FISCAL ALLOCATION RESPONSIBILITY SENEGAL RIVER BASIN DEVELOPMENT PROJECT

VOLUME V

HYDROLOGIC ROUTING OF THE PROBABLE MAXIMUM FLOOD FOR THE MANANTALI DAM

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DEVELOPMENT AND HYDROLOGIC ROUTING OF THE PROBABLE MAXIMUM FLOOD FOR THE PROPOSED MANANTALI DAM IN THE SENEGAL RIVER BASIN

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ABSTRACT

Development and Hydrologic Routing of the Probable Maximum Flood for the Proposed Manantali Dam in the Senegal River Basin

by

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The objective of this project was to develop and route the probable maximum flood for the Manantali Dam in the Senegal River Basin, West Africa. The study was divided into two main areas of research: 1) the formulation and calibration of a hydrologic rainfall-runoff-routing model, and 2) an estimation of the probable maximum precipitation.

The precipitation value was input into the model in order to develop the probable maximum flood. A number of reservoir routings were conducted for various initial soil moisture conditions and assumed lake levels. Results of the routings were analyzed to determine the capacity of the reservoir to safely pass and attenuate large floods. The runs were also reviewed as to their relationship to the operation of the dam during flood conditions.

(130 pages)

CHAPTER I

INTRODUCTION

Background

The Senegal River is the second largest river in West Africa. The headwaters originate in the rainfed Fouta Djallon Mountains and the river empties into the Atlantic Ocean near Saint-Louis, Senegal (Figure 1). As the river carves its way from the mountains to the sea, it traces a 1800 km long path and drains approximately 290,000 sq km (Senegal-Consult 1970). The drainage area is located in four countries: Guinea, Mali, Mauritania, and Senegal.

The Senegal River has been under study for potential development since 1935 when the"Mission pour l'Amenagement du Senegal (M.A.S.)" was formed (U. S. Bureau of Reclamation 1979). Many studies have been made over the years identifying feasible and beneficial projects. In 1972, the Senegal River Development Program came under the direction of an international planning organization called Organisation pour la Mise en Valeu du Fleuve Senegal (OMVS). The OMVS was formed to plan and implement the development of the water resources of the Senegal River Basin. The members of the organization are the three West African countries of Mali, Mauritania, and Senegal.

To date, development plans for the Senegal River Basin include the construction of two dams along the river for purposes of irrigation, domestic and industrial water supply, flood control, hydropower,

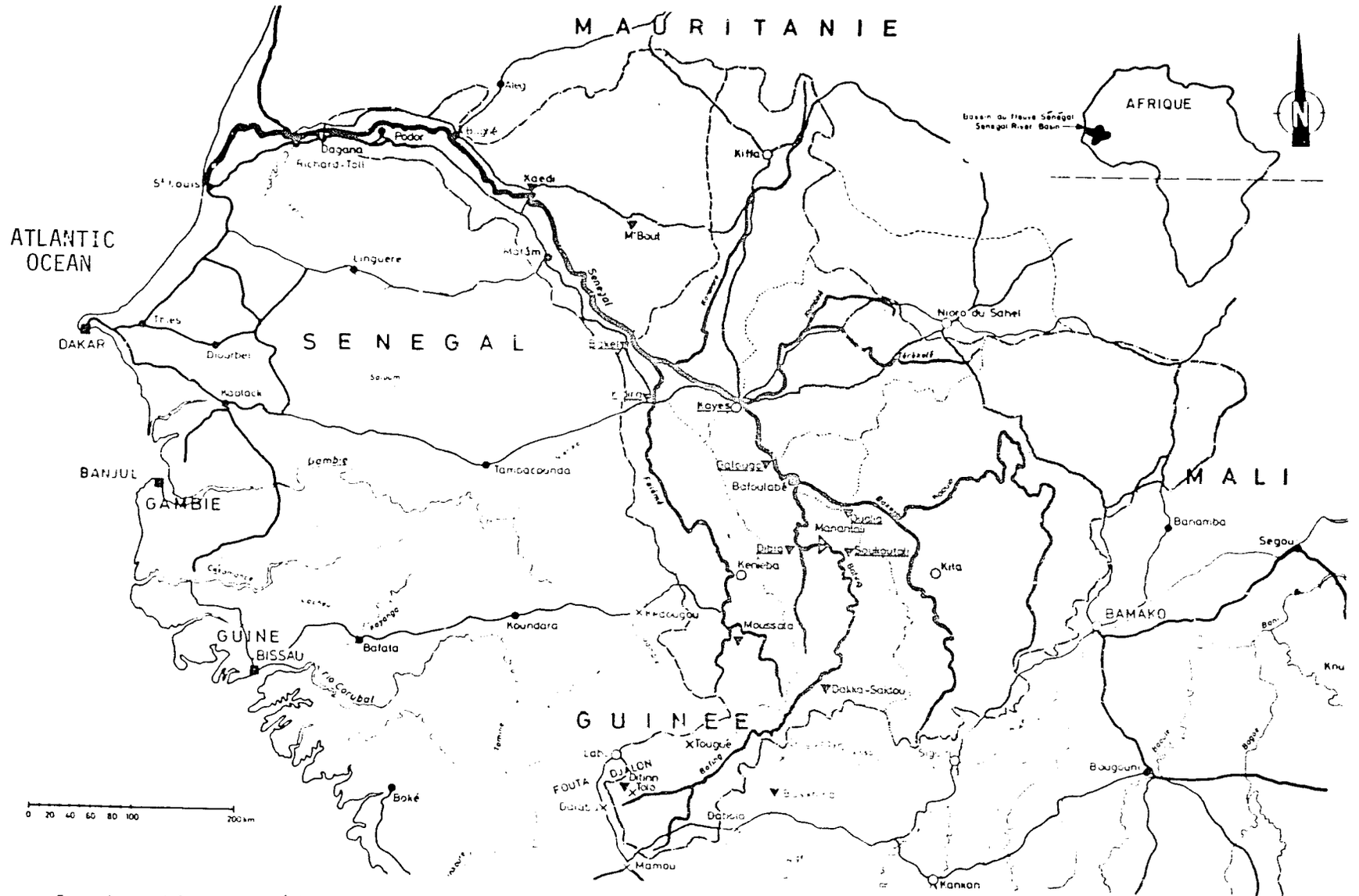


Figure 1. Location map (from Groupement Manantali 1977).

and navigation. The Diama Dam is a low diversion dam and ocean salt-water barrier near the mouth of the Senegal River. The Manantali Dam, located some 1200 km upstream from the ocean on the Bafing River, is designed as a multi-purpose structure.

With respect to flow contribution, the Bafing River is the main tributary to the Senegal River. Its watershed covers 38,000 sq km (18 percent of the total drainage area) yet it delivers more than one-half of the annual streamflow (Gannett Fleming Corddry and Carpenter 1980). The river is situated in a region called the Upper Basin and can be characterized by steep slopes, rapids, and falls. The topographic conditions of the Bafing River valley, upstream of the Manantali Dam site, lends itself to the creation of a large storage reservoir. The reservoir will receive inflow from 27,800 sq km.

Four main studies have been completed on the feasibility and design of the Manantali Dam. These are:

1. Feasibility Survey for the Regulation of the Senegal River--Design of a System of Water Management Planning in the Upper Senegal River Catchment. (Senegal-Consult 1970.)

2. Programms Integre de Developpement du Bassin du Senegal. (Norbert Beyrard 1974.)

3. A study by experts from the Republic of China. 1975 (oral report).

4. Etude d'Execution du Barrage et de l'Usine Hydroelectrique de Manantali. (Groupement Manantali 1977.)

One design criteria that was not addressed in detail in the above reports is the development and hydrologic routing of the probable maximum flood (PMF) on the Bafing River.

Objective

The main objective of this study was to develop and route the inflow hydrograph of the probable maximum flood (PMF) to the Manantali Reservoir. The PMF is used primarily to determine the spillway capacity required to safely discharge the peak flow of the flood without causing any major damages to the structure itself.

Though the design of the dam had been finalized before the onset of this study, the project was continued to serve as a check on the current design of the Manantali spillway and to follow the suggestions and standard practices of such agencies as the U. S. Bureau of Reclamation and World Meteorological Organization (WMO).

It was also desired to determine the effect of various initial water surface elevations of the reservoir on the flood peak. The results for each condition could then provide input to the hydrologic operational studies that will be conducted in a later phase of the Senegal River Project.

Scope of Study

The project was divided into two main areas of research: a) Rainfall-Runoff-Routing Model, and b) Estimation of the probable maximum precipitation (PMP).

To meet the objectives of the project, the following tasks were performed:

1. LITERATURE REVIEW - A general review of the existing reports completed for the Senegal River Development Program was conducted. These reports were considered to be the major source of data for the

project. The review also included research on the hydrologic methods for modeling and estimating the PMP. These references included textbooks, manuals, professional papers and individual contacts.

2. DATA COLLECTION - The hydrologic/hydraulic data extracted from the available project reports provided a fair data base. Additional data concerning rainfall records were obtained through the cooperation of the OMVS staff.

3. DEVELOPMENT OF THE MODEL - To determine the PMF inflow hydrograph, an estimate of the excess precipitation (runoff) and its distribution over time was required (see Task 4). Development of a hydrologic model was required in order to transform this runoff into the PMF inflow and outflow hydrographs at the Manantali Dam. Application of the rainfall-runoff process to predict excess precipitation, channel routing to account for basin storage and lag time, and reservoir routing to attentuate the inflow flood hydrograph were utilized in the model. Calibration and verification of the hydrologic model was necessary before it could be applied to the project area.

4. DEVELOPMENT OF THE PMP - The probable maximum flood is the result of the probable maximum precipitation (PMP). An estimate of the PMP was accomplished by studying past storms and rainfall records and using accepted methods for maximizing storm events.

5. MODEL APPLICATION AND RESULTS - Once the model parameters were calibrated for the Bafing River basin, the probable maximum flood hydrograph was computed and routed through the Manantali Reservoir. A number of routings were conducted for various assumed initial water

levels in the reservoir to determine the capacity of the dam to reduce large flood peaks. The peak discharge and maximum reservoir level elevation for each routing are presented.

5. SUMMARY AND CONCLUSIONS - A summary of this study and an evaluation of the results are given. The evaluation focuses on the spillway capacity and attentuation of the flood peaks provided by the Manantali Reservoir. Suggestions for other applications of the model are also presented.

CHAPTER II

LITERATURE REVIEW

Previous Senegal River Basin Reports

To become familiar with the Senegal River Project, a review of the existing reports on file was conducted. The contents of the reports varied from hydrologic analyses to agricultural practices to socio-economic studies. Of main interest to this author were the reports on the analyses for the Manantali Dam and the summaries of precipitation and streamflow data. Unfortunately, not all of the cited reports and references pertaining to this subject were on file or available. In addition to some missing reports, one other limiting factor during the review was that most of the available material was written in French. Despite these factors, the existing reports provided the majority of the information required in this study.

The following presents a brief summary of selected reports that were most beneficial to the success of this study:

1. Feasibility Survey for the Regulation of the Senegal River - Design of a System of Water Management Planning in the Upper Senegal River Catchment. (Senegal-Consult 1970.) This report, further referenced as the Senegal-Consult Report, was prepared by a group formed by four engineering consulting companies. It consists of two parts. The first part is a preliminary study for the regulation of the Senegal River and the second is an investigation of a control

system for the flows from the upper reaches of the Senegal River. Their work involved the siting and assessment of potential locations for control structures. After detailed hydraulic and economic analyses, Senegal-Consult recommended the reservoir site at Manantali as having the most potential for regulating the river flow. This recommendation was accepted and implemented into the initial stage of the development project. Results of the analyses are presented in nine volumes. The volumes most often referred to during the course of the study were:

Volume 2	Topography
Volume 3A	Hydrology - Text and Annexes
Volume 3B	Hydrology - Appendices

2. Etude d'Execution du Barrage et de l'Usine Hydroelectrique de Manantali. (Groupement Manantali 1977.) Groupement Manantali is a consortium of three organizations. Their study was divided into three phases: 1) design of the Manantali Dam and a preliminary summary report; 2) preparation of a detailed preliminary plan; and 3) selection of a contractor and supervision of the work. Phases 1 and 2 are presented in a multi-volume report, of which Mission A.1.2. - Hydrologie and Mission A.2.2.- Modeles physique et mathematique contained the most useful information.

3. Reports conducted by the U. S. Bureau of Reclamation (1975, 1976, 1979) for the U.S. Agency for International Development provided detailed assessment and evaluations of the relevant studies on the

Senegal River and Manantali Dam. These reports and an evaluation report by Riley (1976) not only provided a translated summary of the previous reports but also recommendations for future work which stimulated this hydrologic study.

4. The Gannett Fleming Corddry and Carpenter report (1980) contains background information necessary for an understanding of the water resources of the Senegal River Basin. It includes general descriptions of the basin, discussions on climate and streamflow, and presents the major results of the Groupement Manantali report.

As stated earlier, the above reports do not constitute an all inclusive listing of studies reviewed. Other documents on file were read to obtain an overall understanding to the developments of the Senegal River Project.

Hydrologic Models and Processes

Three existing rainfall-runoff models were examined during the early phases of this project - HEC-1, the Streamflow Synthesis and Reservoir Regulation Model (SSARR), and the Stanford Watershed Model (SWM).

HEC-1 Flood Hydrograph Package (U. S. Army Corps of Engineers 1973) was originally developed in 1967 by Leo R. Beard and other members of the Hydrologic Engineering Center staff. It can perform most ordinary flood hydrograph computations. The model is applicable only to a single storm event since there is no provision for precipitation loss rate recovery during periods of no precipitation. Capabilities to perform reservoir routing were not provided in the 1973 HEC-1 version but were available in HEC-5 Reservoir System Operation

for Flood Control. Revisions of the model in 1978, made to enable expedient hydrologic evaluations of the safety of small non-federal dams, included the ability to route inflow hydrographs through a reservoir. However, the model requires that a complete discharge curve be inputted so for various spillway scenarios, the user would have to calculate the total discharge curve manually.

The Streamflow Synthesis and Reservoir Regulation Model, SSARR, (U. S. Army Corps of Engineers 1975) was developed initially to meet the needs of the North Pacific Division of the U. S. Corps of Engineers. The needs were to provide mathematical hydrologic simulation for systems analyses as required for the planning, design, and operation of water control works. The model performs similar functions as HEC-1 does but is not limited to a single event. Developments in the model have provided the versatility for operational river forecasting and river management activities. It was designed for large basins and has been applied to various river basins, such as the Mekong River in Southeast Asia (U. S. Army Corps of Engineers 1971).

Crawford and Linsley (1966) designed the Stanford Watershed Model (SWM) to synthesize a continuous hydrograph at the outlet of a watershed. Each process in the hydrologic cycle is defined in a model that gives a detailed accounting of the balancing of precipitation. The model utilizes a hydrologic watershed routing technique to translate the channel inflow to the watershed outlet but does not have the capability to perform flood routing through a reservoir.

In addition to reviewing the previously mentioned models, the author has had experience with two other simulation models, TR-20

(U. S. Soil Conservation Service 1973) and DAMS-2 (U. S. Soil Conservation Service 1971). Both of these models use soil-land use information in developing the runoff hydrographs. The SCS curve number approach as described in the National Engineering Handbook (U. S. Soil Conservation Service 1972) is used to compute the surface runoff. These two models are applicable mainly to small watersheds and do not provide much flexibility when routing through a reservoir with controlled spillways.

Since the advent of the digital computers, many mathematical models have been developed for flood prediction and simulation. Various methods of expressing the rainfall-runoff relationship, channel routing and reservoir routing can be found in the literature. These methods range from deterministic models to parametric models to statistical models to stochastic models. The nature and scope of this study did not warrant a full scale literature review of processes and models as that would constitute a major undertaking in itself. The following presents a review of those methodologies that the author examined throughout the duration of the study.

The rainfall-runoff relationships were reviewed from the textbooks of Chow (1964), Linsley et al. (1975), and Viessman et al. (1977). Also examined were the runoff methods incorporated into the computer models HEC-1, SSARR, and SWM. The relationships between rainfall and runoff can be grossly categorized as those which deal with runoff volumes, peak discharge estimates, and time distribution of runoff.

Time distribution involves the use of unit hydrographs as proposed by Sherman (1932) or the instantaneous unit hydrograph (Clark 1945). Nash (1957) considered a drainage basin as "n" identical linear reservoirs and routed instantaneous inflow through the reservoirs. Dooge (1959) developed the instantaneous unit hydrograph for a basin by envisioning a series of alternating linear channels and linear reservoirs.

Runoff volumes can be calculated by application of the water budget as incorporated in the Stanford Watershed Model. This approach is the basic concept in hydrology. Correlations between rainfall and runoff using the antecedent precipitation index have been developed (Kohler and Linsley 1951). Empirical relationships (not reviewed by the author) exist that correlate two or three variables to runoff.

Flood routing, as defined by Fread (1981), is a mathematical method (model) for predicting the changing magnitude and celerity of a flood wave which propogates through a river, reservoir, or estuary. In his paper, a brief review of the development of hydrologic and hydraulic flood routing is presented.

During the review of hydrologic routing techniques, articles and manuals referenced the Muskingum method quite frequently. A few selected authors included the U. S. Army Corps of Engineers (1960); Carter and Godfrey (1960); Gill (1978); and Cunge (1969). In the SSARR model the Corps of Engineers applied a successive incremental type storage routing known as multiple storage. The channel is visualized as a series of small "lakes" which represent the natural delay of runoff from upstream to downstream points. Gilcrest (1950)

describes the working R and D flood routing procedure. This method is considered more advantageous than the Muskingum method if a variable storage constant must be considered. Flood routing by time displacement of average inflow has also been used to approximate flood wave shapes. Two such methods are the Successive Average-Lag Method (Tatum 1941) and the Progressive Average-Lag Method (U. S. Army Corps of Engineers 1935).

As in channel routing, reservoir routing can be computed using hydrologic and hydraulic methods. The hydraulic routing techniques, normally applied to channels, have been successfully used in reservoir routing for complex unsteady flow conditions. The Tennessee Valley Authority (Garrison et al. 1969) simulated the passage of a flood wave through a proposed reservoir. The National Weather Service Dam Break model (Fread 1982) has the capability to use either dynamic routing or storage routing.

However, for reasons of convenience and economy, the hydrologic routing methods are more frequently used. The modified Puls method is commonly discussed in most hydrology textbooks (Chow 1964, Linsley et al. 1975, Viessman et al. 1977). The progressive average-lag method, mentioned above, has also been applied to reservoir routing.

Probable Maximum Precipitation

Probable maximum precipitation (PMP) is defined as the greatest depth of precipitation for a given duration meteorologically possible for a given basin at a particular time of year. A review of historical reasons for the PMP concept and its evolution in the United States is given by Myers (1967).

The procedures for estimating the PMP based on the meteorologic, or traditional approach are described in "Manual for Estimation of Probable Maximum Precipitation" (World Meteorological Organization 1973). Essentially, the approach consists of moisture maximization and transposition of observed storms. These methods have been used for specific project areas and for generalized mapping. (See the Hydrometeorological Reports No. 39-53, National Oceanic and Atmospheric Administration (1963-1982)). Generally, these methods are applicable in the middle latitudes for basin sizes of up to about 50,000 km^2 and are considered to be inappropriate for the tropics, because rainfall cannot be related to minor fluctuations in humidity.

Hershfield (1965) has developed a statistical method of estimating the PMP based on the general frequency equation (Chow 1961):

$$X_t = X_n + KS_n$$

where

X_t = rainfall for return period t,

X_n = mean of a series of n annual maxima,

S_n = standard deviation of a series of n annual maxima, and

K = statistical variable.

The value of K has been computed to be 15. Application of this method is demonstrated in the WMO manual.

Jennings (1950) enveloped the maximum point rainfall values observed throughout the world. Fletcher (1950) subsequently developed an equation to envelope maximum precipitation in the United States using the duration of the storm event and areal extent.

Procedures using the above three methods are presented by Koelzer and Bitoun (1964). The authors direct their attention to areas with limited hydrologic data, primarily areas outside the United States. To contrast, a differing viewpoint on the concept of the probable maximum precipitation for design floods is presented by Benson (1973). He states that the method is subject to serious criticism on both technical and ethical grounds. Technical because of subjective factors in the computational process and the lack of specific or inconsistent meaning in the result. Ethical because it implies a design that is virtually risk free.

CHAPTER III

AVAILABLE DATA

Topographic Maps

Complete mapping of the Senegal River basin has been done by the "Institut Geographique National (France)" (IGN). Maps at a scale of 1:500,000 for most of the basin and at a scale of 1:200,000 for the whole basin above the city of Bakel were available to Senegal-Consult. Since these existing maps were not suited for detailed work, Senegal-Consult undertook mapping in the reservoir areas and dam sites at a larger scale.

Of these maps, only a few are included in the Senegal-Consult Report - Volume 2, Topographical Data. The maps available for this project covered 74 percent of the drainage area above the Manantali Dam site. Of this mapped area, 49 percent was at a scale of 1:600,000 and covered the lower portion of the catchment. The remaining fifty-one percent of the area was available at 1:200,000. Though not complete, the maps were sufficient enough to delineate subbasins and to provide pertinent input data used in the computer model.

Precipitation and Evaporation Data

Observation stations in and around the Upper Basin of the Senegal River are presented in Volume 3A-Annex 3.07 of the Senegal-Consult Report (1970). Summarizing, there are:

- 11 synoptic stations (main climatological stations of the region).
- 18 climatological stations (auxiliary climatological stations)
- 52 rainfall stations

For the stations located in the country of Mali, daily and monthly precipitation values were obtained for the years of record up to 1965 (ORSTOM 1966). The only source of precipitation records was the Mali data, of which, only eight stations were pertient to the study. Those stations used in the analyses were:

Name	Years of Data
Bafing Makana	3
Bafoulabe	34
Bamako-Aero	45
Falea	9
Guene-Gore	10
Kenieba	24
Kita	35
Sagabari	7

The National Center of Atmospheric Research (NCAR) has world monthly surface climatological data for all stations (global) in the WMO network through 1980 on two magnetic tapes. These tapes were obtained from George H. Hargreaves, Research Director for the International Irrigation Center at Utah State University, and the data for the study area were transferred to hard copies. Though there were numerous gaps in a number of the station records, the data was quite useful for the analysis of the probable maximum precipitation.

Maps of annual, and some monthly, isohyetes of precipitation are given in both the Senegal-Consult and the Groupement Manantali reports.

Evaporation data have been recorded in the Senegal River Basin for at least 20 years at several spots. Rates of evaporation from free water surfaces have been recorded by the use of Piche evaporimeters, the Colorado-type evaporation pan, and estimated by the Penman equation (Senegal-Consult, Volume 3A 1970). These data for selected stations on an annual and monthly time basis are presented in the Senegal-Consult report.

Streamflow Records

To study the flow regime of the Upper Basin, the Senegal-Consult team examined many stream gage records and discharge measurements. Their detailed analysis of the data led to a considerable reduction of the list. The records finally used for their study are presented in section 4.2.2 of the Hydrology volume. A description of the gaging stations is in the Appendices to the Hydrology volume.

Also included in the Appendices are the correlation equations and coefficients that were computed to estimate the missing discharges in the existing data. These correlations were limited to monthly and annual discharges. A summary of these streamflow values from 1903-68 are presented for 12 gaging locations throughout the Upper Basin of the Senegal River, in particular the Dibia, Soukoutali and Dakka - Saidou stations on the Bafing River (see Figure 1).

Frequency analyses of the 66 years of available records (1903-68) were conducted by Senegal-Consult for peak flood discharge values and

annual yields of runoff volume. The Gauss-Gibrat probability distribution was used to estimate both the flood peaks and yields.

Flood hydrographs at four gaging locations - Bakel, Galougo, Dibia, and Gourbassi - for six years of record are included in the Senegal-Consult report. Those figures provided the only source of daily flood measurements that were helpful in the current study. A report published by ORSTOM (Rochette 1965) lists daily flows on the Senegal River from 1903-65 at Bakel, Matam, and Dagana (see Figure 1). However, these stations are downstream from the study area and thus were not directly used in the analyses performed by the author.

The project conducted by Groupement Manantali reports similar types of hydrologic data as those reported by Senegal-Consult. Linear regression equations of streamflow were determined to complete the records through 1977. Differences exist between the two sets of regression data but result in only minor differences in the computed flood flows.

The Pearson III distribution was used for the frequency analysis conducted by Groupement Manantali. Comparison of the flood flows associated with particular frequencies indicates that the Senegal-Consult values are consistently larger than those computed by Groupement Manantali.

Hydrograhs at Soukoutali 1966-77 and Dibia 1951-77 are given in the Hydrologic (Mission A.12) portion of the Groupement Manantali study. Though these hydrographs present a large volume of data, the scales are such that daily flow rates cannot be interpolated from the plots.

Other historic data and evaluations are presented and discussed in the two reports (Senegal-Consult and Groupement Manantali). This information, though it may not have been used directly, provided valuable knowledge for understanding the hydrologic characteristics of the Bafing River basin.

Dam and Reservoir Characteristics

The most recent dam design and relevant reservoir data were taken from the Groupement Manantali report (1979). In Mission A.2.2., Modeles physique et mathematique, complete descriptions of the spillway system and rating curves are reported. The spillway consists of 8 gated openings, 9 x 6.5 meters, for emergency releases and 7 gates, 3.8 x 4.8 meters, for regular operation of the reservoir. In the appendix to Mission A.2.2, the emergency and principal spillway rating curves for a single gate are presented as a family of curves for varying gate openings.

The elevation-surface area and elevation-storage volume curves, as presented in Volume 7 of the Senegal-Consult report, were used as input for the reservoir routing subroutine in the hydrologic model.

A summary sheet taken from an OMVS report (1979) is included as Table 1 to provide the reader with data to develop a better understanding of the complete Manantali Dam project.

Table 1. Manantali Dam summary table (from OMVS 1979).

MANANTALI DAM
ANALYTIC DATA SHEET

I – DAM SITE

The Manantali dam site is on the Bafing river, one of the major tributaries of the Senegal river and is located 90 km South-East from Bafoulabe, in the Republic of Mali

II – PROJECT OBJECTIVES

Exhaustive studies conducted by the OMVS have shown that in order to achieve the fundamental objectives of integrated development of the Senegal river basin resources, a regulated 300 m3/sec. river flow should be secured from the start

The Manatali dam, as it has been designed, is the work that best meets this prerequisite. Its construction will satisfy the following multiple purposes :

- irrigation of 255 000 ha of land in the valley ;
- year round navigability of the Senegal river between Saint-Louis and Kayes, by securing a 100 m3/sec. flow that could be recovered in the river mouth area for the irrigation of an additional 120 000 ha ,
- a firm 800 GWh/year electric power generation ;
- the development at a later date of two downstream hydro-electric production sites that would double the capacity of the Manantaly plant ;
- abatment of the crest in order to reduce sufficiently the 1 000 year and 100 year flood to respectively the present 100 year and 10 year flood crests, and thus ensure protection of the population centres and of the crops cultivated in the valley, and to reduce the cost of the embankments ;
- the supply during a transition period of a 200 m3/sec. regulated flow and the release for one month in August - September of a 2 500 m3/sec. artificial flood to maintain optimal conditions for traditional flood recession farming, pending expansion of irrigated agriculture throughout the Senegal river valley.

III – TECHNICAL CHARACTERITICS

Dam : it is a dam of the combined type (flow regulation and hydro-electric power production) composed of a concrete structure in its middle part that houses all surfaces hydraulic structures, and of two lateral rock-filled dikes.

— length of the concrete structure	492.8 meters
— length of the rockfilled dikes	992.0 »
maximum height above foundations	65 »
normal water storage level	208.0 meters IGN
— water level corresponding to the 1 000 year flood	210.5 » »
— crest elevation	212.5 » »
storage volume	11 billion m3

— flood spillway : eight (8) 9 meter wide bays equipped with radial gates

penstocks : seven (7) 3.8 m x 4.8 bays equipped with radial gates.

— headworks : five 14 m x 15.4 m extended by five conduits of a 4.8 m diameter.

Power plant : The power plant will be constructed at the foot of the concrete dam buttresses

a) *Turbines*

— design capacity	40 MW
— maximum discharge	113 m3/s
normal head	40 to 53 m
— rotation velocity	150 r.p.m. ; 50 HZ
— diameter of runner	3.80 m

b) *Generators*

— unit capacity	46 MVA
— rotation velocity	150 r.p.m. 50 HZ 20 pairs — 6 poles
— normal voltage	13 800 volts
— annual power production	
● firm :	800 GWh
● marginal :	150 to 200 GWh.

c) *Transformers*

— capacity	46 M/A
— length	6.5 meters
— width	4.5 »
— height	6.0 »

CHAPTER IV

HYDROLOGIC MODEL DEVELOPMENT

Introduction

The planning and development of water resources usually involves a dam and other types of control structures. For major projects, such as the Manantali Dam, the design criteria should be stringent enough to ensure a sound, safe project. It is for these reasons that the hydraulic structures of such projects must be designed to operate under extreme conditions. A standard practice is to design the spillway system to pass the probable maximum flood (PMF) without extensive damage to the structure. The PMF results from the probable maximum precipitation (PMP) that could occur in the drainage basin during a storm containing the most critical meteorological conditions considered probable for a particular time of year (Viessman et al. 1977).

In previous studies (Senegal-Consult 1970, Groupement Manantali 1977) conducted for OMVS, the design flood for the Manantali Dam was based on statistical analyses of runoff data in the Senegal River basin. The results of the analyses predict the design-peak discharge associated with a recurrence interval of 10,000 years. Though the PMP or PMF are not usually assigned a probability level, it is widely accepted engineering practice to estimate the probable maximum values using a return period of 10,000 years. Therefore, the previous

methods and results from the other studies are considered acceptable. However, to serve as a check on the current design of the Manantali spillway and to follow standard practices (U. S. Bureau of Reclamation and World Meteorological Organization), the probable maximum flood (design flood) was developed based on the concept of the PMP (World Meteorological Organization 1973).

To estimate a flood peak from the PMP, one must develop a relationship between rainfall and its consequential runoff for that particular basin under consideration. When this relationship is expressed as mathematical formulas, a model is formed. For the Bafing River basin, within which the Manantali Dam is located, no such model existed or was being applied. Therefore, it was considered appropriate to establish the use of a hydrologic model for the Bafing River basin. There are numerous existing hydrologic models that could be used to develop the PMF, such as HEC-1 (U.S. Army Corps of Engineers 1973), SSARR (U.S. Army Corps of Engineers 1975), and Stanford Watershed Model (Crawford and Linsley 1966). These models were considered but due to the limited availability of data on the watershed and the limited computer capabilities available to OMVS it was decided to build a model more specific for the basin.

A computer program was written in standard FORTRAN to be compatible with OMVS's Digital PDP-11 Computer. The program was developed to simulate the rainfall-runoff processes on large, natural watersheds, such as the Bafing River watershed. The primary intent of the model was to determine the PMF for the Bafing River basin and to route this flood through the Manantali Reservoir using the current design characteristics of the reservoir (Groupement Manantali 1979). The

model can also be used to simulate outflow hydrographs for lesser floods and to help establish operating rules to meet the requirement of the artificial flood releases during the early phases of the Senegal River Development Project.

General Description

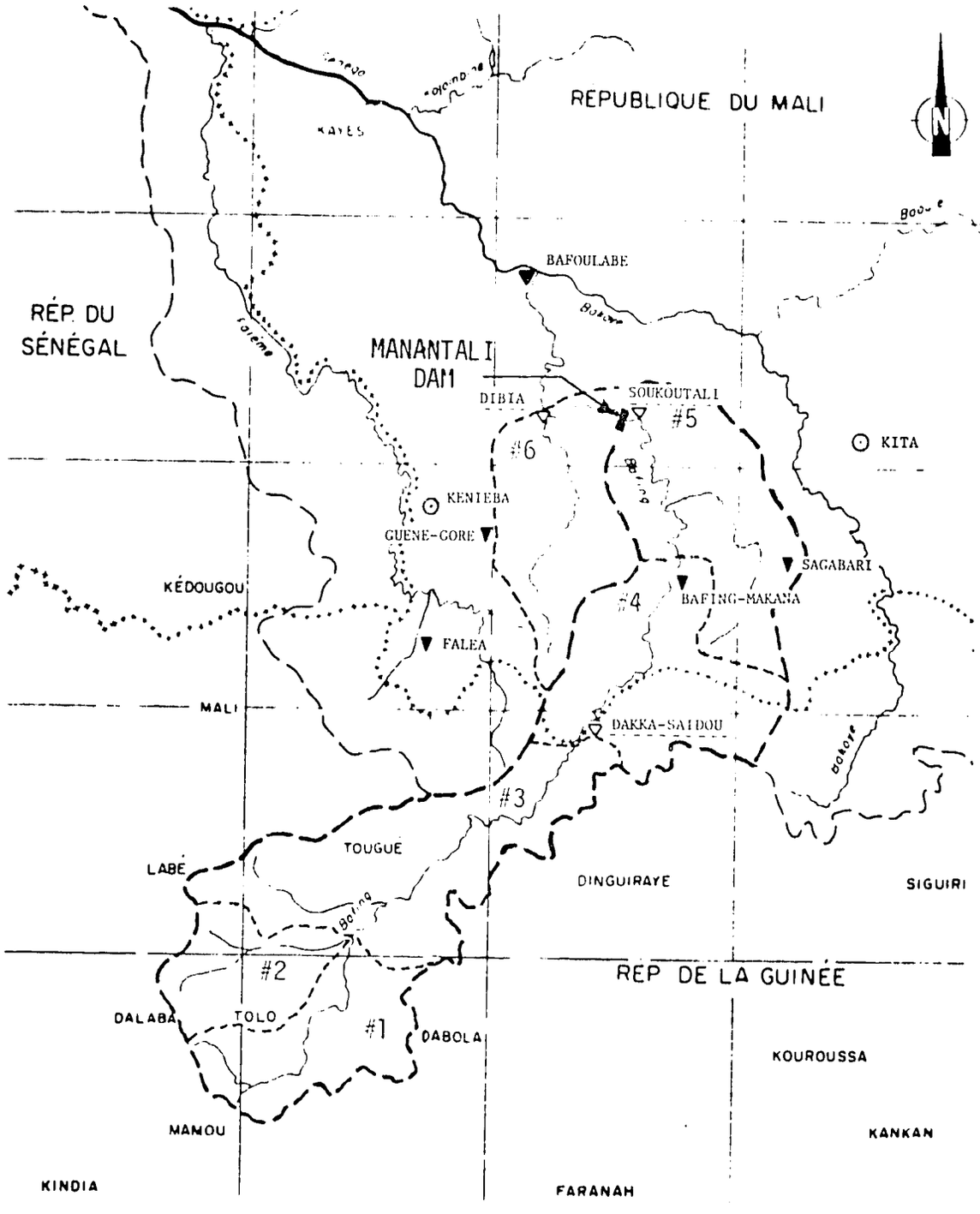
The model incorporates three major components of hydrology. These are the rainfall-runoff relationship, channel routing, and reservoir routing. It is classified as a hydrologic model which implies that only the conservation of mass is employed to define the shape of the flood wave as it progresses downstream. Due to the limited availability of detailed hydrologic and climatic data, the computer model is based on simple hydrologic concepts and processes. That is, given a rainfall event and an estimate of the soil moisture conditions, the rainfall excess is estimated as a percentage of the precipitation. This percentage is obtained from a curve that relates the runoff coefficient to the soil moisture condition. A more detailed description is presented later in this chapter under the section - Component Description.

Processes such as interception, depression storage, infiltration and deep percolation are not included in the model. During large flood events, the impacts of these processes, in general, occur only during the first few hours of a storm and have little effect on the runoff hydrograph from a storm system which may last several days or whose rainfall values are significantly higher than their upper limits.

The concept of the model is to divide the watershed of interest into subbasins, develop runoff hydrographs for each subbasin, combine and channel route the hydrographs to the upper end of the reservoir, and then route the resulting flood hydrograph through the reservoir to obtain the outflow hydrograph downstream of the dam. Figure 2 shows the Bafing River basin as it was divided into subbasins for this project.

The rainfall-runoff relationship is used to predict the quantity of precipitation that accounts for the increase in streamflow. This quantity, also referenced as runoff, is presented as a hydrograph which gives streamflow rates with respect to time. After computing the runoff hydrograph from a watershed, channel routing techniques are applied to account for channel storage and lag time as the flood hydrograph progresses downstream. These two process components are utilized for each subarea within the basin of interest and the resultant is a single, inflow hydrograph at the upstream end of the reservoir. The hydrologic reservoir routing process is used to attentuate and translate the inflow hydrograph to the dam, where the outflow is regulated by the spillway system.

The computer program called MAIN was written as a modular program. All of the major components of the model are separate subroutines, the order in which they are called is given by the user through an input file called SEQEXE.DAT (sequence of execution). The three main subroutines of the program are RUNOFF, ROUTE, and RESVOR with five supplemental routines called SUM, DTCURV, INTERP, APPROX, and ERROR. Each component is further discussed in the following section.



▽ Stream gage location
⊙ Climatic station
▼ Precipitation station
— — — Watershed boundary
- - - - - Subbasin boundary

Figure 2. Bafing River watershed (from Senegal-Consult 1970).

Component Description

MAIN

MAIN provides the central control to the model. In this part of the model the type of measurement units being used (English/Metric) is established and appropriate conversion factors are assigned as constant variables. The sequence of execution is read during this mode and an output file is established. All of the three main subroutines, SUM, and ERROR are controlled through MAIN.

RUNOFF

This subroutine calculates the amount of excess precipitation from an individual subbasin and transforms that value into a runoff hydrograph at the outlet of the subbasin. The methods used are a combination of the ideas presented in the HEC-1 and SSARR models (U. S. Army Corps of Engineers 1973, 1975). In general, the runoff hydrograph is computed based on a time-volume histogram of the basin which presents the volume and time distribution of total runoff. These volumes are converted to a flowrate and stored in a one-dimensional array called QIN(I). To take into account the basin storage and to define the shape of the hydrograph, an artificial linear reservoir is imposed at the outlet of the subbasin. A straight-forward routing in the form of

$$QOUT(I) = CQIN * QIN(I) + (1-CQIN) * QOUT(I-1) \quad .....(1)$$

is then applied where QIN(I) and QOUT(I) represent the inflow and outflow streamflow respectively and CQIN is a routing constant. The array QOUT(I) now contains the runoff hydrograph from the basin.

The parameter CQIN is a routing constant which is related to the routing time interval and an attentuation constant for the subbasin. CQIN must be calibrated for each new river basin to which the model is being applied. Initial values of the parameter can be obtained from known flood hydrographs, if available, using a graphical procedure. The procedure, fully illustrated in the HEC-1 User's Manual (U. S. Army Corps of Engineers 1973) is to determine the flowrate (Q) and slope (dQ/dt) of the hydrograph at the inflection point of the recession limb. These two values then define the attentuation constant, R, having the dimension of time.

$$R = -Q/(dQ/dt) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

$$CQIN = \frac{2\Delta t}{2R + \Delta t} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

For the Bafing River, a value of 1 day was used for Δt, the routing time interval.

The values of QIN(I) are computed using two input tables characteristic of the subbasin and the amount of precipitation that occurs. One of the input tables, called a time-area curve (Linsley et al. 1975) defines the shape of the time-volume histogram previously mentioned. The time-area curve is determined by dividing the subbasin into time zones separated by isochrones, or lines of equal travel time to the watershed outlet. The area between isochrones is measured and

a table is then constructed of time of travel versus cumulative drainage area. Magnitudes of the time-volume histogram are calculated by multiplying the ordinates of the time-area curve by the precipitation excess.

The second input curve, a relationship of soil moisture content versus runoff (SMI-ROP), is defined and calibrated for the subbasin. Actual soil moisture values are not available and so a soil moisture index is used in the model. The table defines an average relationship between the soil moisture index and the percentage of rainfall that contributes to direct surface runoff. The user has the option of defining 1-5 curves to represent the relationship. If rainfall intensity is expected to be a major factor in determining the runoff coefficient, more than one curve should be developed for various rainfall rates.

Knowing the antecedent soil moisture index, designated as SMIT, the SMI-ROP table is entered and an interpolated value of ROP, named ROCOEF, is used to calculate the amount of excess rain, EXRAIN, that contributes to the runoff. This EXRAIN value is applied to the ordinates of the time-area curve to establish the magnitude of QIN(I). The soil moisture index is adjusted at the end of each time interval in the following fashion:

$$EXRAIN = TRAINT * ROCOEF \qquad (4)$$

$$INFILT = TRAINT - EXRAIN \qquad (5)$$

$$SMIT_{t+1} = SMIT_t + INFILT - ET \qquad (6)$$

in which

EXRAIN = excess precipitation (mm/in)

TRAINT = total rainfall during the time interval (mm/in)

ROCOEF = runoff coefficient associated with SMIT as defined by SMI-ROP curve

INFILT = amount of precipitation assumed to infiltrate into the soil (mm/in)

SMIT = soil moisture index at the beginning of the time interval (mm/in)

ET = amount of moisture lost through evapotranspiration (mm/in).

The above procedure is carried out for the duration of the storm and for each subbasin separately. This then allows for time and spatial variability of rainfall and differences in subbasin characteristics. The outflow hydrograph as calculated by Equation 1 is automatically extended by 25 time increments past the duration of the storm event.

ROUTE

ROUTE performs channel flood routing using the Muskingum method. The channel routing procedure is used to move the computed flood hydrograph downstream to either the upper end of the reservoir or to a point where it will be combined with a runoff hydrograph from another subbasin. General effects of channel routing are an attentuated and lagged runoff hydrograph. These effects occur because of channel storage and the time of travel required to propogate the flood wave downstream.

The Muskingum method, developed by McCarthy (1938) based on observed data from the Muskingum Conservancy District, Ohio, is cited as one of the most commonly used hydrologic channel routing procedure (Chow 1964, Linsley et al. 1975, and Viessman et al. 1977). The two parameters which are used in the method are calculated from known flood hydrographs or are easily assumed for a subbasin. The first of the parameters, K, is a storage time constant having units consistent with the time interval and the second, x, is a weighting factor applied to the inflow ordinates. Equation 7 shows the form of relationship according to McCarthy.

$$S = K[xI + (1-x)\ O] \qquad (7)$$

in which

S = storage at time t

I = inflow

O = outflow

Substituting this equation into the basic hydrologic principle of continuity, the Muskingum form is written as:

$$O_t = C_0 I_t + C_1 I_{t-1} + C_2 O_{t-1} \qquad (8)$$

The coefficients are defined as:

$$C_0 = \frac{-Kx + 0.5\ \Delta t}{K - Kx + 0.5\ \Delta t} \qquad (9)$$

$$C_1 = \frac{Kx + 0.5\ \Delta t}{K - Kx + 0.5\ \Delta t} \qquad (10)$$

$$C_2 = \frac{K-Kx - 0.5\ \Delta t}{K-Kx + 0.5\ \Delta t} \quad \text{.....(11)}$$

As a general principle, K can be assumed equal to the time of travel through the channel reach. A value of x, between 0.0 - 0.5, is chosen based on engineering judgment and experience or from an analysis of results from several trials of varying x. The most common value is about 0.2 (Linsley et al. 1975).

RESVOR

This third major component of the model is the reservoir routing subroutine. Its purpose, as implied, is to hydrologically route the computed runoff hydrograph from the entire watershed through the reservoir and calculate the discharge hydrograph from the dam. In RESVOR, the basic hydrologic storage-routing procedure, also known as the Modified Puls Method (Viessman et al. 1977), was used. This method is represented by Equation 12.

$$I_{t-1} + I_t - O_{t-1} + 2/\Delta t * S_{t-1} = 2/\Delta t * S_t + O_t \quad \text{.....(12)}$$

Characteristics of the reservoir are provided through a data file named DAM.DAT. This information includes the stage-storage curve and stage-discharge data for the principal and emergency spillways. Since the rating curve for a multi-gate spillway system consists of a family of curves, a table of elevation and corresponding discharge from each curve is given (see the section on INPUT in this chapter).

The user provides an initial set of gate openings for the spillway system and from this information an elevation versus total discharge curve, H-TOTALQ, is computed internally. Also computed is a curve known as the storage-indication curve, TOTALQ-STOQ, (Linsley et al. 1975). It defines a relationship between the total discharge and the term expressed as the right side of Equation 12. Once these curves are developed, the actual routing procedure can begin, given the initial water surface elevation of the lake.

For consecutive time intervals, the left side of Equation 12 is calculated since all of the variables are known. With this value, the TOTALQ-STOQ curve is entered to obtain the corresponding discharge from the dam, TOTALQ. The elevation of the water level in the reservoir, WSEL, is interpreted based on the H-TOTALQ curve that was computed earlier. The volume of stored water is obtained by entering the stage-storage curve at a stage equal to WSEL. This procedure is repeated for a maximum limit of 50 time intervals.

A feature designed to let the user vary the gate settings during a run has been built into RESVOR. At one meter increments of water surface elevations, the routing procedure is interrupted and the user is given an opportunity to reset the spillway gates. If the user does not wish to change the existing settings, the program continues the routing using the previously calculated curves. If a change in the spillway gate openings is desired, the program returns to that portion of the model where the gate openings are set and then recalculates the necessary curves based on the new spillway conditions. Once the reservoir routing procedure is completed, the user also has the option

to reroute the inflow flood hydrograph through the reservoir for varying initial water surface elevations and/or spillway scenarios.

SUM

This subprogram is used to combine two hydrographs at the outlet of a subbasin. All hydrographs are referenced to the time of the first occurrence of rainfall anywhere within the basin. For example, suppose that the hydrographs from subbasins 1 and 2 in Figure 2 are to be added, however, the storm in subbasin 1 started on day 1 and the storm in subbasin 2 did not begin until two days later. The hydrograph from subbasin 1 is stored as FLOW(1) through FLOW(51). Likewise, the hydrograph from subbasin 2 is stored as QOUT(1) through QOUT(51). But QOUT(1) andQOUT(2) are equal to zero since the storm in subarea 2 did not start until day 3.

Note that the hydrograph from subbasin 1 is not stored in array QOUT but in array FLOW. This was accomplished by executing SUM before the runoff from subbasin 2 was calculated. In order to reduce the required storage space for arrays, the runoff hydrographs from subbasins are printed to an output file, and QOUT is used only as a temporary array. When SUM is called, the resulting summation is stored as FLOW.

FLOW(I) = FLOW(I) + QOUT(I).....(13)

FLOW is passed to the subroutines ROUTE, RESVOR and ERROR and, therefore, SUM must be executed immediately following the execution of

RUNOFF. The argument in arrays FLOW and QOUT are relative to the first occurrence of rainfall anywhere in the basin.

ERROR

This subroutine was designed as an aid in calibrating the model. Its purpose is to compare the computed runoff hydrograph to a known flood hydrograph. The subroutine computes various measurements which are used to evaluate the effectiveness of the program. These measurements include the absolute errors, relative errors, an average relative error for the run, the sum of errors squared, and a coefficient of determination. The absolute error, ABSERR, is calculated as the simulated flow (SIMQ) minus the recorded flow (RECQ).

$$ABSERR = SIMQ - RECQ \qquad (14)$$

Relative error, RELERR, defines the absolute error as a percentage of the recorded streamflow.

$$RELERR = \frac{ABSERR}{RECQ} * 100\% \qquad (15)$$

The coefficient of determination, RSQ, is computed as the standard statistical definition of

$$RSQ = \frac{cov^2\ (SIMQ,\ RECQ)}{var\ (SIMQ) * var\ (RECQ)} \qquad (16)$$

Also calculated are the runoff volumes for the simulated and recorded hydrographs, presented in units of streamflow rate x time.

ERROR can be executed at any location within the watershed as long as there is a known hydrograph at that location.

DTCURV, APPROX and INTERP

These three subprograms cannot be invoked by the user, but are automatically called when RESVOR is executed.

DTCURV is called to set the initial openings of the spillway gates and whenever the user wishes to adjust the gates. Within the subroutine, the stage-total discharge curve and storage-indication curve (discussed in the section on RESVOR) are calculated.

APPROX and INTERP are interpolation routines used in the program to estimate values from either the input tables or calculated curves. APPROX interpolates and extrapolates using Lagrange's formula (Carnahan et al. 1969); and INTERP is a linear interpolation routine.

Model Calibration and Verification

To apply the computer model, MAIN, to the Bafing River basin, the parameters used in the model required calibration. Since the number of parameters to be calibrated is small, a manual procedure was used instead of an internal optimization routine such as the systematic pattern search refined by Lumb et al. (1975). The parameters calibrated were: XMUSK, the weighting factor in the Muskingum routing equation; CQIN, the routing constant used to define the shape of the runoff hydrograph from the subbasin; and the SMI-ROP curve which relates percent runoff to soil moisture conditions.

The manual procedure employed was that of a trial-and-error method whereby the parameter values were adjusted for each execution

of the program until the computed hydrograph satisfactorily matched the known hydrograph. The subroutine ERROR was used during this procedure to provide some statistical analysis of the match between the two hydrographs. A satisfactory match was a subjective judgement but based on the following criteria: 1) magnitude and timing of the peak discharge; 2) match along the rising segment of the hydrograh to the inflection point on the receding segment of the hydrograph, 3) the overall correlation between the computed and known ordinates using the coefficient of determination, R^2, desired to be larger than 80 percent, and 4) the relative error and average relative error of 20 percent or less.

In this study initial values for the SMI-ROP curve were taken from a U. S. Corps of Engineers study on the Mekong River (U. S. Army Corps of Engineers 1971). This reference provided a good estimate which was later adjusted to closely approximate the volume of runoff given a recorded hydrograph. Changes to the initial curve were made after examining the computed daily soil moisture indices and runoff coefficients with the computed and recorded hydrographs. It was not possible to define a unique SMI-ROP curve for each subbasin since the flood hydrographs were recorded at only one gaging location. Accordingly, an assumption of hydrologic homogeneity was made in order to determine a general curve that would be characteristic of the entire basin.

During the calibration it became clear that a single curve was not representative of the runoff during different storm events. Examination of the rainfall-runoff records suggested that rainfall intensity played a major role in determining the percentage of

rainfall excess. Therefore, a two additional SMI-ROP curves were added to the model for estimating runoff during high intensity rainfalls. During the calibration process, the following three intensity ranges were established and a different SMI-ROP curve was developed for each: 0-19 mm/day, 20-44 mm/day, and > 45 mm/day. The SMI-ROP curves were a major parameter in defining the volume of runoff and, to a large extent, the peakedness and general shape of the hydrograph.

The parameter XMUSK could not be calibrated from historical hydrographs at two adjacent streamgages as presented in the hydrologic textbooks such as Linsley et al. (1975) and Viessman et al. (1977). Actual upstream and downstream hydrograhs were not available and local inflows are significant, so that an accurate calculation of channel storage was not possible. The procedure used involved reconstituting a flood hydrograph at the Soukoutali gaging location (Figure 2) based on a recorded hydrograph at Dibia and regression parameters determined by Groupement Manantali (1977). Local inflow, due mainly to the tributary Balin-ko, was computed by executing RUNOFF (using average estimates of the parameters). The reconstituted hydrograph was routed down the 50 km reach of the Bafing River, local inflow was added to the routed hydrograph and the resulting hydrograph was then compared to the recorded values at the Dibia station. This procedure was repeated for different values of XMUSK. The value of XMUSK which yielded the greatest coefficient of determination was chosen to be characteristic of the other reaches in the Bafing basin.

COIN was calibrated based on runs which modeled the basin in a sequential fashion. Estimates of COIN were determined by the

procedure described earlier in this Chapter. Flood hydrographs at the gaging stations of Soukoutali and Dakka - Saidou were estimated using the recorded hydrographs at Dibia as presented in the Senegal-Consult Vol 3A (1970) and the regression parameters in the Groupement Manantali report.

First, the upper portion of the Bafing River basin, the drainage area above the Dakka-Saidou streamflow station (see Figure 2), was modeled. Subbasins 1 and 2 are similar (drainage area, channel length, and slope) so it was assumed that CQIN for each would also be similar. Different values and combinations of CQIN were inputted for both subbasins 1 and 2 and for subbasin 3 during various runs. Those values which resulted in hydrographs correlating well with the estimated hydrographs at Dakka-Saidou were used as the initial values for the second stage of modeling.

The second stage modeled the basin above the Manantali Dam site at Soukoutali. During this stage parameters for subbasins 4 and 5 were determined. In addition, the previously estimated parameters for the upper three subbasins were altered as required to match the hydrograph expected at Soukoutali. Finally, the entire watershed above the Dibia station was modeled to reconstitute the hydrographs obtained from the Senegal-Consult Report. The parameter values obtained through this third stage were considered to be the calibrated parameters for the basin.

Inherent to the calibration of a hydrologic model is the use of good input data, and consistent procedures and criteria. Since rainfall is the major input to a rainfall-runoff model, inaccurate or inconsistent data can greatly affect the results. Therefore, for

calibration of this model, the Thiessen method of areal distribution of precipitation was used. Changes to the diagram were considered if there were reasonable causes and if the corrections were consistently followed.

Four of the six years of flood hydrographs available were used to calibrate the model parameters following the above procedures. The four sets of calibrated parameters were examined for similarity (the objective of a lumped parametric model) and a compromised set of values was established and used in the verification of the model.

In verification or model testing, the basic issue is to determine if the hydrologic estimates achieved by calibration are acceptable (James and Burges 1982). Therefore, the process involves running the model for an event or period of time other than that used for calibration. In this study, other events were not available or applicable. Instead, verification was defined as showing the model's applicability to the Bafing River Basin. Verification of MAIN consisted of executing the model for the four floods using the averaged parameters. Results of these runs were judged on the four criteria presented at the beginning of this discussion. If the computed hydrographs were considered acceptable, the averaged values were chosen as the parameters used to represent the basin. If unacceptable, the calibration runs and model itself were examined to define problem areas. The results of the calibration are described in Chapter VI and selected verification runs are presented in Appendix B.

INPUT and OUTPUT

INPUT

Most of the data required to execute MAIN is read from input files that the user must establish prior to running the program. Data files are read during the routines MAIN, RUNOFF, and RESVOR. The first data file associated with running the program is named SEQEXE.DAT. This file contains the sequence of subroutines that are to be executed during the run.

While executing RUNOFF, information concerning the subbasin and rainfall pattern must be supplied. These data are read sequentially from a file named SBX.DAT, where "X" corresponds to the number of the respective subbasin. From the schematic of the watershed in Figure 2, one would require five data files: SB1.DAT; SB2.DAT; SB3.DAT and so forth. The type of data required are:

1. Watershed characteristics.
2. Time-area curve.
3. Storm hyetograph.
4. Soil moisture index - runoff coefficient curve.

DAM.DAT is a data file used by RESVOR and contains the physical characteristics of the reservoir and dam, such as the stage-storage relationship and stage-discharge curves.

Other information used in the program is supplied by the user through interactive commands. The user is prompted to enter such data as gate settings and initial water surface elevation as well as responses to some instructive questions. See the INPUT-OUTPUT section of Appendix A for exact details and formatting of the data files.

OUTPUT

Results from a run are stored in output files but are not automatically printed to the terminal. The main output file is called FLOOD.DAT and contains results from ROUTE, SUM, and RESVOR (see example printout in Appendix A). Intermediate values of the runoff hydrograph are printed each time the hydrograph is routed through a channel reach or whenever runoff from a subbasin is added to the hydrograph. These intermediate values are shown in the tables entitled, "Routed Hydrograph at the End of Reach # X" and "Combined Hydrograph at the Outlet of Subbasin # X". The hydrograph values in these two tables represent only the direct surface runoff and do not include the baseflow. Results from the reservoir routing are listed under "Outflow Hydrograph from Manantali Dam" and are presented as the inflow hydrograph to the reservoir including baseflow, the outflow hydrograph, water surface elevations, and storage volumes. Also printed are the initial water surface elevation, maximum water surface elevation, and maximum discharge from the dam.

The runoff hydrographs computed during RUNOFF are stored in a separate file named HYDRO.DAT, and the table, "Runoff Hydrograph from Subbasin # X", presents the calculated runoff hydrographs from the model, TOTALQ. If ERROR is executed within a run, an output table is established which presents the computed and recorded streamflow values as well as the statistics mentioned in the description of the subroutine.

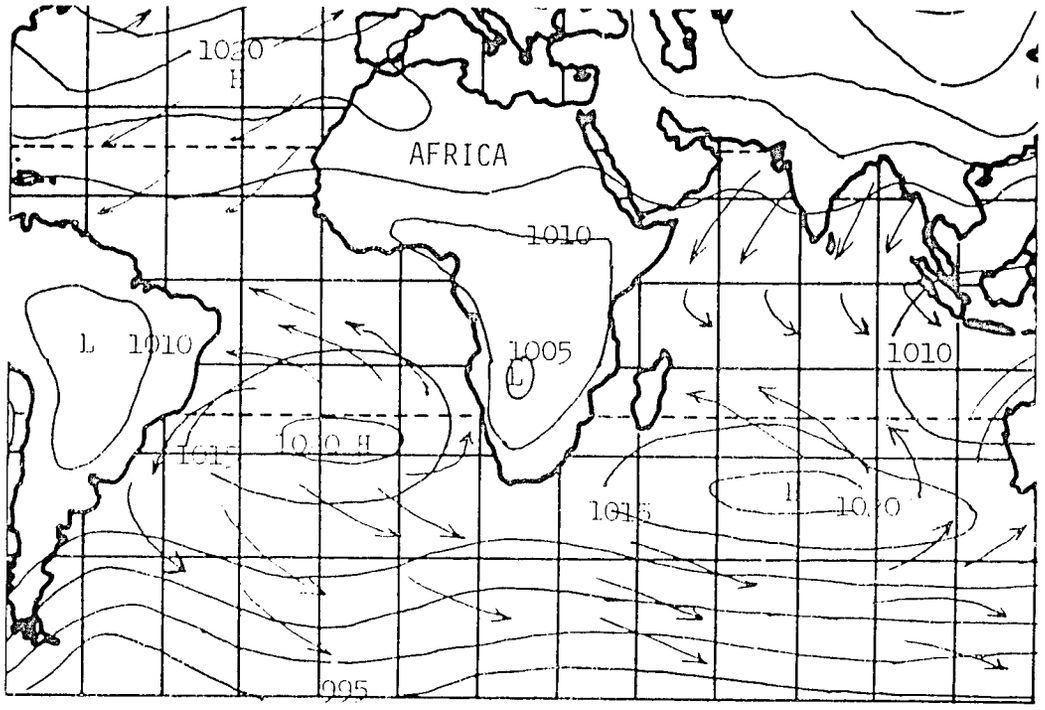
CHAPTER V

DEVELOPMENT OF THE PMP

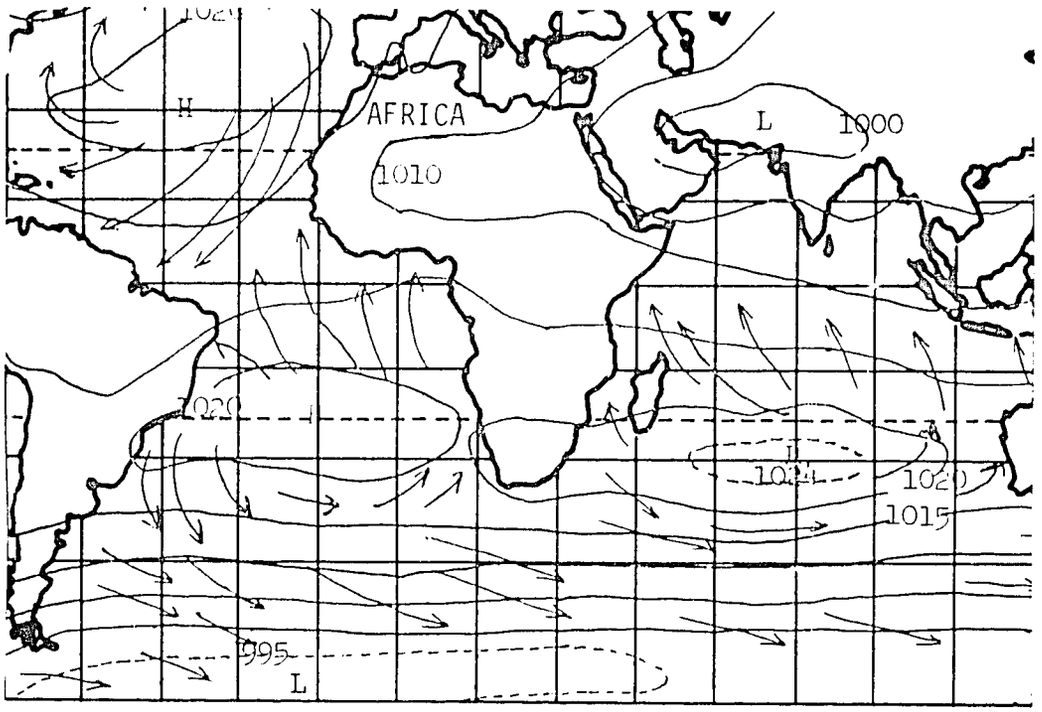
Introduction

The Senegal River basin lies within a meteorological zone whose climate is influenced primarily by prevailing easterly winds and the southwest monsoon (Figure 3). During the rainy season (late spring through summer), the monsoon brings the humid Atlantic air inland while during the dry season, the prevailing easterlies bring the dry air from the Sahara Desert. Throughout the watershed, the climate varies significantly. At the northern boundary, a semi-arid climate prevails. As one moves south, the climate becomes more sub-tropical and finally tropical in the southernmost portion of the basin. Three climatic regions have been defined, the Sahelian, Sudanian, and Foutanian (Senegal-Consult 1970).

The Sahelian region is characterized by its semi-arid climate. Rainfall is very irregular with annual values ranging from 250-700 mm occurring within a three month period (July-Sept). Much of the Senegal River basin, including the lower portion of the Bafing River, lies in the Sudanian region. The rainy season in this region usually extends from June through October and delivers 600-1200 mm of rainfall in the form of intense, short duration storms. In the Foutanian



Mean Sea Level Pressure in January (Millibars) and wind directions



Mean Sea Level Pressure in July (Millibars) and wind directions.

Figure 3. Maps of the prevailing winds (from State Climatologist, Utah 1982).

region, the climate shows its tropical characteristics with a wet season that lasts 6 to 8 months and receives 1500-2000 mm of precipitation per year. Though the regions differ greatly in annual precipitation amounts, all three exhibit a well defined rainy and dry season. The length of the rainy season and quantity of precipitation received generally decreases from the Foutanian region (south) to the Sahelian region (north) (Figure 4).

The climatic variations in the basin, as well as the entire West African subtropics, are controlled by the annual north-south movement of the Intertropical Convergence Zone (ITCZ) (Nicholson 1980). Figure 5 shows its typical range between January and July. A gross summary and the implications of the ITCZ are presented here so that the reader may appreciate some of the complexities and variances in the precipitation patterns.

The ITCZ is a zone which separates the moisture-laden maritime air mass over the Atlantic Ocean and the dry continental air mass over the Sahara Desert. The northward migration of the zone follows the northward shifts in the location of the overhead sun and so by July or August the ITCZ reaches its maximum northern extent. The migration of the ITCZ and the duration of the prevalence of each of the air masses account for the differences in the lengths of the wet seasons in the regions described above. In addition to its variations in the seasonal positions, the ITCZ also exhibits considerable variations in the diurnal positions it occupies. From figures presented in Ojo (1977), these fluctuations range between 36-60 km.

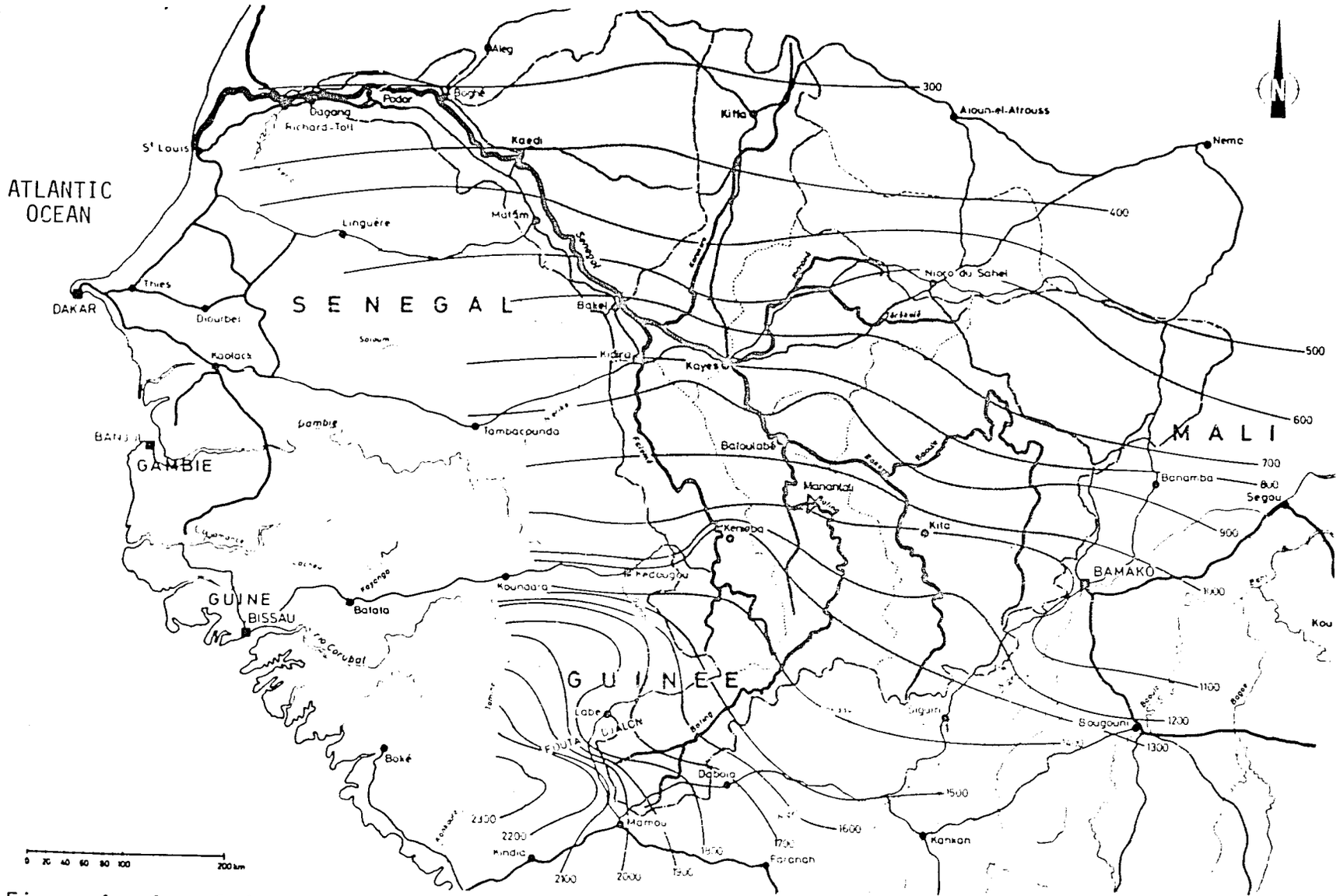


Figure 4. Annual isohyetes of precipitation (mm) in the Senegal River basin (from Groupement Manantali 1977).

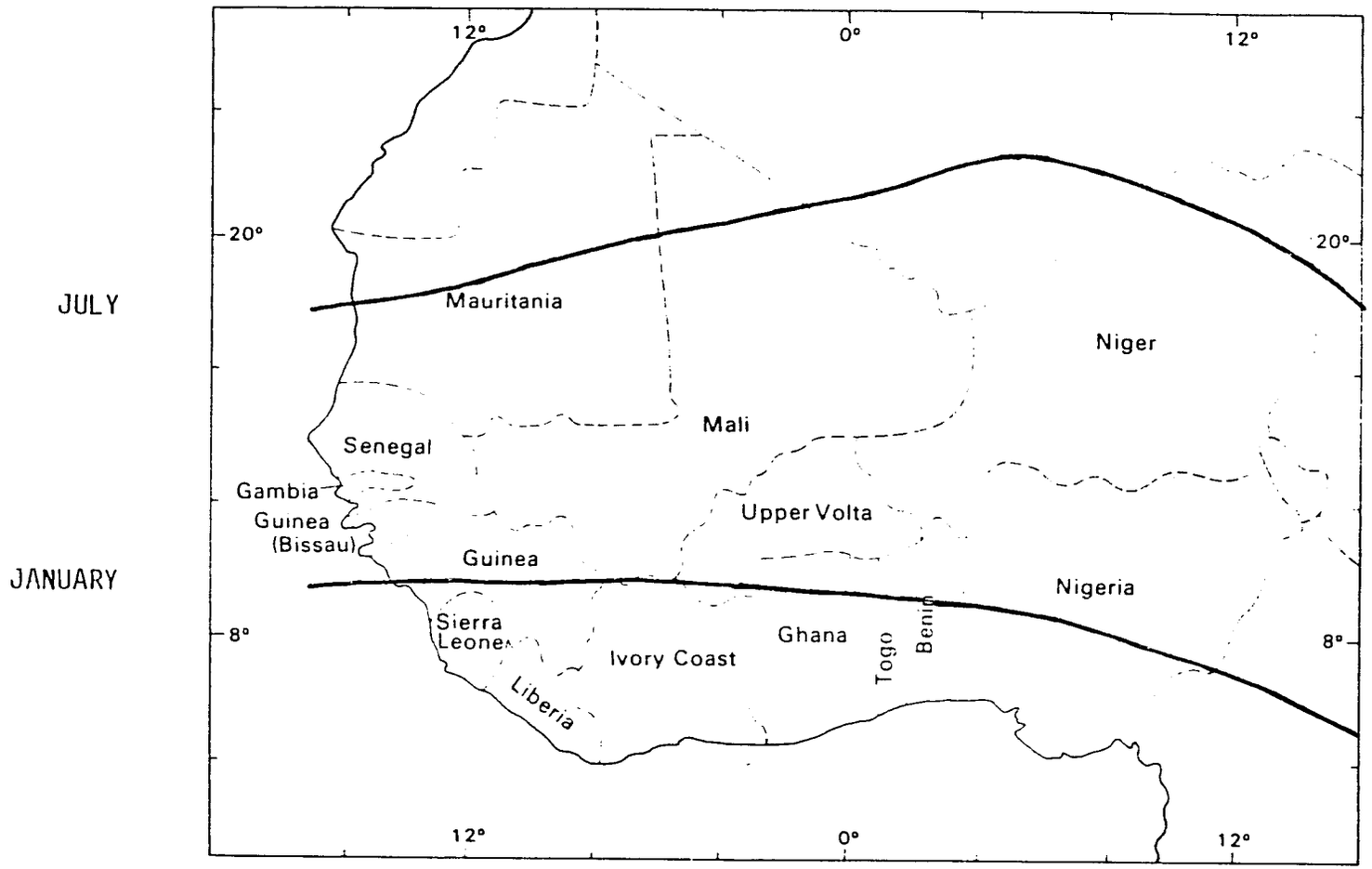


Figure 5. Annual movement of the ITCZ (from Nicholson 1980 and Ojo 1977).

To the south of the ITCZ lies the moist air associated with the southwesterly winds and the high potential for rainfall. Though the potential for precipitation is high, rainfall is not guaranteed. The reasons for this are the influences of the local conditions and the upper-air mass above the moist air that may inhibit the development of rain-bearing clouds.

During the wet season in the Bafing River basin, rainfall is commonly generated by convective disturbances. Other disturbances that produce significant precipitation are associated with the southwesterly flow, line squalls, and disturbance lines. The disturbance lines and line squalls are generally oriented north-south and move east-west. Their occurrences are largely associated with the easterly winds and, therefore, occur frequently during the early and late times of the rainy season when the southwesterly flow is weak. Rainfall during these disturbances is intense but usually lasts for only several hours. The southwesterly disturbance is usually accompanied by an extensive area of bad weather with rainfall less intense but longer than the other disturbances (Ojo 1977). As indicated by the above descriptions, storm events are of short duration. Recorded daily precipitation indicates substantial quantities over 2-3 consecutive days and then a 1-2 day period of little or no rainfall before the next burst.

Such disturbances can be widespread. Riehl (1979) describes a mesoscale rain system that usually covers an average area of 2000 sq km. Most of the precipitation of rainstorms comes from these systems, where a rainstorm is defined as an event whose daily area-averaged precipitation exceeds the seasonal daily average precipitation. By

this definition, the duration of a rainstorm is variable and ranges of 1-9 days has been observed by Riehl. Though the disturbances are widespread, rainfall is still variable and sporadic. As Riehl (1979) points out, there is no apparent continuity in the day to day rainfall pattern. He believes that the rainfall results from in-situ developments rather than any type of travelling system.

Analysis of Rainfall Data

Daily rainfall records for stations in Mali were available for varying years of record up to 1965. Prior to using these data, they were checked for any inconsistencies that may be reflected in the values by conducting a double-mass curve analysis. Cumulative August rainfall totals were plotted against the cumulative values recorded at Bafoulabe. In general, a double-mass curve will plot as a straight line if the data have a constant proportionality between them. Plots of the Mali data showed a more-than-desirable deviation from a straight line as seen in Figure 6. Analysis of Figure 6 indicates that during a period of time (line B), the Kita data does not have the same relationship with the Bafoulabe data as represented by the earlier years (line A). Similar breaks are represented by lines C and D. Since information pertaining to the history of the rainfall stations is not available, no logical reason can be given for these discrepancies. In general though, the deviations are not too great and no continuous trends or breaks could be identified in the data analyzed. It was therefore concluded that, for this study, the recorded values were consistent and that no adjustments were required.

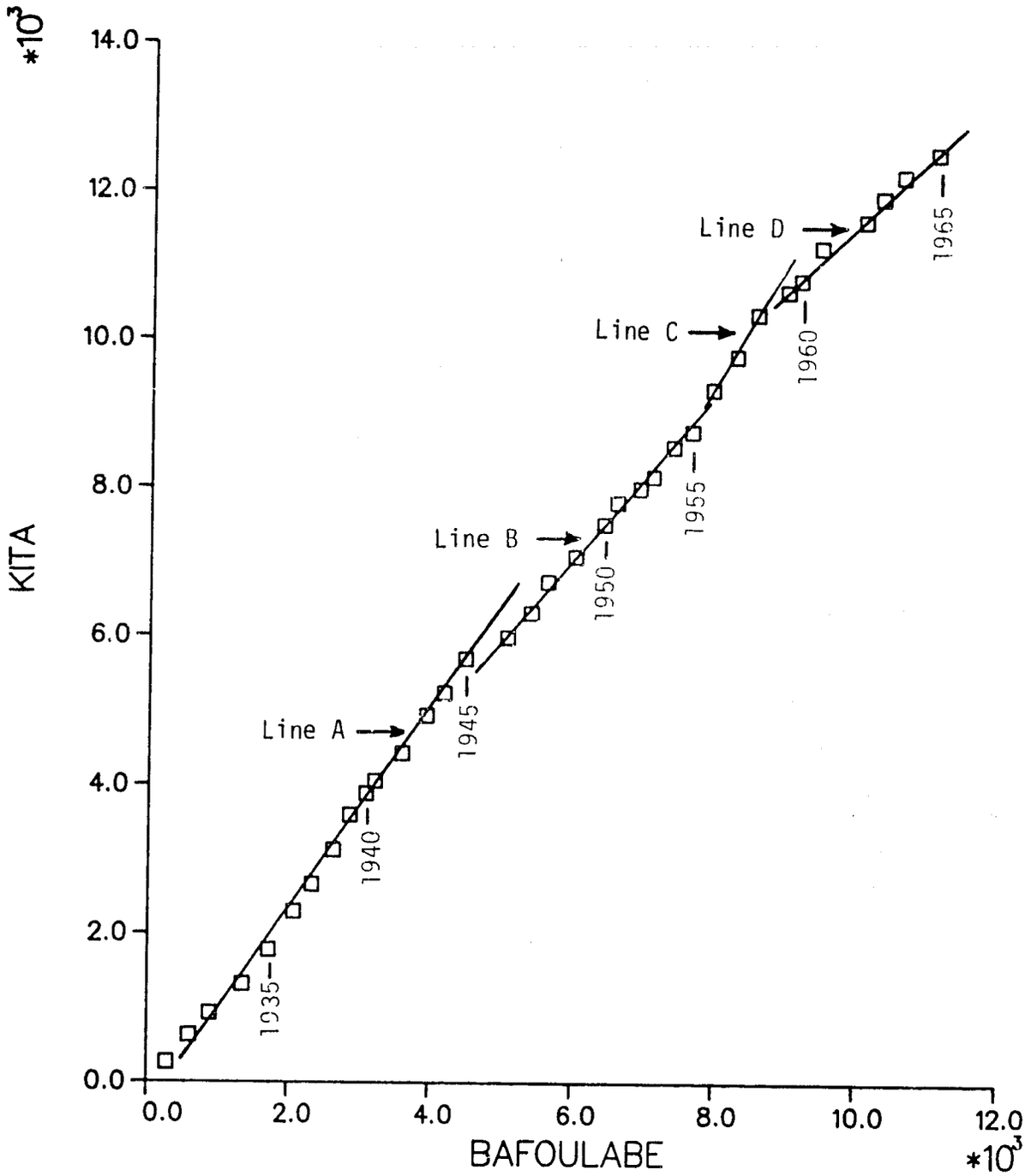


Figure 6. Double-mass plot of August rainfall (mm).

As mentioned earlier, the rainfall in the basin is variable and sporatic. To examine this characteristic, single mass curves were drawn for selected periods corresponding with the available flood hydrographs. For the flood period, a chart of cumulated rainfall versus time was plotted for each precipitation station on the same figure (see Figure 7). The curves verify that no definite daily correlation exist among the gages. However, the curves do indicate widespread areal extent of rainfall throughout the flood period as suggested by Riehl (1979). This suggests that the assumption of rainfall occurring over much of the basin at the same time is quite realistic.

The traditional or meteorological approach to estimating the probable maximum precipitation was not applicable to the Bafing River basin. This is because a high atmospheric moisture prevails during most of the rainy season, and therefore, there is no theoretical or empirical reason to relate rainfall potential to minor fluctuations in humidity (World Meteorological Organization 1973). Instead, an approach combining the statistical and historical methods was developed. The steps involved are:

1. Fitting a probability distribution to the annual maximum monthly precipitation.

2. Selecting a return period that is related to the probable maximum value of rainfall.

3. Determining a reasonable duration for a flood producing storm and a reasonable percentage of the maximum monthly rainfall that would be likely to occur over the duration selected.

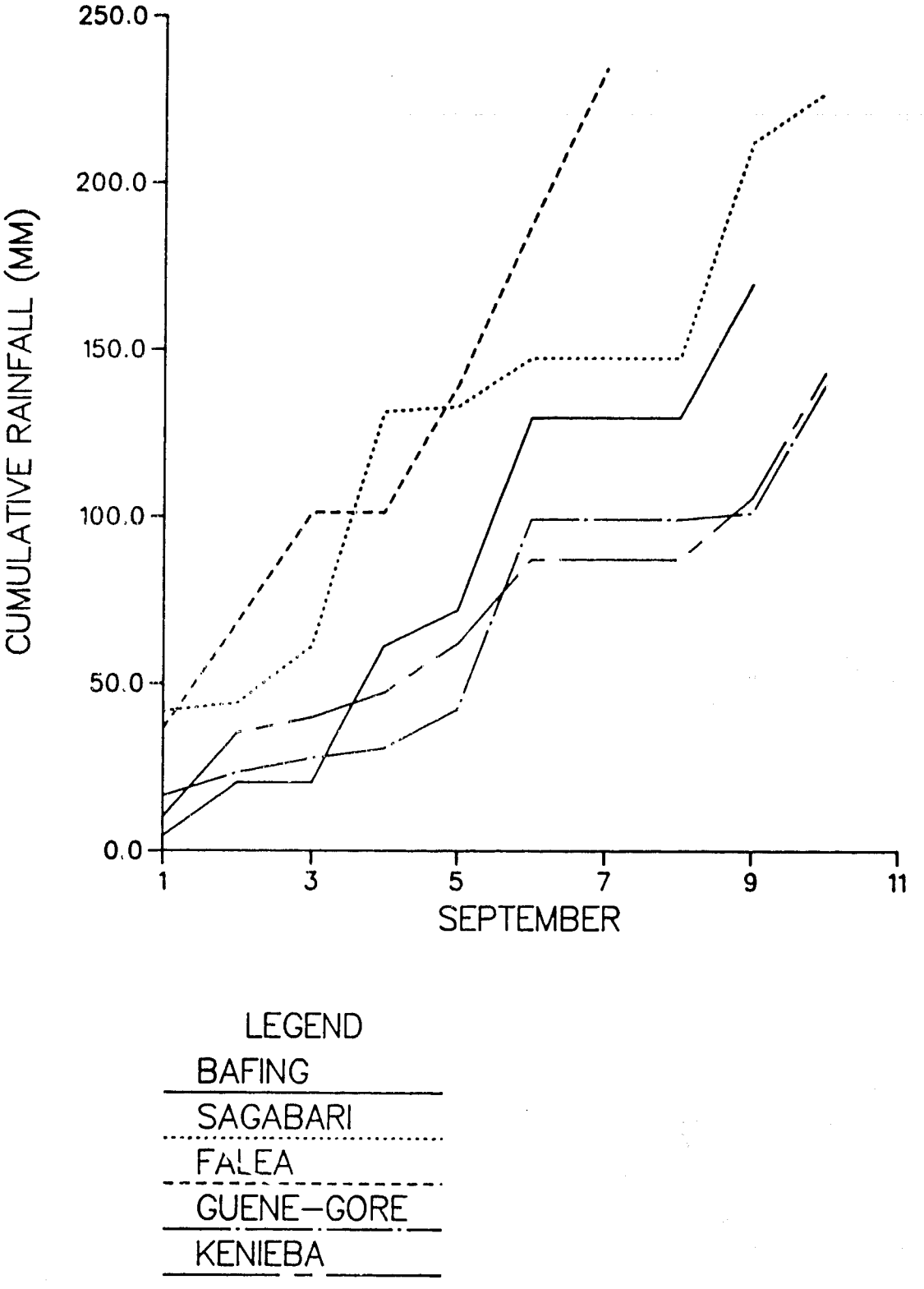


Figure 7. Rainfall mass curve for September 1-10, 1965.

4. Distributing the selected proportion of rainfall over the chosen duration to produce the most critical runoff situation with respect to peak discharge from the reservoir.

In step 1, the maximum monthly precipitation recorded for each year at a station was ranked and plotted using the Weibull plotting position (Haan 1979). Only those stations whose record length exceeded 17 years were examined. The gamma distribution then was fitted to the data using a computer program presented by Hardee (1971). Selection of the gamma distribution was based on the Hardee (1971) report and on the references presented in Haan (1979). These articles recommended fitting rainfall data for long durations, such as weeks or months, with the gamma distribution. Figure 8 shows a plot of the gamma distribution fitted to the Kita station data.

As cited earlier in this report, the probable maximum precipitation (PMP) is not normally associated with a probability level. However, it has become accepted engineering practice to use a 10,000-year return period when referring to an extremely rare event. Based on such practice, a recurrence interval of 10,000 years (P(X) = 0.0001) was selected to estimate the PMP for a monthly duration.

Thus far in this study, the procedures used were fairly straightforward and objective in nature. The remaining steps of the methodology are more subjective and judgment and assumptions were required in analyzing the historical data.

To determine the duration and percentage of the monthly rainfall that should be used to develop the probable maximum storm, rainfall events were examined that corresponded with peak flood hydrographs.

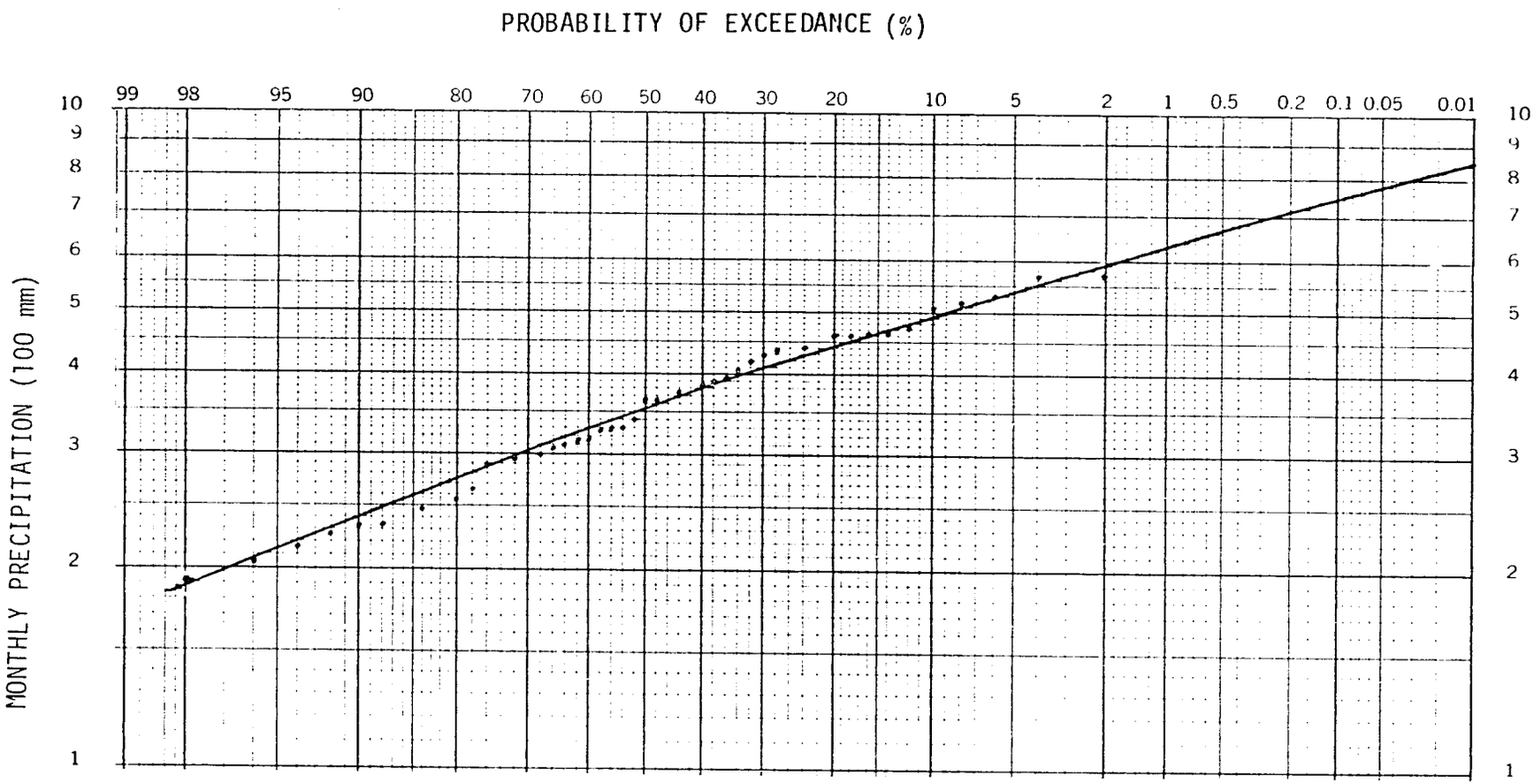


Figure 8. Gamma distribution for the maximum monthly precipitation at Kita.

Storm events were selected on the criteria that they started on or after the day preceding the initial rise in the hydrograph and ended by the time the peak discharge occurred. Using these criteria, daily precipitation records, and the recorded flood hydrographs, 25 storms were selected. The data showed storm durations of 4 to 10 days, with an average duration of 7 days. Assuming 7 days as an appropriate duration for the maximum storm, an analysis was conducted of the maximum consecutive rainfall amounts for durations of 1 through 7 days. Three such studies, described in the following paragraphs, were completed using selected rainfall stations and years of record. These were performed to aid in determining the temporal distribution of the rainfall.

Initially, a duration - percentage curve was completed for each of the 8 stations listed in Chapter 3. Only one curve was developed for each location using the daily data of the recorded maximum month. Once plotted the initial curves indicated a slight tendency to differentiate between two sets of data. One set seemed to represent those stations located in the lower portions of their respective river basins, while the other curves represented the stations located in the headwater portions. Average curves were drawn to reflect the two sets.

Upon completing the initial duration curves, two other sets of data were analyzed in a similar fashion. The data used were from two stations only, Kita and Falea. Data were selected on the criterion that the total precipitation for a particular month was larger than the average monthly total for that station. The plotted results clearly identified one outstanding curve for each station. The three

sets of results were compared and a composite curve was constructed representing the maximum percentages for each duration from 1 to 7 days. Depicted on Figure 9 is the composite curve along with the two average curves developed in the initial analysis. The composite curve shows a maximum daily percentage of 26.6 percent and a maximum 7-day percentage of 53 percent of the monthly total while the durations in-between follow a smooth curve. Figure 9 was developed by analyzing the daily data of a maximum month. When data for the entire wet season were similarly anlyzed, the duration-percentage curves resulted in slightly lower values.

One other study related to step 3 was conducted on the 25 storms that corresponded to five flood peaks. The concern of this study was concentrated on the maximum daily precipitation that occurred within these storms. Findings of the study indicated that for each flood, at least one station recorded a maximum daily precipitation between the range of 45-55 percent of the total storm rainfall. This range corresponds well with the percentages obtained through the duration - percentage curves if one assumes the 7-day percentage as 100 percent. A trend in the temporal distribution of the one day amount during an event does not exist in the data. That is, the maximum one day amount randomly occurs at any time during the storm event.

To further verify the maximum daily amount, the data at the stations of Kita and Falea were examined. This study yielded a maximum daily percentage in the range of 17-35 percent of the monthly total, with the exception of a few isolated one day events. Based on

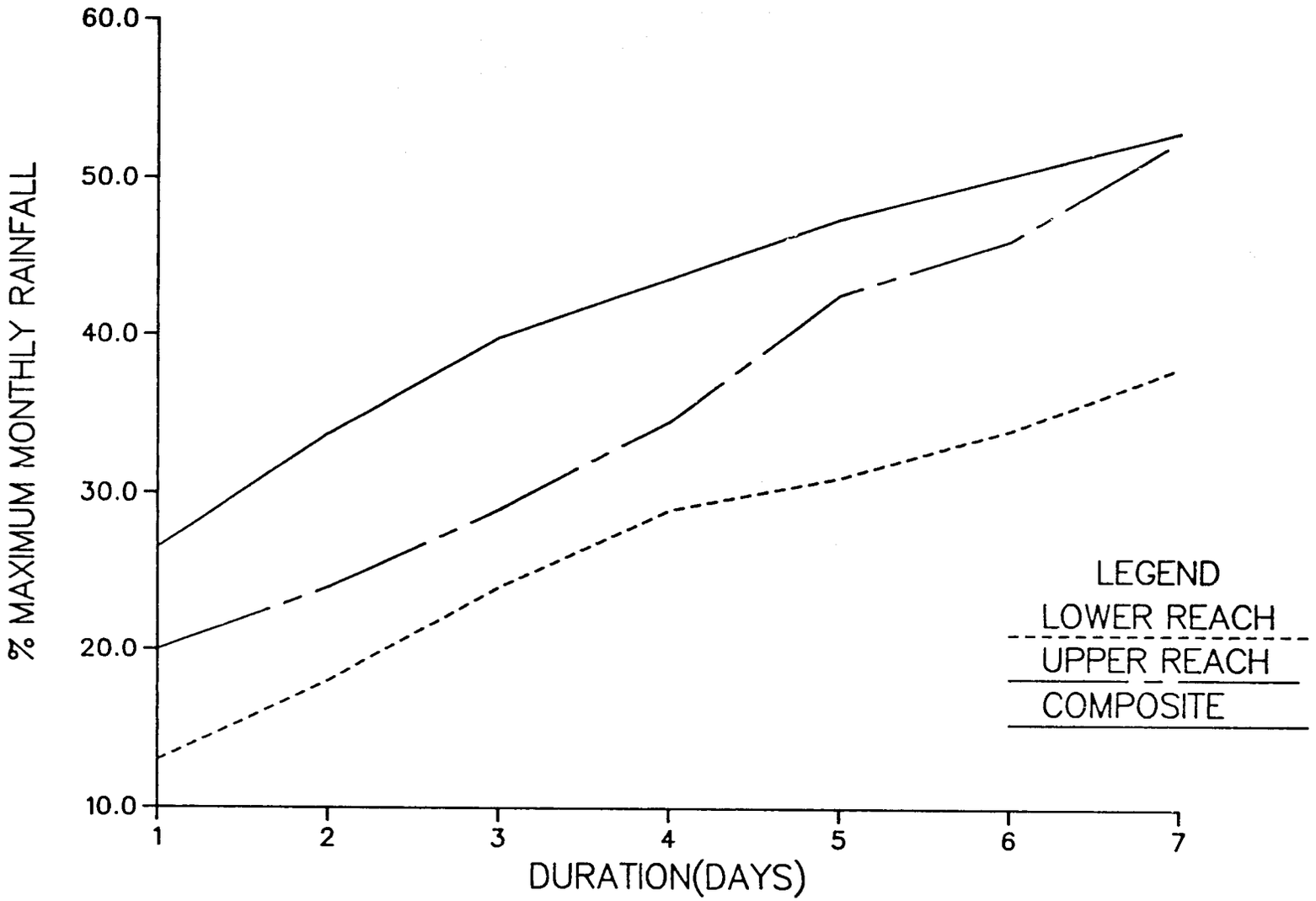


Figure 9. Duration - percentage curve.

this and the preceding analyses, a maximum one-day precipitation amount equal to 25-35 percent of the monthly value would be a realistic occurrence in the PMP storm.

Guidelines for the temporal distribution of the PMP value are very limited. Distribution of the rainfall to obtain the most critical situation is generally placed upon the individual hydrologist for his specific needs (Hansen et al. 1982). The Hydrometeorological Branch of the National Weather Service presents some guidance for sequencing the rainfall based on the historical storms that they have studied (Hansen et al. 1982). In general, it is suggested that the increments of rainfall progressively decrease on either side of the greatest incremental value. This procedure defines a storm with a single peak. Placement of the peak is arbitrary but it is recommended that it not be placed at the beginning or end of the sequence.

The same agency also conducted a study to estimate the probable maximum precipitation in the Mekong River basin which is under the influence of monsoon winds (U. S. Weather Bureau 1970). The time distribution developed in their study is a 7-day duration with a sequence of two 3-day storms interrupted by a day of little rainfall.

These recommendations were followed for distributing the rainfall data over the 7 day period. Various distribution schemes were tried and the final selection was made based on which scheme resulted in the highest lake level during the flood routing analysis.

CHAPTER VI

MODEL APPLICATION AND RESULTS

Calibration Results

Numerous runs of the computer model were made to determine the values of the model parameters that would best represent the runoff characteristics of the Bafing River basin. Throughout the calibration procedure, various sets of parameters were established that would better simulate one flood than the next. However, the main goal of the calibration was to find one set of parameters to adequately simulate all floods. After averaging and compromising, a final set of parameter values was determined that reproduced the four recorded flood hydrographs reasonably well using the available input data. Table 2 presents the calibrated parameters for the subbasins and channel reaches in the study area.

Also shown in Table 2 are the results of the Thiessen polygon method used to establish the rainfall input data. Because of the varying years of record, two Thiessen maps were constructed. One was drawn for years prior to 1963 using the stations of Falea, Guene-Gore, Kenieba, and Kita. The second map was used for floods after 1963 and included the stations of Bafing-Makana and Sagabari. Subbasins No. 5 and No. 6 required a 50/50 weighting of precipitation from two stations. The recorded point precipitation amounts were applied

Table 2. Calibrated model parameters and results of Thiessen rainfall weighting.

SOIL MOISTURE INDEX - RUNOFF COEFFICIENT CURVES (SMI-ROP)

SMI (mm)	ROP (%) 0-19[1]	20-44[1]	45 and above[1]
100	5	5	5
120	10	10	10
130	13	13	20
150	18	30	45
200	40	50	65
250	45	65	75
300	50	70	80

SUBBASIN

Parameter	1	2	3	4	5	6
CQIN	0.19	0.19	0.17	0.17	0.17	0.17
ET (mm/day)	4	4	4	7	7	7
SMIT (1965)	110	110	110	110	110	110
SMIT (1964)	130	130	130	130	130	130
SMIT (1958)	140	140	140	140	140	140
SMIT (1957)	120	120	120	120	120	120
Rainfall weighting						
Station (- 1963)	FALEA	FALEA	GUE-GOR	GUE-GOR	KITA	KEN/GG
Station (1963 -)	FALEA	FALEA[2]	GUE-GOR	BAF-MA	BAT-MA/SAG	KEN/GG

ROUTING REACH

	1	2	3
XMUSK	0.4	0.4	0.2
KMUSK (days)	1.3	0.7	0.8

ABBREVIATION	STATION NAME	
GUE-GOR	Guene-Gore	
BAF-MA	Bafing Makana	
BAF-MA/SAG	Bafing Makana/Sagabari	(50/50 weighting)
KEN/GG	Kenieba/Guene-Gore	(50/50 weighting)

[1]Rainfall intensities (mm/day).
[2]In the absence of Falea rainfall, KEN/GG was used.

directly to the subbasins in accordance with the Thiessen maps. Observed precipitation quantities were not adjusted for areal extent because there was insufficient data to determine a reasonable estimate of the adjustment factor. Also the daily data indicates the magnitudes of widespread rainfall to be very similar.

Copies of the output tables and plotted hydrographs for the calibration runs are presented in Appendix B. It should be noted here that the initial soil moisture conditions (SMIT) for the four calibrated floods were not constant values. The initial SMIT value ranged from 110 for the 1965 flood to 140 for the 1958 flood. Varying the SMIT value was considered acceptable because the parameter is used to define the conditions prior to a rainfall event. The importance of not restricting the SMIT to a constant is further demonstrated by comparing the rainfall and runoff for each calibrated flood (see Table 3). The 1958 flood recorded the largest flow rate and runoff volume yet received the least rainfall of all the floods. A possible explanation is that the soil moisture conditions were high at the beginning of the rainfall event.

PMP Results

The gamma distribution was fitted to the maximum monthly precipitation at five stations in the Upper River basin. Two of the five stations, Bafoulabe and Bamako Aero, were used only to check the assumption of using the gamma distribution and were not used in further analyses of the PMP. The three stations used to estimate

Table 3. Rainfall/runoff comparison of the floods used in calibration.[1]

Subbasin	Flood 1965	1964	1958	1957
1	234.4	171.9	132.4	175.3
2	234.4	171.9	132.4	175.3
3	139.6	138.0	139.2	172.7
4	169.8	132.5	139.2	172.7
5	198.8	151.7	192.2	60.0
6	142.0	171.9	125.2	132.4
Duration (days)	10	7	7	10
WGT Rainfall	180.0	153.0	143.0	148.0
Runoff	44.6	34.6	45.8	33.5
Runoff (%)	24.8	22.6	32.0	22.6

[1]Values listed are in units of millimeters unless noted.

the PMP value were: Kenieba, Kita, and Mamou. In general, the distribution fitted the observed values quite well although the Kenieba data exhibited some variance in the middle range of the distribution. Estimates of the monthly probable maximum precipitation based on a 10,000-year return period are given in Table 4. A surprising result of the statistical analysis was the higher values predicted for the Kenieba station when compared to the values at Mamou. This was not expected because the computed average of the maximum monthly rainfall at Mamou is 473 mm whereas the average of Kenieba is 434 mm. The reasons for the difference in the predicted values seem to be two-fold.

1. The standard deviation at Kenieba is significantly higher than the value at Mamou. This would tend to decrease the values of the two parameters in the gamma distribution which in turn relates to higher estimates.

Table 4. Monthly probable maximum precipitation (mm) and storm distribution.

Station Name	PMP	Station Mean	Standard Deviation	Record Length (yrs)
Kenieba	1116	434	125	38
Kita	847	361	99	49
Mamou	883	470	83	17

Time Days	Distribution of Monthly PMP (%) A	B	Duration - Percentage Curve (%)
1	3.9	2.8	26.6
2	4.8	9.6	33.7
3	7.4	2.8	39.8
4	7.4	0.0	43.5
5	25.0	7.4	47.4
6	3.7	25.0	50.2
7	2.8	7.4	55.0
Σ	55.0	55.0	

2. Only 17 years of record were available at the Mamou station compared to 38 at Kenieba. Review of the Kenieba data corresponding to the 17 years at Mamou showed that those years were typically lower than the other years. If this trend exists throughout the basin, the recorded Mamou data may not be a very representative sample of the true averages.

As previously indicated, to develop the storm event that would produce the probable maximum flood, a 7-day duration was used. Seven days was chosen after reviewing the historical rainfall events corresponding to flood hydrographs. This duration also was chosen on the basis of the U. S. Weather Bureau report (1970) on the probable maximum precipitation in the Mekong River basin. Based on the duration-percentage analysis, 55 percent was chosen to be the maximum

amount of a month's total rainfall expected to occur over a 7-day period. (Actual value from the analysis was 53 percent.) The maximum daily amount had a range of 25-35 percent of the month's total.

The computer model, MAIN, was used to determine the most critical temporal distribution of the 7-day event. Selection of the PMP distribution was based on that distribution which produced the highest reservoir elevation given a constant set of model parameter values. Several distributions were derived and modeled. Two precipitation distributions were selected in order to define two possible storm systems. Distribution A represents a continuous 7-day storm pattern and distribution B defines a system of two short duration events. These distributions, A and B, also are given in Table 4 along with the tabulated values of the composite duration-percentage curve that was presented in Figure 9.

Model Application

With the calibration completed and the PMP event defined, the model was applied to the Bafing River basin to meet the objectives of the project, namely, to develop and route the probable maximum flood (PMF). A Thiessen polygon, different than the ones used in the calibration procedure, was developed to areally distribute the rainfall of the three stations used in the estimates of the PMP. The previous maps were not applicable to the PMF development because they included precipitation weightings of stations that could not be used in the analyses for the PMP. Results of this diagram are as follows:

Subbasin	Station Rainfall
1-2	Mamou
3	Mamou/Kenieba
4	Kenieba/Kita
5	Kita

In the basins that show the influence of two precipitation stations, a 50/50 weighting was used. The same temporal distribution of the rainfall was used in each subbasin. This assumption was made because the recorded data did not indicate any type of general storm system movement across the basin.

Execution of the model to develop and route the PMF was conducted for varying initial conditions. The initial soil moisture index (SMIT) ranged from 110-140 and the initial lake surface elevation varied between 202.0 - 208.0 masl (meters above sea level). The normal operating level for the reservoir is planned at 208.0 which also is the elevation above which storage is allocated for flood control. Therefore, it was considered a reasonable assumption that the lake would not exceed this elevation except during the passage of a flood event. The baseflow was assumed to be 1500 m^3/s as used in the Senegal-Consult and Groupement Manantali reports.

Table 5, summary of the flood routings, lists the maximum inflow to the reservoir, peak discharge from the dam, maximum lake level and, if applicable, the number of days the lake level exceeded elevation 212.50 masl, the crown of the dam. Flow over the crest of the dam was not included in the stage-discharge curve so elevations computed to be

Table 5. Summary of the flood routings.*

DISTRIBUTION A:

	INITIAL LAKE LEVEL 202.00	204.000	206.00	208.00
SMIT = 110	7837.0	8096.0	8422.0	8886.0
INFLOW = 12610	211.54	211.85	212.25	212.81
	--	--	--	3
SMIT = 120	8039.0	8288.0	8594.0	9070.0
INFLOW = 12868	211.78	212.09	212.46	213.04
	--	--	--	5
SMIT = 130	8210.0	8468.0	8757.0	8902.0
INFLOW = 13122	211.99	212.30	212.66	212.83
	--	--	2	4
SMIT = 140	8392.0	8661.0	8929.0	9061.0
INFLOW = 13365	212.21	212.54	212.86	213.03
	--	2	4	4

DISTRIBUTION B:

	INITIAL LAKE LEVEL 202.00	204.00	206.00	208.00
SMIT = 110	7793	8029	8349	8829
INFLOW = 12583	211.48	211.77	212.16	212.74
	--	--	--	3
SMIT = 120	8058	8278	8554	8720
INFLOW = 12792	211.81	212.08	212.41	212.61
	--	--	--	2
SMIT = 130	8262	8555	8779	8925
INFLOW = 13000	212.06	212.41	212.68	212.86
	--	--	2	3
SMIT = 140	8011	8771	8990	9103
INFLOW = 13216	211.75	212.67	212.94	213.08
	--	2	4	4

*Table values represent the following:
maximum outflow (m^3/s)
peak lake level (masl)
duration of overtopping (day)

above 212.50 would, in reality, be less. The values reported in Table 5 are based on the following operating rules:

- initial settings of the spillway gates are such that outflow equals the baseflow.
- the spillway gates are not fully-opened until the reservoir level reaches elevation 209.00 masl.

Tables 6 to 9 give the tabulated results of four selected reservoir routings while Figures 10 to 13 illustrate these results. Both the tables and figures show the attenuation effect of the reservoir on the inflow flood hydrograph (assuming the PMP distribution A) for various reservoir levels. Also shown in each of the figures is the trace of the lake level as the flood wave passes through the reservoir. The tables and plots are based on an initial SMIT value of 120. Note that in Figure 10 to 13 the sharp rise in the outflow hydrograph is due to the sudden opening of the spillway gates as defined in the operating rules above.

As mentioned, the reported routings were made using the same temporal throughout the entire watershed. Additional runs were performed assuming a storm system moving through the basin from the headwaters to the outlet. The results were that the lake levels increased by less than 0.08 m.

Table 6. Results of the PMF routing: Initial lake level = 208.0.

OUTFLOW HYDROGRAPH FROM MANANTALI DAM

TIME (DAYS)	INFLOW (M3/S)	OUTFLOW (M3/S)	ELEV (MASL)	STORAGE (M3)
1	1553.1	1621.4	208.00	0.112E+11
2	1784.9	1622.3	208.01	0.112E+11
3	2547.0	1632.6	208.09	0.112E+11
4	3738.6	1660.8	208.33	0.113E+11
5	6938.2	1729.5	208.94	0.116E+11
6	9568.4	1851.2	210.02	0.122E+11
7	11687.8	7440.9	211.05	0.127E+11
8	12867.7	8073.1	211.83	0.131E+11
9	11881.0	8640.2	212.51	0.135E+11
10	10468.1	8973.1	212.92	0.137E+11
11	8963.1	9069.6	213.04	0.137E+11
12	7660.7	8970.0	212.92	0.137E+11
13	6573.8	8725.3	212.62	0.135E+11
14	5676.6	8381.5	212.20	0.133E+11
15	4937.8	7978.2	211.71	0.131E+11
16	4330.0	7543.8	211.18	0.128E+11
17	3829.9	7108.4	210.63	0.125E+11
18	3418.5	6681.9	210.07	0.122E+11
19	3079.9	6301.1	209.56	0.120E+11
20	2801.2	5949.2	209.06	0.117E+11
21	2571.8	5660.2	208.64	0.115E+11
22	2383.0	5414.9	208.26	0.113E+11
23	2227.5	5189.6	207.90	0.111E+11
24	2099.5	4972.4	207.55	0.109E+11
25	1994.1	4757.6	207.18	0.108E+11
26	1907.2	4596.3	206.90	0.106E+11
27	1835.7	4472.4	206.68	0.105E+11
28	1776.7	4375.4	206.50	0.105E+11
29	1728.2	4297.9	206.36	0.104E+11
30	1688.2	4241.2	206.25	0.104E+11
31	1655.2	4197.5	206.17	0.103E+11
32	1628.0	4161.6	206.10	0.103E+11
33	1584.1	4134.5	206.05	0.103E+11
34	1552.9	4110.7	206.00	0.103E+11
35	1528.0	4091.4	205.97	0.102E+11
36	1504.0	4075.2	205.93	0.102E+11
37	1501.0	4060.3	205.91	0.102E+11

INITIAL WATER SURFACE ELEVATION : 208.00
MAXIMUM WATER SURFACE ELEVATION : 213.04
MAXIMUM DISCHARGE FROM THE DAM : 9069.58

Table 7. Results of the PMF routing: Initial lake level = 206.0.

OUTFLOW HYDROGRAPH FROM MANANTALI DAM

TIME (DAYS)	INFLOW (M3/S)	OUTFLOW (M3/S)	ELEV (MASL)	STORAGE (M3)
1	1553.1	1505.3	206.00	0.103E+11
2	1784.9	1513.4	206.03	0.103E+11
3	2547.0	1544.0	206.12	0.103E+11
4	3738.6	1615.8	206.38	0.104E+11
5	6938.2	1774.3	207.09	0.107E+11
6	9568.4	1956.3	208.22	0.113E+11
7	11687.8	2162.7	209.65	0.120E+11
8	12867.7	7351.4	210.94	0.127E+11
9	11881.0	8006.3	211.75	0.131E+11
10	10468.1	8424.0	212.25	0.133E+11
11	8963.1	8593.7	212.46	0.134E+11
12	7660.7	8555.6	212.41	0.134E+11
13	6573.8	8365.1	212.18	0.133E+11
14	5676.6	8070.9	211.82	0.131E+11
15	4937.8	7711.7	211.38	0.129E+11
16	4330.0	7317.3	210.89	0.126E+11
17	3829.9	6917.7	210.38	0.124E+11
18	3418.5	6528.4	209.86	0.121E+11
19	3079.9	6172.1	209.36	0.119E+11
20	2801.2	5842.6	208.86	0.116E+11
21	2571.8	5549.3	208.36	0.113E+11
22	2383.0	5259.6	207.87	0.111E+11
23	2227.5	4960.0	207.36	0.108E+11
24	2099.5	4683.5	206.84	0.106E+11
25	1994.1	4456.6	206.34	0.104E+11
26	1907.2	4247.4	205.87	0.102E+11
27	1835.7	4058.5	205.44	0.100E+11
28	1776.7	3886.1	205.00	0.983E+10
29	1728.2	3750.4	204.62	0.966E+10
30	1688.2	3631.9	204.25	0.951E+10
31	1655.2	3526.2	203.92	0.936E+10
32	1628.0	3448.0	203.68	0.926E+10
33	1584.1	3391.4	203.45	0.916E+10
34	1552.9	3334.8	203.14	0.903E+10
35	1528.0	3288.5	202.90	0.892E+10
36	1504.0	3260.9	202.72	0.885E+10
37	1501.0	3241.1	202.55	0.877E+10

INITIAL WATER SURFACE ELEVATION : 206.00
MAXIMUM WATER SURFACE ELEVATION : 212.46
MAXIMUM DISCHARGE FROM THE DAM : 8593.73

Table 8. Results of the PMF routing: Initial lake level = 204.0.

OUTFLOW HYDROGRAPH FROM MANANTALI DAM

TIME (DAYS)	INFLOW (M3/S)	OUTFLOW (M3/S)	ELEV (MASL)	STORAGE (M3)
1	1553.1	1314.7	204.00	0.940E+10
2	1784.9	1327.2	204.06	0.942E+10
3	2547.0	1355.3	204.20	0.948E+10
4	3738.6	1413.8	204.51	0.962E+10
5	6938.2	1530.4	205.26	0.994E+10
6	9568.4	1680.0	206.59	0.105E+11
7	11687.8	1840.6	208.21	0.113E+11
8	12867.7	1978.6	209.94	0.122E+11
9	11881.0	7598.0	211.24	0.128E+11
10	10468.1	8067.5	211.82	0.131E+11
11	8963.1	8284.9	212.08	0.132E+11
12	7660.7	8288.4	212.09	0.132E+11
13	6573.8	8134.6	211.90	0.131E+11
14	5676.6	7873.2	211.58	0.130E+11
15	4937.8	7541.2	211.17	0.129E+11
16	4330.0	7175.2	210.72	0.126E+11
17	3829.9	6796.5	210.22	0.123E+11
18	3418.5	6432.5	209.73	0.121E+11
19	3079.9	6090.8	209.24	0.118E+11
20	2801.2	5779.1	208.75	0.115E+11
21	2571.8	5493.1	208.26	0.113E+11
22	2383.0	5204.0	207.77	0.110E+11
23	2227.5	4908.1	207.27	0.108E+11
24	2099.5	4645.6	206.76	0.106E+11
25	1994.1	4423.4	206.27	0.104E+11
26	1907.2	4218.4	205.81	0.102E+11
27	1835.7	4033.7	205.38	0.999E+10
28	1776.7	3867.8	204.95	0.981E+10
29	1728.2	3735.5	204.58	0.965E+10
30	1688.2	3618.3	204.21	0.949E+10
31	1655.2	3515.7	203.89	0.935E+10
32	1628.0	3441.7	203.66	0.925E+10
33	1584.1	3386.1	203.43	0.915E+10
34	1552.9	3328.8	203.11	0.901E+10
35	1528.0	3284.3	202.88	0.891E+10
36	1504.0	3258.5	202.70	0.884E+10
37	1501.0	3238.9	202.52	0.876E+10

INITIAL WATER SURFACE ELEVATION : 204.00
MAXIMUM WATER SURFACE ELEVATION : 212.09
MAXIMUM DISCHARGE FROM THE DAM : 8288.36

Table 9. Results of the PMF routing: Initial lake level = 202.0.

OUTFLOW HYDROGRAPH FROM MANANTALI DAM

TIME (DAYS)	INFLOW (M3/S)	OUTFLOW (M3/S)	ELEV (MASL)	STORAGE (M3)
1	1553.1	1529.5	202.00	0.854E+10
2	1784.9	1532.0	202.08	0.857E+10
3	2547.0	1547.7	202.48	0.875E+10
4	3738.6	1600.9	203.01	0.897E+10
5	6938.2	1711.5	203.74	0.923E+10
6	9568.4	1835.3	205.04	0.984E+10
7	11687.8	1958.8	206.80	0.106E+11
8	12867.7	2083.0	208.63	0.115E+11
9	11881.0	2187.2	210.34	0.124E+11
10	10468.1	7733.3	211.41	0.129E+11
11	8963.1	7995.3	211.73	0.131E+11
12	7660.7	8038.8	211.78	0.131E+11
13	6573.8	7920.4	211.64	0.130E+11
14	5676.6	7688.8	211.36	0.129E+11
15	4937.8	7382.0	210.98	0.127E+11
16	4330.0	7041.1	210.54	0.125E+11
17	3829.9	6680.6	210.07	0.122E+11
18	3418.5	6339.4	209.60	0.120E+11
19	3079.9	6010.2	209.12	0.117E+11
20	2801.2	5714.7	208.64	0.115E+11
21	2571.8	5435.5	208.16	0.112E+11
22	2383.0	5146.8	207.68	0.110E+11
23	2227.5	4854.2	207.17	0.108E+11
24	2099.5	4605.4	206.67	0.105E+11
25	1994.1	4387.6	206.19	0.103E+11
26	1907.2	4186.9	205.74	0.101E+11
27	1835.7	4006.5	205.31	0.996E+10
28	1776.7	3847.7	204.90	0.978E+10
29	1728.2	3719.0	204.53	0.962E+10
30	1688.2	3603.0	204.15	0.946E+10
31	1655.2	3503.6	203.86	0.934E+10
32	1628.0	3434.1	203.63	0.924E+10
33	1584.1	3379.3	203.39	0.914E+10
34	1552.9	3321.0	203.06	0.899E+10
35	1528.0	3278.7	202.84	0.890E+10
36	1504.0	3254.9	202.67	0.883E+10
37	1501.0	3235.7	202.49	0.875E+10

INITIAL WATER SURFACE ELEVATION : 202.00
MAXIMUM WATER SURFACE ELEVATION : 211.78
MAXIMUM DISCHARGE FROM THE DAM : 8033.34

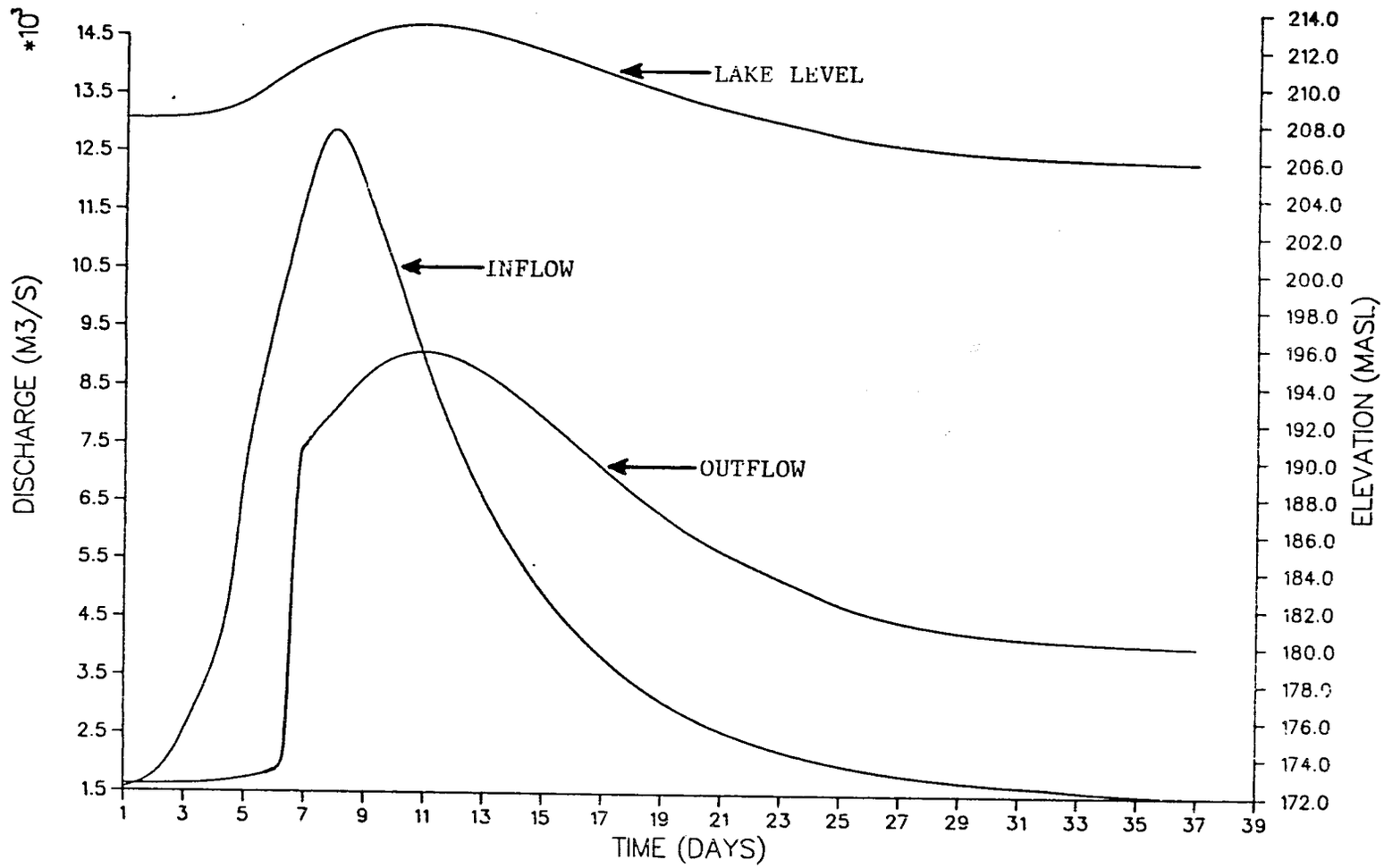


Figure 10. Plot of the PMF routing: Initial lake level = 208.0.

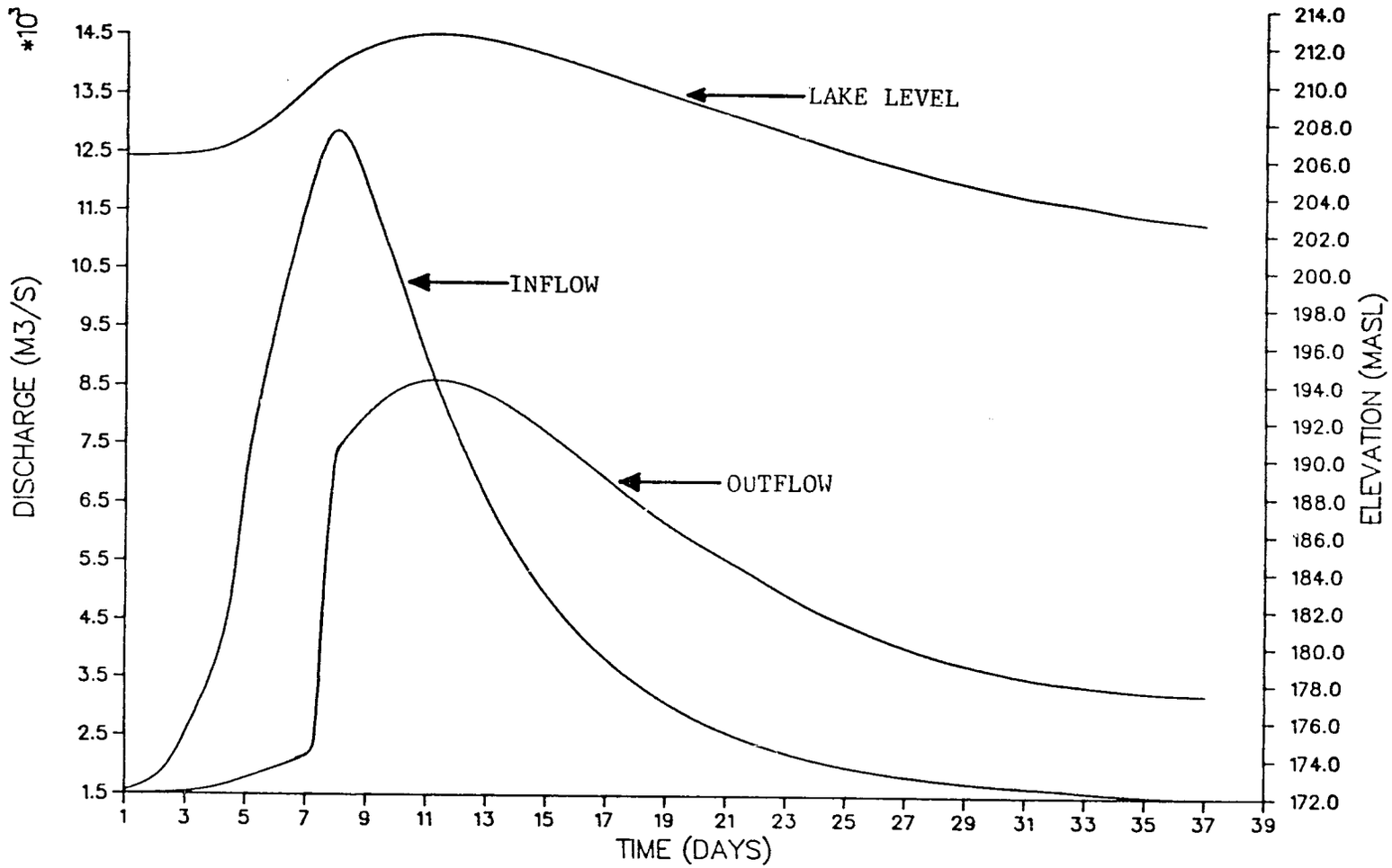


Figure 11. Plot of the PMF routing: Initial lake level = 206.0.

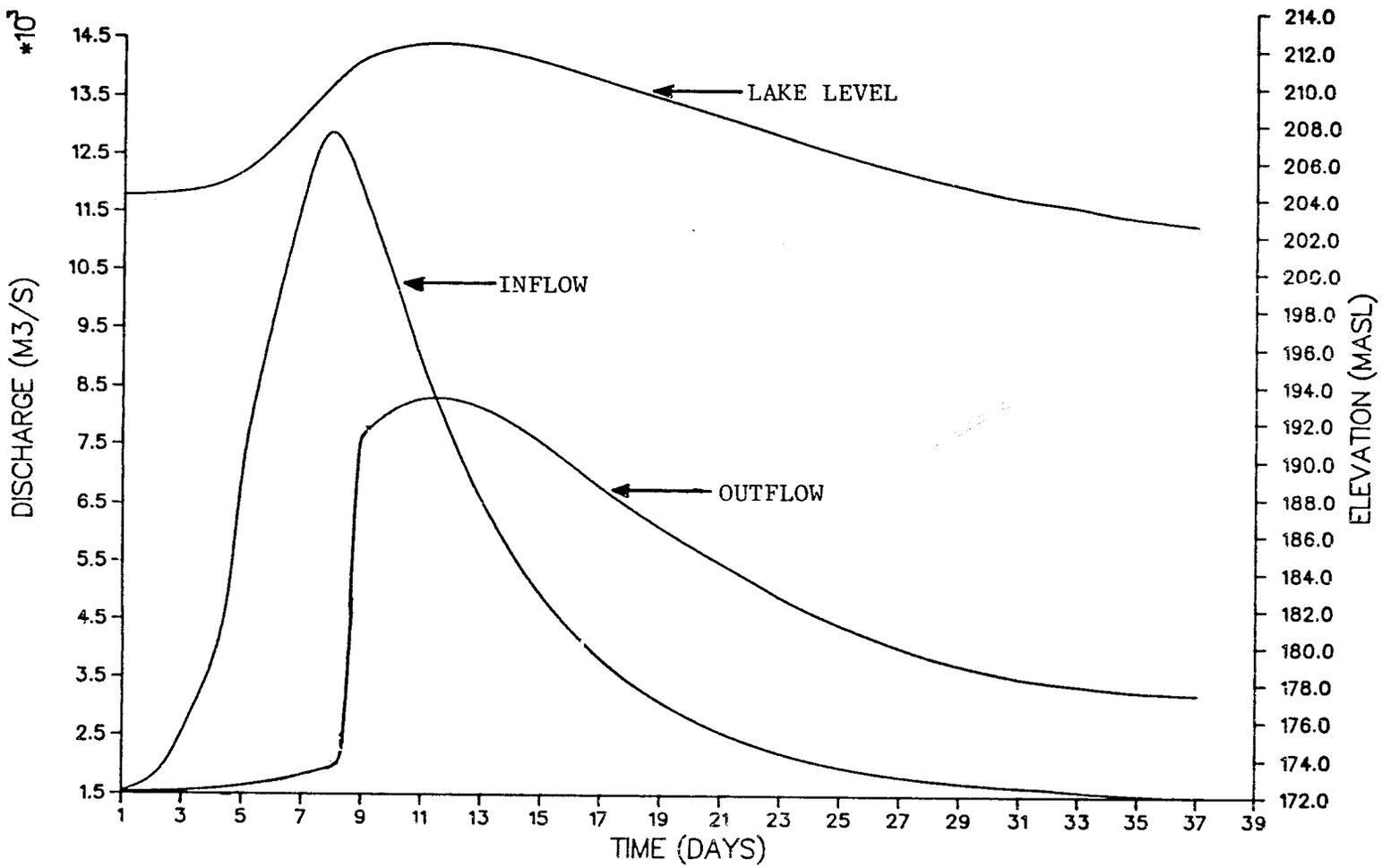


Figure 12. Plot of the PMF routing: Initial lake level = 204.0.

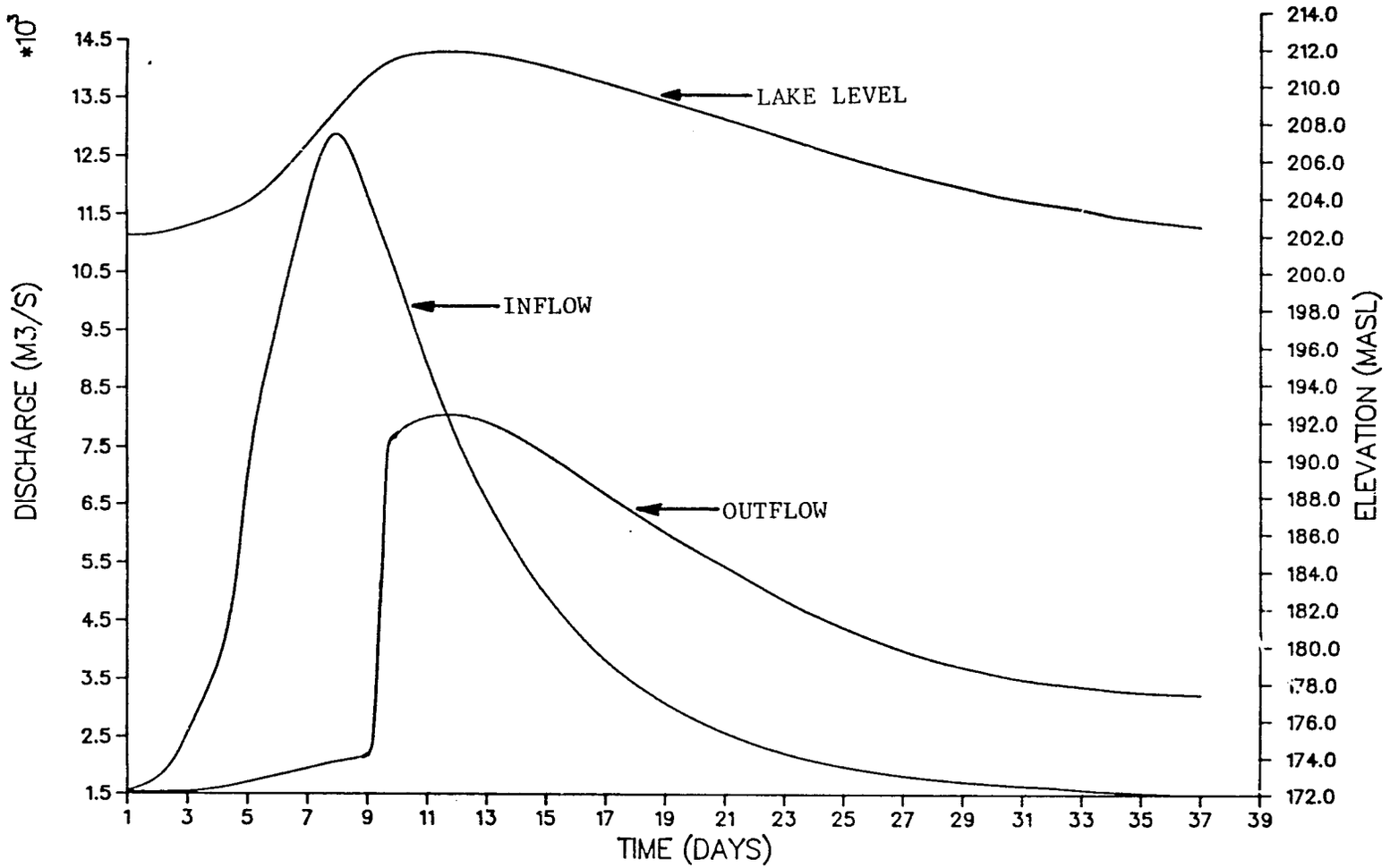


Figure 13. Plot of the PMF routing: Initial lake level = 202.0.

CHAPTER VII

SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

Summary

Development of the water resources in the Senegal River basin, located in West Africa, initially includes the construction of two dams on the river system. The Manantali dam on the Bafing River will be the largest of the two. Designed as a multi-purpose structure, the dam will provide benefits to agriculture, navigation, hydropower, and domestic and industrial water users, as well as providing flood protection. Because the Manantali dam will be the only regulating structure on a river which is subject to highly variable flows, it must be able to operate under extreme conditions.

To provide an independent check on the design and to examine operating procedures during large flood events, this study was designed to develop and route the probable maximum flood (PMF) through the Manantali reservoir. The PMF was developed on the design concept of the probable maximum precipitation (PMP). Estimates of the PMP values, and the storm duration and temporal distribution were determined from a statistical and historical approach. A hydrologic computer model was formulated and calibrated to simulate the rainfall-runoff and routing processes in the Bafing River basin. The data used throughout the course of the project were obtained mainly from previous reports about the Senegal River and its tributaries.

Application of the model to the study area resulted in an estimate of the probable maximum flood that would be expected to occur at the dam site. Various computer runs were performed assuming different initial state conditions. The routings were analyzed to determine the capacity of the reservoir to safely pass and attentuate large floods. In addition to examining the performance of the spillway system, the runs were reviewed with respect to the operation of the dam during floods events.

Conclusions

The hydrologic computer model developed during this study was calibrated to four flood hydrographs recorded at the Dibia stream gage. Results of the calibration runs indicate the model can adequately simulate the rainfall-runoff process in the Bafing River basin. Therefore, it was concluded that the computer model, MAIN, is applicable to the study area to predict and route flood hydrographs.

During the development of the probable maximum precipitation the gamma distribution was shown to fit the recorded precipitation. The historical records also showed flood producing rainfall events to last from 4 to 10 days and so using a duration of 7-days for the PMP event was considered acceptable and a realistic time period. Comparison of this PMP estimate (see Table 4) to an estimate by Riedel (1983) for the Narmada River in India concluded that the value was in a reasonable range.

Applying the model and PMP estimate to the watershed resulted in an estimate of the probable maximum flood. The peak discharge calculated for four different initial conditions ranged from 97 to 108

percent higher than the design peak discharge estimated by Groupement Manantali. Results of 32 flood routings indicate that the lake elevation rises above the crest of the dam, elevation 212.50 masl, for 14 of these floods.

Although the results suggest that the potential for overtopping the dam during the PMF is rather high, the operation of the spillway gates has a significant impact on controlling these high lake levels. Computer runs were made assuming a high initial soil moisture condition, a lake level of 208.00, and the spillway gates fully opened. Under these conditions, the maximum pool elevation reached was 211.93 masl.

In conclusion, the study revealed that although the design flood based on the probable maximum precipitation yielded larger peak discharges than the statistical approach used in previous reports, the results of routing the design floods of both methods are comparable. Overtopping of the dam by the PMF developed in this study can be prevented if the current spillway system is carefully and effectively operated during extreme runoff events.

Recommendations

It is strongly recommended that a more complete and detailed study be performed to establish operating rules and procedures for control of the lake level during the annual flood season. The computer model developed during this project could be used to help establish such regulations. It could also be used to set standard practices of operation during the period of "artifical flood release". In addition, it is suggested that the model could be used as a tool for

management decisions in regards to risk analysis of using allocated flood storage for hydropower production. During periods of low inflow, the drawdown of the reservoir can be simulated based on the water requirements downstream of the dam.

A warning system should be implemented in the basin to alert the dam operators of rising streamflow conditions. Use of the staff gage at Dakka-Saidou, as one example, would provide valuable information regarding the onset of a flood. A water level recorder at the upstream end of the reservoir could also serve as part of a warning system. With the advanced warning, the dam could be operated in such a fashion as to prevent overtopping and unnecessary variations in the discharge from the reservoir. The information from such a system must be transmitted via some type of communication system that is dependable and requires minimal manual operation.

Monitoring the river stage is not the only warning system that could be implemented. By having a reliable and timely estimate of the precipitation occurring throughout the entire watershed, simulation of the expected runoff can be made more accurately. These forecasts would then be used for the day-to-day operation of the day. The precipitation values can be sent by either satellite transmissions, telephone lines, or radio service. The use of radar to estimate the intensity and extent of rainfall is also becoming widely used. Selection of the communciation system is, of course, dependent on the individual conditions and resources available for establishing the necessary control centers.

Better calibration of the model could be achieved by obtaining the streamflow records at Soukoutali and Dakka-Saidou and precipita-

tion data in the upper portion of the Bafing River. Additional stream gages, located in the upper portion of the basin (near the confluence of subbasins 1 and 2, Figure 2), as well as more precipitation stations within the basin would also help in achieving a better calibration and in understanding the hydrologic charactersitics of the area. As needed, refinements in the model, such as variable evapotranspiration rates, could be made to better simulate and forecast the expected stream flow.

One final suggestion for future research and studies is to conduct a study that would check the correlation of precipitation patterns at the start of the rainy season to the patterns of the whole season. Riehl (1979) references such a study by Winstanley (1974) for West Africa between 10^0 and 20^0 north latitudes, from the Atlantic to Lake Chad. If a high correlation exists, some general decisions on the reservoir operation can be made early in the rainy season.

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APPENDICES

Appendix A

Hydrologic Computer Model

1. MODEL LISTING
2. INPUT - OUTPUT

MODEL LISTING

```
C *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *
C      "MAIN"-- A HYDROLOGIC RAINFALL-RUNOFF-ROUTING MODEL
C      THIS PROGRAM WILL CALCULATE THE RUNOFF FROM A LARGE
C        WATERSHED AND ROUTE THE FLOOD HYDROGRAPH THROUGH A RESER-
C        VOIR.  THE PROGRAM CONSISTS OF THREE MAIN SUBROUTINES.
C        (1) RUNOFF
C        (2) ROUTE
C        (3) RESVOR
C      OTHER SUBROUTINES THAT SUPPLEMENT "MAIN" ARE:
C            ERROR
C            DTCURV
C            INTERP
C            SUM
C *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *

      INTEGER SUBBAS,NEXE,ICALL,ROUTIN
      REAL CK,REACHL,KMUSK,RECQ,ABSERR,RELERR,AVERR,RSQ
      DIMENSION QOUT(100),FLOW(100)
      COMMON /BLOCK1/SUBBAS,CK,QOUT,FLOW
      COMMON /BLOCK2/REACHL,RINCR,KMUSK,XMUSK,RTIME
      COMMON /ALPHA/UNITS,TUNITS

C  OPEN FILE TO STORE THE RESULTS OF 'MAIN'
      OPEN(UNIT=2,NAME='FLOOD.DAT',TYPE='NEW')
      OPEN(UNIT=3,NAME='HYDRO.DAT',TYPE='NEW')
      WRITE(5,*)' WHAT TYPE OF UNITS ARE BEING USED?  ENGLISH OR METRIC'


      WRITE(5,*)' IF ENGLISH TYPE E.  IF METRIC TYPE M.'
      READ(5,10)UNITS
 10   FORMAT(A1)
      WRITE(5,*)' WHAT TIME UNITS ARE BEING USED?  HOURS OR DAYS'
      WRITE(5,*)' IF HOURS TYPE H.  IF DAYS TYPE D.'
      READ(5,10)TUNITS
      DO 50 I=1,100
 50    FLOW(I)=0.0
      ICALL=0

C  OPEN FILE CONTAINING THE SEQUENCE C  INSTRUCTIONS TO BE EXECUTED BY
C     'MAIN' PROGRAM.  USING A FILE OF THIS SORT WILL ALLOW THE USER MORE
C     VERSITILTY IN OPTIMIZING AND CALIBRATING.
      OPEN(UNIT=1,NAME='SEQEXE.DAT',TYPE='OLD')

      READ(1,*)NEXE
      DO 100 I=1,NEXE
      READ(1,30)ROUTIN,SUBBAS
 30   FORMAT(I2,1X,I2)
      IF(ROUTIN.EQ.11)CALL RUNOFF
      IF(ROUTIN.NE.22)GO TO 1900
        ICALL=ICALL+1
        CALL ROUTE(ICALL)
 1900 CONTINUE
      IF(ROUTIN.EQ.33)CALL SUM
      IF(ROUTIN.EQ.44)CALL RESVOR
      IF(ROUTIN.EQ.55)CALL ERROR(FLOW,UNITS,TUNITS)
 100  CONTINUE
      CLOSE(UNIT=1,DISPOSE='KEEP')
      CLOSE(UNIT=2,DISPOSE='KEEP')
      CLOSE(UNIT=3,DISPOSE='KEEP')
      WRITE(5,90)
 90   FORMAT(' THE RESULTS OF THIS PROGRAM ARE STORED IN FILE=FLOOD.DAT.

     1 '/,' THE RESULTS INCLUDE:',T23,'SUMMED HYDROGRAPHS'/,
     2 T23,'RESULTS FROM CHANNEL ROUTING'/,
     3 T23,'RESULTS FROM RESERVOIR ROUTING'/,
     4 ' THE RUNOFF HYDROGRAPHS FOR EACH SUBBASIN ARE STORED IN
     5  FILE=HYDRO.DAT.')
      STOP
      END
```

```
C   THIS SUBROUTINE CALCULATES THE AMOUNT OF EXCESS PRECIPITATION
C       FROM AN INDIVIDUAL SUBBASIN AND TRANSFORMS THAT VALUE INTO
C       AN OUTFLOW HYDROGRAPH AT THE OUTLET.
C *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *
      SUBROUTINE RUNOFF
      COMMON /BLOCK1/SUBBAS,CK,QOUT,FLOW
      COMMON /BLOCK2/REACHL,RINCR,KMUSK,XMUSK,RTIME
      COMMON /ALPHA/UNITS,TUNITS
      DIMENSION QOUT(100),FLOW(100)
      REAL*8 FNAME
      INTEGER NPAIRS,SUBBAS,T,BGRAIN,EDRAIN,NQINS,PT,NPOINT
      REAL ACAREA(10),AREA,DA,DELTIM,DFAREA,DIFPER,DROV,EXRAIN,
     1 HYEDIM(50),CK,NRAINT,PERTC(10),PERT,PRECIP,QIN(100),
     2 ROCOEF,TC,TIME,TRAINT,CQIN,CQOUT,REACHL,INAREA,
     3 SAREA(20),QRATE,KEY,RECQ,ABSERR,RELERR,AVERR,SUM,BASEQ,
     4 BFLOW,DBQDT,TOTALQ,SMI(10),ROP(10,5),ROPMAX,ROPMIN,FIELDC,WILTPT,
     5 SMIT,DFROP,DIFSMI,ET,INFILT,RSQ,KMUSK,PSAREA,RI(5)
      WRITE(5,*)' PERFORMING "RUNOFF" ON SUBBASIN ',SUBBAS

C ---------------------------------------------------------------
C  THE FOLLOWING OPENS THE EXISTING DATA FILES ON EACH SUBBASIN
      OPEN(UNIT=7,NAME='NAME.DAT',TYPE='NEW',CARRIAGECONTROL='LIST')
      WRITE(7,998)SUBBAS
 998  FORMAT('SB ',I2,'.DAT')
      REWIND 7
      READ(7,999)FNAME
 999  FORMAT(A8)
      WRITE(5,001)FNAME
 001  FORMAT(' ',A8)
      OPEN(UNIT=4,NAME=FNAME,TYPE='OLD')
      READ(4,*)DA,TC,DELTIM,NPAIRS,CQIN
      READ(4,*)REACHL,RINCR,KMUSK,XMUSK,RTIME
      WRITE(5,*)' CQIN = ',CQIN,' DOES IT NEED TO CHANGE'
      READ(5,15)ANSWER
 15   FORMAT(A1)
      IF(ANSWER.EQ.'N')GO TO 9999
      WRITE(5,*)' GIVE VALUE OF CQIN'
      READ(5,*)CQIN
9999  CONTINUE
C  THE ABOVE DATA READ IN MUST BE EXPRESSED IN THE FOLLOWING UNITS
C       DA    -DRAINAGE AREA OF THE SUBBASIN,SQUARED KM
C       TC    -TIME OF CONCENTRATION,DAYS
C       DELTIM-TIME INCREMENT FOR RUNOFF HYDROGRAPH,DAYS
C       NPAIRS-NUMBER OF POINTS THAT DEFINE THE TIME-AREA CURVE
C       CQIN  -ROUTING COEFFICIENT USED TO DEFINE THE INSTANTANEOUS
C               HYETOGRAPH
C       REACHL-REACH LENGTH TO THE NEXT DOWNSTREAM SUBBASIN OUTLET
C       RINCR -INCREMENTAL LENGTH USED IN CHANNEL ROUTING.
C       KMUSK -MUSKINGUM COEFFICIENT IN UNITS OF TIME.
C       XMUSK -MUSKINGUM X COEFFICIENT.
C       RTIME -TIME INTERVAL FOR ROUTING.
C
C ---------------------------------------------------------------
C  NEXT READ IN THE TIME-AREA RELATION
C       PERTC(I) -PERCENTAGE OF THE TC FOR THE SUBBASIN.  TC FOR
C                 SUBAREA DIVIDED BY THE SUBBASIN TC.
C                 RANGE: 0.0 - 1.0
C       ACAREA(I)-ACCUMULATED SUBAREAS WHOSE TIME OF CONCENTRATION
C                 IS EQUAL OR LESS THAN PERTC(I)*TC
C                 RANGE: 0.0 - DA
      DO 100 I=1,NPAIRS
 100  READ(4,*)PERTC(I),ACAREA(I)
C
C ---------------------------------------------------------------
```

```
C   READ IN STORM EVENT.  THIS MAY BE KNOWN DATA OR A DIMENSIONLESS
C       HYETOGRAPH.
C       HYEDIM(T)-HYETOGRAPH ORDINATE FROM TIME ZEREO
C       PRECIP-TOTAL AMOUNT OF PRECIPITATION DURING THE STORM.  IF
C              ACTUAL EVENT IS GIVEN,THEN PRECIP=1.0.
C       T      -TIME IN DAYS SINCE TZEREO,WHERE TZEREO MARKS THE
C              BEGINNING OF ANY RAIN IN THE BAFING RIVER BASIN.
C       BGRAIN-TIME AT WHICH THE EVENT BEGINS
C       EDRAIN-TIME AT WHICH THE PRECIP STOPS IN THE SUBBASIN
      READ(4,*)PRECIP,BGRAIN,EDRAIN
      KEY=0.0
      DO 125 J=1,50
      IF (KEY.EQ.1.0)GO TO 1000
        READ(4,*)T,HYEDIM(J)
        IF(T.EQ.EDRAIN)KEY=1.0
 125  CONTINUE
 1000 CONTINUE
C
C ------------------------------------------------------------
C  READ IN DATA CONCERNING THE SOIL MOISTURE-RUNOFF RELATIONSHIP.  AN
C    INITIAL ESTIMATE OF THE SOIL MOISTURE INDEX MUST BE GIVEN.
C       SMI(I)-SOIL MOISTURE INDEX (MM/IN)
C       ROP(I)-PERCENT RUNOFF THAT CORRESPONDS TO SMI(I)
C       FIELDC-DEPTH OF WATER HELD IN A COLUMN OF SOIL AFTER FREE
C              DRAINAGE. (MM/IN)
C       WILTPT-DEPTH OF WATER HELD IN A COLUMN OF SOIL WHEN SOIL WATER
C              TENSION EQUALS THE SUCTION FORCE OF THE PLANTS. (MM/IN)
C       ET    -AVERAGE DAILY EVAPOTRANSPIRATION RATE (MM/DAY,IN/DAY)
C       SMIT  -SOIL MOISTURE INDEX AT TIME =T
C       ROPMAX-THE MAXIMUM RUNOFF COEFFICIENT THAT APPLIES WHEN
C              SMIT=FIELDC
C       ROPMIN-THE MINIMUM RUNOFF COEFFICIENT THAT APPLIES WHEN
C              SMIT=FIELDC
C       NPOINT-NUMBER OF POINTS THAT DEFINE THE SOIL MOISTURE-RUNOFF CUR
      READ(4,*)FIELDC,WILTPT,ET,SMIT,ROPMAX,ROPMIN,NPOINT,IOPT
      IF(IOPT.EQ.1)GO TO 149
      READ(4,*)(RI(L),L=1,IOPT-1)
 149  DO 150 I=1,NPOINT
 150  READ(4,*)SMI(I),(ROP(I,J),J=1,IOPT)

 155  WRITE(3,10)SUBBAS
 10   FORMAT('1',T21,'RUNOFF HYDROGRAPH FROM SUBBASIN #',I2,/)
      WRITE(3,71)
 71   FORMAT('0',T12,'TIME',T23,'COMPUTED Q',T38,'TOTAL Q')
D     WRITE(3,72)
D72   FORMAT('+',T53,'RECORDED Q',T68,'ABS ERROR',T83,'REL ERROR')
        IF(UNITS.EQ.'M')GO TO 4500
        IF(TUNITS.EQ.'H')GO TO 4100
        CK=26.78
        WRITE(3,80)
 80     FORMAT(' ',T11,'(DAYS)')
        GO TO 4300

 4100   CK=645.0
        WRITE(3,81)
 81     FORMAT(' ',T11,'(HOURS)')
 4300   WRITE(3,82)
 82     FORMAT('+',T26,'(CFS)',T39,'(CFS)')
D       WRITE(3,73)
D73     FORMAT('+',T56,'(CFS)',T70,'(CFS)',T87,'(%)')
        GO TO 5100

 4500   IF(TUNITS.EQ.'H')GO TO 4700
        CK=0.012
        WRITE(3,80)
        GO TO 5000
```

```
 4700   CK=0.278
        WRITE(3,81)
 5000   WRITE(3,83)
 83     FORMAT('+',T25,'(M3/S)',T39,'(M3/S)')
D       WRITE(3,74)
D74     FORMAT('+ ',T55,'(M3/S)',T70,'(M3/S)',T87,'(%)')


 20   FORMAT(T12,I3,T22,F9.1,T35,F9.1)
D21   FORMAT('+',T52,F9.1,T67,F8.1,T82,F7.2)
 5100 CONTINUE
C-----------------------------------------------------------------------
C  BEGIN TIME LOOP FOR RAINFALL-RUNOFF.  RUNOFF STEPS CARRIED OUT
C       ONLY DURING THE TIME OF ACTUAL PRECIPITATION.
C  ESTABLISH A TIME-AREA DIAGRAM USING THE EXISTING TIME-AREA
C       RELATION.
      I=2
      DURAT=DELTIM
      PERT=DURAT/TC
        IF(PERT.GT.1.0)PERT=1.0
      J=1
      PSAREA=0.0
      INAREA=0.0
      SAREA(1)=0.0
      DO 175 K=2,50
        IF(PERT.GT.1.0)GO TO 1500
        SAREA(J)=0.0
        DO 180 KK=1,20
          IF(SAREA(J).GT.0.0)GO TO 2000
          IF((PERTC(I-1).LE.PERT).AND.(PERTC(I).GE.PERT))GO TO 1600
           GO TO 1700
 1600      DFAREA=ACAREA(I)-ACAREA(I-1)
           DIFPER=PERTC(I)-PERTC(I-1)
           AREA=ACAREA(I-1)+(PERT-PERTC(I-1))/DIFPER*DFAREA
           INAREA=INAREA+PSAREA
           SAREA(J)=AREA-INAREA
           PSAREA=SAREA(J)
          GO TO 180
 1700      I=I+1
 180    CONTINUE
 2000   J=J+1
        DURAT=DURAT+DELTIM
        PERT=DURAT/TC

C THE FOLLOWING INSTRUCTIONS ARE USED WHEN TC/DELTIM DOES NOT
C   EQUAL AN INTEGER.  THIS WILL NORMALLY BE THE CASE BECAUSE
C   OF THE LACK OF DETAILED RAINFALL DATA IN TIME INCREMENTS
C   LESS THAN 1 DAY.
        TCHECK=(DURAT-DELTIM)/TC
        IF((PERT.GT.1.0).AND.(TCHECK.LT.1.0))GO TO 2100
        GO TO 175
 2100     SAREA(J)=DA-AREA
          J=J+1
 175  CONTINUE
 1500 NQINS=J-1
C NOTE: NQINS IS THE NUMBER OF ORDINATES IN THE TIME-AREA DIAGRAM

C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *
C CALCULATE THE INSTANTANEOUS HYDROGRAPH AT THE SUBBASIN OUTLET.
C   CONVERT THIS INTO A TRUE OUTFLOW HYDROGRAPH USING
C   CLARK'S PROCEDURE.

      CQOUT=1.0-CQIN
      MM=NQINS-BGRAIN+EDRAIN
C  NOTE: MM EQUALS THE TOTAL NUMBER OF INSTANTEANEOUS INFLOWS
C        DURING THE STORM DURATION
```

```
      DO 200 M=1,MM
 200    QIN(M)=0.0
      DO 220 IV=1,51
 220    QOUT(IV)=0.0
C  THE PREVIOUS SET OF LINES WAS INITIALIZATION OF VARIABLES

C -------------------------------------------
C -------------------------------------------
C BEGIN LOOP THAT CALCULATES THE SURFACE RUNOFF
C    'M' IS A COUNTER OF THE NUMBER OF RAIN INCREMENTS FOR THE STORM.
C    'MMAX' IS THE TOTAL NUMBER OF RAIN INCREMENTS
      MMAX=EDRAIN-BGRAIN+1
      DO 250 M=1,MMAX
      TRAINT=PRECIP*HYEDIM(M)
C     -----------------------------------------------------------
C     THIS SECTION COMPUTES THE RUNOFF COEFFICIENT FOR THE TIME INTERVAL
C     BEGINNING AT TIME=T.
        IF(SMIT.LT.FIELDC)GO TO 2200
          ROCOEF=ROPMAX
          SMIT=FIELDC
          KEY=1.0
          GO TO 2400
 2200     IF(SMIT.GT.WILTPT)GO TO 2300
          ROCOEF=ROPMIN
          SMIT=WILTPT
          KEY=1.0
          GO TO 2400
 2300     KEY=0.0

C  THE FOLLOWING 5 LINES ESTABLISH WHICH SMI-ROP CURVE TO BE USED
        JK=1
        IF(IOPT.EQ.1)GO TO 2350
        DO 274 IRI=1,IOPT-1
          IF(TRAINT.GE.RI(IRI))JK=IRI+1
 274    CONTINUE
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC

 2350   I=2
        DO 275 L=1,20
          IF((SMI(I-1).LE.SMIT).AND.(SMI(I).GE.SMIT))GO TO 2500
            GO TO 2600
 2500       PERSMI=(SMIT-SMI(I-1))/(SMI(I)-SMI(I-1))
            DFROP=ROP(I,JK)-ROP(I-1,JK)
            ROCOEF=ROP(I-1,JK)+PERSMI*DFROP
 2575       KEY=1.0
          GO TO 2400
 2600       I=I+1
 275    CONTINUE
C-------------------------------------------------------------

 2400   EXRAIN=TRAINT*ROCOEF
        INFILT=TRAINT-EXRAIN
        WRITE(3,*)SMIT,ROCOEF
        SMIT=SMIT+INFILT-ET
        IF(EXRAIN.LT.0.0)GO TO 250
C       --------------------------------------
C       CALCULATE THE INSTANTANEOUS RUNOFF FROM EACH INCREMENTAL AREA
C       OF THE SUBBASIN.
        N=1
        DO 300 J=M,(M+NQINS-1)
          DROV=EXRAIN*SAREA(N)
          QRATE=DROV/DELTIM*CK
          QIN(J)=QRATE+QIN(J)
 2      WRITE(3,*)DROV,QRATE,QIN(J)
 300      N=N+1
 250    CONTINUE
```

```
C END LOOP OF RAINFALL DURATION

C ---------------------------------------------
C ---------------------------------------------
C  ROUTE QIN BASED ON CLARK'S PROCEDURE TO PROVIDE FOR STORAGE
C        EFFECTS.
      T=BGRAIN
      IV=BGRAIN
      PQOUT=0.0
        READ(4,*)BASEQ,DBQDT

      DO 350 J=1,MM
C  NOTE: DIFFERENT COUNTERS ARE USED TO TAKE INTO ACCOUNT
C         THE SPACIAL VARIABILITY OF THE PRECIPITATION,IF ANY.
        QOUT(IV)=(QIN(J)*CQIN)+(PQOUT*CQOUT)
        PQOUT=QOUT(IV)
        BFLOW=BASEQ+(DBQDT*FLOAT(J))
        TOTALQ=QOUT(IV)+BFLOW
D       READ(4,*)RECQ
D         CALL ERROR(TOTALQ,J,RECQ,ABSERR,RELERR,AVERR,RSQ,COE,DIFSQ)
        WRITE(3,20)T,QOUT(IV),TOTALQ
D       WRITE(3,21)RECQ,ABSERR,RELERR
        T=T+DELTIM
        IV=IV+1
 350  CONTINUE
C
C  EXTEND OUTFLOW HYDROGRAPH BY 25 INCREMENTS PAST THE EXTENT OF
C        EVENT DURATION
      MM=MM+1
      DO 400 J=1,25
        QOUT(IV)=QOUT(IV-1)*CQOUT
        BFLOW=BASEQ+(DBQDT*FLOAT(MM))
        TOTALQ=QOUT(IV)+BFLOW
D       READ(4,*)RECQ
D         CALL ERROR(TOTALQ,MM,RECQ,ABSERR,RELERR,AVERR,RSQ,COE,DIFSQ)
D       IF(RECQ.LT.0.0)GO TO 500
        WRITE(3,20)T,QOUT(IV),TOTALQ
D       WRITE(3,21)RECQ,ABSERR,RELERR
        T=T+DELTIM
        IV=IV+1
 400    MM=MM+1
D500      WRITE(3,30)AVERR,RSQ
D30   FORMAT('0THE AVERAGE RELATIVE ERROR,AVERR=',F7.2,'%',/
D    1 ' THE COEFFICIENT OF DETERMINATION,R SQUARED,=',F5.3)
      CLOSE(UNIT=4,DISPOSE='KEEP')
      CLOSE(UNIT=7,DISPOSE='DELETE')
      RETURN
      END
```

```
C *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *
C      THIS PROGRAM PERFORMS CHANNEL FLOOD ROUTING USING THE
C        MUSKINGUM METHOD.
C *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *

      SUBROUTINE ROUTE(ICALL)
      COMMON /BLOCK1/SUBBAS,CK,O2,FLOW
      COMMON /BLOCK2/REACHL,RINCR,KMUSK,XMUSK,RTIME
      COMMON /ALPHA/UNITS,TUNITS
      INTEGER NROUTE,TIME,SUBBAS
      REAL C0M,C1M,C2M,ADJK,O1,DENOM,TCHECK,DIF,O2,KMUSK
      DIMENSION O2(100),FLOW(100)
      WRITE(5,*)' PERFORMING "ROUTE" ON REACH ',ICALL

D     WRITE(5,*)SUBBAS,CK
L     DO 900 I=1,25
D900  WRITE(5,*)FLOW(I)

      NROUTE=REACHL/RINCR
D       WRITE(5,*)NROUTE
      ADJK=KMUSK/FLOAT(NROUTE)
D       WRITE(5,*)' KMUSK,ADJK AND RTIME ARE:',KMUSK,ADJK,RTIME
      TCHECK=2.*ADJK*XMUSK
      IF((RTIME.LT.TCHECK).OR.(RTIME.GT.ADJK))GO TO 1000
      GO TO 1100
 1000   WRITE(5,10)RTIME,TUNITS,TCHECK,ADJK
 10     FORMAT(' ROUTING TIME=',F4.2,A1,' SHOULD BE WITHIN',
     1   F4.2,'( RTIME )',F4.2)
 1100 CONTINUE

C ---------------------------------------------------------------------
C  CALCULATE THE COEFFICIENTS USED IN THE MUSKINGUM METHOD
      DENOM=ADJK*(1.-XMUSK)+(RTIME*0.5)
      C0M=(-ADJK*XMUSK +(RTIME*0.5))/DENOM
      C1M=(ADJK*XMUSK + (RTIME*0.5))/DENOM
      C2M=(ADJK*(1-XMUSK) - (RTIME*0.5))/DENOM
D       WRITE(5,*)C0M,C1M,C2M
      DIF=C0M+C1M+C2M-1.0
      IF(ABS(DIF).LE.0.01)GO TO 1200
        WRITE(5,20)C0M,C1M,C2M
 20     FORMAT(' THE MUSKINGUM COEFFICIENTS DO NOT SUM TO 1.0.
     1   THEY ARE:'/T41,'C0M=',F5.3/T41,'C1M=',F5.3/T41,'C2M=',F5.3)
        WRITE(5,*)' TYPE THE ADJUSTED VALUE OF "C0M" THAT IS REQUIRED.'
        READ(5,*)C0M
        WRITE(5,*)(' THE NEW VALUE OF C0M=',C0M)
 1200 CONTINUE
C ---------------------------------------------------------------------

C  INITIALIZE VARIABLES AND BEGIN ACTUAL ROUTING PROCEDURE.
      ICOUNT=1
      O2(1)=FLOW(1)*C0M
      O1=O2(1)
      DO 150 MM=1,10
        IF(ICOUNT.GT.NROUTE)GO TO 1500
        DO 100 I=2,100
        O2(I)=(C0M*FLOW(I)) + (C1M*FLOW(I-1)) + (C2M*O1)
 100    O1=O2(I)
        DO 200 J=1,100
 200    FLOW(J)=O2(J)
 150  ICOUNT=ICOUNT+1
 1500 CONTINUE
      WRITE(2,40)ICALL
 40   FORMAT('0',T21,'THE ROUTED HYDROGRAPH AT THE END OF REACH #',
     1 I2,//)
        L=1
        LL=6
        DO 500 I=1,8
        WRITE(2,50)(TIME,TIME=L,L+LL)
 50     FORMAT( T10,'TIME',T21,7(5X,I2,4X))
        WRITE(2,60)(FLOW(IV),IV=L,L+LL)
 60     FORMAT( T10,'DISCHARGE',T21,7(F9.1,2X),/)
        L=L+7
        IF(L.EQ.50)LL=1
 500    CONTINUE
      RETURN
      END
```

```
C  THIS SUBROUTINE SUMS TWO HYDROGRAPHS.  THE COMBINED HYDROGRAPH IS
C     STORED IN THE ARRAY VARIABLE 'FLOW'.
      SUBROUTINE SUM
      INTEGER SUBBAS,TIME
      COMMON /BLOCK1/SUBBAS,CK,QOUT,FLOW
      COMMON /ALPHA/UNITS,TUNITS
      DIMENSION QOUT(100),FLOW(100)
      WRITE(5,*)' PERFORMING "SUM" ON SUBBASIN ',SUBBAS

D       WRITE(5,*)TUNITS
      WRITE(2,40)SUBBAS
 40   FORMAT('0 ', T21,'THE COMBINED HYDROGRAPH AT THE OUTLET OF SUBBASIN

     1 #',I2,//)
      DO 100 IV=1,100
 100  FLOW(IV)=FLOW(IV)+QOUT(IV)
      J=1
      JJ=6
      DO 130,M=1,8
        WRITE(2,50)(TIME,TIME=J,J+JJ)
 50     FORMAT( T10,'TIME',T21,7(5X,I2,4X))
        WRITE(2,60)(FLOW(IV),IV=J,J+JJ)
 60     FORMAT( T10,'DISCHARGE',T21,7(F9.1,2X),/)
        J=J+7
        IF(J.EQ.50) JJ=1
 130  CONTINUE
      RETURN
      END
```

```
C * * * * * * * * * * * * * * * * * * * * * *
C     THIS SUBROUTINE PERFORMS HYDROLOGIC RESERVOIR ROUTING.
C       THE METHODOLOGY COMBINES THE MUSKINGUM AND MODIFIED
C       PULS METHOD.
C * * * * * * * * * * * * * * * * * * * * * *

      SUBROUTINE RESVOR
      INTEGER SET,NPGATE,GATES,NEGATE,TIME,NPSETS,NESETS,PGATES,EGATES
      REAL*8 LEVEL
      REAL IN,INWSEL,EWSEL,KMUSK
      COMMON/RES1  SET,NPGATE(15),GATES,NEGATE(15),TIME,
     1 NPSETS,NESETS,EGATES,PGATES,PCREST,ECREST,NPPAIR,NEPAIR
      COMMON/RES1/ ELEV(20),STORE(20),BFLOW,
     1 HP(20),HE(20),PQ(20,20),EQ(20,20),LMAX,
     2 O2(100),H(20),TOTALQ(20),S,S1,S2,STOQ(20),CTIME
      COMMON/BLOCK1/NROUTE,WSEL,IN,FLOW
      COMMON/BLOCK2/DELSTO,ELINC,KMUSK,XMUSK,RTIME
      COMMON/ALPHA/UNITS,TUNITS
      DIMENSION IN(100),FLOW(100)
      WRITE(5,*)' PERFORMING "RESVOR" ROUTING'

C  READ IN CHARACTERISTICS OF THE RESERVOIR
C       (1) STAGE VS. STORAGE
C       (2) STAGE VS. DISCHARGE
      OPEN(UNIT=4,NAME='DAM.DAT',TYPE='OLD')

C  READ IN STAGE-STORAGE RELATIONSHIPS
      READ(4,*)NPAIRS,BASEQ,DRQDT
      DO 100 I=1,NPAIRS
 100  READ(4,*)ELEV(I),STORE(I)

C  READ IN THE STAGE-DISCHARGE DATA FOR BOTH THE PRINCIPAL AND
C     EMERGENCY SPILLWAY.  DATA IS READ IN THE FOLLOWING FORM:
C       ELEV, FLOWS CORRESPONDING TO THE DIFFERENT GATE SETTINGS
C       FOR THAT ELEVATION.
C     ALSO READ IN SOME GENERAL INFORMATION ON THE SPILLWAY SYSTEM
C     SUCH AS THE NUMBER OF GATES AND CREST ELEVATIONS.

      READ(4,*)NPSETS,PGATES,PCREST,NPPAIR
      DO 200 J=1,NPPAIR
 200  READ(4,*)HP(J),(PQ(J,N),N=1,NPSETS)
C       1- GATES FULLY OPENED
C       2- SETTING @ 0.4M
C       3- SETTING @ 1.0M
C       4- SETTING @ 2.0M
C       5- SETTING @ 3.0M
C       6- SETTING @ 4.0M
C       7- GATE CLOSED

C  READ STAGE-DISCHARGE DATA FOR EMERGENCY SPILLWAY
      READ(4,*)NESETS,EGATES,ECREST,NEPAIR
      DO 300 J=1,NEPAIR
 300  READ(4,*)HE(J),(EQ(J,N),N=1,NESETS)
C       1- GATES FULLY OPENED
C       2- SETTING @ 0.8M
C       3- SETTING @ 1.6M
C       4- SETTING @ 2.4M
C       5- SETTING @ 3.2M
C       6- SETTING @ 4.0M
C       7- SETTING @ 4.8M
C       8- SETTING @ 5.6M
C       9- GATES CLOSED

      IF(PGATES.GT.EGATES)MTGATE=PGATES
      IF(PGATES.LE.EGATES) MTGATE = EGATES
```

```
D       WRITE(5,*)' NPSETS=',NPSETS,'PGATES=',PGATES
D       WRITE(5,*)' NESETS=',NESETS,'EGATES=',EGATES
        CLOSE(UNIT=4,DISPOSE='KEEP')

C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *

C  INITIALIZE VARIABLES AND BEGIN ACTUAL ROUTING PROCEDURE.
        ICOUNT=1
          DO 325 J=1,51
 325      FLOW(J)=FLOW(J)+BASEQ+DBQDT*FLOAT(J-1)


C  COMPUTE THE NECESSARY DATA CURVES USING THE SUBROUTINE DTCURV
        CALL DTCURV(RTIME)

C * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * * *


C  COMPLETE THE RESERVOIR ROUTING USING THE MODIFIED PULS METHOD
 1000 WRITE(5,*)' GIVE THE INITIAL WATER SURFACE ELEVATION.'
        READ(5,*)WSEL
        INWSEL=WSEL
        CWSEL=INT(WSEL)
        WRITE(2,10)
 10     FORMAT('1',///,T21,'OUTFLOW HYDROGRAPH FROM MANANTALI DAM',
       2 ///' ',T12,'TIME',T20,'INFLOW',T32,'OUTFLOW',
       3 T45,'ELEV',T56,'STORAGE',/)
        IF(TUNITS.EQ.'H')GO TO 5100
        WRITE(2,20)
 20     FORMAT('+',T11,'(DAYS)')
        GO TO 5300

 5100 WRITE(2,21)
 21     FORMAT('+',T11,'(HOURS)')

 5300 IF(UNITS.EQ.'M')GO TO 5500
        WRITE(2,50)
 50     FORMAT('+',T21,'(CFS)',T33,'(CFS)',T43,'(FT.MSL)',T56,'(AC-FT)'/)
        GO TO 1050

 5500 WRITE(2,60)
 60     FORMAT('+',T20,'(M3/S)',T32,'(M3/S)',T44,'(MASL)',T58,'(M3)'/)

C       ------------------------------------
 1050     JJ=2
          CALL INTERP(WSEL,ELEV,STORE,JJ,S)
          JJ=2
          CALL APPROX(WSEL,H,TOTALQ,JJ,LMAX,QOUT)
          JJ=2
          JQ=2
          JS=2
          TIME=1
          QMAX=QOUT
          ELMAX=INWSEL
        WRITE(2,90)TIME,FLOW(1),QOUT,WSEL,S
        DO 720 I=2,51
 700    STOIND=FLOW(I-1)+FLOW(I)-QOUT+2./RTIME*S/CTIME
          CALL INTERP(STOIND,STOQ,TOTALQ,JJ,QOUT)
          CALL APPROX(QOUT,TOTALQ,H,JQ,LMAX,WSEL)
          CALL INTERP(WSEL,ELEV,STORE,JS,S)
          IF(QOUT.GT.QMAX)QMAX=QOUT
          IF(WSEL.GT.ELMAX)ELMAX=WSEL
          IF((WSEL.LT.ELMAX) .AND. (FLOW(I-1).LE.(BASEQ+1.)))GO TO 750
          TIME=TIME+1
        WRITE(2,90)TIME,FLOW(I),QOUT,WSEL,S
 90     FORMAT (T13, I2, T17, F9.1, T29, F9.1, T41, F9.2, T52, E12.3)
```

```
C  ------------------------------------------------------------------
C  OPTION TO ADJUST GATES DURING A RUN

      IF(WSEL.GE.CWSEL .AND. WSEL.LT.(CWSEL+1.))GO TO 720
        IF(WSEL.LT.CWSEL)GO TO 1100
          ENCODE(8,998,LEVEL)
 998      FORMAT(8HABOVE   )
          CWSEL=CWSEL+1.
          GO TO 1200

 1100   ENCODE(8,999,LEVEL)
 999    FORMAT(8HBELOW   )

 1200 WRITE(5,95)LEVEL,CWSEL,WSEL,QOUT
 95   FORMAT(' THE LAKE LEVEL IS NOW ',A5,' ELEVATION',F9.2,
     1 /' WATER SURFACE ELEVATION IS ',F9.2,' THE DISCHARGE IS ',F9.1,
     2 /' DO YOU WISH TO RESET THE GATE OPENINGS?  AT PRESENT THEY ARE:'
,
     3 // T5,'SETTING',T20,'PRINCIPAL GATES',T40,'EMERGENCY GATES')
      WRITE(5,96)(NN,NPGATE(NN),NEGATE(NN),NN=1,MTGATE)
 96   FORMAT(T8,I2,T25,I2,T45,I2)
      WRITE(5,*)' TYPE Y/N'
      READ(5,15)ANSWER
 15   FORMAT(A1)
      CWSEL=INT(WSEL)
      IF(ANSWER.EQ.'Y')CALL DTCURV(RTIME)
C  ------------------------------------------------------------------

 720  CONTINUE
 750  WRITE(2,82)INWSEL,ELMAX,QMAX
 82   FORMAT('0', T12,'INITIAL WATER SURFACE ELEVATION :',F7.2,/
     1 T12,'MAXIMUM WATER SURFACE ELEVATION :',F7.2,/
     2 T12,'MAXIMUM DISCHARGE FROM THE DAM :',F8.2)
      WRITE(5,*)' DO YOU WISH TO MAKE ANOTHER RESERVOIR ROUTING ?'
      WRITE(5,*)' TYPE Y OR N.'
      READ(5,15)ANSWER
      IF(ANSWER.EQ.'N')GO TO 2000
        WRITE(5,*)' DO THE SPILLWAY GATES NEED ADJUSTMENTS ? Y/N'
        READ(5,15)ANSWER
        IF(ANSWER.EQ.'Y')CALL DTCURV(RTIME)
        GO TO 1000
 2000 RETURN
      END



CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C THIS IS A STRAIGHT LINE INTERPOLATION PROCEDURE.
      SUBROUTINE INTERP(CVALUE,XAXIS,YAXIS,J,RESULT)
      REAL CVALUE,XAXIS(20),YAXIS(20),RESULT,DIFX,DIFY
      RESULT=0.0
      IF(CVALUE.LT.XAXIS(J-1))GO TO 1000
        M=1
      GO TO 1100
 1000   M=-1
        J=J-1
 1100 CONTINUE
      DO 100 KK=1,30
       IF(RESULT.GT.0.0)GO TO 1500
        IF((XAXIS(J-1).GT.CVALUE).OR.(XAXIS(J).LT.CVALUE))GO TO 1600
          DIFX=XAXIS(J)-XAXIS(J-1)
          DIFY=YAXIS(J)-YAXIS(J-1)
          RESULT=YAXIS(J-1)+(CVALUE-XAXIS(J-1))*DIFY/DIFX
        GO TO 1500
```

```
 1600      J=J+M
 100  CONTINUE
 1500 RETURN
      END
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC


C *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *
      SUBROUTINE DTCURV(RTIME)
      INTEGER SET,NPGATE,GATES,NEGATE,TIME,NPSETS,NESETS,PGATES,EGATES
      COMMON/RES1/ ELEV(20),STORE(20),BFLOW,
     1 HP(20),HE(20),PQ(20,20),EQ(20,20),LMAX,
     2 Q2(100),H(20),TOTALQ(20),S,S1,S2,STOQ(20),CTIME
      COMMON/RES2/ SET,NPGATE(15),GATES,NEGATE(15),TIME,
     1 NPSETS,NESETS,EGATES,PGATES,PCREST,ECREST,NPPAIR,NEPAIR
      COMMON /ALPHA/UNITS,TUNITS
C --------------------------------------------------------------
C  ESTABLISH THE POSITION OF THE SPILLWAY GATES.
 475  WRITE(5,*)' GIVE GATE SETTING AND NUMBER OF GATES AT THAT SETTING'
      WRITE(5,*)'    FOR THE PRINCIPAL SPILLWAY.'
      WRITE(5,*)' INPUT DATA FOLLOWING THE FORMAT:SETTING,NGATES'
      WRITE(5,*)' THE FOLLOWING IS THE KEY ASSUMED'
      WRITE(5,*)'        1- GATES FULLY OPENED'
      WRITE(5,*)'        2- SETTING @ 0.4M'
      WRITE(5,*)'        3- SETTING @ 1.0M'
      WRITE(5,*)'        4- SETTING @ 2.0M'
      WRITE(5,*)'        5- SETTING @ 3.0M'
      WRITE(5,*)'        6- SETTING @ 4.0M'
      WRITE(5,*)'        7- GATES CLOSED'
      GATES=0
      DO 480 NN=1,NPSETS
 480  NPGATE(NN)=0
      DO 485 KK=1,NPSETS+1
        IF(GATES.EQ.PGATES)GO TO 2000
        READ(5,*)SET,NPGATE(SET)
 485    GATES=GATES+NPGATE(SET)

C  --------------------------------------
 2000 WRITE(5,*)' GIVE THE SAME INFORMATION FOR THE EMERGENCY SPILLWAY'
      WRITE(5,*)'        1- GATES FULLY OPENED'
      WRITE(5,*)'        2- SETTING @ 0.8M'
      WRITE(5,*)'        3- SETTING @ 1.6M'
      WRITE(5,*)'        4- SETTING @ 2.4M'
      WRITE(5,*)'        5- SETTING @ 3.2M'
      WRITE(5,*)'        6- SETTING @ 4.0M'
      WRITE(5,*)'        7- SETTING @ 4.8M'
      WRITE(5,*)'        8- SETTING @ 5.6M'
      WRITE(5,*)'        9- GATES CLOSED'
      GATES=0
      DO 490 NN=1,NESETS
 490  NEGATE(NN)=0
      DO 495 KK=1,NESETS+1
        IF(GATES.EQ.EGATES)GO TO 2100
        READ(5,*)SET,NEGATE(SET)
 495    GATES=GATES+NEGATE(SET)
C  -----------------------------------------
C  COMPUTE A STAGE-TOTAL DISCHARGE CURVE FOR THE CONDITIONS GIVEN
 2100 CONTINUE
D        WRITE(2,*)' STAGE-TOTAL DISCHARGE CURVE'
      DO 600 L=1,NPPAIR
      TOTALQ(L)=0.0
      H(L)=HP(L)
      IF(HP(L).GT.ECREST)GO TO 800
```

```
      DO 500 K=1,NPSETS
        IF(NPGATE(K).EQ.0)GO TO 500
          TOTALQ(L)=TOTALQ(L)+ NPGATE(K)*PQ(L,K)
 500  CONTINUE
D       WRITE(2,*)H(L),TOTALQ(L)
 600  CONTINUE
 800  LP=L
      DO 900 JE=2,NEPAIR
      TOTALQ(L)=0.0
      H(L)=HE(JE)

        DO 700 KE=1,NESETS
          IF(NEGATE(KE).EQ.0)GO TO 700
           TOTALQ(L)=TOTALQ(L)+ NEGATE(KE)*EQ(JE,KE)
 700    CONTINUE

        DO 750 KP=1,NPSETS
          IF(NPGATE(KP).EQ.0)GO TO 750
          CALL APPROX(H(L),HP,PQ(1,KP),LP,NPPAIR,QOUT)
D         WRITE(2,*)H(L),QOUT,'PLUS',TOTALQ(L),'='
          TOTALQ(L)=NPGATE(KP)*QOUT + TOTALQ(L)
 750    CONTINUE
D       WRITE(2,*)'+',TOTALQ(L)
      L=L+1
 900  CONTINUE
      LMAX=L-1
D     WRITE(2,*)' LMAX IS :',LMAX

C  DEVELOP A STORAGE-INDICATION CURVE, STOQ VS. QOUT
D       WRITE(2,*)' STORAGE-INDICATION CURVE'
        KK=1
      DO 1000 K=1,LMAX
        CALL INTERP(H(K),ELEV,STORE,KK,S1)
        IF(UNITS.EQ.'E')GO TO 3000
        IF(TUNITS.EQ.'D')CTIME=86400.0
        IF(TUNITS.EQ.'H')CTIME=3600.0
        GO TO 3100

 3000   IF(TUNITS.EQ.'D')CTIME=0.504
        IF(TUNITS.EQ.'H')CTIME=12.10

 3100   STOQ(K)=2.*S1/RTIME/CTIME + TOTALQ(K)
D       WRITE(2,*)H(K),TOTALQ(K),S1,STOQ(K)
 1000 CONTINUE
      RETURN
      END



C * * * * * * * * * * * * * * * * * * * * *
C  THIS SUBROUTINE USES THE LAGRANGE'S FORMULA FOR INTERPOLATING
C    A POLYNOMIAL, SUCH AS THE STAGE-TOTAL DISCHARGE CURVE.
C    THE FORMULA USES THE ORGINAL VALUES OF THE FUNCTION.  FOUR
C    KNOWN POINTS ARE USED TO APPROXIMATE Y, TWO POINTS ON THE
C    LEFT SIDE OF X AND TWO POINTS TO THE RIGHT OF X.  AT THE END
C    POINTS, FOUR DATA TO THE KNOWN SIDE ARE USED.
C * * * * * * * * * * * * * * * * * * * * *

      SUBROUTINE APPROX(X,XX,YY,I,IMAX,Y)
      DIMENSION XX(20),YY(20)


      Y=0.0
      IF(I.GE.IMAX)GO TO 1200
      IF(X. LT. XX(I)) GO TO 1000
```

```
      M=1
      GO TO 1100
1000   M=-1
      I=I-1
1100 CONTINUE

      DO 100 KK=1,IMAX
        IF(Y.GT.0.0)GO TO 2000
        IF((XX(I).GT.X).OR.(XX(I+1).LT.X))GO TO 1500
          LL=I-1
          IF(I.EQ.1)LL=I
          IF(I.EQ.IMAX-1)LL=I-2
1200      IF(I.GE.IMAX)LL=IMAX-3

        DO 200 N=LL,LL+3
          TERM=YY(N)

          DO 300 NM=LL,LL+3
            IF(N.EQ.NM)GO TO 300
            TERM=TERM*((X-XX(NM))/(XX(N)-XX(NM)))
300       CONTINUE
200     Y=Y+TERM

1500 I=I+M
100  CONTINUE
2000 RETURN
     END
```

```
C       THIS SUBROUTINE COMPUTES THE MAGNITUDE AND RELATIVE ERRORS
C        GIVEN A KNOWN OUTFLOW HYDROGRAPH.  IT ALSO COMPUTES THE
C        COEFFICIENT OF DETERMINATION,COEFFICIENT OF EFFICIENCY
C        AND THE SUM OF THE DEPARTURES SQUARED.
C *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *  *
      SUBROUTINE ERROR(FLOW,UNITS,TUNITS)
      INTEGER J
      DIMENSION FLOW(100)
      REAL TOTALQ,RECQ,ABSERR,RELERR,SUM,BASEQ,FLOW,XSQ,
     1 YSQ,RSQ,SUMX,SUMY,VARX,VARY,XYSUM,COE,DIFSQ
        READ(1,*)BASEQ,DBQDT
        WRITE(2,70)
 70     FORMAT('1',T16,'RUNOFF HYDROGRAPH FROM THE BASIN')
        WRITE(2,71)
 71     FORMAT('0',T12,'TIME',T23,'COMPUTED Q',T39,'TOTAL Q',
     1  T53,'RECORDED Q',T68,'ABS ERROR',T82,'REL ERROR')
        IF(UNITS.EQ.'M')GO TO 2500
        IF(TUNITS.EQ.'H')GO TO 2100
          WRITE(2,80)
 80       FORMAT(' ',T11,'(DAYS)')
        GO TO 2300

 2100   WRITE(2,81)
 81     FORMAT(' ',T11,'(HOURS)')
 2300   WRITE(2,82)
 82     FORMAT('+',T26,'(CFS)',T39,'(CFS)',T56,'(CFS)',T70,'(CFS)',
     1  T85,'(%)')
        GO TO 3100

 2500   IF(TUNITS.EQ.'H')GO TO 2700
        WRITE(2,80)
        GO TO 3000

 2700   WRITE(2,81)
 3000   WRITE(2,83)
 83     FORMAT('+',T25,'(M3/S)',T39,'(M3/S)',T55,'(M3/S)',T70,'(M3/S)',
     1   T85,'(%)')

 3100     SUM=0.0
          SUMX=0.0
          SUMY=0.0
          XSQ=0.0
          YSQ=0.0
          XYSUM=0.0
          DIFSQ=0.0

        DO 200 II=1,100
          READ(1,*)RECQ
          IF(RECQ.LT.0.0)GO TO 1100
          TOTALQ=FLOW(II)+BASEQ+DBQDT*FLOAT(II-1)
C         TOTALQ: COMPUTED SURFACE RUNOFF PLUS BASE FLOW

          RECRO=RECQ - BASEQ + FLOAT(II-1)*DBQDT
C         RECRP: RECORDED SURFACE RUNOFF ASSUMING THE ABOVE BASE FLOW

      ABSERR=TOTALQ-RECQ
      RELERR=ABSERR/RECQ*100
      SUM=SUM+ABS(RELERR)
      SUMX=SUMX+ FLOW(II)
      XSQ=XSQ+(FLOW(II)*FLOW(II))
      SUMY=SUMY+RECRO
      YSQ= YSQ + (RECRO * RECRO)
```

```
      XYSUM=XYSUM+(FLOW(II)*RECRO)
      DIFSQ=DIFSQ + (FLOW(II)-RECRO)*(FLOW(II)-RECRO)
          WRITE(2,72)II,FLOW(II),TOTALQ,RECQ,ABSERR,RELERR
 72       FORMAT(T12,I3,T22,F9.1,T37,F9.1,T52,F9.1,T67,F8.1,T82,F7.2)
 200  CONTINUE

 1100 J=II-1
      VARX=((FLOAT(J)*XSQ)-(SUMX*SUMX))
      VARY=((FLOAT(J)*YSQ)-(SUMY*SUMY))
      RSQ=((FLOAT(J)*XYSUM)-(SUMX*SUMY))**2/VARX/VARY
      COE=(VARY-DIFSQ)/VARY
      AVERR=SUM/FLOAT(J)

D       WRITE(2,74)SUMX,SUMY
D74     FORMAT('0RUNOFF VOLUMES IN UNITS OF RATE * TIME: COMPUTED=',F9.0
,/
D    1  ,50X,'RECORDED=',F9.0)
        WRITE(2,75)AVERR,RSQ
 75     FORMAT('0THE AVERAGE RELATIVE ERROR,AVERR=',F7.2,'%',/
     1  ' THE COEFFICIENT OF DETERMINATION,R SQUARED=',F5.3)
        WRITE(2,85)DIFSQ,COE
 85     FORMAT('0THE SUM OF THE DEPARTURES SQUARED=',F9.0,/
     1  ' THE COEFFICIENT OF EFFICIENCY,COE=',F5.3)

      RETURN
      END
```

INPUT - OUTPUT

SEQEXE.DAT

Line: NEXE - the number of instructions that will be performed during the run. Example: for a simple test run of RUNOFF, SUM, ROUTE, RESVOR, NEXE equals 4.

Line: ROUTINE - integer code for the name of the subroutine that is to be executed. See table below.

SUBBAS - the subbasin number associated with the routine. Each routine and subbasin number must be typed on one line and in the exact order of performance that is desired.

col 1-2 integer code for the subroutine

col 3 comma or blank

col 4-5 subbasin integer number

Subroutine	Integer Code
RUNOFF	11
ROUTE	22
SUM	33
RESVOR	44
ERROR	55

If ERROR is part of the instructions, then the following additional data must be included in SEQEXE.DAT immediately after the command.

Line: BASEQ - the value of the baseflow at the beginning of the storm.

DBQDT - the incremental change in the baseflow per unit of time.

Line: RECQ - value of the recorded streamflow. After the last recorded streamflow value, a negative flow must be inputted to act as a default mechanism. A value equal to -1.0 is sufficient.

NOTE - the baseflow component of the flood hydrograph is an inputted value. For a known hydrograph, the straight line method of baseflow separation (Chow 1964; Lindsey et al 1975) is used. BASEQ represents the flow at the point of rise on the hydrograph and DBQDT is the slope of the line drawn from BASEQ to an arbitrary point on the lower portion of the recession segment. DBQDT can be a positive or negative value, representing an increase or decrease in the baseflow, respectively.

SBX.DAT

1. Watershed characteristics.

   Line: DA - drainage area of the subbasin (km^2/mi^2)

   TC - time of concentration (days/hrs)

   DELTIM - time increment for the runoff hydrograph. Must be consistent with the precipitation data.

NPAIRS - number of points that define the time-area curve.

COIN - routing coefficient used to shape the runoff hydrograph.

2. Time-area curve.

Line: One pair of data per line.

PERTC(I) - percentage of TC, range 0.0 - 1.0.

ACAREA(I) - corresponding cumulative area whose travel time is equal or less than PERTC(I) * TC, range 0.0 - DA

3. Storm hyetographs - dimensionless or actual.

Line: PRECIP - total amount of precipitation during the storm. If an actual event is given, then PRECIP = 1.0.

BGRAIN - time at which the storm event begins in the subbasin.

EDRAIN - time at which the storm ends.

Line: T - time of the precipitation (day/hr)

HYEDIM(J) - hyetograph ordinate corresponding to T (mm/in)

(One pair of data per line).

4. Soil moisture index - runoff coefficient.

Line: FIELDC - maximum depth of water held in a column of soil after free drainage, analogous to field capacity (mm/in).

	WILTPT	- minimum depth of water held in a column of soil when soil water tension equals the suction force of plants (mm/in).
	ET	- average evapotranspiration loss during the time interval
	SMIT	- initial soil moisture index,
	ROPMAX	- maximum runoff coefficient that applies when SMIT > FIELDC
	ROPMIN	- minimum runoff coefficient that applies when SMIT < WILTPT
	NPOINT	- number of points that define the soil mositure-runoff coefficient curve.
	IOPT	- number of curves to be used to define the runoff characteristics based on rainfall intensity, maximum value of 5.
Line:	RI(J)	- rainfall intensity associated with SMI-ROP curves. If only one curve will be used, this line is omitted.
Line:	SMI(I)	- soil moisture index
	ROP(I,J)	- percent runoff corresponding to SMI(J) and curve J.

DAM.DAT

Line:	NPAIRS	- number of points that define the stage-storage relationship
	BASEQ	- as previously defined
	DBQDT	- as previously defined

1. Stage-storage relationship

Line: ELEV(I) - elevation (m/ft)

STORE(I) - storage volume corresponding to ELEV(I) (m^3/ac-ft)

2. Stage-discharge

Line: NPSETS - the number of curves that define the performance of the principal spillway. Corresponds to the number of gate settings.

PGATES - the number of gates for the principal spillway

PCREST - the minimum crest elevation of the principal spillway

NPPAIR - the number of elevation points and corresponding discharges that define the principal spillway rating.

Line: HP - elevation

PQ(J) - discharges corresponding to elevation HP. (J ranges from 1 to NPSETS)

(One set of data per line)

Line: NESETS - the number of curves that define the per formance of the emergency spillway. Corresponds to the number of gate settings.

EGATES - the number of gates for the emergency spillway

ECREST - the minimum crest elevation of the emergency spillway

NEPAIR - the number of elevation points and corresponding discharges that define the emergency spillway rating.

Line: HE - elevation

EQ(I) - discharges corresponding to elevation HE. (I ranges from 1 to NESETS)

(One set of data per line)

The following pages are samples of the input and output for the September 1965 flood used in the calibration process.

SEREKE.DAT

```
16
11.01   (RUNOFF)
33.01   (SUM)
11.02   (RUNOFF)
33.02   (SUM)
22.01   (ROUTE)
11.03   (RUNOFF)
33.03
22.02
11.04
33.04
22.03
11.05
33.05
11.06
33.06
55.06   (ERROR)
1400.0,0.0
1400.0
1480.0
1900.0
2030.0
2120.0
2250.0
2450.0
2540.0
2800.0
3130.0
3100.0
2900.0
2670.0
2550.0
2490.0
2200.0
2030.0
1850.0
1550.0
1500.0
1510.0
1460.0
-1.0
```

SB1.DAT

```
4100.,1.0,1.0,2,0.19
130.,130.,1.0,0.4,1.0
0.0,0.0
1.0,4100.
1, 1, 7
1, 36.5
2, 32.3
3, 32.5
4, 0.0
5, 38.3
6, 48.3
7, 46.5
300., 100., 4.0, 110.0, 0.7, 0.05, 7, 3
20.0, 45.0
100., 0.05, 0.05, 0.05
120., 0.10, 0.10, 0.10
130., 0.13, 0.13, 0.20
150., 0.18, 0.30, 0.45
200., 0.40, 0.50, 0.65
250., 0.45, 0.65, 0.75
300., 0.50, 0.70, 0.80
250.,0.0
```

SB2.DAT

```
4400.0, 1.0, 1.0, 2, 0.19
210.0, 210.0, 1.3, 0.4, 1.0
0.0, 0.0
1.0, 4400.0
1, 1, 7
1, 36.5
2, 32.3
3, 32.5
4, 0.0
5, 38.3
6, 48.3
7, 46.5
300., 100., 4.0, 110.0, 0.7, 0.05, 7, 3
20.0, 45.0
100., 0.05, 0.05, 0.05
120., 0.10, 0.10, 0.10
130., 0.13, 0.13, 0.20
150., 0.18, 0.30, 0.45
200., 0.40, 0.50, 0.65
250., 0.45, 0.65, 0.75
300., 0.50, 0.70, 0.80
250.,0.0
```

SB3.DAT

```
7200.0, 1.3, 1.0, 3, 0.17
110.0, 110.0, 0.7, 0.4, 1.0
0.0, 0.0
1.0, 3340.0
1.3, 7200.0
1, 1, 10
1, 16.5
2, 7.0
3, 4.3
4, 2.9
5, 11.6
6, 57.0
7, 0.0
8, 0.0
9, 1.9
10, 38.4
300., 100., 4.0, 110.0, 0.7, 0.05, 7, 3
20.0, 45.0
100., 0.05, 0.05, 0.05
120., 0.10, 0.10, 0.10
130., 0.13, 0.13, 0.20
150., 0.18, 0.30, 0.45
200., 0.40, 0.50, 0.65
250., 0.45, 0.65, 0.75
300., 0.50, 0.70, 0.80
250.0,0.0
```

SB4.DAT

```
6769.0, 1.0, 1.0, 2, 0.17
120.0, 120.0, 0.8, 0.2, 1.0
0.0, 0.0
1.0, 6769.0
1,1,7
1, 4.5
2, 16.0
3, 0.0
4, 40.8
5, 10.8
6, 52.7
7, 0.0
8, 0.0
9, 40.0
300., 100., 7.0, 110.0, 0.7, 0.05, 7, 3
20.0, 45.0
100., 0.05, 0.05, 0.05
120., 0.10, 0.10, 0.10
130., 0.13, 0.13, 0.20
150., 0.18, 0.30, 0.45
200., 0.40, 0.50, 0.65
250., 0.45, 0.65, 0.75
300., 0.50, 0.70, 0.80
250.0,0.0
```

SB5.DAT

```
5331.0, 1.0, 1.0, 2, 0.17
120.0, 120.0, 0.8, 0.2, 1.0
0.0, 0.0
1.0, 5331.0
1,1,9
1, 23.2
2, 9.2
3, 8.6
4, 55.5
5, 6.2
6, 36.1
7, 0.0
8, 0.0
9, 52.4
300., 100., 7.0, 110.0, 0.7, 0.05, 7, 3
20.0, 45.0
100., 0.05, 0.05, 0.05
120., 0.10, 0.10, 0.10
130., 0.13, 0.13, 0.20
150., 0.18, 0.30, 0.45
200., 0.40, 0.50, 0.65
250., 0.45, 0.65, 0.75
300., 0.50, 0.70, 0.80
250.,0.0
```

SB6.DAT

```
5700.0, 1.0, 1.0, 2, 0.17
1.0, 1.0, 1.0, 0.2, 1.0
0.0, 0.0
1.0, 5700.0
1,1,10
1, 13.4
2, 16.1
3, 4.4
4, 5.2
5, 13.2
6, 41.5
7, 0.0
8, 0.0
9, 10.2
10, 38.0
300., 100., 7.0, 110.0, 0.7, 0.05, 7, 3
20.0, 45.0
100., 0.05, 0.05, 0.05
120., 0.10, 0.10, 0.10
130., 0.13, 0.13, 0.20
150., 0.18, 0.30, 0.45
200., 0.40, 0.50, 0.65
250., 0.45, 0.65, 0.75
300., 0.50, 0.70, 0.80
250.0,0.0
```

FLOOD.DAT

THE COMBINED HYDROGRAPH AT THE OUTLET OF SUBBASIN # 1

TIME	1	2	3	4	5	6	7
DISCHARGE	25.6	85.0	173.6	140.6	256.4	489.5	685.7
TIME	8	9	10	11	12	13	14
DISCHARGE	555.4	449.9	364.4	295.2	239.1	193.7	156.9
TIME	15	16	17	18	19	20	21
DISCHARGE	127.1	102.9	83.4	67.5	54.7	44.3	35.9
TIME	22	23	24	25	26	27	28
DISCHARGE	29.1	23.5	19.1	15.4	12.5	10.1	8.2
TIME	29	30	31	32	33	34	35
DISCHARGE	6.6	5.4	4.4	3.5	0.0	0.0	0.0
TIME	36	37	38	39	40	41	42
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

THE COMBINED HYDROGRAPH AT THE OUTLET OF SUBBASIN # 2

TIME	1	2	3	4	5	6	7
DISCHARGE	53.1	176.3	359.9	291.5	531.5	1014.8	1421.6
TIME	8	9	10	11	12	13	14
DISCHARGE	1151.5	932.7	755.5	611.9	495.7	401.5	325.2
TIME	15	16	17	18	19	20	21
DISCHARGE	263.4	213.4	172.8	140.0	113.4	91.8	74.4
TIME	22	23	24	25	26	27	28
DISCHARGE	60.3	48.8	39.5	32.0	25.9	21.0	17.0
TIME	29	30	31	32	33	34	35
DISCHARGE	13.8	11.2	9.0	7.3	0.0	0.0	0.0
TIME	36	37	38	39	40	41	42
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

THE ROUTED HYDROGRAPH AT THE END OF REACH # 1

TIME	1	2	3	4	5	6	7
DISCHARGE	-0.8	39.3	143.5	313.6	292.6	471.7	889.6
TIME	8	9	10	11	12	13	14
DISCHARGE	1309.4	1189.5	991.6	809.4	657.0	532.4	431.3
TIME	15	16	17	18	19	20	21
DISCHARGE	349.4	283.0	229.2	185.7	150.4	121.8	98.7
TIME	22	23	24	25	26	27	28
DISCHARGE	79.9	64.7	52.4	42.5	34.4	27.9	22.6
TIME	29	30	31	32	33	34	35
DISCHARGE	18.3	14.8	12.0	9.7	8.0	1.7	0.4
TIME	36	37	38	39	40	41	42
DISCHARGE	0.1	0.0	0.0	0.0	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

THE COMBINED HYDROGRAPH AT THE OUTLET OF SUBBASIN # 3

TIME	1	2	3	4	5	6	7
DISCHARGE	5.7	60.2	170.2	342.0	525.6	565.4	1066.0
TIME	8	9	10	11	12	13	14
DISCHARGE	1455.8	1313.3	1167.4	1079.1	880.0	718.2	585.5
TIME	15	16	17	18	19	20	21
DISCHARGE	477.4	389.2	317.4	258.9	211.1	172.2	140.5
TIME	22	23	24	25	26	27	28
DISCHARGE	114.7	93.6	76.4	62.3	50.9	41.5	33.9
TIME	29	30	31	32	33	34	35
DISCHARGE	27.7	22.6	18.5	15.1	12.4	5.5	3.5
TIME	36	37	38	39	40	41	42
DISCHARGE	2.6	0.0	0.0	0.0	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

THE ROUTED HYDROGRAPH AT THE END OF REACH # 2

TIME	1	2	3	4	5	6	7
DISCHARGE	1.4	19.1	90.1	218.2	348.8	390.9	701.1
TIME	8	9	10	11	12	13	14
DISCHARGE	1190.9	1444.8	1267.0	1137.6	1026.5	829.2	676.0
TIME	15	16	17	18	19	20	21
DISCHARGE	551.7	449.8	366.8	299.1	243.9	199.0	162.3
TIME	22	23	24	25	26	27	28
DISCHARGE	132.4	108.1	88.2	72.0	58.8	48.0	39.2
TIME	29	30	31	32	33	34	35
DISCHARGE	32.0	26.1	21.3	17.4	14.3	10.6	4.5
TIME	36	37	38	39	40	41	42
DISCHARGE	3.2	2.0	-0.2	0.0	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

THE COMBINED HYDROGRAPH AT THE OUTLET OF SUBBASIN # 4

TIME	1	2	3	4	5	6	7
DISCHARGE	6.0	38.0	105.8	270.7	415.2	706.5	971.4
TIME	8	9	10	11	12	13	14
DISCHARGE	1415.2	1814.9	1574.2	1392.6	1238.2	1004.9	822.6
TIME	15	16	17	18	19	20	21
DISCHARGE	672.7	550.3	450.2	368.3	301.4	246.7	201.9
TIME	22	23	24	25	26	27	28
DISCHARGE	165.3	135.3	110.8	90.8	74.3	60.9	49.9
TIME	29	30	31	32	33	34	35
DISCHARGE	40.9	33.5	27.5	22.5	18.5	14.1	4.5
TIME	36	37	38	39	40	41	42
DISCHARGE	3.2	2.0	-0.2	0.0	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

THE ROUTED HYDROGRAPH AT THE END OF REACH # 3

TIME	1	2	3	4	5	6	7
DISCHARGE	1.8	15.0	55.4	148.8	298.8	487.8	752.7
TIME	8	9	10	11	12	13	14
DISCHARGE	1077.6	1493.0	1703.6	1535.9	1364.2	1184.1	972.5
TIME	15	16	17	18	19	20	21
DISCHARGE	796.3	651.4	532.8	435.9	356.7	291.9	238.9
TIME	22	23	24	25	26	27	28
DISCHARGE	195.5	160.1	131.1	107.3	87.9	72.0	59.0
TIME	29	30	31	32	33	34	35
DISCHARGE	48.3	39.6	32.5	26.6	21.8	17.8	11.7
TIME	36	37	38	39	40	41	42
DISCHARGE	5.0	3.0	1.5	0.1	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

THE COMBINED HYDROGRAPH AT THE OUTLET OF SUBBASIN # 5

TIME	1	2	3	4	5	6	7
DISCHARGE	20.7	42.1	88.7	274.3	419.9	727.0	957.2
TIME	8	9	10	11	12	13	14
DISCHARGE	1242.4	1922.0	2059.7	1831.5	1609.5	1387.7	1141.5
TIME	15	16	17	18	19	20	21
DISCHARGE	936.6	767.8	629.5	516.1	423.2	347.1	284.7
TIME	22	23	24	25	26	27	28
DISCHARGE	233.6	191.7	157.3	129.1	106.0	87.0	71.4
TIME	29	30	31	32	33	34	35
DISCHARGE	58.7	48.2	39.6	32.5	26.7	21.7	11.7
TIME	36	37	38	39	40	41	42
DISCHARGE	5.0	3.0	1.5	0.1	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

THE COMBINED HYDROGRAPH AT THE OUTLET OF SUBBASIN # 6

TIME	1	2	3	4	5	6	7
DISCHARGE	32.4	68.3	116.1	303.1	458.2	810.9	1026.8
TIME	8	9	10	11	12	13	14
DISCHARGE	1300.2	1988.0	2210.6	1956.7	1713.4	1474.0	1213.1
TIME	15	16	17	18	19	20	21
DISCHARGE	996.0	817.1	670.4	550.1	451.4	370.5	304.1
TIME	22	23	24	25	26	27	28
DISCHARGE	249.7	205.0	168.4	138.3	113.6	93.3	76.7
TIME	29	30	31	32	33	34	35
DISCHARGE	63.0	51.8	42.6	35.0	28.8	23.4	13.1
TIME	36	37	38	39	40	41	42
DISCHARGE	5.0	3.0	1.5	0.1	0.0	0.0	0.0
TIME	43	44	45	46	47	48	49
DISCHARGE	0.0	0.0	0.0	0.0	0.0	0.0	0.0
TIME	50	51					
DISCHARGE	0.0	0.0					

RUNOFF HYDROGRAPH FROM THE BASIN

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)	RECORDED Q (M3/S)	ABS ERROR (M3/S)	REL ERROR (%)
1	32.4	1432.4	1400.0	32.4	2.31
2	68.3	1448.3	1480.0	-11.7	-0.79
3	114.1	1514.1	1900.0	-383.9	-20.20
4	303.1	1703.1	2030.0	-326.9	-16.10
5	458.2	1858.2	2120.0	-261.8	12.35
6	610.9	2210.9	2250.0	-39.1	-1.74
7	1026.8	2426.8	2450.0	-23.2	-0.95
8	1300.2	2700.2	2540.0	160.2	6.31
9	1988.0	3388.0	2800.0	588.0	21.00
10	2210.6	3610.6	3130.0	480.6	15.35
11	1956.7	3356.7	3100.0	256.7	8.28
12	1713.4	3113.4	2900.0	213.4	7.36
13	1474.0	2874.0	2670.0	204.0	7.64
14	1213.1	2613.1	2550.0	63.1	2.48
15	996.0	2396.0	2490.0	-94.0	-3.77
16	817.1	2217.1	2200.0	17.1	0.78
17	670.4	2070.4	2030.0	40.4	1.99
18	550.1	1950.1	1850.0	100.1	5.41
19	451.4	1851.4	1650.0	201.4	12.21
20	370.5	1770.5	1600.0	170.5	10.66
21	304.1	1704.1	1510.0	194.1	12.86
22	249.7	1649.7	1460.0	189.7	12.99

RUNOFF VOLUMES IN UNITS OF RATE * TIME: COMPUTED= 19081.
RECORDED= 17310.

THE AVERAGE RELATIVE ERROR,AVERR= 9.34%
THE COEFFICIENT OF DETERMINATION,R SQUARED=0.898

THE SUM OF THE DEPARTURES SQUARED= 1249456.
THE COEFFICIENT OF EFFICIENCY,COE=0.991

HYDRO.DAT

RUNOFF HYDROGRAPH FROM SUBBASIN # 1

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)
1	25.6	275.6
2	85.0	335.0
3	173.6	423.6
4	140.6	390.6
5	256.4	506.4
6	489.5	739.5
7	685.7	935.7
8	555.4	805.4
9	449.9	699.9
10	364.4	614.4
11	295.2	545.2
12	239.1	489.1
13	193.7	443.7
14	156.9	406.9
15	127.1	377.1
16	102.9	352.9
17	83.4	333.4
18	67.5	317.5
19	54.7	304.7
20	44.3	294.3
21	35.9	285.9
22	29.1	279.1
23	23.5	273.5
24	19.1	269.1
25	15.4	265.4
26	12.5	262.5
27	10.1	260.1
28	8.2	258.2
29	6.6	256.6
30	5.4	255.4
31	4.4	254.4
32	3.5	253.5

RUNOFF HYDROGRAPH FROM SUBBASIN # 2

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)
1	27.5	277.5
2	91.3	341.3
3	186.3	436.3
4	150.9	400.9
5	275.1	525.1
6	525.3	775.3
7	735.9	985.9
8	596.1	846.1
9	482.8	732.8
10	391.1	641.1
11	316.8	566.8
12	256.6	506.6
13	207.8	457.8
14	168.3	418.3
15	136.4	386.4
16	110.5	360.5
17	89.5	339.5
18	72.5	322.5
19	58.7	308.7
20	47.5	297.5
21	38.5	288.5
22	31.2	281.2
23	25.3	275.3
24	20.5	270.5
25	16.6	266.6
26	13.4	263.4
27	10.9	260.9
28	8.8	258.8
29	7.1	257.1
30	5.8	255.8
31	4.7	254.7
32	3.8	253.8

RUNOFF HYDROGRAPH FROM SUBBASIN # 3

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)
1	6.5	256.5
2	20.9	270.9
3	26.7	276.7
4	28.3	278.3
5	33.0	283.0
6	93.7	343.7
7	176.3	426.3
8	146.4	396.4
9	123.8	373.8
10	175.7	425.7
11	269.7	519.7
12	223.8	473.8
13	185.8	435.8
14	154.2	404.2
15	128.0	378.0
16	106.2	356.2
17	88.2	338.2
18	73.2	323.2
19	60.7	310.7
20	50.4	300.4
21	41.8	291.8
22	34.7	284.7
23	28.8	278.8
24	23.9	273.9
25	19.9	269.9
26	16.5	266.5
27	13.7	263.7
28	11.4	261.4
29	9.4	259.4
30	7.8	257.8
31	6.5	256.5
32	5.4	255.4
33	4.5	254.5
34	3.7	253.7
35	3.1	253.1
36	2.6	252.6

RUNOFF HYDROGRAPH FROM SUBBASIN # 4

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)
1	4.7	254.7
2	18.9	268.9
3	15.7	265.7
4	52.5	302.5
5	66.4	316.4
6	325.6	575.6
7	270.3	520.3
8	224.3	474.3
9	370.2	620.2
10	307.2	557.2
11	255.0	505.0
12	211.7	461.7
13	175.7	425.7
14	145.8	395.8
15	121.0	371.0
16	100.5	350.5
17	83.4	333.4
18	69.2	319.2
19	57.4	307.4
20	47.7	297.7
21	39.6	289.6
22	32.8	282.8
23	27.3	277.3
24	22.6	272.6
25	18.8	268.8
26	15.6	265.6
27	12.9	262.9
28	10.7	260.7
29	8.9	258.9
30	7.4	257.4
31	6.1	256.1
32	5.1	255.1
33	4.2	254.2
34	3.5	253.5

RUNOFF HYDROGRAPH FROM SUBBASIN # 5

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)
1	18.9	268.9
2	27.0	277.0
3	33.4	283.4
4	125.6	375.6
5	121.0	371.0
6	239.2	489.2
7	198.5	448.5
8	164.8	414.8
9	429.0	679.0
10	356.1	606.1
11	295.6	545.6
12	245.3	495.3
13	203.6	453.6
14	169.0	419.0
15	140.3	390.3
16	116.4	366.4
17	96.6	344.6
18	80.2	330.2
19	66.6	316.6
20	55.3	305.3
21	45.9	295.9
22	38.1	288.1
23	31.6	281.6
24	26.2	276.2
25	21.8	271.8
26	18.1	268.1
27	15.0	265.0
28	12.4	262.4
29	10.3	260.3
30	8.6	258.6
31	7.1	257.1
32	5.9	255.9
33	4.9	254.9
34	4.1	254.1

RUNOFF HYDROGRAPH FROM SUBBASIN # 6

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)
1	11.7	261.7
2	26.3	276.3
3	27.4	277.4
4	28.8	278.8
5	38.3	288.3
6	83.9	333.9
7	69.6	319.6
8	57.8	307.8
9	65.9	315.9
10	150.9	400.9
11	125.2	375.2
12	103.9	353.9
13	86.3	336.3
14	71.6	321.6
15	59.4	309.4
16	49.3	299.3
17	40.9	290.9
18	34.0	284.0
19	28.2	278.2
20	23.4	273.4
21	19.4	269.4
22	16.1	266.1
23	13.4	263.4
24	11.1	261.1
25	9.2	259.2
26	7.7	257.7
27	6.4	256.4
28	5.3	255.3
29	4.4	254.4
30	3.6	253.6
31	3.0	253.0
32	2.5	252.5
33	2.1	252.1
34	1.7	251.7
35	1.4	251.4

DAM.DAT

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14,1500.0,0.0
160.0,0.0E9        --STAGE-STORAGE
170.0,0.45E9
175.5,1.00E9
180.0,1.19E9
185.0,2.97E9
190.0,4.22E9
195.0,5.81E9
200.0,7.68E9
207.5,10.90E9
210.0,12.19E9
215.0,14.71E9
220.0,17.61E9
225.0,20.45E9
230.0,24.64E9     -END CURVE
6,7,187.0,11          NPSETS,PGATES,PCREST,NPPATP
187.0,365.0,23.8,65.0,118.8,175.0,251.2
190.0,383.0,26.2,67.5,123.8,183.8,266.2
192.5,400.0,28.8,70.0,128.8,191.2,277.8
195.0,415.0,30.0,72.5,133.8,198.8,287.5
197.5,430.0,31.2,75.0,137.5,205.0,297.5
200.0,445.0,32.5,76.2,142.5,212.5,307.5
202.0,457.0,33.5,78.3,145.5,218.5,314.5
205.0,475.0,35.0,80.0,150.0,226.2,325.0
207.5,488.8,36.2,81.2,153.8,233.8,333.7
210.0,502.5,37.5,83.8,157.5,241.2,341.2
211.0,508.8,37.8,83.9,158.8,243.8,343.7
8,8,202.0,13          NESETS,EGATES,ECREST,NEPATP
202.0,  0.0,  0.0,  0.0,  0.0,  0.0,  0.0,  0.0,  0.0,  0.0
203.0,  8.9,  8.9,  8.9,  8.9,  8.9,  8.9,  8.9,  8.9,  8.9
204.0, 34.4, 22.2, 34.4, 34.4, 34.4, 34.4, 34.4, 34.4, 34.4
205.0, 70.0, 31.1, 55.5, 70.0, 70.0, 70.0, 70.0, 70.0, 70.0
206.0,117.8, 37.8, 70.0,100.0,117.8,117.8,117.8,117.8,117.8
207.0,170.0, 43.3, 81.1,114.4,148.9,170.0,170.0,170.0,170.0
208.0,237.8, 50.0, 92.3,131.1,168.9,207.8,237.8,237.8,237.8
209.0,306.7, 54.4,102.2,144.4,185.6,227.8,270.0,306.7,306.7
210.0,388.9, 60.0,110.0,157.8,203.3,250.0,295.6,345.1,345.1
211.0,480.0, 64.4,116.7,168.9,220.0,268.9,318.9,368.9,368.9
212.0,575.6, 68.9,123.3,180.0,235.6,288.9,342.2,397.8,397.8
213.0,671.3, 74.8,130.8,193.4,250.0,314.4,367.8,435.7,435.7
214.0,762.7, 83.4,140.2,211.4,263.1,349.8,399.0,517.2,517.2
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Appendix B

Calibration Runs

SEPT 1965

RUNOFF HYDROGRAPH FROM THE BASIN

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)	RECORDED Q (M3/S)	ABS ERROR (M3/S)	REL ERROR (%)
1	32.4	1432.4	1400.0	32.4	2.31
2	68.3	1468.3	1480.0	-11.7	-0.79
3	116.1	1516.1	1900.0	-383.9	-20.20
4	303.1	1703.1	2030.0	-326.9	-16.10
5	458.2	1858.2	2120.0	-261.8	-12.35
6	810.9	2210.9	2250.0	-39.1	-1.74
7	1026.8	2426.8	2450.0	-23.2	-0.95
8	1300.2	2700.2	2540.0	160.2	6.31
9	1988.0	3388.0	2800.0	588.0	21.00
10	2210.6	3610.6	3130.0	480.6	15.35
11	1956.7	3356.7	3100.0	256.7	8.28
12	1713.4	3113.4	2900.0	213.4	7.36
13	1474.0	2874.0	2670.0	204.0	7.64
14	1213.1	2613.1	2550.0	63.1	2.48
15	996.0	2396.0	2490.0	-94.0	-3.77
16	917.1	2217.1	2200.0	17.1	0.78
17	670.4	2070.4	2030.0	40.4	1.99
18	550.1	1950.1	1850.0	100.1	5.41
19	451.4	1851.4	1650.0	201.4	12.21
20	370.5	1770.5	1600.0	170.5	10.66
21	304.1	1704.1	1510.0	194.1	12.86
22	249.7	1649.7	1460.0	189.7	12.99

RUNOFF VOLUMES IN UNITS OF RATE X TIME: COMPUTED- 12681.
RECORDED- 17310.

THE AVERAGE RELATIVE ERROR(%)OVER- 9.74%
THE COEFFICIENT OF DETERMINATION,R SQUARED= 0.98

THE SUM OF THE DERIVATIVES SQUARED= 12[illegible]
THE COEFFICIENT OF EFFICIENCY(%)= [illegible]

AUG 1964

RUNOFF HYDROGRAPH FROM THE BASIN

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)	RECORDED Q (M3/S)	ABS ERROR (M3/S)	REL ERROR (%)
1	46.8	1446.8	1400.0	46.8	3.35
2	433.9	1833.9	1440.0	393.9	27.35
3	592.1	1992.1	2200.0	-207.9	-9.45
4	1060.8	2460.8	2850.0	-389.2	-13.66
5	1356.4	2756.4	2850.0	-93.6	-3.28
6	1474.6	2874.6	2780.0	94.6	3.40
7	1761.8	3161.8	2800.0	361.8	12.92
8	1673.7	3073.7	3290.0	-216.3	-6.58
9	1719.7	3119.7	3100.0	19.7	0.63
10	1703.3	3103.3	2830.0	273.3	9.66
11	1449.0	2849.0	2370.0	479.0	20.21
12	1202.4	2602.4	2300.0	302.4	13.15

RUNOFF VOLUMES IN UNITS OF RATE * TIME: COMPUTED= 14474.
RECORDED= 13410.

THE AVERAGE RELATIVE ERROR-AVERR= 10.30%
THE COEFFICIENT OF DETERMINATION-R SQUARED=0.790

THE SUM OF THE DEPARTURES SQUARED= 943408.
THE COEFFICIENT OF EFFICIENCY-COE=0.980

AUG 1958

RUNOFF HYDROGRAPH FROM THE BASIN

TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)	RECORDED Q (M3/S)	ABS ERROR (M3/S)	REL ERROR (%)
1	204.5	2404.5	2200.0	204.5	9.30
2	393.6	2593.6	2230.0	363.6	16.30
3	1294.3	3494.3	2800.0	694.3	24.80
4	1652.0	3852.0	3260.0	592.0	18.16
5	1842.9	4042.9	4200.0	-157.1	-3.74
6	2110.2	4310.2	5270.0	-959.8	-18.21
7	2212.9	4412.9	5380.0	-967.1	-17.98
8	2069.3	4269.3	4900.0	-630.7	-12.87
9	1826.0	4026.0	4320.0	-294.0	-6.81
10	1570.9	3770.9	3730.0	40.9	1.10
11	1299.1	3499.1	3000.0	499.1	16.64
12	1071.6	3271.6	2800.0	471.6	16.83
13	883.3	3083.3	2270.0	813.3	35.83

RUNOFF VOLUMES IN UNITS OF RATE * TIME: COMPUTED= 18431
RECORDED= 17760.

THE AVERAGE RELATIVE ERROR,AVERR= 15.27%
THE COEFFICIENT OF DETERMINATION,R SQUARED=0.854

THE SUM OF THE DEPARTURES SQUARED= 4506658.
THE COEFFICIENT OF EFFICIENCY,COE=0.978

OCT 1957

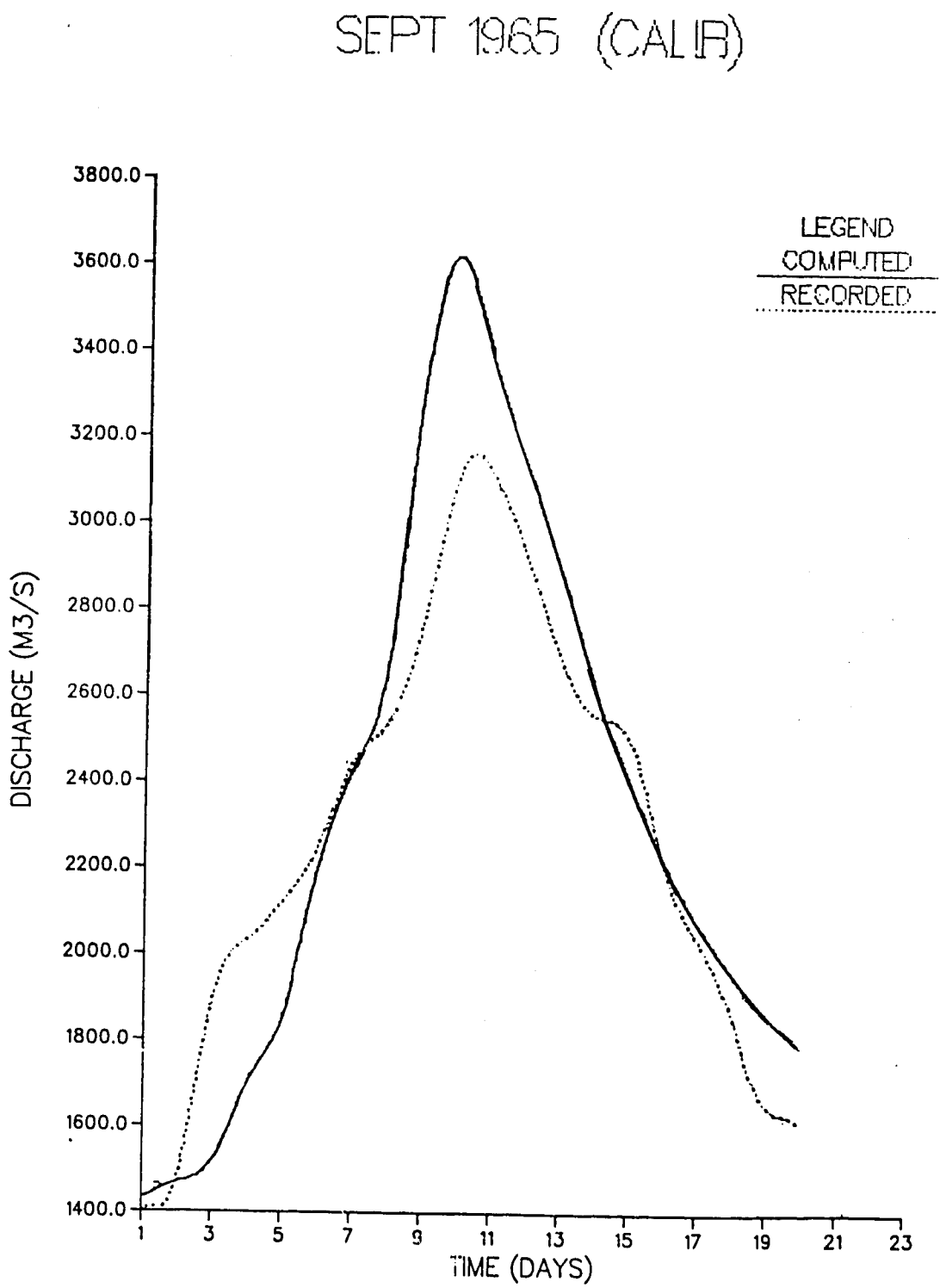
RUNOFF HYDROGRAPH FROM THE BASIN

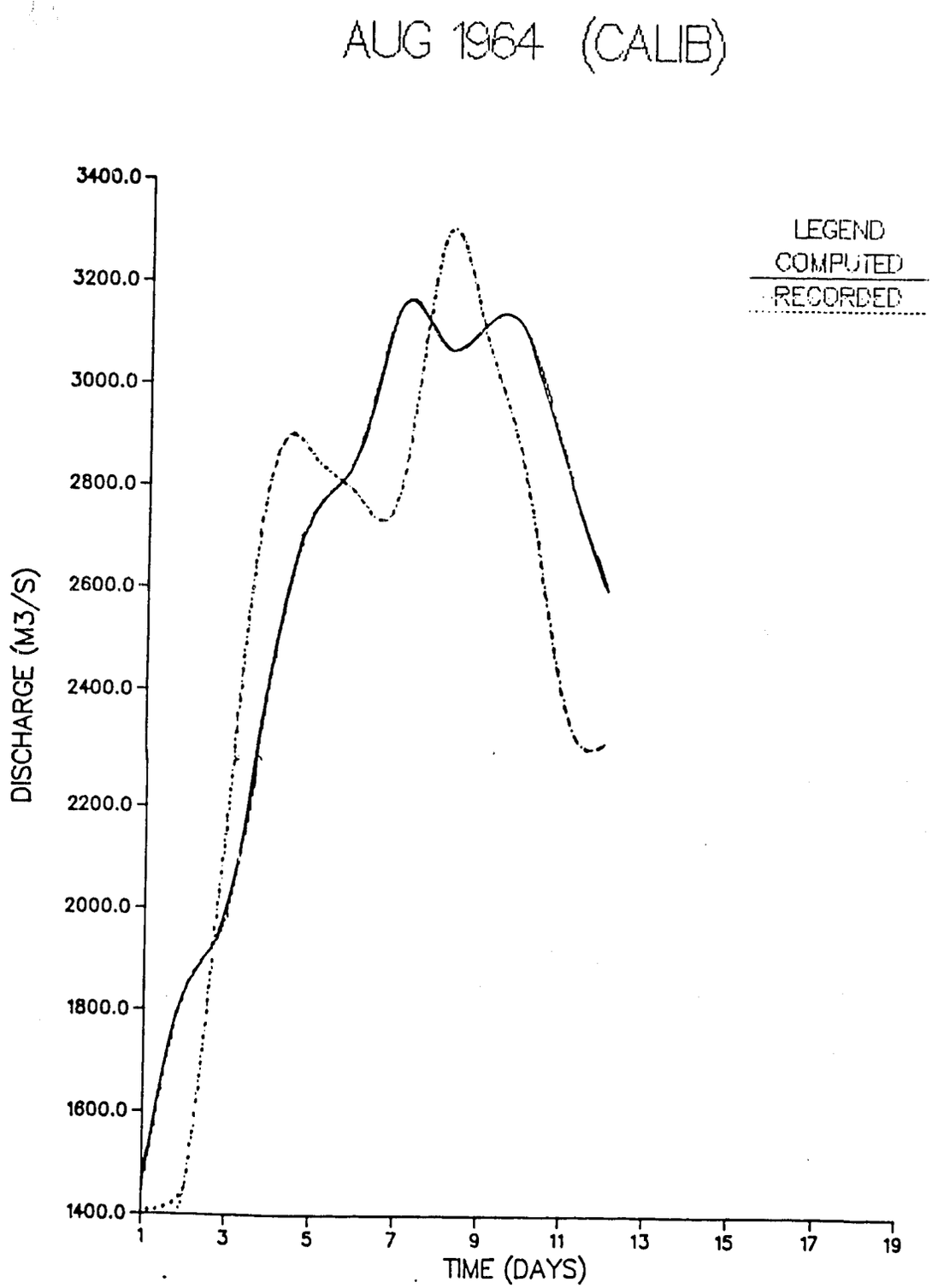
TIME (DAYS)	COMPUTED Q (M3/S)	TOTAL Q (M3/S)	RECORDED Q (M3/S)	ABS ERROR (M3/S)	REL ERROR (%)
1	32.8	1562.8	1530.0	32.8	2.15
2	83.1	1613.1	1570.0	43.1	2.75
3	120.0	1650.0	1610.0	40.0	2.48
4	302.1	1832.1	1750.0	82.1	4.69
5	607.3	2137.3	1880.0	257.3	13.69
6	770.8	2300.8	1910.0	390.8	20.46
7	894.7	2424.7	2110.0	314.7	14.91
8	914.8	2444.8	2920.0	-475.2	-16.28
9	1046.0	2576.0	2780.0	-204.0	-7.34
10	1141.0	2671.0	2790.0	-119.0	-4.27
11	1209.6	2739.6	2940.0	-200.4	-6.82
12	1304.2	2834.2	2900.0	-65.8	-2.27
13	1283.6	2813.6	2800.0	13.6	0.49
14	1159.3	2689.3	2530.0	159.3	6.72
15	977.0	2507.0	2260.0	247.0	10.93
16	807.1	2337.1	2100.0	237.1	11.29
17	662.8	2192.8	1920.0	272.8	14.21
18	543.5	2073.5	1900.0	173.5	9.13
19	445.5	1975.5	1860.0	115.5	6.21

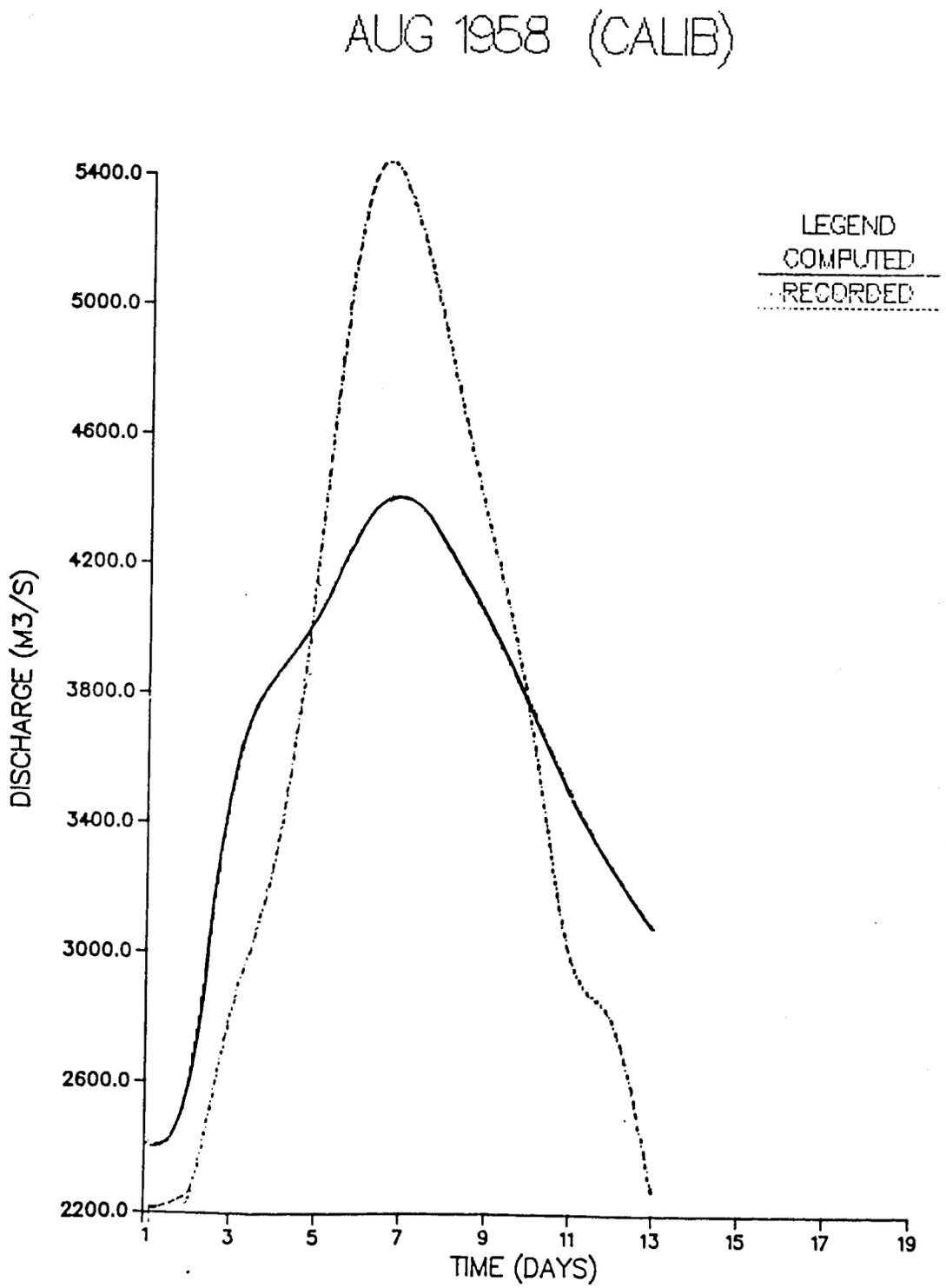
RUNOFF VOLUMES IN UNITS OF RATE * TIME: COMPUTED= 11305.
RECORDED= 12980.

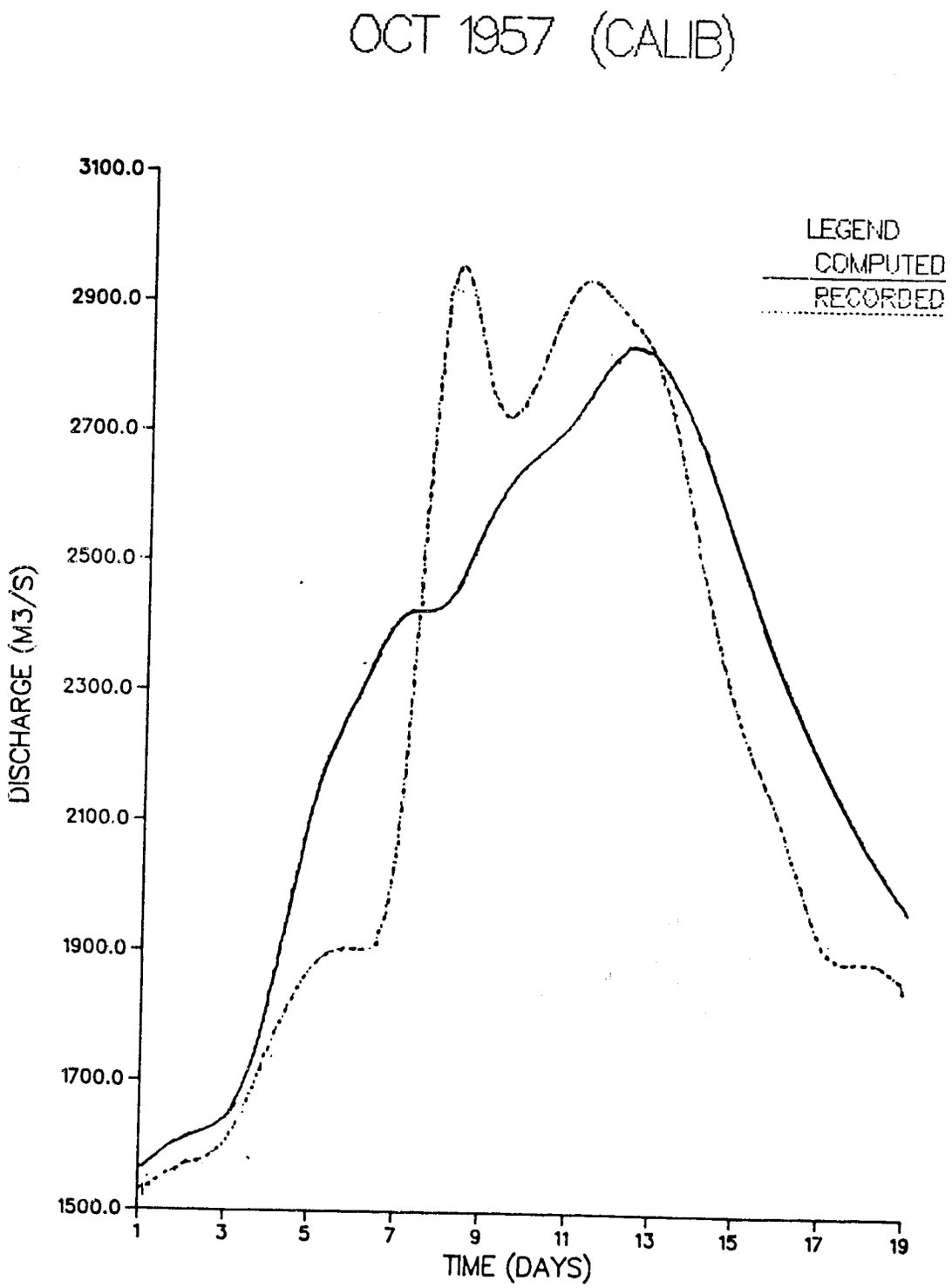
THE AVERAGE RELATIVE ERROR,AVERR= 8.27%
THE COEFFICIENT OF DETERMINATION,R SQUARED=0.990

THE SUM OF THE DEPARTURES SQUARED= 91935.
THE COEFFICIENT OF EFFICIENCY,COE=0.989


SEPT 1965 (CALIB)
LEGEND
COMPUTED
RECORDED
DISCHARGE (M3/S)
3800.0
3600.0
3400.0
3200.0
3000.0
2800.0
2600.0
2400.0
2200.0
2000.0
1800.0
1600.0
1400.0
1
3
5
7
9
11
13
15
17
19
21
23
TIME (DAYS)


AUG 1964 (CALIB)
LEGEND
COMPUTED
RECORDED
DISCHARGE (M3/S)
3400.0
3200.0
3000.0
2800.0
2600.0
2400.0
2200.0
2000.0
1800.0
1600.0
1400.0
1
3
5
7
9
11
13
15
17
19
TIME (DAYS)


AUG 1958 (CALIB)
LEGEND
COMPUTED
RECORDED
DISCHARGE (M3/S)
5400.0
5000.0
4600.0
4200.0
3800.0
3400.0
3000.0
2600.0
2200.0
1
3
5
7
9
11
13
15
17
19
TIME (DAYS)


OCT 1957 (CALIB)
LEGEND
COMPUTED
RECORDED
DISCHARGE (M3/S)
3100.0
2900.0
2700.0
2500.0
2300.0
2100.0
1900.0
1700.0
1500.0
1
3
5
7
9
11
13
15
17
19
TIME (DAYS)