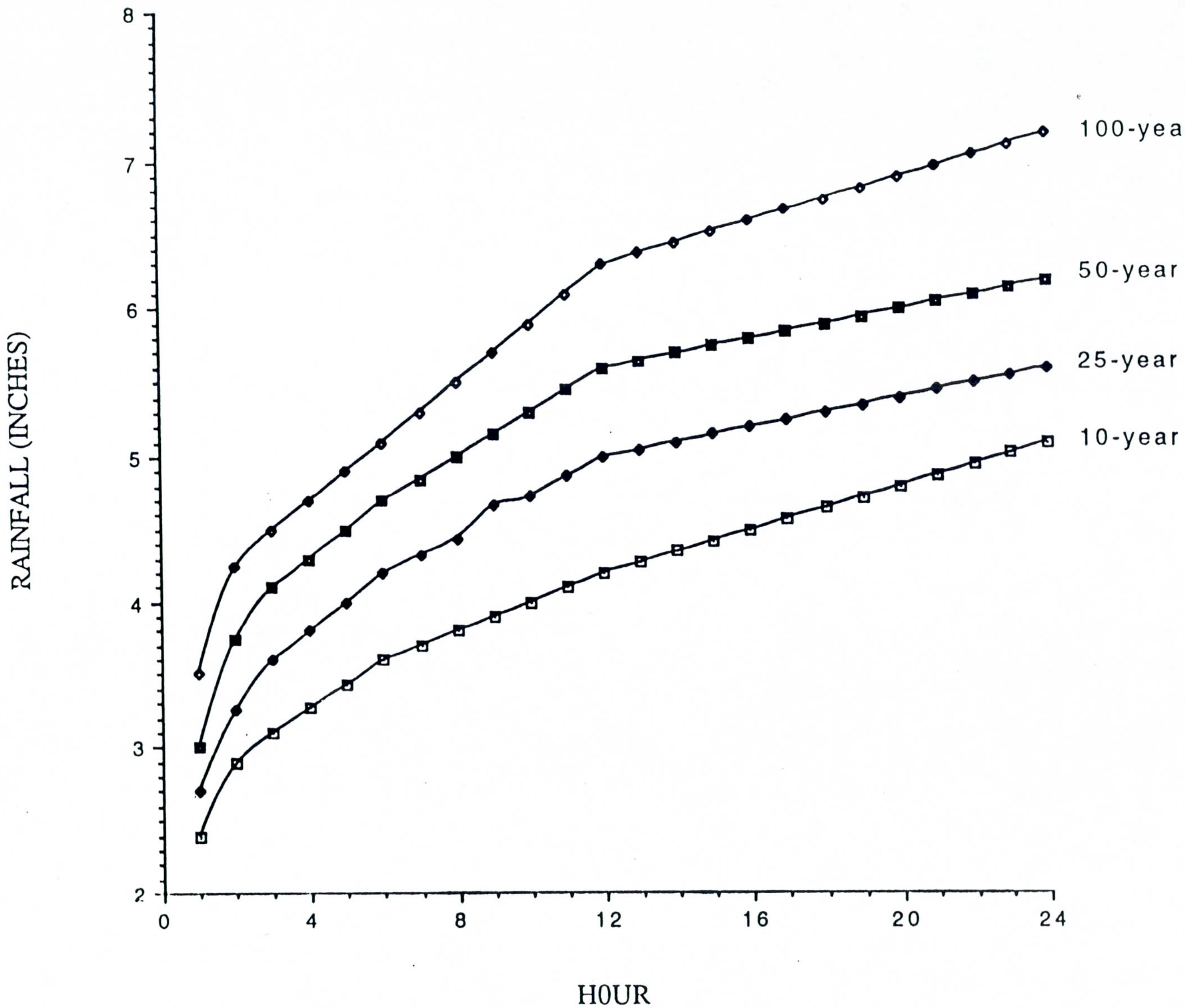
$$S = \frac{1000}{CN} - 10$$

CN = the curve number

WINTERS RUN DATA FOR SCS DIMENSIONLESS UNIT HYDROGRAPH METHOD

SUB-AREA #	AREA (mi^2)	LENGTH TO DIVIDE (ft)	AVG. SLOPE (ft/ft)	MAX. RET. S	LAG TIME, t_l (hrs)
1	3.44	14,900	0.062	3.33	1.28
2	5.63	36,500	0.069	3.89	2.72
3	6.11	22,200	0.069	3.89	1.82
4	4.43	18,400	0.098	3.89	1.32
5	5.09	15,500	0.09	3.89	1.20
6	3.06	25,800	0.082	2.5	1.49
7	3.55	20,000	0.062	3.33	1.63
8	1.68	13,000	0.037	2.5	1.00
9	2.10	12,400	0.085	2.5	0.82
TOTAL AREA	35.09	68,000	0.075	3.33	3.94

RAINFALL AMOUNTS FROM NWS TP-40



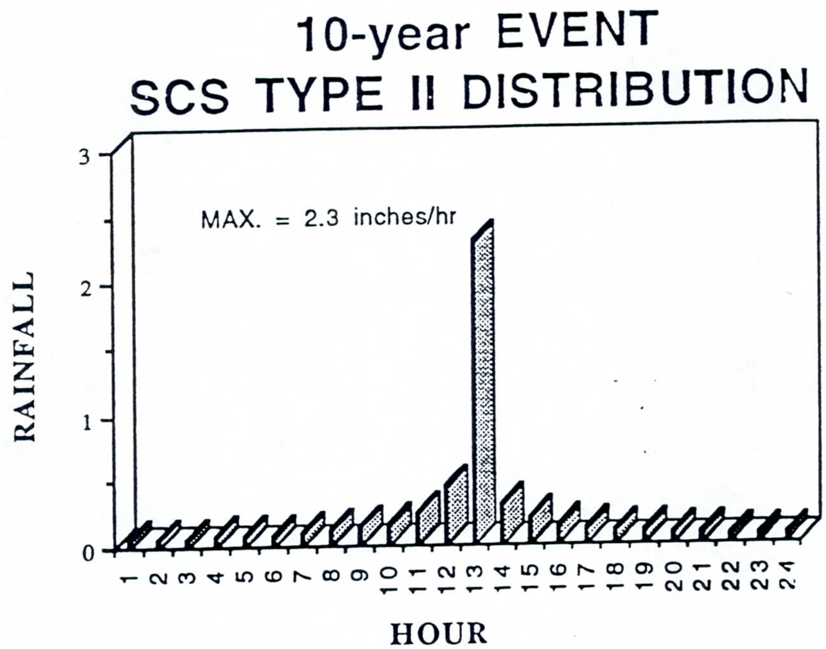


Figure 27 - SCS Type II distribution for a 10-year event.

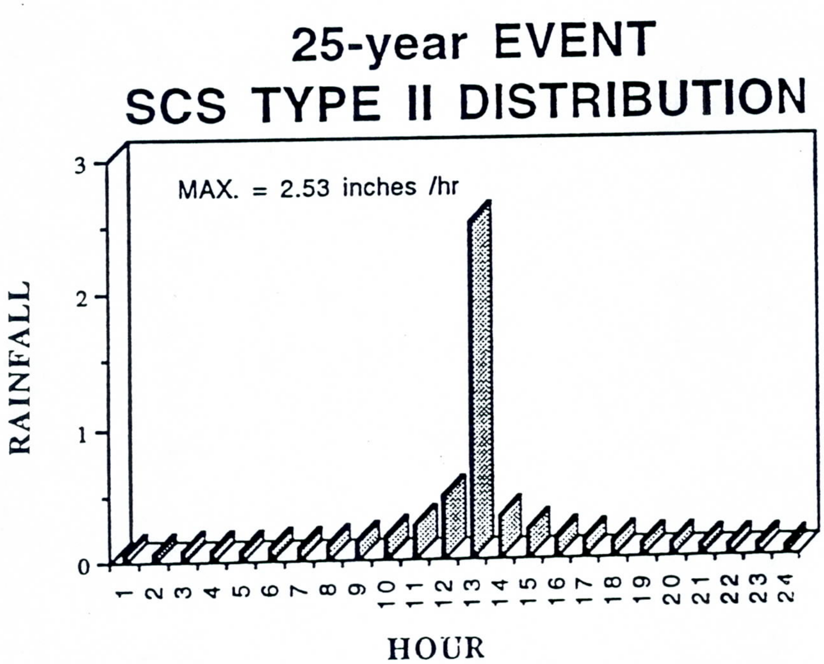


Figure 28 - SCS Type II distribution for a 25-year event.

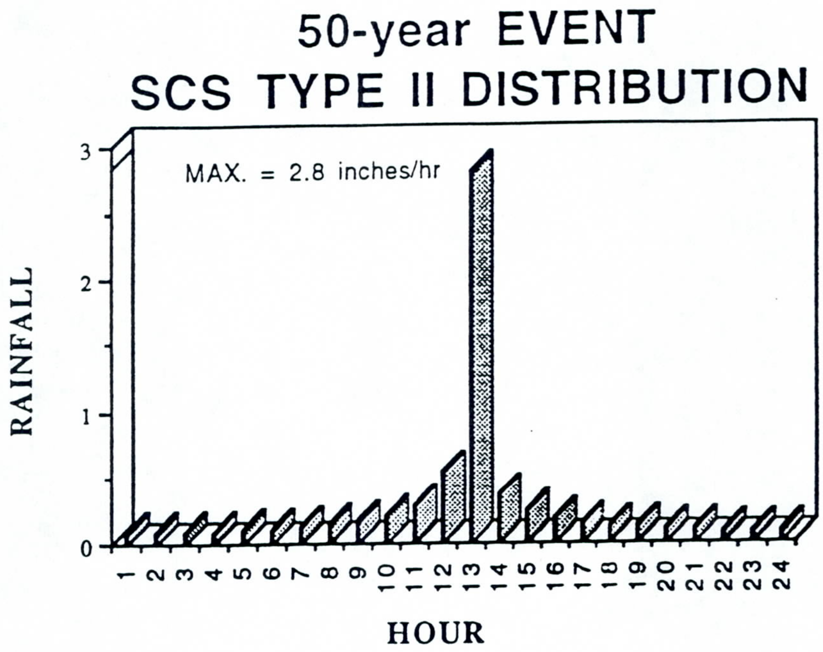


Figure 29 - SCS Type II distribution for a 50-year event.

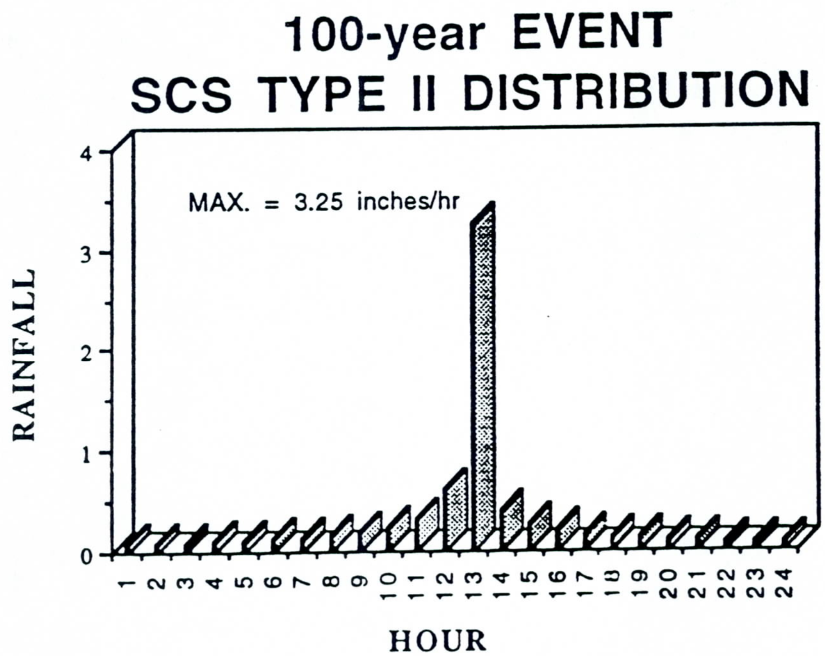


Figure 30 - SCS Type II distribution for a 100-year event.

These four synthetic storms were then combined with the average unit hydrograph (developed from the four historical events and shown in figure 21) to obtain the storm hydrographs. As in the previous section, HEC-1 was used to accomplish this task. Table 7 lists the peak flows for each of the four(4) return periods.

Table 7 - Peak flows for unit hydrograph for gaged sites.

STORM EVENT	PEAK FLOW (cfs)
10-year	2,789
25-year	4,476
50-year	5,293
100-year	6,627

2.2.2 Unit Hydrograph Development for Ungaged Watersheds

Two synthetic unit hydrograph methods were used to calculate the runoff for the 10, 25, 50, and 100-year rainfall events: Snyder and SCS. In this section the emphasis was not to provide a detailed calibration of the methods but rather to make best estimates of the input parameters and compare results.

2.2.2.1 Snyder Synthetic Unit Hydrograph Method

The Snyder Synthetic unit hydrograph development relies upon a correlation of the dependent variables of lag time, peak discharge, and physiographic watershed characteristics. The following lag-time relationship was developed by Snyder for watersheds ranging from 10 to 10,000 sq. mi.

$$t_l = C_t (L\,L_{ca})^{0.3} \tag{5}$$

where:

t_l = lag time (hrs).

C_t = coefficient representing variations of watershed slopes and storage.

L = length of the main stream channel from the outlet to the divide (miles).

L_{ca} = length of the main channel from the outlet to a point nearest the watershed centroid (miles).

Snyder's equation for peak discharge is as follows:

$$Q_p = \frac{640\, C_p\, A}{t_l} \tag{6}$$

where:

Q_p = peak discharge (cfs).

C_p = coefficient accounting for flood wave and storage conditions. It is a function of lag time, duration of runoff producing rainfall, effective area contribution to peak flow, and drainage area.

A = watershed area (sq. mi.)

t_l = lag time (hrs).

Table 8 summarizes the Snyder C_p and C_t coefficients determined by three(3) methods: regression equations developed at the Pennsylvania State University [4], HEC-1's Parameter

Optimization Routine using the four historical events used in Section 2.2.1.1., and best estimates from textbook sources.

Table 8 - C_p and C_t estimates by the various methods.

METHOD	C_p	C_t
PSU Study	0.37	1.15
Optimization	0.10	0.30
Textbook Est.	0.40	0.60

Table 9 lists the relevant data used in the Snyder method. Most of the data was obtained from USGS quadrangle maps. The curve number estimates are based on the land use, land slopes, and vegetation types.

Table 9 - Winters Run data for Snyder's method.

SUB-AREA No.	AREA (mi^2)	LENGTH of MAIN CHANNEL (miles)	LENGTH TO CENTROID (miles)	CURVE NUMBER
1	3.44	2.86	1.36	75
2	5.63	7.12	4.68	72
3	6.11	3.94	1.99	72
4	4.43	3.31	1.97	72
5	5.09	2.61	1.12	72
6	3.06	2.65	3.43	80
7	3.55	3.79	1.89	75
8	1.68	2.50	1.42	80
9	2.10	1.03	0.28	80
TOTAL AREA	35.09	13.09	6.96	75

2.2.2.1.1 Comparison of Actual vs Computed Runoff

Three Snyder synthetic unit hydrographs were developed, one for each estimate of the Snyder coefficients (Ct and Cp). Each synthetic hydrograph was then combined (via HEC-1) with the four historical rainfall events used in previous sections in order to produce runoff hydrograph estimates for each of the four storms. The runoff hydrographs were then compared with the actual runoffs as recorded by the USGS gaging station. Figures 31 through 34 illustrate the actual and computed runoff hydrographs for each of the storm events. In each of the figures there are three computed hydrographs corresponding to the various estimates of the Snyder coefficients.

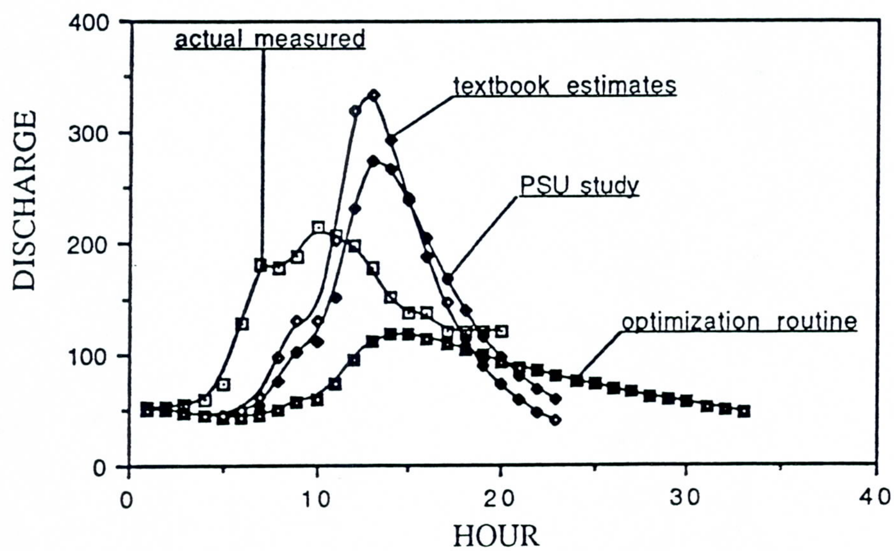


Figure 31 - Storm #1 comparison of Snyder methods.

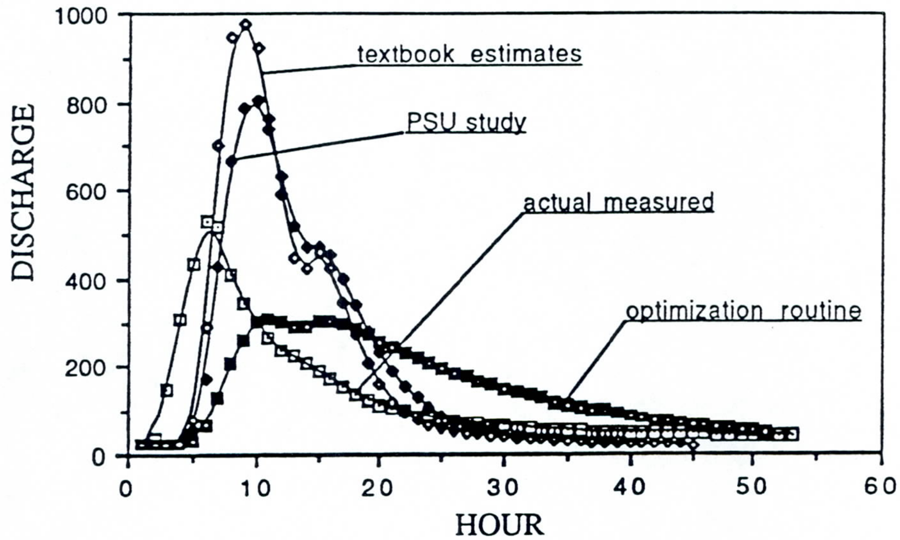


Figure 32 - Storm #2 comparison of Snyder methods.

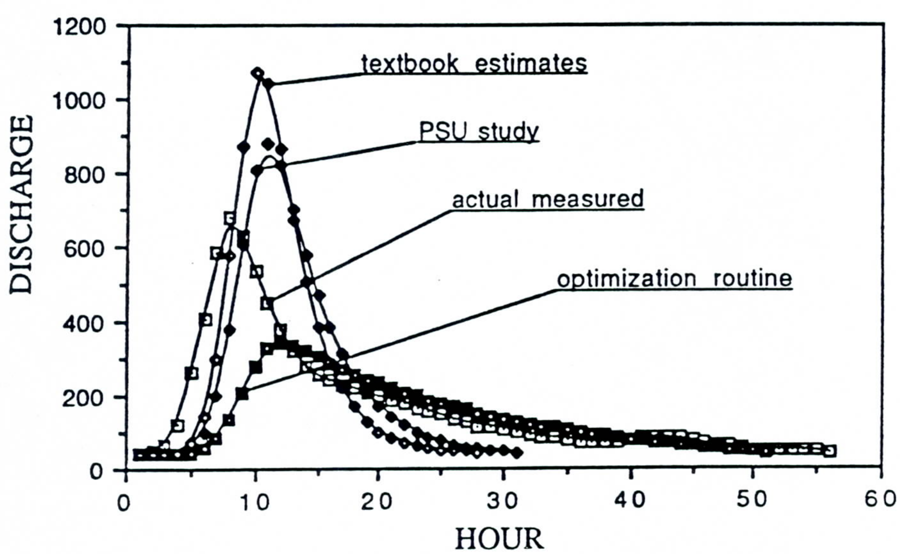


Figure 33 - Storm #4 comparison of Snyder methods.

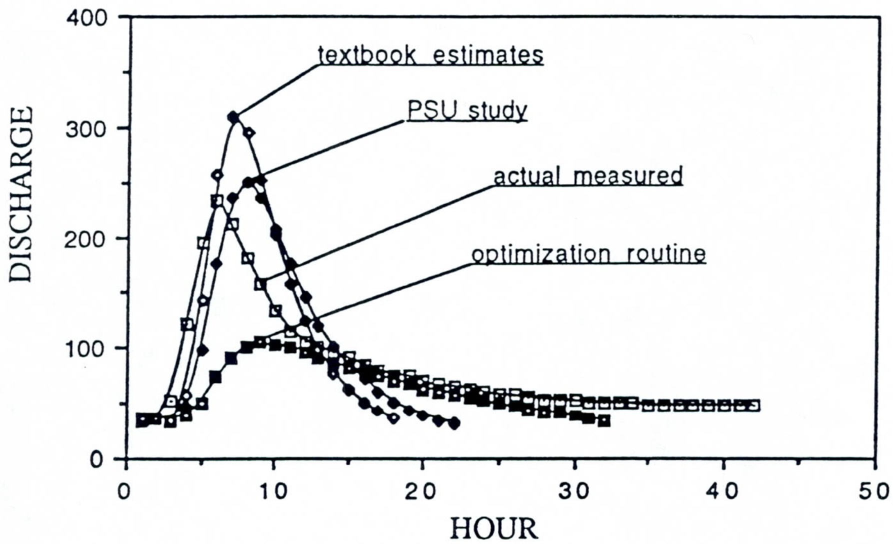


Figure 34 - Storm #5 comparison of Snyder methods.

Upon analyzing the figures, one can see that overall the hydrograph using the Snyder coefficients developed by the Penn State regression equations came the closest to matching the peaks of the actual recorded hydrographs.

2.2.2.1.2 Peak Flow Prediction

The synthetic storms developed in section 2.2.1.4 were used with the three Snyder unit hydrographs to predict the 10, 25, 50, and 100-year peak flows. Runoff predictions as computed by HEC-1 are shown in table 10.

Table 10 - Peak flow predictions for Snyder's method.

METHOD	10-year	25-year	50-year	100-year
PSU Study C_p= .37 & C_t=1.15	5,770	6,797	8,105	10,321
HEC-1 Opt. C_p = 0.1 & C_t = 0.3	2,058	2,405	2,838	3,582
Textbook Est. C_p = 0.4 & C_t = 0.6	7,445	8,715	11,414	13,232

Referring to Table 10, the peak flows predicted using the hydrograph developed from the optimized coefficients are suspiciously low. This is undoubtedly due to the fact that the coefficients were generated using more frequent storms. Consequently, they may not be suitable for predicting major flood events.

2.2.2.2 SCS Dimensionless Unit Hydrograph Method

The Soil Conservation Service's unit hydrograph method is based on a dimensionless unit hydrograph. This hydrograph is the result of an analysis of a large number of natural unit hydrographs from a wide range of sizes and geographic locations. This method employs the following two equations for time-to-peak and peak discharge, respectively:

$$t_p = \frac{D}{2} + t_l \quad (7)$$

where:

t_p = time from beginning of rainfall to peak discharge (hrs).

D = duration of rainfall (hrs).

t_l = lag time from the centroid of the rainfall to the peak discharge (hrs).

and

$$q_p = \frac{484\ A}{t_p} \qquad (8)$$

where:

q_p = peak discharge (cfs).

A = drainage area (mi^2).

t_p = the time to peak (hr).

Equation 9 is often used by the SCS to compute lag time and will be used as one of the methods for computing lag time in this study:

$$t_l = \frac{l^{0.8}\,(S+1)^{0.7}}{1900\ Y^{0.5}} \qquad (9)$$

where:

t_l = the lag time (hrs).

l = length to divide in feet

Y = average watershed slope in percent

S = the potential maximum retention (in) where:

$$S = \frac{1000}{CN} - 10$$

CN = the curve number

Table 11 lists the data used to develop one of the unit hydrographs for this study.

Table 11 - Winters Run data for SCS Dimensionless Unit hydrograph method.

SUB-AREA #	AREA (mi^2)	LENGTH TO DIVIDE (ft)	AVG. SLOPE (ft/ft)	MAX. RET. S	LAG TIME, t_l (hrs)
1	3.44	14,900	0.062	3.33	1.28
2	5.63	36,500	0.069	3.89	2.72
3	6.11	22,200	0.069	3.89	1.82
4	4.43	18,400	0.098	3.89	1.32
5	5.09	15,500	0.09	3.89	1.20
6	3.06	25,800	0.082	2.5	1.49
7	3.55	20,000	0.062	3.33	1.63
8	1.68	13,000	0.037	2.5	1.00
9	2.10	12,400	0.085	2.5	0.82
TOTAL AREA	35.09	68,000	0.075	3.33	3.94

The second unit hydrograph was developed using a lag time generated by the HEC-1 optimization routine. Optimization was achieved by assigning the watershed a constant curve number of 75 and using the four historical rainfall events used throughout this report. The optimized value of lag time for Winters Run was 9.4 hours

2.2.2.2.1 Comparison of Actual vs Computed Runoff

The two SCS unit hydrographs were combined with the historical rainfall events to predict the outflow hydrographs.

Figures 35 through 38 show the two computed hydrographs versus the actual runoff hydrograph.

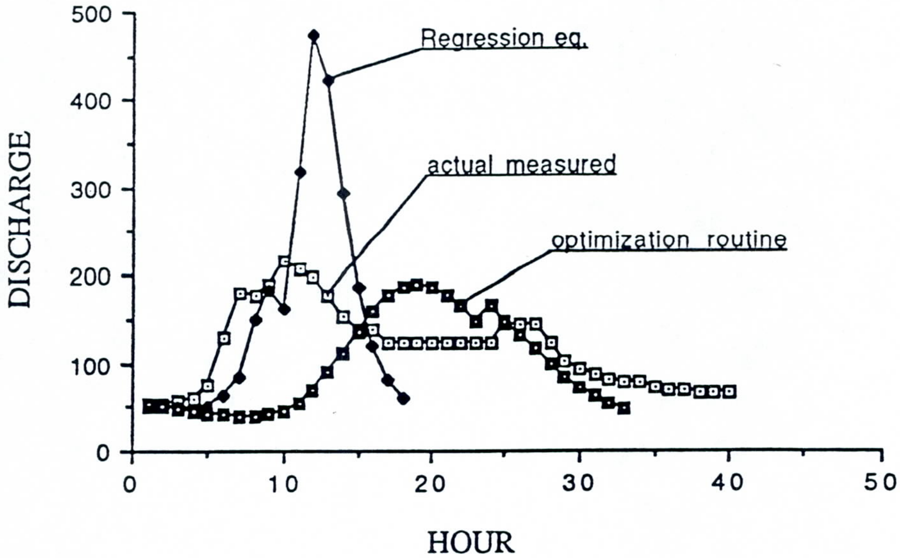


Figure 35 - Storm #1 comparison of SCS methods.

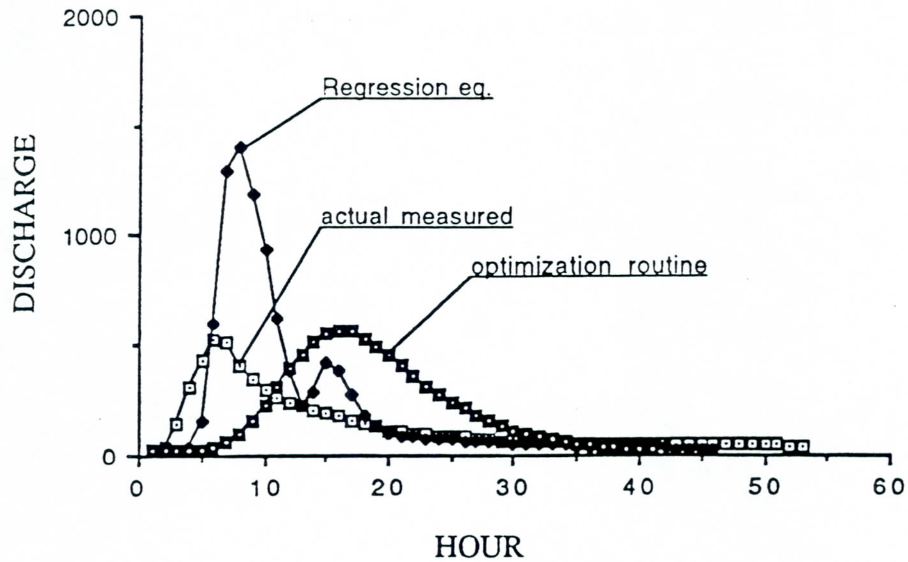


Figure 36 - Storm #2 comparison of SCS methods.

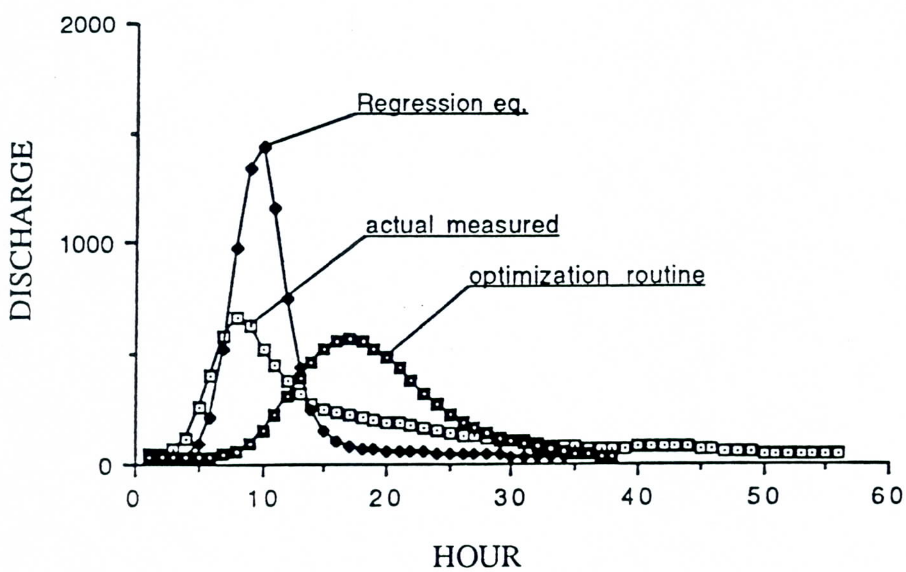


Figure 37 - Storm #4 comparison of SCS methods.

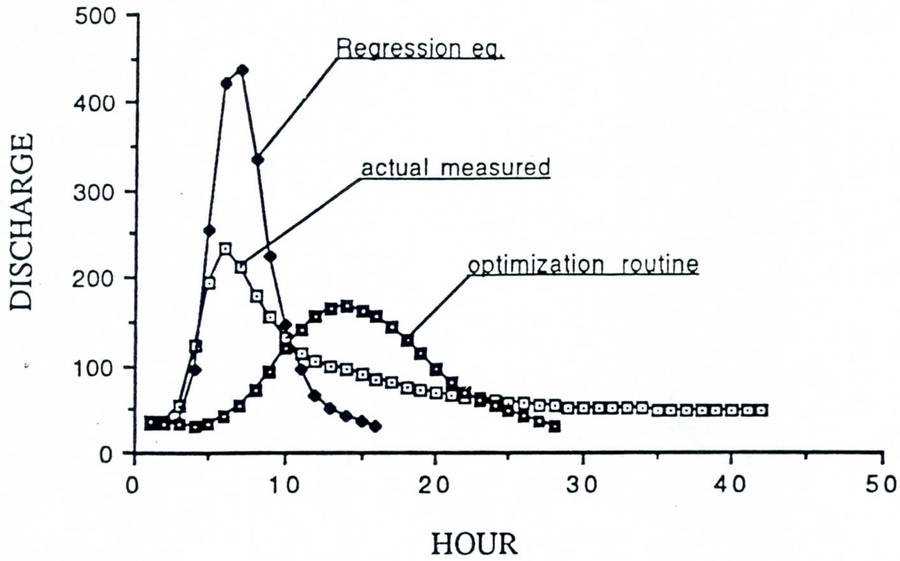


Figure 38 - Storm #5 comparison of SCS methods.

As one can see, in each case the large difference in lag times produced by the regression equation and optimization resulted in two extremely different hydrographs. However, the optimization hydrograph matches the actual hydrograph in terms of peak discharge much better than does the regression equation hydrograph.

2.2.2.2.2 Peak Flow Predictions

The TP-40 rainfalls and SCS Type II distribution were once again used to develop a major synthetic storm for the 10, 25, 50, and

100-year return periods. Peak flow predictions as computed using HEC-1 are shown in table 12.

Table 12 - Peak flow predictions for SCS method.

METHOD	10-year	25-year	50-year	100-year
LAG TIME, t_l, by SCS REGRESSSION EQUATION	12,111	14,264	16,903	21,411
LAG TIME, t_l, by HEC-1 OPTIMIZATION ROUTINE	3,749	4,385	5,129	6,427

Referring to table 12, the peak flows computed using the hydrograph developed from the optimized lag time are approximately one-third of the peak flows computed using the hydrograph developed from the regression equation lag time. Once again, this may be attributed to the fact that the optimized lag times were developed using more frequent storms and could underpredict the peaks of major storm events.

Section 3: EXECUTIVE SUMMARY

Executive Summary

A total of sixteen methods were used to predict the 10, 25, 50, and 100-year peak flows for the Winters Run watershed above the U. S. Route #1 bridge crossing. Each of the sixteen methods fell into one of two basic categories: peak flow determination or unit hydrograph generation. Table 13 lists the flood peaks generated by each method for all four return periods. Also shown are the overall average peak and standard deviation of the sixteen methods.

Looking specifically at the 100-year values presented in table 13, the peak flows range from 3,582 cfs, predicted by Snyder unit hydrograph method with optimized coefficients (method 13) to 57,709 cfs predicted by the Log Gumbel distribution (method 4). The overall average and standard deviation are 15,563 cfs and 12,615 cfs, respectively.

The flood flow frequency methods (methods #1 thru #7) were conducted using stream gaging records of only twenty-four years. Having such a limited amount of statistical data reduces the probability of having a sufficient number of storms of all magnitudes to complete an accurate statistical analysis. For this reason, these

Table 13 - Summary of Peak Flows

No.	METHOD	10-year	25-year	50-year	100-year
1	NORMAL DISTRIBUTION	6,097	7,110	7,762	8,350
2	LOG NORMAL DISTRIBUTION	7,401	10,949	14,085	17,667
3	GUMBEL DISTRIBUTION	6,753	8,633	10,027	11,414
4	LOG GUMBEL DISTRIBUTION	9,537	19,716	33,779	57,709
5	LOG-PEARSON SKEW = -.7375	6,792	8,691	9,998	11,203
6	LOG PEARSON SKEW = -.075	7,359	10,710	13,615	16,869
7	LOG PEARSON SKEW = .645	7,718	12,933	18,479	25,883
8	USGS REGR. EQUATIONS UNGAGED	4,324	6,447	8,488	12,556
9	USGS REGR. EQS. GAGED (TABLES 3&12)	4,304	5,979	7,580	10,371
10	USGS REGRESSION EQS. GAGED (LOG PEARSON)	6,651	9,418	11,956	15,390
11	UNIT GRAPH FOR GAGED WATERSHEDS	2,789	4,476	5,293	6,627
12	SNYDER UNIT GRAPH W/PSU STUDY	5,770	6,797	8,105	10,321
13	SNYDER UNIT GRAPH W/OPTIMIZED	2,058	2,405	2,838	3,582
14	SNYDER UNIT GRAPH W/TEXTBOOK	7,445	8,715	11,414	13,232
15	SCS W/REGR. EQ. LAG TIMES	12,111	14,264	16,903	21,411
16	SCS W/OPT. LAG TIME	3,749	4,385	5,129	6,427
N/A	AVERAGE	6,303	8,852	11,591	15,563
N/A	STD. DEV.	2,523	4,275	7,275	12,615

* - All flows are in cubic feet per second, cfs.

methods may under or overpredict the flows of higher magnitudes, depending on the characteristics of the individual distribution methods. For instance, the characteristic skew of the Gumbel distribution places the 100-year event at 3.75 standard deviations away from the mean of this particular data set. This fact combined with taking the logarithms of the flows accounts for the extreme prediction by this method; therefore, little confidence is placed in this predicted value.

Methods #13 and #16 used an optimization routine based on historical storm events to determine coefficients for the Snyder and SCS unit hydrographs. Method #11 also relies upon historical storm events for its predictions. The particular storm events used in this study were of rather low rainfall amounts and intensities. Therefore, any methods which rely on the characteristics of these storm events to be representative of all storm events for the watershed, run the risk of underpredicting the peak flows for storms of higher magnitudes, such as the 100-year event. Methods #11, #13, and #16 are considered poor estimates, particularly if a conservative estimate is sought.

Consequently, knowing that the historical data set used in the study resulteed in poor estimates from methods #4, #11, #13, and #16, the four values were discarded and a new average and standard deviation were calculated from the remaining twelve methods. The new average (100-year) decreased by 1000 cfs to 14,555 cfs, while the standard deviation was reduced by more than a factor of two to a value of 5,150. The results from nine methods from nine methods

fall within one standard deviation (9,405 cfs - 19,705 cfs) of the new average: #2, #3, #5, #6, #8, #9, #10, #12, and #14).

Of the nine results presented in table 13, which fall within one standard deviation of the new average, three were predicted by methods used for ungaged watersheds (#8, #12, and #14). The values predicted by these three methods are relativley close to one another (12,566, 10,321, and 13,232 cfs respectively). However, the level of effort used in applying each method was not the same. The Snyder unit hydrograph method with various lag times (#12 and #14) requires a great deal of data collection and parameter calibration and a massive amount of calculations if a computer is not available. On the other hand, the USGS regression equaitons (#8) require minimal data and straight forward calculations; and the value predicted by this method is every bit as valid as those predicted by the more time consuming Snyder uint hydrograph methods.

The remaining results which fell within one standard deviation were predicted by gaged methods. Of these, the USGS regression equations (#10) predicted the value (15,390 cfs) closest to the overall average. However, a few of the flood flow frequency methods (#2, #3, #5, and #6) have estimates not too far from the average.

DEVELOPING WATERSHED PARAMETERS FOR UNGAGED WATERSHEDS

HYDROLOGY

Wednesday 11:20 a.m.

EXAMPLE: AUSTEN

Wednesday 1:00 p.m.

Probable Maximum Flood Studies

Example 4: Austen Hydroelectric Project

June 1994

Purpose: *To illustrate a deviation from the recommended procedures in the Guidelines, justified by unusual basin hydrology and data limitations.*

Summary

Subbasin Division:	None
Routing:	No channel routing; reservoir routing using Modified Puls in the HEC-1 model
Unit Hydrograph Analysis:	Adopted parameters from previous studies after verification on recent flood event
Loss Rates:	Detailed modeling using Agricultural Research Service's KINEMAT model on subbasin; transferred as equivalent HEC-1 parameters to whole basin
Initial Reservoir Level:	Annual maximum normal operating level at run-of-river project
Snowpack:	Not applicable
Snowmelt:	Not applicable
Sensitivity Analysis:	None for unit hydrograph parameters; sensitivity of the peak PMF inflow to KINEMAT parameters K_s and G

Example 4
Austen Project
Probable Maximum Flood

June 1994

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Example 4
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June 1994

Summary

This study was performed to estimate the Probable Maximum Flood (PMF) at the Austen Hydroelectric Project, Federal Energy Regulatory Commission (FERC) Project No. BBBB. The study departs from the recommended procedures in Chapter VIII of the FERC *Engineering Guidelines for the Evaluation of Hydropower Projects* (Reference 1) because of unique considerations in the basin hydrology and data availability. Specifically, a detailed subbasin modeling approach was adopted to evaluate loss rates in the highly permeable basin soils. The PMF inflow calculated in this study is 12,956 cubic feet per second (cfs), and PMF outflow is 12,740 cfs. Since the PMF outflow is less than the spillway capacity of 19,600 cfs, no further hazard study or remedial action is required.

I. Project Description

A. Project Data

The Austen Hydroelectric Project is located on the Arrowhead River in Sand County, Anystate, at a drainage area of 1,273 square miles. The project impounds an 860-acre reservoir at normal headwater elevation 963.4 feet National Geodetic Vertical Datum (NGVD). Normal storage volume is 9,300 acre-feet. From left to right facing downstream, the project structures include a 750-foot earth dike; a gated spillway with three 22-foot-long by 18-foot-high tainter gates; a log sluice; a powerhouse with integral intake; and another 350-foot-long earth dike (Reference 2). Normal tailwater is 938 feet, and the maximum height of the earth dikes is approximately 28 feet. The spillway capacity is 19,600 cfs when the pool is at the top of the earth dikes.

The project is operated as a run-of-river project, with outflows set to maintain normal headwater level. An operator is on the site 8 hours a day and on call from the licensee's operations center, 18 miles away, 24 hours a day. Headwater and tailwater levels are monitored electronically from the operations center. The tainter gates are controlled by a moveable electric hoist.

B. Basin Hydrologic Data

The 1,273-square-mile basin is undeveloped, much of it managed as national or state forest. Relief is moderate, with poorly developed drainage and closed depressions in many areas. The majority of the basin soils are deep, clean sands, except where wetlands have formed saturated, organic-rich soils. Approximately 15 percent of the basin is wetland, based on land cover maps (Reference 2).

Historic flooding on the Arrowhead River has been minor. The flood of record (1942–present) at the project site is 4,380 cfs. A U.S. Geological Survey (USGS) stream gage (No. 38884000) is located at the State Highway 55 Bridge near Loup City, approximately one mile upstream from the project dam. The drainage area at the Loup City gage is 1,267 square miles—more than 99 percent of the project drainage area.

Other stream gages are located in various subbasins upstream of the project. All the basin gages and their floods of record are summarized in Table 1 (Reference 3).

TABLE 1
Stream Gages in Arrowhead River Basin

Gage Name and Number	Drainage Area (sq. mi.)	Period of Record	Flood of Record (cfs)	Date
Arrowhead River at Loup City (38884000)	1,267	1942–present	4,380	5/25/48
South Branch Arrowhead River near Geneva (38876600)	401	1966–1989 1990–present	1,120	3/28/76
Arrowhead River at Conestoga (38878700)	110	1942–present	274	6/2/43
East Branch Arrowhead River near Conestoga (38878500)	73	1959–1984	207	3/28/76

Hourly recording rain gages are located in the Arrowhead River basin at New London and Conestoga, and there are nonrecording gages (daily totals only) at Loup City and Doyle Lake. Table 2 shows the rain gages and their periods of record.

TABLE 2
Rain Gages in Arrowhead River Basin

Gage Location	Data Type	Period of Record
Loup City	Daily	1939–present
Conestoga	Hourly	1948–present
Doyle Lake	Daily	1963–1968, 1978–present
New London	Hourly	1971–present

A basin map showing locations of the project and the rain and stream gages is presented in Exhibit 1.

Relative to other similar-sized basins in the region, historic floods on the Arrowhead have been extraordinarily small. Table 3 presents a comparison of 100-year floods estimated from gage records on the Arrowhead and four other forested, regulated basins in the area.

TABLE 3
100-Year Floods at
Selected Regional Gaging Stations

Gage No.	Gage Name	Drainage Area (sq. mi.)	100-Year Flood (cfs)
38884000	Arrowhead River at Loup City	1,267	4,160
41457500	Goulet River near Bannister	1,120	9,800
41690000	Bear River near Boston	2,350	17,000
38674000	Lac des Morts River near Bibbs Island	1,000	11,000
36003700	White River at Peru	1,200	6,300

Floods on the Arrowhead have historically been unusually small, even when compared to apparently similar basins in the same region. It has been established in previous studies that the unique behavior of the basin can be attributed to the depth and permeability of the sandy basin soils, which are 30 to 200 feet deep throughout the

Arrowhead basin (Reference 3). As explained in *Sections IV and VII* of this report, these conditions generate special difficulties in applying conventional watershed modeling techniques to the basin.

C. Upstream Dams

There are no major dams upstream of the Austen Hydroelectric Project. Small check dams are located on several tributaries, but none have significant storage. Therefore, none of these dams were considered separately in the PMF study.

D. Field Visit

The project hydrologists visited the basin in March 1992 prior to beginning the studies described below. Both a driving and an aerial reconnaissance were conducted. The visit verified information previously gained anecdotally. Deep, sandy soils were evident throughout the basin, with up to 40-foot-high exposures of sand in road cuts and reservoir banks. The drainage pattern was quite poorly developed, with internally draining wetlands and large flat areas with no discernible drainage pattern. At the time of the visit, the main stem of the Arrowhead River near the Loup City gage was flowing within about 2 feet of the top of the bank. Just beyond the banks were numerous trees of all ages, with no visible evidence of overbank flooding.

The project works and their operation were reviewed during the independent consultant's physical inspection in June 1992. Each of the three tainter gates was tested at that time and found to be fully operable.

E. Previous Studies

PMF studies were conducted for the Austen Hydroelectric Project in 1983 and 1989 (References 2 and 3). In the 1983 study, Snyder unit hydrograph parameters were estimated from two floods at the Loup City gage. The 1983 study also included a determination of the Probable Maximum Storm (PMS) from Hydrometeorological Report No. 51 (Reference 4), and the use of the Soil Conservation Service Runoff Curve Number method of estimating runoff losses. The resulting PMF peak was 54,000 cfs (Reference 2).

The 1989 study adopted the unit hydrograph ordinates from the 1983 study and added the techniques of Hydrometeorological Report No. 52 (Reference 5) to the PMS determination. Instead of the SCS Curve Number method, an initial and constant loss rate were applied. First, a loss rate equal to the area-averaged soil infiltration rate for the entire basin was applied. The resulting basin-averaged loss rate was approximately 11 inches per hour and the predicted PMF peak was 5,400 cfs. It was recognized that the averaging process had generated a basin average loss rate greater than the peak PMS intensity and had thus overlooked the potential for some less permeable basin soils to generate runoff. The model had actually predicted runoff only from the 3.8 percent of the basin designated as impervious. A calibration of the basin loss rate on the flood of record yielded a calibrated basin average loss rate of 0.8 inches per hour. Applying this loss rate in the watershed model resulted in a PMF peak of 32,400 cfs (Reference 3). This method is also logically flawed, as described in *Sections IV and VII*.

This study adopts a new method for estimating basin losses, since previous studies yielded such divergent results. The method is a deviation from the recommended procedures in the FERC guidelines (Reference 1, *Section 8-10.3*), because, as explained below, neither the approximate nor the detailed method as presented in the guidelines works well with available data.

II. Watershed Model and Subdivision

A. Watershed Model Methodology

The entire Arrowhead River watershed above the project was modeled with the U.S. Army Corps of Engineers (COE) HEC-1 model. However, loss rates input to the HEC-1 model were determined by a detailed model study of a 73-square-mile subbasin, the East Branch of the Arrowhead River near Conestoga. This study used the Agricultural Research Service's KINEMAT model, which represents the watershed as a linked set of overland flow planes and channels.

The use of the KINEMAT model to attain a better representation of distributed loss rates deviates from the recommended procedures in the guidelines (Reference 1, *Section 8-10.3*). The KINEMAT model was selected for its ability to model many small elements of the subbasin separately, routing flow from the plane in which it is generated over other overland flow planes, which may have the capability of infiltrating the runoff generated upslope. These model characteristics were identified as critical for the following reasons:

- It appears that a relatively small group of hydrologically sensitive soils generates virtually all direct runoff; in other words, the contributing area of the basin is much less than the topographic drainage area. Techniques based on averaging loss rates will not adequately represent these soils, because the very high permeability of the majority of the basin soils will tend to cancel out the runoff-producing soils in the average value.

- Some low-permeability soils are probably hydrologically unconnected to the channel network, due to the poor drainage development and the preponderance of high-permeability soils throughout the basin. Therefore, secondary infiltration—infiltration of runoff somewhere between its generation point and the channel—is potentially an important consideration.

For these reasons, it is believed that a relatively small proportion of the area that is topographically within the basin boundaries actually contributes direct runoff to the river. The use of the KINEMAT model on a small subbasin depends on the assumption that the KINEMAT subbasin represents this proportion for the basin as a whole.

B. Subbasin Definition

The HEC-1 model used to generate the PMF hydrograph at the project has no subdivision. The Loup City gage used for unit hydrograph calibration is essentially at the project; there are no major impoundments affecting flow from upstream; and the basin's land cover, soil distribution, and topography are fairly homogeneous.

C. Channel Routing Method

Since there was no subbasin division, channel routing was not performed in this study in the HEC-1 model. Flows were routed through the reservoir using the modified Puls reservoir routing in the HEC-1 model. The KINEMAT model, used to derive a loss function that would be extended to the basin as a whole, uses kinematic wave routing for both overland and channel flow. However, this applies only to the part of the study performed to estimate loss rates. No channel routing was performed for the final PMF determination.

III. Historic Floods

A. Stream Gages

The Loup City stream gage was found to have the best record of flooding for calibration purposes. The upstream gages listed in Table 1 are either located at very small drainage areas or were found to have insufficient records of flood events suitable for calibration. For the purposes of this study, a flood is "suitable for calibration" if it has a distinct peak clearly related to a precipitation event and is one of the largest floods of record.

B. Historic Floods

The five largest historic floods at the Loup City gage are shown in Table 4.

TABLE 4
Floods of Record at Loup City Stream Gage

Date	Flood Peak (cfs)
May 25, 1948	4,380
March 16, 1976	3,800
September 23, 1966	3,650
September 1, 1966	3,200
June 14, 1986	3,100

The flood of record (4,380 cfs on May 25, 1948) had an estimated total volume of 18,500 acre-feet, excluding assumed baseflows. This is equivalent to 0.3 inch of runoff over the entire basin.

The unit hydrograph study included in the 1983 PMF determination (Reference 2) used the floods of May 1948 and September 1966 to derive unit hydrograph parameters.

For this study, the 1983 study's unit hydrograph parameters were verified against the flood of June 1986. Although none of these events yielded anywhere near one inch of runoff—which would be preferable for unit hydrograph analyses—they are the best available data for unit hydrograph calibrations.

C. Precipitation Associated with Historic Floods

Daily precipitation totals associated with each of the above floods were estimated by weighting the daily totals at Loup City and Doyle Lake. Hourly distributions used in the 1983 study's unit hydrograph calibrations and the present study's verification were estimated by distributing the daily basin totals proportionately to the hourly precipitation sequence at New London. The New London hourly rain gage was considered more representative of the basin average than the Conestoga rain gage, as New London is more centrally located than Conestoga (see Exhibit 1). Rainfall totals from the daily gages for each storm are shown In Table 5. The estimated 6-hour incremental sequence for the June 1986 storm is shown in the HEC-1 verification run in Exhibit 2.

TABLE 5
Precipitation Associated with Historic Floods
at Loup City Stream Gage

Date of Flood	Storm Duration (Days)	Total Precipitation (inches)
May 25, 1948	5	4.4
March 16, 1976	2	3.9
September 23, 1966	1	2.9
September 1, 1966	4	4.7
June 14, 1986	2	3.3

The storms shown in Table 5 do not include the record 72-hour precipitation, which totalled 5.9 inches and occurred in June 1951. This storm produced a flood of only 2,500 cfs. This event followed a relatively dry spring and a period of low baseflow. In contrast, the storm of September 23, 1966, produced a large flood from a relatively small amount of rain. When the September 23 storm began, baseflows were still high from the flood occurring earlier that month. These examples illustrate the importance of antecedent conditions in producing major flood events on this watershed.

The storm used to calibrate the model loss rates on the East Branch subbasin, as described in *Section VII* of this report, occurred on July 10 and 11, 1984. This storm produced a significant flood on the East Branch subbasin but did not produce basinwide flooding. The total 11-hour rainfall over the East Branch subbasin for this event was 3.02 inches, estimated from the Conestoga rain gage. Although the New London rain gage is preferable to the Conestoga gage for whole-basin precipitation estimates, the Conestoga gage is approximately one mile outside the East Branch subbasin and was therefore used for the detailed estimate of loss functions in this subbasin.

D. Snowpack and Snowmelt During Historic Floods

Snowmelt was not associated with any of the historical floods listed above. Snowfall occurs infrequently in this region and significant snowpacks do not develop.

IV. Unit Hydrograph Development

A. Discussion of Approach and Tasks

The Arrowhead River basin at the Austen Hydroelectric Project is considered gaged for the purposes of unit hydrograph analysis (Reference 1, *Section 8-7.1.1*). The analyses performed for the 1983 PMF study (Reference 2) yielded a Snyder's Tp of 48 hours and Snyder's Cp of 0.45. These values were converted to Clark parameters and verified against the flood of June 1986 as described below.

B. Baseflow Separation

In the 1983 PMF study (Reference 2) baseflow was defined graphically by assuming a constant baseflow, equal to the baseflow at the start of the storm, through the rising limb of the hydrograph, then a linear increase to the inflection point on the falling limb. The resulting baseflow hydrograph was then subtracted from the observed flow hydrograph before calibrating Snyder parameters. For the 1986 verification storm, the baseflow parameters required in the HEC-1 model were estimated using a semilogarithmic plot of the hydrograph as described in the PMF guidelines

(Reference 1, *Section 8-8.4*). This plot is included as Exhibit 3. The resulting baseflow parameters are shown in Table 6.

TABLE 6
Estimated Baseflow Parameters for Flood of June 1986

HEC-1 Parameter Name	Meaning	Value
STARTQ	Baseflow at Beginning of Storm (cfs)	1,013
QRSCN	Flow at which baseflow recession begins (cfs)	1,700
RTIOR	Baseflow Recession Constant	1.007

C. Preliminary Estimates of Clark Parameters

There were no preliminary estimates of unit hydrograph parameters, as unit hydrograph analysis in this study consisted only of verification of previously estimated parameters.

D. Estimate of Infiltration During Historic Floods

In the 1983 study (Reference 2) the HEC-1 model was run iteratively to calibrate loss rates and unit hydrograph parameters. The calibrated initial and constant loss rates are shown in Table 7.

TABLE 7
Initial and Constant Loss Rates as Calibrated in 1983 PMF Study

Date of Flood	Calibrated Initial Loss Rate (in.)	Calibrated Constant Loss Rate (in./hr.)
May 1948	1.6	0.8
September 1966	2.0	0.2

However, these estimates were not used in the final PMF determinations in the 1983 study. The 1989 PMF study (Reference 3) did use the calibrated loss rate of 0.8 inches per hour from the 1948 flood.

The calibrated basin-averaged loss rates from two historic floods differ significantly, and neither was used for this study. One reason for this variability—and the reason for choosing the KINEMAT model, which does not require averaging—is that any basin-averaged calibrated loss rate is a function of rainfall intensity. For example, as long as any runoff occurs at all from the basin, the calibrated loss rate must always be less than the rainfall intensity in order to generate the observed runoff. However, in a more intense storm, the basin-averaged loss rate could be considerably higher and still yield the observed runoff hydrograph. Therefore, it is likely that loss rates estimated from historic floods, such as the 1948 flood, are lower than those that would be calculated, if possible, for the PMS.

A simplified computation demonstrating this problem is included as Exhibit 4.

E. Subbasin Unit Hydrograph Parameters

The Snyder unit hydrograph parameters for the Arrowhead basin at Loup City, as estimated in the 1983 PMF study, are:

$$Tp = 48 \text{ hours}$$
$$Cp = 0.45$$

Equivalent Clark unit hydrograph parameters computed by the HEC-1 program are:

$$Tc = 48.7 \text{ hours}$$
$$R = 75.9 \text{ hours}$$

These parameters were adopted following unit hydrograph verification as described below.

V. Unit Hydrograph Verification

The Clark parameters listed above were entered into the basin HEC-1 model used in the 1983 study to verify the fit of the computed hydrograph at Loup City against the historical hydrograph of June 1986. The HEC-1 model run used for verification is included in Exhibit 2. The model was also allowed to optimize initial and constant loss rates. The optimized initial loss was 0.5 inch. The optimized constant loss rate was

1.0 inch per hour. As explained in *Section IV* and Exhibit 4 of this report, these loss rates were not used in the final estimate of the PMF, because they are storm-specific.

The verification HEC-1 model run (Exhibit 2) shows that the Clark parameters derived from the 1983 study produce a slightly steeper, higher-peaked hydrograph than that actually produced by the June 1986 storm. The Clark parameters resulted in a 6.5 percent overestimation of the lag time (center of mass of rainfall to center of mass of flow). The peak flow was overestimated by approximately 3.3 percent. These errors are considered insignificant in light of the natural variation in storm distribution and timing, which is not captured by available data. Table 8 summarizes the unit hydrograph verification data from the HEC-1 analysis.

TABLE 8
Summary of Unit Hydrograph Verification Data

Data Type	Historical Hydrograph	Simulated Hydrograph	Percent Difference
Total precipitation depth (in.)	3.02	3.02	–
Initial T_c (hrs.)	48.7	48.7	–
Initial R (hrs.)	75.9	75.9	–
Runoff depth (in.)	0.47	0.50	6.4
Peak flow (cfs)	3,074	3,177	3.3
Lag time (hrs.)	93.5	99.6	6.5

The Clark parameters derived from the Loup City stream gage were used without further adjustment to represent the hydrograph entering the Austen Pond.

VI. Probable Maximum Precipitation

A. Probable Maximum Precipitation Data

The depth-area-duration relationship for the Probable Maximum Precipitation (PMP) in the Arrowhead River basin was determined from *Hydrometeorological Report No. 51*

(Reference 4). These values were plotted and smoothed (Exhibit 5) and are summarized in Table 9.

TABLE 9
Probable Maximum Precipitation
Depth-Area-Duration Data (in.)

Area (Sq. mi.)	Duration (hrs.)				
	6	12	24	48	72
10	15.6	18.6	20.9	22.8	24.5
200	13.7	16.6	18.8	20.7	22.4
1,000	11.0	13.5	15.5	17.4	19.1
5,000	7.5	9.4	11.1	13.0	14.7
10,000	5.5	7.4	9.1	11.0	12.7
20,000	3.5	5.4	7.1	9.0	10.5

B. Candidate Storms for the PMF

The spatial and temporal distributions of various candidate PMSs were determined with the COE HMR52 computer program. The HMR52 program determines the storm pattern—consistent with a given set of depth-area-duration values—that maximizes precipitation depth over a specified subbasin or group of subbasins. Since the basin was analyzed without subdivision, only one centering and basin optimization was tested. However, the HMR52 computer program tests various orientations and storm sizes to maximize the basin average precipitation.

The HMR52 program selected an optimized storm area of 1,000 square miles and an orientation 140 degrees from north. HMR52 input and output are shown in Exhibit 6, and PMS isohyets are shown in Exhibit 7.

VII. Loss Rates

A. Discussion of Loss Rate Methodology

As discussed above, previous studies have shown that loss rates are highly variable within the basin and, if calibrated to a historic storm, apply only to that storm or one with similar intensity. Furthermore, a basin-averaged loss rate based on physical parameters of the soils, as applied in the 1989 PMF study (Reference 3) tends to cancel the effect of the few, but critical, runoff-producing soils. Finally, the extent of these soils is a function of rainfall intensity, since soils that allow all rainfall to infiltrate during a moderate storm may produce runoff in a more intense storm.

These conditions cause difficulties when a drainage basin with highly variable soils (some of which commonly produce runoff and others of which virtually never produce runoff) is assigned a single, "average" loss rate. It is almost always inappropriate to select such an average loss rate by averaging physical properties of the soils, because the resulting average may be greater than the rainfall intensity. In this case, the model will predict no runoff from the storm. It is also inappropriate to assume that an average loss rate calculated for one storm applies to others of different intensities. This is because when a low rate is calibrated to an actual event, even if there is no runoff at all, the calibrated loss rate will not exceed the maximum rainfall intensity.

These difficulties would not arise if the entire basin could be modeled, with soils of different permeabilities treated as separate runoff-producing units. One way to do this is with the "detailed method" of estimating loss rates as outlined in the FERC engineering guidelines (Reference 1, *Section 8-10.3.2*). However, a review of the STATSGO data for the basin showed that a very large range of possible soil permeabilities is given for each soil unit, and there is no documented reason for selecting any particular loss rate within this range. (In these cases, the SCS recommends using the average infiltration rate listed, as there is no documented basis for further refinement of the infiltration rate.) Furthermore, the detailed STATSGO analysis does not admit the possibility of secondary infiltration of runoff, as it travels downslope from its generation point and infiltrates into permeable soils before reaching the stream.

B. Warm-Season

To overcome these difficulties, a warm-season basin average loss rate specific to the PMS was estimated by analogy to a 73-square-mile subbasin. This subbasin, the East Branch of the Arrowhead River at Conestoga, was modeled in detail with the KINEMAT model (Reference 6). The modeling effort included a calibration of individual soil unit loss rates for a historical storm. A detailed description in the KINEMAT model study is given in the following paragraphs.

1. KINEMAT Model Description

Note: When a program that is not in common use in PMF studies is adopted for a particular analysis, the user must be prepared to provide FERC staff with the executable program and documentation.

The Agricultural Research Service describes the KINEMAT model as:

> ...an event oriented, physically based model developed to describe the hydrologic processes of interception, infiltration and surface runoff from small agricultural and urban watersheds (Reference 6).

KINEMAT is a component of the more comprehensive KINEROS model, which also includes the capability to model erosion. Like almost all available models and all infiltration accounting methods within these models, KINEMAT has been tested primarily on smaller agricultural watersheds. However, discussions with authors of the model at the ARS revealed that it has been used on drainage areas of 50 square miles and more. Furthermore, since the basic area unit of calculation is small and is described by physical parameters of the land and soils, there is no theoretical limit to the basin size that may be modeled. (There are, however, computational limits to the number of area elements that may be used, effectively imposing limits on the drainage area size.)

Conceptually, KINEMAT is similar to HEC-1 in that the basin is divided into subbasins. The difference is that in KINEMAT, the user further divides the subbasins into nearly homogeneous "elements" and models the basin as a cascade of planes and channels. Each plane is characterized by its area, slope, soil and land cover parameters, and connections to other planes or channels.

Channels are assumed to be trapezoidal in cross section and are used to collect and route the storm flow hydrograph.

Because of the detail required in KINEMAT, it would be impractical to apply it to the entire Arrowhead River basin. For this reason, a detailed study was conducted on a small, representative watershed within the basin. The results from this study were then extended to the HEC-1 model for the entire basin.

2. Study Basin

The watershed used for this study is a 73-square-mile gaged portion of the East Branch of the Arrowhead River (USGS Gage No. 38878500, East Branch Arrowhead River near Conestoga). This subbasin was chosen primarily for the availability of a gage record with at least one significant runoff event, with a runoff peak distinguishable from the subsurface flow hydrograph. Although gage records for other subbasins were reviewed, only this one was found to meet this criterion. In addition, this subbasin (called the East Branch subbasin in this report) was considered an appropriate surrogate for the runoff characteristics of the entire basin because it represents a typical watershed with respect to soil type and land cover. The 5 soil units in the East Branch basin account for 76 percent, by area, of the soils found in the Arrowhead basin as a whole. If anything, extrapolating the East Branch's soil distributions to the basin as a whole is slightly conservative, as 82.7 percent of the East Branch subbasin is classified as a sand or sandy loam while 86.3 percent of the entire Arrowhead basin is a sand or sandy loam. The majority of the land cover in both basins is heavily forested.

A map showing the location of the East Branch subbasin is presented in Exhibit 8.

3. Model Development

To construct a KINEMAT parameter file, input information is required on topography, soil cover, and land cover. The topography gives the slope for each element and defines the flow path network. The flow path network defines the order in which elements are processed (e.g. Plane A flows into Plane B, which flows into Channel C). Each element and its relation to the other elements is defined by the user on the basis of map information. The soil cover provides the saturated hydraulic conductivity (K_s), effective capillary head (G), porosity, maximum saturation, rock content, and the infiltration recession factor. Land cover information provides Manning's *n* values for overland flow and interception heights.

First, the basin is divided into small, homogeneous elements. These are divided primarily on the basis of soil type and secondarily by slope. Natural boundaries such as forests and streams serve to divide the basin even further.

Second, the order in which the elements are processed must be determined. This sequence should simulate the overland flow path occurring in the field and can be determined from topographic maps.

Interception, infiltration, and overland flow parameter values were estimated for each computational element. Land cover data affecting interception and overland flow were estimated using satellite remote sensing data from the January 1991 *Conestoga Lakes Land Cover Mapping Project* (Reference 6), field investigation data, and the KINEMAT documentation (Reference 6). Infiltration values based on soil texture were estimated using the KINEMAT documentation and other literature (References 6, 8, and 9). Average values were used when the KINEMAT manual listed a range of values. Detailed definitions of KINEMAT parameters are included in the program documentation (Reference 7). The subdivision of the East Branch subbasin into KINEMAT model elements is shown in Exhibit 9.

4. Model Calibration

Next, the model was calibrated to an historic storm. A thorough search of the gage record was conducted to obtain hydrographs representing critical conditions. The criteria used to select hydrographs for calibration are listed below:

- large single peak
- short, intense rainfall
- quick recession, indicating direct runoff rather than subsurface flow

Of the 15 largest floods at the gage, only the storm of July 10 and 11, 1984, met all criteria. The peak instantaneous flow was 158 cfs and occurred at 1 a.m. The 11-hour cumulative rainfall over the East Branch subbasin for this event was 3.02 inches. Although several additional floods were investigated, they were found to have either inadequate rainfall data or a very small runoff peak relative to the slower subsurface component. The fact that this was by far the most significant runoff peak of all recorded events suggests that the East Branch subbasin may have been hydrologically "primed" to produce runoff. The Conestoga rain gage record indicates that approximately 3 inches of rain fell on the East Branch watershed on July 6, 7, and 8, 1984.

The hourly streamflows for the 1984 storm were obtained from the regional office of the USGS Water Resources Division. The hourly rainfall data at the Conestoga rain gage was assumed representative of the watershed, as no other rain gages are located in or near the East Branch subbasin.

Even for the 1984 storm, which produced by far the most significant runoff peak in the gage record, it was concluded that much of the streamflow is attributable to subsurface flow. Since KINEMAT cannot account for interflow, the 1984 storm was separated into subsurface flow and direct runoff. The direct runoff hydrograph peaks at approximately 85 cfs. The KINEMAT model was then calibrated to the runoff. It should be noted that the portion of the hydrograph attributed to "subsurface" flow consists of all delayed flow transmitted through the subsurface, some of which is true "baseflow" and some of which is shallow subsurface flow. The shallow subsurface flow, although it travels considerably faster than deep groundwater flow, is still delayed and diffuse relative to the direct runoff portion of the hydrograph.

The July 1984 hydrograph, with baseflow separation, is shown in Exhibit 10.

Initially, soil parameters required in the model, such as K_s (hydraulic conductivity) and G (capillary head), were estimated from the SCS Soil Interpretation Records (Reference 10).

To calibrate the model, adjustments were later made to the two most sensitive parameters: the saturated hydraulic conductivity (K_s) and the effective capillary head (G). The K_s and G values in the sandy upland soils were high enough that major changes did not affect the hydrograph, even when adjustments were made to the entire basin. The only areas in which a change did affect the model results were low-lying areas adjacent to the stream (these areas are described generically as "wetlands" in this report). K_s and G were reduced in the wetlands to produce results which matched the observed hydrograph. The revised values are still well within the ranges given in the KINEMAT manual. The calibrated runoff parameters for each soil unit in the East Branch subbasin are shown in Table 10. All KINEMAT and other input and output data files are included in Exhibit 11.

TABLE 10
Calibrated Runoff Parameters
East Branch Arrowhead River

Soil Unit	Percent of Subbasin	K_s (in/hr)	G (in)
109 (Carlin–Vermont)	17.6	0.85	9.8
116 (Bannister)	42.6	6.4	5.3
119 (Conestoga–Montcalm)	19.6	8.3	4.0
126 (Okee–Conestoga)	2.5	7.5	4.5
127 (Okee–Grinell–Anderton)	14.2	9.9	7.9
126 wetland	1.0	1.5	4.5
127 wetland	2.4	1.5	7.9
Stream Channel	0.1	0.0	0.0

5. Transfer of Parameters from KINEMAT to Whole-Basin HEC-1

The KINEMAT model, calibrated to the 1984 storm, was used to estimate the flood hydrograph on the East Branch resulting from the PMP. The warm-season PMP values were selected based on a 1,000-mi^2 storm, which optimizes PMP for the entire Arrowhead River basin. The East Branch subbasin's response to the whole-basin PMP is then representative of the entire basin's response to the same storm.

The East Branch PMF hydrograph predicted by the KINEMAT model has a peak of approximately 1,534 cfs when the whole-basin PMP is applied to the model. This hydrograph was entered into a HEC-1 model of the East Branch subbasin as an optimization hydrograph. The HEC-1 model was allowed to calibrate initial and constant loss rates to fit the KINEMAT results for the PMP. This procedure was carried out in order to determine equivalent basin average ("lumped") loss parameters which, for the PMP case, would yield the same volume and rate of runoff as the distributed model. The HEC-1 calibration yielded a basin average loss rate of 2.82 inches per hour for the PMS and was insensitive to initial loss. Since the East Branch subbasin is assumed representative of conditions in the entire basin, these optimized loss parameters are assumed applicable throughout the Arrowhead River basin. However, they are not applicable for any storm other than the whole-basin optimized PMS.

In addition to the calibrated loss rate, the entire Arrowhead basin was assumed to contain 3 percent impervious area, which accounts for lakes, reservoirs, rivers, and highways that do not occur in the East Branch subbasin (Reference 7).

C. Cool-Season

A separate analysis for cool-season conditions was not conducted for this study. A cool-season analysis is not relevant because daytime temperatures generally stay above freezing even in the winter months and snow is not an important factor in flooding on the Arrowhead River.

VIII. Coincident Hydrometeorological and Hydrological Conditions for the Probable Maximum Flood

A. Reservoir Level

The Austen Dam is operated as a run-of-river project and the annual maximum normal operating level is equal to the target pool level of 963.4 feet NGVD. The reservoir was assumed to be at this level at the beginning of the PMS.

B. Baseflow

It is apparent that high initial baseflows, probably as well as shallow subsurface flows, are associated with observed floods on the Arrowhead River. Each of the floods shown in Table 3 was associated with a high initial baseflow. The flood of September 1, 1966, has no discernible direct runoff component and appears to be almost entirely baseflow and interflow. All of the significant historical floods have extremely long hydrographs, without a well-defined end to the runoff period. To create a predictive model of the subsurface flow component of the PMF would be extraordinarily time-consuming and costly, and given the very limited database, the reliability of the results would not justify the effort. Instead, an empirical approach to the problem was taken by reviewing historic floods and considering a reasonable "envelope" value for the subsurface contribution.

The following two assumptions are the basis for this determination:

- The maximum rate of subsurface discharge to the stream network is physically limited, due to a number of components including drainage density, topography, and hydraulic conductivity in the subsurface. This rate is much slower than the overland flow component of the flood, as hydraulic conductivities in sands are on the order of inches per hour at the greatest. (Equating hydraulic conductivity with flow velocity assumes a hydraulic gradient of 1:1, which is much higher than the horizontal gradient occurring in a natural watershed).

- The maximum rate of subsurface discharge to the stream network physically possible has probably been approached over the last 40 years of record. Although no rainstorms approaching the PMP intensity have occurred, it is likely that some other combination of long- and short-term hydrologic conditions (seasonal groundwater table, rainfall, and antecedent storms) has resulted in a near-maximum condition of watershed saturation.

Therefore, it was assumed that the annual flood series at the Loup City gage reflects a reasonable "envelope" of subsurface contributions. Many of these floods, of course, also had a surface flow component, which is conservatively neglected in the estimate. Based on the flood records at Loup City and the other gage sites in the basin (Reference 11), it was determined that a conservative estimate of the subsurface flow coinciding with the PMP would be 3 cfs per square mile. Some of this would probably be quick-return flow due to the PMP, while some would be a function of antecedent groundwater conditions. This subsurface contribution was assumed to occur as a steady flow coincident with the PMS.

C. Snowpack

Snow is uncommon in this region and permanent snowpacks do not develop. Therefore, snowpack analysis is not applicable to this basin.

D. Snowmelt

Snowmelt is not applicable.

IX. PMF Hydrographs

A. Inflow PMF Hydrograph

A summary of the HEC-1 input parameters for the Austen Dam PMF is shown in Table 11.

TABLE 11
Summary of Input to HEC-1 Model
Austen Dam PMF

Total precipitation	17.0 in.
Peak 1-hour precipitation	3.09 in.
Basin area	1,267 sq. mi.
Starting baseflow	3,810 cfs
Recession constant	1.007
Flow at which recession begins	7,100 cfs
Clark Tc	48.7 hrs.
Clark R	75.9 hrs.
Initial Loss	2.0 in.
Constant loss rate	2.81 in./hr.
Impervious percent of basin	7.0
Starting reservoir elevation	963.4 ft.

The PMF inflow hydrograph is plotted in the HEC-1 output in Exhibit 12. The peak of the PMF inflow hydrograph is 12,956 cfs.

B. Sensitivity Analysis

Unit Hydrograph Parameters. No sensitivity analysis was conducted for unit hydrograph parameters. The verification unit hydrograph analysis justifies the use of the unit hydrograph parameters determined in the 1983 PMF study and shows that these parameters may be slightly conservative.

Loss Rates. Using the KINEMAT model study of the East Branch subbasin to represent loss rates in the entire basin is also conservative, as the East Branch subbasin has a slightly higher percentage of wetlands and low-permeability soils than the basin as a whole. However, there is some uncertainty as to the calibration of loss rates, since only one storm was available for analysis and the resulting parameters could not be verified. Therefore, a sensitivity analysis was conducted by varying the KINEMAT parameters K_s and G by ± 20 percent and repeating the KINEMAT/HEC-1 calibration and transfer process. The sensitivity of the peak PMF inflow to these parameters is summarized in Table 12. KINEMAT and HEC-1 runs for these analyses are shown in Exhibit 13.

TABLE 12
Sensitivity of the Peak PMF Inflow to KINEMAT Parameters K_s and G

Change in K_s (in./hr.)	Change in G	Q Peak (cfs)	Percent Change in Q Peak
+20%	–	11,580	–11
–20%	–	14,750	+14
–	+20%	12,400	–4.3
–	–20%	13,170	+1.7

For all the sensitivity cases investigated, the resulting peak PMF discharge is much less than the spillway capacity of 19,600 cfs. To change the conclusions of this report regarding spillway adequacy, K_s and G would have to be greatly different (much more than 20 percent) from the calibrated values. Therefore, for the purposes of evaluating spillway adequacy, the analyses reported here are sufficient.

Precipitation. The worst-case precipitation pattern was determined by using the HMR52 computer model for several possible storm sizes and orientations. No further sensitivity analysis was performed.

C. Reservoir Outflow PMF

The PMF inflow hydrograph was routed through the Austen Pond with the modified Puls reservoir routing routine in the HEC-1 model. The peak PMF outflow is 12,740 cfs. The flood could be passed at normal pool level of 963.4 feet.

References

1. Federal Energy Regulatory Commission, 1993. *Guidelines for the Evaluation of Hydropower Projects*. Chapter VIII, "Determination of the Probable Maximum Flood."

2. SCF Consultants, 1983. *Report on Inspection*, Austen Hydroelectric Project.

3. SCF Consultants, 1989. *Probable Maximum Flood Study*, Arrowhead River Basin.

4. National Weather Service, 1978. Hydrometeorological Report No. 51. Probable Maximum Precipitation Estimates - United States East of the 105^{th} Meridian. U.S. Department of Commerce (Weather Bureau), Washington, DC.

5. National Weather Service, 1982. Hydrometeorological Report No. 51. Application of Probable Maximum Precipitation Estimates - United States East of the 105^{th} Meridian. U.S. Department of Commerce (Weather Bureau), Washington, DC.

6. U.S. Department of Agriculture, Agricultural Research Service, 1990. KINEROS: A Kinematic Runoff and Erosion Model, Documentation and User Manual. ARS77.

7. Anystate Technical Information Bureau, 1991. *Conestoga Lakes Land Cover Mapping Project*.

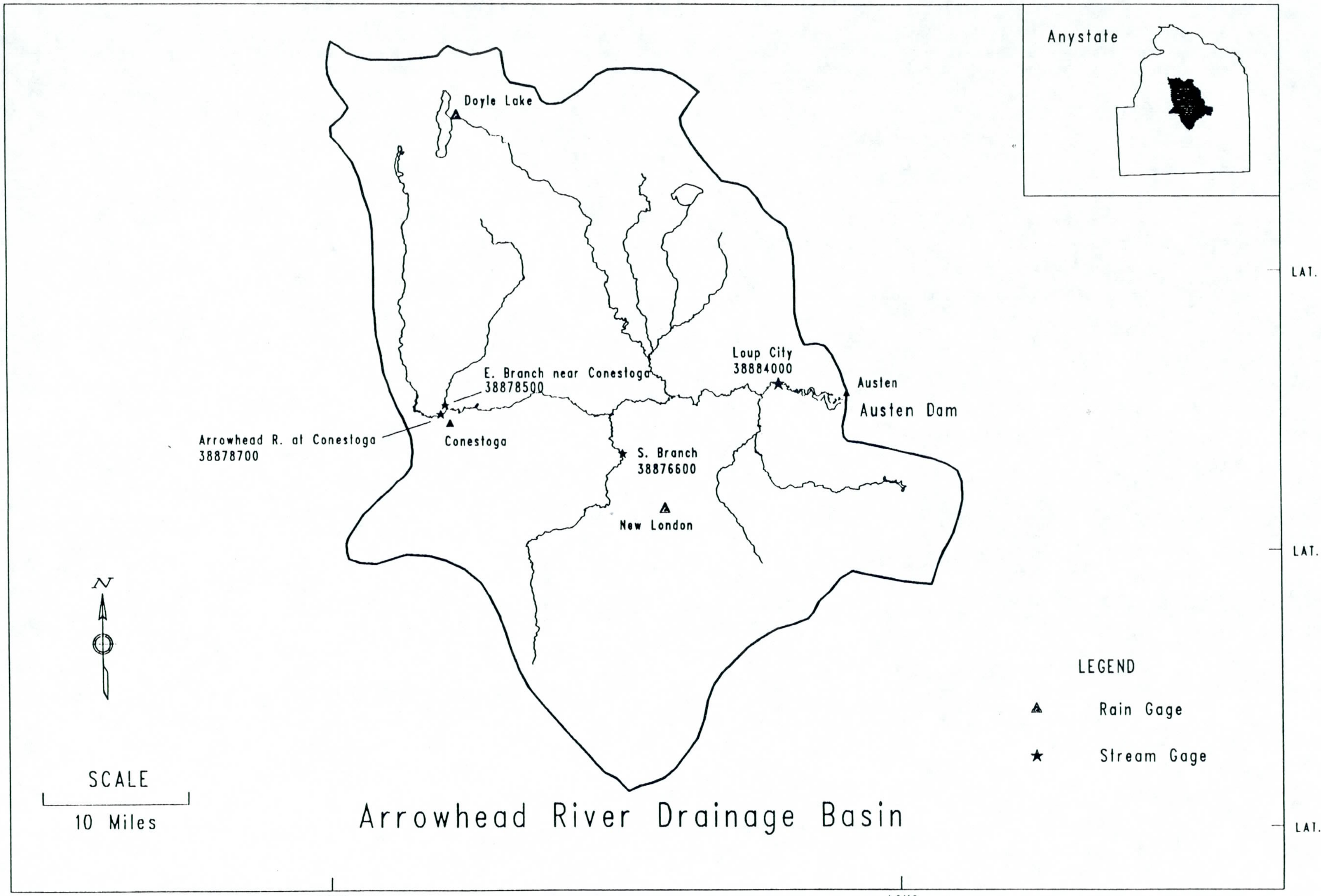
8. U.S. Department of Agriculture, Soil Conservation Service, 1981. Soil Survey Manual.

9. Trimble, G.R., T.S. Sartz and R.S. Pierce, 1987. *How Type of Soil Frost Affects Infiltration*, Journal of Soil and Water Conservation, 13(1) 81-82.

10. Soil Conservation Service, 1990-1992. Soil Interpretations Records.

11. U.S. Geological Survey Water Data Report, 1983. Water Resources Data, Anystate, Water Year 1983, Vol. 1.

EXHIBIT 1
Basin Map



Arrowhead River Drainage Basin

EXHIBIT 2

HEC-1 Unit Hydrograph Verification Input and Output

Note: Complete HEC-1 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

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*            VERSION 4.0.1 (LOCAL)          *          *        609 SECOND STREET            *
*                                           *          *     DAVIS, CALIFORNIA 95616         *
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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM
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                                   HEC-1 INPUT                                          PAGE  1

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    4          ID    FERC Project No. BBBB
    5          ID
    6          ID      Verifies to the flood of June, 1986
    7          ID
*** FREE ***
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31	QO	1113	1100	1088	1075	1063	1050	1038	1025	1017	1008
32	QO	1000	992	983	975	963	950	938	925	913	900
33	QO	892	883	875	867	858	850	833	825	817	808
34	QO	800	792	783	775	767	758	750	742	733	725
35	QO	717	708	700	692	683	675	667	658	650	642
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44	PW	1.00									
45	BA	1267									
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47	UC	48.7	75.9								
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*    HYDROLOGIC ENGINEERING CENTER      *
*          609 SECOND STREET            *
*       DAVIS, CALIFORNIA 95616         *
*           (916) 551-1748              *
*                                       *
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PMF Verification
Austen Hydroelectric Project
FERC Project No. BBBB

Verifies to the flood of June, 1986

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                  NDTIME        2200  ENDING TIME
                   ICENT          19  CENTURY MARK

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                 TOTAL TIME BASE  274.00 HOURS

      ENGLISH UNITS
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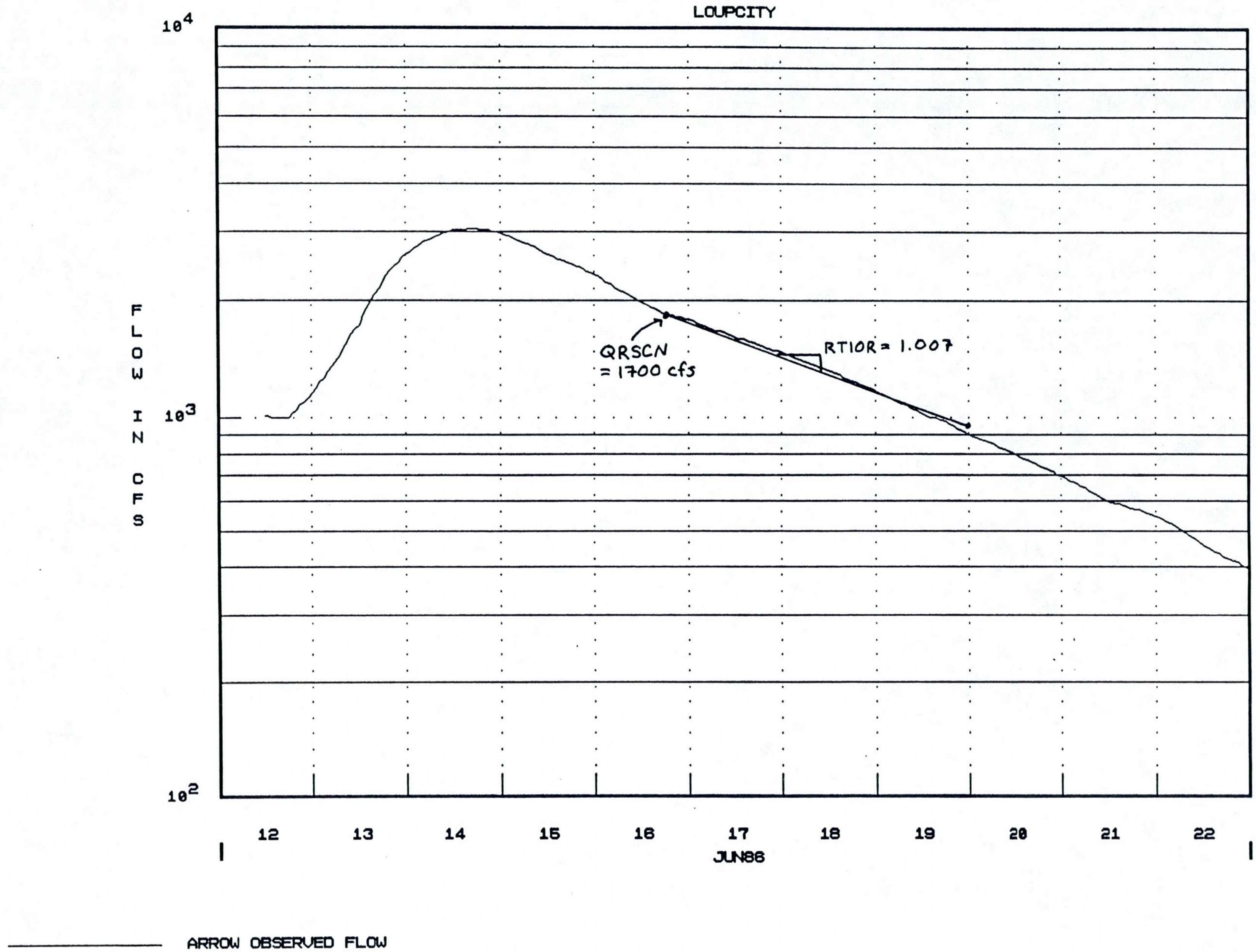
EXHIBIT 3

Baseflow Recession Plot for June 1986 Flood



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10^3
10^2
FLOW IN CFS
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RTIOR = 1.007
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14
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16
17
18
19
20
21
22
JUN86
ARROW OBSERVED FLOW

EXHIBIT 4
Relation of Calibrated Loss Rate to Rainfall Intensity

Exhibit 4
Relation of Calibrated Loss Rate to Rainfall Intensity

The following is a quantitative example why—for a simplified basin with both contributing and highly permeable (essentially noncontributing) areas—both model calibration and prediction can be in error when the infiltration characteristics of the two types of areas are spatially averaged. Although the example is very simple, its conclusions can be applied to more complex, real watersheds.

EXAMPLE:

1. Assume: **100-square-mile** basin, of which:
90 square miles have infiltration rate of **6 inches/hour**
10 square miles have infiltration rate of **0 inches/hour**

The calibration storm's peak 1-hour period produced **2 inches per hour**, and the maximum PMP increment is **4 inches per hour.**

2. Calculate: The basin average infiltration (loss) rate, based on the given infiltration rates, is equal to:

(10 sq. mi. X 0"/hr + 90 sq. mi. X 6"/hr)/(100 sq. mi) = 5.4 inches/hour.

3. Consider: Only the peak hour of each storm. Then a total of **2 inches** fell in the calibration storm, and a total of **4 inches** will fall in the PMP.

4. Calculate: The volume of runoff produced for:

(1) the calibration storm **(2 inches per hour)**:

90 square miles of the basin produced **0 inches** of runoff, because 2 inches/hour < infiltration rate of 6 inches/hour;
10 square miles of the basin produced **2 inches** of runoff.

Then the area average runoff is:

(10 sq. mi. X 2" + 90 sq. mi X 0")/100 sq. mi = 0.2 inches

And the calibrated loss rate is:

(2 inches-0.2 inches)/ 1 hour = 1.8 inches/hour.

(2) the PMP **(4 inches per hour)**.

Note that if the basin average of 5.4 inches/hour is applied to the PMP, the model will predict 0 runoff. But if the areas are considered separately:

90 square miles produces 0 inches of runoff, because the PMP is still less than the infiltration rate, and

10 square miles produces 4 inches of runoff.

Then the area average runoff is:

(10 sq. mi. X 4" + 90 sq. mi. X 0")/100 sq. mi = 0.4 inches.

And the average loss rate is:

(4 inches - 0.4 inches)/1 hour = 3.6 inches per hour.

(3) a larger PMP of **6 inches per hour**.

90 square miles produces **0 inches** of runoff;

10 square miles produces **6 inches** of runoff.

The area average runoff is:

(10 sq. mi. X 6" + 90 sq. mi. X 0")/100 sq. mi. = 0.6 inches

and the area average loss rate is:

(6 inches - 0.6 inches)/1 hour = 5.4 inches/hour, which is equal to the basin average loss rate estimated just from soil data.

(4) Finally, consider an even larger PMP of **8 inches per hour.**

90 square miles produces **2 inches** of runoff;

10 square miles produces **8 inches** of runoff.

The area average runoff is:

(90 sq. mi. X 2" + 10 sq. mi. X 8")/(100 sq. mi) = 2.6 inches.

and the area average loss rate is:

(8 inches - 2.6 inches)/1 hour = 5.4 inches per hour.

EXHIBIT 5

PMP Depth-Area-Duration Curves
Arrowhead River Basin

Austen Project -- PMP

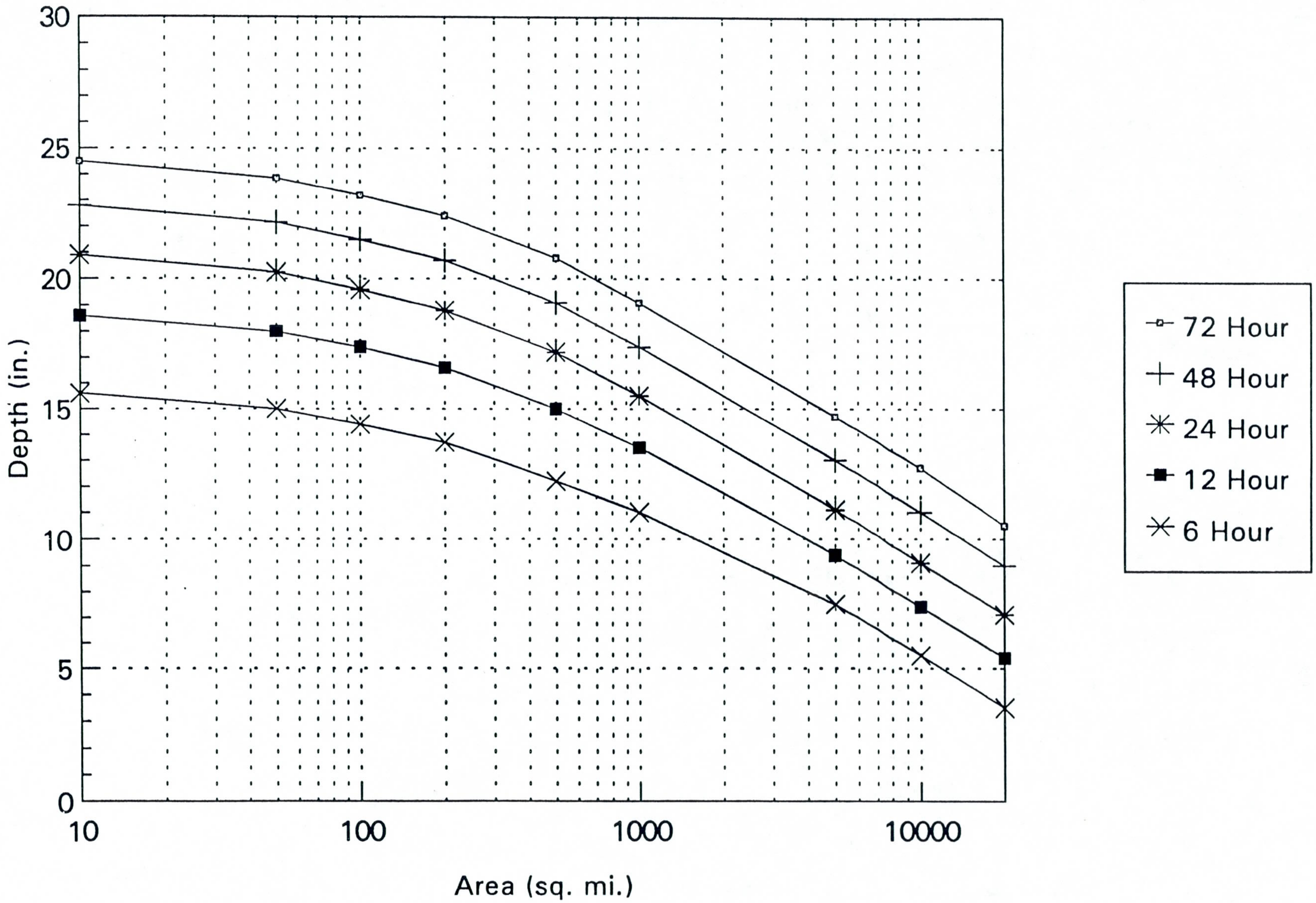

EXHIBIT 6
HMR52 Input and Output Data
(Probable Maximum Storm)

Note: Complete HMR-52 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

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*  THE HYDROLOGIC ENGINEERING CENTER    *
*          609 SECOND STREET            *
*        DAVIS, CALIFORNIA 95616        *
*   (916) 551-1748 OR (FTS) 460-1748    *
*                                       *
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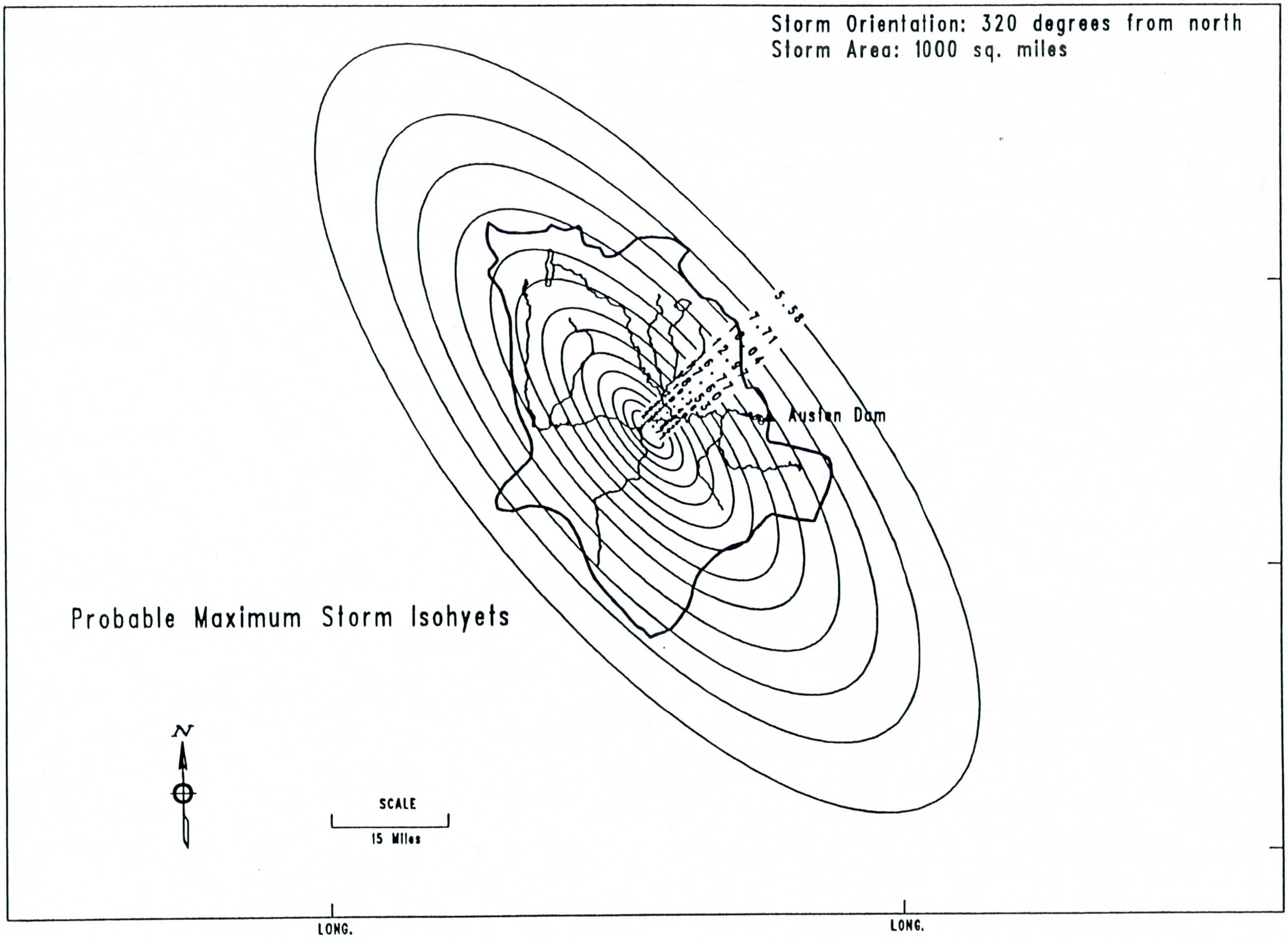
H     H  M     M  R   R          5     2
H     H  M     M  R    R   5     5   2
H     H  M     M  R     R   55555   2222222
```


LINE	ID.......1	2	3	4	5	6	7	8	9	10
1	ID									
2	ID									
3	ID									
4	ID PROBABLE MAXIMUM STORM FOR THE ARROWHEAD RIVER BASIN ***ALL SEASON***									
5	ID									
6	ID									
7	ID									
8	ID									
9	ID									
10	ID									
*** FREE ***										
11	PU ON									
12	BN ARRW									
13	BS 7.891									
14	BX 5.28	5.34	5.62	5.71	5.67	6.65	6.70	6.45	5.70	5.27
15	BX 4.53	4.15	3.66	2.78	2.33	1.89	1.37	1.18	1.84	1.57
16	BX 1.55	1.19	1.35	1.21	1.54	2.12	2.26	2.45	2.70	2.82
17	BX 3.04	3.11	3.30	3.57	4.13	4.49	4.30	4.68	4.85	5.18
18	BY 4.85	4.06	3.96	3.56	3.22	3.00	2.69	1.90	2.00	1.60
19	BY 1.11	0.44	0.14	1.10	2.05	2.26	2.20	2.27	3.16	4.06
20	BY 5.09	5.98	6.23	6.81	6.63	6.62	6.73	6.65	6.69	6.42
21	BY 6.35	6.23	6.25	6.47	6.49	6.29	6.03	5.51	5.48	5.12
22	HO 280									
23	HP 10	15.6	18.6	20.9	22.8	24.5				
24	HP 200	13.7	16.6	18.8	20.7	22.4				
25	HP 1000	11.0	13.5	15.5	17.4	19.1				
26	HP 5000	7.5	9.4	11.1	13.0	14.7				
27	HP 10000	5.5	7.4	9.1	11.0	12.7				
28	HP 20000	3.5	5.4	7.1	9.0	10.5				
29	SA 0	0	3							
30	ST 60	0.309	7	1.0						
31	ZZ									


```
1*****************************************
 *                                       *
 *   PROBABLE MAXIMUM STORM  (HMR52)     *
 *            NOVEMBER 1982              *
 *          REVISED 26 JUL 86            *
 *                                       *
 * RUN DATE  30/11/1993  TIME   9:47:53  *
 *                                       *
```

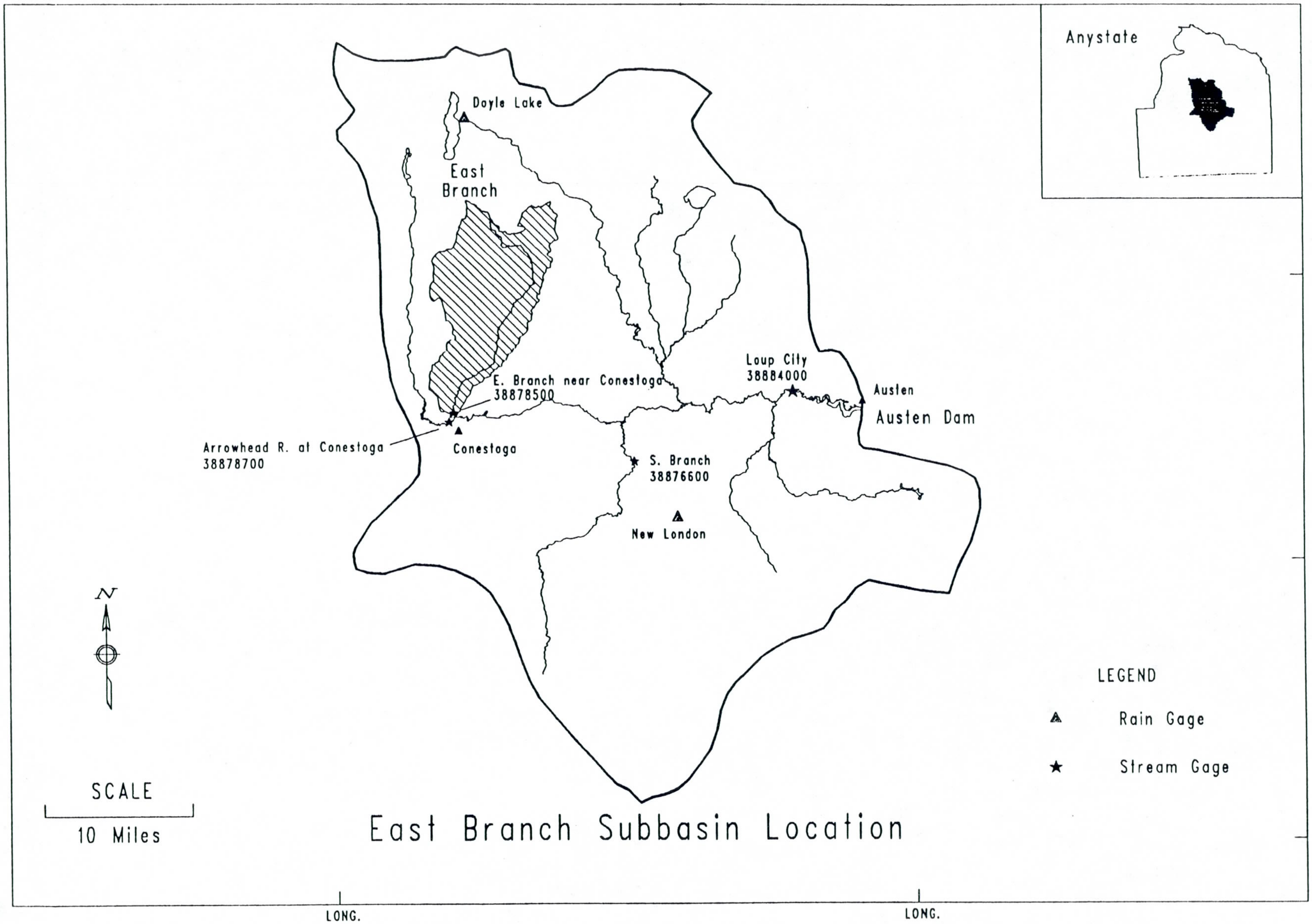
```
*****************************************
*                                       *
*    U.S. ARMY CORPS OF ENGINEERS       *
*  THE HYDROLOGIC ENGINEERING CENTER    *
*          609 SECOND STREET            *
*        DAVIS, CALIFORNIA 95616        *
*   (916) 551-1748 OR (FTS) 460-1748    *
*                                       *
```

EXHIBIT 7
Probable Maximum Storm Isohyetal Maps



Storm Orientation: 320 degrees from north
Storm Area: 1000 sq. miles
5.58
7.71
12.97
6.77
7.60
Austen Dam
LAT.
LAT.
LAT.
Probable Maximum Storm Isohyets
N
SCALE
15 Miles
LONG.
LONG.

EXHIBIT 8
East Branch Subbasin Location Map



East Branch Subbasin Location

EXHIBIT 9
KINEMAT Model Elements

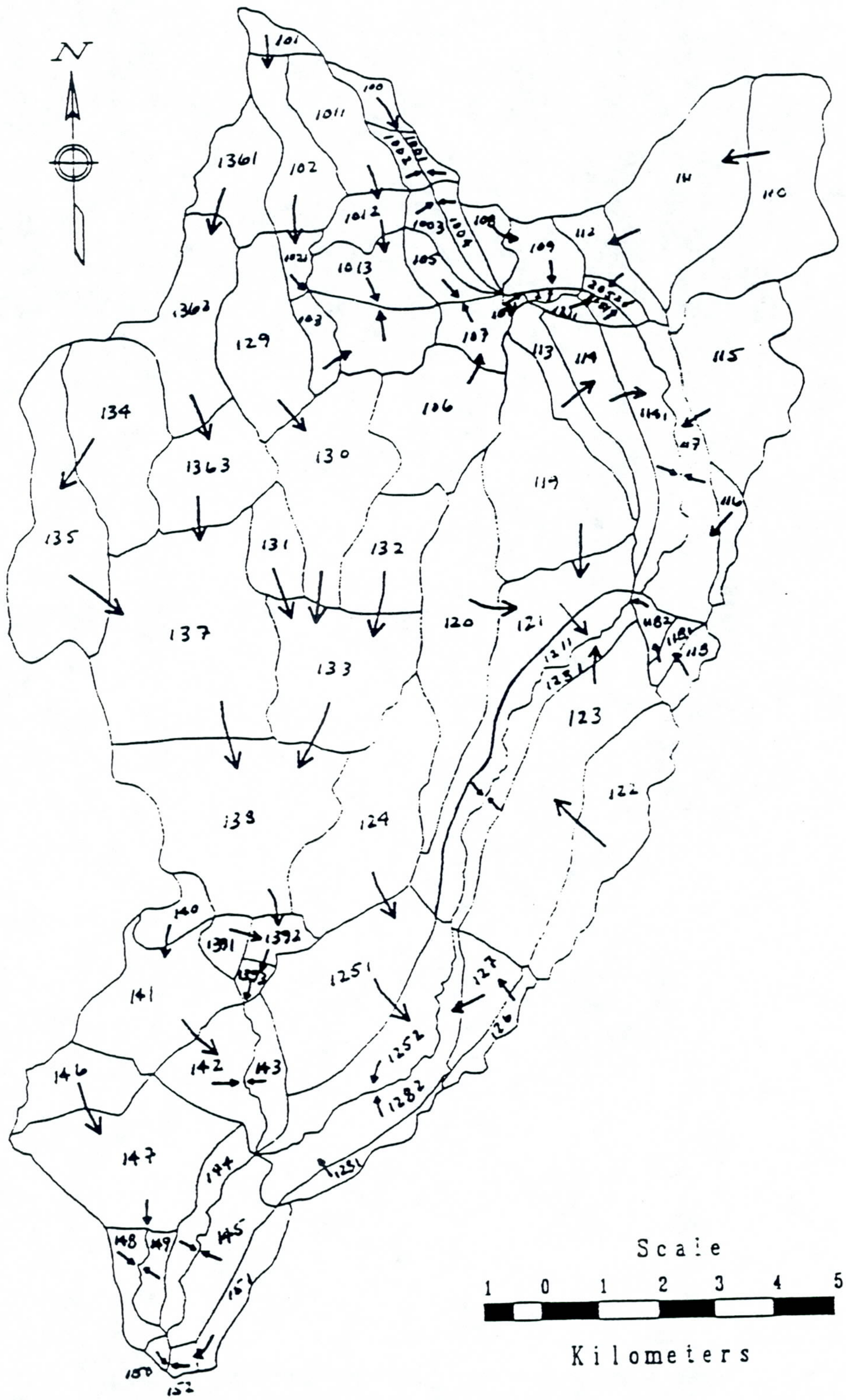
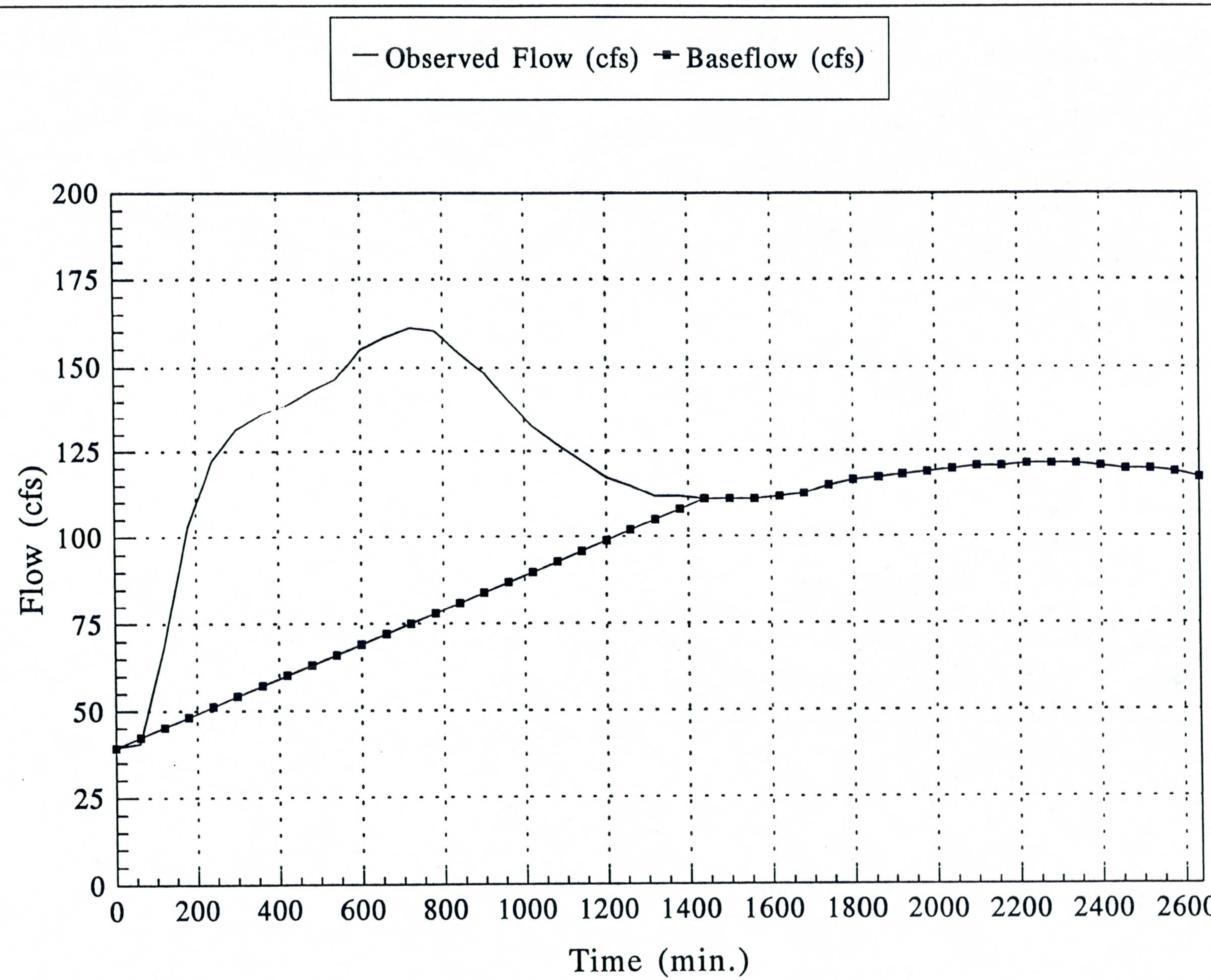

N
101
1011
102
1361
1363
1012
1013
129
134
1363
130
106
135
131
132
137
133
120
121
119
123
122
138
124
114
110
109
115
1251
1252
141
142
143
146
147
148
149
145
150
152
Scale
1 0 1 2 3 4 5
Kilometers

EXHIBIT 10
July 1984 Calibration Hydrograph

(A) With Baseflow Separation
(B) Calibration Hydrograph

East Branch Arrowhead River
1984 Storm -- Runoff Separated



East Branch Arrowhead River
1984 Storm

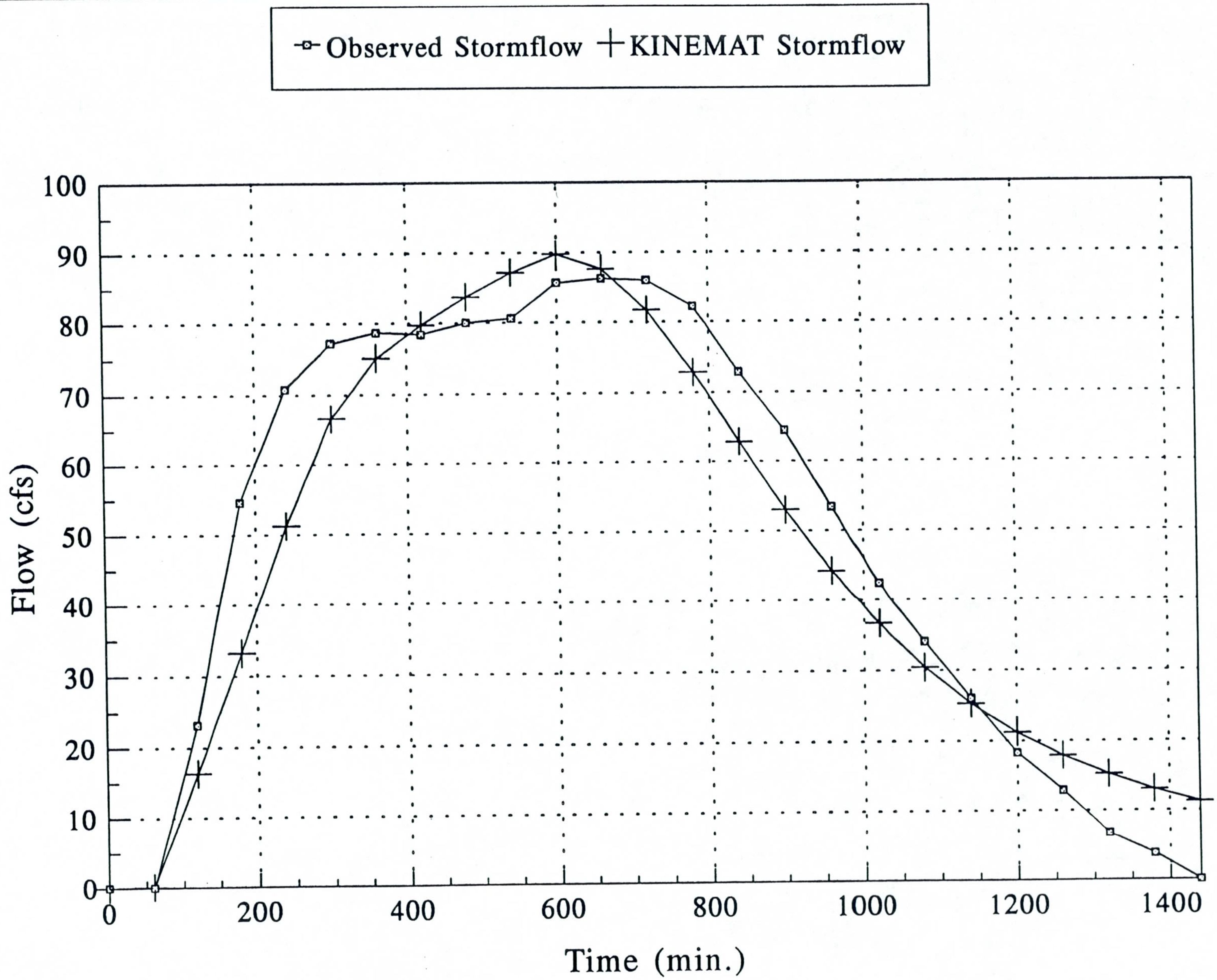


EXHIBIT 11
KINEMAT and HEC-1 Input and Output
East Branch Subbasin

Note: Complete KINEMAT and HEC-1 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

KINEMAT - Parameters

CHAR LENGTH	ENGLISH	METRIC
49564 1 0		

Duplicate from * to * for each element:

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
100	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
5200	1318	.006	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1001	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1400	4328	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1002	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
750	4488	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
200	2	100	1001	1002	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
3500	5	.01	.9	.9	5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.04	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1003	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
5500	7418	0.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1004	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1000	6930	.06	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
201	2	200	1004	1003	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
7000	5	.01	.9	.9	5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.04	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1011	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
8000	2873	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC FINC GSLOPE

0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1012	1	1011	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	5426	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1013	1	1012	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
3000	6904	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
101	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2400	2845	0.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
102	1	101	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
10000	2800	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	0.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1021	1	102	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
4000	4138	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
103	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
4000	1663	.008	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
104	1	103	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
3000	6284	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
202	2	1021	1013	104	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
6500	5	.01	.9	.9	5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.04	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3

105	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
5500	2203	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.02	1.0

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
106	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
7500	5597	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
107	1	106	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2500	4340	.04	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
203	2	202	107	105	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2700	5	.004	.9	.9	5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.04	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
108	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1250	6020	.06	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
109	1	108	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
4000	4455	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
7.5	4.5	.44	.95	.07	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1091	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
750	2904	.1	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1092	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
500	4356	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
7.5	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****
****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
204	2	1091	109	1092	203	201

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
5500	6	.008	.8	.8	7.5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.04	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
110	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
5500	11116	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
8.3	4.0	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
111	1	110	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
5000	11808	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
7.5	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
112	1	111	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2500	6691	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
1.5	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1121	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1500	3168	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER

.4	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
1.5	4.5	.44	.95	.35	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
2051	1	0	1121	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
300	2500	.0001	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.010	0	1	0	0	0	1.0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
2052	1	0	0	112	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
300	2500	.0001	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.010	0	1	0	0	0	1.0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
205	2	204	2051	2052	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	5	.0001	.9	.9	7.5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.010	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
113	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1650	10850	.08	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

```
KS          G           POR         SMAX        ROCK        RECS
6.4         5.3         .44         .92         .05         5.0
----------------------------------------------------------------------
FIDC        FINC        GSLOPE
0           0           0
**********************************************************************
**********************************************************************
ID NO       TYPE        NU1         NL1         NL2         NU2         NU3
114         1           113         0           0           0           0
----------------------------------------------------------------------
LENGTH      WID/DIA     SLOPE       BANK 1      BANK 2      MAX HT      IWOOL
2250        11358       0.007       0           0           0           0
----------------------------------------------------------------------
MANNING     CHEZY       GAGE        X           Y           INTER       COVER
.40         0           1           0           0           .020         1.00
----------------------------------------------------------------------
KS          G           POR         SMAX        ROCK        RECS
7.5         4.5         .44         .95         .05         5.0
----------------------------------------------------------------------
FIDC        FINC        GSLOPE
0           0           0
**********************************************************************
**********************************************************************
ID NO       TYPE        NU1         NL1         NL2         NU2         NU3
1141        1           114         0           0           0           0
----------------------------------------------------------------------
LENGTH      WID/DIA     SLOPE       BANK 1      BANK 2      MAX HT      IWOOL
2250        11358       0.007       0           0           0           0
----------------------------------------------------------------------
MANNING     CHEZY       GAGE        X           Y           INTER       COVER
.40         0           1           0           0           .020         1.00
----------------------------------------------------------------------
KS          G           POR         SMAX        ROCK        RECS
1.54        4.5         .44         .95         .05         5.0
----------------------------------------------------------------------
FIDC        FINC        GSLOPE
0           0           0
**********************************************************************
**********************************************************************
ID NO       TYPE        NU1         NL1         NL2         NU2         NU3
115         1           0           0           0           0           0
----------------------------------------------------------------------
LENGTH      WID/DIA     SLOPE       BANK 1      BANK 2      MAX HT      IWOOL
4200        13327       0.01        0           0           0           0
----------------------------------------------------------------------
MANNING     CHEZY       GAGE        X           Y           INTER       COVER
.40         0           1           0           0           .020         1.00
----------------------------------------------------------------------
KS          G           POR         SMAX        ROCK        RECS
8.3         4.0         .44         .95         .05         5.0
----------------------------------------------------------------------
FIDC        FINC        GSLOPE
0           0           0
**********************************************************************
**********************************************************************
ID NO       TYPE        NU1         NL1         NL2         NU2         NU3
116         1           0           0           0           0           0
----------------------------------------------------------------------
LENGTH      WID/DIA     SLOPE       BANK 1      BANK 2      MAX HT      IWOOL
1800        4476        .02         0           0           0           0
----------------------------------------------------------------------
MANNING     CHEZY       GAGE        X           Y           INTER       COVER
.40         0           1           0           0           .020         1.00
----------------------------------------------------------------------
KS          G           POR         SMAX        ROCK        RECS
8.3         4.0         .44         .95         .05         5.0
```

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
117	1	115	0	0	116	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
3000	16784	.008	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
1.54	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
206	2	205	1141	117	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
17000	6	.001	.8	.8	7.5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.05	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
118	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	3903	.04	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
7.5	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1181	1	118	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1700	8188	.002	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
7.5	4.5	.44	.95	.05	5.0

FIDC FINC GSLOPE

0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1182	1	1181	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	700	.0001	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.010	0	1	0	0	0	1.00

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
119	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
11600	5566	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
120	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
7000	8840	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
121	1	120	0	0	119	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1550	17088	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
7.5	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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*********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
1211      1         121       0         0         0       0
---------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
1550      17088     .02       0         0         0         0
---------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.40       0         1         0         0         .020       1.00
---------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
1.54     4.5       .44       .95       .05       5.0
---------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
*********************************************************************
*********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
122       1         0         0         0         0         0
---------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
3100      14902     .03       0         0         0         0
---------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.40       0         1         0         0         .020       1.00
---------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
8.3       4.0       .44       .95       .05       5.0
---------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
*********************************************************************
*********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
123       1         122       0         0         0         0
---------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
2550      16768     .004      0         0         0         0
---------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.40       0         1         0         0         .020       1.00
---------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
7.5       4.5       .44       .95       .05       5.0
---------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
*********************************************************************
*********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
1231      1         123       0         0         0         0
---------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
2550      16768     .004      0         0         0         0
---------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.40       0         1         0         0         .020       1.00
---------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
1.54     4.5       .44       .95       .05       5.0
---------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
*********************************************************************
*********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
```

207	2	1182	1231	1211	0	206

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
22000	7.5	.001	.7	.7	7.5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.055	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
124	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
8500	7284	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1251	1	124	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
5250	12054	.007	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
1.54	7.9	.45	.90	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1252	1	1251	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	12047	0.005	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
1.54	7.9	.45	.90	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
126	1	0	0	0	0	0

```
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
900       7685      .07       0         0         0         0
----------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.40       0         1         0         0         .020      1.00
----------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
8.3       4.0       .44       .95       .05       5.0
----------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
**********************************************************************
**********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
127       1         126       0         0         0         0
----------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
4000      3687      .04       0         0         0         0
----------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.40       0         1         0         0         .020      1.00
----------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
1.54      4.5       .45       .90       .05       5.0
----------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
**********************************************************************
**********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
1281      1         0         0         0         0         0
----------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
1800      8000      .05       0         0         0         0
----------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.40       0         1         0         0         .020      1.00
----------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
8.3       4.0       .44       .95       .05       5.0
----------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
**********************************************************************
**********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
1282      1         127       0         0         1281      0
----------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
2100      12173     .005      0         0         0         0
----------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.40       0         1         0         0         .020      1.00
----------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
 1.54      7.9       .45       .90       .05       5.0
----------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
**********************************************************************
**********************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
208       2         0         1252      1282      0         207
----------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
17400     10        .001      .6        .6        7.5       0
```

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.07	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
129	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
4500	9236	.003	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
130	1	129	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
5800	9085	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
131	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
6350	2485	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
132	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
4000	7283	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER

.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
133	1	130	0	0	131	132

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
8100	6914	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
.85	9.8	.45	.91	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
134	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2900	18244	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
6.4	5.3	.44	.92	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
135	1	134	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
7050	8636	.006	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
8.3	4.0	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1361	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
6000	6634	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

```
KS       G        POR      SMAX     ROCK     RECS
.85      9.8      .45      .91      .05      5.0
----------------------------------------------------------------------
FIDC     FINC     GSLOPE
0        0        0
**********************************************************************
**********************************************************************
ID NO    TYPE     NU1      NL1      NL2      NU2      NU3
1362     1        1361     0        0        0        0
----------------------------------------------------------------------
LENGTH   WID/DIA  SLOPE    BANK 1   BANK 2   MAX HT   IWOOL
11000    3420     .0001    0        0        0        0
----------------------------------------------------------------------
MANNING  CHEZY    GAGE     X        Y        INTER    COVER
.40      0        1        0        0        .020      1.00
----------------------------------------------------------------------
KS       G        POR      SMAX     ROCK     RECS
6.4      5.3      .44      .92      .05      5.0
----------------------------------------------------------------------
FIDC     FINC     GSLOPE
0        0        0
**********************************************************************
**********************************************************************
ID NO    TYPE     NU1      NL1      NL2      NU2      NU3
1363     1        1362     0        0        0        0
----------------------------------------------------------------------
LENGTH   WID/DIA  SLOPE    BANK 1   BANK 2   MAX HT   IWOOL
5500     6346     .02     0        0        0        0
----------------------------------------------------------------------
MANNING  CHEZY    GAGE     X        Y        INTER    COVER
.40      0        1        0        0        .020      1.00
----------------------------------------------------------------------
KS       G        POR      SMAX     ROCK     RECS
6.4      5.3      .44      .92      .05      5.0
----------------------------------------------------------------------
FIDC     FINC     GSLOPE
0        0        0
**********************************************************************
**********************************************************************
ID NO    TYPE     NU1      NL1      NL2      NU2      NU3
137      1        1363     0        0        135      0
----------------------------------------------------------------------
LENGTH   WID/DIA  SLOPE    BANK 1   BANK 2   MAX HT   IWOOL
12700    8292     .01      0        0        0        0
----------------------------------------------------------------------
MANNING  CHEZY    GAGE     X        Y        INTER    COVER
.40      0        1        0        0        .020      1.00
----------------------------------------------------------------------
KS       G        POR      SMAX     ROCK     RECS
6.4      5.3      .44      .92      .05      5.0
----------------------------------------------------------------------
FIDC     FINC     GSLOPE
0        0        0
**********************************************************************
**********************************************************************
ID NO    TYPE     NU1      NL1      NL2      NU2      NU3
138      1        137      0        0        133      0
----------------------------------------------------------------------
LENGTH   WID/DIA  SLOPE    BANK 1   BANK 2   MAX HT   IWOOL
15100    6962     .01      0        0        0        0
----------------------------------------------------------------------
MANNING  CHEZY    GAGE     X        Y        INTER    COVER
.40      0        1        0        0        .020      1.00
----------------------------------------------------------------------
KS       G        POR      SMAX     ROCK     RECS
6.4      5.3      .44      .92      .05      5.0
```

FIDC	FINC	GSLOPE				
0	0	0				

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1391	1	0	0	0	0	0
LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	2930	.04	0	0	0	0
MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00
KS	G	POR	SMAX	ROCK	RECS	
8.3	4.0	.44	.95	.05	5.0	
FIDC	FINC	GSLOPE				
0	0	0				

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1392	1	1391	0	0	138	0
LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
3000	2508	.01	0	0	0	0
MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00
KS	G	POR	SMAX	ROCK	RECS	
9.9	7.9	.45	.90	.05	5.0	
FIDC	FINC	GSLOPE				
0	0	0				

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
1393	1	1392	0	0	0	0
LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
750	1000	.0001	0	0	0	0
MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.020	0	1	0	0	0	1.00
KS	G	POR	SMAX	ROCK	RECS	
0	0	0	0	0	0	
FIDC	FINC	GSLOPE				
0	0	0				

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
140	1	0	0	0	0	0
LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
3500	4000	.004	0	0	0	0
MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00
KS	G	POR	SMAX	ROCK	RECS	
6.4	5.3	.44	.92	.05	5.0	
FIDC	FINC	GSLOPE				

0	0	0

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****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
141	1	140	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
4300	5500	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	.020	1.00

KS	G	POR	SMAX	ROCK	RECS
8.3	4.0	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
142	1	0	0	0	141	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	8500	.03	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	0.020	1.00

KS	G	POR	SMAX	ROCK	RECS
1.54	7.9	.45	.90	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
143	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1500	10000	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	0.020	1.00

KS	G	POR	SMAX	ROCK	RECS
1.54	7.9	.45	.90	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

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****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
209	2	1393	142	143	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
14000	7.5	.002	.6	.6	5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.12	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
144	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
750	9000	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	0.02	1.0

KS	G	POR	SMAX	ROCK	RECS
1.54	7.9	.45	.90	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
145	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	9000	.007	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	0.02	1.00

KS	G	POR	SMAX	ROCK	RECS
1.54	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
210	2	209	144	145	208	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
13000	12.5	.002	.5	.5	5	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.07	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
146	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
4000	6000	.02	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	0.020	1.0

KS	G	POR	SMAX	ROCK	RECS
8.3	4.0	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3

```
147      1        146       0        0        0        0
------------------------------------------------------------------
LENGTH   WID/DIA  SLOPE     BANK 1   BANK 2   MAX HT   IWOOL
9000     6600     .02       0        0        0        0
------------------------------------------------------------------
MANNING  CHEZY    GAGE      X        Y        INTER    COVER
.40      0        1         0        0        0.02     1.0
------------------------------------------------------------------
KS       G        POR       SMAX     ROCK     RECS
 1.54     7.9      .45       .90      .05       5.0
------------------------------------------------------------------
FIDC     FINC     GSLOPE
0        0        0
******************************************************************
******************************************************************
ID NO    TYPE     NU1       NL1      NL2      NU2      NU3
148      1        0         0        0        0        0
------------------------------------------------------------------
LENGTH   WID/DIA  SLOPE     BANK 1   BANK 2   MAX HT   IWOOL
2300     5000     .006      0        0        0        0
------------------------------------------------------------------
MANNING  CHEZY    GAGE      X        Y        INTER    COVER
.40      0        1         0        0        0.02     1.0
------------------------------------------------------------------
KS       G        POR       SMAX     ROCK     RECS
 1.54     4.5      .44       .95      .05      5.0
------------------------------------------------------------------
FIDC     FINC     GSLOPE
0        0        0
******************************************************************
******************************************************************
ID NO    TYPE     NU1       NL1      NL2      NU2      NU3
149      1        0         0        0        0        0
------------------------------------------------------------------
LENGTH   WID/DIA  SLOPE     BANK 1   BANK 2   MAX HT   IWOOL
1000     3500     .01       0        0        0        0
------------------------------------------------------------------
MANNING  CHEZY    GAGE      X        Y        INTER    COVER
.40      0        1         0        0        0.020    1.0
------------------------------------------------------------------
KS       G        POR       SMAX     ROCK     RECS
 1.54     7.9      .45       .90      .05       5.0
------------------------------------------------------------------
FIDC     FINC     GSLOPE
0        0        0
******************************************************************
******************************************************************
ID NO    TYPE     NU1       NL1      NL2      NU2      NU3
211      2        0         148      149      147        0
------------------------------------------------------------------
LENGTH   WID/DIA  SLOPE     BANK 1   BANK 2   MAX HT   IWOOL
6250     7.5      .001      .5       .5       5        0
------------------------------------------------------------------
MANNING  CHEZY    GAGE      X        Y        INTER    COVER
.12      0        1         0        0        0        0
------------------------------------------------------------------
KS       G        POR       SMAX     ROCK     RECS
0        0        0         0        0        0
------------------------------------------------------------------
FIDC     FINC     GSLOPE
0        0        0
******************************************************************
******************************************************************
ID NO    TYPE     NU1       NL1      NL2      NU2      NU3
150      1        0         0        0        0        0
------------------------------------------------------------------
```

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
1800	2000	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	0.02	1.0

KS	G	POR	SMAX	ROCK	RECS
1.54	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
151	1	0	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
8500	2000	.003	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	0.02	1.0

KS	G	POR	SMAX	ROCK	RECS
1.54	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
152	1	151	0	0	0	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
2000	2200	.01	0	0	0	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.40	0	1	0	0	0.02	1.00

KS	G	POR	SMAX	ROCK	RECS
1.54	4.5	.44	.95	.05	5.0

FIDC	FINC	GSLOPE
0	0	0

****

****

ID NO	TYPE	NU1	NL1	NL2	NU2	NU3
212	2	211	152	150	210	0

LENGTH	WID/DIA	SLOPE	BANK 1	BANK 2	MAX HT	IWOOL
3200	15	.001	.25	.25	10	0

MANNING	CHEZY	GAGE	X	Y	INTER	COVER
.055	0	1	0	0	0	0

KS	G	POR	SMAX	ROCK	RECS
0	0	0	0	0	0

FIDC	FINC	GSLOPE
0	0	0

****

****
******* Computed Hydrograph for East Branch Arrowhead River at Mouth ******
****

CHANNEL NO:	212		INFLOW:	.56017E+08	CU.FT
			INFILT:	.00000E+00	CU.FT
PEAK FLOW:	.15336E+04	CFS	STORAGE:	.17701E+05	CU.FT
AT TIME:	.27600E+04	MIN	OUTFLOW:	.55999E+08	CU.FT
			ERROR:	.49056E-03	PERCENT

MINUTES	CFS	IN/HR
.00	.00000	.00
60.00	.02056	.00
120.00	.07065	.00
180.00	.14066	.00
240.00	.21724	.00
300.00	.27948	.00
360.00	.32546	.00
420.00	.36624	.00
480.00	.40847	.00
540.00	.48338	.00
600.00	.56235	.00
660.00	.64263	.00
720.00	.71893	.00
780.00	.79859	.00
840.00	.89470	.00
900.00	1.00284	.00
960.00	1.18401	.00
1020.00	1.35094	.00
1080.00	1.53159	.00
1140.00	1.78234	.00
1200.00	2.13430	.00
1260.00	2.54778	.00
1320.00	2.94703	.00
1380.00	3.55320	.00
1440.00	4.21482	.00
1500.00	5.13515	.00
1560.00	6.42386	.00
1620.00	7.92834	.00
1680.00	9.37230	.00
1740.00	10.59131	.00
1800.00	12.99712	.00
1860.00	14.90875	.00
1920.00	16.63906	.00
1980.00	18.41293	.00
2040.00	20.26544	.00
2100.00	22.22286	.00
2160.00	24.37733	.00
2220.00	30.56834	.00
2280.00	40.56111	.00
2340.00	57.58884	.00
2400.00	196.83280	.00
2460.00	530.75200	.01
2520.00	909.39690	.02
2580.00	1188.63100	.03
2640.00	1384.29000	.03
2700.00	1510.15300	.03
2760.00	1533.55600	.03
2820.00	1433.39400	.03
2880.00	1241.96100	.03
2940.00	1017.92000	.02
3000.00	806.19260	.02
3060.00	628.83360	.01
3120.00	490.66370	.01

3180.00	385.52810	.01
3240.00	305.93220	.01
3300.00	245.43810	.01
3360.00	199.08050	.00
3420.00	163.18710	.00
3480.00	134.79050	.00
3540.00	112.40950	.00
3600.00	94.57173	.00
3660.00	80.24908	.00
3720.00	68.68152	.00
3780.00	59.28164	.00
3840.00	51.58587	.00
3900.00	45.08012	.00
3960.00	39.68143	.00
4020.00	35.13257	.00
4080.00	31.25966	.00
4140.00	27.94280	.00
4200.00	25.09230	.00
4260.00	22.63766	.00
4320.00	20.52104	.00
4380.00	18.34594	.00
4440.00	16.54613	.00
4500.00	15.00912	.00
4560.00	13.66969	.00
4620.00	12.50096	.00
4680.00	11.48261	.00
4740.00	10.59592	.00
4800.00	9.82160	.00
4860.00	9.13800	.00
4920.00	8.52429	.00
4980.00	7.96341	.00
5040.00	7.44318	.00
5100.00	6.95545	.00
5160.00	6.49492	.00
5220.00	6.05846	.00
5280.00	5.64476	.00
5340.00	5.25349	.00
5400.00	4.88479	.00
5460.00	4.53866	.00
5520.00	4.21465	.00
5580.00	3.91189	.00
5640.00	3.62929	.00
5700.00	3.36567	.00
5760.00	3.12000	.00
5820.00	2.89130	.00
5880.00	2.67882	.00
5940.00	2.48178	.00
6000.00	2.29951	.00

EVENT SUMMARY

WATERSHED AREA:	2028871000.00	SQ.FT
RAINFALL:	17.00	IN
INJECTED INFLOW:	.00	IN
BASEFLOW:	.00	IN
INTERCEPTION:	.02	IN
INFILTRATION:	16.55	IN
SURFACE STORAGE:	.00	IN
PEAK FLOW RATE:	.03	IN/HR
TOTAL OUTFLOW:	.33	IN
ERROR:	.59	PERCENT

```
*******************************************
*                                         *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)     *
*           FEBRUARY  1991                *
*           VERSION 4.0.1 (LOCAL)         *
*                                         *
* RUN DATE  12/01/93  TIME  10:29:01      *
*                                         *
*******************************************
```

```
*****************************************
*                                       *
*   U.S. ARMY CORPS OF ENGINEERS        *
*   HYDROLOGIC ENGINEERING CENTER       *
*       609 SECOND STREET               *
*     DAVIS, CALIFORNIA 95616           *
*       (916) 551-1748                  *
*                                       *
*****************************************
```

```
X     X  XXXXXXX   XXXXX          X
X     X  X        X     X        XX
X     X  X        X               X
XXXXXXX  XXXX     X       XXXXX   X
X     X  X        X               X
X     X  X        X     X         X
X     X  XXXXXXX   XXXXX         XXX
```

THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

```
LINE          ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

  1           ID    File = ARROW.HC1
  2           ID
  3           ID    OPTIMIZATION OF KINEMAT PMF HYDROGRAPH - KINEMAT FILE = EBRANCH.DAT
  4           ID    THIS RUN OPTIMIZES UNIT HYDROGRAPH PARAMETERS AND CONSTANT LOSS RATES
  5           ID      FOR THE EAST BRANCH ARROWHEAD SUBBASIN
  6           ID    RESULTS WERE THEN EXTENDED TO THE ENTIRE BASIN IN THE FILE = AUSTEN.HC1
  7           ID
  8           ID
  9           ID    OPTIMIZATION OF KINEMAT PMF HYDROGRAPH
 10           ID
*** FREE ***
 11           IT      60 01AUG99    0100     100
 12           IN      60
 13           OU      38      76
 14           IO       0       2
 15           PG     PMS
 16           PI   0.041   0.041   0.041   0.041   0.041   0.041   0.050   0.050   0.050   0.050
 17           PI   0.050   0.050   0.064   0.064   0.064   0.064   0.064   0.064   0.088   0.088
 18           PI   0.088   0.088   0.088   0.088   0.124   0.129   0.136   0.143   0.151   0.161
 19           PI   0.263   0.283   0.316   0.361   0.420   0.493   0.778   1.272   1.783   3.089
 20           PI   1.596   1.129   0.246   0.224   0.206   0.190   0.178   0.169   0.108   0.108
 21           PI   0.108   0.108   0.108   0.108   0.074   0.074   0.074   0.074   0.074   0.074
 22           PI   0.056   0.056   0.056   0.056   0.056   0.056   0.045   0.045   0.045   0.045
 23           PI   0.045   0.045
              *

 24           KK   ARROW
              *      FLOW FROM KINEMAT
 25           QO      .0      .0      .0      .1      .2      .2      .3      .3      .4      .4
 26           QO      .5      .6      .7      .7      .8     1.0     1.1     1.3     1.5     1.7
 27           QO     2.1     2.5     2.9     3.5     4.2     5.1     6.4     7.9     9.3    10.5
 28           QO    12.9    14.9    16.6    18.4    20.2    22.2    24.3    30.5    40.5    57.5
 29           QO   196.8   530.7   909.3  1188.6  1384.2  1510.1  1533.5  1433.3  1241.9  1017.9
 30           QO   806.1   628.8   490.6   385.5   305.9   245.4   199.0   163.1   134.7   112.4
 31           QO    94.5    80.2    68.6    59.2    51.5    45.0    39.6    35.1    31.2    27.9
 32           QO    25.0    22.6    20.5    18.3    16.5    13.6    12.5    11.4    10.5     9.8
 33           QO     9.1     8.5     7.9     7.4     6.9     6.4     6.0     5.2     4.8     4.2
 34           QO     3.9     3.6     3.3     3.1     2.8     2.4     2.2
 35           PR     PMS
 36           PW    1.00
 37           BA      73
 38           BF       0     .01   1.007
 39           UC    7.12    5.06
 40           LU     2.0   2.815    0.25
 41           ZW   A=EAST BRANCH B=ARROWHEAD C=FLOW F=CALCULATED
 42           ZZ
```

EXHIBIT 12

HEC-1 Input and Output
Austen Dam Basin

Note: Complete HEC-1 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

```
LINE            ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

   1            ID
   2            ID   AUSTEN HYDROELECTRIC PROJECT
   3            ID      PMF
   4            ID    FERC PROJECT NO.
   5            ID
   6            ID   LOSS RATES COMPUTED IN FILE: ARROW.HC1
   7            ID
*** FREE ***
   8            IT      60 01AUG99     0100      360
   9            IO       1       2
  10            PG   CONE
                *
  11            PI  0.041   0.041   0.041   0.041   0.041   0.041   0.050   0.050   0.050   0.050
  12            PI  0.050   0.050   0.064   0.064   0.064   0.064   0.064   0.064   0.088   0.088
  13            PI  0.088   0.088   0.088   0.088   0.124   0.129   0.136   0.143   0.151   0.161
  14            PI  0.263   0.283   0.316   0.361   0.420   0.493   0.778   1.272   1.783   3.089
  15            PI  1.596   1.129   0.246   0.224   0.206   0.190   0.178   0.169   0.108   0.108
  16            PI  0.108   0.108   0.108   0.108   0.074   0.074   0.074   0.074   0.074   0.074
  17            PI  0.056   0.056   0.056   0.056   0.056   0.056   0.045   0.045   0.045   0.045
  18            PI  0.045   0.045
                *

  19            KK  ARROW
  20            PR   CONE
  21            PW   1.00
  22            BA   1267
  23            BF   3801   -0.55   1.007
  24            UC   48.7    75.9
  25            LU    2.0   2.815     7.0
  26            ZW  A=AUSTEN B=ARROWHEAD C=FLOW F=CALC-INFLOW

  27            KK  ROUTE
  28            KO      1       2
  29            RS      1    ELEV   963.4
  30            SV   6000    8250   12750   14000   16200   17500   19300
  31            SE  963.2   963.4   964.0   965.5   967.5   969.5   971.5
  32            SQ      0    3801   43500   62000   81000  102000  122000  142000
  33            SE  963.2   963.4   965.5   967.5   969.5   971.5   973.5   975.5
  34            ZW  F=CALC-OUTFLOW
  35            ZZ
```

STATION ROUTE

```
                              (I) INFLOW,   (O) OUTFLOW
            0.       2000.       4000.      6000.      8000.      10000.     12000.     14000.       0.         0.         0.         0.         0.
                                                                                       (S) STORAGE
            0.         0.          0.         0.         0.         0.      6000.      8000.     10000.     12000.        0.         0.         0.
DAHRMN PER
 10100   1.---------.--------I.---------.---------.---------.---------.---------.S--------.---------.---------.---------.---------.
 10200   2.         .        I.         .         .         .         .        .S         .         .         .         .         .
 10300   3.         .        I.         .         .         .         .        .S         .         .         .         .         .
 10400   4.         .        I.         .         .         .         .        .S         .         .         .         .         .
 10500   5.         .       IO.         .         .         .         .        .S         .         .         .         .         .
 10600   6.         .       IO.         .         .         .         .        .S         .         .         .         .         .
 10700   7.         .       IO.         .         .         .         .        .S         .         .         .         .         .
 10800   8.         .       IO.         .         .         .         .        .S         .         .         .         .         .
 10900   9.         .       IO.         .         .         .         .        .S         .         .         .         .         .
 11000  10.         .       IO.         .         .         .         .        .S         .         .         .         .         .
 11100  11. . . . . . . . . I . . . . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . . . . . . . . . . .
 11200  12.         .       I .         .         .         .         .        .S         .         .         .         .         .
 11300  13.         .       I .         .         .         .         .        .S         .         .         .         .         .
 11400  14.         .       I .         .         .         .         .        .S         .         .         .         .         .
 11500  15.         .      IO .         .         .         .         .        .S         .         .         .         .         .
 11600  16.         .      IO .         .         .         .         .        .S         .         .         .         .         .
 11700  17.         .      IO .         .         .         .         .        .S         .         .         .         .         .
 11800  18.         .      IO .         .         .         .         .        .S         .         .         .         .         .
 11900  19.         .      IO .         .         .         .         .        S          .         .         .         .         .
 12000  20.         .      IO .         .         .         .         .        S          .         .         .         .         .
 12100  21. . . . . . . . .IO . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . .
 12200  22.         .      I  .         .         .         .         .        S          .         .         .         .         .
 12300  23.         .      I  .         .         .         .         .        S          .         .         .         .         .
 20000  24.         .      I  .         .         .         .         .        S          .         .         .         .         .
 20100  25.         .      I  .         .         .         .         .        S          .         .         .         .         .
        26.         .      I  .         .         .         .         .        S          .         .         .         .         .
        27.         .      I  .         .         .         .         .        S          .         .         .         .         .
 20400  28.         .      I  .         .         .         .         .        S          .         .         .         .         .
 20500  29.         .      I  .         .         .         .         .        S          .         .         .         .         .
 20600  30.         .      I  .         .         .         .         .        S          .         .         .         .         .
 20700  31. . . . . . . . .I. . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . .
 20800  32.         .      I  .         .         .         .         .        S          .         .         .         .         .
 20900  33.         .      I  .         .         .         .         .        S          .         .         .         .         .
 21000  34.         .      I  .         .         .         .         .        S          .         .         .         .         .
 21100  35.         .      I  .         .         .         .         .        S          .         .         .         .         .
 21200  36.         .      I  .         .         .         .         .        S          .         .         .         .         .
 21300  37.         .      I  .         .         .         .         .        S          .         .         .         .         .
 21400  38.         .      OI .         .         .         .         .        S          .         .         .         .         .
 21500  39.         .      OI .         .         .         .         .        S          .         .         .         .         .
 21600  40.         .      OI .         .         .         .         .        S          .         .         .         .         .
 21700  41. . . . . . . . .OI . . . . . . . . . . . . . . . . . . . . . . . . S . . . . . . . . . . . . . . . . . . . . . . . . .
 21800  42.         .       OI.         .         .         .         .        S          .         .         .         .         .
 21900  43.         .       OI.         .         .         .         .        .S         .         .         .         .         .
 22000  44.         .       O I         .         .         .         .        .S         .         .         .         .         .
 22100  45.         .       O .I        .         .         .         .        .S         .         .         .         .         .
 22200  46.         .        O. I       .         .         .         .        .S         .         .         .         .         .
 22300  47.         .        O.  I      .         .         .         .        .S         .         .         .         .         .
 30000  48.         .         O    I    .         .         .         .        . S        .         .         .         .         .
 30100  49.         .         .O    I   .         .         .         .        . S        .         .         .         .         .
 30200  50.         .         . O    I  .         .         .         .        . S        .         .         .         .         .
 30300  51. . . . . . . . . . . .O. .I. . . . . . . . . . . . . . . . . . . . . . .S. . . . . . . . . . . . . . . . . . . . . . . .
 30400  52.         .         .  O     I.         .         .         .        .  S       .         .         .         .         .
 30500  53.         .         .   O     I         .         .         .        .   S      .         .         .         .         .
        54.         .         .    O   .I         .         .         .        .   S      .         .         .         .         .
        55.         .         .     O  .  I       .         .         .        .   S      .         .         .         .         .
 30600  56.         .         .      O .   I      .         .         .        .    S     .         .         .         .         .
 30900  57.         .         .        O    I     .         .         .        .    S     .         .         .         .         .
 31000  58.         .         .        .O      I  .         .         .        .     S    .         .         .         .         .
 31100  59.         .         .        . O      I .         .         .        .     S    .         .         .         .         .
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31200  60.         .         .         .  O      I         .         .         .      S  .         .         .         .         .
31300  61.................................O.....I....................................S.........................................
31400  62.         .         .         .     O   .  I      .         .         .       S .         .         .         .         .
31500  63.         .         .         .       O .   I     .         .         .        S.         .         .         .         .
       64.         .         .         .        O.     I   .         .         .        S.         .         .         .         .
       65.         .         .         .         O      I  .         .         .         S         .         .         .         .
31800  66.         .         .         .         . O      I.         .         .         S         .         .         .         .
31900  67.         .         .         .         .  O      I         .         .         .S        .         .         .         .
32000  68.         .         .         .         .    O    . I       .         .         .S        .         .         .         .
32100  69.         .         .         .         .     O   .  I      .         .         . S       .         .         .         .
32200  70.         .         .         .         .       O .   I     .         .         .  S      .         .         .         .
32300  71.................................................O.....I..................................S...................................
40000  72.         .         .         .         .         O      I  .         .         .   S     .         .         .         .
40100  73.         .         .         .         .         . O     I .         .         .   S     .         .         .         .
40200  74.         .         .         .         .         .  O     I.         .         .    S    .         .         .         .
40300  75.         .         .         .         .         .   O     I         .         .    S    .         .         .         .
40400  76.         .         .         .         .         .    O    I         .         .     S   .         .         .         .
40500  77.         .         .         .         .         .     O   .I        .         .     S   .         .         .         .
40600  78.         .         .         .         .         .      O  . I       .         .     S   .         .         .         .
40700  79.         .         .         .         .         .       O .  I      .         .      S  .         .         .         .
40800  80.         .         .         .         .         .        O.  I      .         .      S  .         .         .         .
40900  81............................................................O...I...........................S.................................
41000  82.         .         .         .         .         .         .O  I     .         .       S .         .         .         .
41100  83.         .         .         .         .         .         .O  I     .         .       S .         .         .         .
41200  84.         .         .         .         .         .         . O  I    .         .       S .         .         .         .
41300  85.         .         .         .         .         .         . O  I    .         .       S .         .         .         .
41400  86.         .         .         .         .         .         .  O I    .         .        S.         .         .         .
41500  87.         .         .         .         .         .         .  O I    .         .        S.         .         .         .
41600  88.         .         .         .         .         .         .  O I    .         .        S.         .         .         .
41700  89.         .         .         .         .         .         .   I     .         .        S.         .         .         .
41800  90.         .         .         .         .         .         .   I     .         .        S.         .         .         .
41900  91..............................................................IO..............................S..................................
       92.         .         .         .         .         .         .  IO     .         .        S.         .         .         .
       93.         .         .         .         .         .         . IO      .         .        S.         .         .         .
42200  94.         .         .         .         .         .         .I O      .         .        S.         .         .         .
42300  95.         .         .         .         .         .         .I O      .         .        S.         .         .         .
50000  96.         .         .         .         .         .         I O       .         .       S .         .         .         .
50100  97.         .         .         .         .         .         I O       .         .       S .         .         .         .
50200  98.         .         .         .         .         .        I.O        .         .       S .         .         .         .
50300  99.         .         .         .         .         .       I .O        .         .       S .         .         .         .
50400 100.         .         .         .         .         .       I O         .         .       S .         .         .         .
50500 101........................................................I..O...........................S...................................
50600 102.         .         .         .         .         .      I O.         .         .      S  .         .         .         .
50700 103.         .         .         .         .         .     I  O.         .         .      S  .         .         .         .
50800 104.         .         .         .         .         .    I  O .         .         .      S  .         .         .         .
50900 105.         .         .         .         .         .    I  O .         .         .      S  .         .         .         .
51000 106.         .         .         .         .         .   I  O  .         .         .     S   .         .         .         .
51100 107.         .         .         .         .         .   I O   .         .         .     S   .         .         .         .
51200 108.         .         .         .         .         .  I  O   .         .         .     S   .         .         .         .
51300 109.         .         .         .         .         . I  O    .         .         .     S   .         .         .         .
51400 110.         .         .         .         .         . I  O    .         .         .     S   .         .         .         .
51500 111...................................................I..O..............................S.....................................
51600 112.         .         .         .         .         .I O      .         .         .    S    .         .         .         .
51700 113.         .         .         .         .         I  O      .         .         .    S    .         .         .         .
51800 114.         .         .         .         .        I. O       .         .         .   S     .         .         .         .
51900 115.         .         .         .         .        I. O       .         .         .   S     .         .         .         .
52000 116.         .         .         .         .       I .O        .         .         .   S     .         .         .         .
52100 117.         .         .         .         .       I .O        .         .         .   S     .         .         .         .
52200 118.         .         .         .         .      I  O         .         .         .   S     .         .         .         .
52300 119.         .         .         .         .      I O.         .         .         .  S      .         .         .         .
      120.         .         .         .         .     I  O.         .         .         .  S      .         .         .         .
      121............................................I..O..............................S.......................................
60200 122.         .         .         .         .    I  O .         .         .         .  S      .         .         .         .
60300 123.         .         .         .         .   I  O  .         .         .         . S       .         .         .         .
60400 124.         .         .         .         .   I  O  .         .         .         . S       .         .         .         .
60500 125.         .         .         .         .  I  O   .         .         .         . S       .         .         .         .
```

EXHIBIT 13
KINEMAT and HEC-1 Runs for Sensitivity Analysis

Note: Complete KINEMAT and HEC-1 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

```
KINEMAT - Parameters
--------------------------------------------------------------------------
CHAR LENGTH         ENGLISH   METRIC
49564 1 0
--------------------------------------------------------------------------
Duplicate from * to * for each element:
**************************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
100       1         0         0         0         0         0
--------------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
5200      1318      .006      0         0         0         0
--------------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.4        0         1         0         0         .020       1.00
--------------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
6.4       5.3       .44       .92       .05       5.0
--------------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
**************************************************************************
**************************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
1001      1         0         0         0         0         0
--------------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
1400      4328      .01       0         0         0         0
--------------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.4        0         1         0         0         .020       1.00
--------------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
.85       9.8       .45       .91       .05       5.0
--------------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
**************************************************************************
**************************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
1002      1         0         0         0         0         0
--------------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
750       4488      .01       0         0         0         0
--------------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.4        0         1         0         0         .020       1.00
--------------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
.85       9.8       .45      .91        .05       5.0
--------------------------------------------------------------------------
FIDC      FINC      GSLOPE
0         0         0
**************************************************************************
**************************************************************************
ID NO     TYPE      NU1       NL1       NL2       NU2       NU3
200       2         100       1001      1002      0         0
--------------------------------------------------------------------------
LENGTH    WID/DIA   SLOPE     BANK 1    BANK 2    MAX HT    IWOOL
3500      5         .01       .9        .9         5        0
--------------------------------------------------------------------------
MANNING   CHEZY     GAGE      X         Y         INTER     COVER
.04       0         1         0         0         0         0
--------------------------------------------------------------------------
KS        G         POR       SMAX      ROCK      RECS
0         0         0         0         0         0
```

SPECIAL CONSIDERATIONS

Wednesday 3:20 p.m.

8-3 Antecedent and Coincident Hydrometeorological Conditions

The inflow PMF hydrograph that produces the critical conditions within the reservoir and at the dam may depend on either the peak inflow rate or the timing and volume of PMF inflow, depending on spillway capacity and reservoir storage available at the beginning of the flood. Thus, the inflow PMF hydrograph could result from a high-intensity local storm, a general storm with a long duration, or a winter storm. This section discusses these considerations and their influence on PMF development procedures.

Caution: Although it may be possible to assess in advance whether the peak outflow and/or the maximum reservoir water-surface elevation will be produced by a local or a general storm, flood inflow hydrographs should be generated for each storm and then routed through the reservoir to clearly establish the PMF event.

8-3.1 Antecedent Conditions

The question has been raised as to whether a PMP flood hydrograph based solely on runoff from the PMP provides sufficiently small risk of exceedance for consideration of dam safety. In general, it does. Severe storms may be preceded by lesser ones; the real question of interest is: <u>What reservoir level is reasonable as the starting elevation when routing the inflow PMF through the reservoir, considering the possibility of antecedent storms</u>? It is advisable to determine if a water resources agency has conducted regional special studies related to antecedent storms. If so, the results should be considered for application. In the absence of antecedent storm information, the following four approaches are recommended as acceptable alternatives:

(1) <u>Consider that the reservoir surface is at a predefined annual maximum level at the start of PMF inflow</u>. It will be necessary to determine the annual maximum reservoir level for each dam, depending on the characteristics of the dam, its spillway and outlet works, and the historic and specified operation plans. If flashboards are normally used on the dam during the time of the PMF, they should be assumed to be in place for the determination of the annual maximum reservoir level. Routing of the PMF through the reservoir should assume that flashboards fail or collapse at their design level.

- For hydroelectric projects, the annual maximum reservoir level should be defined as the annual maximum normal operating level.

(2) <u>Use an operating rule curve, when available, to identify the reservoir surface corresponding to the maximum storage level for the season of the controlling PMP</u>. A 100-year, 24-hour storm—using the percentages of the 24-hour maximum temporal distribution developed for the PMP—should be assumed to end three days prior to the PMP. The runoff hydrograph from this 100-year storm should be routed through the reservoir using established project operating rules, with the beginning reservoir level at the normal maximum storage level for the season. The reservoir level at the beginning of inflow from PMP runoff should be taken as the level produced by the routed inflow from the 100-year storm, but it need not be greater than the annual maximum reservoir level.

(3) <u>Use or develop a wet-year rule curve to establish the reservoir level that would exist at the start of the inflow PMF</u>. To develop this rule curve, assume that the reservoir level at the

beginning of the inflow PMF is at the average of the five consecutive, highest wet-year reservoir levels occurring during the season of the critical PMP. The assumed starting level need not be higher than the annual maximum reservoir level.

(4) Analyze historical extreme floods and antecedent storms for the region. A possible procedure can be found in HMR 56 [NWS 1986]. If the analysis shows it is probable that antecedent storms do occur in the region and could significantly influence the maximum reservoir level and the magnitude of the routed PMF outflow, develop a storm that could reasonably be expected to occur antecedent to the PMP as follows:

(a) Prepare an arithmetic plot of the antecedent storm rainfall expressed as a percentage of the principal storm versus the principal storm rainfall in inches. Draw an envelope line of the maximum values and extrapolate to the estimated PMP depth.

(b) Determine the average time between the beginning of the antecedent storm and the following one.

(c) Read a total rainfall depth for the antecedent storm from plot obtained in step (a) by the total PMP depth.

(d) Set the time between the antecedent storm and the PMP equal to the average time interval determined in step (b).

(e) Use both the antecedent storm and the PMP to develop an inflow PMF hydrograph.

Average monthly flow should be obtained for the months during the season when the critical PMP would occur. Tabulated monthly average data are available in USGS water data reports. The average monthly flow for the month of the critical PMP should be added to the inflow PMF hydrograph before routing through the reservoir. When using HEC-1 this initial flow is the parameter STRTQ. For the particular case when the basin has been subdivided, the initial flow will already have been added as described in Section 8-10.5. For "ungaged" basins, the average monthly flow per square mile of drainage area, obtained from records for nearby "gaged" basins, should be used to compute the initial flow.

A reservoir cannot be drawn down at the beginning of the PMF storm when flood routing, unless a drawdown is documented as the normal operating procedure for the reservoir during an impending storm.

8-3.2 Coincident Hydrometeorological Conditions

Assume the pertinent physical conditions of soil-moisture content, frozen ground (see Section 8-10.3.3), and snowpack water equivalent that could reasonably be expected to occur antecedent to the PMP. If snowpack is apt to exist in at least part of the drainage area in the season when the critical PMP would occur, an antecedent 100-year snowpack (covering the area that could be subject to snowpack) should be assumed to exist at the time when the PMP occurs (see Section 8-10.2.1).

8-4 Data Acquisition

Hydrologic and meteorologic data are necessary to develop unit hydrographs. Primary objectives of data collection are as follows:

- To obtain basic precipitation and streamflow data to use in subsequent analysis.
- To enable the engineer to understand the hydrologic response of the basin to properly simulate the runoff process for the season when the critical PMF would occur.

In general, four types of data are recommended to develop a unit hydrograph, as follows:

- Streamflow records for major historic floods.
- Precipitation records for the storms that produced the historic floods.
- Physical characteristics of the watershed including topography, soil types, and land use.
- Snowpack and temperature records in the basin if snowmelt was a factor in historic floods.

In addition, it is necessary to understand the project's physical features, as well as those of upstream dams, to properly route flood hydrographs through the reservoir. This section describes the specific data needs.

Caution: Delays may be experienced in data collection. These can take the form of extended periods to retrieve data in storage and seasonal weather delays for field data collection. Appropriate time should be allotted (i.e., four to six months) for data collection.

8-4.1 Information from Previous Studies

As stated earlier, unit-hydrograph theory is recommended to develop the PMF inflow hydrograph. Since unit hydrographs are commonly developed and used in flood-control studies, local, state, or federal agencies with flood-control responsibilities may have already developed one for the basin of interest. If available and applicable, the use of such unit hydrographs can save considerable time and cost to develop the inflow PMF. This is particularly true for basins where the available streamflow or rainfall records may be less than desirable—in which case, it may be necessary to develop a new unit hydrograph with more recent data. Thus, it is necessary to search for previous flood studies for nearby dams and to inquire about the availability of relevant information. Sources of information about regional flood studies include:

- Local flood control districts
- COE district and division offices
- USBR regional offices
- TVA
- SCS state and district offices
- USGS district offices

APPENDIX II-A

Dambreak Studies

The evaluation of the downstream consequences in the event of a dam failure is a main element in determining hazard potential and formulating emergency action plans for hydroelectric projects. The solution requires knowledge of the lateral and longitudinal geometry of the stream, its frictional resistance, a discharge-elevation relationship at one boundary, and the time-varying flow or elevation at the opposite boundary.

The current state-of-the-art is to use transient flow or hydraulic methods to predict dambreak wave formation and downstream progression. The transient flow methods solve and therefore account for the essential momentum forces involved in the rapidly changing flow caused by a dambreak. Another technique, referred to as storage routing or the hydrologic method, solves one-dimensional equations of steady flow ignoring the pressure and acceleration contributions to the total momentum force. For the same outflow hydrograph, the storage routing procedures will always yield lower water surface elevations than hydraulic or transient flow routing.

When routing a dambreak flood through the downstream reaches appropriate local inflows should be included in the routing which are consistent with the assumed storm centering.

The mode and degree of dam failure involves considerable uncertainty and cannot be predicted with acceptable engineering accuracy; therefore, conservative failure postulations are necessary. Uncertainties can be circumvented in situations where it can be shown that the complete and sudden removal of a dam (or dams) will not endanger human life or cause significant property damage.

The following provides references on dambreak analyses and criteria which may prove useful as indicators of reasonableness of the breach parameters, peak discharge, depth of flow, and travel time determined by the licensee. In addition, **Section 6-2 and Appendix VI-C of Chapter VI of these Guidelines provides additional criteria on analytical requirements for dambreak analyses.**

I. REFERENCES

Suggested acceptable references regarding dam failure studies include the following:

A. Fread, D. L. "DAMBRK - The NWS Dam-Break Flood Forecasting Model," National Weather Service, Silver Spring, Maryland, 1988 Version. This (or the most recent version) is the preferred method for performing dambreak studies.

B. Fread, D. L. "NWS FLDWAV Model: The Replacement of DAMBRK for Dam-Break Flood Prediction", Proceedings, Association of State Dam Safety Officials, 10th Annual Conference, Kansas City, Missouri, September 26-29, 1993. ***Since this model combines the NWS DAMBRK model and the NWS DWOPER model, it is also considered the preferred method.***

C. Westmore, Jonathan N. and Fread, Danny L., "The NWS Simplified Dam-Break Flood Forecasting Model," National Weather Service, Silver Spring, Maryland, 1981. (Copy previously furnished to each Regional Office with a detailed example).

D. Fread, D. L., 1977: The development and testing of a dam-break flood forecasting model, "Proceedings, Dam-Break Flood Modeling Workshop," U.S. Water Resources Council, Washington, D.C., 1977, pp. 164-197.

E. Hydrologic Engineering Center, "Flood Hydrograph Package (HEC-1) Users Manual for Dam Safety Investigations," September, 1990.

F. Gandlach, D. L. and Thomas, W. A., "Guidelines for Calculating and Routing a Dam-Break Flood," Research Note No. 5, U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1977.

G. Cecilio, C. B. and Strassburger, A. G., "Downstream Hydrograph from Dam Failure," Engineering Foundation Conference on Evaluation of Dam Safety, 1976.

H. Soil Conservation Service, "Simplified Dam-Breach Routing Procedure," March 1979. (To be used only for flood routing technique, not dambreak discharge).

I. Chow, V. T., Open Channel Hydraulics, McGraw-Hill Book Company, Inc., New York, 1959, Chapter 20.

J. Henderson, F. M., Open Channel Flow, McMillan Company, New York, 1966, Chapters 8 and 9.

K. Hydrologic Engineering Center, "Flood Emergency Plans, Guidelines for Corps Dam," June 1980. (Forwarded to all Regional Engineers by memorandum dated February 11, 1981).

L. Hydrologic Engineering Center, "UNET, One-Dimensional Unsteady Flow Through a Full Network of Open Channels", September 1992.

II. CRITERIA

The following criteria may prove useful as an indicator of the reasonableness of a dambreak study:

A. If the dambreak analysis has been performed by an acceptable method (**References A and B are the preferred methods**), then generally only the breach parameters, peak discharge, and flood wave travel time should be verified as an indicator of the licensee's correct application of the method selected. Downstream routing parameters (i.e., Manning's "n") should be reviewed for acceptability and inundation maps should be reviewed for clarity and completeness of information (i.e., travel times). The following criteria are considered to be adequate and appropriate for verifying the selected breach parameters and peak discharge:

1. Breach Parameters - Most serious dam failures result in a situation resembling weir conditions. Breach width selection is judgmental and should be made based on the channel or valley width with failure occurring at the deepest section. The bottom of the breach should generally be assumed to be at the foundation elevation of the dam. Pages 2-A-8 through 2-A-11 of this appendix contain suggested breach parameters and should be used when verifying the selected breach parameters. For worst case scenarios, the breach width should be in the upper range while the time of failure should be in the lower range. However a sensitivity analysis is recommended to determine the reasonableness of the assumptions.

2. Peak Discharge - The peak discharge may be verified by use of equations (11) and (13) of Reference No. 1. Although the equations assume a rectangular-shaped breach, a trapezoidal breach may be analyzed by specifying a rectangular breach width that is equal to the average width of the trapezoidal breach.

<u>Equation 11</u>:

$$C=\frac{23.4A_s}{\overline{BR}}$$

Where:	C	=	constant
	A_s	=	reservoir surface area, in acres
	$\overline{BR}$	=	<u>average</u> breach width, in feet

<u>Equation 13</u>:

$$Q_{bmax}=3.1\overline{BR}\left(\frac{C}{(t_f+\frac{C}{\sqrt{H}})}\right)^3$$

Where:	Q_{bmax}	=	maximum breach outflow, in cfs
	t_f	=	time of failure, in hours
	H	=	maximum head over the weir, in feet

This equation for Q_{bmax} has been found to give results within +5% of the Q_{peak} from the full DAMBRK model.

In a rare case where a dam impounding a small storage volume has a large time of failure, the equations above will predict a much higher flow than actually occurs.

At a National Weather Service Dam-Break Model Symposium held in Tulsa, Oklahoma, June 27-30, 1983, Dr. Danny Fread presented an update to his simplified method. Equation 13 has been modified as follows to include additional outflow not attributed to breach outflow:

$$Q_{bmax}=Q_o+3.1\overline{BR}\left(\frac{C}{\left(t_f+\frac{C}{\sqrt{H}}\right)}\right)^3$$

Where: Q_o = Additional (non-breach) outflow (cfs) at time t_f (i.e., spillway flow and/or crest overflow) (optional data value, may be set to 0).

This equation has also been modified to address instantaneous failure, because in some situations where a dam fails very rapidly, the negative wave that forms in the reservoir may significantly affect the outflow from the dam.

3. Flood Wave Travel Time - Reasonableness of the flood wave travel time may be determined by use of the following "rule-of-thumb" approximation for average wave speed:

(a) Assume an equivalent rectangular channel section for the selected irregular channel section.

(b) Assume a constant average channel slope.

(c) Compute depth of flow from the following adjusted Manning's equation.

$$d=\left(\frac{Qn}{1.46B(S)^{0.5}}\right)^{0.6}$$

Where:

	d	=	depth of flow for assumed rectangular section, ft.
	Q	=	peak discharge, cfs
	B	=	average width (rectangular), ft.
	S	=	average slope, ft./ft.
	n	=	Manning's roughness coefficient

(d) Compute average velocity from Manning's Equation:

$$V=\frac{1.49\,(S)^{0.5}\,(d)^{0.67}}{n}$$

Where: V = average velocity, fps

(e) Compute wave speed, C (Kinematic velocity):

$$C=\frac{5}{3}V(0.68)$$

Where: C = wave speed (mph)

Note : 1 fps = 0.68 mph

(f) Determination travel time, TT

$$TT=\frac{X}{C}$$

Where: TT = travel time, hr.
X = distance from dam, mi.

Note: If the slope is flat, the following "rule-of-thumb" provides a very rough estimate of the wave speed:

$$C=2\,(S)^{0.5}$$

Where: C = wave speed, mph
S = average slope, ft./mi.

In addition, as a "rule-of-thumb", the dynamic routing (NWS) method should be used whenever severe backwater conditions at downstream areas occur and/or the slope is less

than 20 ft/mi. When these restrictions are not present normal hydrologic routing (HEC-1) may provide reasonable results. It is recommended that HEC-2 be used to determine the resulting water surface elevations when HEC-1 is used for the dambreak study.

The HEC-I Manual (Reference E) states that when "a higher order of accuracy is needed, then an unsteady flow model, such as the National Weather Service's DAMBRK should be used." Experience demonstrates that the higher order of accuracy is usually required. Therefore, the NWS DAMBRK model and the more recent NWS FLOODWAV model are the preferred methods and recommended for all situations requiring dambreak studies.

B. If a dambreak analysis has been performed by a method other than one of the suggested acceptable methods, the selected breach parameters, peak discharge, depth of flow and travel time of the flood wave shall be verified by one of the two methods:

1. **Unsteady Flow - Dynamic Routing Method (Recommended)**

The NWS "DAMBRK" Model (Reference A) and the NWS "FLOODWAV" Model (Reference B) are the recommended methods. Each FERC Regional Office has received the software using the NWS DAMBRK program and should use this program, as necessary, to verify dambreak studies. As the flood wave travels downstream, the peak discharge and wave velocity generally, but not always, decrease. This attenuation in the flood wave is primarily due to energy dissipation when it is near the dam and to valley storage as it progresses in an unsteady flow downstream. It is important that the NWS model be calibrated to historical floods, if at all possible.

2. **Steady Flow Method (Provides a rough estimate)**

If this method is selected, the breach parameters and peak discharge shall be verified as in part "A" above. The method described below should be utilized only for preliminary assessments and the obtained values may be far from the actually expected results. Sound judgement and extensive numerical experience is necessary when evaluating the results.

For a rough estimate of the travel time and flood wave, it is recommended that one of the following two steady state methods be used for verification of the licensee's values:

a. When steam gage data are available, the depth of flow and travel time can be estimated as follows (This method will indirectly take valley storage into consideration):

(1) Identify existing stream gages located downstream of the dam.

(2) Obtain the stage-discharge curve for each gage.

(3) Assuming Q_{peak} remains constant, extrapolate the curves to the Q_{peak} value of the flood wave and determine the corresponding water surface elevation.

(4) Using the continuity equation to determine the velocity, estimate the travel time between each cross-section.

b. When stream gage data is not available, the depth of flow and travel time can be estimated based on the following steady-state method:

(1) Assume the area downstream of the dam is a channel. This will neglect valley storage.

(2) Identify on topographic maps all abrupt changes in channel width and/or slope. Using this as a basis, select and plot channel cross-sections.

(3) Assume Q_{bmax} remains constant throughout the entire stream length under consideration.

(4) Selecting a fairly rough Manning's n value, determine the depth of flow by applying Manning's equation to each cross-section. Assume the energy slope is equal to the slope of the channel.

(5) Using the continuity equation to determine the velocity, estimate the travel time between each cross-section.

C. The above criteria for breach parameters, peak discharge, depth of flow, and travel time should provide the necessary "ballpark figures" needed for comparison with licensee's estimates. When large discrepancies in compared values exist, or questions arise about assumptions to be made, or it appears that an extensive review will be necessary, the Regional Director should contact the Washington Office, D2SI for guidance. The methodology used by the licensee should be a part of the study and should be requested if not included.

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
	$\overline{BR}$ = Multiple Slabs	Buttress
	$\overline{BR}$ = Width of 1 or more	Masonry, Gravity Monoliths,
	Usually BR $\leq$ 0.5 W	
	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 = O	Masonry, Gravity Timber Crib, Buttress
	$\frac{1}{4} \leq Z \leq 1$	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, Buttress
	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 2-A-12 for definition Sketch

**Comments: See Page 2-A-10 - 2-A-11*

Comments:

1. $\overline{BR}$ is the <u>average</u> 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 width will probably be. Time to failure is the time from the start of the breach formation until the complete breach is formed. It does not include the time leading up to the start of the breach formation. For example, the time to erode away the downstream slope of an earth dam is not included. In this situation, the time to failure commences after sufficient erosion of the downstream slope has occurred and actual formation of the breach (the lowering of the crest) has begun.

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

5. Breach width assumptions should be based on the type of dam, the height of dam, the volume of the reservoir, and the type of failure (e.g. piping, sustained overtopping, etc.). Slab and buttress dams require sensitivity analyses that vary the number of slabs assumed to fail.

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 the range, and the Manning's "n" value should be in the upper portion of the recommended range. In order to fully evaluate the impacts of a failure on downstream areas, a sensitivity analysis is required to estimate the confidence and relative differences resulting from varying 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 for sections immediately below the dam and up to several thousand feet or more downstream of the dam should be assumed to be larger than the maximum value suggested by field investigations in order to account for uncertainties 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 resulting water surface elevation at selected locations downstream from the dam for each run on the same graph. Compare the differences in elevation with respect to distance downstream from the dam for the two cases.

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 (elevation-distance downstream) of both runs on the same graph to compare 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 natural flood without dambreak assuming maximum probable Manning's "n" value.

2. Use criteria in a. 1.

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

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

8-5 Review and Assessment of Data

Before using the data obtained in Section 8-4 to develop the PMF for the project basin, the data must be reviewed for accuracy and adequacy. The selection of antecedent conditions as addressed in Section 8-3 will be assessed in relation to data collected in Section 8-4 and applied in Section 8-10. This section discusses the review processes and acceptance criteria.

8-5.1 Unit Hydrographs

Any unit hydrograph available from a previous study for the project basin or from a regional study must be reviewed and tested for its ability to reproduce major flood hydrographs. The best means of proving applicability of the unit hydrograph is to use it to reconstitute the largest of the historic flood hydrographs chosen for review.

- If the reconstituted flood hydrograph agrees well with the historic flood hydrograph, the unit hydrograph can normally be accepted without adjustment. Acceptance will depend on the historic flood magnitude and is further discussed in Section 8-10.

- If the available unit hydrograph does not reasonably reproduce major floods or is judged not to do so due to changes in basin characteristics or error in the assumed time distribution of rainfall excess, a new unit hydrograph will be required. Unit-hydrograph development is discussed further in Sections 8-8 and 8-9.

Caution: It is important to determine the magnitude and importance of the flood hydrographs that were used in producing the unit hydrograph. If the floods used were not of major significance, the unit hydrograph may not accurately predict the peak and timing of major floods. Compensating for such nonlinear effects is considered in Section 8-10.

8-5.2 Flood Data

The first task in the review of the flood data is to ensure that the historic floods used are the largest for which records are available. They should be the maximum floods of record and should preferably have occurred during the season of the critical PMP.

- It is important to note the cause of the floods (e.g., thunderstorm, general storm, hurricane, snowmelt, or rain-on-snow).

Caution: Floods caused by ice jams, debris blockage, or dam break should not be used in unit-hydrograph analysis.

Flood data must be reviewed for accuracy. The flood hydrographs should be plotted to detect discontinuities and suspicious peaks or lows in the recorded flow. Historical ratings, including methods used to extend the range for extreme floods, should be reviewed to make certain that the conversion of recorded stage to discharge was done correctly. Original stage records can usually be obtained from the local USGS district office or the gage owner if questions arise regarding accuracy of recorded flood flows.

- If a slope-area method was originally used to extend the rating curve, a check should be made to ensure that control did not shift to another location during the flood. This may require a computed water surface profile for the reach.

- If questionable aspects of the flood data cannot be resolved, the data should not be used further in unit-hydrograph development.

- If changes in watershed characteristics have occurred since the time of the historic flood, adjustments may be necessary to adequately model the new situation. For example, if the percentage of a watershed's impervious area has changed, the input to the runoff model can be adjusted to reflect the new percentage. Clearcutting of large areas of forests may require changes in both initial abstractions and infiltration rates to reflect changes. Such land use changes will affect the unit hydrograph as well as losses.

If no floods have been recorded within the basin of interest, flood records from other basins in the region will need to be evaluated for applicability to unit-hydrograph development. This procedure has been covered separately in Section 8-9.

- Ideally, unit hydrographs should not be developed from storms that produced less than 1 inch of runoff.

  Caution: Noncontributing areas may cause average runoff over the total drainage area to be less than 1 inch. Special studies may be required to develop an appropriate unit hydrograph.

8-5.3 Precipitation Data

Hyetographs for each storm at each recording rain gage should be plotted and examined for consistency, continuity, accuracy, and completeness. Storm totals and the time distributions for all rain-gage records should be compared to detect obvious inconsistencies. Gaps in records can usually be filled by using regression and correlation analysis with records from nearby gages. An isohyetal map of total rainfall for the storms of interest should be prepared using all acceptable rain-gage records. The location of individual isohyets, for zones obviously influenced by orographic effects, can be drawn parallel to elevation contours when the density of rain gages is insufficient to clearly define the rainfall pattern throughout the area. The general pattern should be compared to mean annual or 100-year isohyetal patterns, which can be obtained from Technical Paper 40 or National Oceanic and Atmospheric Administration Atlas II, published for individual states by the NWS.

Comparisons of the hyetographs and the flood hydrographs should be made to identify suspicious differences in timing between a storm's beginning and end and the rise, recession, and peak of the flood hydrographs.

- If a major timing difference is noted, additional study of the original recorded data records should be performed.

- The hyetographs from nearby rain gages should be checked to determine if the timing difference is due to a clock problem with the rain gage or the stage recorder.

- Rainfall records at the gage should also be analyzed to detect any trends that may coincide with changes in locations of gages or in conditions around them.

- Double-mass analysis or regression methods may be used to adjust rain-gage records to remove spurious trends and produce a homogeneous rainfall record.

Caution: Timing adjustments should not be made to the records unless the irregularity is minor or the source of the error can be positively identified.

The lag time should be measured as the elapsed time between the centroid of the hyetograph and the peak of the flood hydrograph. Other definitions of the lag time are often used and some are included in the Glossary.

Because most rain gage records will be available only as daily totals, the records from the most appropriate recording gage(s)—usually the nearest gage with a complete record—should be used in disaggregating daily records to the required temporal distribution. In assembling daily records it is important to note the time at which each daily gage was read, so that all daily totals can be adjusted to a common daily total.

8-5.4 Snowpack Data

Snowpack data will be required for those basins where snowmelt has been or may be a contributing factor to major floods. The required snowpack-related data include the portion of the basin covered by snow, water equivalent of the snow depth, and hourly or maximum average daily temperatures.

8-5.4.1 Water-Equivalent Data

Snowpack water-equivalent data for snowcover that existed during historic storms should be reviewed for completeness, consistency, and adequacy. Adequacy is determined by plotting the recorded snowpack water-equivalent depths against elevation. It is necessary to decide if data are sufficient to define an altitude-depth relationship for the basin, including the lowest elevation of snowcover for mountainous regions.

- If data are available from only one snow course in the basin, which is often the case, data from other basins with a similar orientation and exposure should be obtained.

- If applicable data from other snow courses are not available in sufficient quantity at different altitudes, undefined portions of the altitude-snowpack estimate can be proportioned in accordance with the isohyetal maps for annual basin rainfall.

- It is possible to reconstitute snowpack data for historic floods through the use of runoff models such as the Hydrological Simulation Program-Fortran [Crawford and Linsley 1966] or the Sacramento Model [Burnash, et al. 1973]. If no snowpack is available, but is required to study the historic floods, such a procedure may be necessary.

8-5.4.2 Temperature Data

Temperature data should be reviewed for accuracy and for applicability in analyzing historical snowmelt.

8-5.5 Data on Reservoir Volume, Spillway and Outlet-Works Capacity, and Operation Policy

Data on the operating history and performance characteristics of the spillway and outlet works, as well as on the reservoir storage volume, are required. Knowledge of operating policies during extreme floods will also be required for routing the inflow PMF hydrograph.

8-5.5.1 Reservoir Volume

Data for reservoir area and volume should be reviewed for accuracy and possible changes occurring since the relationship was formulated.

- Available data on sediment deposition in the active storage of the reservoir should be reviewed to assess the need for adjustment of the reservoir area and volume characteristics.

- If measured data are not available, visual observations of the reservoir's upper reaches should be made.

- If deposition in the active storage area at the head of the reservoir appears to be significant, an estimate of the deposited volume should be made using whatever data can be readily assembled.

- Unless the volume of deposition is large, its effects on the PMF hydrograph will not be important. However, if the reduction in active storage volume appears to be 5 percent or greater, a survey of sediment deposited in the active storage volume and the development of new reservoir area-elevation-capacity curves should be considered.

- If it appears that deposition exceeds one percent per year, an allowance should be made for future deposition between the time the survey is made and the time the next inspection is due.

8-5.5.2 Spillway and Outlet Works Capacity

The relationships for capacity of spillways and outlet works should be checked in accordance with available discharge coefficients for tested hydraulic structures, such as those given in the COE Hydraulic Design Criteria [COE 1989]. For unusual spillway crest shapes, the USBR publication "Discharge Coefficients for Irregular Overfall Spillways" [Bradley 1952] and the "Handbook of Hydraulics" [King and Brater 1954] provide additional guidance. Because approach conditions and site-specific geometry can affect the magnitude of discharge coefficients, precise agreement should not be expected.

- If differences of 10 percent or more are apparent, the source of the original discharge-capacity estimates should be reviewed.
- If adequate physical model studies have been made for the structures to experimentally determine the discharge relationships, they can be accepted.
- If such studies have not been made, values from verified references of discharge coefficient should be used for routing of the PMF inflow. Checks should also be made to determine if any structural modifications that could have produced a change have been made.
- A check should be made to ascertain that a common datum has been used for elevations of reservoir levels and the dam's appurtenances.

8-5.5.3 Operation History and Policy

Data on historical operation should be reviewed for correctness, especially if these data will be required to determine historical inflow floods by reverse-reservoir routing. The location of the reservoir stage recorder should be evaluated to ensure that measured stages are not influenced by drawdown due to spillway or outlet works operation or wind-generated waves.

- If stage records are available for any other location on the reservoir, the records should be compared to detect any inconsistencies, which will also aid in assessing the degree to which the reservoir surface is sloped during passage of extreme floods.

It is necessary to review operation policy and procedures for the passage of extreme floods to develop criteria to be used in routing the inflow PMF.

- If it is possible for operators to be present at the project and to perform the required operations during the PMF, and if redundant operation systems exist, assume that gates and valves that have been tested under head can be operated as proposed during flood passage.
- If gates and valves that would be operated during passage of an extreme flood have not been tested under head to ensure their operation, it will be necessary to make a detailed evaluation of their condition and reliability. Assumptions on the operation of the gates during passage of a PMF should then be made based on that evaluation.
- If the gates are operated remotely, it is necessary to assess the reliability of operation that can be expected during an extreme flood. Operations during historical floods should be reviewed to determine whether the operational policies have been consistently applied.

Spillways equipped with flashboards or stoplogs must be reviewed to determine the operation policy relative to their installation and removal. In addition, if the flashboards are designed to fail or collapse, it will be necessary to obtain detailed information on their structural design. The head at which the flashboards will fail or collapse must be checked.

- If the flashboards are designed to be tripped, the tripping operation should be reviewed to ensure that it can be accomplished at the planned time during passage of an extreme flood.

- If the spillway is sometimes blocked with stoplogs that must be removed manually, it will be necessary to determine if sufficient warning time and the needed equipment would logically be available to allow for removal.

- It is important to consider the possibility that a spillway or outlet works may be at least partially blocked by debris. The degree to which debris has been handled successfully during past major floods should be assessed. If a debris-handling operation plan that has worked successfully in the past is in place, it is acceptable to assume that blockage will be insignificant during passage of the PMF.

SUMMARY

Wednesday 4:00 p.m.

EVALUATIONS

Wednesday 4:30 p.m.

SPECIAL CONSIDERATIONS

(SEC. 8-3, 8-10.2, 8-10.5, 8-11 of FERC Guidelines on the Determination of Probable Maximum Flood)

Materials under TAB 31 in Volume 2 of the Notebook

- **DAM BREAK PARAMETERS**
- **ANTECEDENT HYDRO-METEOROLOGICAL CONDITIONS**
- **CO-INCIDENT HYDRO-METEOROLOGICAL CONDITIONS**
- **START Q AT THE INCEPTION OF PMF**
- **GATE AND FLASHBOARD OPERATIONS**
- **SEDIMENT CONSIDERATIONS**

DAM BREAK PARAMETERS

- Dam Break analysis is required for the development of Emergency Action Plan (EAP) for all FERC licensed projects

- Two scenarios
  - Fair-weather dam break scenario with a reservoir water level at the normal maximum pool

  - PMF dam break scenario with the reservoir water level at the maximum during the passage of the PMF

- National Weather Service DAMBRK model is preferred; this model is now marketed by BOSS and HAESTEAD METHODS with user-friendly interface

- HEC-1 also has a dam break routine; if a short time interval is used, the results are acceptable

- Guidelines for dam breach parameters:

  - Average breach width ($\overline{BR}$)

  - Horizontal component of side slope of breach (Z)

  - Time to failure (TFH)

  are provided in the FERC "Engineering Guidelines for the Evaluation of Hydropower Projects", Appendix IIA - Dambreak Studies

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
	$\overline{BR}$ = Multiple Slabs	Buttress
	$\overline{BR}$ = Width of 1 or more	Masonry, Gravity Monoliths,
	Usually BR $\leq$ 0.5 W	
	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 = O	Masonry, Gravity Timber Crib, Buttress
	$¼ \leq Z \leq 1$	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, Buttress
	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:

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 2-A-12 for definition Sketch

**Comments: See Page 2-A-10 - 2-A-11*

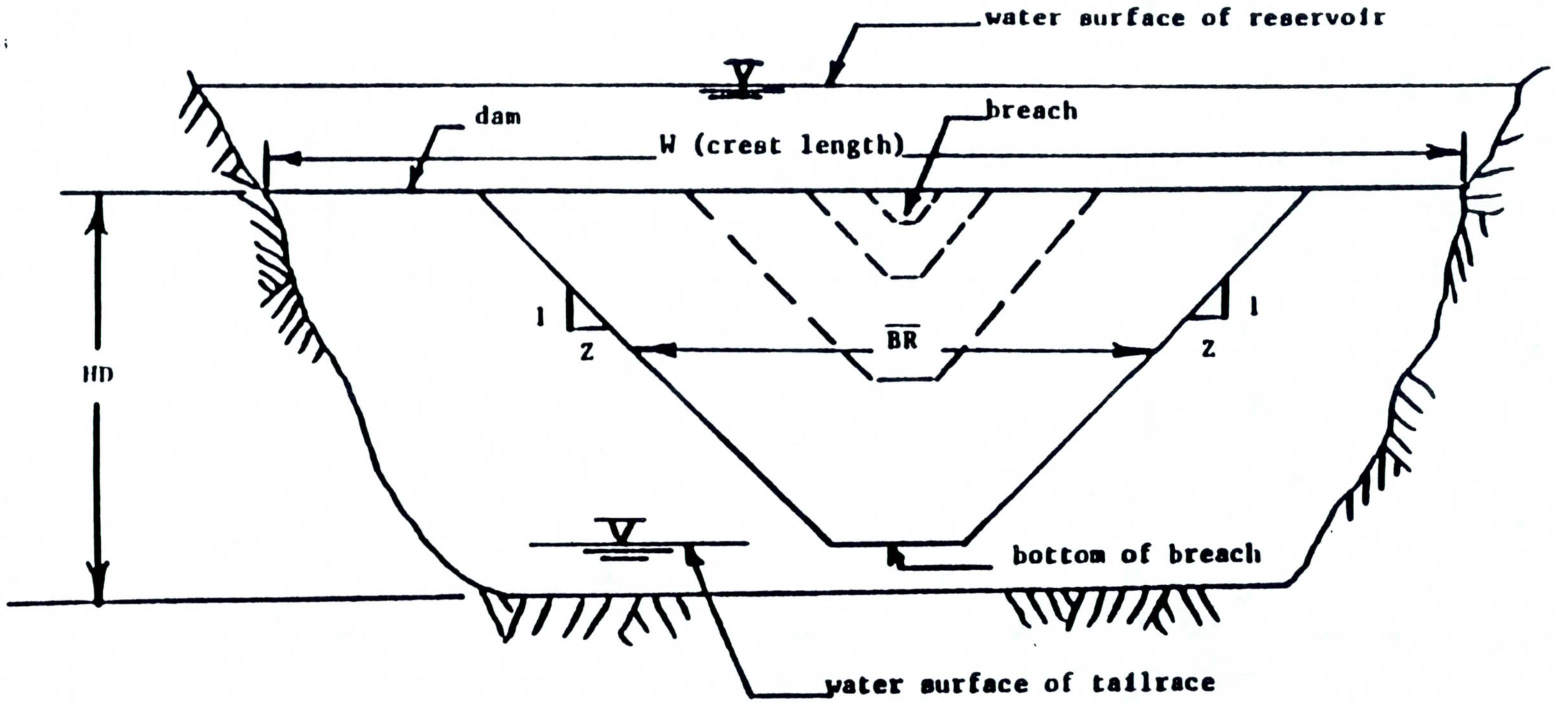


FIGURE 1. DEFINITION SKETCH OF BREACH PARAMETERS

Two Scenerios:

- **Use the low Manning's n value to estimate the time of arrival of the flood wave and high Manning's n value to estimate the depth of flood inundation**

ANTECEDENT HYDRO-METEOROLOGICAL CONDITIONS

- Primarily relates to establishing the reservoir water level in the inception of the PMF

- FERC policy on this topic is presented in Sec 8-3.1 of the Guidelines

  - annual maximum reservoir water level

  - 24-hour 100-year flood antecedent flood condition

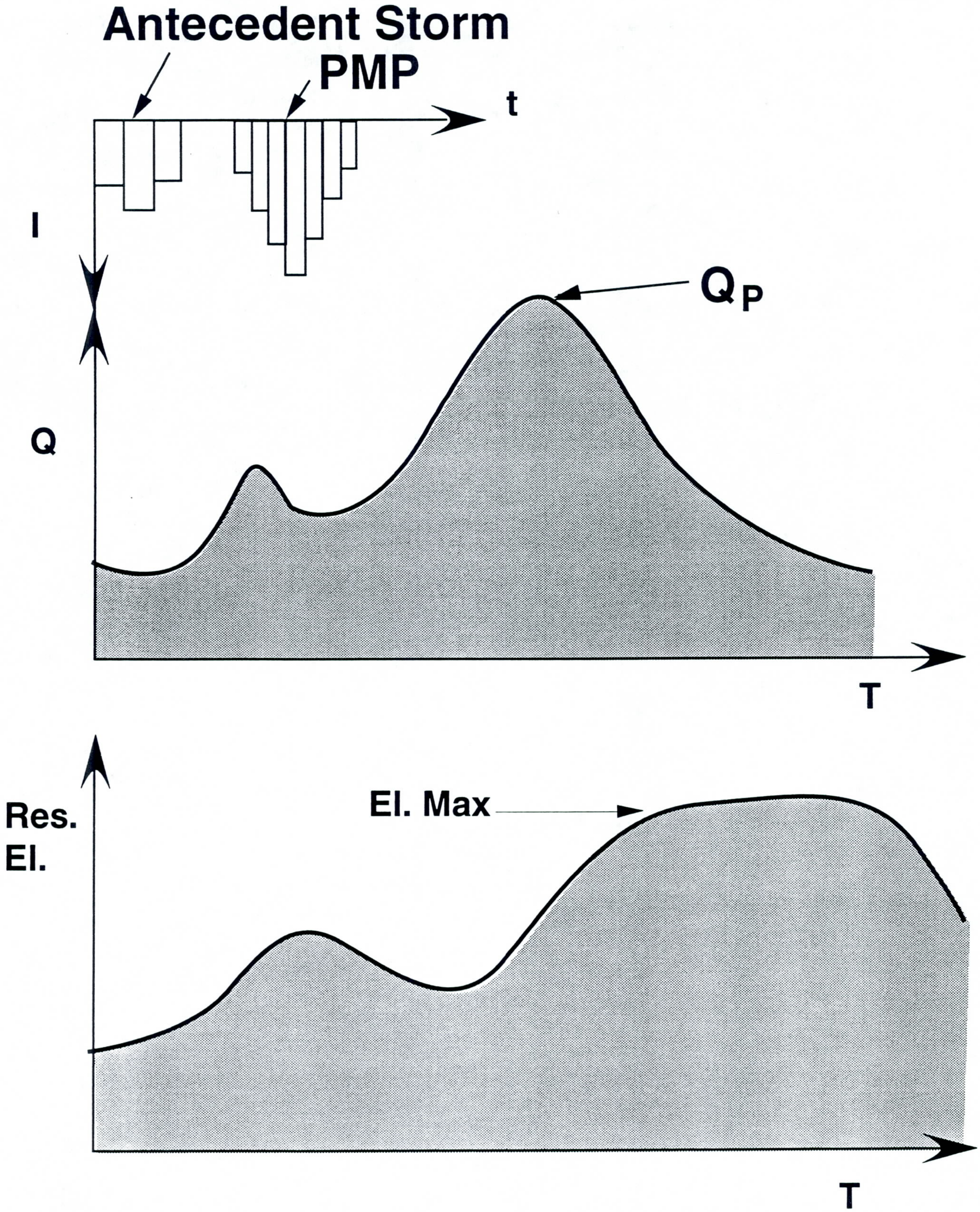
  - wet year rule curve

  - historical analysis of antecedent hydro-meteorological conditions associated with major floods

- Normally not a concern for a run-of-the-river type project. Antecedent flood will generally have receded before the PMF

- Always a concern when dealing with storage dams which fluctuate from year to year

- For most projects with uncontrolled spillway,the starting water level will be at the spillway crest elevation unless the antecedent storm gives a surcharge


Antecedent Storm
PMP
t
I
Q
Q_P
T
Res.
El.
El. Max
T

CO-INCIDENT HYDRO-METEOROLOGICAL CONDITIONS

- In the case when snowmelt is a critical element of the runoff process

- Assume 100-year snowpack covering the watershed areas which generally experience snowfall

- Use the temperature sequence associated with the major snowmelt floods in the region or as given in the Hydromet reports for PMP estimates published by National Weather Service

- Use a 3° F lapsed rate per 1,000 feet altitude change

- Use infiltration rate as given by STATSGO; assume frozen ground conditions unless the soil type is characterized as deep sand (very high infiltration rate) or areas are well-forested with considerable amount of humus accumulation

COE LA Dist:

- Empirical
- San Gabriels Mont.
- applicable to S. Calif.
- event yield basis
- for areas of 0.1 to 200 sq mile
- for area with high ~~rooted~~ proportion of their total area in steep, mountainous terrain
- Runoff > 3 cfs/sq mile. & max 1 hr precip. > 0.3"/hr.
- For areas other than Costal Region Use adjustment/transposition factor
- Precipitation is based on
  - Rainfall/Peak unit runoff Rate
  - Basin physiographic parameters
  - fire history — 10-yr after burn.

STARTING Q AT THE INCEPTION OF PMF

- River flow at the inception of PMF should be set at the average for month during which PMP would occur

TABS:

- 2-d. model.
- very computationally intensive.
- best to use it to predict localize area; such as near intakes.

GATE AND FLASHBOARD OPERATIONS

- Are gates operable?
- Is there back-up power for gate operations?
- Can gates be operated by hand?
- Will operational staff be able to get to the plant during a PMF?
- Proper maintenance and scheduled testing of gates are essentially to successful operations during emergency conditions, such as PMF
- Can the flashboards be tripped during the PMF?

HEC-6 :

- One Dim.
- need sed. char.
- need Run-off hydrograph
- for reservoirs, need a long-term run-off hydrograph; 30 to 50 yrs.
-

SEDIMENT CONSIDERATIONS

- What has happened during the life of the reservoir?

- Has flood storage space in the reservoir been encroached due to sediment depositions in the upper reaches of the reservoir?

- Can sediment affect gate operations?

USBR Area Red. Method
Design of small Dams

* Empirical.

* classify Reservoir sediment dep. char. by its Elev-Storage relationship.

* Deposition Pattern

* Est. total Res. Sed. inflow; total Sed. Rating Curve.

* Use est. empirical Relationship to reduce the res. area

*

DAM BREAK PARAMETERS

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  - Average breach width ($\overline{BR}$)

  - Horizontal component of side slope of breach (Z)

  - Time to failure (TFH)

  are provided in the FERC "Engineering Guidelines for the Evaluation of Hydropower Projects", Appendix IIA - Dambreak Studies

Est of Sed

- USBR – Area Red. Method
- HEC-6 1-dinensional
- TABS 2-dimensional
- COE – LA Dist.

  Los Angles Dist. Method for Prediction of Debris yield from Coastal Southern Calif. Watershed", April 1989 ??