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Volume 5
Hypothetical Floods

March 1975

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13. ABSTRACT *(Maximum 200 words)* This is Volume 5 of the 12 volume report prepared by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers as a contribution to the International Hydrological Decade. This volume describes techniques and procedures for synthesizing hypothetical rainfall-snowmelt-runoff events. Standard Project and Probable Maximum floods simulation are discussed as used by the Corps of Engineers in the United States. Storm centering, use of rainfall frequency data, balanced hydrographs and several computer programs are discussed. Examples and illustrations are included. The following computer programs are included as appendices: 1) Unit Hydrograph and Hydrograph Computation; 2) Hydrograph Combining and Routing; 3) Balanced Hydrograph; and, 4) Streamflow Routing Optimization.			
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FOREWORD

This volume is part of the 12-volume report entitled "Hydrologic Engineering Methods for Water Resources Development", prepared by the Hydrologic Engineering Center as part of the U. S. Army Corps of Engineers' participation in the International Hydrological Decade. Volume 5 describes methods and procedures for computing hypothetical floods from rainfall and snowmelt for various purposes in flood control studies. Although many of the methods and procedures described herein have been used successfully in Corps of Engineers studies, the volume should not be construed to represent official policy or criteria of the Corps.

This volume was written by Leo R. Beard, Technical Director, Center for Research and Water Resources, University of Texas. Mr. Beard was formerly the director of the Hydrologic Engineering Center. Much of the material contained in this volume has been extracted from Corps manuals and other publications. Review and ammendments have been made by various members of the HEC staff, with final editing assigned to Dale R. Burnett.

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Chapter 1

Nature of Hypothetical Floods

CHAPTER 1. NATURE OF HYPOTHETICAL FLOODS

Section 1.01. Definition

Hypothetical floods consist of hydrographs of artificial flood flows at one or more locations that can be used as a basis for flood-control planning, design and operation decisions or evaluations. These artificial floods represent classes of floods of a specified or implied range of severity. They can be used as a direct basis of design or as representative flood magnitudes for use in evaluating the over-all effectiveness of a project or system of projects in preventing flood damages at one or more locations at or downstream of projects.

Hypothetical floods are ordinarily derived from rainfall or snowmelt or both, with ground conditions that are appropriate to the objectives of the study, but they can also be derived from runoff data alone, usually on the basis of runoff volume and peak-flow frequency studies and representative time sequences of runoff.

Section 1.02. Need

Hypothetical floods are needed in order to analyze the effect of floods of a specific magnitude and sequence where gaged historical data are inadequate to define the event being considered. In addition, hypothetical floods are needed for assessing the effects of changes in river basin conditions (including project construction, urban development, deforestation, etc.) on downstream flood potential.

Large hypothetical floods that represent relatively extreme conditions (such as the standard project or probable maximum floods described in Chapters 3 and 4) are needed as a basis of design of projects whose inadequacy would result in major property damage or loss of life. Such extreme hypothetical floods can also be used as reference levels for design floods that are smaller, thus permitting their indirect use in a set of standards and guides for design of projects where potential property damage and loss of life is not sufficient to warrant design for the full flood magnitude.

The need for probable maximum floods that represent the upper feasible limit of anticipated flood magnitudes is of particular importance. These are used to design spillways of major dams where failure would cause a disaster far greater than would be caused by natural flooding. Next in importance is the need for standard project floods that are representative of maximum events that occur within entire regions. These events rarely occur on individual drainage areas, but their occurrence within a region demonstrates that they are a definite threat at any specific location. Lastly, hypothetical floods that have expected probabilities of occurrence at specific locations are needed for economic and other planning and design purposes. This last category of hypothetical floods must be representative of large ranges of magnitude and of time and area distributions so that evaluations of project effects can be made by simulating project operation using a minimum number of hypothetical floods.

Section 1.03. Application

Hypothetical floods are used primarily to simulate the operation of a project or system of projects in order to verify that facilities and operation rules are adequate.

In the case of simple limited channel improvement, floods of all magnitudes are usually reduced in stage, and it is simple to compute the average annual damage with and without the improvement in order to determine project feasibility. In the case of extensive channel improvements that might serve to accelerate flows in a significant portion of a river system, the accelerating effect must be simulated for a large range of flood magnitudes in order to determine the extent to which flow magnitudes downstream are augmented.

In the case of levee protection, the frequency and severity of sudden flooding from levee failures and the increased flooding of lands immediately upstream of a levee system must be evaluated, and the consequences must be compared with the reduced flooding due to levee protection against floods of design magnitude and smaller. As in the case of extensive channel improvements, the accelerating effects of extensive levee improvements can be substantial and should be evaluated for a large range of flood magnitudes.

In the case of reservoir projects, it is usually necessary to simulate the effects of each reservoir on downstream flows for all relevant magnitudes of peaks and volumes of inflows. Here it is particularly important that each hypothetical flood has a peak flow and volumes for all pertinent durations that are commensurate in severity, so that each computed regulated flow will have a probability or frequency that is comparable to that of the corresponding unregulated flow. This balance between peak flow and volumes is of special importance when studies are made to select alternative reservoir or channel improvement projects.

Where a number of reservoirs operate in a system or where one or more reservoirs affect damages at remote downstream locations, it is important that a proper balance is attained among inflows to the various reservoirs and with runoff downstream of the reservoirs so that regulatory effects can be accurately evaluated and so that the correct weight will be assigned to each reservoir when determining its proportional effect in reducing downstream flooding.

Chapter 2

General Procedures

CHAPTER 2. GENERAL PROCEDURES

Section 2.01. Storm magnitude

Most hypothetical floods are computed from storm rainfall, the severity of which is a function of the depth-area-duration relationship and the time sequence and areal distribution of rainfall conforming to that relationship. An illustrative depth-area-duration relationship is shown in figure 2.01. Because all storms are more severe for some area sizes and durations than for others, it is virtually impossible to assign an exceedence probability to any particular storm. Probability estimates are confined to total precipitation amounts for specified durations and areas, and such estimated probabilities do not necessarily apply to resulting runoff, since the time and area distributions of rainfall and the ground conditions will also affect runoff severity.

Section 2.02. Storm types

There is some merit in considering general storms separately from local storms. Precipitation in both types is caused by lifting of moist air to higher levels, causing the air to cool through expansion and thus to reduce its capacity to retain water vapor.

General storms are associated with tropical or extra-tropical cyclones, the latter generally associated with distinct frontal systems. Precipitation in general storms is caused by convergence of air over extensive areas through cyclonic or orographic action. Usually large general storms are accompanied by high winds that persist for relatively long durations over large areas.

Local storms are those that result from individual convection cells, often accompanied by thunder and lightning. The horizontal extent of the cells is limited by the height of vertical development. In the most

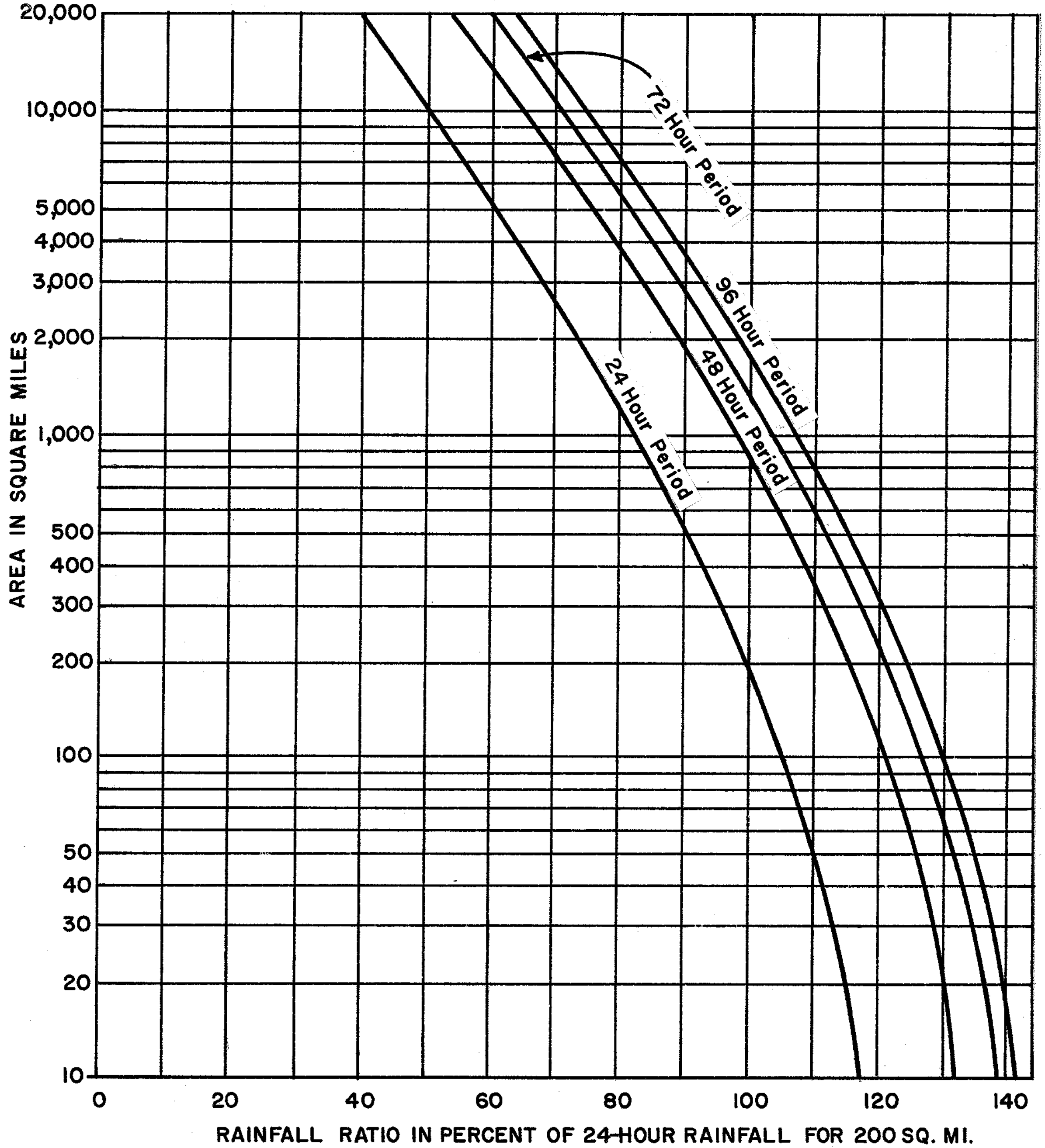


Figure 2.01 Typical depth - area - duration curves

severe cases, the single convective cell can extend from near sea level to the tropopause, a vertical distance of 8 to 14 kilometers, depending on latitude and season. The horizontal diameter of such cells is limited to 2 or 3 times the vertical dimension, so it can be seen that the areal extent of local-storm cells ranges from a few square kilometers up to about 1,000 square kilometers. Rapid overturning of the air mass accompanied by a strong convergence of the falling rain can result in extremely high intensities of rainfall over an area much smaller than the total extent of the convective cell. When the air mass is completely overturned, it becomes relatively stable, and precipitation essentially ends. This usually takes 2 or 3 hours.

Both general and local storms move during the course of their life. Thus, the pattern of storm-total precipitation does not necessarily reflect the patterns of precipitation at various instants during the storm's life time. Also, convective activity is often associated with general-storm activity, and thus the irregularity of the space-time patterns of general storms can be extreme. However, heavy precipitation over large areas for sustained durations requires high winds. The high wind shear accompanying high winds usually effectively inhibits the vertical development of convective cells. Thus, precipitation is somewhat uniform in time and area during large cyclonic and orographic storms (except for topographic effects discussed below), although convective precipitation often follows in the wake of a general storm.

These generalizations are based on experience in the northern hemisphere and particularly in the United States, but are believed to apply generally throughout the world.

Section 2.03. Geographic variations of storm potential

Storm potential at any specific location is a function of (a) the distance from sources of moisture and the characteristics of intervening terrain and (b) location in relation to the prevailing paths of storm centers. Extratropical cyclones occur in the zone of prevailing westerly winds, generally between 30 and 60 degrees of latitude and derive most of their energy from temperature differences in the vicinity of the polar front that separates polar and tropical air masses. Tropical cyclones (hurricanes) occur in the zone of prevailing easterly winds, generally between 10 and 20 degrees of latitude, and derive most of their energy from moisture evaporation in tropical seas. They usually terminate when they travel over land for any substantial distance, and, of course, they frequently move into the zone of prevailing westerlies, where they usually dissipate after a few days of intense activity.

The severity of storm activity, in the case of extra-tropical cyclones can be great over the entire zone of prevailing westerlies, but the amount of storm precipitation depends on the amount of moisture available. This is usually greater nearer the equator and nearer the oceanic sources of moisture in the warm air mass. Maximum moisture over the ocean is that corresponding to the moist adiabat that intersects sea-level pressure at ocean-surface temperatures, as indicated on an adiabatic diagram. If the moisture must travel great overland distances, it is subject to substantial depletion through precipitation on the way. If mountain barriers intervene, only moisture that can be retained when the air masses are lifted over the mountains can be carried beyond them.

In the case of tropical cyclones, storm potential is greatest along the coasts in the general paths of the cyclones and as much as 1,000 kilometers or more inland. Again, rainfall amounts are diminished substantially as the distance from the ocean increases. Since tropical hurricanes originate in the zones of prevailing easterly winds, associated rainfall potential is greatest along the east coasts and coasts toward the equator in latitude zones below 45 degrees.

In the case of convective rainfall, amounts are greatest where precipitable moisture in the air masses is greatest. They occur under conditions where air warms at the surface or cools aloft and where conditionally unstable air is lifted by orographic barriers, by cold fronts, by convergence in low-pressure areas and by vertical divergence where wind speeds are reduced as air enters land areas from the ocean.

Section 2.04 Topographic effects

Any orographic barrier that causes air to be lifted or deflected upward tends to initiate convective action and consequent precipitation. Thus island and mountain peaks often are surrounded by clouds and rain. As the air masses pass such a barrier, moisture precipitates on the windward side of the barrier and as it subsides and warms on the leeward slope the remaining air mass becomes dry. Consequently, where wind patterns are steady, the windward side of mountains received precipitation, and the leeward side is desert.

Since most of the air's moisture is in the lowest 2,000 meters, heaviest precipitation usually occurs below 3,000 meters elevation. Moisture condensed at lower elevations can be carried to elevations of 3,000 meters by high winds. Because of irregularities in mountain surfaces, local wind patterns develop that can depend on wind direction or that can be rather uniform from time to time. In either event, there is a tendency for large variations in precipitation from place to place within mountainous regions. This pattern cannot easily be related to topographic features because of the extreme complexity of wind and moisture condensation interactions. Nevertheless, there is a general pattern of higher precipitation with higher ground elevations and with the degree of windward exposure of the ground surface.

Section 2.05. Storm transposition

The fact that meteorological conditions vary only gradually within many large regions of the world suggests that recorded storms could have occurred with about equal severity at any of many locations in the region where they actually occurred. Thus, if a large flood occurs on one river in a region, it is possible to estimate floods that could have resulted on other streams in that region assuming the storm was centered over the drainage area tributary to any one of the other streams. This is done by transposing the isohyetal pattern of total storm rainfall to the drainage area in such a position that maximum rainfall will occur over the drainage area. Historical events having a large areal coverage usually have several centers of high rainfall. When transposing this type of storm to another river basin, it usually will require several trials before the most critical centering can be determined. Figure 2.02 illustrates the centering of an isohyetal pattern over the drainage basin below a flood control reservoir and upstream of a local protection project. This pattern is an hypothetical areal pattern but is reasonably typical of severe historical occurrences to be valid. The centering illustrated is centered to maximize the storm rainfall-runoff most adversely for the local protection project. If the major concern were the flood control storage requirement at the reservoir, the storm could be recentered so as to result in maximizing rainfall and runoff upstream of the dam. Either centering is equally valid in most instances, and both centerings would normally be investigated for purposes of sizing both the reservoir and the local protection works.

As storms occur, they usually do not center over any particular river basin. Consequently, even where maximum flood flows occur, some transposition of the storm would ordinarily increase those flows. There is some question as to whether the orientation of isohyets, or their general shape, can reasonably be changed, because meteorological factors that result in major storm precipitation might be associated closely with the isohyetal patterns. Each storm must be considered individually to evaluate the extent it can be moved and rotated.

If storms are transposed over long distances, the rainfall amounts should be modified in accordance with the maximum precipitable moisture variations that are characteristic of the respective large regions. If

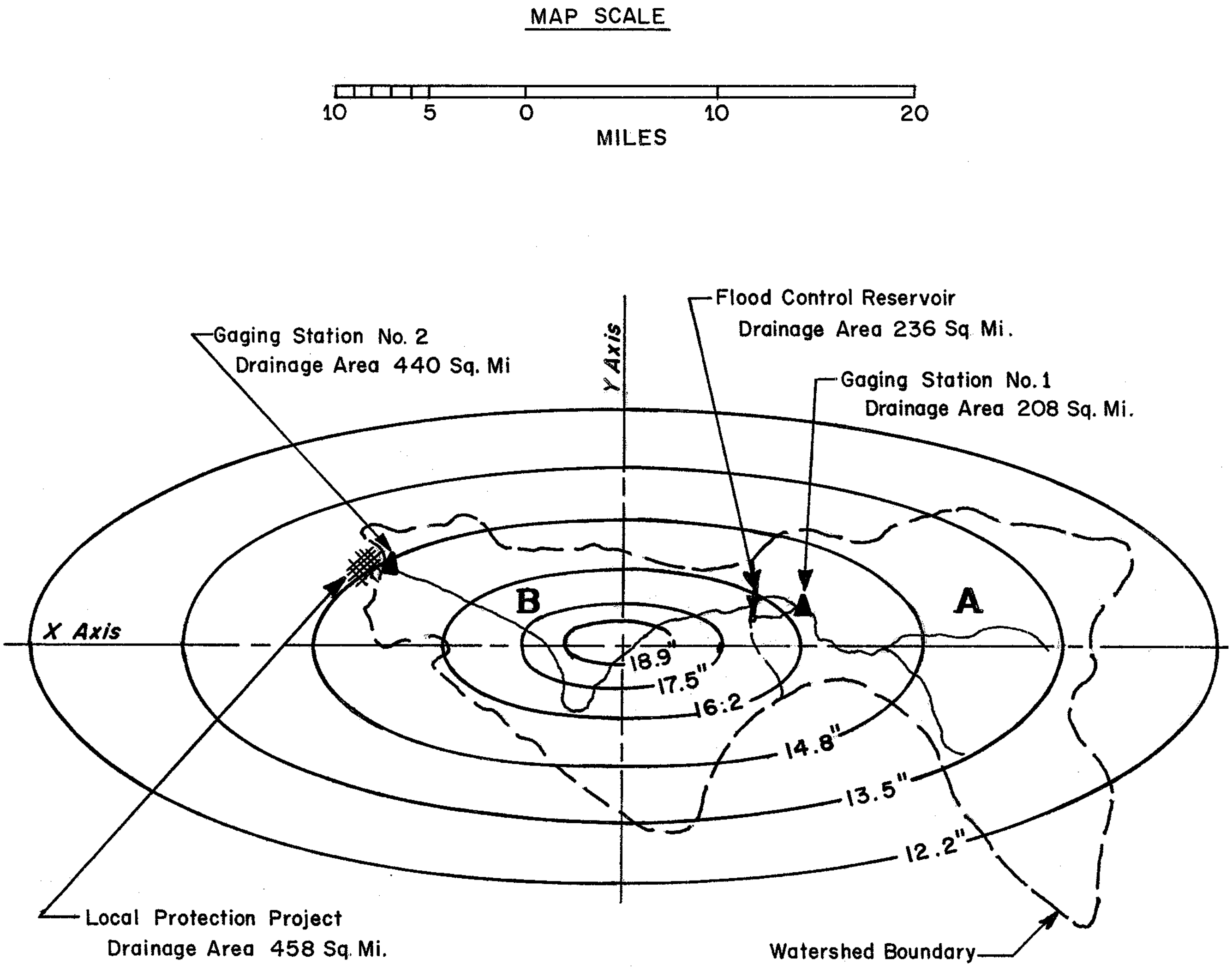


Figure 2.02 Illustrative storm transposition

storms are transposed in mountainous areas, account must be taken of the intricate effects of irregular topography, barrier orientation, prevailing wind direction and relative storm occurrence. Usually a base pattern of normal orographic precipitation can be constructed from average seasonal precipitation amounts computed from all available data, in conjunction with correlation studies relating such average precipitation to orographic features. Then lines of storm precipitation expressed as a ratio to the base-pattern amount can be transposed over a much wider areal range than if expressed as direct rainfall depth. Such normalization does not entirely remove orographic related patterns but is generally acceptable when combined with other adjustment techniques.

It is not ordinarily possible to express in terms of probability or frequency the flood that would result from a transposed storm. It does not even necessarily follow that floods in different areas resulting from transposition of the same storm to those areas would be equally severe (equally probable), since an actual storm is not necessarily balanced (equally severe) with respect to different areas and time patterns that different river basins respond to most strongly. In the chapters below, transposition of a balanced storm will be used in order to approach equal flood severity as nearly as possible, but actual probability evaluations will be based on observed flood frequencies wherever adequate runoff data are available. In the case of very rare events, some judgment or inference of extreme probabilities might be made for standard project and probable maximum floods independent of recorded runoff at the location.

Section 2.06. Rainfall frequency

The frequency or probability of occurrence of maximum rainfall amounts for a specified location and duration can be estimated from recorded rainfall at that location using methods described in Volume 2. Point rainfall estimates thus obtained can be used to construct maps of precipitation

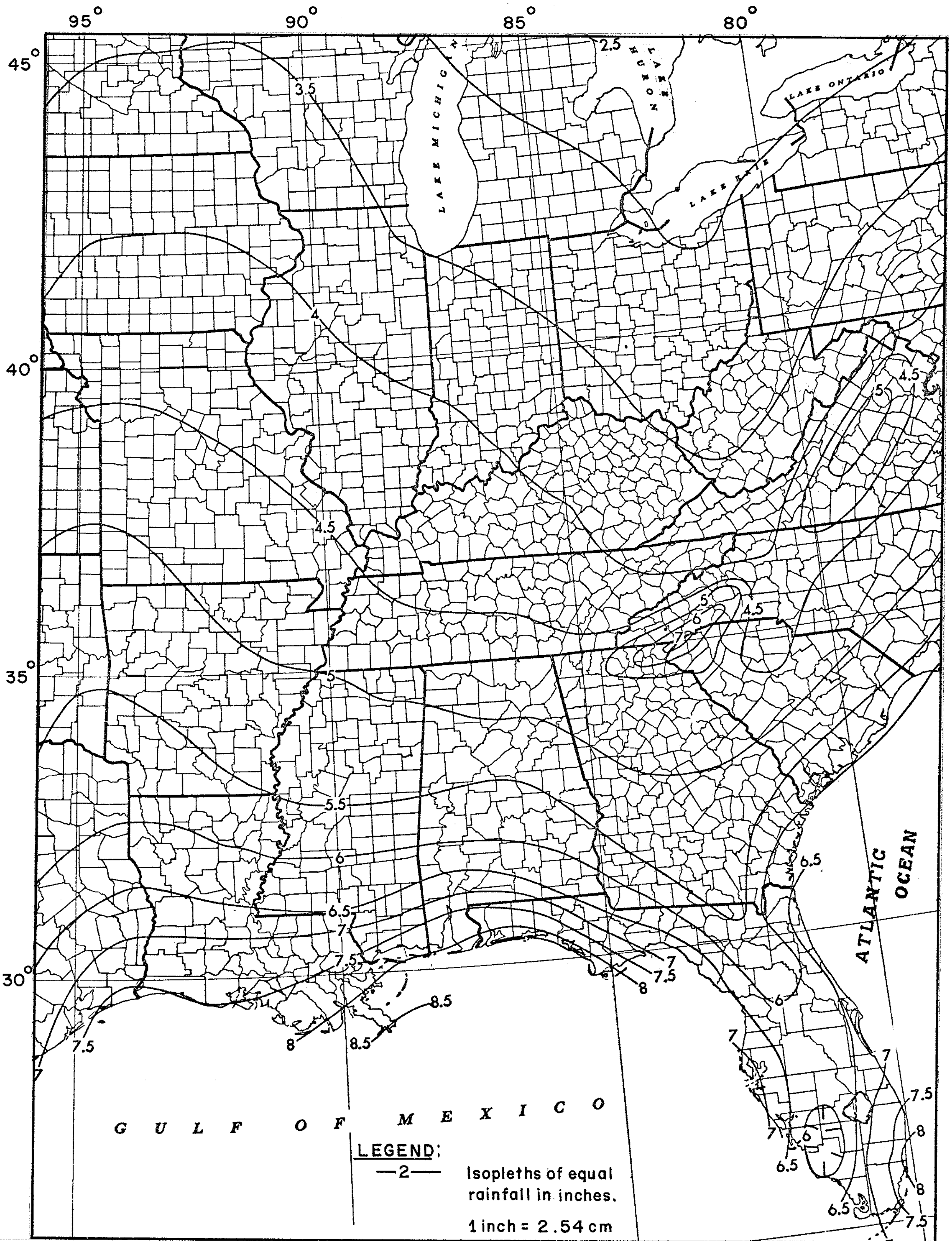


Figure 2.03 Point-Rainfall Amounts For Specified Frequency

24-Hour Amounts Exceeded Once in 5 Years

Eastern United States

(Taken from U.S. National Weather Service, Technical Paper No. 40) amounts corresponding to specified frequencies and durations. Such maps have been prepared by the U.S. National Weather Service for the United States, and a portion of one of those maps is shown in figure 2.03.

Such point-rainfall estimates are often converted to area-rainfall estimates by means of general relationships developed between point rainfall and area rainfall. The proper method to do this is to perform frequency studies of average rainfall recorded simultaneously at a number of stations and to compare point-rainfall amounts with area-rainfall amounts for various specified frequencies. This should be done for various station groupings and frequencies. In each case, the area size represented by the stations whose amounts are averaged should correspond to an area circumscribing the stations a distance beyond outlying stations equal to about half of the average distance between stations in the group. Relationships developed for this purpose are illustrated in figure 2.04.

Depth-area-duration diagrams representing precipitation for a specified frequency at a river basin can be determined by averaging map values of point precipitation over the river basin for each of various pertinent durations. Relationships similar to those illustrated in figure 2.04 can then be applied to these point values in order to develop the full depth-area-duration relationship. This relationship can be illustrated as shown in figure 2.01, but the frequency relationship will be different quantitatively from the relationship for a single storm, particularly in that amounts will not decrease as rapidly with increasing area size.

Section 2.07. Snowpack and snowmelt

In regions where snowmelt is a major contribution to flood runoff, snow accumulates on the ground during the winter season and melts during the spring. The primary parameter of snowpack of interest from a snow-

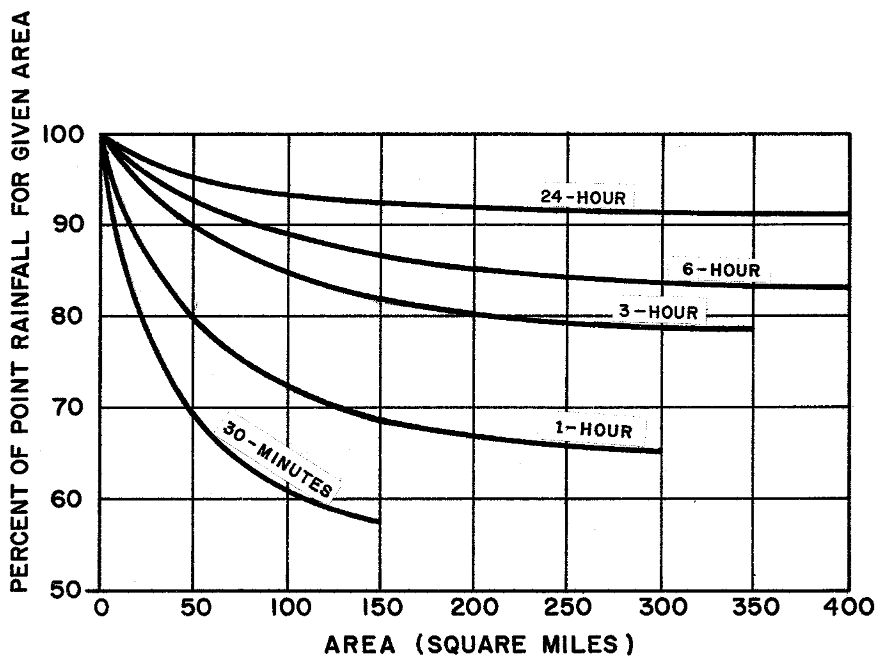


Figure 2.04 Relation of Area Average Rainfall to Point Rainfall

(Taken from U. S. National Weather Service, Technical Paper No. 40) melt flood standpoint is the maximum accumulation before the start of the snowmelt season. Maximum accumulation will not ordinarily occur at all points simultaneously, as melt is greater in lower latitudes, lower elevations and on the sunny slopes of the ground, but measurements at a number of points in a basin can be used to estimate the maximum simultaneous accumulation for the basin.

Frequency studies of maximum water equivalent of snowpack at a point or for a basin can be constructed in the same manner as are frequency curves of rainfall or runoff (as described in Volume 2). These can be used to determine the snowpack corresponding to any specified frequency and to make some estimate of maximum snowpack potential. Maximum snowpack potential is ordinarily not determined by integrating maximum snowfall during a winter season, because of the great uncertainties in estimating the frequency and sizes of storms that can occur and the losses that occur through evaporation and intermittent melt during the accumulation season.

Daily snowmelt can be estimated by use of a simple relationship to daily temperatures as follows:

$$M = C_1 (T - T_0) \tag{1}$$

in which:

M = Daily melt in millimeters

C_1 = Calibration constant

T = Daily average (or daily maximum) temperature in degrees centigrade

T_0 = Base temperature in degrees centigrade (usually freezing temperature, 0° C).

In most river basins, such a simple relationship is most satisfactory, because detailed information on other melt factors is usually inadequate for successfully using more elaborate relationships.

Where more detailed information is available, a number of energy-budget relationships can be used. Particularly useful are the following two simplified relationships:

$$M = 2.3C_2 + C_2 (1.33 + .51W + .013R) (T - T_0) \qquad (2)$$

$$M = 0.5C_3S(1 - A) + C_3(.11W + 6.6) (T - T_0) + .40W(D-T_0) \qquad (3)$$

in which:

C_2 = Calibration constant

C_3 = Calibration constant

W = Wind speed in meters per second 15 meters above the snow surface

R = Rainfall millimeters

S = Solar radiation in langleys

D = Dewpoint in degrees centigrade

Precipitation that occurs during the snowmelt season occurs as snowfall and is added to the snowpack when and where temperatures are below 1 or 2 degrees centigrade. Otherwise it occurs as rain and is absorbed by the snowpack until saturation and melting temperatures are reached and the excess is added to the snowmelt for that computation interval.

Temperature sequences that might be used in the computation of hypothetical snowmelt floods are very difficult to derive, because critical conditions are not usually related to maximum temperatures throughout the melt season. Often the most critical conditions are those where early temperatures remain low in order to preserve the snowpack until the season of higher temperature potential. Maximum temperatures for various durations at any particular time of year can be derived through frequency studies of temperatures recorded at that time each year. These temperatures are a function of elevation. Critical temperature sequences should not exceed maximum selected amounts for any specified time of year, and can be derived through a process of successive approximations, computing flood runoff from various trial sequences, using equation 1.

Selection of critical sequences of melt factors in equations 2 and 3 is much more complex than the selection of temperature sequences alone, since variations of the different parameters must be consistent. The process is largely a matter of judgment and of successive approximation of resulting runoff severity.

Complete snowpack and snowmelt computation techniques described above are contained in the computer programs described in Appendix 1.

Section 2.08. Ground conditions

Different wetness of the ground and different vegetal cover can cause large differences in runoff from storm rainfall and snowmelt. In very porous soils particularly, loss rates can vary greatly with ground wetness. In cold regions, freezing of the ground can reduce substantial losses to practically zero. Since a major portion of the rainfall in most storms is commonly lost to interception and infiltration into the ground, it is apparent that selection of ground conditions is a major consideration in deriving hypothetical floods.

For various reasons, loss rates in natural drainage basins average much less than rates observed in controlled laboratory experiments with similar soils--sometimes an order of magnitude smaller. Consequently, loss rates used in the computation of hypothetical floods should be derived from data on a watershed similar to the one under study. Rates in nature range from less than 1 to more than 10 millimeters per hour when expressed as average for an entire basin.

Loss rates can be related to ground conditions where data on rainfall and runoff are available for a large number of floods. Usually an index of ground conditions such as total antecedent precipitation is used, although soil moisture indexes would be useful where such measurements are available.

Selection of ground conditions for the various types of hypothetical floods is discussed in the appropriate chapters below.

Section 2.09. Runoff computations

Runoff is computed at regular intervals that are short enough to define the shape of the flood hydrograph adequately. These intervals can range from a few minutes in very small areas to a full day in very large areas. A good guide is generally that the tabulation interval should be about 1/5 to 1/3 of the time of concentration or of the unit-hydrograph lag for the drainage area.

For each computation interval, rainfall that occurs is added to snowmelt. Infiltration (and any other) loss is then subtracted. The remaining quantity, designated as rainfall and snowmelt excess, is translated in time and transformed for storage detention effects by various means. Most commonly, the unit hydrograph method, described in Volume 4, is used. Computation of rainfall and snowmelt losses and runoff in accordance with these principles can be accomplished using the computer program described in Appendix 1.

It is often necessary to divide a drainage basin into sub-basins in order to compute runoff from units where the areal patterns of precipitation and other factors are relatively constant for different flood events. Also, design considerations may require subdivision of a drainage basin in order to obtain flood estimates for various specific locations. When this is necessary, runoff computed for each sub-basin is routed through downstream channels and reservoirs, and combined with other hydrographs at downstream locations of interest. Stream system computation and routing techniques are discussed in the following sections.

Section 2.10. Routing and combining operations

Computation of flood hydrographs from specified storm rainfall and snowmelt for each sub-basin is accomplished first for the most upstream areas in each tributary, and each succeeding sub-area runoff computation is made as the routing and combining operation proceeds downstream. This is a straightforward computation for cases where the storm rainfall quantities do not vary with size of drainage area. In cases where lower precipitation amounts are used for larger areas, some means of recomputing upstream contributions for each successive downstream location must be used. One technique for doing this is described in Chapter 7.

Routing computations in each stream reach between points where determinations are required or at reservoirs must use a computation interval equal to that used in subsequent sub-basin runoff computations so that hydrographs can be combined in a digital computation (or subsequent computation intervals can be an exact multiple of earlier computation intervals).

Routing and combining operations for a stream system can be accomplished using the computer program described in Appendix 2.

Section 2.11. River routing of floods

Routing of floods through river reaches to account for the time of travel and storage effects can be accomplished by a number of methods. A few that are widely used in project studies are described in the following paragraphs.

The Muskingum or coefficient method of flood routing uses the following function of the rate of change of inflow during the computation interval and difference between inflow and outflow at the start of the computation interval to determine the rate of change of outflow during the computation interval:

$$O_2 - O_1 = C_1 (I_1 - O_1) + C_2 (I_2 - I_1) \tag{4}$$

in which:

O_1 = outflow at start of interval

O_2 = outflow at end of interval

I_1 = inflow at start of interval

I_2 = inflow at end of interval

C_1 = routing coefficient

C_2 = routing coefficient

Coefficients in the above equation are expressed in terms of the Muskingum coefficients, K and X as follows:

$$C_1 = 2\Delta t / (2K - KX + \Delta t) \tag{5}$$

$$C_2 = (\Delta t - 2KX) / (2K - KX + \Delta t) \tag{6}$$

It can be seen that K must be expressed in the same time units as the computation interval, Δt. Values of K and X should be derived by successive approximations to reconstitute recorded flows of the downstream hydrograph where data at 2 points on a stream are available. The value of K should approximately equal the computation interval, and this can be accomplished by dividing a reach into sub-reaches if necessary. Routing for each successive sub-reach is then accomplished by applying equation 4 during each successive computation interval (Δt), assuming that outflow equals inflow at the start of the computation for each sub-reach. The theory for this routing method is explained in the U.S. Army Corps of Engineers manual, EM 1110-2-1408, "Routing of Floods Through River Channels," 1 March 1960.

The working storage and discharge method of river routing is based on the following equations:

$$R_2 = R_1 + (I_1 + I_2) / 2 - D_1 \Delta t \qquad (7)$$

$$D_2 = f\ (R_2) \qquad (8)$$

$$O_2 = D_2 - X\ (I_2 - D_2) / (1 - X) \qquad (9)$$

in which:

R_1 = working storage index at start of interval

R_2 = working storage index at end of interval

D_1 = working discharge index at start of interval

D_2 = working discharge index at end of interval

X = same as in the Muskingum routing method

A curve of D vs R can be established from the following equation:

$$R = D\ (K - KX + .5\Delta t) \qquad (10)$$

in which K is the variable used in the Muskingum method but need not be constant for different discharges. Routing procedure consists of applying equations 7 to 9 for each computation interval, usually assuming that outflow equals inflow at the start of the computation.

The straddle-stagger method of flood routing is an artificial and empirical technique that might be satisfactory in many applications and is very simple in concept. This technique consists of averaging a fixed number of successive inflows to obtain the outflow that would occur some time after the mid-time of the inflows averaged. Straddle refers to the number of inflows to be averaged. Stagger refers to the number of ordinates of time delay between the mid-time of the inflows averaged and the time that such average becomes the outflow. This method is also referred to as the progressive average-lag method. In order that timing at the downstream point coincides with timing at the upstream point, stagger should be an integer if the straddle is an odd number. If straddle is an even number, the stagger should be an odd multiple of .5 time intervals.

For example, if straddle is 5 computation intervals, stagger can be an integer of 2 or larger such as 2, 3, 4, etc. If straddle is 4, stagger should have a value such as .5, 1.5, 2.5, etc.

The Tatum method, also known as the successive average lag method, consists simply of a number of successive straddle-stagger routings where straddle is 2 and stagger is .5. The number of successive routings is usually taken as twice the time of travel through the reach divided by the computation interval.

The multiple storage routing consists of a succession of reservoir-type storage routings (where outflow is a direct and unique function of storage) using a time-of-storage factor as an index of the linear storage-outflow relationship. The equation used is:

$$O_2 = \Delta t \, \frac{(I_1 + I_2) / 2 - O_1}{T \, O_1^{-0.2} + \Delta t / 2} \tag{11}$$

in which:

T = time of storage in the same time units as Δt.

The exponent in the denominator may vary somewhat from -0.2, but this value usually gives satisfactory results if sufficient data in a stream system are not available for empirical determination of this exponent. The routing computation is accomplished by successively computing the outflow at the end of each computation interval using equation 11, assuming that outflow equals inflow at the start of the computation. The time of storage, T, is derived empirically through reconstitution of recorded flood hydrographs for the stream. Usually 4 or 5 successive storage computations are used per reach.

Routing techniques described in this section are contained in computer programs described in Appendix 2 of this volume and Appendix 1 of Volume 1. Automatic derivation of routing coefficients from recorded upstream and downstream data can be accomplished using the computer program described in Appendix 4 of this Volume or Appendix 1 of Volume 1.

Section 2.12. Reservoir routing

Reservoir routings are made to determine the outflow that would result from a specified hypothetical flood hydrograph, with a specified reservoir stage at the start and specified operation rules. Hypothetical flood computations described in this report are restricted to those where operation rules are based on conditions at the reservoir and not on conditions at remote locations downstream. Thus, outflow is a unique function of reservoir storage in these circumstances. Routing of hypothetical floods through the reservoir and spillway is needed to determine maximum pool elevation during the spillway design flood. Freeboard requirements are added to establish the elevation of top of dam. The spillway design flood may be based on the probable maximum flood, standard project flood or a flood of lesser magnitude depending on the resulant loss in event of failure from overtopping.

The most simple and satisfactory technique for performing a reservoir routing is a special form of the Modified Puls method. In order to perform this routing, a curve or table of storage indication versus outflow is first constructed from stage-storage-outflow data. Storage indication is the sum of storage and one-half of the corresponding outflow, when the storage is expressed in volume units equal to one unit of outflow lasting for one computation interval. When these units are used, inflow and outflow can be directly added to or subtracted from storage without converting units each time. The routing equation is based only on the law of continuity and the assumption that average outflow for a computation interval is equal to the average of instantaneous outflow quantities at the start and end of that interval. The equation is derived as follows:

$$\frac{S_2 - S_1}{\Delta t} = I_{12} - \frac{O_1}{2} - \frac{O_2}{2} \tag{12}$$

$$\left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) = \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) + I_{12} - O_1 \tag{13}$$

in which:

S_2 = storage at end of interval expressed in flow units for the computation interval

S_1 = storage at start of interval expressed in same units

I_{12}= average inflow for interval

O_1 = outflow at start of interval

O_2 = outflow at end of interval

Quantities in parenthesis in equation 13 are storage indication at the end and start of the interval. The routing is started by establishing an initial value of storage indication. If outflow equals inflow at the start, an initial storage indication value is obtained from the storage indication vs. outflow relation. If storage at the start is known, storage indication is computed by converting units and adding one-half of the corresponding outflow.

When storage indication at the start of each interval is known, it can be computed for the end of that interval (the start of the next interval) using equation 13 by simply subtracting the outflow at the start of the interval and adding average inflow for the interval. Outflow at the start of each interval is obtained from the storage indication using the relationship established as discussed in the preceding paragraph.

Reservoir routings by this method are performed in the computer programs described in Appendix 2 of this volume and Appendix 1 of Volume 1.

Chapter 3

Standard Project Floods

CHAPTER 3. STANDARD PROJECT FLOODS

Section 3.01. Definition

The standard project flood represents the flood that can be expected from the most severe combination of meteorologic and hydrologic conditions that are considered reasonably characteristic of the geographic region involved, excluding extremely rare combinations. It is usually computed by examining all of the major storms that have occurred in the region and selecting a storm magnitude and pattern that is as severe as any of the transposed storms, with the possible exception of any storm or storms that are exceptionally larger than others and are considered to be extremely rare events. In the case of snowmelt floods, it is ordinarily the flood that would result from the most reasonably severe combination of snowpack, rainfall and snowmelt factors. Ground conditions in either type of flood should be reasonably conducive to high runoff, but not necessarily the most extreme observed. Standard project flood estimated completed to date in the United States indicate that SPF discharge on detailed studies usually are about 40 to 60 percent of the probable maximum flood for the same basin. (See Chapter 4.) The standard project flood is intended as a practicable expression of the degree of protection that should be sought as a general rule in the design of flood control works for communities where protection of human life and unusually high-valued property is involved. The SPF procedure is used in lieu of the discharge-frequency approach because of the unreliability inherent in estimating large magnitude infrequent events from short record, or even regional, discharge frequency analyses.

Section 3.02. Storm severity

Storm rainfall severity is not simply a function of the average total rainfall for a drainage basin, but is a function of the area and time distribution within the storm. In general, the greater the degree of concentration of rainfall in time and space, the greater is the severity as measured by its flood-producing capability. In the case of the space distribution, however, there are some areas within a basin that are more conducive to flood production than others, and it is usual practice to center the heavier storm rainfall over these areas. Locations of such storm centers may be different from computation of flood flows at different points in the basin.

Storm rainfall severity is usually represented by depth-area-duration relationships such as illustrated in figure 2.01. In the case of the standard project rainflood, maximum rainfall depths characteristic of the region should be used for the durations and area sizes that are critical to the study. Figure 3.01 illustrates an evaluation of standard project rainstorm magnitude for a 24-hour duration and 200 square mile area in different zonal regions of the United States east of the 105th meridian. Figure 3.01 indicates that the standard project rainfall depth has been exceeded by approximately 5 to 16 percent of the historical events studied. Based on all 400 storms, about 10 percent of the events exceeded the SPS 200 square mile 24-hour index amount, in some cases by as much as 80 percent. This illustrates that the standard project storm is not of unprecendented magnitude regionally, although it is definitely of a major category. Amounts of precipitation should normally be expressed as a ratio to a precipitation potential index representing potential moisture or topographic effects discussed in section 2.04 and 2.05.

After depth-area-duration criteria are derived, space and time patterns for the standard project storm must be developed. These can be taken from actual storm records, but it is not likely that a single recorded storm could produce a space or time pattern that is satisfactoy for application to a variety of drainage basins. It may be desirable to devise simplified space and time patterns that are sufficiently balanced and representative of general storm characteristics. Such patterns are illustrated in figures 3.02 and 3.03.

In many computations, a space (isohyetal) pattern is not used, but a transposition coefficient is used that accounts for the degree to which the isohyetal pattern does not coincide with the basin boundary. Unless the basin shape is very unusual, transposition coefficients for actual storms on actual drainage basins usually are very high - in the order of 0.95.

Derivation of storm rainfall for a particular drainage basin from generalized criteria is illustrated for probable maximum storm criteria in paragraph 4.03. Similar techniques can advantageously be used for standard project storm determination.

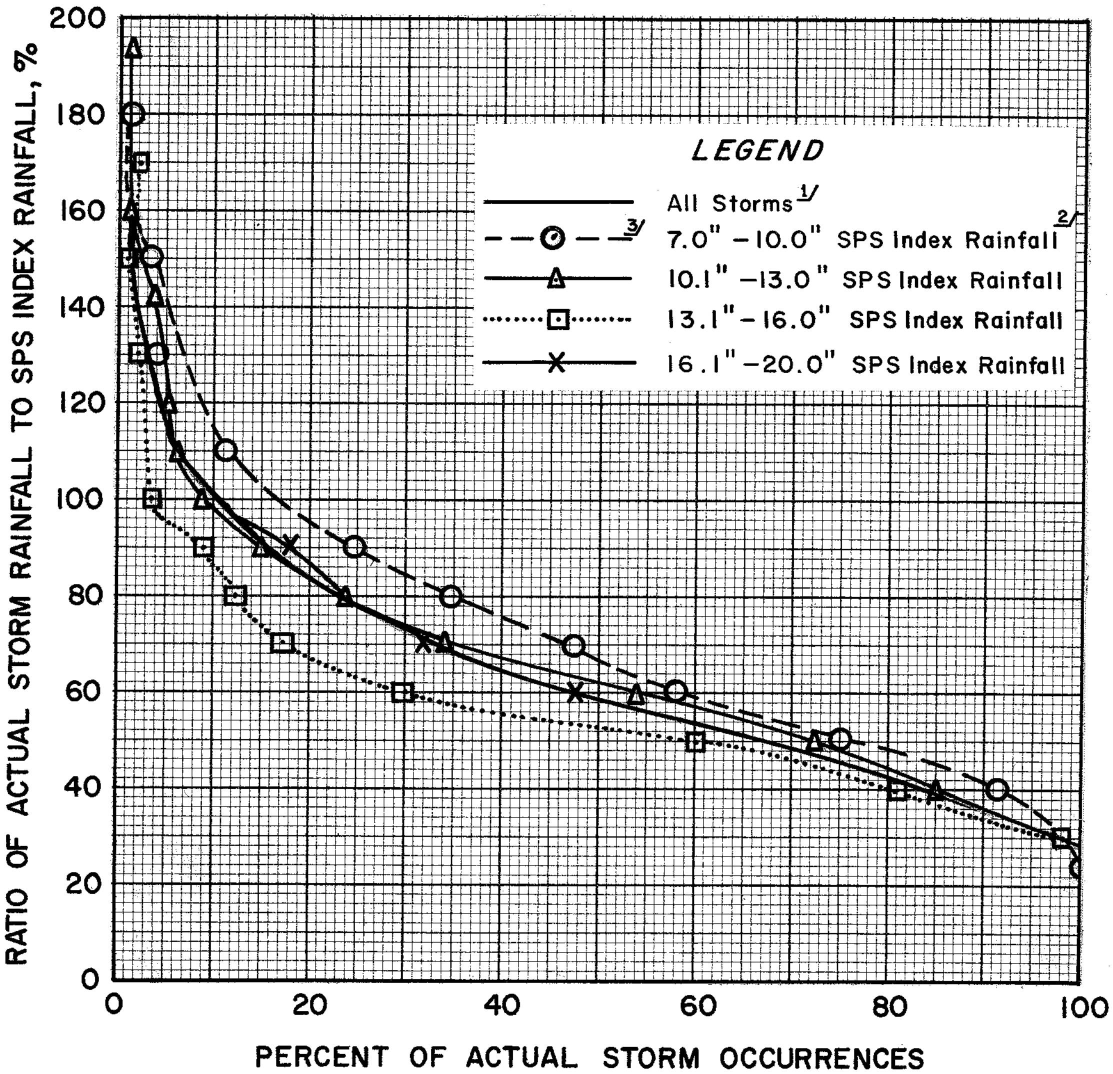


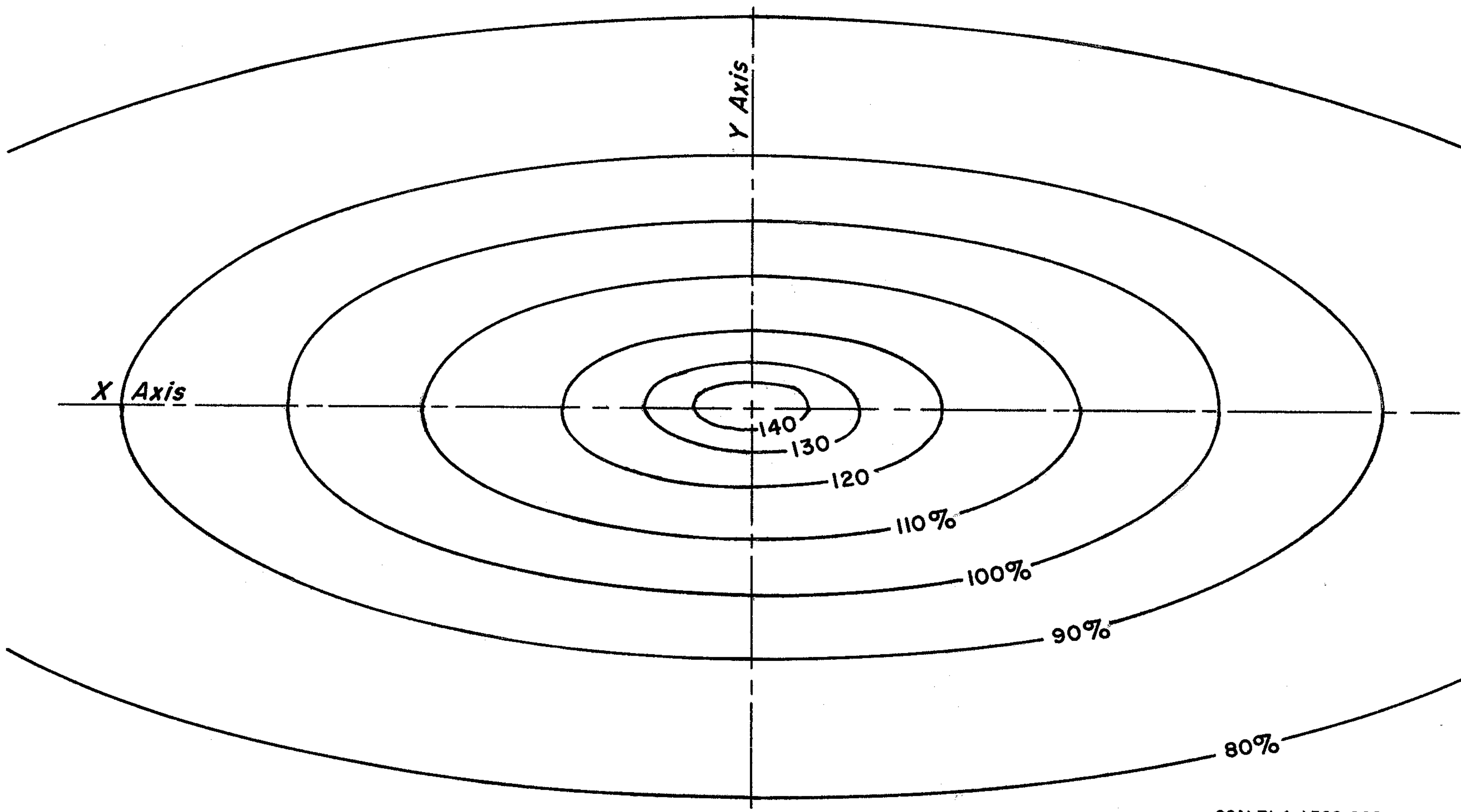
Figure 3.01 Ratio of Standard Project Storm to Maximum Recorded Rainfall

Footnotes:

1/ Based on 400 storms

2/ Average 24 hour rainfall over an area of 200 square miles

3/ Lines represent regions of differing rainfall potential



Total storm rainfall depth expressed as a percent of an index value such as those shown in Figure 2.01.

(Reference Plate 12, EM 1110-2-1411)

Figure 3.02 Example of rainfall spatial distribution

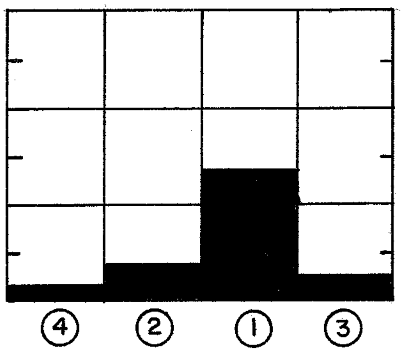


Fig. (b) Typical arrangement of 6-hr rainfall quantities

INDEX RAINFALL IN INCHES	PERCENTAGE OF 24-HOUR RAINFALL IN DESIGNATED 6-HR PERIOD			
	4	2	1	3
1	2	3	4	5
8	1.0	8.0	87.0	4.0
9	2.1	9.5	83.0	5.4
10	3.2	11.0	79.2	6.6
11	4.3	12.3	75.9	7.5
12	5.3	13.8	72.5	8.4
13	6.1	14.9	69.6	9.4
14	7.0	16.0	66.9	10.1
15	7.6	17.0	64.5	10.9
16	8.1	17.9	62.4	11.6
17	8.8	18.9	60.3	12.0
18	9.1	19.7	58.5	12.7
19	9.8	20.3	56.8	13.1
20	10.1	21.0	55.1	13.8

Fig.(c) Tabulation of data from Fig (a)

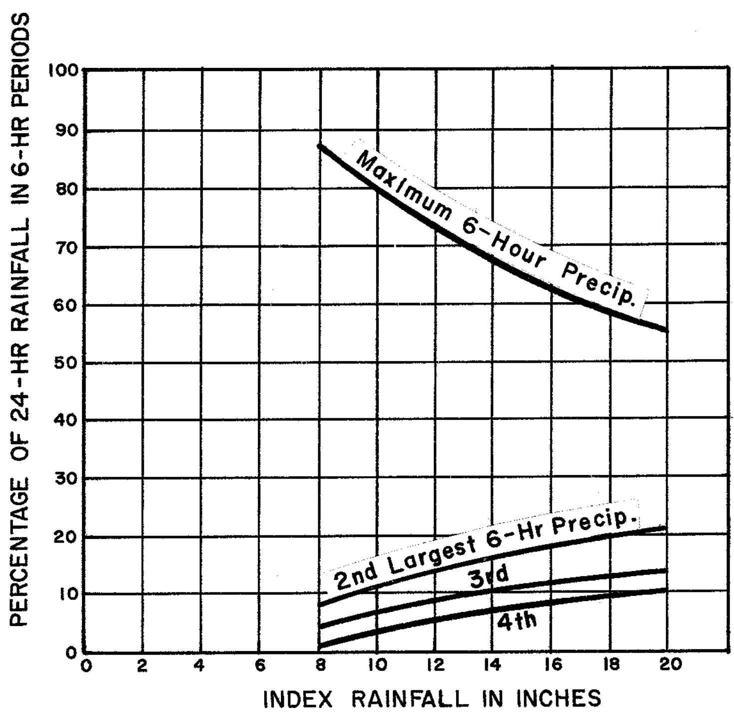


Fig. (a) 24-hour precipitation over 200 sq. mi. vs Percent in 6-hour periods

Figure 3.03 Example of rainfall time distribution

Section 3.03. Standard project loss rates

As discussed in Section 2.08, loss rates during major storms, consisting principally of infiltration, can best be estimated from records of observed storms and floods. Loss rates for use in computing standard project floods should correspond to those considered reasonably likely to occur during storms of such magnitude, estimated on the basis of rates observed in floods that have occurred in the basin or in similar areas.

As discussed in Volume 4, some loss-rate functions are complex and cannot be expressed in a single parameter. Loss rates change with precipitation intensity and with changing ground conditions during the storm. These rates of change can be different during different storms in the same basin. Accordingly, selection of a severe loss rate function for standard project flood computation may require use of a simplified index.

One index of loss-rate severity is the "infiltration index," which is the uniform loss rate that would produce the same volume of runoff as that which actually occurs with the complex function. Use of this index ignores the variation of average loss with average precipitation intensity over a drainage basin, but often produces adequate results. A better index might be the initial value of loss coefficient in the following formulas that relate loss to rainfall intensity and accumulated loss during the storm:

$$L = K R^{E}$$

$$K = K_0 e^{-C\Sigma L}$$

in which:

L = loss rate

K = loss coefficient

R = rainfall rate

E = empirical constant between 0.0 and 1.0

K_0 = loss coefficient at start of storm

C = empirical constant

ΣL = accumulated loss during storm

e = natural log base 2.718

Losses computed with this equation should be constrained between 0.0 and some reasonable upper limit such as 50 mm/hr. In order to compare initial loss coefficients for different storms, it is necessary to derive each one using the same empirical constants E and C. This loss function is one of the methods used in the computer program described in Appendix 1. A simpler function also contained in that program may be preferable in some applications. This simply consists of an initial loss such as 10 to 20 mm followed by a uniform loss rate ranging from 2 to 10 mm/hr. Another procedure widely used by hydrologists consists of an exponential decay of a starting loss rate until a specified minimum is reached and then continuing at a constant rate. Some methods also include a specified recovery rate during periods of no rainfall excess.

Section 3.04. Standard project snowpack

Where the standard project flood is primarily a snowmelt flood, standard project snowpack should envelope maximum observed snowpack and a percentage should be added as a safety factor, depending on the length of snowpack record and history of snowmelt floods. If the record is long or the greatest snowmelt flood in a long time occurred during the period of snowpack record, a small or no factor may be added to the largest observed snowpack. A better alternative procedure would be to construct frequency curves of snowpack and select values of about 100 to 200-year exceedence interval. This would be the best procedure where only short records are available. If no snowpack records are available, it is ordinarily best to select the most severe snowmelt flood of record and add a factor for determining a standard project flood, rather than to compute one from snowpack. However, it is possible, but highly questionable, to derive snowpack from precipitation and temperature records. This is not recommended, because precipitation records of snowfall are of poor quality, and losses during the accumulation season are difficult to estimate.

Where the standard project flood is a rainflood with snowmelt contribution, a moderate snowpack is ordinarily most critical, because deeper packs tend to retain or retard runoff from rain and decrease peak runoff rates. Depending on a number of factors such as rainfall temperatures and storm duration, critical snowpack water equivalent at the start of a standard project rainflood would be approximately 10 percent of the standard project storm rainfall.

In mountainous regions, deeper snowpack occurs at higher elevations where melt and storm temperatures are lower, and critical conditions are best represented at intermediate levels where the greatest proportion of the flood runoff originates. Snowpack at the lower levels will then be depleted early in the storm or season, and some snowpack at the higher levels will remain after the end of the flood.

Section 3.05. Standard project snowmelt

Temperatures and other melt factors during the snowmelt season must be selected with care for standard project use, because, as explained earlier, maximum melt factors throughout the melt season are not necessarily the most critical. In many cases, it would be best to retain very low temperatures (and other melt factors) during the first quarter of the melt season and then to use maximum values during the remainder of the season, but care should be exercised so that probabilities of the components are not unreasonably compounded.

Since the melt season usually lasts 2 to 4 months, and occurs during the spring when normal temperatures are changing rapidly, a curve of temperature vs. duration should be constructed in terms of temperatures above (or below) normal for each day. Envelope curves or frequency curves of temperature for various durations can be constructed and applied to the pattern of normal daily temperatures. Curves of other melt factors can be established in the same manner. Envelope values for a long period of record or values exceeded once in about 50 years should be satisfactory for standard project use. The time sequence of temperatures must be consistent with time sequences of precipitation and other factors during the melt season. Critical sequences may be different for different applications, and it is ordinarily necessary to make several runoff computation trials before a critical sequence can be selected for any particular application.

In the case of rainfloods, the temperatures and other factors must be maximum consistent with rainfall. Usually maximum surface dew points and pseudo-adiabatic lapse rates will determine maximum rainstorm temperatures.

Section 3.06. Standard project base flow

Base flow is the amount of runoff that occurs during a flood but is not computed directly from rainfall and snowmelt. It is flow that would occur if the current storm did not occur plus any return flow that is not directly computed from the current rain and snowmelt. Some hydrologists prefer to include an added amount to account for an increase in subsurface flow resulting from the current storm and any remaining water stored in the channel from an antecedent event.

Base flow to be used in standard project flood computations is ordinarily obtained by enveloping the largest base flow amounts that have occurred in past floods and applying this to the standard project flood in the manner in which it occurred in past floods.

Section 3.07. Standard project runoff computation

Where unit hydrographs have been derived from recorded rainfall or snowmelt and runoff data, data inaccuracies and other factors usually result in different unit hydrographs for different storms. Those derived for the larger floods are considered to be the most reliable for standard project use. If no large floods at the location are available for study, some increase in the maximum ordinate and decrease in time to peak of unit hydrographs derived from smaller floods would normally be advisable to account for increase flow velocities in those rivers where larger flood flows can be expected to be accompanied by increased hydraulic efficiency. If future urbanization on channel modifications are anticipated, adjustments should be made in the unit hydrographs to reflect these changes also.

Where methods other than the unit hydrograph method are used in runoff computation, adequate adjustments may likewise be necessary in order to assure that severe rainfall-runoff relations are applied. Care must be exercised, where new reservoirs eliminate the retarding effects of channel storage, that such changes are properly accounted for (by reducing the time of concentration or the routing effects).

Runoff computations are then straight-forward. Losses are subtracted from rainfall and snowmelt for each computation interval, direct runoff is computed from the resulting excess values by use of the unit hydrograph or other method, and base flows are added.

Section 3.08. Standard project flood series

In the design of storage facilities, it is occasionally necessary to examine the operation of the facility over a long period of time. In the case of rainfloods, it is ordinary to compute runoff only for a period of 3 or 4 days, because uncertainties in the rainfall-runoff relation become very serious as the longer durations (with lower rainfall rates and greater time and area variations) occur.

Where it is important to consider longer sequences of rainflood runoff, as in studies to assure sufficient time between storms to empty flood control space, flood periods can be added before and after the standard project computed runoff. These can be sequences that occurred in relation to large recorded rainfloods or can be derived from flood volume frequency curves. In the latter case, incremental runoff volumes for incremental durations of about 3 days can be computed for an exceedence interval of about 200 years. These volumes can be used with a typical time pattern to develop hydrograph components, the larger of which should ordinarily precede the main period of computed runoff.

Section 3.09. Summary of procedure--rainfloods

Standard project rainfloods are derived in the following steps:

a. Develop a map of the entire hydrologic region showing relative values of storm precipitation potential. This map would not be necessary if the storm precipitation potential is relatively uniform throughout the region, as in some plains areas far removed from ocean sources of moisture. This could be a map of normal annual precipitation or an isohyetal map of 3-day precipitation having an exceedence frequency of 10 years per hundred years (3-day 10-year precipitation).

b. For each large recorded storm in the region, develop the depth-area-duration relationship of maximum total precipitation expressed as a ratio to the index of storm precipitation potential from step a.

c. Construct a standard project depth-area-duration relationship that envelops all but any radically unusual values in the various storms.

d. For the drainage area for which the standard project flood is to be derived, select a representative time distribution of precipitation from recorded storms and adjust the quantities to correspond to standard project depths for the drainage area size and for each of a representative set of durations.

e. By studies of rainfall-runoff relations in the basin or nearby, develop for each subarea a unit hydrograph, loss rates and base flow that are representative of those for the most severe floods of record. Techniques are discussed in Volume 4. Reconstitution of a major rainflood is illustrated in figure 3.04.

f. Determine routing characteristics and coefficients for reservoirs and river reaches pertinent to the study.

g. Using runoff computation techniques described in Volume 4 and routing techniques described in Sections 2.11 and 2.12 herein, compute the standard project flood hydrograph for the design location.

h. Computations described through step g are usually confined to a duration of 3 or 4 days. Where appropriate, a series of floods can be computed as described in Section 3.08. These are illustrated in figures 3.05 and 3.06.

Section 3.10. Summary of procedure--snowmelt floods

The standard project snowmelt flood for a specified location can be computed as follows:

a. From records of snowpack water equivalent or a combination of winter snowfall and winter snowpack losses, determine the maximum snowpack water equivalent that would occur at the various points within the drainage basin if the most severe snow accumulation conditions of the entire region over a long period of time (40 or 50 years) should occur over the specific basin. This may require some generalized estimates of snowpack variations and an upward adjustment of snowpack quantities if records are short.

b. Adopt either the degree-day or energy-budget melt computation technique, depending on the adequacy of available meteorological data.

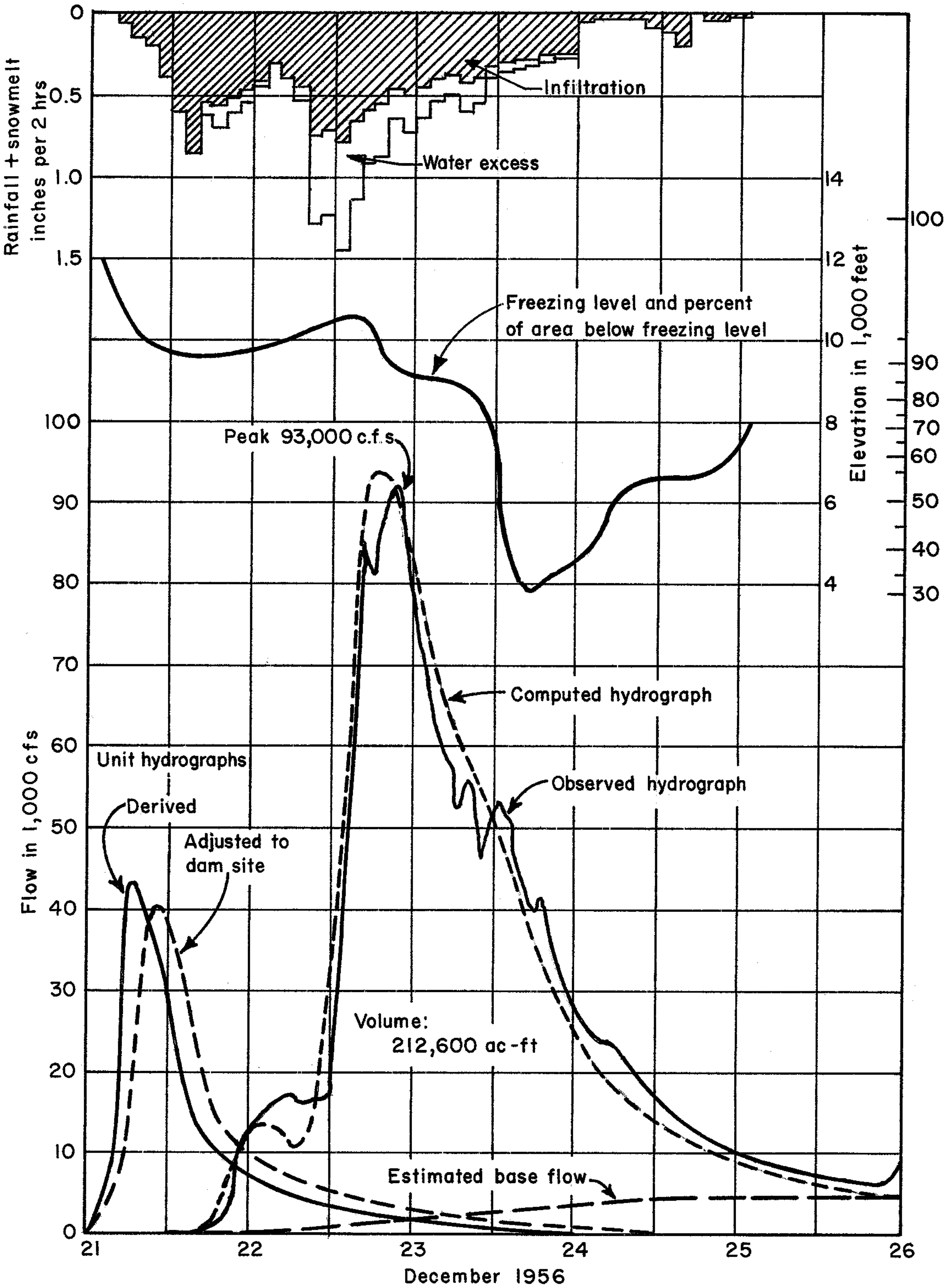


Figure 3.04 Illustration of unit hydrograph and loss derivation

FIGURE 3.05

STANDARD PROJECT FLOOD COMPUTATION

Hour	Computation of total water: Basin precipitation: Percent	Basin precipitation: in.	34°F elev. feet	Precip. as rain: percent	Rain in.	Snow-melt in.	Total ΔP in.	Computation of water excess: ΔL in.	ΣL in.	We in.	Computation of runoff: Unit graph	Base flow	Total runoff
											Mean flows 1,000 c.f.s.		
0									16.00				
6	0.8	0.16	8,800	83	.13	0.06	0.19	0.19	16.19	0	5.5	1.0	1.0
12	6.9	1.35	8,800	83	1.12	0.16	1.28	0.90	17.09	0.38	33.8	1.0	3.1
18	12.1	2.36	9,100	86	2.03	0.26	2.29	1.33	18.42	0.96	28.2	1.0	19.1
24	1.0	0.20	9,100	86	0.17	0.06	0.23	0.23	18.65	0	12.4	1.0	44.2
30	0.3	0.06	8,800	83	0.05	0.10	0.15	0.15	18.80	0	7.9	1.1	32.9
36	0	0	8,800	83	0	0.12	0.12	0.12	18.92	0	5.7	1.2	16.1
42	1.5	0.29	10,200	97	0.28	0.12	0.40	0.40	19.32	0	4.4	1.3	11.0
48	11.6	2.26	10,200	97	2.19	0.22	2.41	1.32	20.64	1.09	3.5	1.4	14.5
54	10.3	2.01	10,500	98	1.97	0.20	2.17	1.20	21.84	0.97	2.8	1.5	49.2
60	12.9	2.52	10,500	98	2.47	0.24	2.71	1.39	23.23	1.32	2.2	1.6	76.8
66	21.3	4.15	10,900	99	4.11	0.18	4.29	1.83	25.06	2.46	1.6	1.8	104.3
72	13.9	2.71	10,900	99	2.68	0.12	2.80	1.39	26.45	1.41	1.0	2.0	153.5
78	4.9	0.95	10,300	97	0.92	0.08	1.00	0.67	27.12	0.33	0.5	2.2	153.2
84	1.4	0.27	10,300	97	0.26	0.08	0.34	0.31	27.43	0.03	0.2	2.3	105.8
90	1.1	0.21	9,600	92	0.19	0	0.19	0.19	27.62	0		2.4	65.8
96												2.5	44.8
102												2.5	34.1
108												2.5	27.0
114												2.5	21.5
120												2.5	16.8
126												2.5	12.6
132												2.5	8.9
138												2.5	6.0
144												2.5	4.1
150												2.5	3.0
156												2.5	2.6
Total	100.0	19.50			18.57	2.00	20.57	11.62		8.95	109.7	50.3	1,031.9

FIGURE 3.06

STANDARD PROJECT FLOOD SERIES

Day and Hour	Mean flow 1,000 c.f.s.	Day and Hour	Mean flow 1,000 c.f.s.	Day and Hour	Mean flow 1,000 c.f.s.
1-6	1.8	11-6	1.2	21-6	2.0
12	7.1	12	1.1	12	2.0
18	11.2	18	1.1	18	2.0
24	15.2	24	1.1	24	2.3
2-6	22.4	12-6	1.0	22-6	9.3
12	22.4	12	1.0	12	14.6
18	15.4	18	3.1	18	19.8
24	9.7	24	19.1	24	29.3
3-6	6.5	13-6	44.2	23-6	29.3
12	5.0	12	32.9	12	20.0
18	4.0	18	16.1	18	12.4
24	3.2	24	11.0	24	8.6
4-6	2.5	14-6	14.5	24-6	6.5
12	1.9	12	49.2	12	5.1
18	1.3	18	76.8	18	4.0
24	0.9	24	104.3	24	3.2
5-6	0.6	15-6	153.5	25-6	2.5
12	0.4	12	153.2	12	1.9
18	0.3	18	105.8	18	1.6
24	0.3	24	65.8	24	1.5
6-6	4.3	16-6	44.8		
12	17.6	12	34.1		
18	27.4	18	27.0		
24	37.2	24	21.5		
7-6	54.8	17-6	16.8		
12	54.7	12	12.6		
18	37.7	18	8.9		
24	23.5	24	6.0		
8-6	16.1	18-6	4.1		
12	12.2	12	3.0		
18	9.7	18	2.6		
24	7.7	24	2.4		
9-6	6.0	19-6	2.4		
12	4.5	12	2.3		
18	3.3	18	2.3		
24	2.3	24	2.2		
10-6	1.7	20-6	2.2		
12	1.4	12	2.1		
18	1.3	18	2.1		
24	1.2	24	2.1		

Reference:

See par. 3-08 and Chapter 5.

In most cases, the former technique will be used, because only temperature and precipitation data are required, and the technique has proven to be relatively satisfactory.

c. Develop a relationship of maximum temperature vs. duration that envelops maximum values recorded during past snowmelt seasons. Values of temperature should be expressed for this purpose in degrees above normal for each calendar day, using a smooth curve of normal temperature vs. time. If the energy-budget method of snowmelt computation is used, derive corresponding maximum values of other melt variables such as wind and solar insolation.

d. In many cases, the most severe snowmelt runoff occurs if low temperatures prevail during the early part of the snowmelt season when normal temperatures are low and then extremely high temperatures follow. By a series of approximations, derive a temperature sequence (and corresponding sequences of other melt factors if the energy-budget method is used) that results in the most severe runoff computed in accordance with the following step. Determine also the maximum precipitation that would occur in the melt season consistent with these melt factors.

e. Taking account of different temperatures in different elevation zones, compute snowmelt during each computation interval of the storm period in accordance with techniques described in Section 2.07 and add any precipitation that falls as rainfall. Develop unit hydrograph, loss rate and base flow criteria as described in paragraph 3.09e and apply to the snowmelt and rainfall totals for each period using techniques described in Volume 4 in order to obtain the standard project snowmelt hydrograph for each sub-area. Route and combine sub-area hydrographs as described in paragraphs 3.09f and g.

Section 3.11. Use of electronic computers

Computations described in this chapter can be readily performed with existing computer programs such as those described in the appendices of this chapter. When generalized criteria for a region are formulated, they can be included in a computer program such as those described in Appendix 1 of this Volume and Appendix 1 of Volume 1.

Chapter 4

Probable Maximum Floods

CHAPTER 4. PROBABLE MAXIMUM FLOODS

Section 4.01. Definition

The probable maximum flood is that flood discharge which would result from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. Since there is great uncertainty in estimating potential hydrologic magnitudes, extreme conservatism in estimating each variable would ordinarily result in probably maximum floods that are unreasonable and of no practical utility. Accordingly, a great amount of judgment is ordinarily required in selecting rainfall, snowpack, loss rates, etc. A moderately conservative estimate of extreme values of each variable should be used, and these should be consistent with the selected application as to be made. The moderate, rather than extreme, degree of conservatism is the factor that has led to the terminology probable maximum rather than maximum possible. Nevertheless, the resulting flood must be one that the engineer considers is virtually impossible of exceedence, because the flood is ordinarily used to assure the integrity of a dam whose failure would cause a great loss of life and major property damage that would not occur under natural conditions. Probably maximum flood estimates are appliable to projects where consideration is to be given to virtually complete security against potential floods.

Section 4.02. Storm maximization

There are two general factors that determine the severity of precipitation during storms. One is the amount of moisture present in the atmosphere and the second is the intensity of the precipitation-forming mechanism.

Moisture that exists in the atmosphere at the start or during a storm can be measured by integrating the specific humidity measured in radiosonde observations at locations in the storm area. Total moisture in the atmosphere above the earth's surface can then be expressed in millimeters as a depth. Amounts vary greatly, but the general order of magnitude is about 50 millimeters. Maximum moisture that can occur during a storm is usually considered to be that which would correspond to saturated air at temperatures that would correspond to a pseudo-adiabatic lapse rate curve passing through the pressure and temperature point at the source of moisture. Since the source of moisture is usually an ocean surface, the maximum ocean-surface water temperature at the time of year would be used as the maximum surface temperature (and dew point). Curves of temperature and moisture vs. surface temperature for pseudo-adiabatic lapse rates are given in figures 4.01 and 4.02.

It is usual practice to assume that storm precipitation for the same storm mechanism would be proportional to the precipitable water. Thus, insofar as the influence of moisture in the air mass is concerned, recorded storm precipitation can be maximized by multiplying by the ratio of maximum probable precipitable water to recorded storm precipitable water.

The factor of storm mechanism is more difficult to measure and assess in relation to the effect on storm precipitation. In convective type storms, it is usually assumed that such large numbers of these storms occur that a near-maximum mechanism occurs someplace in the region during a long period of observation. Thus a simple envelope of rainfall amounts adjusted for maximum moisture might well represent probable maximum convective-storm precipitation. In the case of cyclonic storms, wind velocity is a good index of storm-mechanism intensity. Maximum probable winds can be estimated by an envelope procedure using observed winds in a large number of cyclonic storms. Some mod-

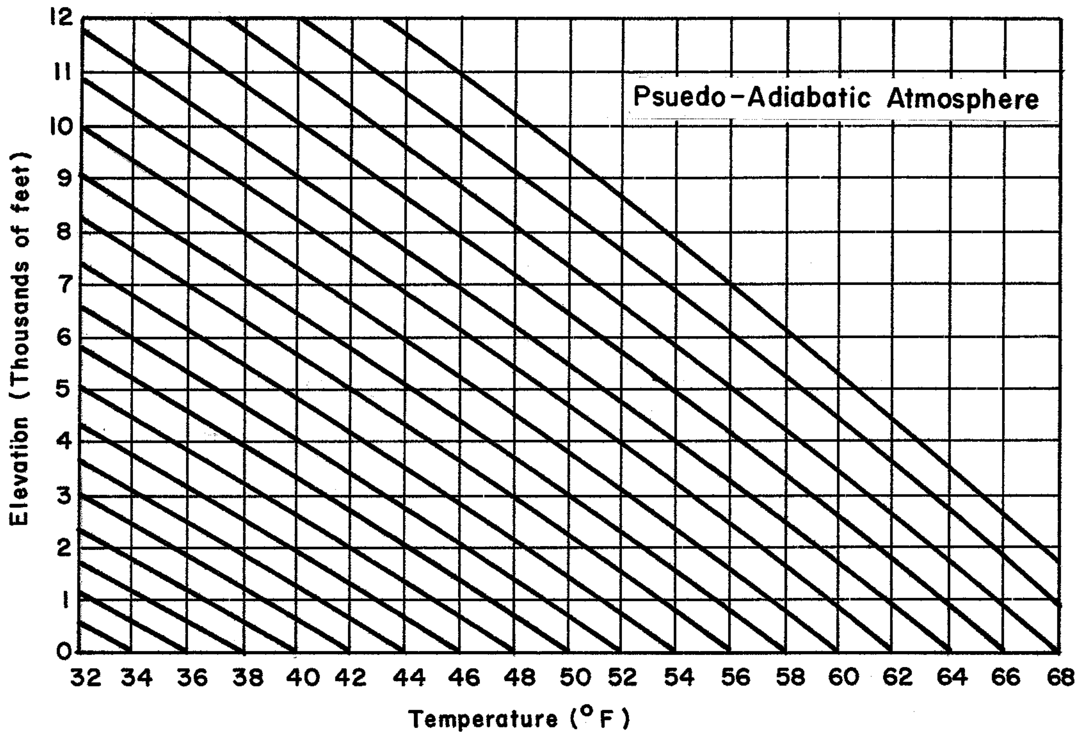


Figure 4.01 Decrease of temperature with elevation

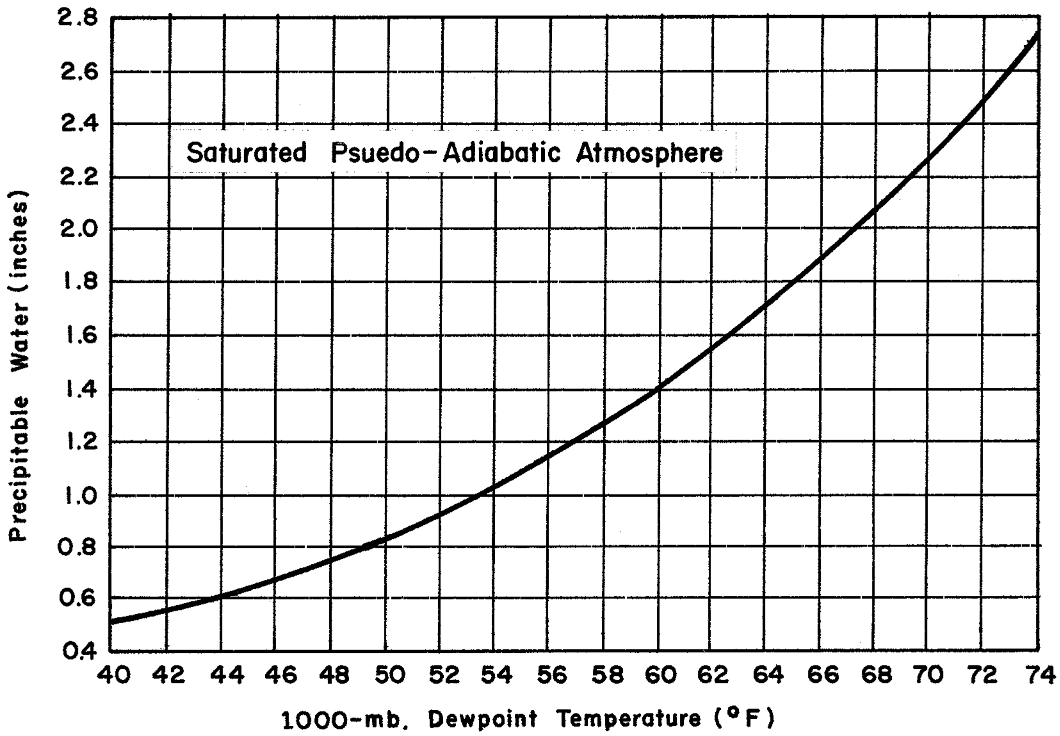


Figure 4.02 Variation of precipitable water with 1000-mb. dew point temperature

erate increase over the maximum observed might be warranted. In the case of cyclonic-storm rainfall, amounts measured in observed storms can be multiplied by the ratios of maximum probable precipitable moisture and winds to values recorded in each storm.

Section 4.03. Probable maximum storm derivation

Probable maximum storm precipitation amounts expressed as a depth-area-duration function are obtained generally by enveloping maximized precipitation amounts in all observed storms for each duration and area size. Where storm potential varies arealy or regionally, as discussed in Sections 2.03 and 2.04, ratios of normal precipitation at the location for which the estimate is being made to normal precipitation at the location of each recorded storm should be applied to precipitation amounts for that storm before the enveloping procedure.

Figures 4.03 and 4.04 illustrate generalized criteria developed by the U.S. National Weather Service for the eastern United States. Derivation of a probable maximum storm hyetograph based on these criteria is illustrated in figure 4.05. Use of such generalized criteria facilitates engineering evaluations and usually assures reliable results. The World Meteorological Organization has published a document which gives considerable detail on the theory and techniques used by the meteorologist in deriving probable maximum precipitation. It is WMO No. 332, World Meteorological Organization, Operational Hydrology Report No. 1, "Manual for Estimation of Probable Maximum Precipitation," 1973.

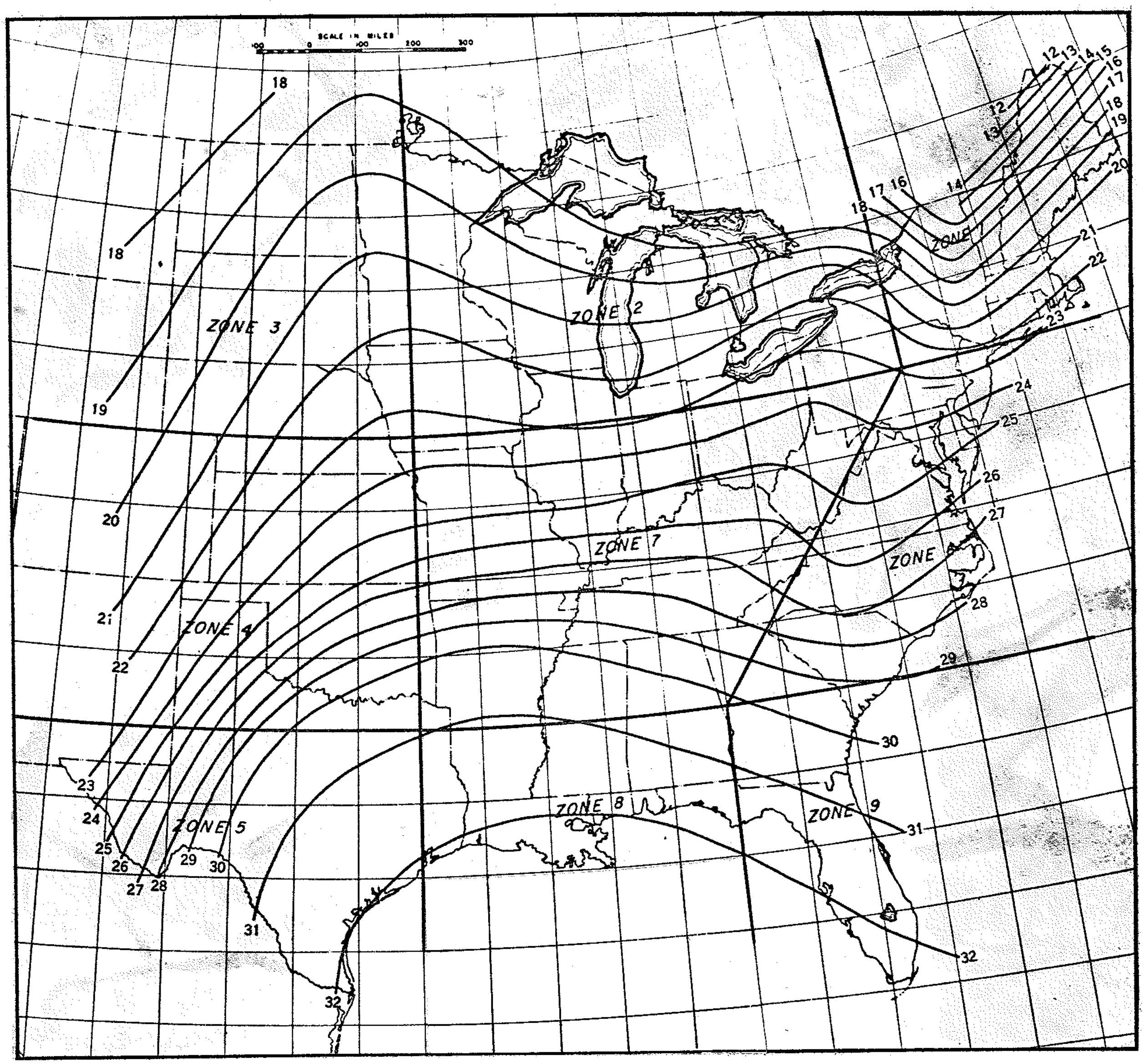


Figure 4.03 Probable Maximum Precipitation. For 200 square miles—24 hours (In inches) The All Season Envelope

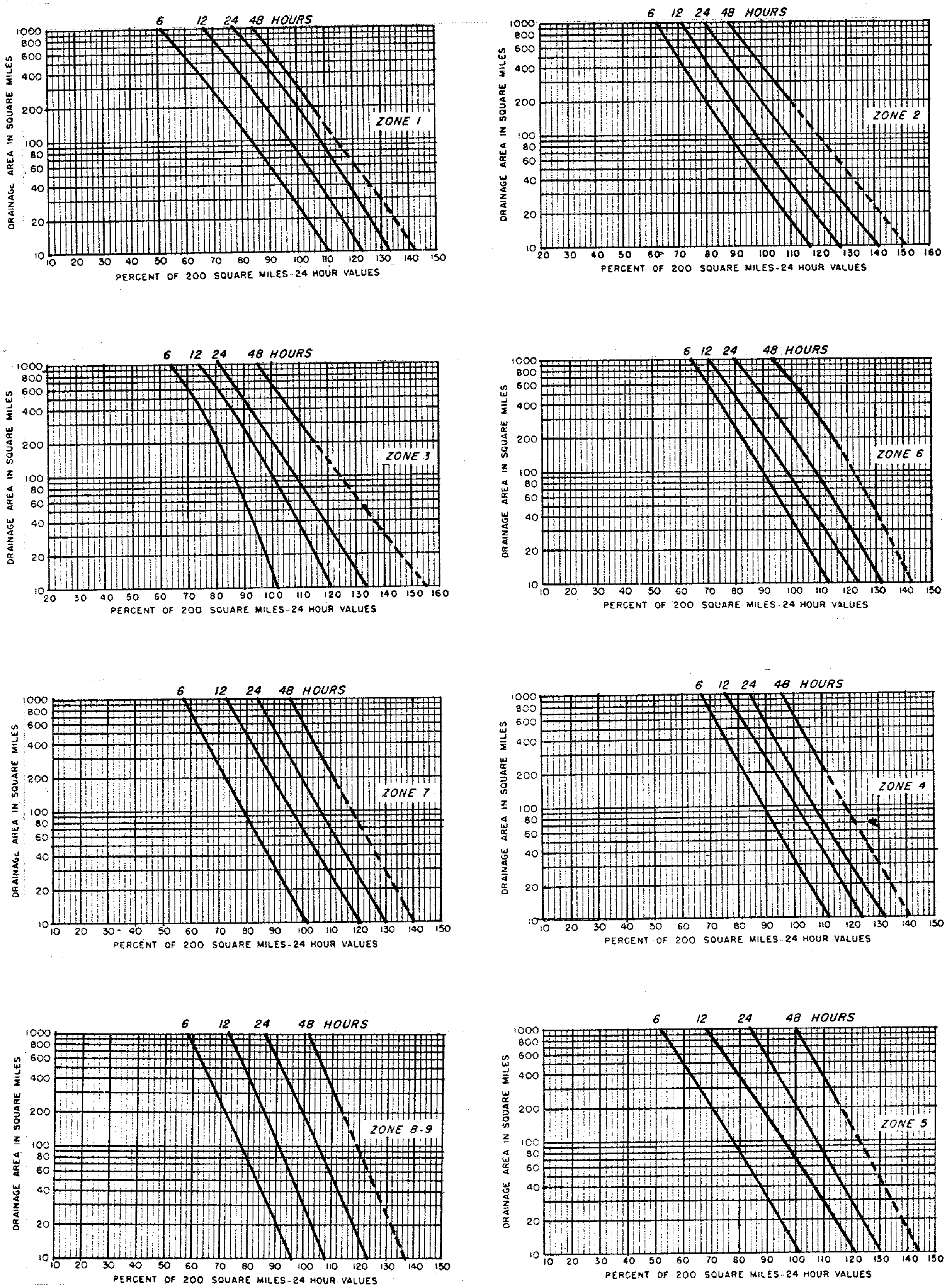


Figure 4.04 SEASONAL VARIATION

Depth– Area– Duration Relationships

Percentage to be applied to 200 square miles 24 hour probable maximum precipitation values for: The All Season Envelope

Figure 4.05

EXAMPLE OF PROBABLE MAXIMUM STORM

1. BASIC DATA
   a. DA = 19.6 sq. mi.
   b. Shape factor 81.2 percent (Imperfect fit of isohyetal pattern to basin shape)
   c. LAT 36° - 30'
   d. LONG 97° - 40'
   e. Index = 26.8" (figure 4.03)
   f. Index with shape factor = 21.76"

2. RAINFALL SUMMARY

Duration hrs	% Index*	Total Rainfall (21.76x Index)	Rainfall Increment	Duration of Increment
6	106	23.06	23.06	6 hr
12	118	25.68	2.62	6 hr
24	124.5	27.09	1.41	12 hr
48	134.0	29.15	2.06	24 hr

* From figure 4.04

3. SIX HOUR RAINFALL IN CRITICAL SEQUENCE

HR	% 24 HR RF	ΔRF	RF TOTALS
6	2.07	.04	
12	9.67	.20	2.06
18	85.12	1.76	
24	3.14	.06	
30	2.07	.56	
36	9.67	2.62	27.09
42	85.12	23.06	
48	3.14	.85	
			29.15

Section 4.04. Probable maximum ground conditions

Ground conditions that affect losses during the probable maximum storm should be the most severe that can reasonably exist in conjunction with maximum probable precipitation. Lowest loss rates that have been observed might be used if there is reasonable assurance that the entire range of possible losses has been experienced. However, loss estimates are subject to major uncertainties, and there are cases where negative loss rates are computed simply because of inadequate precipitation data. Accordingly, some allowance must be made for this uncertainty, and loss rates that are conservatively low should be selected for probable maximum flood computation.

Where it is possible for the ground to be frozen at the start of a rainflood or snowmelt flood, it can be concluded that zero or near-zero loss rates should be used for probable maximum flood computation.

There may exist a seasonal variation in minimum loss rates, in which case, rates selected should be those representative of most extreme conditions for the season for which probable maximum runoff is being computed.

Section 4.05. Probable maximum snowpack

As in the case of standard project snowpack, it is not feasible to compute maximum snowpack accumulation from winter precipitation, temperatures and snowpack losses. Probable maximum snowpack for floods that are primarily snowmelt floods should be estimated from observed snowpack data and should be considerably larger than standard project snowpack. It may be satisfactory to add a factor such as 25 percent to standard project snowpack in order to obtain probable maximum snowpack.

In the case of rainfloods that have some snowmelt contribution, snowpack used for probable maximum rainflood computation should be the maximum that can contribute toward the peak flow and runoff volume of the flood without inhibiting the direct runoff from rainfall. The most critical snowpack for probable maximum rainflood computations is substantially larger than that for the standard project rainflood, because the heavier rainfall and warmer temperatures will have a greater capability to melt snow.

As discussed for standard project snowpack in Section 3.04, the critical snowpack in mountainous regions will ordinarily be located at elevations where most of the rainflood runoff originates. Snowpack is ordinarily greater at higher elevations and less at lower elevations, and hence critical snowpack will not exist at all elevations.

Section 4.06. Probable maximum snowmelt

The discussion in Section 3.05 of factors to be considered in selecting melt factors for the standard project flood also apply to the selection of melt factors for the probable maximum flood. Temperatures should be higher for both snowmelt floods and rain floods, and other melt factors should be correspondingly higher. In the case of snowmelt floods, temperature patterns may be different from standard project snowmelt temperature patterns but the same principles for developing such patterns apply. In the case of rainfloods, greater precipitation amounts are usually accompanied by higher temperatures, but again the same principles for developing temperature patterns apply as for the standard project flood.

Section 4.07. Probable maximum base flow

Base flow for the probable maximum flood is not such a critical item as for the standard project flood (Section 3.06), because the peak flow, which is not greatly affected by base flow, is the primary characteristic of interest in the probable maximum flood. Nevertheless, it is prudent to adopt a base flow value that is more severe than that which would be used for standard project flood derivation. There is no general guide for this, but an additional 10 to 25 percent should suffice.

Section 4.08. Probable maximum flood computation

Many of the guides contained in Section 3.07 are relevant to the computation of the probable maximum flood. Rainfall-runoff factors should be selected in this case as the most severe that are reasonably consistent with the storm and flood conditions, and should be considerably more severe than those selected for the standard project flood. In all cases where the unit hydrograph technique is used, the unit-hydrograph peak should be increased substantially to account for the more rapid concentration of flood flows in the stream system. Channel routing coefficients should likewise be modified toward greater translation speed and less storage effects because of the more efficient hydraulic flow conditions during larger floods.

In application of the probable maximum flood for spillway design, allowance should be made for the accelerating effect of a reservoir in relation to the stream reaches that are inundated, and the reservoir level at the start of the flood should be the highest level reasonably consistent with probable maximum flood conditions.

Section 4.09. Antecedent conditions

In many spillway design applications, flood conditions that precede the probable maximum flood may have substantial influence on the regulatory effect that the reservoir has on the probable maximum flood. In such cases, it is appropriate to precede the probable maximum flood with a flood of major magnitude at a time interval that is minimum consistent with the causative meteorological conditions. While a special meteorological study is desirable where possible for this purpose, it is often considered that the start of a probable maximum flood could reasonably, as an extreme possibility, be preceded by the start of the standard project flood 4 or 5 days earlier.

Section 4.10. Summary of procedure

Procedures used in the computation of probable maximum rainfloods and snowmelt floods are similar to those described in Sections 3.09 and 3.10 for the standard project flood. The only significant differences are that criteria used are as described in this chapter and an antecedent flood is used as described in Section 4.09 in lieu of a standard project rainflood series. Computation procedures are as illustrated for the standard project flood in figures 3.04 to 3.06.

Section 4.11. Use of electronic computers

As in the case of standard project floods, computations of probable maximum floods can be accomplished using any of a number of existing computer programs such as those described in Appendix 1 of this Volume and Appendix 1 of Volume 1. Generalized criteria for probable maximum flood derivation can also be programmed for computer use, as has been done in these two computer programs for criteria in the eastern USA.

Chapter 5

Balanced Floods

CHAPTER 5. BALANCED FLOODS

Section 5.01. Definition and need

A balanced flood is one that is of equal severity for all possible critical durations of project design. Severity is expressed in terms of exceedence probability or exceedence frequency.

In the planning of a flood control project involving storage or in the development of reservoir operation rules, it is not ordinarily known what the critical duration will be, because this depends on the amounts of reservoir space and release in relation to flood magnitude. When alternative types of projects are considered, critical durations will be different, and a design flood should reflect a degree of protection that is comparable for the various types of projects. Accordingly, balanced hypothetical floods are useful for planning, design and operation purposes.

Section 5.02. Varying volume-duration relationships of floods

Time patterns of floods that occur at any particular location are of a great variety. Some floods have high peak flows and are of short duration. Others have the reverse characteristics, and, of course, short-duration volumes can be small in relation to long-duration volumes in some floods and large in others. No one historical flood would ordinarily be representative of the same severity of peak flow and runoff volumes for all durations of interest.

If a project is designed to regulate all floods of record, it is likely that one flood will dictate the type of project and its general features, because the largest flood for peak flows is also usually the largest-volume flood. Yet, there is information in the other floods that can be used to obtain a better balance of the peak flow and volumes for various durations of a design flood. It is important that this information be used effectively.

Section 5.03. Runoff volume frequency curves

The varying characteristics of runoff volume vs. duration at any location can be effectively represented by a set of runoff peak and volume frequency curves developed as described in Volume 3 and illustrated in figure 5.01. Floods that dominate the peak-flow and short-duration-volume curves are not necessarily the same ones that dominate the long-duration volume curves. Yet, a set of values for peak flow and each duration that have the same exceedence probability would represent the same degree of severity as defined in Section 5.01. Such a set of values, possibly with interpolated values added, can be used as described below for the construction of a balanced hypothetical flood.

Section 5.04. Representative hydrograph

Even though a balance is obtained among runoff peak flow and volumes for various durations at a given location, the time sequences of flows in a hypothetical flood that would make up this balance can be any of a great variety of time sequences. For example, flows could rise rapidly and recede gradually or vice versa, and still retain the same volume-vs.-duration relationship. Although this item is of secondary importance, it does have some influence on design and therefore should be considered carefully. This can be done by studying the relative timing of maximum volumes for specific durations in relation to the time of peak flows for the largest floods of record. A representative discharge sequence can then be selected from among recorded floods. This would be representative only with respect to flow sequences and not with respect to absolute or relative magnitudes of flow for various durations.

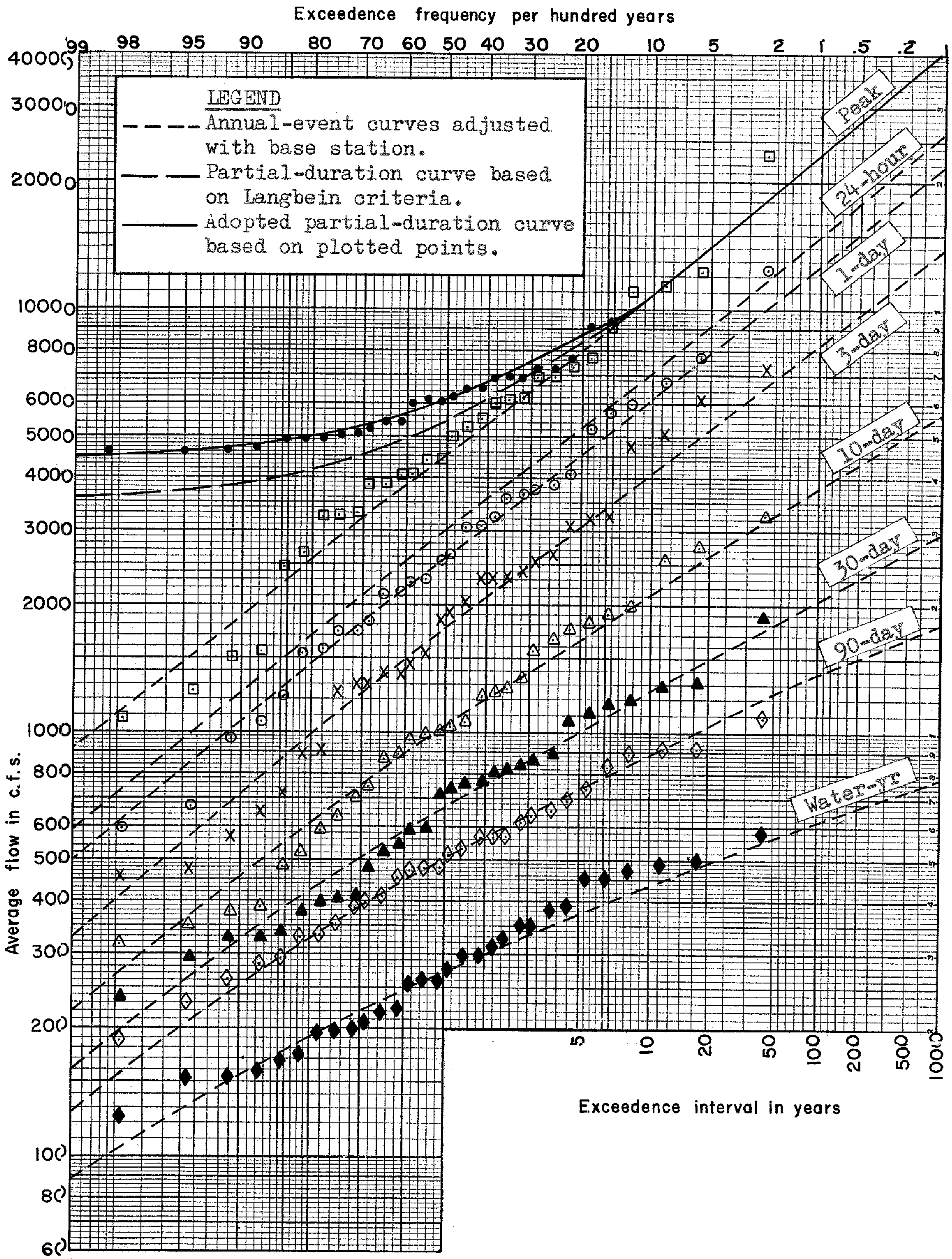


Figure 5.01 Max-runoff volume frequency curves

Section 5.05. Hydrograph balance routine

Runoff peak and volume frequency curves are used in conjunction with a representative hydrograph to obtain a balanced hydrograph as follows:

a. Select an exceedence frequency suited to the needs of a particular problem. This may be a design frequency or a frequency that represents a range of frequencies particularly pertinent to flood damage evaluation.

b. Obtain target flow peak and volumes corresponding to the selected exceedence frequency, using a set of frequency curves such as that illustrated in figure 5.01.

c. Change the maximum ordinate of the representative hydrograph to the target value for the duration equal to the hydrograph tabulation interval. This is done ordinarily, because each ordinate of a hydrograph is considered to represent a volume of flow lasting one computation interval of time. In most cases, it may be desirable to use the target peak flow for this and to ignore the small error in volume thus incurred. This is because it is important to represent the maximum instantaneous flow, and the computation interval should be short enough that instantaneous flows effectively represent the average of flows for a computation interval.

d. When some balanced hydrograph values have been adjusted for any specific duration, the segment of the unadjusted representative hydrograph that has maximum volume for the next longer duration is determined. All adjusted flows within this segment are expressed as a total runoff volume, which is then subtracted from the target volume for the next longer duration. All of the unadjusted flows within this duration are then adjusted so that the total volume they represent will correspond to the target value. This step is repeated until the last duration is reached.

e. It is possible, although a rare occurrence, that the representative hydrograph will not have maximum volumes for all durations within the time period of maximum volumes for longer durations. It is also possible that successive adjustments as described in step d will substantially modify the shape of the representative hydrograph. Either of these circumstances could result in a hydrograph whose volume-duration characteristics do not conform to target values. Consequently, it is necessary to check this at the end of each adjustment sequence and, if substantial differences exist, steps c and d should be repeated using the adjusted hydrograph as the new representative hydrograph. A number of such repetitions might be necessary in some cases.

A computer program that accomplishes the balancing routine as described above is contained in Appendix 3. An example of a balanced hydrograph is shown in figure 5.02.

Section 5.06. Application

A balanced flood can be used as a design flood to assure that the project will provide protection against a flood having an exceedence frequency equal to that selected for the balanced flood. If a project design is sensitive to peak flows or short-duration volumes, it will not be sensitive to long-duration volumes, and vice versa. Accordingly, there is no significant escalation of severity by including maximum volumes for all durations in the same design hydrograph.

Application of balanced floods for economic evaluations is discussed in Chapter 6.

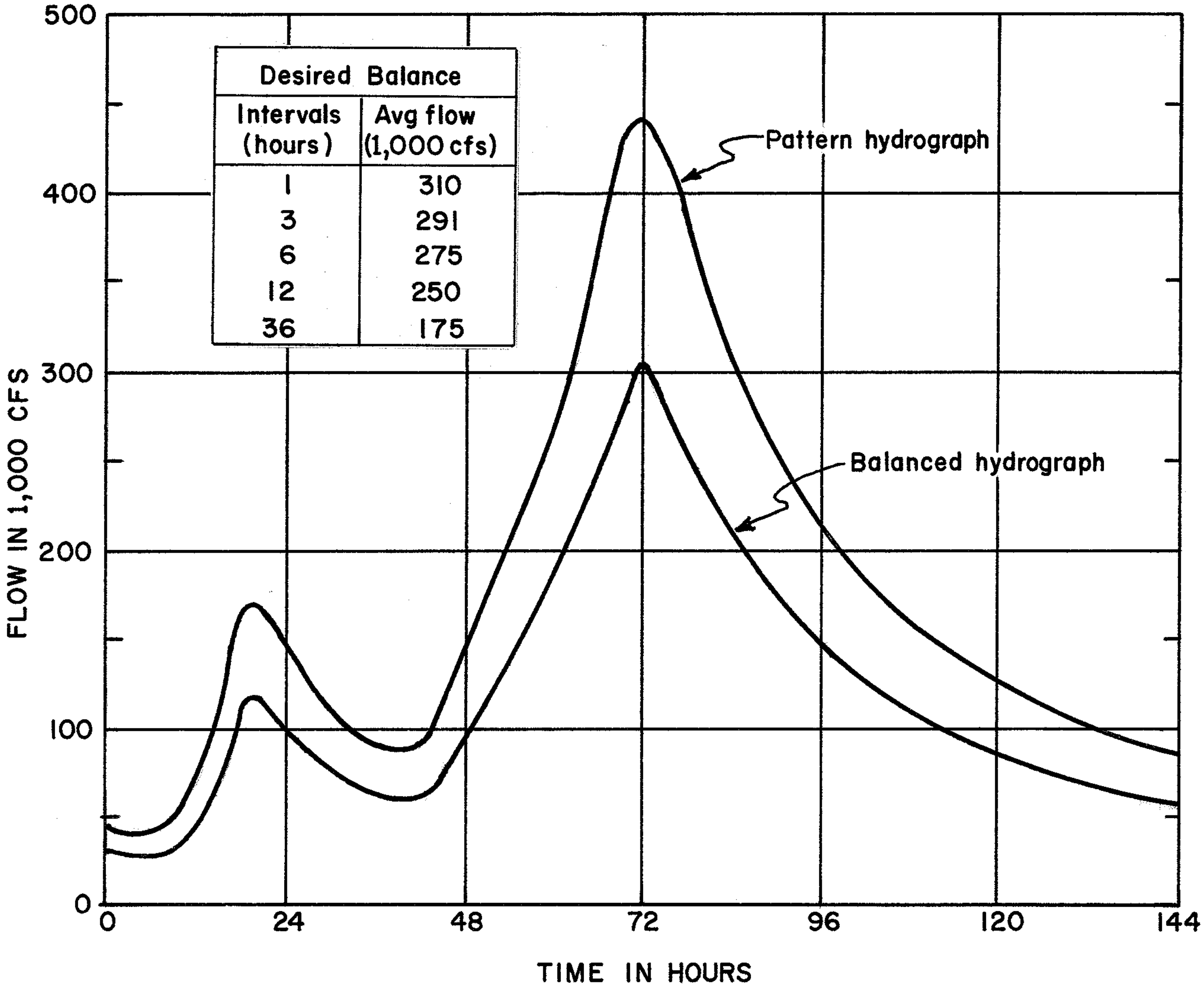


Figure 5.02 Example of balanced hydrograph

Chapter 6

Stream System Runoff Frequency Computation

CHAPTER 6. STREAM SYSTEM RUNOFF FREQUENCY COMPUTATION

Section 6.01. Need and basic concept

In evaluating a plan of water resource project development or operation, it is ordinarily necessary to establish peak flow frequency relations for conditions without the project (non-project conditions) and conditions with the project (project conditions). When conditions without the project are essentially the same as conditions that prevailed when records of flood flows were taken, a frequency curve of non-project conditions can be obtained at each record location from recorded data as described in Volume 3. Otherwise the frequency curve of recorded peak flows will represent a third condition that is not directly used in project evaluations.

In order to evaluate the effects of reservoir, channel and other improvements, at or upstream of any location, on flows at that location, flood hydrographs must be routed through the improvements and down to the location of interest, which is usually a damage index point. In order that alternative project development or operation plans be comparably evaluated, hydrographs should be balanced in time, as discussed in Section 5.01, as well as balanced in space, so that projects on some streams will be appropriately evaluated in relation to those on other streams insofar as effects at a common downstream point are concerned. This can be accomplished by computing runoff from rainfall having the same exceedence frequency at all points in the river system and for all pertinent durations, using loss functions that are of equal severity at all locations. Various magnitudes of balanced floods computed in this manner for non-project and project conditions can be used to derive a frequency curve for project conditions from one of non-project conditions. Routings can also be used to interpolate frequencies for non-project as well as project conditions between locations where records are available. This is particularly useful in establishing an Intermediate Regional Flood, which is a flood having an exceedence frequency of about once in 100 years and is used for flood plain management criteria.

Section 6.02. Flood computation procedure

One flood of intermediate magnitude is computed from rainfall amounts that are consistent in time and area using procedures described in Sections 2.09 and 2.10. Since the rainfall patterns are consistently severe in time and space, runoff should be reasonably balanced throughout the system.

A number of reasonably balanced floods for each sub-area can be obtained by computing runoff from various ratios of the precipitations or simply by multiplying all ordinates of the derived flood by a set of coefficients smaller and larger than 1.0. These can be routed through reservoirs and channels differently for each plan of development. The sub-area computations themselves might be different for each plan if changes such as urbanization or watershed management are involved.

The net result of the flood computations is a set of hydrographs for each location and for each plan of development or operation. The respective hydrographs for each plan have the same exceedence frequency, because the floods are balanced and all larger floods in one plan will be equal to or larger than the corresponding flood in any other plan. Thus, the frequency with which the flood computed from a given ratio for one plan is equaled or exceeded is also the frequency with which the flood computed from the same ratio for another plan is equaled or exceeded.

Section 6.03. Flood frequency designation

The exceedence frequency of a flood computed from rainfall and snowmelt is difficult to determine from exceedence frequencies of the causative factors because of the great variations that ordinarily occur in loss and other runoff functions. Accordingly, the exceedence frequency of floods computed as described in the preceding section is best determined from runoff records at locations where sufficient records exist, using techniques described in Volume 3. Then the exceedence frequency of each flood computed for the plan corresponding to conditions that prevailed during the period of streamflow records can be directly determined. As stated above, exceedence frequencies at the same location for all other plans will be the same. Frequencies of floods for locations other than where records exist can be interpolated in relation to the size of the contributing drainage area.

Section 6.04. Summary of procedure

Computation of peak flow frequencies for various plans of development or operation at damage locations throughout a river basin is accomplished as follows:

a. Derive peak flow frequency curves for all locations where runoff data exist using procedures described in Volume 3.

b. Establish a storm rainfall pattern for point rainfall of equal exceedence frequency for all locations and pertinent durations. The exceedence frequency should be representative of the frequency of damaging floods. Background information for this computation is contained in Section 2.06.

c. Using representative unit hydrographs and loss rates, compute the flood runoff from this rainfall at all locations for conditions under which runoff was recorded, using techniques described in Volume 4.

d. Derive runoff from each sub-area for a number of ratios of the precipitation pattern, or simply take a number of ratios of runoff computed for each sub-area and route resulting hydrographs through the basin, using procedures described earlier in this Volume. Each ratio should be constant for the entire basin, and ratios should be selected to represent the entire range of damaging floods at all locations.

e. Using the runoff frequency curves derived in step a, determine the exceedence frequency for each ratio at each location from the computed peak flow.

f. For each other plan, derive floods using the same rainfall pattern and the same ratios, and assign the corresponding exceedence frequencies to the new computed peak flows. These can then be used to establish frequency curves of modified flows for each plan and location.

Section 6.05. Use of electronic computers

Computations described in the individual steps of Section 6.04 can be done using any number of existing computer programs. The entire procedure, except for runoff-frequency-curve derivations, has been programmed as a unified operation in the computer program described in Appendix 1 of Volume 1.

Chapter 7

Stream System Design Flood Computation

CHAPTER 7. STREAM SYSTEM DESIGN FLOOD COMPUTATION

Section 7.01. Need and general concepts

As discussed in Section 2.06, average storm rainfall corresponding to a specified exceedence frequency decreases as the size of the tributary area increases. Likewise, as discussed in Sections 3.02 and 4.03, standard project and probable maximum precipitation amounts decrease as the size of the tributary area increases. Thus, if design rainfloods computed for various tributaries of a stream system are routed and combined at a downstream point, the resulting flood would represent a higher level of severity than the severity of the components, because the rainfall is not reduced for increased area size. One way to obtain a comparable flood downstream is to recompute all of the upstream flood hydrographs for each successive point downstream, using design rainfall amounts for the larger area tributary to the downstream point. This would be satisfactory, but if a large number of successive downstream points require design values, there would be a great deal of repetition in the computations. More important, probably, is the need to continuously distinguish between the design flood for a given location and the smaller floods at that location which contribute to the design floods at downstream locations.

There is no simple technique for performing the comprehensive design flood hydrograph computations for a stream system or a storm drain system, but the technique described herein provides a systematic approach that is easily computerized. It consists essentially of computing 4 or 5 base floods from rainfall of uniform severity over the entire river basin. The rainfall severity would range from lowest amounts representing design rainfall for the largest areas of interest to highest amounts representing design rainfall for the smallest areas of interest. The size of area corresponding to each of the 4 or 5 base-storm rainfall amounts is related directly to that amount.

Four or five base floods are then computed for every pertinent location in the river system, and the design flood for each location is then interpolated on the basis of drainage area size between the base-flood hydrographs that represent drainage-area sizes nearest the drainage area size of that location.

Section 7.02. Base-storm rainfall patterns

If normal storm rainfall intensity or rainfall potential does not vary appreciably over a river basin, depth-area-duration relationships such as that illustrated in figure 2.01 would suffice as a basis for obtaining rainfall patterns that are balanced in time for each of the 4 or 5 selected area sizes. If normal storm rainfall intensity varies within the basin, the depth-area-duration curves should be expressed as a ratio to a base areal pattern of rainfall intensity, as discussed in Section 2.05. The depth-area-duration relationship and time pattern of storm rainfall are derived for standard project rainfall, probable maximum rainfall or rainfall of any specified frequency as discussed in the previous chapters on those subject areas.

It is usual that rainfall depth-duration relationships change with area size. In small areas, very high short-time intensities can occur, but these do not extend over large areas simultaneously. There is less tendency for this disparity in long-time intensities. Accordingly, rainfall time patterns of the same degree of severity usually have higher proportions of the rain in short periods for small areas than for large areas. As a consequence of this, the time distribution of total rainfall should ordinarily be different for the 4 or 5 base storms, with the larger storm-total amounts (representing rain over the smaller areas) having a greater concentration of rainfall in short periods. This would be a direct result of using the derived depth-duration relations for each selected area size.

Section 7.03. Rainfall-runoff relationships

Factors used in deriving runoff from rainfall should be selected in accordance with guides given in the chapters corresponding to the type of design flood being developed (Chapters 3 through 5).

Section 7.04. Flood interpolation

Runoff computations for stream system design floods proceed from an upstream to a downstream direction. At each location where the 4 or 5 base floods are computed, two are selected whose precipitation amounts correspond to drainage-area sizes nearest to the drainage area at that location, one larger and one smaller. A direct interpolation on the basis of the logarithms of the drainage areas is made between these two base flood hydrographs in order to obtain the design hydrograph for that location.

It should be noted particularly that the sum of design floods on streams immediately above a confluence would be larger than the design flood immediately below the confluence. Of course, the sum could not be considered as a design flood, since the maximum precipitation has been centered on each upstream component and it could only physically center over one of the components at any one time. When the runoff characteristics differ greatly for the different tributaries, it is possible that the interpolation process would cause the interpolated flood below the confluence to be smaller in some respect than one of the tributary floods, which is a physical impossibility. Accordingly, it is advisable to check peak flows and volumes for various durations of the hydrograph below the confluence with those on each tributary, and, if any one tributary value is greater, the downstream hydrograph should be adjusted to equal the tributary hydrograph with regard to that value.

Section 7.05. Summary of procedure

Stream system design rainflood computations can advantageously be accomplished in the following steps:

a. Develop a design storm rainfall depth-area-duration relationship in accordance with procedures outlined in Chapter 3, 4, or 5, depending on the nature of the design storm.

b. Select 4 or 5 area sizes that cover the range of sizes of tributary areas above the various design locations. For each of these area sizes, develop a time pattern of rainfall in accordance with procedures outlined for the particular type of design storm (Chapter 3, 4, or 5).

c. For each sub-area select loss rates, unit hydrograph coefficients and base flow appropriate for the type of design flood, as described in Chapters 3, 4, and 5. Determine reservoir and channel characteristics and routing coefficients for all routing operations.

d. Progressing in an upstream to downstream direction on all tributaries and the main stream, compute runoff hydrographs for each of the 4 or 5 base floods.

e. At each location for which a design flood is required, interpolate on the basis of drainage area sizes between the two base floods corresponding to area sizes nearest that of the design location in order to obtain the design flood hydrograph for that location.

f. Check consistency of hydrographs at combining points and make any adjustments required.

Section 7.06. Use of electronic computers

Procedures described in this chapter have been programmed for automatic stream system computation and routing and for flood interpolation. Routines are contained in Appendix 1 to Volume 1.

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Appendix 1

Unit Hydrograph And Hydrograph Computation

FILE

UNIT GRAPH AND HYDROGRAPH COMPUTATION

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L228

JULY 1966

U. S. ARMY ENGINEER DISTRICT
650 CAPITOL MALL
SACRAMENTO, CALIFORNIA

FACE

UNIT GRAPH AND HYDROGRAPH COMPUTATION

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L228

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EXHIBITS

UNIT GRAPH AND HYDROGRAPH COMPUTATION

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L228

1. ORIGIN OF PROGRAM

This program was prepared in the Hydrologic Engineering Center, Corps of Engineers, principally by Leo R. Beard. Up-to-date information and copies of source statement cards for various types of computers can be obtained from the Center upon request by Government and cooperating organizations.

2. PURPOSE OF PROGRAM

a. This program written in Fortran II computes a unit graph, unless unit graph is supplied, given unit rain duration, size of drainage area, time of concentration (TC) and Clark's routing constant, R. Time-area curve should be supplied, if available. If desired, TC and R will be modified so that the unit hydrograph will attain specified values of Snyder's C_p and t_p, each within 1 1 percent.

b. Program will compute standard project rain amounts from EB 52-8 criteria, probable maximum precipitation amounts from HMS 33 criteria, or accept rain amounts in inches or as ratios of a specified storm total. SPS or PMS amounts will be multiplied by a specific basin shape factor or by the "Hop Brook" shape factor, if a shape factor is not specified (can be 1.00). Use of this "shape factor" as an artificial coefficient can permit rapid computation of floods from a number of ratios of given storm rain.

c. Program will compute rain excess by use of initial loss and uniform loss rate or from loss as a function of some power of rain intensity, decreasing exponentially with accumulated loss.

d. Using the computed or supplied unit graph, program will compute flood runoff for any number of basins and any number of storms for each basin. Time, rain, loss, excess, recession and total flow are printed in convenient format. Also, cards are punched for input to the basin routing program (23-J2-L232).

e. A listing of the source program and test input and output are given at the end of this report.

f. An interactive version of this program (UHCOMP) is available for MS DOS microcomputers (PC) and the Harris minicomputer.

3. DESCRIPTION IF EQUIPMENT

a. This program was prepared for use in the IBM 1620 computer with 40,000 digit, variable word length memory, card input and output, and is usable in the GE 225 and RCA 301 computer having comparable memory, if any necessary input and output statement changes are made.

b. An executable interactive version of this program for the PC is available from the Hydrologic Engineering Center.

4. METHODS OF COMPUTATION

a. The Clark unit graph is computed by linear interpolation of time-area ordinates, after they are converted to cfs, in order to obtain ordinates at the end of each TR interval. These, with the exception of reservoir area at the concentration point, are routed through basin storage by use of the standard Clark formulas:

$$O_2 = C1(I_2) + C2\ (O_1) \qquad (1)$$

$$C1 = TRHR/(R+.5TRHR) \qquad (2)$$

$$C2 = 1-C1 \qquad (3)$$

The unit graph is terminated when its flow recedes to 1 percent of its peak flow, except that not more than 60 ordinates are computed. The volume of the truncated unit graph is printed out along with the volume of 1 inch of excess for comparison. The unit hydrograph thus computed includes return base flow, so associated losses represent permanent losses, and only recession from antecedent runoff is added to flows computed from the unit hydrograph and rain excess amounts.

b. Loss rates can be computed in the usual manner of satisfying a given initial loss and then applying a uniform loss rate for the remainder of the storm period. As an alternative procedure, the program can compute losses in the manner described in the September 1958 Sacramento District report, "Standard Project Rainflood Criteria," where loss coefficient AK is expressed as decreasing with accumulated loss and used in the following formula:

$$ALOSS = AK(RAIN)^E \qquad (4)$$

where ALOSS and RAIN are expressed in inches per hour and the exponent, E, can vary from zero (loss independent of intensity) to one (loss directly proportional to intensity of rain). The coefficient, AK, is expressed as an exponential depletion function, with additional initial increment if desired. It is entered as BASEL at the start of the storm, and the exponential component is defined by the following equations:

$$AK = BASEL/(RTIOL)^{(ACUML/10)} \qquad (5)$$

$$ACUML = \Sigma(ALOSS\text{-}RCVRY) \qquad (6)$$

The initial increment (TEMP) is defined as:

$$TEMP = DLTAK\ (1-(ACUML/DLTAL)^2) \qquad (7)$$

Terms are defined at the end of the text and illustrated in exhibit 5.

c. In applying EB 52-8 and HMS 33 criteria, 24-hour rain amounts and 6-hour amounts within each 24-hour period are arranged in the following order, numbers representing decreasing rain amounts:

4, 2, 1, 3

For storms shorter than 96 hours, 24-hour amounts are arranged

2, 1, 3

2, 1

or 1

The largest 6-hour amount for each day is subdivided in accordance with Plate 11 of 52-8 for both SPS and PMS application. The transposition coefficient supplied by the program only if one is not specified is computed by the following equation designed to fit the "Hop Brook" formula:

$$TRSPC = 1-.3008/(DA)^{.17718} \qquad (8)$$

d. Most runoff hydrographs show an approximately uniform exponential recession rate after flows recede below a given value and rain has stopped. This portion of the runoff cannot satisfactorily be computed by unit hydrograph methods using the same rainfall excess that causes the main flood. This program computes the recession below a specified rate (QRCSN) by use of the following formula, but accepts a higher flow computed by unit hydrograph.

$$Q_2 = Q_1/(RTIOR)^{.1} \qquad (9)$$

5. INPUT

a. Input is summarized at the end of the text. Items A through F are basin or job characteristics and given only once per basin. Remaining items are given for each storm.

b. All data are entered consecutively on each card, using 8 columns (digits, including decimal point, if used) per variable and 10 variables per card unless fewer variables are called for, except that the first column on each card is reserved for identification and not read by computer. Thus, the first field on every card is restricted to 7 columns.

6. OUTPUT

a. All input data except time-area table.

b. Tabulation of time, rain, loss, excess, unit graph, recession and total flow, with totals of all columns.

c. When CP and TP specified, successive approximations of these and associated values of Clark's TC and R.

d. Hydrograph specification card and punched tabulation for direct input to basin routing program.

7. OPERATING INSTRUCTIONS

Standard Fortran II operating instructions. No sense switches used.

8. DEFINITION OF TERMS

Terms used in this program are defined in Exhibit 3.

9. EXAMPLES

Examples illustrating various methods of using this program are shown on Exhibits 1 and 2.

10. PROPOSED FUTURE DEVELOPMENT

a. The content of this program has been limited by the 40-K memory capacity of the IBM 1620 computer. However, it is anticipated that additions and improvements will be made from time to time. A modified version of this program containing generalized SPS criteria for the Southern California area has been made for the Los Angeles District.

b. It is requested that any user of this program who finds an inadequacy, desirable addition or modification notify the Hydrologic Engineering Center.

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A                    SAMPLE HYDROGRAPH COMPUTATION
A                 UNIT GRAPH AND HYDROGRAPH COMPUTATION
A                        PROGRAM 23-J2-L228
B      1       2
C    100      60       6       6      .8       6
D
G      STANDARD PROJECT FLOOD
H             .2     1.0                     14.0                    100
G      PROBABLE MAX. FLOOD
H             .1      .5                             29.5            100
I     89     100     107     118
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EXHIBIT 1

1

SAMPLE HYDROGRAPH COMPUTATION
UNIT GRAPH AND HYDROGRAPH COMPUTATION
PROGRAM 23-J2-L228

ISTA	NHT	NUHGQ	NCLRK	IPNCH	QRCSN	EXTA	RTIMP
1	2	0	0	0	0.	1.50	0.000

DA	TR	TP	CP	TC	RTIOR	RTIOL	RCVRY	E	RCLRK
100.00	60.00	6.00	.800	6.00	1.00	0.00	0.00	0.00	6.00
		5.31	.558	6.77					4.18
		5.69	.698	7.12					3.65
		5.87	.745	7.28					3.41
		5.89	.763	7.40					3.25
		5.92	.772	7.50					3.14
		5.93	.779	7.58					3.06
		5.95	.783	7.58					3.00
		5.93	.788	7.67					2.95
		5.95	.789	7.67					2.92
		5.94	.792	7.67					2.92

UNGR NQ= 22 TR= 60.00 MINS SUMQU= 64358. VOL= 64500.

1

STANDARD PROJECT FLOOD

NP	BASEL	DELTAL	STARTQ	STORM	SPFE	PMF	TRSPC	TRSDA
96	.20	1.00	0.	0.00	14.00	0.00	.867	100.00

HR	MIN	RAIN	LOSS	EXCESS	UNIT HG	RECSN	FLOW
1	0	0.00	0.00	0.00	627.	0.	0.
2	0	0.00	0.00	0.00	2219.	0.	0.
3	0	0.00	0.00	0.00	4204.	0.	0.
4	0	0.00	0.00	0.00	6214.	0.	0.
5	0	0.00	0.00	0.00	7825.	0.	0.
6	0	0.00	0.00	0.00	8597.	0.	0.
7	0	.01	.01	0.00	8479.	0.	0.
8	0	.01	.01	0.00	7356.	0.	0.
9	0	.01	.01	0.00	5550.	0.	0.
10	0	.01	.01	0.00	3928.	0.	0.
11	0	.01	.01	0.00	2780.	0.	0.
12	0	.01	.01	0.00	1967.	0.	0.
13	0	.03	.03	0.00	1392.	0.	0.

EXHIBIT 2

E72

14	0	.03	.03	0.00	985.	0.	0.
15	0	.04	.04	0.00	697.	0.	0.
16	0	.11	.11	0.00	493.	0.	0.
17	0	.04	.04	0.00	349.	0.	0.
18	0	.03	.03	0.00	247.	0.	0.
19	0	.01	.01	0.00	175.	0.	0.
20	0	.01	.01	0.00	124.	0.	0.
21	0	.01	.01	0.00	88.	0.	0.
22	0	.01	.01	0.00	62.	0.	0.
23	0	.01	.01	0.00		0.	0.
24	0	.01	.01	0.00		0.	0.
25	0	.02	.02	0.00		0.	0.
26	0	.02	.02	0.00		0.	0.
27	0	.02	.02	0.00		0.	0.
28	0	.02	.02	0.00		0.	0.
29	0	.02	.02	0.00		0.	0.
30	0	.02	.02	0.00		0.	0.
31	0	.05	.05	0.00		0.	0.
32	0	.05	.05	0.00		0.	0.
33	0	.05	.05	0.00		0.	0.
34	0	.05	.05	0.00		0.	0.
35	0	.05	.05	0.00		0.	0.
36	0	.05	.05	0.00		0.	0.
37	0	.13	.13	0.00		0.	0.
38	0	.15	.15	0.00		0.	0.
39	0	.19	.19	0.00		0.	0.
40	0	.48	.20	.28		0.	175.
41	0	.18	.18	0.00		0.	621.
42	0	.14	.14	0.00		0.	1177.
43	0	.03	.03	0.00		0.	1739.
44	0	.03	.03	0.00		0.	2191.
45	0	.03	.03	0.00		0.	2407.
46	0	.03	.03	0.00		0.	2374.
47	0	.03	.03	0.00		0.	2059.
48	0	.03	.03	0.00		0.	1554.
49	0	.15	.15	0.00		0.	1099.
50	0	.15	.15	0.00		0.	778.
51	0	.15	.15	0.00		0.	550.
52	0	.15	.15	0.00		0.	389.
53	0	.15	.15	0.00		0.	275.
54	0	.15	.15	0.00		0.	195.

55	0	.35	.20	.15	0.	232.
56	0	.35	.20	.15	0.	524.
57	0	.35	.20	.15	0.	1126.
58	0	.35	.20	.15	0.	2038.
59	0	.35	.20	.15	0.	3198.
60	0	.35	.20	.15	0.	4477.
61	0	.88	.20	.68	0.	6074.
62	0	1.05	.20	.85	0.	8443.
63	0	1.32	.20	1.12	0.	12050.
64	0	3.34	.20	3.14	0.	18514.
65	0	1.23	.20	1.03	0.	28426.
66	0	.97	.20	.77	0.	39932.
67	0	.22	.20	.02	0.	50844.
68	0	.22	.20	.02	0.	58590.
69	0	.22	.20	.02	0.	61261.
70	0	.22	.20	.02	0.	58641.
71	0	.22	.20	.02	0.	51196.
72	0	.22	.20	.02	0.	40846.
73	0	.01	.01	0.00	0.	30634.
74	0	.01	.01	0.00	0.	22266.
75	0	.01	.01	0.00	0.	16033.
76	0	.01	.01	0.00	0.	11557.
77	0	.01	.01	0.00	0.	8316.
78	0	.01	.01	0.00	0.	5959.
79	0	.02	.02	0.00	0.	4244.
80	0	.02	.02	0.00	0.	3004.
81	0	.02	.02	0.00	0.	2119.
82	0	.02	.02	0.00	0.	1494.
83	0	.02	.02	0.00	0.	1028.
84	0	.02	.02	0.00	0.	691.
85	0	.05	.05	0.00	0.	439.
86	0	.06	.06	0.00	0.	173.
87	0	.07	.07	0.00	0.	77.
88	0	.19	.19	0.00	0.	20.
89	0	.07	.07	0.00	0.	13.
90	0	.05	.05	0.00	0.	8.
91	0	.01	.01	0.00	0.	5.
92	0	.01	.01	0.00	0.	3.
93	0	.01	.01	0.00	0.	1.
94	0	.01	.01	0.00	0.	0.
95	0	.01	.01	0.00	0.	0.

EXHIBIT 2

EY2

96	0	.01	.01	0.00		0.	0.
97	0					0.	0.
98	0					0.	0.
99	0					0.	0.
100	0					0.	0.
101	0					0.	0.
102	0					0.	0.
103	0					0.	0.
104	0					0.	0.
105	0					0.	0.
106	0					0.	0.
107	0					0.	0.
108	0					0.	0.
109	0					0.	0.
110	0					0.	0.
111	0					0.	0.
112	0					0.	0.
113	0					0.	0.
114	0					0.	0.
115	0					0.	0.
116	0					0.	0.
117	0					0.	0.
TOTAL		16.11	7.22	8.89	64358.	0.	571950.

1

PROBABLE MAX. FLOOD

PMS PERCENTS 89.00 100.00 107.00 118.00 0.00 0.00

NP	BASEL	DELTAL	STARTQ	STORM	SPFE	PMF	TRSPC	TRSDA
48	.10	.50	0.	0.00	0.00	29.50	.867	100.00

HR	MIN	RAIN	LOSS	EXCESS	UNIT HG	RECSN	FLOW
1	0	.01	.01	0.00	627.	0.	0.
2	0	.01	.01	0.00	2219.	0.	0.
3	0	.01	.01	0.00	4204.	0.	0.
4	0	.01	.01	0.00	6214.	0.	0.
5	0	.01	.01	0.00	7825.	0.	0.
6	0	.01	.01	0.00	8597.	0.	0.
7	0	.05	.05	0.00	8479.	0.	0.

8	0	.05	.05	0.00	7356.	0.	0.
9	0	.05	.05	0.00	5550.	0.	0.
10	0	.05	.05	0.00	3928.	0.	0.
11	0	.05	.05	0.00	2780.	0.	0.
12	0	.05	.05	0.00	1967.	0.	0.
13	0	.23	.17	.05	1392.	0.	31.
14	0	.28	.10	.18	985.	0.	223.
15	0	.35	.10	.25	697.	0.	766.
16	0	.89	.10	.79	493.	0.	2118.
17	0	.33	.10	.23	349.	0.	4458.
18	0	.26	.10	.16	247.	0.	7323.
19	0	.02	.02	0.00	175.	0.	10158.
20	0	.02	.02	0.00	124.	0.	12325.
21	0	.02	.02	0.00	88.	0.	13306.
22	0	.02	.02	0.00	62.	0.	12961.
23	0	.02	.02	0.00		0.	11370.
24	0	.02	.02	0.00		0.	9013.
25	0	.12	.10	.02		0.	6687.
26	0	.12	.10	.02		0.	4835.
27	0	.12	.10	.02		0.	3522.
28	0	.12	.10	.02		0.	2658.
29	0	.12	.10	.02		0.	2114.
30	0	.12	.10	.02		0.	1791.
31	0	.47	.10	.37		0.	1830.
32	0	.47	.10	.37		0.	2506.
33	0	.47	.10	.37		0.	3913.
34	0	.47	.10	.37		0.	6043.
35	0	.47	.10	.37		0.	8747.
36	0	.47	.10	.37		0.	11726.
37	0	2.28	.10	2.18		0.	15804.
38	0	2.73	.10	2.63		0.	22636.
39	0	3.41	.10	3.31		0.	33608.
40	0	8.65	.10	8.55		0.	52919.
41	0	3.19	.10	3.09		0.	81917.
42	0	2.50	.10	2.40		0.	115380.
43	0	.18	.10	.08		0.	147040.
44	0	.18	.10	.08		0.	169370.
45	0	.18	.10	.08		0.	177020.
46	0	.18	.10	.08		0.	169470.
47	0	.18	.10	.08		0.	148330.
48	0	.18	.10	.08		0.	119020.

EXHIBIT 2

49	0					0.	89893.
50	0					0.	65731.
51	0					0.	47618.
52	0					0.	34542.
53	0					0.	24997.
54	0					0.	18000.
55	0					0.	12856.
56	0					0.	9111.
57	0					0.	6433.
58	0					0.	4538.
59	0					0.	3118.
60	0					0.	2093.
61	0					0.	1335.
62	0					0.	569.
63	0					0.	266.
64	0					0.	83.
65	0					0.	55.
66	0					0.	35.
67	0					0.	21.
68	0					0.	12.
69	0					0.	4.
TOTAL	30.20	3.56	26.64	64358.		0.	1713200.

6

DEFINITIONS - 23-J2-L228

ACUML	- Accumulated loss in inches less NP(RCVRY)
AI	- Conversion of I
AJ	- Conversion of J
AK	- Loss coefficient in formula $L = AK(P)^E$
ALAG	- Snyder's t_p (and intermediate computation constant)
ALOSS	- Loss in inches
BASEL	- Initial value of AK on exponential recession curve (see RTIOL) or uniform loss in inches per TR period if RTIOL is not positive
C1	- Interpolation constant or Clark routing coefficient
C2	- 1-C1
CONST	- Temporary variable
CP	- Snyder's C_p (given)
CPTMP	- Snyder's C_p (computed)
DA	- Drainage area in square miles
DLTAK	- Initial increment of AK above exponential recession curve (see RTIOL)
DLTAL	- Value of ACUML up to which some increment of AK is added to the exponential recession curve value
E	- Exponent for loss computation (see AK)
*EXCES	- Rainfall excess in inches
EXTA	- Exponent for synthetic time-area curve
I	- Various index values, serial number of rain period
IDTHR	- Number of whole hours in TRHR
IDTMN	- Number of minutes left over when IDTHR is subtracted from TRHR
IHR	- Hour at end of any tabulation interval
IMIN	- Minute at end of any tabulation interval
IPNCH	- Positive value causes punch-out for input to routing program 23-J2-L232
ISTA	- Station identification number
ITC	- Time of concentration in TR intervals
J	- Various index values
J1	- Index value
K	- Various index values, temporary fixed-point variable, or index to leave 100 do-loop.
KRAIN	- Dimension limit for RAIN
KUHGQ	- Dimension limit for QUNGR
L	- Iteration index (100 do-loop)
LAG	- Serial number of largest unit graph ordinate
M	- Number of hydrographs already computed
N24HR	- Number of 24-hour periods in storm
NCLRK	- Number of time-area ordinates
NHT	- Number of hydrographs to be computed

* Subscripted variable

NINTV - Number of TR intervals in 6 hours
NP - Number of rain intervals
NPR - Hour serial number
NQ - Number of hydrograph ordinates to be computed
NUHGQ - Number of unit graph ordinates
NX - Number of TR intervals in 1 hour
PMS - Probable maximum storm precipitation in inches
*Q - Flow in cfs or temporary subscripted variable
QB - Recession flow in cfs
QMAX - Maximum ordinate of unit graph in cfs
QRCSN - Flow in cfs below which maximum rate of recession is controlled by RTIOR
*QUNGR - Unit graph ordinate in cfs
R6 - Maximum 6-hour rain in percent of PMF
R12 - " 12 " " " " " "
R24 - " 24 " " " " " "
R48 - " 48 " " " " " "
R72 - " 72 " " " " " "
R96 - " 96 " " " " " "
*R6HR - Successive ratios of 6-hour rain to 24-hour rain
*R24HR - Successive ratios of 24-hour rain to storm total in percent
*RAIN - Rain in inches or temporary value of ratio to storm total
RATR1 - Ratio of recession flow to preceding flow
RCLRK - Clark's R storage routing constant
RCVRY - Decrement in inches applied to ACUML each period to account for loss-rate recovery
RINTV - Reciprocal of NINTV
RNX - Reciprocal of NX
RTIMP - Ratio of contributing area that is impervious
RTIOL - Ratio of AK on exponential recession curve to that at 10 inches more ACUML. Thus, AK component from exponential recession curve is BASEL divided by the quantity RTIOL raised to the power of ACUML/10.
RTIOR - Ratio of recession flow to that 10 TR units later
RTIOT - Number of time-area intervals per TR interval
SPFE - Standard project storm index for eastern U.S., 200 sq.mi., 24-hr, from EB 52-8
STORM - Basin-mean storm rain in inches
STRTQ - Flow in cfs at start of storm
SUME - Total excess in inches
SUML - Total loss in inches for storm
SUMQ - Total flow in cfs-TR units
SUMQB - Total recession from STRTQ in cfs-TR units

* Subscripted variable

SUMQU - Total unit graph flow in cfs-TR units
SUMR - Total rain in inches
TB - Temporary variable
TC - Time of concentration of drainage basin runoff, hours
TEMP - Temporary constant
TMP - Temporary constant
TP - Snyder's t_p in hours
TR - Tabulation interval and rain interval in minutes
TRHR - TR expressed in hours
TRHR2 - Half of TRHR
TRSDA - Drainage area in square miles on which transposition is based
TRSPC - Storm transposition coefficient
VOL - One inch of runoff from basin expressed in cfs-TR units

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*05074
C    PROGRAM 23-J2-L228  UNIT GRAPH AND HYDROGRAPH COMP JULY 1966
C      LIBRARY SUBR  LOGF USED AT STAT NO 620
       DIMENSION QUNGR(40),RAIN(120),Q(179),EXCES(120),R24HR(4),R6HR(4)
C                              STATION DATA                                  129
       KUHGQ=40
       KRAIN=120
    10 M = 0
C                         THREE TITLE CARDS
       READ 50,(Q(J),J=1,120)
       READ 20,ISTA,NHT,NUHGQ,NCLRK,IPNCH
       READ 30,DA,TR,RCLRK,TC,CP,TP,RTIOL,RCVRY,E,RTIMP,QRCSN,RTIOR,EXTA
       IF(TR) 15,15,11
    11 IF (EXTA) 9,9,12
     9 EXTA=1.50
    12 IF (KUHGQ-NUHGQ) 13,40,40
    13 PRINT 14
    14 FORMAT (19H DIMENSION EXCEEDED)
    15 STOP                                                                   1066
    20 FORMAT (1X,I7,9I8)
    30 FORMAT (1X,F7.0,9F8.0)
    50 FORMAT (1X,A1,9A2,15A2,15A2)
    40 PRINT 60
    60 FORMAT(1H1)
       PRINT 50,(Q(J),J=1,120)
C           AFTER FIRST FLOOD, SKIP UNIT HYDRO COMP
       IF (RTIOR) 80,80,90
    80 RTIOR = 1.
    90 RATR1 = 1./RTIOR**.1
       PRINT 100,ISTA,NHT,NUHGQ,NCLRK,IPNCH,QRCSN,EXTA,RTIMP
   100 FORMAT(/63H    ISTA      NHT  NUHGQ   NCLRK  IPNCH  QRCSN     EXTA
      1   RTIMP/4I8,I6,F11.0,F7.2,F7.3/)
       PRINT 110,DA,TR,TP,CP,TC,RTIOR,RTIOL,RCVRY,E,RCLRK
   110 FORMAT(80H      DA       TR       TP       CP       TC   RTIOR   RTIOL
      1   RCVRY        E   RCLRK/3F8.2,F8.3,6F8.2)
       TRHR = TR/60.
       TRHR2=TRHR*.5
       VOL=DA*645./TRHR
       CONST=(.5/.5**EXTA)*VOL
       RAIN(1) = 0.
       IF (NUHGQ) 130,130,570
```

EXHIBIT 4

EX-4

```
  130 IF (NCLRK) 160,160,140
C                                   CLARK UNIT HYDROGRAPH
C     READ TIME-AREA ORDINATES (DUMMY VARIABLE)
  140 READ 30,(Q(I),I=1,NCLRK)
      TEMP=VOL/Q(NCLRK)
      DO 150 I=1,NCLRK
C                                             CONVERT TO CFS (DUMMY VARIABLE)
  150 RAIN (I)    = Q(I)*TEMP
      RAIN (NCLRK+1)=RAIN (NCLRK)
  160 DO 520 L=1,10
      IF (NCLRK) 190,190,170
C                                INTERPOLATE TIME-AREA CURVE
C                         ITC IS TC IN TR UNITS, TRUNCATED
  170 ITC = TC/TRHR
      TB = NCLRK-1
C     NUMBER OF TIME-AREA INTERVALS PER UNIT GRAPH INTERVAL
      RTIOT = TB/TC*TRHR
      DO 180 I=1,ITC
      AI = I
C     AJ IS UNIT HGR TIME IN TERMS OF TIME-AREA TABULATION INTERVAL
      AJ = AI*RTIOT+1.
C     +1. IS ADDED SINCE FIRST AREA IS AT ZERO TIME
      J = AJ
      TEMP = J
C                                   INTERPOLATION CONSTANTS
      C1 = AJ-TEMP
      C2 = 1.-C1
  180 Q(I) = RAIN (J)*C2 + RAIN (J+1)*C1
      ITC = ITC+1
      Q(ITC)  = VOL
      GO TO 230
C     COMPUTED TIME-AREA CURVE
  190 TMP = TC/TRHR
      ITC=TMP
      DO 220 I=1,ITC
      AI = I
      IF(AI-TMP*.5) 200,200,210
  200 Q(I) = CONST*(AI/TMP)**EXTA
      GO TO 220
  210 Q(I)=VOL-CONST*(1.-AI/TMP)**EXTA
  220 CONTINUE
```

2

```
      ITC=ITC+1
      Q(ITC)=VOL
C                                ROUTING CONSTANTS AND AVERAGING
  230 QUNGR(1)=Q(1)
      DO 240 I=2,ITC
  240 QUNGR(I) = Q(I)-Q(I-1)
      C1 = TRHR/(RCLRK + TRHR2)
      C2 = 1.-C1
C     C1 CHANGED TO OBTAIN HALF FLOWS TO AVOID DIVIDING IN 280 + 1
      C1=C1*.5
C                           TMP IS RESERVOIR AREA CONTRIBUTION
      TMP=RAIN (1)
      Q(1)=(QUNGR(1)-TMP)*C1
      QMAX = Q(1)
      QUNGR(1)=Q(1)+TMP
      SUMQU=QUNGR(1)
      DO 270 I=2,60
      IF (I-ITC) 250,250,260
  250 Q(I) = QUNGR(I)*C1 + Q(I-1)*C2
      GO TO 270
  260 Q(I) = Q(I-1)*C2
  270 CONTINUE
  280 DO 320 I=2,60
      K= Q(I)+Q(I-1) +.5
      QUNGR(I)=K
      SUMQU=SUMQU+QUNGR(I)
      IF(QUNGR(I)-QMAX) 290,310,310
  290 IF(QUNGR(I)-.01*QMAX) 300,320,320
  300 NUHGQ = I
      GO TO 340
  310 QMAX = QUNGR(I)
      LAG=I
  320 CONTINUE
      NUHGQ = 60
      PRINT 330
  330 FORMAT(13H UH TRUNCATED)
C     ADJUSTMENT TO SNYDER COEFFICIENTS, IF SPECIFIED
  340 ALAG=LAG
      IF(LAG-1) 350,350,360
  350 ALAG=    1.5-(QUNGR(1)-QUNGR(2))/QUNGR(1)*.5
      GO TO 390
```


EXHIBIT 4

```
360 IF(QUNGR(LAG-1)-QUNGR(LAG+1)) 370,390,380
370 ALAG = ALAG +.5-(QUNGR(LAG)-QUNGR(LAG+1))/(QUNGR(LAG)-QUNGR(LAG-1)
   1)*.5
    GO TO 390
380 ALAG = ALAG-.5+(QUNGR(LAG)-QUNGR(LAG-1))/(QUNGR(LAG)-QUNGR(LAG+1))
   1*.5
390 ALAG=(ALAG-.75)*TRHR*1.048
    CPTMP = QMAX*ALAG/(645.*DA)
    IF (CP) 480,480,400
400 TEMP = CP/CPTMP
    K=0
    IF (TEMP-1.01) 410,410,420
410 IF (TEMP-.99) 420,440,440
420 RCLRK = RCLRK/TEMP
    K=1
    IF (RCLRK- TRHR2) 430,440,440
430 RCLRK= TRHR2
440 TEMP = TP/ALAG
    IF (TEMP-1.01) 450,450,460
450 IF (TEMP-.99) 460,490,490
460 TC = TC*TEMP
    K = 1
    IF (TC-TRHR2) 470,490,490
470 TC = TRHR2
480 K = 0
490 PRINT 500,ALAG,CPTMP,TC,RCLRK
500 FORMAT (F24.2,F8.3,F8.2,F40.2)
510 IF (K) 540,540,520
520 CONTINUE
    PRINT 530
530 FORMAT(32H CP OR TP POSSIBLY NOT SATISFIED)
540 PRINT 550,NUHGQ,TR,SUMQU,VOL
    IF (NHT) 560,560,580
550 FORMAT(9H UNGR NQ=I3,5H  TR=F8.2,13H MINS  SUMQU=F10.0,6H  VOL=F10
   1.0)
560 PRINT 30,(QUNGR(L),L=1,NUHGQ)
    GO TO 10
570 READ 30,(QUNGR(I),I=1,NUHGQ)
C                          RAINFALL COMPUTATION
580 READ 50, (Q(J),J=1,40)
    PRINT 60
```

```
      PRINT 50, (Q(J),J=1,40)
      READ 30,TMP,BASEL,DLTAL,STRTQ,STORM,SPFE,PMS,TRSPC,TRSDA
      NP=TMP
      IF (KRAIN-NP) 13,585,585
  585 M = M+1
      IF (SPFE+PMS) 880,880,590
  590 IF (TRSPC) 600,600,610
  600 TRSPC = 1. - .3008/(TRSDA**.17718)
  610 IF (SPFE) 780,780,620
C                   STD PROJ 24-HOUR PERCENTAGES
  620 R24HR(3) = 182.15-14.3537*LOGF(TRSDA+80.)
      R24HR(1)= 03.5
      R24HR(2) = 15.5
      R24HR(4) = 06.
      N24HR = 4
C                      6-HOUR RATIOS OF 24-HR AMOUNTS
      R6HR(3) =13.42/(SPFE+11.)**.93
      R6HR(2) = .055*((SPFE-6.)**.51)
      R6HR(4) = (1.-R6HR(3)-R6HR(2))*.5 + .0165
      R6HR(1) = R6HR(4) - .033
      TEMP = SPFE*TRSPC*.01
  630 NINTV = 360./TR
      NP = NINTV*4*N24HR
      IF (KRAIN-NP) 13,635,635
  635 RINTV = TR/360.
      I = 0
      TMP = NINTV
      K=360.5-TMP*TR
      IF (K) 640,660,640
  640 PRINT 650
  650 FORMAT(22H UNACCEPTABLE INTERVAL)
      GO TO 1440
C                     SUBDIVISION OF MAX 6-HR RAIN EACH DAY
  660 IF (NINTV-2)  670,680,690
  670 Q(1)=1.
      GO TO 720
  680 Q(1)=.33
      Q(2)=.67
      GO TO 720
  690 IF (NINTV-6) 700,710,710
  700 Q (1)=.26
```

```
      Q (2)=.53
      Q (3)=.21
      GO TO 720
  710 Q (1)=.10
      Q (2)=.12
      Q (3)=.15
      Q (4)=.38
      Q (5)=.14
      Q (6)=.11
      NX=NINTV/6
      RNX=NX
      RNX=1./RNX
  720 DO 770 J =1,N24HR
      DO 770 K =1,4
      TMP = TEMP*R24HR(J)*R6HR(K)
      DO 770 L = 1,NINTV
      I=I+1
      IF (K -3) 730,740,730
  730 RAIN(I) = TMP*RINTV
      GO TO 770
  740 IF (NINTV-6) 750,750,760
  750 RAIN(I)=TMP* Q(L)
      GO TO 770
  760 NPR=(L-1)/NX+1
      RAIN(I)=TMP* Q(NPR)*RNX
  770 CONTINUE
      GO TO 910
C     COMPUTE DIFFS OF MAX 6,12,24,48,72,96-HR RAIN, PERCENT
  780 READ 30,R6,R12,R24,R48,R72,R96
      PRINT 790,R6,R12,R24,R48,R72,R96
  790 FORMAT(13H PMS PERCENTS6F8.2)
      IF (R96) 810,810,800
  800 N24HR = 4
      R24HR(3) = R24
      R24HR(2) =(R48-R24)
      R24HR(4) =(R72-R48)
      R24HR(1) =(R96-R72)
      GO TO 870
  810 IF (R72) 830,830,820
  820 N24HR = 3
      R24HR(3) =(R72-R48)
```

6

574

```
      GO TO 850
  830 IF (R48) 860,860,840
  840 N24HR = 2
  850 R24HR(2) = R24
      R24HR(1) =(R48-R24)
      GO TO 870
  860 N24HR = 1
      R24HR(1) = R24
  870 TEMP = PMS*TRSPC*.01
      R6HR(3) = R6/R24
      R6HR(2) =(R12-R6)/R24
      R6HR(1) = (R24-R12)*.4/R24
      R6HR(4) = R6HR(1)*1.5
      GO TO 630
  880 READ 30,(RAIN(J), J = 1,NP)
      IF (STORM) 910,910,890
  890 DO 900 J=1,NP
  900 RAIN(J) = STORM * RAIN(J)
  910 ACUML = 0.
      IF (NP-200) 940,940,920
  920 PRINT 930
  930 FORMAT(13H NP TOO LARGE)
      GO TO 1440
  940 IHR = 0
      IMIN = 0
      IDTHR = TRHR
      TEMP = IDTHR
      IDTMN = TR - 60. * TEMP
      PRINT 950,NP,BASEL,DLTAL,STRTQ,STORM,SPFE,PMS,TRSPC,TRSDA
  950 FORMAT(/72H       NP   BASEL  DELTAL  STARTQ   STORM    SPFE      PM
     1S   TRSPC   TRSDA/I8,2F8.2,F8.0,3F8.2,F8.3,F8.2)
      SUMQ = 0.
      SUMQU=0.
      SUMR = 0.
      SUMQB=0.
      SUME = 0.
      QB=STRTQ                                                         236
      DLTAK = 0.2*DLTAL
      PRINT 970
  970 FORMAT(/52H  HR MIN  RAIN  LOSS EXCESS  UNIT HG   RECSN    FLOW)
C                       LOSS COMPUTATION
```

```
      NQ = NP + NUHGQ - 1
      DO 1410 J=1,NQ
      IF (J-NP) 980,980,1110
  980 K=RAIN(J)*100.+.5
      RAIN(J) = K
      RAIN(J) = RAIN(J)*.01
      IF (RTIOL) 1040,1040,990
  990 IF (ACUML) 1000,1010,1010
 1000 ACUML=0.
 1010 AK    = BASEL / RTIOL**(ACUML*.1)
      IF (ACUML - DLTAL) 1020,1030,1030
 1020 TMP=1.-ACUML/DLTAL
      TEMP = DLTAK * TMP*TMP
      AK = TEMP + AK
 1030 ALOSS = AK *TRHR * (RAIN(J)/TRHR)**E
      GO TO 1080
 1040 TMP=DLTAL-ACUML
      IF (RAIN(J)-TMP) 1090,1090,1050
 1050 IF (TMP) 1060,1060,1070
 1060 ALOSS = BASEL
      GO TO 1080
 1070 ALOSS=BASEL*(1.-TMP/RAIN(J))+TMP
 1080 IF (RAIN(J)-ALOSS) 1090,1100,1100
 1090 ALOSS = RAIN(J)
 1100 ACUML = ACUML + ALOSS - RCVRY
      ALOSS=ALOSS*(1.-RTIMP)
      EXCES(J) = RAIN(J) - ALOSS
      K=EXCES(J)*100.+.5
      TEMP=K
      EXCES(J)=TEMP*.01
      SUMR = SUMR + RAIN(J)
      SUME = SUME + EXCES(J)
C                          RUNOFF COMPUTATION
 1110 QB=QB*RATR1
      SUMQB=SUMQB+QB
      Q(J) = QB
      IF (J-NP) 1120,1120,1170
 1120 IF (J-NUHGQ) 1130,1130,1150
 1130 DO 1140 I=1,J
      K = J + 1 - I
 1140 Q(J) = Q(J) + EXCES(K) * QUNGR(I)
```

```
      SUMQU=SUMQU+QUNGR(J)
      GO TO 1230
 1150 J1 = J-NUHGQ + 1
      DO 1160 I=J1,J
      K = J + 1 -I
 1160 Q(J) = Q(J) + EXCES(I) * QUNGR(K)
      GO TO 1230
 1170 IF (J-NUHGQ) 1180,1180,1200
 1180 DO 1190 I=1,NP
      K = J+1-I
 1190 Q(J) = Q(J) + EXCES(I) * QUNGR(K)
      SUMQU=SUMQU+QUNGR(J)
      GO TO 1230
 1200 J1 = J - NUHGQ + 1
      IF (J1-NP) 1210,1210,1230
 1210 DO 1220 I=J1,NP
      K = J+1-I
 1220 Q(J) = Q(J) + EXCES(I) * QUNGR(K)
 1230 IF (Q(J)-QRCSN) 1240,1270,1270
 1240 IF (J-1) 1270,1270,1250
 1250 IF (Q(J)-Q(J-1)*RATR1) 1260,1270,1270
 1260 Q(J)=Q(J-1)*RATR1
 1270 SUMQ = SUMQ + Q(J)
      IMIN = IMIN +IDTMN
      IF (IMIN - 60) 1290,1280,1280
 1280 IMIN = IMIN - 60
      IHR = IHR +IDTHR + 1
      GO TO 1300
 1290 IHR = IHR +IDTHR
C                            PRINT OUT
 1300 IF (J-NP) 1310,1310,1360
 1310 IF (J-NUHGQ) 1320,1320,1340
 1320 PRINT 1330,IHR,IMIN,RAIN(J),ALOSS,EXCES(J),QUNGR(J),QB,Q(J)
 1330 FORMAT(2I4,3F6.2,3F9.0)
      GO TO 1410
 1340 PRINT 1350, IHR,IMIN,RAIN(J),ALOSS,EXCES(J),QB,Q(J)
 1350 FORMAT (2I4,3F6.2,F18.0,F9.0)
      GO TO 1410
 1360 IF (J-NUHGQ) 1370,1370,1390
 1370 PRINT 1380,IHR,IMIN,QUNGR(J),QB,Q(J)
 1380 FORMAT(2I4,F27.0,2F9.0)
```

EXHIBIT 4

```
      GO TO 1410
 1390 PRINT 1400,IHR,IMIN,QB,Q(J)
 1400 FORMAT (2I4,F36.0,F9.0)
 1410 CONTINUE
      SUML=SUMR-SUME
      PRINT 1420, SUMR,SUML,SUME,SUMQU,SUMQB,SUMQ
 1420 FORMAT (/6H TOTAL F9.2,2F6.2,3F9.0//)
      IF(IPNCH) 1440,1440,1425
 1425 PUNCH 1435,     ISTA,NQ,TR
 1435 FORMAT (1HC,I7,I8,F8.0)
      PUNCH 1438,     (Q(J),J=1,NQ)
 1438 FORMAT (1HD,F7.0,9F8.0)
 1440 IF (M-NHT) 580,10,10
      END
```

EXHIBIT 5

INPUT DATA# - 23-J2-L228

A. Three output title cards

B. Station data card

1. ISTA - Station identification number, needed only if output to be used in routing program
2. NHT - Number of hydrographs to be computed, excluding unit hydrograph.
3. NUHGQ - Number of unit graph ordinates given. Should be zero if unit hydrograph to be computed
4. NCLRK - Number of time-area ordinates to be read starting with area at zero time. Cannot exceed 21 unless dimension is changed. Can be in any units of area equally spaced in time. If zero, ordinates will be computed using EXTA(D3). Omit if NUHGQ is positive.
5. IPNCH - Positive value calls for punched output for input to routing program 23-J2-L232.

C. Station data card

1. DA - Drainage area in square miles used for unit hydrograph computation
2. TR - Rainfall and runoff interval in minutes
3. RCLRK - Routing constant, R, for Clark unit graph in hours. Needed only if unit graph to be computed.
4. TC - Time of concentration in hours. Needed only if unit graph to be computed
5. CP - Snyder's C_p. Supplied only if unit graph is to be computed and adjusted to conform with given TP and CP
6. TP - Snyder's t_p in hours required if unit graph is to be computed and adjusted to conform with given TP and CP
7. RTIOL - Ratio of AK on exponential recession AK curve to that at 10 more inches of accumulated loss. Zero value causes fixed initial loss (DLTAL) and uniform loss rate (BASEL) to be used (H3, H2).
8. RCVRY - Quantity in inches to be subtracted from accumulated loss each interval to account for loss rate recovery between storms. Should be value used in deriving AK curve. Omit if RTIOL (C7) is zero.
9. E - Exponent of rain, when rain is expressed in inches per hour, in loss computation, Loss = $AK(P)^E$. Omit if RTIOL (C7) is zero.

Data entered on cards in 10 fields of 8 columns each per card, except that column 1 is reserved for identification, leaving 7 columns for first field.

10. RTIMP - Ratio of imperviousness of drainage area.

D. Station data card

1. QRCSN - Flow in cfs below which specified recession will control when flow computed from rain recedes faster. Needed only if hydrograph to be computed, in which case it can be zero, if desired.
2. RTIOR - Ratio of recession flow to that 10 TR units later. Needed if STARTQ or QRCSN is positive (H4, D1).
3. EXTA - Exponent (1 or larger) of time-area curve. Used only when NCLRK (B4) is zero. If not supplied, program will adopt 1.5. Omit if NUHGQ or NCLRK (B3 and B4) is positive.

E. Time-area ordinates. Number must correspond with NCLRK (B4), so this item is omitted if NCLRK is zero. Supplied only once for each station for all storms.

Q - Accumulated areas in any units at equal travel time intervals from station, starting with value at zero time. This first value would be zero unless the station is a reservoir in which case it would be the reservoir area. Program assumes no rainfall losses in reservoir area.

F. Given unit graph ordinates

QUNGR - Consecutive ordinates in cfs of unit graph starting with that at end of first period. Number must correspond with NUHGQ (B3), so this item is omitted when NUHGQ is zero. Supplied only once for each station for all storms.

G*. One flood title card

H*. Flood data card

1. NP - Number of rain periods to be supplied. Will be ignored if SPFE or PMS are positive (H6, H7).
2. BASEL - If RTIOL (C7) is positive, this is initial value of AK on exponential recession k curve. Otherwise, this is uniform loss rate in inches per TR period (C2).

* Items G through J repeated in sequence for each of NHT(B2) floods.

3. DLTAL - If RTIOL (C7) is positive, this calls for an initial increment of AK equal to .2(DLTAL) above the exponential recession AK curve. Otherwise, this is initial loss in inches.
4. STRTQ - Flow in cfs at start of storm.
5. STORM - Total storm rain in inches. Given only when SPFE and SPS are zero and ratios for each TR (C2) period are to be supplied (J).
6. SPFE - Standard project storm index for eastern U.S. from Engineer Bulletin 52-8 (24-hr., 200 sq.mi.). If a positive value is given, SPS and STORM will be disregarded (H7, H5).
7. PMS - Probable maximum storm index in inches from HMS Report 33. Indicator to call for depth-duration ratios (I) and probable maximum storm computation
8. TRSPC - Storm transposition coefficient by which rain amounts will be multiplied. If not given, the coefficient using the "Hop Brook" formula will be supplied by program, using following item for drainage area.
9. TRSDA - Drainage area in square miles for which storm is transposed, used in computing SPS amount and Hop Brook shape factor, when needed.

I*. Depth-duration rain data. Supplied only if PMS (H7) is positive. At least first 3 values must be given.

1. R6 - Maximum 6-hour rain in percent of PMS
2. R12 - " 12- " " " " " "
3. R24 - " 24- " " " " " "
4. R48 - " 48- " " " " " "
5. R72 - " 72- " " " " " "
6. R96 - " 96- " " " " " "

J*. Rain or rain ratio values for each period. Must number NP (H1.). This item is omitted if SPFE or PMS is positive.

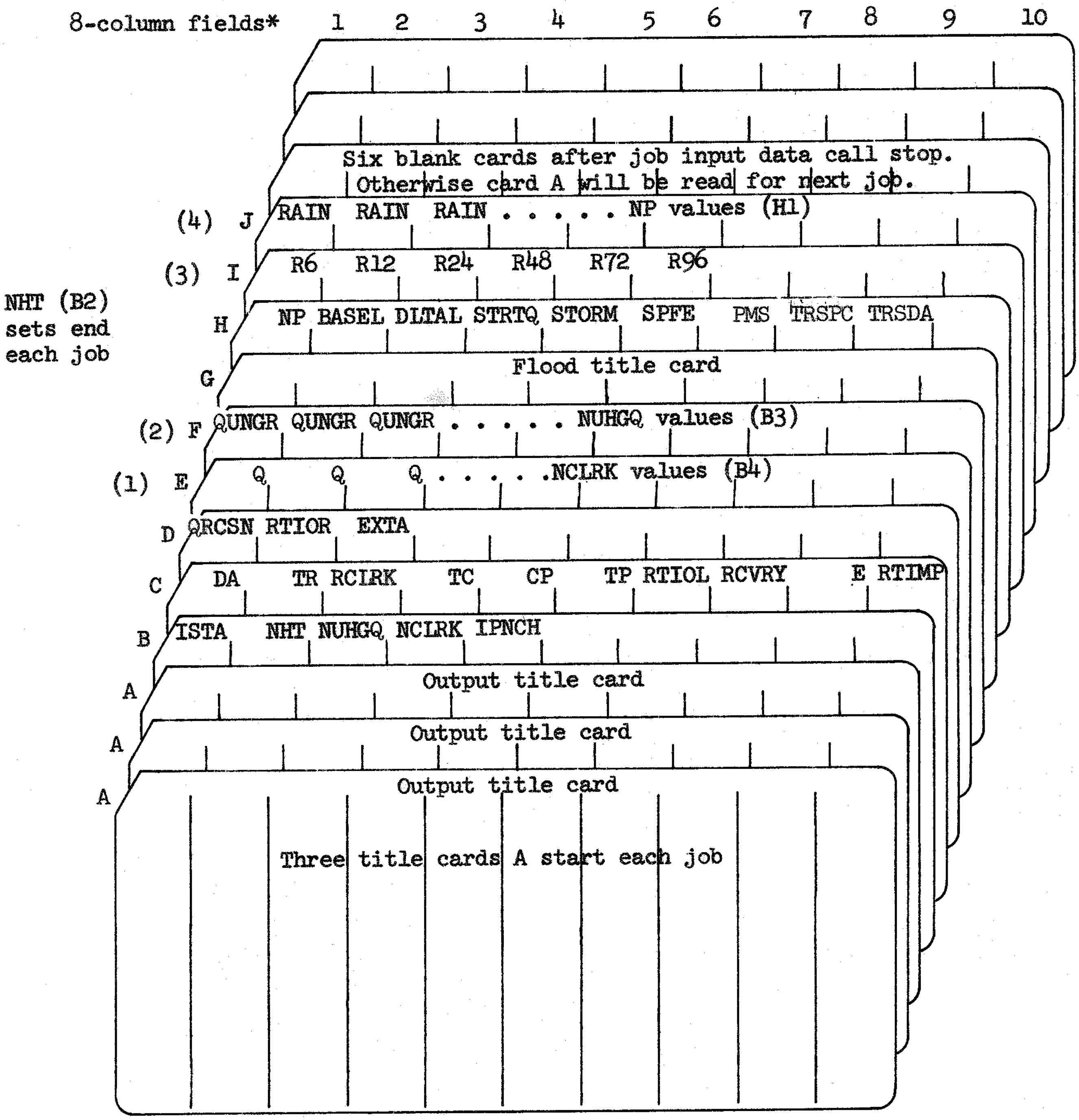
RAIN - Rain in inches per TR period or, if STORM (H5) is positive, as ratio to STORM (not in percent).

* Items G through J repeated in sequence for each of NHT(B2) floods. Six blank cards following job input data will cause computer to stop. Otherwise computer will branch back to read cards A for new job.

EXHIBIT 6

SUMMARY OF REQUIRED CARDS

23-J2-L228



Notes:

1. Supplied only if NUHGQ is zero and NCLRK is positive (B3, B4).
2. Supplied only if NCLRK is zero and NUHGQ is positive (B4, B3).
3. Supplied only if PMS is positive and SPFE is zero (H7, H6).
4. Supplied only if SPFE and PMS are both zero (H6, H7).

* Column 1 on each card is reserved for identification and not read by computer. Consequently first field is limited to 7 columns.

Appendix 2

Hydrograph Combining And Routing

HYDROGRAPH COMBINING AND ROUTING

COMPUTER PROGRAM 723-G1-L7310

AUGUST 1966

THE HYDROLOGIC ENGINEERING CENTER
U.S. ARMY ENGINEER DISTRICT, SACRAMENTO
609 SECOND STREET
DAVIS, CALIFORNIA 95616

HYDROGRAPH COMBINING AND ROUTING

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L232

CONTENTS

EXHIBITS

HYDROGRAPH COMBINING AND ROUTING

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L232

1. ORIGIN OF PROGRAM

This program was prepared in the Hydrologic Engineering Center, Corps of Engineers, 650 Capitol Mall, Sacramento, California, principally by Leo R. Beard. Up-to-date information and copies of source statement cards for various types of computers can be obtained from the Center upon request by Government and cooperating organizations.

2. PURPOSE OF PROGRAM

a. This program written in Fortran II will route hydrographs through river channels, given routing coefficients, and through reservoirs, given a table of fixed storage-outflow relation, for a river basin of any size and complexity to accomplish any one of the following:

(1) Given reservoir hold-outs and observed hydrographs downstream, compute unregulated flows

(2) Given observed or unregulated flows, compute local inflows

(3) Given hydrographs for all sub-areas (and local flows at combining points, if any), compute routed and combined hydrographs.

b. The program is also practical for single routings. Specified ratios of input hydrographs can be routed, and any combination of uniform and coefficient channel losses can be used.

c. Choices of methods for each routing are:

(1) Puls method for reservoir routings, where outflow is a function of storage only.

(2) Successive Puls method for channel routing, which is similar to that used in Columbia River Basin studies Program 24-J3-H001.

(3) Puls-lag method for channel routing, which uses storage-outflow table, and lag of an exact multiple of routing interval.

(4) Muskingum method for channel routing with K, X and number of identical reaches given.

(5) Tatum method for channel routing with number of steps designated as NTATM.

(6) Straddle-stagger method for channel routing.

d. Routing through reservoirs with downstream control and intermediate uncontrolled runoff is not possible with this program as written.

e. A listing of the source program and test input and output data are given at the end of this report.

3. DESCRIPTION OF EQUIPMENT

a. This program was prepared for use on the IBM 1620 computer with 40,000 digit, variable word length memory, card input and output, and is usable in the GE 225 and RCA 301 computers having comparable memory.

b. Source decks for these and other types of computers are available in the Hydrologic Engineering Center.

4. METHODS OF COMPUTATION

a. Procedures for routing are described in EM 1110-2-1408, "Routing of Floods Through River Channels", in ES-171 Technical Bulletin No. 22 (Multiple Storage), and in Handbook of Applied Hydrology, by Ven Te Chow. These are briefly described as follows:

(1) Puls: Outflow is a function of storage and therefore of storage indication (S+Q/2), which is determined from equation 1.

$$(S_2+O_2/2) = (S_1+O_1/2)+I_{1,2}-O_1 \qquad (1)$$

(2) Muskingum: Outflow is a function of prism and wedge storage, which are functions of inflow and outflow, determined as follows:

$$O_2 = (C_1-C_2)\ I_1+C_2\ (I_2+O_1) \qquad (2)$$

$$C_1 = 2(\Delta t)/(2K(1-X)+\Delta t) \qquad (3)$$

$$C_2 = (\Delta t-2KX)/(2K(1-X)+\ t) \qquad (4)$$

(3) Tatum: Instantaneous flows a routing interval apart are averaged to obtain new instantaneous flow at end of interval. This procedure is repeated until the number of steps (NTATM) times the routing interval is equal to twice the travel time for the reach. The routing interval can be longer (an exact multiple) than the inflow interval, to cause greater storage effect.

(4) Straddle-stagger: Successive inflows numbering NSTRL (at least 2) are averaged and the average is lagged LAG intervals beyond middle of range over which flows were averaged. In order to have outflows on correct timing, 1/2 time interval is added by program to specified value of LAG if straddle (NSTRL) is an even number. Thus, Tatum routing can be effected (but less efficiently) by specifying NRCHS (number of reaches) instead of NTATM as number of Tatum steps and 2 for NSTRL and zero for LAG.

b. Hydrographs can be routed and combined downstream at any interval that is an exact multiple of the input interval. All hydrographs must be read and must begin at the same time, but may be tabulated at different intervals that are exact multiples of each other. All routings (except Tatum, where it is not necessary can be repeated any number of times, using previous outflow for new inflow, without intermediate print-out. Hydrograph output from program 23-J2-L228, Unit Hydrograph and Hydrograph Computation, can be used directly for input to this program. All routing procedures include the assumption that flow has been uniform prior to the beginning of each hydrograph at the flow rate of its first ordinate.

5. INPUT

a. Input for hydrograph reading and routing and combining operations are summarized at the end of the text. All data are entered consecutively on each card, using 8 columns (digits, including decimal, if used) per variable and 10 variables per card, unless fewer variables are called for, except column 1 of each card is reserved for identification of the card series (A-F) and not read by the computer. Thus, the first field on each card is limited to 7 columns.

b. Each hydrograph (C, D) to be routed (local inflow is not routed) must be followed by routing cards (E, F). Whenever a hydrograph card contains a non-negative value or routing card contains a positive value for NHGT (which calls for a combining operation) the operation must be followed by the routing cards, unless it is the last operation for the run (positive value for FIN). If the hydrograph specification card contains a negative value of NHGT, the operation must be followed by another set of hydrograph cards. Accordingly, NHGT should be left blank except as follows:

(1) NHGT should be the number of hydrographs to be combined immediately following the operation in which it is located.

(2) NHGT should be -1 if one hydrograph operation is to be followed by another without an intermediate routing operation.

c. Hydrographs are stored automatically in the lowest available computer locations, and the highest locations used for hydrograph storage are released as soon as hydrographs are combined at a combining point. Hydrographs combined are those in the highest computer locations, so once a hydrograph above any combining point is read, all remaining hydrographs above that combining point must be read before any others, so that they will be in the highest computer locations. Local inflow or observed flow, if any, should be read last, since it calls for a combining operation. Since only a limited number of hydrographs can be retained in memory at any one time, the first hydrograph read above any combining point should be one of those having the most combining points to pass through in order to reach the final combining point of the system, so that hydrographs are not stored before they are needed. All routings to the same combining point must have the same outflow interval, but local inflow can have a shorter tabulation interval (exact fraction). Ordinates are normally in cfs at end of each interval (ITQI). However, average values in cfs can be entered if a positive value of IQAVG is given.

d. All values of ITQI (C3) ITQR (E1) and ITQO (E2) should for purposes of standardization be 5, 15 or 30 minutes or 1, 2, 6, 12 or 24 hours expressed in minutes. No routing or outflow interval (ITQO or ITQR) can be shorter than the longest upstream hydrograph interval (ITQI) or upstream outflow interval (ITQO).

e. LNGTH is length of flood in hours and cannot exceed KQ (see Fortran listing) times shortest interval used for any ITQI, ITQR or ITQO.

f. At the end of each run (complete basin routing for one flood) data for the next run will be accepted.

6. OUTPUT

a. Input (including hydrograph tabulations, if IPNCH is positive)

b. Routed hydrograph in cfs at end of each reach

c. Combined hydrograph (or derived local flow) in cfs at each combining point

d. Reservoir inflow and outflow in cfs and storage in acre-feet.

7. OPERATING INSTRUCTIONS

Standard Fortran II operating instructions. No sense switches used.

8. DEFINITION OF TERMS

Terms used in this program are defined at the end of the text.

9. EXAMPLE

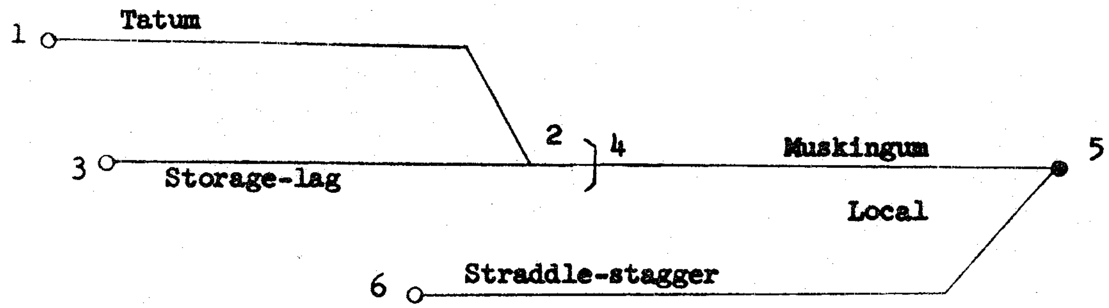
An example of each of the 3 basic program applications is given as Exhibits 1, 2 and 3.

10. PROPOSED FUTURE DEVELOPMENT

a. It is anticipated that additions to this program will be made from time to time. Alternative methods of routing and inclusion of remote downstream controls on reservoir releases are contemplated.

b. It is requested that any user of this program who finds an inadequacy or desirable addition or modification notify the Hydrologic Engineering Center.

TEST DATA SEQUENCE
23-J2-L232



Type of Input		Inflow location LOCI	Outflow location LOCO	Combine hydrographs NHGT	End of run FIN
COMPUTING UNREGULATED FLOWS ROUTINE					
Three output title cards					
Basin data card					
Holdouts	(C,D)	4			
Routing	(E)	4	5		
Hydrograph	(C,D)	5		2	1
COMPUTING LOCAL FLOWS ROUTINE					
Three output title cards					
Basin data card					
Hydrograph	(C,D)	4			
Routing	(E)	4	5		
Hydrograph	(C,D)	6			
Routing	(E)	6	5		
Hydrograph	(C,D)	5		3	1
HYDROGRAPH COMBINING ROUTINE					
Three output title cards					
Basin data card					
Hydrograph	(C,D)	1			
Routing	(E)	1	2		
Hydrograph	(C,D)	3			
Routing	(E,F)	3	2	2	
Routing	(E,F)	2	4		
Routing	(E)	4	5		
Hydrograph	(C,D)	6			
Routing	(E)	6	5		
Hydrograph	(C,D)	5		3	1

EXHIBIT 1

SAMPLE INPUT

```
A                                TEST NO. 1
A               HYDROGRAPH COMBINING AND ROUTING PROGRAM
A                  COMPUTING UNREGULATED FLOWS ROUTINE
B       1               66       5      17      65    1200
C       4      66      60
D                                       36      73     218     351     448     544
D     617     690     726     774     859     968    1065    1162    1246    1307
D    1367    1404    1428    1500    1585    1718    1924    2190    2493    2686
D    2735    2638    2493    2311    2069    2287    1537    1234     968     702
D     448     230      36    -133    -254    -339    -411    -436    -448    -496
D    -520    -557    -593    -605    -605    -629    -629    -641    -629    -641
D    -629    -629    -629    -617    -617    -605
E      60     120       1       5
E     3.2      .2
C       5      33     120                              2       1
D      76     124     343     535     671     807     983    1317    1617    1809
D    1995    2300    2802    3566    3822    3573    3463    3239    2897    2496
D    1970    1465    1129     869     631     463     369     318     290     264
D     233     211     198
A                                TEST NO. 2
A               HYDROGRAPH COMBINING AND ROUTING PROGRAM
A                    COMPUTING LOCAL INFLOWS ROUTINE
B               1      66       5      17      65    1200
C       4      66      60
D     192      44     186      60     200     200     202     205     209     213
D     218     224     230     237     244     252     260     270     280     291
D     304     321     339     358     377     399     422     449     482     516
D     552     586     618     648     674     698     717     734     746     755
D     761     764     764     763     759     755     750     744     738     732
D     725     718     710     702     694     686     678     670     662     654
D     645     637     629     621     613     605
E      60     120               5
E     3.2      .2
C       6      30     120
D       8      50     262     457     541     584     648     863    1038    1103
D    1166    1348    1722    2332    2398    1868    1478    1096     714     429
D     222     109     104     121      85      40      19       9       4       2
E     120     120       1       5               3       4
E
C       5      33     120                              3       1
D      76     124     343     535     671     807     983    1317    1617    1809
```

EXHIBIT 2

D	1995	2300	2802	3566	3822	3573	3463	3239	2897	2496
D	1970	1465	1129	869	631	463	369	318	290	264
D	233	211	198							
A	TEST NO. 3									
A	HYDROGRAPH COMBINING AND ROUTING PROGRAM									
A	HYDROGRAPH COMBINING ROUTINE									
B			66	5	17	65	1200			
C	1	60	60	.90		1				
D	76	53	83	267	489	599	580	570	637	625
D	598	617	691	845	955	974	1001	1044	1045	1026
D	1033	1087	1230	1360	1532	1836	2316	2427	2061	1756
D	1515	1348	1207	1004	807	651	518	413	292	192
D	124	86	61	56	109	163	148	102	66	43
D	8	18	12	8	5	3	1	0	0	
E	60	60		2	3					
E			20	.10						
C	3	60	60							
D	76	53	83	267	489	599	580	570	637	625
D	598	617	691	845	955	974	1001	1044	1045	1026
D	1033	1087	1230	1360	1532	1836	2316	2427	2061	1756
D	1515	1348	1207	1004	807	651	518	413	292	192
D	124	86	61	56	109	163	148	102	66	43
D	8	18	12	8	5	3	1	0	0	
E	60	60	2	2		1		2	-1	
F					2					
F	0	0	2000	1000						
E	60	60	1	4				5	-1	43
E					1					
F	5000		5000	200	6000	300	7000	450	9000	760
E	60	120	1	5						
E	3.2	.2								
C	6	30	120							
D	8	50	262	457	541	584	648	863	1038	1103
D	1166	1348	1722	2332	2398	1868	1478	1096	714	429
D	222	109	104	121	85	40	19	9	4	2
E	120	120	1	5		3	4			
E										
C	5	30	120				3	1		
D	8	50	262	457	541	584	648	863	1038	1103
D	1166	1348	1722	2332	2398	1868	1478	1096	714	429
D	222	109	104	121	85	40	19	9	4	2

EXHIBIT 2

1

SAMPLE OUTPUT

```
                              TEST NO. 1
                 HYDROGRAPH COMBINING AND ROUTING PROGRAM
                  COMPUTING UNREGULATED FLOWS ROUTINE

  IUNRG    LOCAL    LNGTH    IMNTH     IDAY      IYR    ITIME    RATIO
      1        0       66        5       17       65     1200     0.00

   LOCI      NQI     ITQI     RTIO    IQAVG    IPNCH     NHGT      FIN
      4       66       60     0.00        0        0        0       0.

FLOW AT       4  MULTIPLIED BY    1.000
     0.       0.       0.       0.      36.      73.     218.     351.     448.     544.
   617.     690.     726.     774.     859.     968.    1065.    1162.    1246.    1307.
  1367.    1404.    1428.    1500.    1585.    1718.    1924.    2190.    2493.    2686.
  2735.    2638.    2493.    2311.    2069.    2287.    1537.    1234.     968.     702.
   448.     230.      36.    -133.    -254.    -339.    -411.    -436.    -448.    -496.
  -520.    -557.    -593.    -605.    -605.    -629.    -629.    -641.    -629.    -641.
  -629.    -629.    -629.    -617.    -617.    -605.

   ITQR     ITQO    NRCHS     LOCO    NTATM      LAG    NSTRL    NPULS    STORA      RES
     60      120        1        5        0        0        0        0       0.       0.
  AMSKK        X    QLOSS    CULOS     NHGT      FIN
  3.200     .200      0.0    0.000        0       0.

HYDROGRAPH     4    ROUTED TO      5    NO. FLOWS=   33   INTERVAL= 120 MINUTES
     0.       0.       8.      80.     252.     430.     580.     706.     873.    1053.
  1208.    1319.    1437.    1639.    2021.    2404.    2497.    2314.    2091.    1556.
  1024.     540.     140.    -141.    -304.    -414.    -502.    -557.    -596.    -616.
  -626.    -628.    -622.

   LOCI      NQI     ITQI     RTIO    IQAVG    IPNCH     NHGT      FIN
      5       33      120     0.0C        0        0        2       1.

FLOW AT       5  MULTIPLIED BY    1.000
    76.     124.     343.     535.     671.     807.     983.    1317.    1617.    1809.
  1995.    2300.    2802.    3566.    3822.    3573.    3463.    3239.    2897.    2496.
  1970.    1465.    1129.     869.     631.     463.     369.     318.     290.     264.
   233.     211.     198.

UNRFGULATFD FLOW
```

EXHIBIT 3

AT 5 NQ= 33 120-MIN INTRVL

76.	124.	351.	615.	923.	1237.	1563.	2023.	2490.	2862.
3203.	3619.	4239.	5205.	5843.	5977.	5960.	5553.	4988.	4052.
2994.	2005.	1269.	727.	326.	48.	-133.	-239.	-306.	-352.
-393.	-417.	-424.							

EXHIBIT 3

```
                         TEST NO. 2
          HYDROGRAPH COMBINING AND ROUTING PROGRAM
              COMPUTING LOCAL INFLOWS ROUTINE

   IUNRG   LOCAL   LNGTH   IMNTH    IDAY     IYR   ITIME   RATIO
       0       1      66       5      17      65    1200    0.00

    LOCI     NQI    ITQI    RTIO   IQAVG   IPNCH    NHGT     FIN
       4      66      60    0.00       0       0       0      0.

FLOW AT     4  MULTIPLIED BY   1.000
    192.     44.    186.     60.    200.    200.    202.    205.    209.    213.
    218.    224.    230.    237.    244.    252.    260.    270.    280.    291.
    304.    321.    339.    358.    377.    399.    422.    449.    482.    516.
    552.    586.    618.    648.    674.    698.    717.    734.    746.    755.
    761.    764.    764.    763.    759.    755.    750.    744.    738.    732.
    725.    718.    710.    702.    694.    686.    678.    670.    662.    654.
    645.    637.    629.    621.    613.    605.

    ITQR    ITQO   NRCHS    LOCO   NTATM     LAG   NSTRL   NPULS   STORA     RES
      60     120       1       5       0       0       0       0      0.      0.
   AMSKK       X   QLOSS   CQLOS    NHGT     FIN
   3.200    .200     0.0   0.000       0      0.

HYDROGRAPH     4   ROUTED TO      5   NO. FLOWS=  33   INTERVAL= 120 MINUTES
    198.    161.    147.    176.    193.    205.    216.    229.    243.    260.
    280.    307.    339.    377.    424.    483.    547.    609.    662.    704.
    734.    750.    756.    754.    747.    737.    724.    710.    695.    679.
    663.    647.    630.

    LOCI     NQI    ITQI    RTIO   IQAVG   IPNCH    NHGT     FIN
       6      30     120    0.00       0       0       0      0.

FLOW AT     6  MULTIPLIED BY   1.000
      8.     50.    262.    457.    541.    584.    648.    863.   1038.   1103.
   1166.   1348.   1722.   2332.   2398.   1868.   1478.   1096.    714.    429.
    222.    109.    104.    121.     85.     40.     19.      9.      4.      2.

    ITQR    ITQO   NRCHS    LOCO   NTATM     LAG   NSTRL   NPULS   STORA     RES
     120     120       1       5       0       3       4       0      0.      0.
   AMSKK       X   QLOSS   CQLOS    NHGT     FIN
```

```
   0.000    0.000     0.0    0.000       0       0.

HYDROGRAPH      6    ROUTED TO      5    NO. FLOWS=  33    INTERVAL= 120 MINUTES
       8.       8.       8.      18.      82.     194.     327.     461.     557.     659.
     783.     913.    1042.    1163.    1334.    1642.    1950.    2080.    2019.    1710.
    1289.     929.     615.     368.     216.     139.     104.      87.      66.      38.
      18.       8.       3.

    LOCI      NQI     ITQI     RTIO    IQAVG    IPNCH     NHGT      FIN
       5       33      120     0.00        0        0        3       1.

FLOW AT      5  MULTIPLIED BY    1.000
      76.     124.     343.     535.     671.     807.     983.    1317.    1617.    1809.
    1995.    2300.    2802.    3566.    3822.    3573.    3463.    3239.    2897.    2496.
    1970.    1465.    1129.     869.     631.     463.     369.     318.     290.     264.
     233.     211.     198.

LOCAL FLOW
AT     5  NQ=  33    120-MIN INTRVL
    -130.     -45.     187.     339.     395.     407.     438.     626.     815.     889.
     931.    1080.    1420.    2024.    2062.    1447.     965.     549.     215.      81.
     -53.    -214.    -242.    -254.    -332.    -413.    -460.    -479.    -471.    -453.
    -448.    -444.    -436.
1
```

EXHIBIT 3

4

```
                              TEST NO. 3
                  HYDROGRAPH COMBINING AND ROUTING PROGRAM
                      HYDROGRAPH COMBINING ROUTINE

 IUNRG   LOCAL   LNGTH   IMNTH    IDAY     IYR   ITIME   RATIO
     0       0      66       5      17      65    1200    0.00

  LOCI     NQI    ITQI    RTIO   IQAVG   IPNCH    NHGT     FIN
     1      60      60     .90       0       1       0      0.
   76.     53.     83.    267.    489.    599.    580.    570.    637.    625.
  598.    617.    691.    845.    955.    974.   1001.   1044.   1045.   1026.
 1033.   1087.   1230.   1360.   1532.   1836.   2316.   2427.   2061.   1756.
 1515.   1348.   1207.   1004.    807.    651.    518.    413.    292.    192.
  124.     86.     61.     56.    109.    163.    148.    102.     66.     43.
    8.     18.     12.      8.      5.      3.      1.      0.      0.      0.

FLOW AT      1  MULTIPLIED BY      .900
   68.     47.     74.    240.    440.    539.    522.    513.    573.    562.
  538.    555.    621.    760.    859.    876.    900.    939.    940.    923.
  929.    978.   1107.   1224.   1378.   1652.   2084.   2184.   1854.   1580.
 1363.   1213.   1086.    903.    726.    585.    466.    371.    262.    172.
  111.     77.     54.     50.     98.    146.    133.     91.     59.     38.
    7.     16.     10.      7.      4.      2.      0.      0.      0.      0.

  ITQR    ITQO   NRCHS    LOCO   NTATM     LAG   NSTRL   NPULS   STORA     RES
    60      60       1       2       3       0       0       0      0.      0.
 AMSKK       X   QLOSS   CQLOS    NHGT     FIN
 0.000   0.000    20.0    .100       0      0.

HYDROGRAPH      1   ROUTED TO      2   NO. FLOWS=   66   INTERVAL=   60 MINUTES
   43.     41.     37.     58.    143.    280.    398.    447.    456.    470.
  483.    480.    484.    525.    607.    697.    754.    784.    807.    821.
  821.    823.    854.    927.   1033.   1170.   1377.   1643.   1817.   1757.
 1540.   1320.   1151.   1013.    871.    720.    578.    460.    360.    268.
  183.    116.     71.     44.     37.     54.     85.     97.     81.     52.
   26.      5.      0.      0.      0.      0.      0.      0.      0.      0.
    0.      0.      0.      0.      0.      0.

  LOCI     NQI    ITQI    RTIO   IQAVG   IPNCH    NHGT     FIN
     3      60      60    0.00       0       0       0      0.
```


EXHIBIT 3

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```
FLOW AT     3  MULTIPLIED BY    1.000
    76.      53.      83.     267.     489.     599.     580.     570.     637.     625.
   598.     617.     691.     845.     955.     974.    1001.    1044.    1045.    1026.
  1033.    1087.    1230.    1360.    1532.    1836.    2316.    2427.    2061.    1756.
  1515.    1348.    1207.    1004.     807.     651.     518.     413.     292.     192.
   124.      86.      61.      56.     109.     163.     148.     102.      66.      43.
     8.      18.      12.       8.       5.       3.       1.       0.       0.       0.

   ITQR     ITQO    NRCHS     LOCO    NTATM      LAG    NSTRL    NPULS    STORA      RES
     60       60        2        2        0        1        0        2      -1.       0.
  AMSKK        X    QLOSS    CQLOS     NHGT      FIN
  0.000    0.000      0.0    0.000        2       0.
STOR-OUTFLOW TABLE
      0.       0.       2000.   10000.

HYDROGRAPH      3    ROUTED TO       2    NO. FLOWS=  66    INTERVAL=  60 MINUTES

     PER   INFLOW OUTFLOW     STOR

HYDROGRAPH      3    ROUTED TO       2    NO. FLOWS=  66    INTERVAL=  60 MINUTES
    76.      76.      76.      75.      73.      78.     104.     157.     227.     300.
   365.     422.     471.     508.     538.     572.     617.     673.     733.     791.
   844.     890.     927.     956.     987.    1028.    1086.    1167.    1286.    1447.
  1618.    1745.    1803.    1797.    1742.    1654.    1540.    1405.    1260.    1110.
   963.     822.     688.     564.     455.     362.     288.     235.     201.     180.
   161.     143.     122.     101.      82.      66.      52.      40.      31.      23.
    17.      12.       9.       6.       4.       3.

COMBINED HYDROGRAPH
AT     2   NQ=   66      60-MIN INTRVL
   119.     117.     113.     133.     217.     359.     502.     604.     684.     770.
   848.     903.     955.    1033.    1146.    1269.    1372.    1457.    1541.    1613.
  1665.    1713.    1781.    1884.    2021.    2199.    2463.    2811.    3103.    3205.
  3158.    3066.    2955.    2810.    2614.    2374.    2118.    1866.    1620.    1378.
  1147.     938.     759.     609.     492.     417.     373.     332.     282.     232.
   188.     149.     122.     101.      82.      66.      52.      40.      31.      23.
    17.      12.       9.       6.       4.       3.

   ITQR     ITQO    NRCHS     LOCO    NTATM      LAG    NSTRL    NPULS    STORA      RES
     60       60        1        4        0        0        0        5      -1.      43.
  AMSKK        X    QLOSS    CQLOS     NHGT      FIN
```

EXHIBIT 3

√

6

133P

0.000 0.000 0.0 0.000 1 0.
STOR-OUTFLOW TABLE
5000. 0. 5000. 200. 6000. 300. 7000. 450.
9000. 760.

HYDROGRAPH 2 ROUTED TO 4 NO. FLOWS= 66 INTERVAL= 60 MINUTES

PER	INFLOW	OUTFLOW	STOR
1	119.	110.	5000.
2	118.	126.	5000.
3	115.	104.	5000.
4	123.	142.	5000.
5	175.	200.	5000.
6	288.	200.	5008.
7	431.	202.	5027.
8	553.	205.	5056.
9	644.	209.	5092.
10	727.	213.	5134.
11	809.	218.	5183.
12	876.	223.	5238.
13	929.	229.	5296.
14	994.	235.	5359.
15	1090.	242.	5429.
16	1207.	250.	5508.
17	1320.	259.	5596.
18	1414.	269.	5691.
19	1499.	279.	5793.
20	1577.	289.	5900.
21	1639.	301.	6011.
22	1689.	318.	6125.
23	1747.	336.	6242.
24	1833.	354.	6365.
25	1952.	374.	6496.
26	2110.	395.	6639.
27	2331.	419.	6798.
28	2637.	446.	6980.
29	2957.	478.	7186.
30	3154.	512.	7406.
31	3181.	546.	7625.
32	3112.	579.	7836.
33	3010.	610.	8035.

34	2882.	639.	8222.
35	2712.	665.	8393.
36	2494.	689.	8543.
37	2246.	708.	8671.
38	1992.	725.	8777.
39	1743.	738.	8861.
40	1499.	748.	8923.
41	1262.	754.	8966.
42	1042.	758.	8990.
43	848.	759.	8998.
44	684.	758.	8992.
45	551.	756.	8975.
46	455.	752.	8950.
47	395.	747.	8921.
48	353.	742.	8888.
49	307.	737.	8853.
50	257.	730.	8813.
51	210.	724.	8770.
52	168.	717.	8725.
53	135.	709.	8677.
54	112.	702.	8628.
55	92.	694.	8579.
56	74.	686.	8527.
57	59.	678.	8476.
58	46.	670.	8424.
59	35.	662.	8371.
60	27.	654.	8319.
61	20.	646.	8267.
62	15.	638.	8215.
63	10.	630.	8163.
64	7.	622.	8112.
65	5.	614.	8062.
66	3.	606.	8011.

ITQR	ITQO	NRCHS	LOCO	NTATM	LAG	NSTRL	NPULS	STORA	RES
60	120	1	5	0	0	0	0	0.	0.
AMSKK	X	QLOSS	CQLOS	NHGT	FIN				
3.200	.200	0.0	0.000	0	0.				

HYDROGRAPH 4 ROUTED TO 5 NO. FLOWS= 33 INTERVAL= 120 MINUTES

109.	110.	144.	175.	192.	205.	216.	228.	243.	259.

√

EXHIBIT 3

```
  279.    305.    337.    375.    422.    480.    542.    602.    654.    696.
  726.    744.    751.    750.    745.    735.    723.    710.    695.    679.
  664.    647.    632.

  LOCI     NQI    ITQI    RTIO   IQAVG   IPNCH    NHGT     FIN
     6      30     120    0.00       0       0       0      0.

FLOW AT      6  MULTIPLIED BY   1.000
    8.     50.    262.    457.    541.    584.    648.    863.   1038.   1103.
 1166.   1348.   1722.   2332.   2398.   1868.   1478.   1096.    714.    429.
  222.    109.    104.    121.     85.     40.     19.      9.      4.      2.

  ITQR    ITQO   NRCHS    LOCO   NTATM     LAG   NSTRL   NPULS   STORA     RES
   120     120       1       5       0       3       4       0      0.      0.
 AMSKK       X   QLOSS   CQLOS    NHGT     FIN
 0.000   0.000     0.0   0.000       0      0.

HYDROGRAPH     6   ROUTED TO     5   NO. FLOWS=  33   INTERVAL= 120 MINUTES
    8.      8.      8.     18.     82.    194.    327.    461.    557.    659.
  783.    913.   1042.   1163.   1334.   1642.   1950.   2080.   2019.   1710.
 1289.    929.    615.    368.    216.    139.    104.     87.     66.     38.
   18.      8.      3.

  LOCI     NQI    ITQI    RTIO   IQAVG   IPNCH    NHGT     FIN
     5      30     120    0.00       0       0       3      1.

FLOW AT      5  MULTIPLIED BY   1.000
    8.     50.    262.    457.    541.    584.    648.    863.   1038.   1103.
 1166.   1348.   1722.   2332.   2398.   1868.   1478.   1096.    714.    429.
  222.    109.    104.    121.     85.     40.     19.      9.      4.      2.

COMBINED HYDROGRAPH
AT     5  NQ=   33    120-MIN INTRVL
  125.    168.    414.    651.    815.    983.   1192.   1552.   1838.   2021.
 2228.   2566.   3101.   3870.   4155.   3990.   3970.   3778.   3387.   2835.
 2237.   1782.   1470.   1240.   1046.    914.    847.    806.    765.    720.
  682.    656.    635.
```


EXHIBIT 3

DEFINITIONS - 23-J2-L232

AMSKK	- Muskingum K
C1	- Routing coefficient
C2	- Routing coefficient
COEF	- Channel loss coefficient
CQLOS	- Channel loss as ratio of flow remaining after uniform loss (QLOSS) is subtracted
DISCH(K)	- Outflow in storage-outflow table
FIN	- Indicator ends run where positive
I	- Temporary index
IX	- Temporary index
IDAY	- Day of flood beginning
IMNTH	- Month number of flood beginning
IPNCH	- Indicator to punch input hydrograph when positive
IQAVG	- Indicator that inflows are average for period when positive
ITIME	- Time on 24-hour scale at start of first flood period
ITQI	- Inflow interval in minutes
ITQO(L)	- Outflow interval in minutes
ITQR	- Routing interval in minutes
IUNRG	- Indicator to compute unregulated flows when positive
IYR	- Year of flood beginning
J	- Temporary index
JX	- Temporary index
K	- Temporary index
KHYD	- Dimension limit, number of hydrographs
KQ	- Dimension limit, number of flows
KSTR	- Dimension limit, storage-discharge table
L	- Hydrograph number in computer
LAG	- Lag (translation) time of hydrograph as a multiple of ITQI
LNGTH	- Duration in hours of desired outflow tabulations
LOCAL	- Indicator for routing to compute local flow from observed flows when positive
LOCI	- Identification number of inflow location
LOCO	- Identification number of outflow location
M	- Flow index
MI	- Temporary index
MX	- Temporary index
NHGT	- Indicator calling for hydrograph combining, number of hydrographs to be combined
NOI	- Ratio of outflow to inflow interval
NPULS	- Indicator calling for Puls (storage-lag) routing, number of points to be read on storage-outflow table
NQI	- Number of inflows to be read
NQI1	- NQI-1
NQO1	- Temporary variable
NQOMX	- Number of outflows

EXHIBIT 4

NRCHS - Number of identical routings to be repeated
NRI - Ratio of routing to inflow interval
NSTRL - Number of successive flows to be averaged in straddle-stagger routing
NTATM - Number of steps in Tatum routing
Q(M,L) - Flow in cfs
QLOSS - Uniform loss in cfs
QO - Initial outflow in Puls routing
RATIO - Ratio by which all hydrographs are multiplied before routing
RTIO - Ratio by which individual hydrographs are multiplied before routing
RESQ - Indicator for reservoir routing
RSTRL - Reciprocal of NSTRL
STND - Storage indication (storage + $\frac{1}{2}$ outflow)
STORA - Storage in thousand ac-ft.
STR(K) - Storage and subsequently storage indication in table of STR versus outflow
STRDL - NSTRL
STRMN - Minimum storage in reservoir routing
TR - Routing interval in hours
X - Muskingum X

SOURCE PROGRAM LISTING

```
*05074
C     HYDROGRAPH COMBINING AND ROUTING  PROGRAM 23-J2-L232
C     HYDROLOGIC ENGINEERING CENTER         JULY 1966
C     LIBRARY SUBR NOT USED
      DIMENSION Q(150,5),STR(20),DISCH(20),ITQO(5)
      KQ=150
      KHYD=5
      KSTR=20
    1 FORMAT (1X,I7,9I8)
    2 FORMAT (1X,F7.0,9F8.0)
    3 FORMAT(10F8.0)
  100 PRINT 105
  105 FORMAT (1H1)
C                         THREE TITLE CARDS
      READ 110,(Q(I,1),I=1,120)
      PRINT 110,(Q(I,1),I=1,120)
  110 FORMAT (1X,A1,9A2,15A2,15A2)
      READ 140,IUNRG,LOCAL,LNGTH,IMNTH,IDAY,IYR,ITIME,RATIO
      IF(LNGTH)115,115,120
  112 PRINT 113
  113 FORMAT (19H DIMENSION EXCEEDED)
  115 STOP
  120 PRINT 130
  130 FORMAT(/64H    IUNRG    LOCAL    LNGTH    IMNTH    IDAY     IYR    ITIM
     1E   RATIO)
      PRINT 195, IUNRG,LOCAL,LNGTH,IMNTH,IDAY,IYR,ITIME,RATIO
  140 FORMAT (1X,I7,6I8,3F8.0)
      IF(RATIO)144,144,146
  144 RATIO = 1.
  146 L = 0
  150 L = L + 1
      IF(L-KHYD)180,180,160
  160 PRINT 170
  170 FORMAT(28H TOO MANY HYDROGRAPHS STORED)
      STOP
C                              READ HYDROGRAPH
  180 READ 182, LOCI,NQI,ITQI,RTIO,IQAVG,IPNCH,NHGT,FIN
  182 FORMAT(1X,I7,2I8,F8.0,3I8,F8.0)
      IF (KQ-LNGTH*60/ITQI) 112,185,185
  185 NQOMX=LNGTH*60/ITQI
      PRINT 190
```

```
190 FORMAT(/64H    LOCI     NQI     ITQI     RTIO    IQAVG    IPNCH    NHG
   1T     FIN)
    PRINT 196,LOCI,NQI,ITQI,RTIO,IQAVG,IPNCH,NHGT,FIN
195 FORMAT(7I8,F8.2)
196 FORMAT(3I8,F8.2,3I8,F8.0)
    IF(NQI-NQOMX) 220,220,200
200 PRINT 210
210 FORMAT(25H TOO MANY FLOWS FOR LNGTH)
    STOP
220 DO 230 I = NQI,NQOMX
230 Q(I,L)=0.
    READ 2, (Q(M,L),M=1,NQI)
    IF(IPNCH) 250,250,240
240 PRINT 2,(Q(M,L),M=1,NQI)
250 IF(RATIO    *RTIO) 260,260,290
260 IF(RTIO) 270,270,300
270 IF(RATIO    )330,330,280
280 RTIO=RATIO
    GO TO 300
290 RTIO=RTIO*RATIO
300 DO 310 M=1,NQI
310 Q(M,L)=Q(M,L)*RTIO
    PRINT 320,LOCI,RTIO
320 FORMAT(/8H FLOW ATI6,15H  MULTIPLIED BY F8.3)
    PRINT 2,(Q(M,L),M=1,NQI)
330 IF (NHGT) 340,360,350
340 ITQO(L) = ITQI
    GO TO 150
350 ITQR=0
    RES=0.
    ITQO(L)=ITQO(L-1)
    IF(ITQI-ITQO(L)) 1080,1180,420
C                          ROUTING DATA                                  130
360 READ 361,ITQR,ITQO(L),NRCHS,LOCO,NTATM,LAG,NSTRL,NPULS,STORA,RES
361 FORMAT(1X,I7,7I8,2F8.0)
    IF (NRCHS) 364,364,366
364 NRCHS=1
366 READ 367,AMSKK,X,QLOSS,CQLOS,NHGT,FIN
367 FORMAT(1X,F7.0,3F8.0,I8,F8.0)
    PRINT 370
370 FORMAT (/80H    ITQR     ITQO    NRCHS     LOCO    NTATM     LAG    NST
```

```
   1RL    NPULS    STORA      RES)
      PRINT 361, ITQR,ITQO(L),NRCHS,LOCO,NTATM,LAG,NSTRL,NPULS,STORA,RES
      PRINT 380
  380 FORMAT (48H   AMSKK        X   QLOSS   CQLOS    NHGT     FIN)
      PRINT 390,AMSKK,X,QLOSS,CQLOS,NHGT,FIN
  390 FORMAT(2F8.3,F8.1,F8.3,I8,F8.0)
      NRI = ITQR/ITQI                                                137
      NOI = ITQO(L)/ITQI                                             138
C                           DISCARD EXTRA INFLOWS                    152
      IF (ITQR - ITQO(L)) 400,400,410
  400 IF (ITQR - ITQI) 420,480,460
  410 IF (ITQO(L) - ITQI) 420,480,440
  420 PRINT 430
  430 FORMAT(19H INTERVAL TOO SHORT)
      STOP
  440 NQI = NQI/NOI
      NRI = NRI/NOI                                                  157
      DO 450 M = 1,NQI
      MI = M * NOI                                                   160
  450 Q(M,L) = Q(MI,L)
      NOI = 1                                                        162
      ITQI=ITQO(L)
      GO TO 480
  460 NQI = NQI/NRI
      DO 470 M = 1,NQI
      MI = M * NRI                                                   169
  470 Q(M,L) = Q(MI,L)
      NOI = NOI/NRI                                                  171
      ITQI=ITQR
      NRI = 1                                                        175
  480 NQOMX=LNGTH*60/ITQI
      NQI1 = NQI + 1                                                 172
      DO 490 I = NQI1,NQOMX
  490 Q(I,L) = 0.
      TR = ITQI
      TR = TR/60.
C                           ROUTING                                  176
      IF (NPULS) 500,500,670
  500 NQI1=NQI-1
      IF(IQAVG) 530,530,510
  510 DO 520 I=1,NQI1
```

```
  520 Q(I,L)=(Q(I,L)+Q(I+1,L))*.5
      Q(NQI,L)=Q(NQI,L)*.5
  530 IF (AMSKK) 540,540,580
  540 IF (NSTRL) 550,550,600
  550 IF (NTATM) 560,560,1040
  560 PRINT 570
  570 FORMAT (21H NO ROUTING PERFORMED)
      GO TO 1170
C                          MUSKINGUM ROUTING
  580 C1 = 2.*TR/(2.*AMSKK*(1.-X) + TR)
      C2 = (TR-2.*AMSKK*X)/(2.*AMSKK*(1.-X)+TR)
      DO 590 K=1,NRCHS
      TEMP = Q(1,L)
C     TMP=CURRENT INFLOW   TEMP=PREVIOUS INFLOW   Q(M-1,L)=PREV OUTFLOW
      DO 590 M=2,NQOMX
      TMP = Q(M,L)
      Q(M,L) = (C1-C2)*TEMP + C2*Q(M,L)+Q(M-1,L)*(1.-C1)
  590 TEMP = TMP
      GO TO 1070
C                          STRADDLE-STRAGGER ROUTING
  600 STRDL = NSTRL
      RSTRL = 1./STRDL
      DO 660 K=1,NRCHS
      DO 660 M=1,NQOMX
C           IX = END OF STAGGER
      IX=NQOMX-M+1
C     J=START OF STRADDLE
      J = IX - NSTRL/2-LAG
C     JX = END OF STRADDLE
      JX = J+NSTRL-1
      TMP = 1.
      IF(J) 610,610,620
C     TMP IS NO OF TIMES FIRST FLOW OF FLOOD IS AVERAGED IN AT BEGINNING
  610 TMP = -J+2
      J = 1
  620 IF(JX-1) 630,630,640
  630 Q(IX,L)=Q(1,L)
      GO TO 660
  640 TEMP = TMP*Q(J,L)
      J = J + 1
      DO 650 I=J,JX
```

EXHIBIT 5

```
650 TEMP = TEMP + Q(I,L)
    Q(IX,L) = TEMP*RSTRL
660 CONTINUE
    GO TO 1070
C                             STORAGE-LAG ROUTING
670 READ 2,(STR(I),DISCH(I),I=1,NPULS)
    PRINT 680
680 FORMAT(19H STOR-OUTFLOW TABLE)
    PRINT 682,(STR(I),DISCH(I),I=1,NPULS)
682 FORMAT(4(F10.0,F8.0))
    STRMN=-1.
    IF(STR(1)-STR(2)) 700,690,700
690 STRMN=STR(1)
C   COMPUTE STORAGE INDICATION IN CFS-TR UNITS
700 DO 710 I=1,NPULS
710 STR(I)=STR(I)*12.1/TR+DISCH(I)*.5
    QO = Q(1,L)
    PRINT 1160,LOCI,LOCO,NQOMX,ITQO(L)
    IF(RES)722,722,715
715 PRINT 720
720 FORMAT(/32H      PER   INFLOW OUTFLOW    STOR)
722 DO 1030 J=1,NRCHS
    IF(J-1)725,725,730
725 IF(IQAVG) 730,730,750
730 DO 740 I=2,NQI
    K = NQI-I+2
740 Q(K,L)=(Q(K,L)+Q(K-1,L))*.5
750 IF (RES) 758,758,754
754 IF (STORA) 758,756,756
756 STND = STORA*12.1/TR+QO*0.5
    GO TO 840
758 IF(QO -DISCH(1)) 760,790,790
760 IF(STRMN) 770,790,790
770 PRINT 780
780 FORMAT ( 31H OPERATING BEYOND RANGE OF DATA)
790 DO 800 I=2,NPULS
    IF(QO-DISCH(I)) 810,810,800
800 CONTINUE
810 IF (DISCH(I) - DISCH(I-1)) 830,820,830
820 STND = STR(I-1)
    GO TO 840
```

```
830 STND = (STR(I)*(QO-DISCH(I-1)) + STR(I-1)*(DISCH(I)-QO))/(DISCH
   1(I)-DISCH(I-1))
840 NQO1 = NQOMX-LAG
    DO 990 M=1,NQO1
    TMP=Q(M,L)
    IF(M-1) 850,850,860
850 STND=STND+Q(1,L)-QO
    GO TO 870
860 STND = STND+Q(M,L)-Q(M-1,L)
870 IF(STND-STR(1)) 880,910,910
880 IF(STRMN) 890,900,900
890 PRINT 780
900 I = 2
    GO TO 950
910 IF(STND-STR(NPULS)) 930,930,920
920 PRINT 780
    I=NPULS
    GO TC 950
930 DO 940 IX=2,NPULS
    I = IX
    IF(STND-STR(I)) 950,950,940
940 CONTINUE
950 IF(STR(I)-STR(I-1))954,954,957
954 Q(M,L) = DISCH(I-1)
    GO TO 958
957 Q(M,L)= (DISCH(I)*(STND-STR(I-1))+DISCH(I-1)*(STR(I)-STND))/(S
   1TR(I)-STR(I-1))
958 IF(Q(M,L)) 960,970,970
960 Q(M,L) = 0.0
970 IF(RES) 990,990,980
980 STOR=(STND-Q(M,L)*.5)*TR/12.1+.5
    TEMP=TMP
985 FORMAT (I8,2F8.0,F10.0)
    PRINT 985,M,TMP,Q(M,L),STOR
990 CONTINUE
    IF(LAG) 1030,1030,1000
1000 DO 1010 I=1,NQO1
    M = NQOMX-I+1
    MX = M-LAG
1010 Q(M,L) = Q(MX,L)
    DO 1020 M=2,LAG
```

```
 1020 Q(M,L) = Q(1,L)
      QO=Q(1,L)
 1030 CONTINUE
      GO TO 1070
C                                   TATUM ROUTING
 1040 NQI1 = NQOMX-1
 1050 DO 1060 K = 1,NTATM
      DO 1060 M = NRI,NQI1
      J = NQOMX - M + NRI                                                  180
      I = J - NRI                                                          181
 1060 Q(J,L) = (Q(J,L) + Q(I,L)) * .5
C                                DISCARD EXTRA OUTFLOWS                    183
 1070 IQAVG=0
      IF (ITQO(L)-ITQR)  1100,1100,1080
 1080 NOI=ITQO(L)/ITQI
      NQOMX = NQOMX/NOI                                                    185
      DO 1090 M = 1,NQOMX
      MI = M * NOI                                                         187
 1090 Q(M,L) = Q(MI,L)
C                 ITQR IS ZERO FOR LOCAL INFLOW
       IF (ITQR) 1180,1180,1100
C                                LOSSES                                    192
 1100 IF(QLOSS+CQLOS) 1110,1140,1110
 1110 COEF = 1. - CQLOS
      DO 1130 I = 1,NQOMX
      Q(I,L) = (Q(I,L)-QLOSS) * COEF                                       197
      IF(Q(I,L)) 1120,1130,1130
 1120 Q(I,L) = 0.
 1130 CONTINUE
C                               PUNCH ROUTED HYDROGRAPH
 1140 IF(RES) 1150,1150,1170
 1150 PRINT 1160,LOCI,LOCO,NQOMX,ITQO(L)
      PRINT 2,(Q(J,L),J=1,NQOMX)
 1160 FORMAT(/11H HYDROGRAPHI6,12H   ROUTED TOI6,13H   NO. FLOWS=I4,12H
     1  INTERVAL=I4,8H MINUTES)
C                                COMBINING                                 206
 1170 IF(NHGT-1) 150,1360,1180
C                CHECK COMBINING INTERVAL
 1180 K = L - NHGT + 2
      DO 1210 I=K,L
      IF(ITQO(I)-ITQO(I-1)) 1190,1210,1190
```

EXHIBIT 5

```
1190 PRINT 1200
1200 FORMAT(/28H UNEQUAL COMBINING INTERVALS)
     GO TO 100
1210 CONTINUE
     K = K -1
     J = L-1
C    PRINT OUT Q(M,IX) IX=L FOR LOCAL AND UNREG, K FOR COMBINED
     IX=L
     IF(IUNRG) 1240,1240,1220
1220 PRINT 1230
1230 FORMAT(/17H UNREGULATED FLOW)
     GO TO 1290
1240 IF(LOCAL) 1270,1270,1250
1250 PRINT 1260
1260 FORMAT(/11H LOCAL FLOW)
     GO TO 1290
1270 IX=K
     J = L
     PRINT 1280
1280 FORMAT(/20H COMBINED HYDROGRAPH)
1290 DO 1340 M = 1,NQOMX
     QA = 0.                                                      214
     DO 1300 I=K,J
1300 QA = QA + Q(M,I)
     Q(M,K) = QA
     IF(IUNRG) 1320,1320,1310
1310 Q(M,L)=Q(M,L)+QA
     GO TO 1340
1320 IF(LOCAL) 1340,1340,1330
1330 Q(M,K) = Q(M,L)
     Q(M,L) = Q(M,L) - QA
1340 CONTINUE
     L = K                                                        219
C                        PUNCH COMBINED HYDROGRAPH                2212
     PRINT 1350,LOCO,NQOMX,ITQO(L)
1350 FORMAT(3H AT15,5H  NQ=I4,I6,11H-MIN INTRVL)
     PRINT 3,(Q(M,IX),M=1,NQOMX)
1360 ITQI = ITQO(L)
     NQI = NQOMX                                                  209
     LOCI = LOCO                                                  210
     IF (FIN) 360,360,100
     END
```

INPUT DATA 23-J2-L232

A. 3 Cards - Output title cards

B. Basin data card - (No decimal points for first 7 items)

1. IUNRG - Positive if job is for computing unregulated flows, leave blank normally
2. LOCAL - Positive if job is for computing local flows, leave blank normally
3. LNGTH - Duration of flood output desired, hours, cannot exceed 240 times shortest inflow interval without dimension change
4. IMNTH - Month when flood started, if historical
5. IDAY - Day " " " " "
6. IYR - Year " " " " "
7. ITIME - Time " " " " " , on 2400 time scale
8. RATIO - Ratio by which all input hydrographs will be multiplied (in addition to any value of RTIO, item C3) before routing. Assumes 1.0 if left blank.

C*. Hydrograph specification card (No decimal points for items 1, 2, and 4-7)

1. LOCI - Inflow location number (any positive integer)
2. NQI - Number of inflow values, cannot exceed LNGTH*(60)/ITQI (B3, C4)
3. ITQI - Inflow interval in minutes
4. RTIO - Ratio by which individual hydrograph will be multiplied (in addition to any value of RATIO, item B8) before routing
5. IQAVG - Positive if inflows are average for period, otherwise inflows must be instantaneous at end of period
6. IPNCH - Positive if input hydrograph is to be printed
7. NHGT - Number of hydrographs to be combined (or used to compute unregulated or local flows if item B1 or B2 is positive). Leave blank (or zero) if this hydrograph is to be routed before combining. Use -1 if another hydrograph is to be read before routing or combining.
8. FIN - Use 1 if this is last input for job, otherwise leave blank

*These hydrograph cards (C and D) to be followed by next set of hydrograph cards (C and D) if NHGT on the current hydrograph cards is negative, otherwise by next set of routing cards (E and F).

EXHIBIT 6

D*. Hydrograph cards - Flows or reservoir holdouts in cfs at end of period unless IQAVG, item C5 is positive, in which case these are average flows or holdouts for period, enough cards to contain NQI flows, item C2, 10 per card.

E#. Two data cards for each routing (No decimal points for items 1-8 and 15)

1. ITQR - Routing interval (Δt) in minutes (can equal or be an exact multiple of inflow interval)
2. ITQO - Outflow interval in minutes, must be same for all routings to same combining location, can equal or be an exact multiple of inflow interval. Must equal or exceed ITQR for straddle-stagger and Puls-lag routings.
3. NRCHS - Number of successive reaches to be routed with identical routing specification without intermediate print-out (one for Tatum method and 1 or more for any other method)
4. LOCL - Outflow location number
5. NTATM - Number of routing steps for Tatum method (zero if Tatum method is not used).
6. LAG - Lag (stagger) as number of routing intervals (ITQR, item E1) for reach
7. NSTRL - Number of routing (ITQR) values (straddle) to be averaged (zero if straddle-stagger method not used), must be 2 or greater but cannot exceed 2* LAG + 2.
8. NPULS - Number of points to be read for outflow-vs-storage table (cards F) for modified Puls routing (zero if storage-lag method or reservoir routing not used). Limited to 20 points (4 cards) unless dimension changed.
9. STORA - Starting storage in acre-feet for reservoir routing, -1 if starting storage to be determined from tables (cards F) based on starting inflow, as in channel routings.
10. RES - Positive value indicates reservoir routing (by modified Puls method)
11. AMSKK - K coefficient for Muskingum routing (zero if Muskingum method not used)

*These hydrograph cards (C and D) to be followed by next set of hydrograph cards (C and D) if NHGT on the current hydrograph cards is negative, otherwise by next set of routing cards (E and F).

#These routing cards (E and F) to be followed by next set of routing cards (E and F) if NHGT on the current routing cards is positive, otherwise by next set of hydrograph cards (C and D).

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12. X - X coefficient for Muskingum routing
13. QLOSS - Constant loss in cfs
14. CQLOS - Loss as ratio of remaining outflow
15. NHGT - Number of hydrographs to be combined immediately after this routing operation (zero if other hydrograph to be read before routing or combining, one if routed hydrograph is to be immediately routed further)
16. FIN - Use 1 if this is last input for run, otherwise leave blank

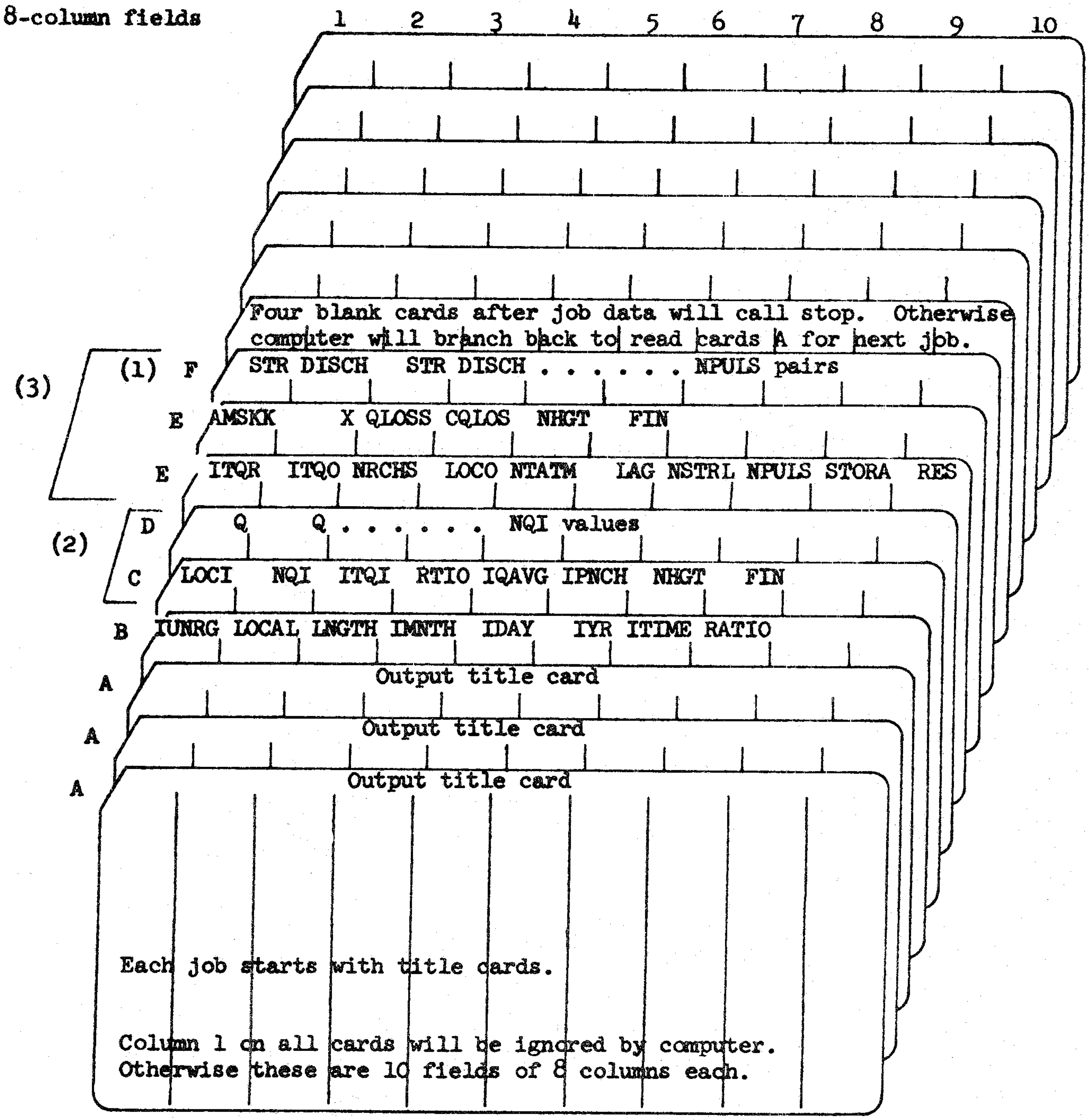
F#. Remaining routing cards - Storage and outflow alternately up to 20 values each (1 to 4 cards), omit if NPULS is zero or negative (E8).

STR, DISCH - Storage in acre-feet and corresponding outflow in cfs. In case of reservoir routing, minimum storage can be set by making first two items of STR equal, the first DISCH equal to zero and the second DISCH equal to the desired value for that storage.

#These routing cards (E and F) to be followed by next set of routing cards (E and F) if NHGT on the current routing cards is positive, otherwise by next set of hydrograph cards (C and D)

Note: A combining operation can be performed only one time at a station, therefore all required routings to a station must be performed before combining.

SUMMARY OF REQUIRED CARDS



(1) Omit if NPULS (E8) is zero or negative.

(2) Hydrograph cards to be followed by next set of hydrograph cards if NHGT (C7 on this set) is negative, otherwise by next set of routing cards (E and F).

(3) Routing cards to be followed by next set of routing cards if NHGT (E15 on this set) is positive, otherwise by next set of hydrograph cards (C and D).

Appendix 3

Balanced Hydrograph

BALANCED HYDROGRAPH

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L237

NOVEMBER 1965

BALANCED HYDROGRAPH

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L237

CONTENTS

EXHIBITS

BALANCED HYDROGRAPH

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L237

1. ORIGIN OF PROGRAM

This program, written in Fortran II, was prepared in the Hydrologic Engineering Center, Corps of Engineers, 650 Capitol Mall, Sacramento, California, principally by Leo R. Beard. Up-to-date information and copies of source statement cards for various types of computers can be obtained from the Center upon request by Government and cooperating agencies.

2. PURPOSE OF PROGRAM

a. A "balanced hydrograph" is defined herein as one that conforms to a specified set of volume-duration values. Given a specified set of durations and a pattern hydrograph whose duration equals or exceeds the longest specified duration, the program will compute any number of balanced hydrographs for specified sets of average flows corresponding to the set of durations. Each time such a job is completed, the computer will be ready to accept data for another job.

b. If several pattern hydrographs for the same basin are to be used for future routing and combining, their relative timing can be preserved by entering the same period of flows for all hydrographs, and the duration of that period must correspond to the longest specified duration, which must be uniform for all locations.

3. DESCRIPTION OF EQUIPMENT

a. This program was prepared for use in the IBM 1620 computer with 40,000 digit, variable word length memory, card input and output, and is usable in the GE 225 and RCA 301 computers having comparable memory, if any necessary input and output statement changes are made.

b. Source decks for these and other types of computers are available in the Hydrologic Engineering Center.

4. METHODS

a. Two arrays of flows are carried. The first (Q) consists of the pattern hydrograph and the second (Q2) of the adjusted hydrograph. Starting with the shortest duration specified, the period of maximum flow of the pattern hydrograph is determined, and the sum of all flows within each period that have not already been used in shorter-duration computation is designated SUMQ. Adjusted flows (Q2) within that period are subtracted from the specified volume for that duration to obtain VOL. Then all unadjusted flows within that duration are multiplied by the ratio, VOL/SUMQ.

b. When each period of maximum flows is within all such periods for longer durations, this solution should be exact. Otherwise, it is possible that non-coincidence will substantially change the hydrograph shape. Consequently, if any flow adjustment has exceeded 5 percent, the adjusted hydrograph is re-adjusted using the same procedure, and this process is repeated until no adjustment grater than 5 percent is made.

5. INPUT

Input is summarized in exhibits 5 and 6. All data are entered consecutively on each card, using 8 columns (digits, including decimal, if used) per variable and 10 variables per card, unless fewer variables are called for. The first column of each card is not read.

6. OUTPUT

a. Input volume-duration data and specification

b. Comparison of specified and derived average flows for each duration

c. Adjusted flows

7. OPERATING INSTRUCTIONS

Standard Fortran II operating instructions. No sense switches used.

8. DEFINITION OF TERMS

Terms used in this program are defined in exhibit 3.

9. EXAMPLE

An example of program input and output is given in exhibits 1 and 2.

10. PROPOSED FUTURE DEVELOPMENT

It is requested that any user of this program who finds an inadequacy or desirable addition or modification notify the Hydrologic Engineering Center.

TEST INPUT

```
A                                      BALANCED FLOOD HYDROGRAPHS
A                                      FEATHER RIVER AT OROVILLE
A                                               TEST RUN
B     120        72         6         3
C   42000     40000     41000     45000     54000     69000     95000    131000    163000    170000
C  158000    148000    133000    118000    107000     99000     92000     90000     86000     85000
C   88000     99000    128000    149000    173000    189000    208000    230000    254000    277000
C 300        326000    361000    402000    428000    440000    431000    411000    395000    353000
C  322000    296000    281000    268000    254000    238000    224000    210000    199000    189000
C  180000    172000    164000    158000    151000    145000    139000    134000    130000    126000
C  122000    118000    114000    111000    107000    104000    100000     97000     94000     91000
C   87000     84000
D       2         6        12        24        72       120
E                                         100-YEAR FLOOD
F  310000    291000    275000    250000    175000    143000
E                                          50-YEAR FLOOD
F  260000    247000    233000    212000    145000    120000
E                                          20-YEAR FLOOD
F  197000    187000    176000    160000    111000     91000
```

EXHIBIT 1

TEST OUTPUT

BALANCED FLOOD HYDROGRAPHS
FEATHER RIVER AT OROVILLE
TEST RUN

TR	NQ	NDUR	NHGT
120	72	6	3

100-YEAR FLOOD

DUR HOURS	QAV GIVEN	QAV COMP
2	310000	310000
6	291000	291000
12	275000	275000
24	250000	250000
72	175000	175000
120	143000	143000

ADJUSTED FLOWS IN CFS

107416	133655	139395	129555	121355	109056	96756	87737	81177	75437
73797	70517	69697	72157	81177	103919	103841	120567	131718	144960
160292	177018	193047	206844	224770	248902	258571	280517	310000	282483
264360	254069	243386	222012	204086	195835	186775	177018	165867	156110
146353	138687	131718	125446	119870	114295	110114	105235	101054	96872
93387	91504	103316	100036	96756	93476	91017	87737	85277	81997

EXHIBIT 2

50-YEAR FLOOD

DUR HOURS	QAV GIVEN	QAV COMP
2	260000	260000
6	247000	247000
12	233000	233000
24	212000	212000
72	145000	145000
120	120000	120000

ADJUSTED FLOWS IN CFS

93316	116110	121097	112549	105425	94740	84055	76220	70521	65535
64110	61261	60548	62685	70521	84299	84236	97743	106783	117518
129948	143507	156502	175587	190805	211290	218637	239660	260000	241340
223532	214830	206608	188464	173246	158762	151417	143507	134468	126558
118648	112433	106783	101698	97178	92658	89268	85313	81924	78534
75709	79443	89698	86905	84055	81206	79069	76220	74083	71233

20-YEAR FLOOD

DUR HOURS	QAV GIVEN	QAV COMP
2	197000	197000
6	187000	187000
12	176000	176000
24	160000	160000
72	111000	111000
120	91000	91000

ADJUSTED FLOWS IN CFS

68997	85851	89538	83218	77951	70050	62150	56356	52143	48456
47403	45296	44769	46349	52143	62330	62284	75828	82841	91169
100811	111331	121412	132380	143853	159297	164727	181364	197000	182636
168415	161858	155767	142088	130615	123165	117467	111331	104318	98182
92045	87224	82841	78896	75389	71883	69253	66185	63555	60925
58734	61630	69586	64257	62150	60043	58463	56356	54776	52670

DEFINITIONS - 23-J2-L237

DUR	-	Duration in hours
FLOWS	-	Number of flow ordinates in hydrographs
I	-	Various index values, flow sequence number
IADJ	-	Positive value calls for new iteration to adjust flow magnitudes
IDUR	-	Duration in TR units
J1	-	Flow serial number at start of period of maximum flow for each duration
J11	-	Flow serial number at start of period of maximum flow for maximum duration
J2	-	Flow serial number at end of period of maximum flow for each duration
K1	-	Flow serial number at start of any period of flow for each duration
K2	-	Flow serial number at end of any period of flow for each duration
L	-	Serial number of durations
NDUR	-	Number of durations
NHG	-	Number of hydrographs balanced thus far
NHGT	-	Total number of hydrographs to be balanced
NQ	-	Number of flows in hydrograph
NSTPS	-	Number of periods in hydrograph to be examined for each duration
Q	-	Flow ordinate of given and subsequently of adjusted hydrograph
Q2	-	Flow ordinate of adjusted hydrograph
QAV	-	Average flow in cfs for each duration
QMAX	-	Maximum of flow averages for given duration
RATIO	-	Ratio of desired flow to given flow
SUMQ	-	Sum of given ordinates within a given duration that have not already been adjusted for shorter duration
TR	-	Tabulation interval in minutes
VOL	-	Sum of desired ordinates within a given duration that have not already been adjusted for shorter duration

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EXHIBIT 4

LISTING OF SOURCE PROGRAM

```
C 23-J2-J237  BALANCED HYDROGRAPH, HYDROLOGIC ENGR CENTER, 14 JULY 66 LRB
C        SUBROUTINES USED - NONE
      DIMENSION Q(480),Q2(480),DUR(10),FLOWS(10),VOL(10),QAV(10),
     1QMAX(10),RATIO(10)
  109 FORMAT (1H1)
C                            THREE TITLE CARDS
      KQ=480
      KDUR=10
   10 READ 20,(Q(I),I=1,120)
      PRINT 109
      PRINT 20,(Q(I),I=1,120)
   20 FORMAT (1X,A1,9A2,15A2,15A2)
      READ 21,TR,NQ,NDUR,NHGT
   21 FORMAT (1X,F7.0,3I8)
      IF (TR) 25,25,26
   22 PRINT 24
   24 FORMAT (19H DIMENSION EXCEEDED)
   25 STOP
   26 IF (KQ-NQ) 22,28,28
   28 IF (KDUR-NDUR) 22,30,30
   30 READ 40,(Q(I),I=1,NQ)
      READ 40,(DUR(L),L=1,NDUR)
   40 FORMAT (1X,F7.0,9F8.0)
   50 FORMAT (1X,I7,9I8)
      PRINT 60
   60 FORMAT(//32H      TR      NQ     NDUR     NHGT)
      PRINT 21,TR,NQ,NDUR,NHGT
      NHG = 0
C                                      FLOOD TITLE CARD
   70 READ 20,(Q2(I),I=1,40)
      PRINT 80
   80 FORMAT (/1H )
      PRINT 20,(Q2(I),I=1,40)
C                         SPECIFY AVG FLOWS FOR DESIRED FLOOD
      READ 40,(QAV(L),L=1,NDUR)
      IADJ = 1
   90 DO 100 I=1,NQ
  100 Q2(I) = 0.
      J11 = NQ
      DO 210 L=1,NDUR
      IDUR = DUR(L)*60./TR


      FLOWS(L) = IDUR
```

EXHIBIT 4

2

```
      FLOWS(L) = IDUR
      VOL(L) = FLOWS(L)*QAV(L)
      NSTPS = NQ-IDUR+1
      QMAX(L) = 0.
C               DETERMINE PERIOD AND MAX VOL FOR EACH DURATION
      DO 130 K1=1,NSTPS
      K2 = K1+IDUR-1
      SUMQ = 0.
      DO 110 I=K1,K2
  110 SUMQ = SUMQ+Q(I)
      IF(SUMQ-QMAX(L)) 130,130,120
  120 QMAX(L) = SUMQ
      J1 = K1
  130 CONTINUE
      IF(IADJ) 210,210,140
  140 J2 = J1+IDUR-1
      IF(J1-J11)150,160,160
C     J11 IS NUMBER OF FIRST GIVEN ORDINATE FOR LONGEST DURATION
  150 J11 = J1
  160 SUMQ = 0.
C             COMPUTE GIVEN AND DESIRED VOLS OF UNADJUSTED FLOWS
      DO 190 I=J1,J2
      IF(Q2(I))170,170,180
  170 SUMQ = SUMQ+Q(I)
      GO TO 190
  180 VOL(L) = VOL(L)-Q2(I)
  190 CONTINUE
      RATIO(L) = VOL(L)/SUMQ
C                       ADJUST REMAINING UNADJUSTED FLOWS
      DO 210 I=J1,J2
      IF(Q2(I))200,200,210
  200 Q2(I) = Q(I)*RATIO(L)
  210 CONTINUE
      IF(IADJ)270,270,220
  220 IADJ = 0
C               IF ANY ADJUSTMENT EXCEEDED 5 PERCENT, READJUST
      DO 250 L=1,NDUR
      IF(RATIO(L)-1.05)230,230,240
  230 IF(RATIO(L)-.95)240,240,250
  240 IADJ = 1
  250 CONTINUE
```

```
      DO 260 I=1,IDUR
      J=I+J11-1
  260 Q(I) = Q2(J)
      NQ=IDUR
      GO TO 90
  270 PRINT 280
  280 FORMAT(/24H    DUR      QAV       QAV)
      PRINT  290
  290 FORMAT(24H   HOURS   GIVEN    COMP)
      DO 300 L=1,NDUR
      QMAX(L) = QMAX(L)/FLOWS(L)
  300 PRINT 40,DUR(L),QAV(L),QMAX(L)
      PRINT 310
  310 FORMAT(/22H ADJUSTED FLOWS IN CFS)
      PRINT 40,(Q(I),I=1,NQ)
      NHG = NHG+1
      IF(NHG-NHGT) 70,10,10
      END
```


EXHIBIT 4

INPUT DATA 23-J2-L237

A. Three output title cards for job

B. Specification card

1. TR - Tabulation interval in minutes
2. NQ - Number of flows in given hydrograph (up to 480, unless dimension increased)
3. NDUR - Number of durations specified
4. NHGT - Number of balanced hydrographs to be obtained from same given hydrograph

C. Hydrograph cards

Q - Consecutive flows of given hydrograph, NQ values

E. Durations card

DUR - Respective durations in hours for which average flows are to be given. Number must equal NDUR

E. Flood output title card

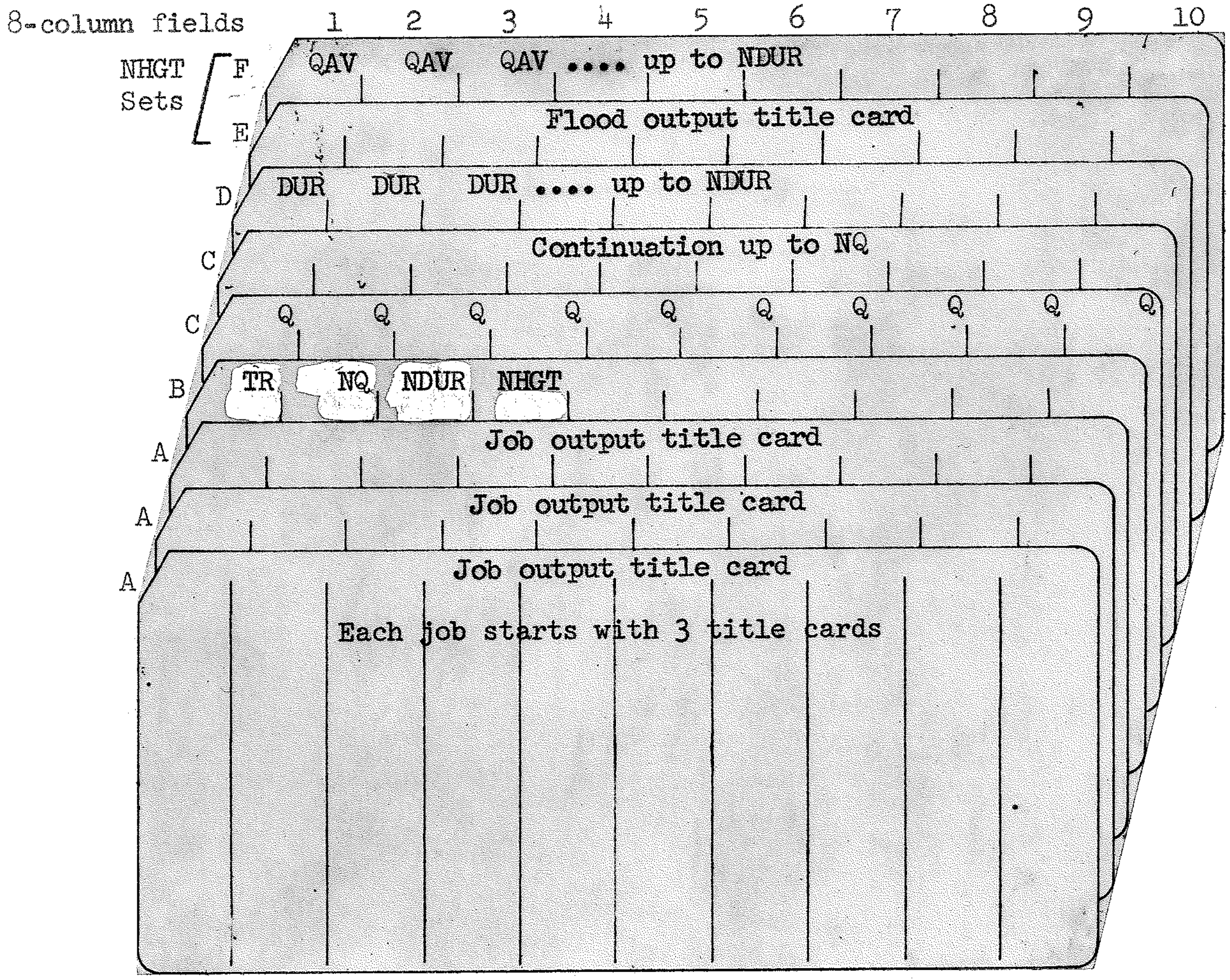
F. Average flows

QAV - Respective given average flows corresponding to durations in IV, NDUR values

EXHIBIT 5

SUMMARY OF REQUIRED CARDS

Four blank cards at end of job stop computer.
Otherwise, computer will read cards A for next job.



Note:

All data are read using 10 fields of 8 columns each, except that col 1 of each card is reserved for card identification, so first field is restricted to 7 columns.

EXHIBIT 6

Appendix 4

Streamflow Routing Optimization

STREAMFLOW ROUTING OPTIMIZATION

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L231

NOVEMBER 1966

U. S. ARMY ENGINEER DISTRICT
650 CAPITOL MALL
SACRAMENTO, CALIFORNIA

STREAMFLOW ROUTING OPTIMIZATION

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L231

CONTENTS

EXHIBITS

STREAMFLOW ROUTING OPTIMIZATION

HYDROLOGIC ENGINEERING CENTER
COMPUTER PROGRAM 23-J2-L231

1. ORIGIN OF PROGRAM

This program was prepared in the Hydrologic Engineering Center, Corps of Engineers, 650 Capitol Mall, Sacramento, California, by Leo R. Beard. Up-to-date information and copies of source statement cards for various types of computers can be obtained from the Center upon request by Government and cooperating organizations.

2. PURPOSE OF PROGRAM

a. This program written in Fortran II will solve by successive approximations for any number of cases, in turn, for the optimum Muskingum routing coefficients, K and X, to reproduce observed outflow hydrographs for a number of floods, given the observed outflow hydrographs and observed inflow hydrographs at one point on the main stem and hydrographs at each of any number of streams that are tributary below that point on the main stem, and given the number of combination points and relative travel times from each point to the next downstream combining points. Local (intermediate) runoff may be given or computed as a ratio to a given hydrograph so that it will equal in volume the difference between observed outflow and inflows, if positive, or it may be ignored. The program is not effective where local runoff is large in comparison to measured inflows or where one measured inflow is small in comparison to the outflow.

b. A listing of the source program is given in Exhibit 4.

3. DESCRIPTION OF EQUIPMENT

This program was prepared for use in the IBM 1620 computer with 40,000 digit, variable word length memory, card input and output, and is usable in the GE 225 and RCA 301 computers having comparable memory, if any necessary input or output statement changes are made. For complex problems, the run time is so long that it is advisable to use a high speed computer of the 7090 class.

4. METHODS OF COMPUTATION

a. The Muskingum routing method is described in EM 1110-2-1408 and many hydrology texts and handbooks. This program estimates the average translation time in hours, time, between the inflow and outflow hydrographs. The reach is then divided into a number of subreaches equal to the nearest integer to TIME/TRHR, so that the travel time in each subreach is about equal to the routing interval in hours (TRHR). The initial estimate of AK (Muskingum K) for each subreach is TRHR and the initial estimate of each Muskingum X is 0.2. Routings are then accomplished by use of the following standard equations:

$$C1 = 2\,(TRHR)/(2AK(1-X) + TRHR) \qquad (1)$$

$$C2 = (TRHR - 2AK(X))/(2AK(1-X) + TRHR) \qquad (2)$$

$$O_2 = O_1 + C1\,(I_1 - O_1) + C2(I_2 - I_1) \qquad (3)$$

b. If local inflow is not specified, its volume for each flood is made equal to the difference between outflow and total given inflow for that flood, and it is determined by multiplying a specified hydrograph by a constant to obtain that volume. Since the volume determination ignores travel time and storage effects, it is important to encompass all important flows in selecting the flood period, if local inflow is appreciable.

c. Successive approximations of X and AK in turn for each inflow station in turn are made with the objective of minimizing the sum of squares of the differences between each successive observed outflow and the corresponding total of local inflow and routed flows for that period. This is accomplished by computing the increments in standard error resulting from increasing a variable by one percent in each of 2 successive steps. If the second increment is algebraically smaller than the first, divergence is indicated, and a maximum adjustment of the variable (factor of 1.5) is made in the direction that reduces the standard error. If the second increment is algebraically larger, convergence is indicated, and the optimum adjustment is made (by use of Newton-Raphson technique) as follows:

$$X' = X\,(1.005 - .01\ DSER1/DIF2) \qquad (4)$$

Where X stands for any routing constant and DSER1 is the standard error increment for the first 1 percent increase and DIF2 is the difference between the second and first increment. The adjustment is limited to a factor of 1.5 in each step, and values of AK and X are limited as follows in order that C2 in equation 2 will not be negative:

$$X \leq TRHR/2AK \qquad (5)$$

$$AK \leq TRHR/2X \qquad (6)$$

d. Every change is checked before it is accepted to make sure that it does decrease the standard error. If not, the change is reduced 70 percent. This is repeated once if divergence still exists. If the standard error is still larger than the test value after two adjustments of the change, the variable is set back to its value before the change, and the next variable is considered.

e. Optimization is declared after six approximations of all routing constants (6 cycles). During the first 3 cycles, standard error is based on the differences between combined and observed flows, but during the last 3 cycles, combined flows are multiplied by a constant to make their volume equal the observed volume. This is done only for the standard error computation (not for output) so that positive errors and negative errors will balance. It is not done during the first 3 cycles, because the procedure might impede convergence. The ratio used is printed out.

5. INPUT

a. Input is summarized in Exhibits 5 and 6. Items C to F of Exhibit 1 must be consecutive for each flood. Input for each new optimization problem will be read as soon as the preceding computation is completed.

b. All data are entered consecutively on each card, using 8 columns (digits, including decimal point, if used) per variable and 10 variables per card, unless fewer are called for.

6. OUTPUT

a. Input data except flows and number of flows for each inflow station.

b. Standard error of each flood and of all floods for each iteration.

c. Each change of coefficient.

d. Sum of routed flows, local flow, total of routed and local flow, and observed flow.

e. Optimized coefficients, K, X, C_1 and C_2.

f. Ratio of routed plus local volume to observed outflow volume for each flood.

7. OPERATING INSTRUCTIONS

Standard Fortran II operating instructions. No sense switches used.

8. DEFINITION OF TERMS

Terms used in this program are defined in Exhibit 3.

9. EXAMPLE

Examples of two applications of the program are shown in Exhibits 1 and 2.

10. PROPOSED FUTURE DEVELOPMENT

a. It is anticipated that additions to this program will be made from time to time. Use of alternative routing methods is contemplated.

b. It is requested that any user of this program who finds an inadequacy or desirable addition or modification notify the Hydrologic Engineering Center.

TEST INPUT

```
A                        STREAMFLOW OPTIMIZATION TEST
A                          PLATE 3 EM 1110-2-1408
A                                JUNE 1965
B    720       1              1
C      1
E      2      26   1929    0000      15
F   2200   14500  28400   31800   29700   25300   20400   16300   12600    9300
F   6700    5000   4100    3600    2400
H   2000    7000  11700   16500   24000   29100   28400   23800   19400   15300
H  11200    8200   6400    5200    4600
A                      STREAMFLOW OPTIMIZATION TEST NO 2
A                               RED RIVER BASIN
A                             3 INFLOW STATIONS
B   1440       3      4       2       1
C      1       2      3
D      1       2      2       3
E      1       1     50    0000      59
F   7720    8230  12000   21200   23100   24500   25000   23100   19500   16300
F  15500   15500  21700   51200   75400   72600   57400   47000   38000   28800
F  20400   14600  11200    9370    8260    7610    6750    6960    6750    6540
F   6750    9580  45600   69300   63700   47900   34700   24200   16900   13700
F  12000   11200  14000   60400  100000  107000   88400   64600   48700   34100
F  24200   18600  13400   10300   10300   10500    9820    8700    8040   36700
F   5620    6920  20600   28000   31100   30300   27400   23600   18600   15000
F  16600   15500  27400   55300   47500   40900   35200   27100   21100   14500
F   8470    5510   4580    4160    3640    3220    3620    5020    4320    3900
F   4280   21300  44100   38100   29400   24700   19600   13000    7460    5100
F   4240    3560  17200   67900   69200   56400   41700   27000   18200    9770
F   5450    4070   3900    5960    7000    6010    4840    3980    3300     286
F   1090    3610   8940    5350    4410    2310    1480    1140     916    6630
F   5280    2600  17600    7160    3280    2920    1920    1420    1090     869
F    712     630    574     520     455     440     846     892     778     670
F   1310   14200   7230    3180    1800    1230     965     800     650     538
F    440     384  17900   13700    3360    1760    1230     965     778     650
F    538     471    691     916     734     611     504     411     358      42
G    577    1880   1500    1570    1070     835     553     420     344    3110
G   1280    1290   3870    1500    1310    1070     696     544     390     301
G    254     237    217     200     169     156     220     179     166     163
G    557    3310   2410    1070     696     485     380     305     251     211
G    174     156   4000    1990    1010     644     468     369     292     237
G    197     187    211     161     142     124     114     100      95       6
```

EXHIBIT - 1

EXHIBIT 1

H	18800	16900	19200	32000	40000	46400	51200	53600	51800	47600
H	45800	46400	51200	73800	94600	104000	108000	98800	91700	82800
H	71000	60800	50000	41600	35500	30000	23200	17500	15100	13500
H	13800	19200	56600	84400	95500	88800	81200	69800	59000	54200
H	49400	45800	45800	72400	125000	130000	134000	127000	116000	99900
H	86000	71700	55400	41600	36500	34000	29500	24400	19200	38400
E	6	1	51	0000	29					
F	36700	35300	30700	38200	30700	27700	48600	59800	45200	39700
F	59800	79600	86200	88400	92800	100000	102000	93500	73500	58800
F	49100	46600	44000	40300	38400	3700	37500	37300	37200	6780
F	262	256	238	247	546	705	2080	5200	7500	13200
F	17800	22600	23100	28200	32000	31100	24800	18000	11600	8820
F	8710	6980	4550	2960	2150	1680	1400	1160	960	5900
F	38	39	37	37	49	56	77	255	397	322
F	993	1250	990	990	800	592	411	300	225	182
F	154	130	108	93	79	69	64	56	57	1360
G	6	5	7	11	13	13	18	104	113	80
G	641	418	300	314	228	175	127	93	72	59
G	48	39	31	24	20	17	15	12	28	548
H	37800	37300	34000	36800	34600	28800	42000	60500	56600	51000
H	61800	78000	90500	97700	102000	112000	121000	121000	111000	96500
H	83000	73200	66400	61800	57200	52800	49200	46800	45600	12100

TEST OUTPUT

```
                 STREAMFLOW OPTIMIZATION TEST
                   PLATE 3 EM 1110-2-1408
                         JUNE 1965

  TR NISTA  LSTA NFLDS NCOMB IPNCH     A     T1     T2     T3     T4     T5     T6
 720     1    -0     1    -0    -0    10      1
STA 1   COMP 1  STD ERR=    3610    3610
STA 1   COMP 2  STD ERR=    3613    3613
STA 1   COMP 3  STD ERR=    3615    3615
X FOR STA 1  CHANGED FROM   .200   TO  .134
STA 1   COMP 1  STD ERR=    3533    3533
STA 1   COMP 2  STD ERR=    3495    3495
STA 1   COMP 3  STD ERR=    3458    3458
AK FOR STA 1  CHANGED FROM 12.00  TO 18.00
STA 1   COMP 3  STD ERR=    2004    2004
X FOR STA 1  CHANGED FROM   .200   TO  .300
STA 1   COMP 1  STD ERR=    2310    2310
STA 1   COMP 2  STD ERR=    2280    2280
STA 1   COMP 3  STD ERR=    2252    2252
AK FOR STA 1  CHANGED FROM 18.00  TO 20.00
STA 1   COMP 3  STD ERR=    2051    2051
X FOR STA 1  CHANGED FROM   .200   TO  .300
STA 1   COMP 1  STD ERR=    2051    2051
STA 1   COMP 2  STD ERR=    2034    2034
STA 1   COMP 3  STD ERR=    2019    2019
AK FOR STA 1  CHANGED FROM 20.00  TO 20.00
STA 1   COMP 3  STD ERR=    2061    2061
X FOR STA 1  CHANGED FROM   .200   TO  .202
STA 1   COMP 1  STD ERR=    1788    1788
STA 1   COMP 2  STD ERR=    1769    1769
STA 1   COMP 3  STD ERR=    1752    1752
AK FOR STA 1  CHANGED FROM 20.00  TO 22.17
STA 1   COMP 3  STD ERR=    1684    1684
X FOR STA 1  CHANGED FROM   .200   TO  .271
STA 1   COMP 1  STD ERR=    1859    1859
X FOR STA 1  CHANGED FROM   .200   TO  .221
STA 1   COMP 1  STD ERR=    1720    1720
STA 1   COMP 2  STD ERR=    1721    1721
STA 1   COMP 3  STD ERR=    1725    1725
AK FOR STA 1  CHANGED FROM 22.17  TO 22.19
```

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EXHIBIT 2

```
STA 1  COMP 3  STD ERR=     1720    1720
X FOR STA 1  CHANGED FROM  .200  TO  .134
STA 1  COMP 1  STD ERR=     1621    1621
STA 1  COMP 2  STD ERR=     1619    1619
STA 1  COMP 3  STD ERR=     1623    1623
AK FOR STA 1  CHANGED FROM 22.19  TO 22.37
STA 1  COMP 3  STD ERR=     1617    1617
X FOR STA 1  CHANGED FROM  .200  TO  .268
STA 1  COMP 1  STD ERR=     1853    1853
X FOR STA 1  CHANGED FROM  .200  TO  .220
STA 1  COMP 1  STD ERR=     1720    1720
X FOR STA 1  CHANGED FROM  .200  TO  .206
STA 1  COMP 1  STD ERR=     1691    1691
X FOR STA 1  CHANGED FROM  .200  TO  .200
STA 1  COMP 1  STD ERR=     1680    1680
FLOOD STARTING    2   26 1929     0  RATIO COMP TO OBS VOL=    .99  RTIOL=   0.
```

PERIOD	ROUTD Q	LOCAL	TOTAL	OBS Q	ROUTD1	ROUTD2	ROUTD3	ROUTD4	ROUTD5	ROUTD10
1	2200	0	2200	2000	2200					
2	2985	0	2985	7000	2985					
3	9654	0	9654	11700	9654					
4	19284	0	19284	16500	19284					
5	25434	0	25434	24000	25434					
6	27295	0	27295	29100	27295					
7	25981	0	25981	28400	25981					
8	22917	0	22917	23800	22917					
9	19358	0	19358	19400	19358					
10	15754	0	15754	15300	15754					
11	12348	0	12348	11200	12348					
12	9403	0	9403	8200	9403					
13	7135	0	7135	6400	7135					
14	5579	0	5579	5200	5579					
15	4509	0	4509	4600	4509					

STA	AK	X	C1	C2	NRCHS
1	22.37	.200	.502	.064	1

ALTERNATIVE COEFFICIENTS (SEE EQUA 14 AND PLATE 2--EM 1110-2-1408)

STA	C1	C2	C3	C4	C5	C6	C7	NRCHS
1	.064	.470	.234	.117	.058	.029	.029	1

```
                STREAMFLOW OPTIMIZATION TEST NO 2
                        RED RIVER BASIN
                      3 INFLOW STATIONS

  TR NISTA  LSTA NFLDS NCOMB IPNCH     A    T1    T2    T3    T4    T5    T6
 1440    3     4     2     1    -0    10     1     2     3
T=   1.0   N= 2   ISTA   2   3
STA 1  COMP 1  STD ERR=   17502    8643   15165
STA 1  COMP 2  STD ERR=   17509    8647   15171
STA 1  COMP 3  STD ERR=   17515    8651   15177
X FOR STA 1  CHANGED FROM  .200  TO  .134
STA 1  COMP 1  STD ERR=   17290    8528   14980
STA 1  COMP 2  STD ERR=   17244    8493   14938
STA 1  COMP 3  STD ERR=   17198    8459   14896
AK FOR STA 1  CHANGED FROM 24.00  TO 36.00
STA 2  COMP 3  STD ERR=   15229    7012   13103
X FOR STA 2  CHANGED FROM  .200  TO  .300
STA 2  COMP 1  STD ERR=   15444    7065   13281
STA 2  COMP 2  STD ERR=   15428    7055   13266
STA 2  COMP 3  STD ERR=   15411    7044   13251
AK FOR STA 2  CHANGED FROM 24.00  TO 36.00
STA 3  COMP 3  STD ERR=   14638    6558   12563
X FOR STA 3  CHANGED FROM  .200  TO  .300
STA 3  COMP 1  STD ERR=   14601    6560   12534
STA 3  COMP 2  STD ERR=   14605    6560   12538
STA 3  COMP 3  STD ERR=   14610    6560   12542
AK FOR STA 3  CHANGED FROM 24.00  TO 16.08
STA 4  COMP 3  STD ERR=   14448    6554   12414
X FOR STA 4  CHANGED FROM  .200  TO  .134
STA 4  COMP 1  STD ERR=   14341    6517   12324
STA 4  COMP 2  STD ERR=   14326    6506   12311
STA 4  COMP 3  STD ERR=   14312    6495   12298
AK FOR STA 4  CHANGED FROM 24.00  TO 36.00
STA 1  COMP 3  STD ERR=   13662    5995   11704
X FOR STA 1  CHANGED FROM  .200  TO  .300
STA 1  COMP 1  STD ERR=   14710    6824   12666
X FOR STA 1  CHANGED FROM  .200  TO  .230
STA 1  COMP 1  STD ERR=   14221    6437   12217
STA 1  COMP 2  STD ERR=   14212    6429   12208
STA 1  COMP 3  STD ERR=   14203    6422   12200
```


EXHIBIT 2

```
AK FOR STA 1  CHANGED FROM 36.00  TO 43.58
STA 2  COMP 3  STD ERR=   14109    6386   12120
X FOR STA 2  CHANGED FROM  .200  TO  .300
STA 2  COMP 1  STD ERR=   14109    6386   12120
STA 2  COMP 2  STD ERR=   14104    6383   12116
STA 2  COMP 3  STD ERR=   14099    6381   12111
AK FOR STA 2  CHANGED FROM 36.00  TO 40.00
STA 3  COMP 3  STD ERR=   14056    6358   12074
X FOR STA 3  CHANGED FROM  .200  TO  .300
STA 3  COMP 1  STD ERR=   14056    6358   12074
STA 3  COMP 2  STD ERR=   14059    6359   12077
STA 3  COMP 3  STD ERR=   14062    6359   12079
AK FOR STA 3  CHANGED FROM 16.08  TO 10.77
STA 4  COMP 3  STD ERR=   13966    6348   12002
X FOR STA 4  CHANGED FROM  .200  TO  .134
STA 4  COMP 1  STD ERR=   13966    6348   12002
STA 4  COMP 2  STD ERR=   13960    6343   11997
STA 4  COMP 3  STD ERR=   13954    6339   11991
AK FOR STA 4  CHANGED FROM 36.00  TO 54.00
STA 1  COMP 3  STD ERR=   13714    6137   11769
X FOR STA 1  CHANGED FROM  .200  TO  .275
STA 1  COMP 1  STD ERR=   14063    6455   12096
X FOR STA 1  CHANGED FROM  .200  TO  .223
STA 1  COMP 1  STD ERR=   13661    6088   11719
STA 1  COMP 2  STD ERR=   13691    6119   11748
STA 1  COMP 3  STD ERR=   13723    6151   11778
AK FOR STA 1  CHANGED FROM 43.58  TO 29.20
STA 2  COMP 3  STD ERR=   12915    5465   11031
X FOR STA 2  CHANGED FROM  .200  TO  .134
STA 2  COMP 1  STD ERR=   12580    5302   10741
STA 2  COMP 2  STD ERR=   12559    5283   10721
STA 2  COMP 3  STD ERR=   12539    5263   10702
AK FOR STA 2  CHANGED FROM 40.00  TO 60.00
STA 3  COMP 3  STD ERR=   11828    4447   10016
X FOR STA 3  CHANGED FROM  .200  TO  .300
STA 3  COMP 1  STD ERR=   11828    4447   10016
STA 3  COMP 2  STD ERR=   11829    4447   10017
STA 3  COMP 3  STD ERR=   11830    4447   10018
AK FOR STA 3  CHANGED FROM 10.77  TO  7.22
STA 4  COMP 3  STD ERR=   11800    4443    9993
X FOR STA 4  CHANGED FROM  .200  TO  .134
```

EXHIBIT 2

```
STA 4  COMP 1  STD ERR=   11800    4443    9993
STA 4  COMP 2  STD ERR=   11788    4423    9980
STA 4  COMP 3  STD ERR=   11775    4404    9968
AK FOR STA 4  CHANGED FROM 54.00  TO 76.96
STA 1  COMP 3  STD ERR=   11526    3853    9693
X FOR STA 1  CHANGED FROM  .200  TO  .300
STA 1  COMP 1  STD ERR=   11671    4230    9860
STA 1  COMP 2  STD ERR=   11697    4266    9886
STA 1  COMP 3  STD ERR=   11724    4302    9912
AK FOR STA 1  CHANGED FROM 29.20  TO 19.56
STA 2  COMP 3  STD ERR=   11322    3566    9494
X FOR STA 2  CHANGED FROM  .200  TO  .134
STA 2  COMP 1  STD ERR=   11323    3564    9494
STA 2  COMP 2  STD ERR=   11318    3539    9487
STA 2  COMP 3  STD ERR=   11313    3516    9480
AK FOR STA 2  CHANGED FROM 60.00  TO 79.66
STA 3  COMP 3  STD ERR=   11327    2953    9429
X FOR STA 3  CHANGED FROM  .200  TO  .300
STA 3  COMP 1  STD ERR=   11304    2982    9413
STA 3  COMP 2  STD ERR=   11305    2982    9413
STA 3  COMP 3  STD ERR=   11305    2982    9414
AK FOR STA 3  CHANGED FROM  7.22  TO  6.83
STA 4  COMP 3  STD ERR=   11303    2982    9412
X FOR STA 4  CHANGED FROM  .200  TO  .134
STA 4  COMP 1  STD ERR=   11303    2982    9412
STA 4  COMP 2  STD ERR=   11305    2964    9412
STA 4  COMP 3  STD ERR=   11306    2949    9411
AK FOR STA 4  CHANGED FROM 76.96  TO 89.55
STA 1  COMP 3  STD ERR=   11375    2771    9449
X FOR STA 1  CHANGED FROM  .200  TO  .201
STA 1  COMP 1  STD ERR=   11326    2508    9385
STA 1  COMP 2  STD ERR=   11318    2508    9379
STA 1  COMP 3  STD ERR=   11311    2509    9373
AK FOR STA 1  CHANGED FROM 19.56  TO 21.42
STA 2  COMP 3  STD ERR=   11275    2552    9348
X FOR STA 2  CHANGED FROM  .200  TO  .151
STA 2  COMP 1  STD ERR=   11269    2546    9342
STA 2  COMP 2  STD ERR=   11280    2537    9351
STA 2  COMP 3  STD ERR=   11289    2530    9357
AK FOR STA 2  CHANGED FROM 79.66  TO 53.37
STA 3  COMP 3  STD ERR=   11119    3223    9290
```


EXHIBIT 2

```
X FOR STA 3  CHANGED FROM  .200   TO  .134
STA 3  COMP 1  STD ERR=   11159    3121     9311
STA 3  COMP 2  STD ERR=   11160    3121     9312
STA 3  COMP 3  STD ERR=   11161    3121     9313
AK FOR STA 3  CHANGED FROM  6.83  TO  5.59
STA 4  COMP 3  STD ERR=   11150    3122     9304
X FOR STA 4  CHANGED FROM  .200   TO  .134
STA 4  COMP 1  STD ERR=   11149    3122     9304
STA 4  COMP 2  STD ERR=   11146    3093     9298
STA 4  COMP 3  STD ERR=   11142    3067     9292
AK FOR STA 4  CHANGED FROM 89.55  TO 89.55
STA 1  COMP 3  STD ERR=   11147    3127     9302
X FOR STA 1  CHANGED FROM  .200   TO  .202
STA 1  COMP 1  STD ERR=   11150    3124     9304
X FOR STA 1  CHANGED FROM  .200   TO  .201
STA 1  COMP 1  STD ERR=   11149    3121     9303
STA 1  COMP 2  STD ERR=   11149    3124     9303
STA 1  COMP 3  STD ERR=   11149    3129     9304
AK FOR STA 1  CHANGED FROM 21.42  TO 21.43
STA 2  COMP 3  STD ERR=   11149    3121     9303
X FOR STA 2  CHANGED FROM  .200   TO  .134
STA 2  COMP 1  STD ERR=   11153    3121     9306
X FOR STA 2  CHANGED FROM  .200   TO  .180
STA 2  COMP 1  STD ERR=   11144    3123     9300
STA 2  COMP 2  STD ERR=   11143    3100     9296
STA 2  COMP 3  STD ERR=   11140    3081     9292
AK FOR STA 2  CHANGED FROM 53.37  TO 66.59
STA 3  COMP 3  STD ERR=   11167    2717     9276
X FOR STA 3  CHANGED FROM  .200   TO  .300
STA 3  COMP 1  STD ERR=   11143    2736     9258
STA 3  COMP 2  STD ERR=   11142    2736     9258
STA 3  COMP 3  STD ERR=   11143    2736     9258
AK FOR STA 3  CHANGED FROM  5.59  TO  6.01
STA 4  COMP 3  STD ERR=   11144    2736     9259
X FOR STA 4  CHANGED FROM  .200   TO  .134
STA 4  COMP 1  STD ERR=   11145    2736     9259
X FOR STA 4  CHANGED FROM  .200   TO  .180
STA 4  COMP 1  STD ERR=   11035    2730     9170
STA 4  COMP 2  STD ERR=   11044    2705     9175
STA 4  COMP 3  STD ERR=   11048    2689     9177
```

EXHIBIT 2

```
AK FOR STA 4   CHANGED FROM 89.55   TO 60.00
STA 1   COMP 3   STD ERR=    11250      3837      9472
X FOR STA 1   CHANGED FROM   .200   TO   .201
STA 1   COMP 1   STD ERR=    11271      3720      9473
X FOR STA 1   CHANGED FROM   .200   TO   .200
STA 1   COMP 1   STD ERR=    11270      3719      9472
X FOR STA 1   CHANGED FROM   .200   TO   .200
STA 1   COMP 1   STD ERR=    11270      3718      9472
X FOR STA 1   CHANGED FROM   .200   TO   .200
STA 1   COMP 1   STD ERR=    11270      3718      9472
FLOOD STARTING     1     1     50     0   RATIO COMP TO OBS VOL=  1.00   RTIOL=  11.286
```

PERIOD	ROUTD Q	LOCAL	TOTAL	OBS Q	ROUTD1	ROUTD2	ROUTD3	ROUTD4	ROUTD5 — ROUTD10
1	14430	6512	20942	18800	7720	5620	1090	6710	
2	14586	21219	35804	16900	7855	5620	2156	6731	
3	16419	16930	33348	19200	9162	6089	7662	7257	
4	23833	17720	41553	32000	13935	11318	7296	9898	
5	33788	12077	45864	40000	20421	17330	3823	13366	
6	39507	9424	48931	46400	22998	22293	3371	16508	
7	44482	6241	50724	51200	24367	25179	1278	20115	
8	47005	4740	51745	53600	24385	25979	1363	22620	
9	46820	3883	50702	51800	22374	25122	892	24446	
10	44246	35101	79347	47600	19160	22771	4522	25086	
11	42534	14447	56981	45800	16593	19971	6794	25941	
12	41857	14560	56417	46400	15693	18756	2867	26164	
13	41711	43679	85390	51200	17175	17582	11907	24536	
14	55295	16930	72225	73800	28712	21121	13769	26584	
15	83483	14785	98269	94600	53640	33439	1543	29843	
16	102823	12077	114899	104000	70820	38507	3888	32003	
17	104317	7855	112173	108000	68262	39369	1826	36056	
18	94601	6140	100741	98800	56563	37867	1651	38039	
19	84837	4402	89239	91700	46304	33986	1101	38533	
20	74118	3397	77515	82800	37029	29342	945	37088	
21	62341	2867	65207	71000	28028	23993	733	34313	
22	50653	2675	53327	60800	20210	18399	650	30443	
23	40571	2449	43021	50000	14689	13753	585	25882	
24	32620	2257	34877	41600	11331	10447	535	21289	
25	26623	1907	28531	35500	9422	8181	472	17201	
26	22109	1761	23870	30000	8293	6545	437	13816	
27	18620	2483	21103	23200	7503	5346	697	11117	
28	16058	2020	18078	17500	6938	4724	947	9120	
29	14667	1874	16540	15100	6901	4831	794	7766	

EXHIBIT 2

30	13642	1840	15482	13500	6721	4647	702	6921
31	12934	6287	19220	13800	6628	4378	1057	6306
32	13606	37358	50964	19200	7478	4342	9540	6128
33	28082	27200	55283	56600	18745	10454	12056	9337
34	61688	12077	73765	84400	47137	22580	2363	14551
35	82650	7855	90506	95500	63908	28173	2705	18742
36	83032	5474	88506	88800	59554	28515	1007	23478
37	72325	4289	76614	81200	46461	27204	1171	25864
38	60782	3442	64225	69800	33995	24464	762	26788
39	50089	2833	52922	59000	23995	20332	724	26094
40	41329	2381	43710	54200	17304	15693	544	24024
41	34780	1964	36744	49400	13886	11875	473	20895
42	29608	1761	31369	45800	12121	9124	389	17487
43	26638	45146	71784	45800	12104	7118	11400	14534
44	42253	22460	64713	72400	25950	10752	18382	16303
45	86282	11399	97681	125000	64807	31348	4946	21475
46	123132	7268	130400	130000	95645	44990	1592	27487
47	135127	5282	140409	134000	100073	49102	1508	35055
48	125251	4165	129416	127000	84158	46434	930	41093
49	107256	3296	110552	116000	63841	39430	864	43415
50	89544	2675	92219	99900	47506	31779	656	42039
51	71960	2223	74183	86000	33844	23847	577	38116
52	57033	2111	59144	71700	24419	17217	477	32615
53	44923	2381	47304	55400	18250	12479	606	26673
54	34723	1817	36540	41600	13435	9387	873	21288
55	27791	1603	29393	36500	10853	8152	822	16938
56	24252	1400	25652	34000	10450	7737	614	13802
57	21962	1287	23248	29500	10311	7114	542	11650
58	19676	1129	20805	24400	9610	6295	427	10066
59	17463	1072	18535	19200	8686	5460	2428	8777

FLOOD STARTING 6 1 51 0 RATIO COMP TO OBS VOL= .99 RTIOL= 21.328

PERIOD	ROUTD Q	LOCAL	TOTAL	OBS Q	ROUTD1	ROUTD2	ROUTD3	ROUTD4	ROUTD5 — ROUTD10
1	37000	128	37128	37800	36700	262	38	300	
2	36629	107	36736	37300	36329	262	38	300	
3	34564	149	34713	34000	34264	260	38	300	
4	33613	235	33848	36800	33314	252	37	299	
5	35648	277	35925	34600	35353	250	45	295	
6	31024	277	31301	28800	30726	357	55	297	
7	34112	384	34496	42000	33767	482	69	345	
8	49388	2218	51606	60500	48949	1058	193	440	
9	54811	2410	57221	56600	54020	2551	374	790	

EXHIBIT 2

10	46962	1706	48668	51000	45300	4335	361	1662
11	48930	13671	62602	61800	46009	7530	725	2921
12	67705	8915	76620	78000	62609	11231	1283	5096
13	86431	6398	92830	90500	78350	15328	1070	8081
14	96793	6697	103490	97700	85398	18129	951	11396
15	103514	4863	108377	102000	89035	21759	889	14479
16	111792	3732	115524	112000	94042	25450	626	17750
17	120530	2709	123239	121000	99478	27486	462	21052
18	123040	1984	125023	121000	99305	26518	317	23735
19	114119	1536	115655	111000	89229	23448	245	24890
20	96719	1258	97978	96500	72383	19178	188	24336
21	80942	1024	81966	83000	58628	15445	161	22314
22	69754	832	70586	73200	50119	13018	135	19635
23	63583	661	64244	66400	46532	10842	114	17050
24	58082	512	58594	61800	43467	8574	96	14615
25	52599	427	53026	57200	40356	6551	83	12244
26	48387	363	48749	52800	38374	4965	71	10012
27	45412	320	45732	49200	37375	3781	65	8037
28	43801	256	44057	46800	37425	2923	59	6376
29	42328	597	42925	45600	37296	2288	44	5032

STA	AK	X	C1	C2	NRCHS
1	21.43	.200	.824	.265	1
2	66.59	.180	.360	.000	3
3	6.01	.300	1.481	.629	4
4	60.00	.180	.392	.019	1

ALTERNATIVE COEFFICIENTS (SEE EQUA 14 AND PLATE 2--EM 1110-2-1408)

STA	C1	C2	C3	C4	C5	C6	C7	NRCHS
1	.265	.606	.107	.019	.003	.001	.000	1
2	.000	.360	.231	.147	.094	.060	.107	3
3	.629	.549	-.264	.127	-.061	.029	-.010	4
4	.019	.385	.234	.142	.086	.052	.081	1

DEFINITIONS - 23-J2-L231

A - Ratio greater than 1.0 representing a control on the variation of routing coefficients throughout the system

AI - Serial number of flow for each flood

AK - Muskingum K, initially assigned a value of flow tabulation (routing) interval in hours

C1 - Muskingum C_1

C2 - Muskingum C_2

CORR - Factor for adjusting C1 to optimum

CP1 - Muskingum C_1'. CP1, CP2, CP3, CP4 and CPI are alternative routing coefficients used to compute the outflow entirely in terms of inflow. See EM 1110-2-1408.

CP2 - Muskingum C_4'

CP3 - Muskingum $C_4'C_3'$

CP4 - Muskingum $C_4'(C_3')^2$

CPI - Muskingum $C_4'(C_3')^3 + C_4'(C_3')^4 + \ldots\ C_4'(C_3')^{n-2} =$ 1 - (CP1+CP2+CP3+CP4). The sum of remaining coefficients beyond CP4.

DENOM - Temporary variable

DIF2 - Second difference of standard errors of three successive computations

DSER1 - Difference between standard errors of second and first computation (NC2 and NC1)

DSER2 - Difference between standard errors of third and second computation

FIN - Indicator to declare optimization, terminate computations and print results

I - Serial number of flow for all floods in succession (job serial number)

IDAY - Day of start of flood

IFLAG - Indicator when positive that tests have been completed on the original and 2 adjusted sets of routing coefficients.

IFLG - Indicator when positive that station is contributing to downstream location

IMNTH - Month number for start of flood

INDEX - Indicator when positive that an X coefficient is considered and when zero that a K coefficient is considered

IPNCH - Indicator when positive causes diagnostic printout

IROUT - Indicator when zero that associated flows are not be be routed before adding to other flows

ISTA - Identification number of upstream station, corresponds to order number of reading inflows

ITEMP - Temporary fixed-point variable

ITIME - Time of start of flood

ITMP - Temporary fixed-point variable

IYR - Year of start of flood

J - Serial number of inflow station
JA - Variation of J
JX - Variation of J
K - Serial number of reach in routing for a given station
KCOMB - Dimensioned number of combination points
KFLOD - Dimensioned number of floods
KI - Dimensioned number of inflow locations
KQ - Dimensioned number of streamflow intervals
KSTA - Dimensioned number of stations (inflow locations plus combination points)
L - Index number for upstream station
LSTA - Station number of local inflow or of station used as an index of local inflow
M - Serial number of station whose value of C1 is currently being adjusted
M1 - Serial number of previous station whose value of C1 was adjusted
N - Serial number of flood
NADJ - Number of adjustments made that resulted in increased standard error (divergence).
NC - Computation number (complete computation for all floods)
NCOMB - Number of combination points exclusive of outflow combining point
NCYCL - Number of cycles completed (complete computation and adjustment for all stations)
NFLDS - Number of floods used
NHGT - Number of hydrographs to be combined at a combining point
NISTA - Number of inflow stations whose flows are to be routed
NQ - Number of flows (including zeros) in a given flood
NQ1 - Job serial number (I) of first flow of a given flood (one greater than last flow of previous flood)
NQN - Job serial number of last flow of a given flood
NRCH - Number of reaches of each station routing
NRCHS - Number of routing reaches between inflow or combining point to next downstream combining point
NRCHT - Total number of routing reaches in system
NRTG - NISTA + NCOMB
Q - Flow to be routed, cfs
QI - Inflow in cfs
QIMOM - Moment of all inflows to be routed (QI times AI)
QLCL - Local flow in cfs
QO - Observed outflow in cfs
QOMOM - Moment of all observed outflows
QR - Routed flow in cfs
QR1 - Routed flow at beginning of period
QRT - Total routed flow for all stations for one period
QTOTL - Total routed and local flow for one period
RATIO - Ratio of total routed and local flow to total observed outflow for each flood

RNQ - Reciprocal of number of flows for one flood
RTIO - Ratio of observed travel time to arbitrarily selected values of T
RTIOL - Ratio of excess of (a) overflow volume over inflow volume for stations to be routed, to (b) volume of flow at local inflow index station
SMQ - Grand total flow in all reaches used for computing a weighted average of k coefficients
SMQIT - Sum of inflow for all stations to be routed for all floods
SMQL - Sum of flows for local inflow index station for all floods
SMQOT - Total observed outflow for all floods
SMQRT - Sum of routed and local flows for all floods
SMSQT - Sum of squares of errors for all floods
STDER - Standard error for one computation (all floods)
STDR - Standard error for one flood
SUMQ1 - Sum of inflows to be routed for first period of one flood
SUMQI - Sum of inflows to be routed for all periods of one flood
SUMQO - Sum of observed outflows for one flood
SUMQR - Sum of routed flows for one flood
SUMSQ - Sum of squares of errors for one flood
T - Relative travel time for each station
TAVG - Average of relative travel times weighted by inflow volumes for stations
TEST - Lowest value of standard error attained so far
TIME - Average travel time determined from observed inflows and outflows
TMP - Temporary constant
TMPK - Successive approximation of k coefficient
TMPX - Successive approximation of x coefficient
TR - Tabulation and routing interval in minutes
TRHR - Same in hours
TWTD - Sum of products of T and volume for all inflows to be routed
WTDK - Weighted average of k value for all reaches
X - Muskingum routing coefficient, x

. LISTING OF SOURCE PROGRAM

```
C     PROGRAM 23-J2-J231     STREAMFLOW ROUTING OPTIMIZATION
C     HYDROLOGIC ENGINEERING CENTER                    APRIL 1966
C     INDEXES N FLOOD, J STA, I PERIOD, K REACH, M VARIABLE, NC COMP
      DIMENSION QI(900,6),QO(900),IMNTH(5),IDAY(5),IYR(5),ITIME(5),
     1NQ(5),STDR(5),STDER(3),C1(10),SUMQR(5),SUMQO(5),RATIO(5),QRT(900),
     2NRCHS(10),SUMQ1(5),SUMQI(5),T(10),AK(10),SUMI(10,5),QR(900,10),
     3C2(10),Q(900),RTIOL(5),IROUT(10),IFLG(10),TSUM(10),NHGT(10),
     4CPI(7),X(10),ISTA(4,6),SUMQJ(10)
      KSTA =10
      KI = 6
      KQ = 900
      KFLOD = 5
      KCOMB = 4
    1 FORMAT (1XF7.0,9F8.0)
    2 FORMAT (1X,I7,9I8)
  100 PRINT 107
  107 FORMAT (1H1)
      DO 110 J = 1, KSTA
      IROUT(J) = 1
  110 IFLG(J) = 0
C                             THREE TITLE CARDS
      READ 120,((QI(I,J),I=1,40),J=1,3)
      PRINT 120,((QI(I,J),I=1,40),J=1,3)
  120 FORMAT (1XA1,9A2,15A2,15A2)
      READ 122, TR,NISTA,LSTA,NFLDS,NCOMB,IPNCH,A
  122 FORMAT (1X,F7.0,5I8,F8.0)
      IF(A)124,124,125
  124 A = 10.
C      FOUR BLANK CARDS AT END OF INPUT DECK
  125 IF (TR) 140,140,145
  130 PRINT 135
  135 FORMAT(19H DIMENSION EXCEEDED)
  140 STOP
  145 IF(LSTA) 141,141,142
  141 IF(NISTA-KI) 143,143,130
  142 IF(NISTA+1-KI) 143,143,130
  143 IF(NFLDS-KFLOD) 144,144,130
  144 IF(NCOMB-KCOMB) 149,149,130
  149 READ 1, (T(J),J=1,NISTA)
      DO 160 J = 1, NISTA
      IF (T(J)) 150,150,160
```

EXHIBIT 4

EXHIBIT 4

```
 150 IROUT (J) = 0
 160 TSUM(J) = T(J)
 170 PRINT 180
 180 FORMAT(//78H    TR NISTA  LSTA NFLDS NCOMB IPNCH      A     T1     T2
    1    T3    T4    T5    T6)
     PRINT 190, TR,NISTA,LSTA,NFLDS,NCOMB,IPNCH,A,(T(J),J=1,NISTA)
 190 FORMAT (F6.0,5I6,7F6.0)
     NRTG = NISTA + NCOMB
     IF (NCOMB)220,220,200
 200 ITMP = NISTA + 1
     ITEMP = NISTA+ NCOMB
 201 FORMAT (3H T=F6.1,5H   N=I2,7H   ISTA6I4/)
     DO 210 J = ITMP, ITEMP
     JA = J - NISTA
     READ 215,T(J),N,(ISTA(JA,L),L = 1,N)
     PRINT 201, T(J),N,(ISTA(JA,L),L=1,N)
     NHGT (JA) = N
     DO 205 L = 1,N
     JX=ISTA(JA,L)
     IFLG(JX) = 1
 205 TSUM(JX) = TSUM(JX) + T(J)
 210 CONTINUE
 215 FORMAT (1XF7.0,9I8)
 220 QIMOM = 0.
     QOMOM = 0.
     SMQIT = 0.
     SMQOT = 0.
     SMQ = 0.
     TWTD = 0.
     STDER(1) = 999999.
     FIN = 0.
     NQN = 0
     TRHR = TR/60.
     DO 360 N = 1, NFLDS
     SMQL = 0.
     SUMQI(N) = 0.
     SUMQO(N) = 0.
C        READ INFLOWS AND OUTFLOWS FOR ONE FLOOD AT A TIME
     READ 225,IMNTH(N),IDAY(N),IYR(N),ITIME(N),NQ(N),RTIOL(N)
 225 FORMAT (1XI7,4I8,F8.0)
     NQ1 = NQN + 1
```

```
      NQN = NQN + NQ(N)
      SUMQ1(N) = 0.
      DO 240 J= 1,NISTA
      SUMI(J,N) = 0.
      READ 1,(QI(I,J),I=NQ1,NQN)
      SUMQ1(N) = SUMQ1(N) + QI(NQ1,J)
C                    COMPUTE SUMS AND MOMENTS
      DO 230 I=NQ1,NQN
      AI = I-NQ1+1
      SUMI(J,N) = SUMI(J,N) + QI(I,J)
  230 QIMOM = QIMOM + AI*QI(I,J)
      SUMQI(N) = SUMQI(N) + SUMI(J,N)
  240 TWTD = TWTD + TSUM(J)*SUMI(J,N)
      IF(NCOMB)249,249,241
  241 DO 245 JA = 1,NCOMB
      J = JA + NISTA
      SUMI(J,N) = 0.
      ITEMP = NHGT(JA)
      DO 244 JX = 1,ITEMP
  244 SUMI(J,N) = SUMI(J,N) +SUMI(JX,N)
  245 CONTINUE
  249 IF(LSTA) 290,290,250
  250 IF(LSTA-NISTA)270,270,260
  260 LSTA = KI
      READ 1,(QI(I,LSTA),I=NQ1,NQN)
  270 DO 280 I= NQ1, NQN
  280 SMQL = SMQL + QI(I,LSTA)
  290 SUMQO(N) = 0.
      READ 1,(QO(I),I=NQ1,NQN)
      DO 300 I = NQ1,NQN
      AI = I-NQ1+1
      SUMQO(N) = SUMQO(N)+QO(I)
  300 QOMOM = QOMOM + AI*QO(I)
      IF(LSTA)330,330,310
  310 IF (RTIOL(N))320,320,350
  320 RTIOL(N) = (SUMQO(N)-SUMQI(N))/SMQL
      IF (RTIOL(N))340,350,350
  330 LSTA = 1
  340 RTIOL(N) = 0.
  350 SMQOT = SMQOT + SUMQO(N)
  360 SMQIT = SMQIT + SUMQI(N)
```

3

EXHIBIT 4

EXHIBIT 4

```
C                   DETERMINATION OF TRAVEL TIME IN TR INTERVALS
      TAVG = TWTD/SMQIT
      TIME =(QOMOM/SMQOT - QIMOM/SMQIT)
C         DIVIDE EACH ROUTING INTO REACHES OF TRAVEL TIME TR
      RTIO = TIME/TAVG
      NRCHT = 0
      DO 390 J= 1,NRTG
      IF(T(J))365,365,367
  365 IROUT(J)=0
      GO TO 370
  367 NRCHS(J) = T(J)*RTIO + .5
      IF(NRCHS(J))370,370,380
  370 NRCHS(J) =1
  380 NRCHT = NRCHT + NRCHS(J)
      TMP = NRCHS(J)
      SUMQJ(J) = 0.
      DO 375 N=1,NFLDS
  375 SUMQJ(J) = SUMQJ(J) + SUMI(J,N)*TMP
      SMQ = SMQ + SUMQJ(J)
C                    INITIAL K = TRAVEL TIME IN HOURS PER REACH
      AK(J) = TRHR
C                INITIAL X = .2
  390 X(J) = .2
      TMPK = TRHR
      TMPX = .2
      NC=1
      INDEX = 0
      M = 1
      M1 = 0
      IFLAG = 0
      NCYCL = 0
C                             START ITERATION ROUTINE
  400 SMQRT = 0.
      SMSQT = 0.
      DO 410 I = 1, NQN
  410 QRT(I) = 0.
      NQN = 0
      DO 700 N = 1, NFLDS
      NQ1 = NQN+1
      NQ2 = NQN+2
      NQN = NQN + NQ(N)
```

```
      TMP = NQ(N)
      RNQ = 1./TMP
      IF(FIN)450,450,420
  420 ITMP = RATIO(N)*100. + .5
      TMP = ITMP
      RATIO(N) = TMP*.01
      PRINT430,IMNTH(N),IDAY(N),IYR(N),ITIME(N),RATIO(N),RTIOL(N)
  430 FORMAT ( 15H FLOOD STARTING4I5,24H  RATIO COMP TO OBS VOL=F6.2,
     18H  RTIOL=F8.3)
      PRINT 440
  440 FORMAT(/120H  PERIOD ROUTD Q   LOCAL   TOTAL   OBS Q  ROUTD1  ROUT
     1D2  ROUTD3  ROUTD4  ROUTD5  ROUTD6  ROUTD7  ROUTD8  ROUTD9 ROUTD10
     2)
  450 AI = 1.
      QTOTL = QI(NQ1,LSTA)*RTIOL(N)
      DO 456 J=1,NISTA
  456 QTOTL = QTOTL+QI(NQ1,J)
      IF(NCYCL-3)460,470,470
  460 TMP = QTOTL-QO(NQ1)
      GO TO 480
  470 TMP = QTOTL - (QO(NQ1)*RATIO(N))
C                      START ACCUMULATIONS FOR EACH FLOOD
  480 SUMSQ = TMP*TMP
      SUMQR(N) = QTOTL
      NRT =NRTG
      DO 690 J=1,NRT
      IF(J-NISTA)490,490,510
  490 DO 500 I=NQ1,NQN
  500 Q(I)=QI(I,J)
      GO TO 540
  510 JA = J-NISTA
      DO 530 I = NQ1,NQN
      Q(I) = 0.
      ITMP = NHGT(JA)
      DO 520 L= 1,ITMP
      ITEMP=ISTA(JA,L)
  520 Q(I) =Q(I) + QR(I,ITEMP)
  530 CONTINUE
  540 DENOM = 2.*AK(J)*(1.-X(J))+TRHR
      C1(J) = 2.*TRHR/DENOM
      C2(J) = (TRHR-2.*AK(J)*X(J))/DENOM
```


EXHIBIT 4

EXHIBIT 4

6

```
C                        ASSUME FIRST ROUTED FLOW = FIRST INFLOW
      QR1 = Q(NQ1)
      QR(NQ1,J)=QR1
      NRCH = NRCHS(J)
      IF(IPNCH)560,560,550
C                             PUNCH-OUT FOR DIAGNOSTIC PURPOSES ONLY
  550 PRINT 2,  NRCH,NQN,NRTG,NC,N,NCYCL,INDEX,M,M1,IFLAG
  560 DO 680 K=1,NRCH
      DO 675 I=NQ2,NQN
      IF(IROUT(J))570,570,575
  570 QR(I,J)=Q(I)
      GO TO 580
  575 AI = I-NQ1+1
      QR(I,J) = C1(J)*(Q(I-1)-QR1)+C2(J) *(Q(I)-Q(I-1))+QR1
      Q(I-1)=QR1
      QR1 = QR(I,J)
  580 IF(K-NRCH)680,585,585
C              FOLLOWING ROUTINES APPLY TO FULLY ROUTED FLOWS ONLY
C              ADD ONLY FLOWS ROUTED TO OUTFLOW POINT
  585 IF(IFLG(J))590,590,600
  590 QRT(I) = QRT(I) + QR(I,J)
  600 IF(J-NRTG)675,610,610
C              FOLLOWING ROUTINES APPLY TO TOTAL OUTFLOW ONLY
  610 QLCL = QI(I,LSTA)*RTIOL(N)
      QTOTL = QRT(I) + QLCL
      SUMQR(N) = SUMQR(N) + QTOTL
      IF(FIN)640,640,614
  614 IF(I-NQ2)615,615,616
  615 TMP =QI(NQ1,LSTA)*RTIOL(N)
      TEMP  = SUMQ1(N)+TMP
      AI=1.
      PRINT 630,AI,SUMQ1(N),TMP, TEMP ,QO(NQ1),(QR(NQ1,JX),JX=1,NRTG)
      AI=2.
  616 PRINT 630,AI,QRT(I),QLCL,QTOTL,QO(I),(QR(I,JX),JX=1,NRTG)
  630 FORMAT (15F8.0)
  640 IF(NCYCL-3)650,660,660
  650 TMP = QTOTL - QO(I)
      GO TO 670
C     AFTER 3 CYCLES, ADJUST OBSERVED VOL TO ROUTED VOL FOR STD ERR COMP
  660 TMP = QTOTL - QO(I)*RATIO(N)
  670 SUMSQ = SUMSQ + TMP*TMP
```

```
  675 CONTINUE
  680 Q (NQN) = QR1
  690 CONTINUE
      RATIO(N) = SUMQR(N)/SUMQO(N)
      STDR(N) =       (SUMSQ*RNQ)**.5
      SMQRT = SMQRT + SUMQR(N)
  700 SMSQT = SMSQT + SUMSQ
  710 FORMAT(4H STAI2,6H  COMPI2,10H  STD ERR=7F8.0)
      IF(FIN)720,720,1110
  720 TMP=NQN
      STDER(NC) =      (SMSQT/TMP)**.5
      PRINT 710,M,NC,(STDR(N),N=1,NFLDS),STDER(NC)
C              CHECK FOR DIVERGENCE WHEN NC=1 EXCEPT DURING FIRST RUN (M1=0)
      IF(NC-2)730,890,920
  730 IF(M1)760,760,740
  740 IF(STDER(1)-TEST)750,750,810
C                   OPTIMIZATION DECLARED AFTER 6 CYCLES
  750 IF(NCYCL-6)770,800,800
  760 M1=1
  770 NADJ = 0
      IFLAG = 0
      NC = 2
      TEST = STDER(1)
      IF(INDEX)790,790,780
  780 TMPK=AK(M)
      AK(M)=TMPK*1.01
      GO TO 400
  790 TMPX=X(M)
      X(M)=TMPX*1.01
      GO TO 400
  800 FIN = 1.
      GO TO 400
  810 IF(IFLAG)820,820,750
  820 IF(NADJ-2)830,860,860
  830 IF(INDEX)850,850,840
  840 X(M1)=.3*X(M1)+.7*TMPX
      PRINT 1025,M1,TMPX,X(M1)
      NADJ = NADJ + 1
      GO TO 400
  850 AK(M1)=.3*AK(M1)+.7*TMPK
      PRINT 1060,M1,TMPK,AK(M1)
```


EXHIBIT 4

EXHIBIT 4

```
      NADJ = NADJ + 1
      GO TO 400
  860 IFLAG = 1
      IF (INDEX)870,870,880
  870 AK(M1)=TMPK
      PRINT 1060,M1,TMPK,AK(M1)
      GO TO 400
  880 X(M1)=TMPX
      PRINT 1025,M1,TMPX,X(M1)
      GO TO 400
  890 NC = 3
      IF(INDEX)910,910,900
  900 AK(M)=TMPK*1.02
      GO TO 400
  910 X(M)=TMPX*1.02
      GO TO 400
C                               OPTIMIZATION
  920 DSER1 = STDER(2) - STDER(1)
      DSER2 = STDER(3) - STDER(2)
      DIF2 = DSER2 -DSER1
      IF(DIF2)930,940,950
  930 IF(DSER1)980,980,970
  940 IF(DSER1)980,945,970
  945 CORR=0.
      GO TO 990
  950 CORR = -DSER1/DIF2 + .5
      IF(CORR-50.)960,990,980
  960 IF(CORR+33.)970,990,990
  970 CORR=(-33.)
      GO TO 990
  980 CORR = 50.
  990 IF(INDEX)1000,1000,1027
 1000 X(M)=TMPX*(1.+CORR*.01 )
      IF(X(M)-TRHR/(2.*AK(M)))1020,1020,1010
 1010 X(M)=TRHR/(2.*AK(M))
 1020 INDEX=1
      M1 = M
      PRINT 1025,M,TMPX,X(M)
 1025 FORMAT(10H X FOR STAI2,14H  CHANGED FROMF6.3,4H  TO F6.3)
      NC=1
      GO TO 400
```

```
1027 WTDK = 0.
     DO 1028 J = 1,NRTG
1028 WTDK = WTDK + AK(J)*SUMQJ(J)/SMQ
     AK(M)=TMPK*(1.+CORR*.01)
     IF(AK(M)-WTDK*A )1032,1032,1031
1031 AK(M)=WTDK*A
1032 IF(AK(M)-WTDK/A )1033,1034,1034
1033 AK(M) =WTDK/A
1034 IF(AK(M)-TRHR/(2.*X(M)))1050,1050,1040
1040 AK(M)=TRHR/(2.*X(M))
1050 INDEX = 0
     PRINT 1060,M,TMPK,AK(M)
1060 FORMAT (11H AK FOR STAI2,14H  CHANGED FROMF6.2,4H  TOF6.2)
     IF(M-NRTG) 1080,1090,1090
1080 M = M + 1
     GO TO 400
1090 M = 1
     NCYCL = NCYCL + 1
     IF(NCYCL-3)400,1100,400
1100 TEST = 999999.
     GO TO 400
1110 PRINT 1120
1120 FORMAT (/34H STA   AK     X     C1     C2  NRCHS)
     PRINT 1130,(J,AK(J),X(J),C1(J),C2(J),NRCHS(J),J=1,NRTG)
1130 FORMAT (I2,F7.2,3F6.3,I7)
     PRINT 1140
1140 FORMAT(//5X,67H ALTERNATIVE COEFFICIENTS (SEE EQUA 14 AND PLATE 2-
    1-EM 1110-2-1408)//10X,3HSTA,3X,2HC1,4X,2HC2,4X,2HC3,4X,2HC4,4X,2HC
    25,4X,2HC6,4X,2HC7,5X,5HNRCHS/)
     DO 1150 K=1,NRTG
     CP1=C2(K)
     CP2=C1(K)-C2(K)
     CP3=1.-C1(K)
     CP4=CP1*CP3+CP2
     CPI(1)=CP1
     CPI(2)=CP4
     CPI(3)=CP4*CP3
     CPI(4)=CP4*CP3*CP3
     CPI(5)=CP4*CP3*CP3*CP3
     CPI(6)=CP4*CP3*CP3*CP3*CP3
     CPI(7)=1.0 - (CPI(1) + CPI(2) + CPI(3) + CPI(4) + CPI(5) + CPI(6))
```

```
1150 PRINT 1160,K,(CPI(KX),KX=1,7),NRCHS(K)
1160 FORMAT (10X,I3,7F6.3,I8)
     GO TO 100
     END
```

EXHIBIT 4

INPUT DATA DESCRIPTION

23-J2-L231

A.. Three output title cards

B. Job specification card or cards

1. TR - Flow tabulation interval in minutes
2. NISTA - Number of inflow stations (exclusive of local flow not to be routed)
3. LSTA - Station number of local inflow or of station used as an index of local inflow. Blank if no local inflow to be added; NISTA + 1 if not one of the stations to be routed
4. NFLDS - Number of floods used
5. NCOMB - Number of combining points between inflow stations and outflow station, excluding outflow combining point
6. IPNCH - Indicator, when positive, calls for diagnostic output. Leave blank normally.
7. A - Maximum permissible ratio between individual reach K and average K for all reaches. Must considerably exceed 1.0. If blank, program will supply 10.

C. T - Relative travel time for each routing from each inflow station to next combining point, must be equal in number to and in same order as station data are read, must be expressed in same units for all stations.

D. Combining point data (NCOMB sets)

1. T - Relative travel time from combining point to next downstream combining point, same units as item C.

2. N - Number of hydrographs to be combined at this combining location before routing.

3+. ISTA - Identification number of each station whose flows will be combined at this combining point.

E. Hydrograph specification card

1. IMNTH - Month number for start of flood
2. IDAY - Day of start of flood
3. IYR - Year of start of flood
4. ITIME - Time of start of flood on 2400 scale. Since flows are tabulated at end of period, this represents one interval earlier than time of first flow.

EXHIBIT 5

5. NQ - Number of flows tabulated for each station.
6. RTIOL - Ratio of local inflow to index station flow. If not specified, will be computed from volume difference of hydrographs to be routed and outflow hydrograph, but set to zero if this difference is negative.

F. Tabulation cards of inflows (for one flood) to be routed

QI - NQ flows in cfs for each station in same order as T values (items C). All sets must start at same time and include zero flows, if any.

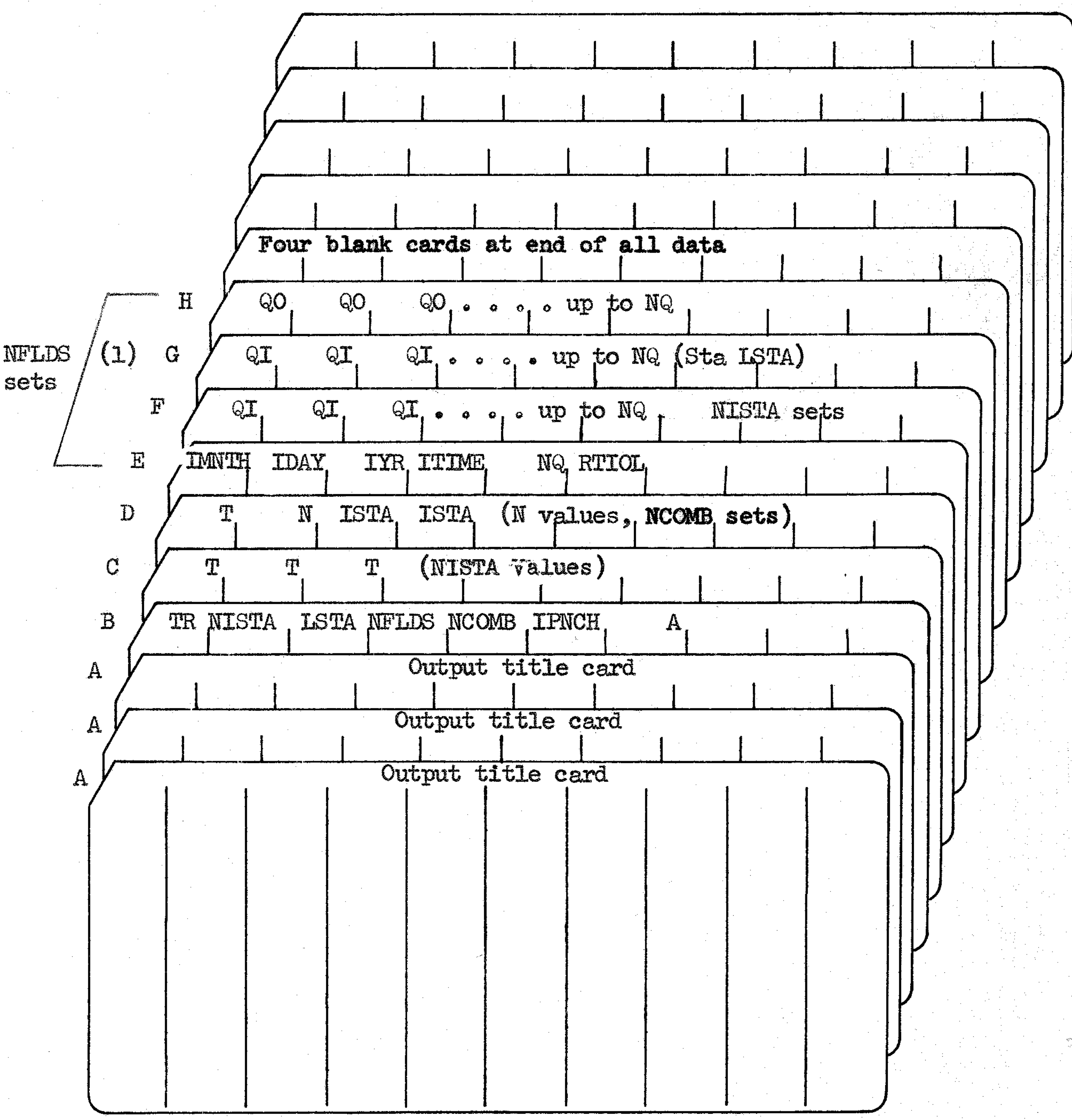
G. Tabulation cards of local inflow index station for same flood, if not one of the stations in item F.

QI - Same definition as item F.

H. Tabulation cards of observed outflow for same flood.

QO - Same definition as item F.

SUMMARY OF REQUIRED CARDS



Note:

(1) Omit if LSTA does not exceed NISTA

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