RAINFALL RUNOFF MODELING OF RAMGANGA CATCHMENT

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

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WATER RESOURCES DEVELOPMENT

(CIVIL)

By

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CANDIDATE'S DECLARATION

I hereby certify that the work, which is being presented in this dissertation entitled "RAINFALL RUNOFF MODELING OF RAMGANGA CATCHMENT" in partial fulfilment of the requirement for the award of the degree of Master of Technology in " WATER RESOURCES DEVELOPMENT (CIVIL)" submitted to the DEPARTMENT OF WATER RESOURCES DEVELOPMENT AND MANAGEMENT, Indian Institute of Technology Roorkee, Roorkee, is an authentic record of my own work carried out during the period from Aug 2007 to June 2008 under the supervision of **Dr. S.K. Mishra,** Associate Professor, Department of Water Resources Development and Management, Indian Institute of Technology Roorkee, Roorkee.

I have not submitted the matter embodied in this dissertation for the award of my any other degree.

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Place: ROORKEE

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This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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Place: Roorkee
Date: …..June, 2008

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LIST IF SYMBOLS

A	area of the catchment in square kilometres
AMC	Antecedent moisture condition
AMC I	AMC corresponding to dry condition
AMC II	AMC corresponding to normal condition
AMC III	AMC corresponding to wet condition
AVP	average annual rainfall
AVQ	average annual runoff
b_c	depletion coefficient
BFI_{max}	maximum value of the baseflow index
CN	Curve number
CN_d	Curve number for drainage flow
CN_I	curve number corresponding to AMC I
CN_{II}	curve number corresponding to AMC II
CN_{III}	curve number corresponding to AMC III
COUR	courant number
D	number of days between the storm crest and the end of quickflow
DP	average daily depletion
ΔS_m	soil moisture storage
E	pan evaporation based on field data
E_t	average monthly lake evaporation for day t
F	infiltration depth
F_d	actual retention after flow begins
I_a	initial abstraction
I_d	initial abstractions for subsurface drainage flow
IV	initial variance
K	storage coefficient
K_b	base flow storage coefficient
M	soil moisture index at the beginning of the first storm
Mt	soil moisture index at any time t
n	total number of observations

P	rainfall for the first storm
PANC	pan coefficient
P_e	effective rainfall
Q	runoff for the first storm
Q_{comp}	simulated runoff
Q_d	drainage flow depth
Qi	computed runoff for ith day
Q_{obs}	observed runoff
Qs	surface runoff
RE	Relative error (%)
RO	direct surface runoff
RV	remaining variance
S	potential maximum retention
SCS-CN	Soil Conservation Service-Curve Number
S_d	potential maximum retention of watershed
T	number of days between the storms
λ	initial abstraction coefficient

ABSTRACT

Information regarding flow rates at any point of interest along a stream is necessary in the analysis and design of many types of water resource projects. Although many streams have been gauged to provide continuous records of stream flow, planners and engineers are some times faced with little or no available stream flow information and must rely on synthesis and simulation as tools to generate artificial flow sequences for use in rationalizing decisions regarding structure size, the effect of land use, flood control measures, water supplies, and the effect of natural or induced watershed or climatic change. Simulation is defined as the mathematical description of the response of a hydrologic water resource system to a series of events during a selected time period. For example, simulation can mean calculating daily, monthly, or seasonal stream flow based on rainfall or computing the discharge hydrograph resulting from a known or hypothetical storm, or simply filling in the missing values in a stream flow record.

The importance of rainfall–runoff modeling in planning and management of water resources systems can hardly be overemphasized. The long- term daily rainfall – runoff models are primarily developed for determination of continuous daily flow series from the available precipitation and other meteorological data for augmentation of record for use in water availability analysis useful in planning and management of water resources projects. Since the rainfall data is generally abundant compared to runoff data, it is necessary to develop models which convert rainfall to runoff. Water yield is generally estimated from rainfall-runoff relationships. Thus rainfall-runoff (R-R) relation is a conceptual simplification of the systematic hydrologic cycle and it is major complex processes in the hydrologic cycle. Many researchers have developed rainfall-runoff models for gauged and ungauged catchments separately that could accurately predict runoff hydrographs, peak flow rates, and times to peak. As every model has got its own limitations, the selection of appropriate method for predicting runoff from a catchment is difficult and most often there is a scope for further improvement. Hence, evaluation, refinements and modifications related to rainfall-runoff modeling remain classical.

The Soil Conservation Service Curve Number (SCS – CN) method is widely used for computation of direct surface runoff from given storm rainfall. In this study this method is first explored for its potential and limitations and then used for long term daily flow simulation. When the method is expressed as an infiltration equation, the infiltration rate becomes dependent on both total storm rainfall and rainfall intensity. When expressed as a spatially varied saturation overland flow model, the method implies that some part of any catchment has infinite surface storage capacity. SCS curve numbers are used to estimate the amount of precipitation which becomes runoff, and the amount which infiltrates into the soil. The concept behind the SCS – CN method can be applied to determination of surface drainage flow from rainfall. When accumulated subsurface drainage flow is plotted against accumulated infiltration, subsurface drainage flow starts after some infiltration has accumulated and the relationship becomes asymptotic to a line of 45° slope, similar to the SCS rainfall – runoff relationship. Extending the basic proportionality concept of the SCS – CN method, an SCS – CN based procedure is suggested for determination of baseflow and it is employed in long term daily flow simulation.

For daily flow simulation, a simple 4 parameter SCS – CN based daily flow simulation model is proposed and applied to the 5 years daily data of rainfall, runoff and evaporation, of Ramganga catchment. To check the versatility of the proposed model, this also tested on the data of catchments from different climatic regions of India and the results are analyzed. The simulation results were compared with the results obtained from the application of an existing 6- parameter, SCS – CN based hydrologic model and the proposed model is found to perform better on the high runoff producing watersheds, such as the Ramganga catchment, than the less runoff producing watersheds, such as Manot, Hridayanagar, Ghodahado watersheds.

CHAPTER I: INTRODUCTION

1.1 GENERAL

Rainfall-runoff modeling is now-a-days a dynamically developing field of Hydrology and Water Management. This development is primarily caused by the rapid progress of computers and information technology. This evolution provides the mankind with new possibilities to use water as its basic need and at the same time to evolve an affective protection against it. Rainfall runoff modeling is meant to model the hydrological processes of the land phase of the hydrological cycle which inputs the rainfall and other hydrologic, climatic and basin parameters and produces the desired output such as runoff, peak discharge etc. In other words, a rainfall - runoff model is a hydrological model which determines the runoff from the rainfall. Obviously hydrological processes are complex phenomena and certain degree of simplification is always involved in modeling. For estimation of runoff, a number of models varying from the simplest empirical relations to the most complex physically based models are available in literature. Since the rainfall data are generally available for a much longer period than the stream flow data, long – term hydrologic simulation helps to extend the gauged data required for the applications in water resources planning and watershed management. Much of the current research in catchment hydrology as well as practical management of water resources is based on computer models for estimating runoff from rainfall and evaporation data. Most of the modern rainfall–runoff models that now number in thousands will give reliable results where some stream flow data are available for calibration of model parameters. However, very little progress has been made in use of these models on ungauged catchments where calibration data are not available.

The response of the catchment for a particular rainfall event is runoff. Stream flow representing the runoff phase of the hydrologic cycle is the most important data for hydrologic studies. The first and foremost requisite for the planning of water resources development is accurate data of stream flow, or in other words, the surface runoff for a considerable period of time to determine the extent and pattern of the available supply of water. The usual practical objective of a hydrologic analysis is to determine the characteristics of the hydrograph that may be expected for a stream draining any particular watershed. Surface runoff is that part of the precipitation, which, during and immediately following a storm event, ultimately appears as flowing water in the drainage network of a watershed. Such flow may result from direct movement of water over the ground surface, precipitation in excess of abstraction demands, or it may result from emergence of soil water into drainage ways.

The long-term daily hydrologic simulation is useful in augmentation of hydrologic data, water resources planning and watershed management (Mishra and Singh, 2003, 2004) and is efficacious in describing the performance of a water resource system under climatic variations of rainfall and other aspects (Kottegoda et al., 2000). The computer-based lumped, conceptual rainfall–runoff models have been widely applied in hydrological modelling since they were first introduced in the late 1960s and early 1970s. Among a multitude of models, a few well known and some recent storage concept-based models worth citing are: Stanford Watershed Model IV (SWM IV) (Franchini and Pacciani, 1991; Singh, 1989), Boughton model (Johnston and Pilgrim, 1976; Mein and Brown, 1978), Kentucky Watershed model (Moore et al., 1983; James, 1972), Institute of Hydrology model (Nash and Sutcliffe, 1970), HYDROLOG (Poter and McMahon 1976), MODHYDROLOG model (Chiew et al., 1993), and Hydrology and River Hydraulics at University of Tokushima (HRUT) model (Yao et al., 1996). Using the storage concept, the Soil Conservation Service Curve Number (SCS-CN) model has also been widely employed in the past for long-term hydrologic simulation (Mishra et al., 1998; Mishra and Singh, 2003, 2004).

Estimation of runoff from a particular rainfall event is of vital significance in planning for irrigation, hydropower, flood control, water supply and navigation. In general rainfall- runoff modeling is basic to design of a wide variety of hydraulic structures, environmental impact assessment, evaluation of the impact of climatic change, irrigation scheduling, flood forecasting, planning of tactical military operation, augmentation of runoff records, pollution abatement, watershed management & so on.

1.2 OBJECTIVE OF THE STUDY

As runoff data are missing or only available during short periods, they can be generated using rainfall – runoff relationship or long- term hydrologic simulation models. This analysis however considers the model application to the catchment as a whole. The primary objective of the study is to conceptualize and develop a lumped model based on the SCS – CN technique for long daily rainfall- runoff simulation model and test its workability using the data of Ramganga catchment and further verify its applicability to other catchments located in different geo-hydro-meteorological settings. The study also compares the model performance with another lumped conceptual model (K. Geetha, 2007) on different watersheds.

1.3 SCOPE OF THE STUDY

The study is organized as follows:

Chapter 2 provides a brief review of literature available on various rainfall – runoff simulation methods, historical background, and other details relevant to the study.

Chapter 3 contains the theory of the SCS- CN method, which has been used in model development in the present study and the proposed model.

Chapter 4 provides a brief description of the watersheds and the data available for the study.

Chapter 5 provides a discussion of the results of model calibration and validation and its comparison with the existing model.

Chapter 6 summarizes and concludes the study.

The developed computer program is incorporated at the end of dissertation work (Appendix – I).

CHAPTER II: REVIEW OF LITERATURE

The simulation of rainfall-generated runoff is very important in various activities of water resources development and management such as flood control and its management, irrigation scheduling, design of irrigation and drainage works, design of hydraulic structures, and hydro-power generation etc. Ironically, determining a robust relationship between rainfall and runoff for a watershed has been one of the most important problems for hydrologists, engineers, and agriculturists since its first documentation by P. Perrault (In: Mishra and Singh 2003) about 330 years ago. The process of transformation of rainfall to runoff is highly complex, dynamic, non-linear, and exhibits temporal and spatial variability, further affected by many and often interrelated physical factors. However an understanding of various hydrologic variations (spatial and temporal) over long periods is necessary for identification of these complex and heterogeneous watershed characteristics.

2.1 RAINFALL – RUNOFF MODELING

Rainfall – runoff modelling is meant to model the hydrological processes of the land phase of the hydrological cycle which input the rainfall and other hydrologic, climatic and basin parameters and produces the desired output such as runoff, peak discharge etc.

The hydrological cycle is a continuous process in which water circulates from the oceans through the atmosphere and rivers back to the oceans. Among the various components of hydrological cycle, the term precipitation denotes all forms of water that reach the earth from the atmosphere. Rain (precipitation) is the major object of hydrologic cycle and this is the primary cause of runoff. The rainfall is subjected to the physical processes which depend on climatological factors like temperature, humidity, wind velocity, cloud cover, evaporation and evapotranspiration, topographical features like depressions, slope of the catchments, vegetation and land use pattern, the soil characteristics like permeability, antecedent moisture content and irrigability characteristics; and the hydrological condition like rock formation, elevation of water table and sub-surface channels too affect this process considerably. Runoff is defined as the portion of the precipitation that makes its way towards river or ocean etc. as surface and subsurface flow. Runoff, representing the response of a catchment to precipitation, reflects the integrated effect of a catchment, climate & precipitation characteristics. Under these influencing parameters, it is utmost difficult task to estimate the likely runoff from a particular storm. Precipitation (rain) falling on the land surface has several pathways as shown in Figure 2. 1.

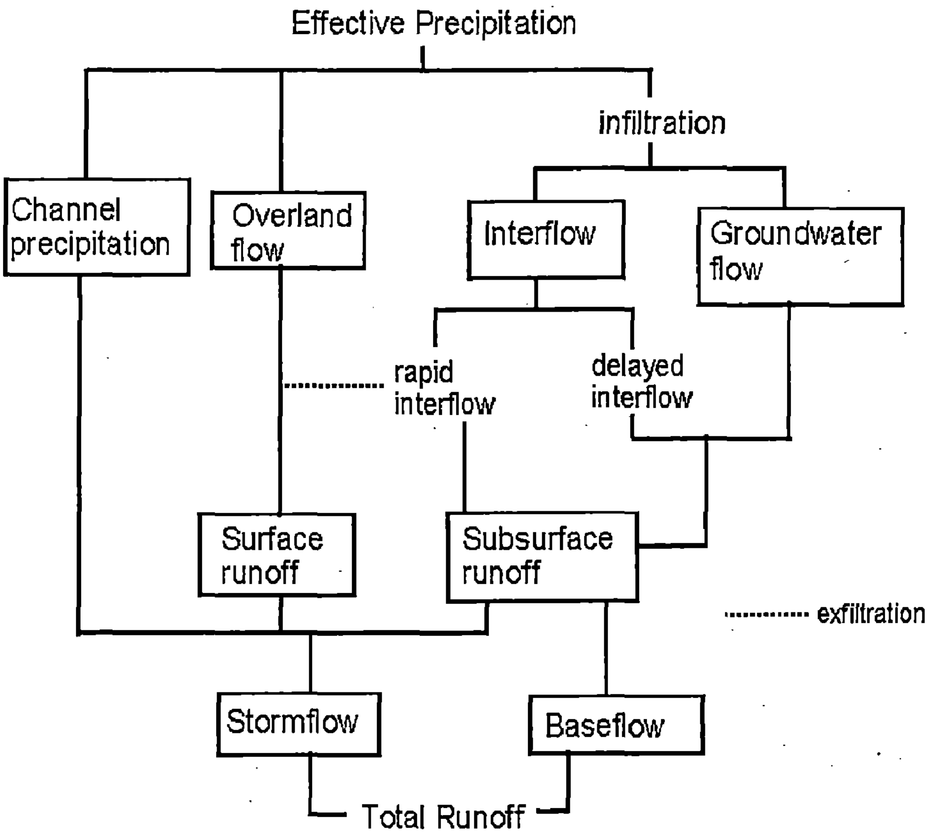


Fig : 2.1 Generation of runoff from effective rainfall in a catchment (source :- www.cartage.org.lb/.....sourcesofrunoff.htm)

The precipitation responsible for the runoff is known as effective precipitation. For a given precipitation the evapotranspiration, initial loss, infiltration and detention storage requirements will have to be first satisfied before the commencement of runoff. When these are satisfied the excess precipitation moves over the land surface to reach smaller channels. The portion of the runoff is called as overland flow and involves building up of storage over the surface and draining the same. Flows from several small channels join bigger channels and flows from there and, in turn, combine to form a large stream and so on till the flow reaches the catchment's outlet. The flow in this mode where it travels all the time over the surface as overland flow and through the channels as open channel flow and reaches the catchment's outlet is called surface runoff. A part of precipitation that infiltrates moves laterally through upper crust of the soil and returns to the surface at some location away from the point of entry into the soil. This component of the runoff is known as interflow. The amount of interflow depends on the geological condition of the soil. Depending upon the time delay between the infiltration and outflow, the interflow is sometimes classified into prompt interflow or rapid interflow i.e. the interflow with the least time lag and delayed interflow. Another route for the infiltrated water is to undergo deep percolation and reach the ground water storage in the soil. The time lag i.e. difference in time between the entry into the soil and outflow from it is very large, being of the order of months and years. This part of runoff is called groundwater runoff or groundwater flow.

Based on the time delay between the precipitation and the runoff, runoff is classified into two categories as direct runoff or storm runoff and base flow. Direct runoff is the part of runoff which enters the stream immediately after the precipitation. It includes surface runoff, prompt interflows and precipitation on channel surface. The delayed flow that reaches stream essentially as groundwater flow is called as base flow.Rainfall-runoff models may be grouped into two general classifications that are illustrated in Figures 2.2 and 2.3. The first approach uses the concept of effective rainfall in which a loss model is assumed which divides the rainfall intensity into losses and an effective rainfall hyetograph. The effective rainfall is then used as input to a catchment model to produce the runoff hydrograph. It follows from this approach that the infiltration process ceases at the end of the storm duration.

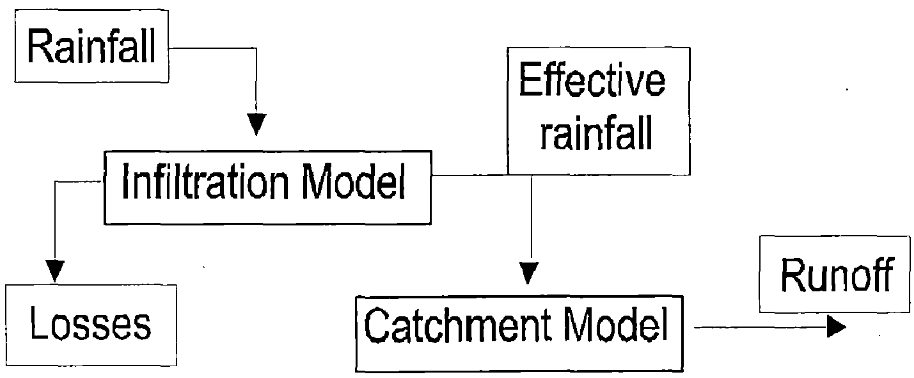


Fig : 2.2 A rainfall-runoff model using effective rainfall

(source :- www.alanasmith.com/theory- calculating../ runoff models. htm)

An alternative approach that might be termed as surface water budget model incorporates the loss mechanism into the catchment model. In this way, the incident rainfall hyetograph is used as input and the estimation of infiltration and other losses is made as an integral part of the calculation of runoff. This approach implies that infiltration will continue to occur as long as the average depth of excess water on the surface is finite. Clearly, this may continue after the cessation of rainfall.

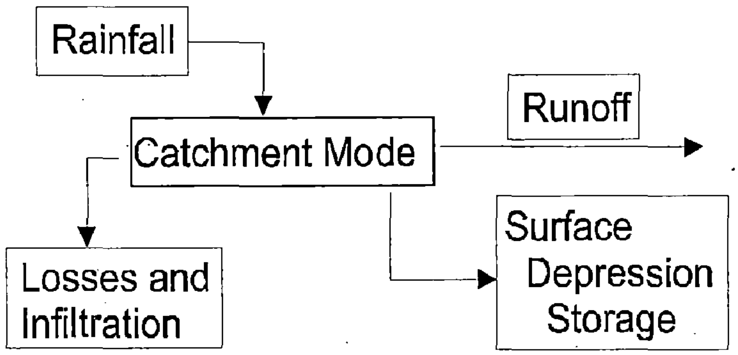


Fig: 2.3 A rainfall-runoff model using a surface water budget

(source :- www.alanasmith.com/theory- calculating../ runoff models. htm)

The origin of rainfall- runoff modelling, widely used for flow simulation, can be found in the second half of the 19th century when engineers faced the problems of urban drainage and river training networks. During the last part of 19th century and early part of 20th century, the empirical formulae were in wide use (Dooge, 1957, 1973). The approaches were mainly confined to small and mountainous watersheds. Later attempts were mainly confined to their application to larger catchments. In 1930's the popular unit hydrograph techniques were developed. With the advent of computers in 1950's, sophistication to models through mathematical jugglery was introduced with the objective of providing the generality of available approaches. The subsequent era saw the development of a number of models and evoked the problem of classification.

The relation between precipitation (rainfall) and runoff is influenced by various storm and basin characteristics. Because of the complexities and frequent paucity of adequate runoff data, many approximate formulae have been developed to relate runoff with rainfall. The earliest of these were usually crude empirical statements, whereas the trend now is to develop descriptive equations based on physical processes.

2.2 CLASSIFICATION OF HYDROLOGICAL MODELS

The simulation of rainfall-runoff (R-R) relationships has been an unavoidable issue of hydrological research for several decades and has resulted in plenty of models proposed in literature. In recent decades the science of computer simulation of groundwater and surface water resources systems has passed from scattered academic interest to a practical engineering procedure. A few of the most descriptive classifications are presented. The available hydrological models can be broadly classified into Deterministic vs. Stochastic / Probabilistic, Conceptual vs. Physically Based Models, Lumped Models vs. Spatial Distributed Models, a brief description of which is provided as follows:

Deterministic vs. Stochastic / Probabilistic models

Water balance models can be referred to as "deterministic" if the statistical properties of input and output parameters are not considered. On the other hand, probabilistic models include random variations in input parameters, whereby known probability distributions are used to determine statistical probabilities of output parameters; i.e deterministic models permit only one outcome from a simulation with one set of input and parameter values. Stochastic models allow for some randomness or uncertainty in the possible outcomes due to uncertainty in input variables.

Conceptual vs. Physically Based Models

Conceptual models rely primarily on empirical relationships between input and output parameters. These are based on overall observations of system behaviour (sometimes called "black box" models). The modeling systems may or may not have clearly defined physical, chemical or hydraulic relationships. Physically based models seek to describe water movement based on physical laws and principles. This may result in more reliable descriptions of water balance relationships. This type of model demands appropriate data for input and requires documentation of processes and assumptions.

Lumped Models vs. Spatially Distributed Models

Lumped models treat a subwatershed as a single system and use the basin-wide averaged data as input parameters. This method assumes that the hydrologic characteristics of subwatersheds are homogeneous. A spatially distributed model accounts for variations in water budget characteristics. Various methods are available, such as division of the watershed into grid cells or use of Hydrological Similar Units

(HSU). For example, a grid cell model uses data for each grid cell inside the basin to compute flow from cell to cell. By this method, the spatial variation in hydrologic characteristics can be handled individually (i.e. assuming homogeneity for each cell), and therefore, may be a more appropriate treatment. Spatially distributed models are suitable for GIS applications.

In this study a simple, lumped, conceptual, and empirical Soil Conservation Service Curve Number (SCS – CN) method, a detailed description of which is provided in the forthcoming chapter, is used for long term hydrological simulation, a brief review of such studies is in order.

2.3 LONG TERM HYDROLOGICAL SIMULATION:

Long-term hydrological simulation is required for augmentation of hydrological data. It is useful for water resources planning and watershed management. Long-term daily flow data are specifically needed for analysis of water availability, computation of fortnightly or monthly flows for reservoir operation and drought analysis. As the rainfall data are generally available for much longer periods than the stream-flow data, long-term hydrological simulation helps extend the gauged stream-flow data required for the applications.

There exist a multitude of models for hydrological simulation. In 1991, the U. S. Bureau of Reclamation prepared an inventory of 64 watershed models into four categories and the inventory is currently being updated. Burton (1993) compiled Proceedings of the Federal Interagency Workshop on Hydrologic Modeling Demands for the 1990's, which contains several important watershed hydrology models. Singh (1995) edited a book that summarized 26 popular models from around the globe. The subcommittee on hydrology of the Interagency Advisory Committee on Water Data (1998) published Proceedings of the First Federal Interagency Hydrologic Modeling Conference, which contains many popular watershed hydrology models developed by federal agencies in the USA. Wurbs (1998) listed a number of generalized water resources simulation models in seven categories and discussed their dissemination.

The hydrological models vary in description of the components of the hydrological cycle, degree of complexity of inputs, number of parameters to be determined, time interval used in simulation, error and risk analyses, and output generated. Most of the models, such as the Hydrologic Simulation Package Fortran (HSPF), USDAHL (Holtan and Lopez, 1971) and its variants, System Hydrologic Europeen (SHE) (Abbott *et al.*, 1986a, b), Hydrologic Engineering Centre Hydrologic Modeling System (HEC-HMS) (HEC, 2000), etc., have a number of parameters, usually use a short time interval, produce hydrographs as well as water yield and provide continuous simulation. The HSPF and SHE models are not applicable to ungauged watersheds for the reason that their application requires apriori calibration with measured runoff data for each watershed. The USDAHL model can, however, be used for ungauged watersheds, but the prediction accuracy is not commensurate with the input detail. These models are better suited for detailed scientific, hydrologic studies. Holtan and Lopez (1971) found the USDAHL model to explain about 90% of the variation in the monthly runoff for four watersheds up to 40 sq. km. The Haan (1975) model has four parameters, uses a 1- d time interval (except for a 1- d interval is used during rains), has simple inputs, and only outputs the runoff volume. In testing this model was reported to explain about 80% of the variation in the monthly runoff from 46 watersheds of generally less than 100 sq. km. However, no provision exists for estimating the parameters of this model for its employment to ungauged watersheds. Woodward and Gburek, (1992) compared some of the available models and found them widely varying in their degree of success.

Despite their comprehensive structure, many of these models have not yet become standard tools in hydrological practice in developing countries, such as India, Pakistan, Nepal, and other countries of Asia as well as African countries. The reason is twofold. First, most basins in these countries are ungauged and there is little hydrological data available. Second, these models contain too many parameters, which are difficult to estimate in practice and vary from basin to basin. Although some of these models have been applied to ungauged basins, the fact is that they are not easy for practical applications. Furthermore, when these models are compared on the same basin, they are found widely varying in their performance (Woodward and Gburek, 1992). Thus, what is needed in developing countries is simple models which can provide reasonable simulations and need few data. The Soil Conservation Service Curve Number (SCS-CN) based simulation models do satisfy these criteria.

The SCS-CN method is an infiltration loss model and, therefore, its applicability is supposedly restricted to modelling storms (Ponce and Hawkins, 1996). Notably, the SCS – CN method is theoretically applicable to any watershed of any size as long as the measured runoff corresponds to the observed rainfall amount (Mishra and Singh, 2003). However, some restrictions regarding its application to watershed of less than 250 sq. km, for practical reasons, have been reported in literature (for example Ponce and Hawkins, 1996). Using theoretical arguments, it is possible to apply the SCS-CN method for long-term hydrological simulation to any basin. It is for this reason that the SCS – CN method computes the rainfall – excess that equals the direct surface runoff. In large watersheds, routing plays an important role in converting the rainfall-excess to the surface runoff hydrograph produced at the basin outlet. On the other hand, small watersheds require minimal routing in long-term hydrological simulation utilizing a time interval of 1 day or larger. Consequently, the SCS-CN method has been used on small basins for long-term hydrological simulation and several models have been developed in the past two decades. The models of Williams and LaSeur (1976), Huber *et al.* (1976), Hawkins (1978), Knisel (1980), Soni and Mishra (1985) and Mishra *et al.* (1998) are notable.

2.3.1 Williams–LaSeur (1976) model

Williams and LaSeur (1976) proposed a model based on the existing SCS – CN method which is based on water balance equation and two fundamental hypotheses (methodology). The SCS – CN parameter potential maximum retention S is linked with the soil moisture (M) as blow:

$$M = S_{abs} - S \tag{2.1}$$

where S_{abs} is the absolute potential maximum retention equal to 20 inches. M is depleted continuously between storms by evapotranspiration and deep storage. Depletion is high when soil moisture and lake evaporation is high, the most rapid immediately after a storm (high M). M is assumed to vary with the lake evaporation as

$$\frac{d(M)}{dt} = -b_c M^2 E \tag{2.2}$$

where t is the time, b_c is the depletion coefficient, and E is the lake evaporation. Equation (2.2) represents a second- order process. The lake evaporation is used as a climatic index. According to Williams and LaSeur, equation (2.2) works well for the average monthly values for runoff predictions. They found their model to perform poorly when used daily pan evaporation and temperature as climatic indices. From equation (2.2) M is solved as

$$M_t = \frac{M}{1.0 + b_c M \sum_{t=1}^{T} E_t} \tag{2.3}$$

where M is the soil moisture index at the beginning of the first storm, M_t is the soil moisture index at any time t, E_t is the average monthly lake evaporation for day t, and T is the number of days between the storms.

For model operation, the amount of water infiltrated during a rainstorm (= rainfall P – direct surface runoff Q) is added to the soil moisture. The rainfall of the first day of the T – day period is added to M before equation (2.3) is solved. However, runoff is not abstracted from rainfall until the end of the T – day period, for the reason that runoff lags rainfall and may be subjected to depletion for several days on large watersheds. Thus equation (2.3) is modified for rainfall P as

$$M_t = \frac{M + P}{1.0 + b_c M \sum_{t=1}^{T} E_t} - Q \tag{2.4}$$

where P and Q are, respectively, the rainfall and runoff for the first storm. The retention parameter S is computed from equation $S = S_{abs} - M$ for $S_{abs} = 20$ inches for computing runoff for the second storm using the popular form of the existing SCS – CN method, expressible as

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \tag{2.5}$$

The procedure is repeated for each storm in the rainfall series. Thus, the Williams and LaSeur model can also be applied to the pre- identified rainstorms other than 1 day. The model is calibrated with data from a gauged watershed by adjusting the depletion coefficient, b_c, until the predicted average annual runoff matches closely with the measured average annual runoff. The initial estimation of b_c is derived from the average annual rainfall and runoff values as

$$DP = \frac{AVP - AVQ}{365} \tag{2.6}$$

where, DP is the average daily depletion, AVP is the average annual rainfall, and AVQ is the average annual runoff. The value of b_c can be computed from equation (2.4) assuming that (a) T = 1; (b) M is the average soil moisture index, M_A ; (c) E is the average lake evaporation; and (d) P= Q=0 for the day. For this situation, equation (2.4) can be recast as

$$M_t = \frac{M_A}{1.0 + b_c M_A E_t} \tag{2.7}$$

In which, M_A is computed from equation $S = \frac{1000}{CN} - 10$ and $S = S_{abs} - M$ for CN corresponding to AMC II. The average daily depletion computed from equation (2.6) is set equal to the change in soil moisture for 1 day as

$$DP = M_A - M_t \tag{2.8}$$

Combining equation (2.7) and (2.8), one obtains

$$DP = M_A - \frac{M_A}{1.0 + b_c M_A E_t} \tag{2.9}$$

From which b_c can be derived as

$$b_c = \frac{-DP}{E_t M_A (DP - M_A)} \tag{2.10}$$

The simulation begins 1 year before the actual calibration period because of apriori determination of the initial soil moisture index. At the end of one year, the soil moisture is taken to represent the actual soil moisture conditions. Here the initial estimate of M is M_A.

In brief, the Williams–LaSeur model has one parameter, uses a 1-day or any other pre-determined time interval, has simple inputs and only outputs the runoff volume. It eliminates, to certain extent, sudden jumps in the CN values when changing from one AMC to the other. Its operation requires (i) an estimate of the AMC-II curve number, (ii) measured monthly runoff, (iii) daily rainfall and (iv) average monthly lake evaporation. The model-computed b_c forces an agreement between the measured and the predicted average annual runoff. The model can be applied advantageously to nearby ungauged watersheds by adjusting the curve number for the ungauged watershed in proportion to the ratio of the AMC-II curve number to the average predicted curve number for the calibrated watershed.

The model, however, has its limitations. It utilizes an arbitrarily assigned value of 20 inches for S_{abs} and simulates runoff on monthly and annual bases although runoff is computed daily, treating rainfall of a day as a storm. Several adjustments for b_c loss the physical soundness of the model apart from the undesirable loss of 1-year rainfall–runoff information (Singh *et al.*, 2001). Owing to physically unrealizable decay of soil moisture with lake evaporation, the model contradicts the SCS-CN approach, as shown below.

Taking $S_{abs} = S_o = S$, which represents S at the beginning of a storm under fully dry conditions, equation $M = S_{abs} - S$ can be written for time t as: $M_t = S_0 - S_t$, if $S_t = 0$ at time $t = 0$, $M_t = S_o$. Its substitution into Equation (2.3) leads to

$$(S_0 - S_t)/S_0 = \frac{1}{(1 + b_c S_0 \overline{Et})} \qquad (2.11)$$

where E is the average rate of evapotranspiration. Here, $(S_0 - S_t) / S_0 = F/ S_0$, consistent with the description of Mishra (1998) and Mishra and Singh (2002a,b). With the assumption that $P/ S_0 = b_c S_0 E_t$ and $I_a = 0$ (here, P/t = uniform rainfall intensity $i_o = b_c S_0^2 E$), a substitution of these relationships into equation $P = I_a + F + Q$ yields $Q = PS_0/(S_0+P)$, which actually holds for F in the existing SCS-CN approach, rather than Q, and therefore, equation (2.3) is physically unrealizable.

2.3.2 Hawkins Model

Hawkins (1978) derived a daily simulation model by expressing Equation (2.5) as

$$Q = P - S\left(1.2 - \frac{S}{(P + 0.8S)}\right) \tag{2.12}$$

which is valid for $P \geq 0.2S$. It is evident from this equation that as $P \to \infty$ the maximum possible water is equal to S_t and it is computed as

$$S_t = 1.2\ S \tag{2.13}$$

which can be derived from equation $S_T = (1 + \lambda)S$, assuming $\lambda = 0.2$. Substitution of equation $S = \frac{1000}{CN} - 10$ for S into equation (2.13) yields a storage relation for any time t as

$$S_{T(t)} = 1.2S_t = 1.2\left(\frac{1000}{CN_t} - 10\right) \tag{2.14}$$

where subscript 't' represents the time level. Taking into account the evapotranspiration (ET), the maximum water loss at a higher time level $(t + \Delta t)$, where Δt is the storm duration, can be derived from the moisture balance as:

$$S_{T(t+\Delta t)} = S_{T(t)} + \lfloor ET - (P - Q)_{(t+\Delta t)} \rfloor \tag{2.15}$$

where the last term in the bracket corresponds to the Δt duration between time t and $(t+\Delta t)$, denoted by subscript $(t, t+\Delta t)$. Following the above argument, equation (2.15) can be alternatively written as

$$S_{T(t+\Delta t)} = 1.2\ S_{(t+\Delta t)} \tag{2.16}$$

Here it is noted that ET also intuitively accounts for the interim drainage, if any. Coupling of equation (2.15) with equation (2.16) and substitution of equation $S = \frac{1000}{CN} - 10$ into the resulting expression leads to

$$1.2\left(\frac{1000}{CN_t} - 10\right) + \left[ET - (P - Q)\right]_{(t+\Delta t)} = 1.2\left(\frac{1000}{CN_{t+\Delta t}} - 10\right) \tag{2.17}$$

which can be solved for $CN_{t+\Delta t}$ as

$$CN_{t+\Delta t} = \frac{1200}{\frac{1200}{CN_t} + [ET - (P - Q)]_{(t+\Delta t)}} \qquad (2.18)$$

Since ET, P, Q in equation (2.18) correspond to the time duration Δt and these are known quantities, Q can be computed from equation (2.5) for a given CN_t. Input of these values along with the known value of ET yields CN at time level $(t+\Delta t)$

It is apparent from the above that the Hawkins model accounts for the site moisture on a continuous basis using the volumetric concept. It is worth emphasizing here that the Hawkins model is analogous to a bottomless reservoir, implying that the reservoir never depletes fully or the reservoir is of infinite storage capacity. Such a description is, however, physically realizable in terms of $\psi - \theta$ relationship, according to which S is directly proportional to the average ψ which approaches infinity as $\theta \rightarrow 0$. Under the situation that the soil is fully saturated or $\theta \rightarrow \eta$ (soil porosity), $\psi \rightarrow 0$. Thus, similar to S, S_t will also vary from 0 to ∞. Following this argument, $S_{abs} = 20$ inches in the Williams–LaSeur model appears to be a forced assumption. While applying the Hawkins model, Soni and Mishra (1985) also employed a similar assumption by fixing the depth of the soil profile to the root zone depth of 1.2 m for computing S.

The advantage of the Hawkins model is that it also eliminates sudden quantum jumps in the CN values when changing from one AMC level to the other, similar to the Williams–LaSeur model. However, the Hawkins model also has the following limitations.

1. It does not distinguish the dynamic infiltration from the static one. The water drained down to meet the water table may not be available for evapotranspiration.
2. The interim drainage is coupled with the evapotranspiration intuitively.
3. According to the model formulation equation (2.21), the term $(I_a + S)$ takes part in the dynamic infiltration process, rather than the S alone, where I_a = initial abstraction. As the initially adsorbed water (= I_a) as a result of very high capillary suction is not available for transpiration, I_a does not play a part in the dynamic infiltration process.

4. The follow up of the above 3 leads to the assumption of the SCS-CN method to be based on the (I_a – S) scheme, whereas I_a is separate from S. It is noted that the Hawkins model considers the maximum F amount equal to (I_a +S).
5. Substitution of P = 0 in equation (2.12) yields Q = 0.05S, which is impossible. Although Equation P = I_a + F + Q, where P = Total rainfall, F = Actual infiltration, Q = Direct surface runoff is stated to be valid for P ≥ 0.2S, equation (2.12) carries its impacts by allowing an additional storage space of 20% of S available for water retention at every time level and, in turn, leads to unrealistic negative infiltration at P → 0. Thus, S at time t (= S_t) corresponds to CN at time t (= CN_t). Equation (2.12) therefore needs modification by substitution of 1000 for 1200.

2.3.3 Pandit and Gopalakrishnan (1996) model:

Pandit and Gopalakrishnan (1996) suggested a continuous simulation model for computing the annual runoff for determination of annual pollutant loads. This model is specifically useful for urban areas characterized primarily by the percentage imperviousness, and involves the following steps.

1. Determine the pervious curve number for AMC II.
2. Determine the directly connected impervious area of the urban watershed according to SCS (1956).
3. Estimate the daily runoff depth for both pervious and impervious areas separately using equation (2.5).
4. Determine the actual AMC based on the previous 5-day rainfall and modify CN as

$$CN_I = \frac{CN_{II}}{2.281 - 0.01281 CN_{II}}; r^2 = 0.996 \text{ and SE} = 1.0 \text{ CN} \quad (2.19a)$$

$$CN_{III} = \frac{CN_{II}}{0.427 - 0.00573 CN_{II}}; r^2 = 0.994 \text{ and SE} = 0.7 \text{ CN} \quad (2.19b)$$

CNs are modified such that these do not exceed 98. NEH – 4 identified three antecedent moisture conditions (AMC): AMC I, AMC II, AMC

III for dry, normal and wet conditions of the watershed, respectively. As shown in Fig (2.1), AMC I corresponds to the lower enveloping CN, and AMC III the upper enveloping CN. NEH- 4 provides conversion table from CN for AMC II to corresponding CNs for AMC I and AMC III.

5. Calculate the yearly storm runoff depth by summing the runoff for each day.

In summary, the method is very simple, allows sudden jumps in the CN values and ignores evapotranspiration, drainage contribution and watershed routing. Since routing is ignored, it is useful for small watersheds, where routing is minimal in daily runoff computation. This model is a specific form of the Mishra *et al.* (1998) model described subsequently.

2.3.4 Mishra et al. model:

The Mishra et al. (1998) model assumes CN variation with time t dependent on AMC (Ponce and Hawkins, 1996) only. The computed rainfall-excess Q (equation 2.5) is transformed to direct runoff amount DO_t using a linear regression approach, analogous to the unit hydrograph scheme. Taking base flow (O_b) as a fraction of F along with the time lag, the total daily flow, Q_t, is computed as the sum of DO_t and O_b. The model parameters are optimized utilizing the objective function of minimizing the errors between the computed and observed data.

The advantage of the Mishra et al. (1998) model is that it allows the transformation of rainfall-excess to direct runoff and takes into account the base flow, enabling its application to even large basins. The model, however, has the following limitations.

1. It does not distinguish between dynamic and static infiltration, similar to the Williams–LaSeur and Hawkins models.
2. It allows sudden jumps in CN values when changing from one AMC to another AMC level.
3. The use of a linear regression equation invokes the problem of mass balance, for the sum of the regression coefficients is seldom equal to 1.0 in long-term hydrological simulation.

4. The base flow is taken as a fraction of F, which is not rational. The water retained in the soil pores may not be available for base flow, rather the water that percolates down to meet the water table may appear at the outlet as base flow.

Thus, there exists a need for an improved model that eliminates for the most part these limitations, leading to the formulation of a model based on the modified SCS-CN method (Mishra and Singh, 2002a; Mishra et al., 2003). In the present dissertation work, since the SCS – CN concept is utilized for computation of base flow, which is an integral part of total runoff from the watershed, a brief review of baseflow computation is in order.

2.4 BASEFLOW COMPUTATION

Base flow analysis, with a wide availability of methodologies, is a valuable strategy to understand the dynamics of groundwater discharge to streams. Stream flow data is commonly collected and made publicly available, so is amenable to desktop analysis prior to any detailed field investigations. However, it is important to remember that the assumption that base flow equates to groundwater discharge is not always valid. Water can be released into streams over different timeframes from different storages such as connected lakes or wetlands, snow or stream banks. As the hydrographical record represents a net water balance, base flow is also influenced by any water losses from the stream such as direct evaporation, transpiration from riparian vegetation, or seepage into aquifers along specific reaches. Water use or management activities such as stream regulation, direct water extraction, or nearby groundwater pumping can significantly alter the base flow component. Hence, careful consideration of the overall water budget and management regime for the stream is required.

Subsurface runoff analysis considers the movement of water throughout the entire hydrologic cycle. The prediction of subsurface runoff is performed with models of varying complexity depending on the application requirements and constraints. The models used may be categorized as event-oriented or continuous simulation. Event-oriented models utilize relatively simple techniques for estimating subsurface contributions to a flood hydrograph. Continuous simulation models continuously account for the movement of water throughout the hydrologic cycle. Continuous accounting of water movement involves the consideration of precipitation, snow melt, surface loss, infiltration, and surface transport processes that have been discussed previously. Other processes that need to be considered are evapotranspiration, soil moisture redistribution, and groundwater transport.

A stream hydrograph is the time-series record of stream conditions (such as water level or flow) at a gauging site. The hydrograph represents the aggregate of the different water sources that contribute to stream flow. These components can be subdivided into quick flow and base flow.

(i) Quick flow – the direct response to a rainfall event including overland flow (runoff), lateral movement in the soil profile (interflow) and direct rainfall onto the stream surface (direct precipitation), and;

(ii) Base flow – the longer-term discharge derived from natural storages.

The relative contributions of quick flow and base flow components change through the stream hydrographic record. The flood or storm hydrograph is the classic response to a rainfall event and consists of three main stages (Figure 2.4).

(i) Prior low-flow conditions in the stream consisting entirely of base flow at the end of a dry period;

(ii) With rainfall, an increase in stream flow with input of quick flow dominated by runoff and interflow. This initiates the rising limb towards the crest of the flood hydrograph. The rapid rise of the stream level relative to surrounding groundwater levels reduces or can even reverse the hydraulic gradient towards the stream. This is expressed as a reduction in the base flow component at this stage;

(iii) The quick flow component passes, expressed by the falling limb of the flood hydrograph. With declining stream levels timed with the delayed response of a rising water table from infiltrating rainfall, the hydraulic gradient towards the stream increases. At this time, the base flow component starts increasing. At some point along the falling limb, quick flow ceases and streamflow is again entirely base flow. Over time, base flow declines as natural storages are gradually drained till the dry period is up and until the next significant rainfall event.

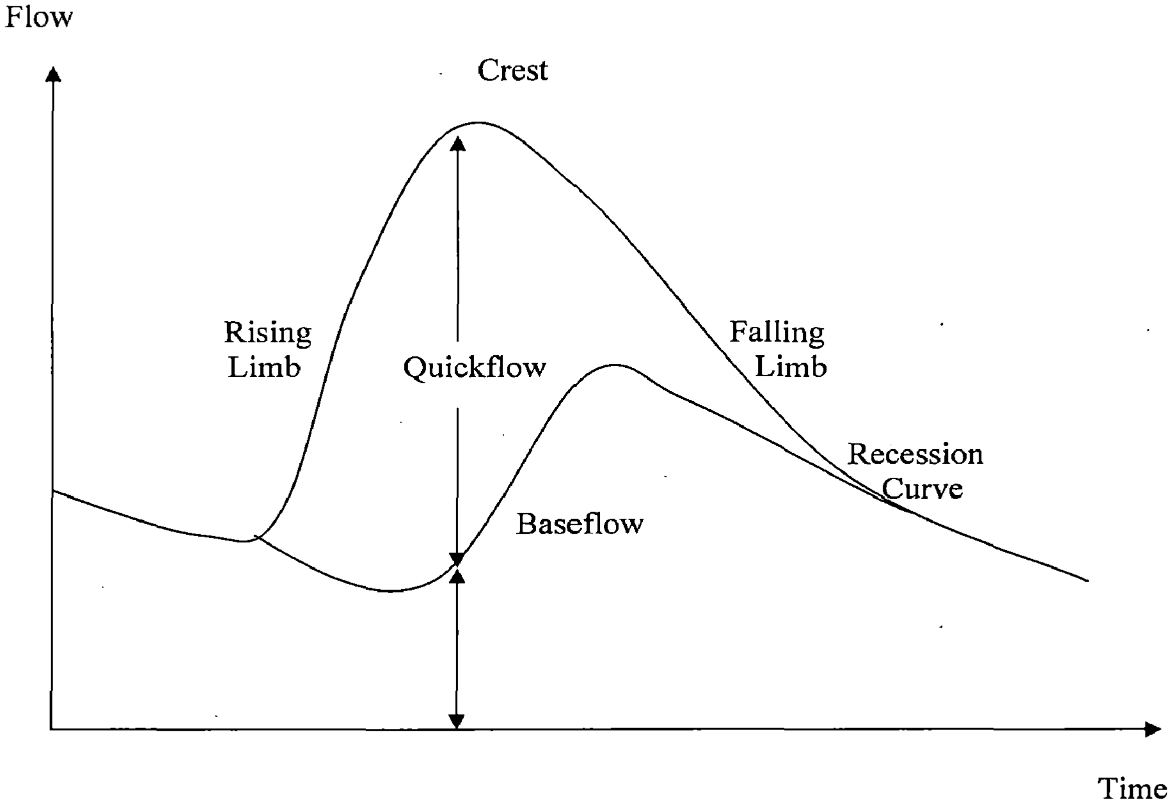


Fig: 2.4 Components of a typical flood hydrograph

Another complication is that base flow is also influenced by any water losses from the stream. The hydrographic record essentially represents the net balance between gains to and losses from the stream. These losses include direct evaporation from the stream channel or from any connected surface water features such as lakes and wetlands, transpiration from riparian vegetation, evapotranspiration from source groundwater seepages, leakage to the underlying aquifer, or rewetting of stream bank and alluvial deposits (Smakhtin, 2001). These processes are often aggregated into a transmission loss for the reach of the stream. Specific activities that can influence base flow include:

(i) Stream regulation where flow is controlled by infrastructure such as dams or weirs. Releases from surface water storages for downstream users can make up the bulk of stream flow during dry periods. Base flow analysis should be undertaken in unregulated reaches, or at least the regulated catchment area should be no more than 10% of the catchment area of the stream flow gauge (Neal et al. 2004);

(ii) Direct pumping of water from the stream for consumptive uses such as irrigation, urban supply or industry;

(iii) Artificial diversion of water into or out of the stream as part of inter-basin transfer schemes;

(iv) Direct discharges into the stream, such as from sewage treatment plants, industrial outfalls or mine dewatering activities;

(v) Seasonal return flows from drainage of irrigation areas;

(vi) Artificial drainage of the floodplain, typically for agricultural or urban development, which can enhance rapid runoff and reduce delayed drainage;

(vii) Changes in land use, such as clearing, re-afforestation or changes in crop type, which can significantly alter evapotranspiration rates;

(viii) Groundwater extraction, sufficient to lower the water table and decrease or reverse the hydraulic gradient towards the stream. Careful consideration of the overall water budget and management regime for the stream is required before the assumption that base flow equates to groundwater discharge can be made.

Several methods for base flow separation are used when actual amount of base flow is unknown. During large storms, the maximum rate of discharge is only slightly affected by base flow, and inaccuracies in separation are fortunately not important.

2.4.1 Baseflow Separation

From a hydrological process view point, baseflow is considered to be that component of the total flow hydrograph that is derived from runoff processes that operate relatively slowly. Thus many of the traditional hydrograph separation approaches have focused on trying to distinguish between rapidly occurring surface runoff, slower moving interflow and even slower discharge from groundwater. However, the conceptual basis for such distinctions can only really apply in small catchments where differential travel times, due to distance from the catchment outlet, play a minor role. In larger catchments the situation is far more complex and hydrograph shapes can be affected by a multitude of processes, some dominated by topography, others by subsurface (soils and geology) characteristics and others by spatial variations in rainfall inputs.

Baseflow separation techniques use the time-series record of stream flow to derive the baseflow signature. The common separation methods are either graphical which tend to focus on defining the points where baseflow intersects the rising and falling limbs of the quickflow response, or involve filtering where data processing of the entire stream hydrograph derives a baseflow hydrograph(www.connectedwater. gov.au/ framework/ baseflow_separation).

Graphical Separation Methods

Graphical methods are commonly used to plot the baseflow component of a flood hydrograph event, including the point where the baseflow intersects the falling limb (Figure 2.5). Stream flow subsequent to this point is assumed to be entirely baseflow, until the start of the hydrographic response to the next significant rainfall event. These graphical approaches of partitioning baseflow vary in complexity and include

(i) An empirical relationship for estimating the point along the falling limb where quickflow has ceased and all of the stream flow is baseflow;

$$D = 0.827A^{0.2} \tag{2.20}$$

where D is the number of days between the storm crest and the end of quickflow, and A is the area of the catchment in square kilometres. The value of the exponential constant (0.2) can vary depending on catchment characteristics such as slope, vegetation and geology.

(ii) The constant discharge method assumes that baseflow is constant during the storm hydrograph. The minimum streamflow immediately prior to the rising limb is used as the constant value.

(iii) The constant slope method connects the start of the rising limb with the inflection point on the receeding limb. This assumes an instant response in baseflow to the rainfall event.

(iv) The concave method attempts to represent the assumed initial decrease in baseflow during the climbing limb by projecting the declining hydrographic trend evident prior to the rainfall event to directly under the crest of the flood hydrograph (Linsley et al. 1958). This minima is then connected to the inflection point on the receeding limb of storm hydrograph to model the delayed increase in baseflow.

(v) Using the trends of the falling limbs before and after the storm hydrograph to set the bounding limits for the baseflow component.

(vi) Using the Boussinesq equation as the basis for defining the point along the falling limb where all of the streamflow is baseflow.

Filtering Separation Methods

The baseflow component of the streamflow time series can also be separated using data processing or filtering procedures. These methods tend not to have any hydrological basis but aim to generate an objective, repeatable and easily automated

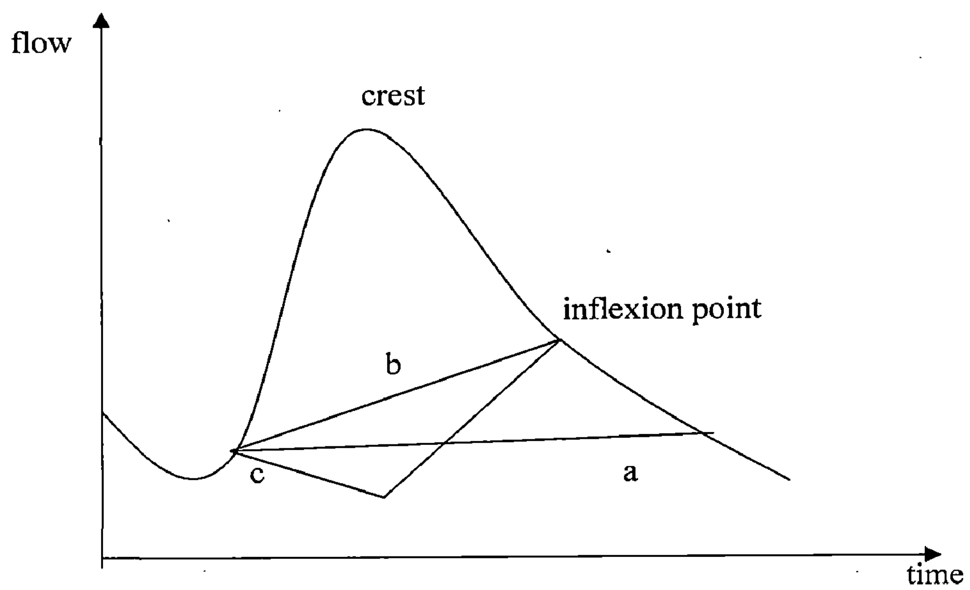


Fig : 2.5 Graphical baseflow separation techniques including (a) constant discharge method, (b) constant slope method, (c) concave method

index that can be related to the baseflow response of a catchment. The baseflow index (BFI) *or* reliability index, which is the long-term ratio of baseflow to total streamflow, is commonly generated from this analysis. Other indices include the mean annual baseflow volume and the long-term average daily baseflow. Examples of continuous hydrographic separation techniques based on processing or filtering the data record include

1. Increasing the base flow at each time step, either at a constant rate or varied by a fraction of the runoff;
2. The smoothed minima technique which uses the minima of 5-day non-overlapping periods derived from the hydrograph. The baseflow hydrograph is generated by connecting a subset of points selected from this minima series. The HYSEP hydrograph separation program uses a variant of this called the local-minimum method;
3. The fixed interval method discretises the hydrographic record into increments of fixed time. The magnitude of the time interval used is calculated by doubling (and rounding up) the duration of quickflow calculated empirically from Equation 2.20. The baseflow component of each time increment is assigned the minimum stream flow recorded within the increment;
4. The sliding-interval method assigns a baseflow to each daily record in the hydrograph based on the lowest discharge found within a fixed time period before and after that particular day;
5. Recursive digital filters, which are routine tools in signal analysis and processing, are used to remove the high-frequency quickflow signal to derive the low-frequency baseflow signal. Table 2.1 outlines some of the digital filters that have been applied to smooth hydrographic data. Eckhardt (2005) has developed a general formulation that can devolve into several of the commonly used one-parameter filters:

$$q_{b(i)} = \frac{(1-BFI_{max})aq_{b(i-1)} + (1-a)BFI_{max}q_i}{1-aBFI_{max}} \qquad (2.21)$$

where $q_{b(i)}$ is the baseflow at time step i, $q_{b(i-1)}$ is the baseflow at the previous time step i-1, q_i is the stream flow at time step i, a is the recession constant and BFI_{max} is the maximum value of the baseflow index that can be measured; and

6. The streamflow partitioning method uses both the daily record of streamflow and rainfall. Baseflow equates to streamflow on a given day, if rainfall on that day and a set number of days previous is less than a defined rainfall threshold value. Linear interpolation is used to separate the quickflow component during high rainfall events.

Table 2.1: Recursive digital filters used in base flow analysis (Grayson *et al,* 1996; Chapman, 1999; Furey and Gupta, 2001)

Filter Name	Filter Equation	Source	Comments
One-parameter algorithm	$q_{b(i)} = \frac{k}{2-k} q_{b(i-1)} + \frac{1-k}{2-k} q_{(i)}$	Chapman and Maxwell (1996)	$q_{b(i)} \leq q_{(i)}$ Applied as a single pass through the data.
Boughton two-parameter algorithm	$q_{b(i)} = \frac{k}{1+C} q_{b(i-1)} + \frac{C}{1+C} q_{(i)}$	Boughton (1993) Chapman and Maxwell (1996)	$q_{b(i)}$ £$q_{(i)}$ Applied as a single pass through the data Allows calibration against other baseflow information such as tracers, by adjusting parameter C
IHACRES three-parameter algorithm	$q_{b(i)} = \frac{k}{1+C} q_{b(i-1)} + \frac{C}{1+C}\left(q_{(i)} + \alpha_q q_{(i1)}\right)$	Jakeman and Hornberger (1993)	Extension of Boughton two-parameter algorithm
Lyne and Hollick algorithm	$q_{f(i)} = \alpha q_{f(i-1)} + \left(q_{(i)} - q_{(i-1)}\right) * \frac{1+\alpha}{2}$	Lyne and Hollick (1979) Nathan and McMahon, (1990)	$q_f(i) \leq 0$ α a value of 0.925 recommended for daily stream data .filter recommended to be applied in three passes Baseflow is $q_b = q - q_f$
Chapman algorithm	$q_{f(i)} = \frac{3\alpha - 1}{3-\alpha} q_{f(i-1)} + \frac{2}{3+\alpha}\left(q_{(i)} - \alpha q_{(i-1)}\right)$	Chapman (1991) Mau and Winter (1997)	Baseflow is $q_b = q - q_f$
Furey and Gupta filter	$q_{b(i)} = (1-\gamma) q_{b(i-1)} + \gamma \frac{c_3}{c_1} * \left(q_{(i-d-1)} - q_{b(i-d-1)}\right)$	Furey and Gupta (2001)	Physically-based filter using mass balance equation for baseflow through a hillside

- $q_{(i)}$ is the original stream flow for the i^{th} sampling instant
- $q_{b(i)}$ is the filtered base flow response for the i^{th} sampling instant
- $q_{f(i)}$is the filtered quick flow for the i^{th} sampling instant
- $q_{(i-1)}$ is the original stream flow for the previous sampling instant to i
- $q_{b(i-1)}$is the filtered base flow response for the previous sampling instant to i
- $q_{f(i-1)}$is the filtered quick flow for the previous sampling instant to i
- k is the filter parameter given by the recession constant
- α, α_q are filter parameters
- c is a parameter that allows the shape of the separation to be altered
- γ, c c_1, c_3 are physically based parameters

Along with the above methods for base flow separation, there are also other methods to calculate base flow. Some are given below :

It is known that infiltration depends on rainfall. Therefore, if P – I_a is less than the gravitational infiltration F_c on a given day, then F_c = P - I_a and direct surface runoff RO_t = 0 or dynamic infiltration Fd_t= 0. It implies that F_c exists even prior to the satisfaction of the capillary demand, which is in contrast with reality. This is because of the assumed equivalence between F_c and the minimum infiltration rate at a time approaching infinity. Considering that the water infiltrating after saturation through F_c percolates down to the water table, it finally appears at the outlet of the basin with assumptions that the basin boundary coincides with the aquifer boundary and no lateral flow contributes to the water table from across the defined watershed boundary. Thus, applying continuity and storage equations, the baseflow (O_b) can be computed as:

$$O_{b(t+\Delta t)} = g_0 F_{c(t)} + g_1 F_{c(t)} + g_2 O_{b(t)} \tag{2.22}$$

where

$$g_0 = \frac{\Delta t / K_b}{2 + \Delta t / K_b}; \tag{2.23a}$$

$$g_1 = g_0; \tag{2.23b}$$

$$g_2 = \frac{2 - \Delta t / K_b}{2 + \Delta t / K_b}; \tag{2.23c}$$

K_b is the base flow storage coefficient [T]; and g_0, g_1, g_2 are the base flow routing coefficients.

The concept behind the SCS – CN method can be applied to determination of surface drainage flow from rainfall (Yuan et. al., 2001). The work of Andrews (1954) and Mockus (1964) was the basis for the generalized SCS rainfall-runoff relationship, which can be expressed as follows: when accumulated natural runoff is plotted against accumulated natural rainfall, runoff starts after some rainfall has accumulated, and the line of the relation curve becomes asymptotic to a line of 45° slope, as illustrated in Figure 2.6.

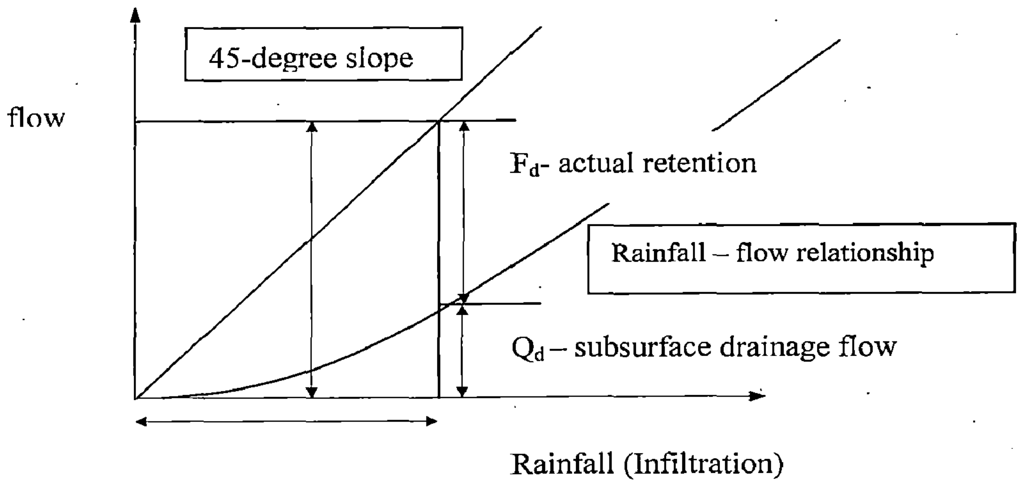


Fig : 2.6 Typical rainfall and flow relationship (Source: Yuan et al., 2001)

By analogy, for subsurface drainage flow, equation becomes:

$$\frac{F_d}{S_d} = \frac{Q_d}{F} \tag{2.24}$$

where F_d = actual retention after flow begins, S_d = potential maximum retention of watershed, Q_d = drainage flow depth ($F > Q_d$), and F= infiltration depth.

If there are no initial abstractions, or if one begins the water accounting after initial abstractions, then equation (2.24) can be rewritten as:

$$\frac{F - Q_d}{S_d} = \frac{Q_d}{F} \tag{2.25}$$

However, initial abstractions, in the form of soil moisture changes, must be considered, and the amount of infiltration available for drainage flow is F - ΔS_m. By substituting F- ΔS_m for F in equation (2.25), the following equation results:

$$\frac{F-\Delta S_m - Q_d}{S_d} = \frac{Q_d}{F-\Delta S_m} \tag{2.26}$$

where F = infiltration depth, ΔS_m = soil moisture storage, Q_d = drainage flow depth ($F > Q_d$), S_d = potential maximum retention of watershed. If no surface runoff occurs, then:

$$F = P - I_a \tag{2.27}$$

where P = rainfall depth (P > I), I_a = interception. If surface runoff occurs, then

$$F = P - I_a - Q_s \tag{2.28}$$

where P = rainfall depth (P >I) , I_a = interception, Q_s = surface runoff.

If, for simplicity, we assume that there is no surface runoff and substitute equation (2.27) into equation (2.26), we obtain:

$$\frac{P-I_a-\Delta S_m - Q_d}{S_d} = \frac{Q_d}{P-I_a-\Delta S_m} \tag{2.29}$$

If we assume

$$I_d = I_a + \Delta S_m \tag{2.30}$$

where I_d is the initial abstraction for subsurface drainage flow. A substitution of equation (2.30) into equation (2.29) yields

$$\frac{P-I_d-Q_d}{S_d} = \frac{Q_d}{P-I_d} \tag{2.31}$$

Solving for Q_d results in

$$Q_d = \frac{(P-I_d)^2}{P-I_d+S_d} \qquad (P > I_d) \tag{2.32}$$

$$Q_d = 0 \qquad (P < I_d) \tag{2.33}$$

Equations (2.32) and (2.33) can be used to estimate subsurface flow (Q_d) from storm rainfall. If surface runoff occurs then equation (2.29) can be written as:

$$\frac{P - I_a - Q_s - \Delta S_m - Q_d}{S_d} = \frac{Q_d}{P - I_a - Q_s - \Delta S_m} \tag{2.34}$$

and solving for Q_d results in

$$Q_d = \frac{(P - I_d - Q_s)^2}{P - I_d - Q_s + S_d} \qquad (P > I_d) \tag{2.35a}$$

$$Q_d = 0 \qquad (P < I_d) \tag{2.35b}$$

Equation (2.35) can be used for computation of base flow. The methodology proposed for baseflow computation in the thesis work is largely based on this concept and its development is discussed in the forthcoming chapter.

CHAPTER III: METHODOLOGY

3.1 GENERAL

In calculation of the quantity of runoff from a basin, the curve number is used to determine the amount of precipitation excess that results from a rainfall event over the basin. This methodology is a standard hydrologic analysis technique that has been applied in a variety of settings and the development and application of the curve number is well documented in Section 4 of the National Engineering Handbook (NEH) in 1956. The Natural Resource Conservation Service - Curve Number (NRCS-CN) model, formerly known as Soil Conservation Service - Curve Number (SCS-CN) model (SCS 1956, 1964, 1969, 1971, 1972, 1985, 1993), is one of the popular models for computing the volume of surface runoff from small to medium-sized agricultural watersheds for a given rainfall event. The SCS-CN technique (USDA, 1972) is an empirical method based on the characteristics of soil type, land use, and the hydrological condition in the watershed. It is a well known and practical tool that is used to estimate direct runoff from rainfall. This technique was originally derived from the examination of annual flood event data. Its application is therefore most suited to design involving high runoff events (Yong et al., 2006). Even though the curve number technique is appropriately used for rainfall – runoff event simulation, it has also been widely used in a number of continuous simulation models since the 1980's.

The SCS-CN model converts rainfall to surface runoff (or rainfall-excess) using a single parameter, called curve number (CN) which is derived from watershed characteristics and 5-day antecedent rainfall. Some of the reasons for its popularity are that (1) it is simple (Bales and Betson, 1981); (2) it is a familiar procedure that has been used for many years around the world; (3) it is computationally efficient; (4) the required inputs are generally available; and (5) it relates runoff to soil type, land use, and management practices. The use of readily available daily rainfall is particularly an important input to the SCS-CN model. This model however has its own limitations and assumptions, which lead to many questionable arguments on its applications. Since its inception, the SCS-CN model has been improved, extended and modified in various ways

The method which is derived to compute the surface runoff from rainfall in small agricultural watersheds is based on water balance equation and the two hypotheses (Mishra and Singh 1999, 2003; SCS, 1956). The curve numbers are a function of the land use type, soil texture type, hydrologic condition and antecedent moisture condition (AMC). Estimation of it requires mapping of the soil and land use with the drainage basin boundaries and specification of unique soil texture type.

3.2 EXISTING SCS – CN METHOD

The SCS curve number method is a simple, widely used and efficient method for determining the direct runoff from a rainfall event in a particular area. Although the method is designed for a single storm event, it can be scaled to find average annual runoff values. The requirements for this method are very low, rainfall amount and curve number. The curve number is based on the area's hydrologic soil group, land use, treatment and hydrologic condition; the former two being of the greatest importance. In the SCS-CN-based long-term hydrologic simulation, daily computation of direct surface runoff largely depends on AMC dependent CN. The computed direct surface runoff (or rainfall excess) is routed to the outlet of the catchment. Since the SCS-CN method is an infiltration loss model (Ponce and Hawkins, 1996), a portion of the infiltrated water is taken as base flow routed to the catchment outlet. The total runoff is the sum of the routed direct surface runoff and base flow.

The existing SCS – CN method (SCS 1956) consists of the following three equations (Mishra and Singh, 2003):

Water Balance Equation

$$P = I_a + F + Q \tag{3.1}$$

Proportionality Hypothesis

$$\frac{Q}{P-I_a}=\frac{F}{S} \quad (3.2)$$

- I_a–S Hypothesis

$$I_a=\lambda S \quad (3.3)$$

where P = total rainfall, I_a = initial abstraction, F = cumulative infiltration, Q = direct surface runoff, S = potential maximum retention, and λ = initial abstraction coefficient. All quantities in above equations are in depth or volumetric unit. Combination of these equations leads to the following popular form of the SCS – CN method

$$Q=\frac{(P-I_a)^2}{P-I_a+S} \quad (3.4)$$

Here, P > I_a and Q= 0 otherwise. By using the volumetric concept of soil water air, Mishra (1998) defined S as the maximum amount of space available in the soil profile under given antecedent moisture. The relation between S and CN is usually expressed in SI units as:

$$S=\frac{25400}{CN}-254 \quad (3.5)$$

where S is in mm and CN = curve number. CN is taken as CN0 valid for AMC II (normal condition), for the first five days beginning from the first day of simulation (June 1 to June 5). As the time (day) advances, CN varies with AMC levels (Hawkins 1978; Mishra et al. 1998) dependent on the amount of antecedent rainfall (ANTRF):

$$\text{ANTRF}_t=P_{(t-1)}+P_{(t-2)}+P_{(t-3)}+P_{(t-4)}+P_{(t-5)} \quad (3.6)$$

where *t*=current day, and *P*=rainfall of the respective day. AMC II (average or normal condition) is taken as the basis from which adjustments to daily curve numbers are made so that they correspond to AMC I or AMC III (Hjelmfelt 1991). Different AMC class limits are provided for the dormant and growing seasons based on five-day antecedent precipitation, i.e., ANTRF and presented in (Mishra et al., 1998; Ponce, 1989; Hawkins et al., 1985). Variation in curve numbers based on the total rainfall in the five days preceding the storm under consideration (Woodward and Croshney, 1992), CN_t of t^{th} day which corresponds to CN_{II} is converted to CN_I or CN_{III} as follows (Hawkins et al., 1985):

$$CN_I = \frac{CN_{II}}{2.3 - 0.013CN_{II}} \tag{3.7}$$

$$CN_{III} = \frac{CN_{II}}{0.43 - 0.0057CN_{II}} \tag{3.8}$$

Table 3.1 Antecedent Moisture Conditions

AMC	Total five-day antecedent rainfall (cm)	
	Dormant season	Growing season
I	Less than 1.3	Less than 3.6
II	1.3 to 2.8	3.6 to 5.3
III	More than 2.8	More than 5.3

which are valid for AMC I or AMC III. It is worth noting that the initial vaue of CN=CN0 at the start of simulation, an optimized value, corresponds to AMC II. Since the months of May and June are usually quite hot with evapotranspiration being the highest of the year, dry soils contain minimum moisture in their pores, leading to availability of maximum pore space of moisture retention. Therefore, a minimum CN value is likely to occur during this period, and is designated as CN0. Thus,

$$S_t = S_o \quad \text{for} \quad S_t \geq S_o \tag{3.9}$$

where S_0 corresponds to CN_0, derivable from Eq.(3.5)

The potential water retention is defined as the maximum possible pore space available for retention of moisture in the soil store after the loss, which is in the form of evapotranspiration and the outflow in the form of base flow. Potential maximum retention S is the maximum depth of storm rainfall that could potentially be abstracted by a given site (Ponce and Hawkins, 1996). The potential retention on the current day is calculated by considering the space availability after evapotranspiration and infiltration inputs, as below:

$$S_t = S_{(t-1)} - (1 - b_f)F_{(t-1)} + \mathrm{EV}_{(t-1)} \tag{3.10}$$

where S_t=space available for water retention for the current day; S_{t-1}=previous day's potential maximum retention (mm); EV_{t-1} =previous day's evapotranspiration (mm), computed using Penman coefficients taken as 0.8 for June – September, 0.6 for October–January, and 0.7 for February–May; and F_{t-1}=previous day's infiltration (mm), computed using the water balance equation

$$F_{(t-1)} = P_{(t-1)} - I_{a(t-1)} - \mathrm{RO}_{(t-1)} \tag{3.11}$$

Here, if $P_{e(t)} \geq 0$, $F_t \geq 0$. The quantity $(1-b_f)\ F_{(t-1)}$ is assumed to be a part of the infiltration available in the soil store on the previous day for balancing the soil storage for moisture retention, and b_f is taken as a factor describing base flow.

Base flow is assumed to be a fraction, b_f, of infiltration F and it is routed to the watershed outlet using the lag and route method as follows:

$$q_{b(t+\mathrm{NLAG})} = b_f F_t \tag{3.12}$$

where NLAG = lag parameter.

3.3 PROPOSED RAINFALL - RUNOFF MODEL:

In this study, based on the existing SCS – CN method, a new method is proposed for long-term hydrologic simulation. Here the direct surface runoff is computed based on the SCS – CN based hydrological simulation and it is routed to the outlet of the catchment. Since the SCS – CN method is an infiltration loss model, a portion of the infiltration is taken as base flow, as described above. The total runoff is the sum of the surface runoff and base flow.

3.3.1 Computation of direct surface runoff

Replacing Q by RO (surface runoff) in Eq. 3.4 (for clarity in text, Eq. 3.4 can be rewritten for daily runoff with time t as subscript) yields

$$RO_{(t,t+\Delta t)} = \frac{(P_{(t,t+\Delta t)} - I_{a(t)})^2}{P_{(t,t+\Delta t)} - I_{a(t)} + S_t} \tag{3.13}$$

where

$$I_{a(t)} = \lambda S_t \tag{3.14}$$

$$S_t = \frac{25400}{CN_t} - 254 \tag{3.15}$$

Equation (3.13) valid for $P_{(t,t+\Delta t)} \geq I_{a(t)}$, $RO_{(t,t+\Delta t)} = 0$ otherwise. Here P = total rainfall, I_a = initial abstraction, S = potential maximum retention, and λ = initial abstraction coefficient.

3.3.2 Soil moisture budgeting:

The total infiltration (F) consists of static infiltration component (Q_d) and dynamic infiltration component (F_r) (Mishra.et.al, 2004) as shown in Fig. 3.1. The dynamic infiltration component of infiltration that occurred during the time period can be computed from water balance equation as:

$$F_{r(t,t+\Delta t)} = F_{(t,t+\Delta t)} - Q_{d(t,t+\Delta t)} \tag{3.16}$$

where

$$F_{(t,t+\Delta t)} = P_{(t,t+\Delta t)} - I_{a(t)} - RO_{(t,t+\Delta t)} \tag{3.17}$$

which is valid for $RO_{(t,t+\Delta t)} \geq 0$, $F_{r(t,t+\Delta t)} = 0$ otherwise. The term $F_{r(t,t+\Delta t)}$ also represents an increase in the amount of soil moisture in the soil profile during the time period, which when added to its antecedent moisture leads to the antecedent moisture amount for the next day.

$$M_{(t,t+\Delta t)} = M_{(t)} + F_{r(t,t+\Delta t)} - ET_{(t,t+\Delta t)} \tag{3.18}$$

where $M_{(t,t+\Delta t)}$ varies from 0 to S_{abs}, S_t can be modified for the next day by balancing the soil moisture as

$$S_{(t+\Delta t)} = S_{(t)} - M_{(t+\Delta t)} \tag{3.19}$$

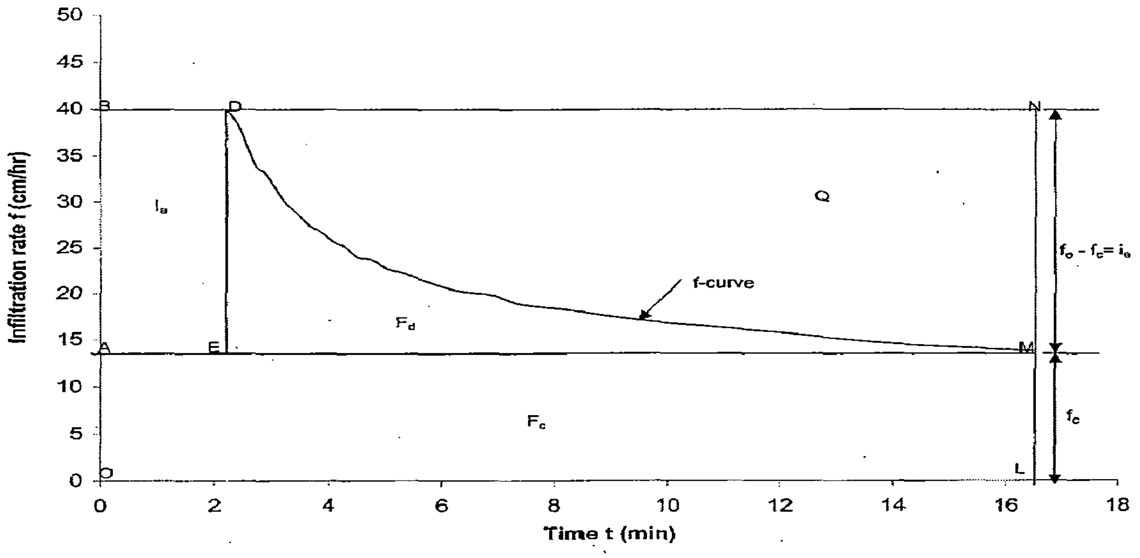


Fig: 3.1 A descriptive infiltration curve of Columbia sandy loam (Mishra and Singh, 2003).

3.3.3 Computation of Evapotranspiration

The daily evapotranspiration (ET) can be computed using the pan evaporation as:

$$PET_{(t,t+\Delta t)} = PANC \times E_{(t,t+\Delta t)} \tag{3.20}$$

where E is the pan evaporation based on field data and PANC is the pan coefficient, assumed as 0.8 for June – September, 0.6 for October – January and 0.7 for February – May in this study.

3.3.4 Base flow

Based on the general hypothesis of SCS – CN method, base flow is computed as:

$$Q_{d(t,t+\Delta t)} = \frac{(P_{(t,t+\Delta t)} - I_{a(t)} - RO_{(t,t+\Delta t)} - I_{d(t)})^2}{P_{(t,t+\Delta t)} - I_{a(t)} - RO_{(t,t+\Delta t)} - I_{d(t)} + S_d} \tag{3.21}$$

where

$$I_{d(t)} = \lambda S_{d(t)} \tag{3.22}$$

$$S_{d(t)} = \frac{25400}{CN_{d_t}} - 254 \quad (3.23)$$

Equation (3.21) is valid for ($P_{(t,t+\Delta t)} - I_{a(t)}$ - $RO_{(t,t+\Delta t)}$) $\geq I_{d(t)}$, $Q_{d(t,t+\Delta t)} = 0$ otherwise. Here RO = direct surface runoff, I_d = initial abstraction, S_d = potential maximum retention, and λ = initial abstraction coefficient for subsurface drainage.

The initial abstraction for drainage flow depends on the soil moisture storage, S_d or (CN_d) for next day is varied by balancing the soil moisture storage follows :

$$S_{d(t+\Delta t)} = S_{d(t)} - \left(M_{(t+\Delta t)} - M_{(t)}\right) \quad (3.24)$$

3.3.5 Routing

Based on the principle of continuity and storage equations, the daily rainfall excess is routed to the outlet of the catchment using single linear reservoir (Mishra and Singh, 2004)

$$R - O = \frac{\Delta S}{\Delta t} \quad (3.25)$$

$$S = KO \quad (3.26)$$

where K=storage coefficient (day); R=inflow (mm/day); O=outflow (mm/day); S=storage (mm). Here, the time step is considered as one-day interval. $\Delta t/K$ is defined as the Courant number (Ponce, 1989). The rainfall excess RO_t corresponding to P_e can be computed only if rainfall P exceeds initial abstraction (I_a), it is otherwise zero. Then RO_t is routed to the outlet of the basin using the single linear reservoir as below:

$$DO_{(t+\Delta t)} = C0RO_t + C1RO_{t-1} + C2DO_{t-1} \quad (3.27)$$

where

$$C0 = \frac{COUR}{2 + COUR} \quad (3.28a)$$

$$C1 = C0 \quad (3.28b)$$

$$C2 = \frac{2 - COUR}{2 + COUR} \quad (3.28c)$$

$$COUR = \frac{1}{K} \quad (3.29)$$

In equation (3.28a-c), K is the storage coefficient and COUR = courant number.

It is known that infiltration depends on rainfall. Therefore, if P- I_a is less than F on a given day, then F = P- I_a. It emphasized that under such situation, $RO_t = 0$. Applying equation (3.27), base flow can be computed as

$$Q_{b(t+\Delta t)} = C0Q_{d\,t} + C1Q_{d\,(t-1)} + C2Q_{b(t-1)} \quad (3.30)$$

where

$$C0 = \frac{COUR}{2+COUR} \quad (3.31a)$$

$$C1 = C0 \quad (3.31b)$$

$$C2 = \frac{2-COUR}{2+COUR} \quad (3.31c)$$

$$COUR = \frac{1}{K_b} \quad (3.32)$$

In equation (3.31a-c), K_b is the base flow storage coefficient

Thus, the total runoff hydrograph, O appearing at the outlet of the catchments is computed as the sum of the routed rainfall excess, DO, and the base flow, O_b. Expressed mathematically,

$$O_t = DO_{(t)} + O_{b(t)} \quad (3.33)$$

which actually represents the computed total runoff hydrograph.

3.4 MODEL PARAMETERS

Model parameters are CN, CN_d, K [T] and K_b [T] and these can be determined using the Marquardt algorithm of constrained least squares or any other appropriate algorithm. It is also possible to derive these parameters physically or from rainfall–Runoff data. The parameter CN represents the curve number on the simulation, assuming that the maximum pore space is available in the soil for water storage or retention on that day of simulation. The curve number can vary from 0 to 100. CN_d is the curve number for the drainage flow and it depends on the soil moisture storage. It also varies from 0 to 100. Parameter K represents the storage coefficient of the surface runoff hydrograph and is analogous to the time lag of the watershed. It can also be derived from the rainfall–runoff data by plotting them on a semi-logarithmic paper. The slope of the fit represents K. The rainfall–runoff data set selected for the derivation should correspond to high rainfall–runoff events excluding base flow. Similarly, parameter K_b, which represents the storage coefficient of the base flow appearing at the outlet of the watershed, can be derived for low rainfall–runoff events or using other methods suggested in standard text books, for example, in the text books by Ponce (1989), Singh (1992) and Singh and Singh (2001), among others. For describing the range of variation of these parameters, the lower bound is taken as zero, because all the parameters are physically non-negative.

CHAPTER IV:
STUDY AREA AND DATA AVAILABILITY

The 'study area chosen for the present work is Ramganga Catchment (Figure 4.1) a major tributary of River Ganga in Uttarakhand State of India. Apart from this catchment the above proposed model is also applied to different catchments falling under different climatic and geographic settings of India. The daily monsoon (June–November) data of study catchments, Hemavati, a tributary of River Cauvery in Karnataka State; Hridaynagar, Manot, and Mohegaon catchments, tributaries of River Narmada in Madhya Pradesh; Kalu catchment, a tributary of River Ulhas, in Maharashtra; and Ghodahado catchment, a tributary of River Rushikulya, in Orissa, State of India, observed at respective gauging stations are used in the analysis. Details of these catchments are given as follows.

4.1 RAMGANGA BASIN

4.1.1 General

The Ramganga river is a major tributary of Ganga and drains a catchment area of 3,134 km^2 (Fig. 4.1). Its catchment lies in the Sivalik ranges of Himalayas and the valley is known as Patlei Dun. River Ramganga originating at Diwali Khel. It emerges out of the hills at Kalagarh (District Almora) where a major multipurpose Ramganga dam is situated. Its catchments lies between elevation 262 and 2,926 m above mean sea level, and it is considerably below the perpetual snow line of the Himalayas. The river traverses approximately 158 km before it meets the reservoir and then continues its journey in the downstream plains for 370 km before joining River Ganga at Farrukhabad.

Fig : 4.1 Ramganga catchment

During its travel up to Ramganga dam, the river is joined by main tributaries: Ganges, Binoo, Khatraun, Nair, Badangad, Mandal, Helgad, and Sona Nadi. About 50% of the drainage basin is covered with forest, 30% is under cultivation on terraced fields, and the remaining 20% is urban/barren land.

4.1.2 Data availability

The climatic condition of the river basin is largely influenced by the orographic effect. The area receiving the majority of precipitation in the form of rainfall. The daily rainfall and evaporation data available from 1985 to 1990 (five years). The runoff data is available for the same period. These data have been processed for the application of the model.

(a) Rainfall: The Ramganga valley experiences approximately an annual precipitation of 1,550 mm. The raingauge network consists of Ranikhet, Chaukhatia, Naula, Marchulla, Lansdowne and Kalagarh besides the other existing stations.

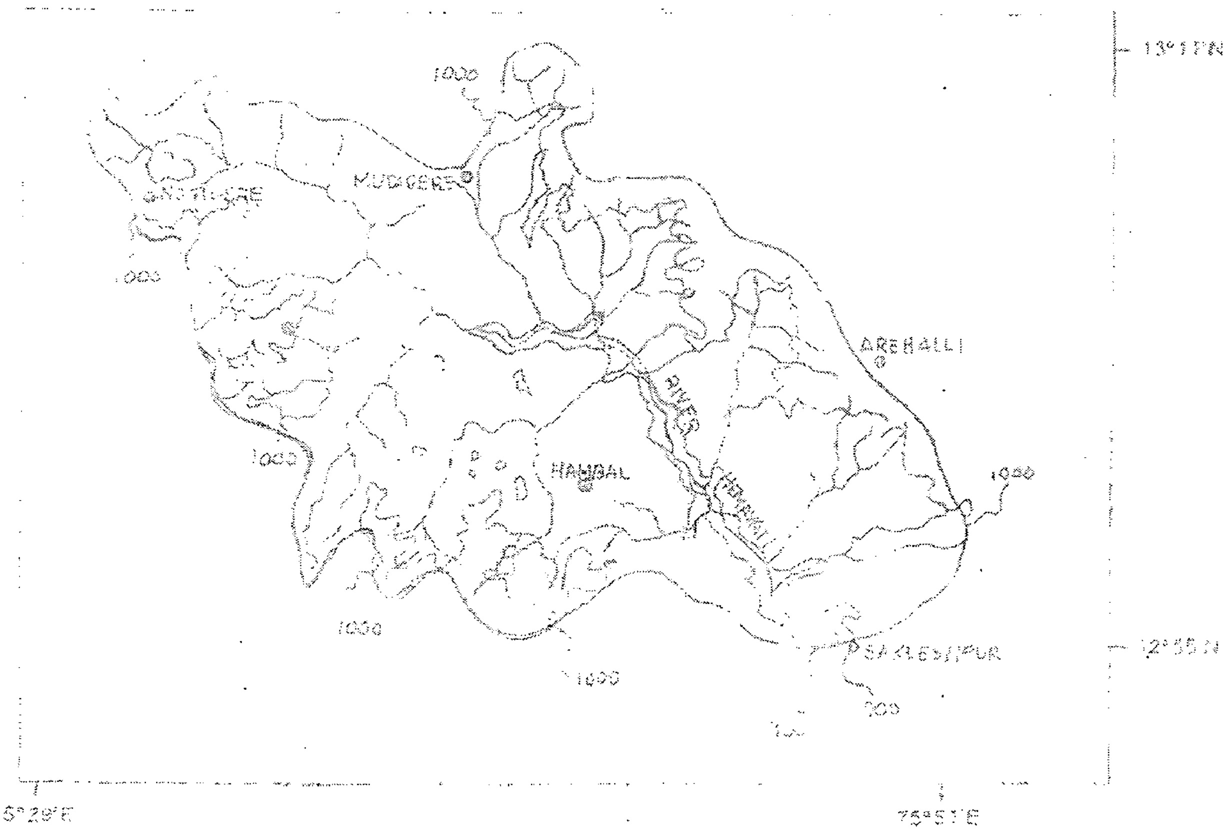
(b) Evaporation: The daily pan evaporation data are available for the catchment, but for a limited period only.

(c) Runoff: Streamflow records of the Ramganga River, including river stages, instantaneous as well as monthly, are available at Kalagarh.

4.2 OTHER CATCHMENTS

4.2.1 Hemavati catchment

River Hemavati is a tributary of River Cauvery, originating in Ballaiarayanadurga in the Western Ghats in Mundgiri Taluk of Chikmanglur district in Karnataka State (Mishra and Singh, 2003b). It passes through a region of heavy rainfall in its early reaches, in the vicinity of Kotigehara and Mudigere. It has Yagachi and Algur tributaries and drains an area of 600 km^2 up to Sakleshpur. The catchment of Hemavati lies between 12° 55′ and 13° 11′ north latitude and 75° 20′ and 75° 51′ east longitude (Fig. 4.2). It is a hilly catchment with steep to moderate slopes (Mishra and



· **Fig 4.2: Hemavati catchment**

Singh, 2003b). The upper part of the catchment is hilly with an average elevation of 1,240m above the mean sea level and the lower part forms a plain terrain with an average elevation of 890m. Agriculture and plantation are the major industries of the basin. Its land use can be characterized by forests (12%), coffee plantations (29%), and agricultural lands (59%). The principal soil types are red loamy soil (67%) and red sandy soil (33%). Soils in the forest area and coffee plantations are greyish due to high humus content.

4.2.2 Narmada catchment

The River Narmada (Fig. 4.3) is one of the major rivers with 41 tributaries flowing through central parts of India. It rises from the Amarkantak plateau of the Maikala range in Shahdol district in Madhya Pradesh at an elevation of about 1059 m above mean sea level. The river travels a distance of 1312 km before it joins the Gulf of Cambay in the Arabian Sea near Bharuch in Gujarat. The stream flow data used in the study belong to the River Narmada at Manot, Banjar at Hridaynagar and Burhner at Mohegaon and described briefly below.

(a) **Narmada up to Manot**: The Narmada catchment up to Manot lies between north latitudes 22° 26′ to 23° 18′ and east longitudes 80° 24′ to 81° 47′. The length of the River Narmada from its origin up to Manot is about 269 km, with a drainage area of 5032 km^2. The catchment is covered by forest and its topography is hilly. Its elevation ranges from 450 m near Manot site to 1110 m in the upper part of the catchment. It has a continental type of climate classified as sub-tropical and sub-humid, with average annual rainfall of 1596 mm. It is very hot in summer and cold in winter. In the major part of the catchment, soils are red, yellow and medium black with shallow to very shallow depth. In some small pockets of plain land, soils are moderately deep dark greyish clay. Approximately 52% of the catchment area is under cultivation, about 35% under forest and 13% under wasteland.

(b) **Banjar up to Hridaynagar**: The Banjar River, a tributary of Narmada in its upper reaches, rises from the Satpura range in the Durg district of Madhya Pradesh near Rampur village at an elevation of 600 m at north latitude 21° 42′ and east longitude 80° 50′. Its catchment area up to Hridaynagar is about 3370 km^2 and the elevation drops from 600 m to 372 m at Hridaynagar gauging site. The climate of the basin can be classified as sub-tropical and sub-humid, with average annual rainfall of 1178 mm. About 90% of the annual rainfall is received during the monsoon season (June–October). The estimates of evapotranspiration

(c) **Burhner up to Mohegaon:** The Burhner River rises in the Maikala range, south-east of Gwara village in the Mandla district of Madhya Pradesh at an elevation of about 900 m at north latitude 22° 32′ and east longitude 81° 22′. It flows in a westerly direction for a total length of 177 km to join the Narmada near Manot. The Burhner drains a total area of about 4661 km^2 and its catchment area up to Mohegaon is about 4103 km^2. The elevation at Mohegaon gauging site drops to 509 m. The climate of the basin can be classified as sub-tropical and sub-humid, with average annual rainfall of 1547 mm. The evapotranspiration varies from 4 mm/day in winter to 10 mm/day in summer. The catchment area comprises both flat and undulating lands covered with forest and cultivated lands. Soils are mainly red and yellow silty loam and silty clay loam. Forest and agricultural lands share nearly 58% and 42% of the catchment area, respectively.

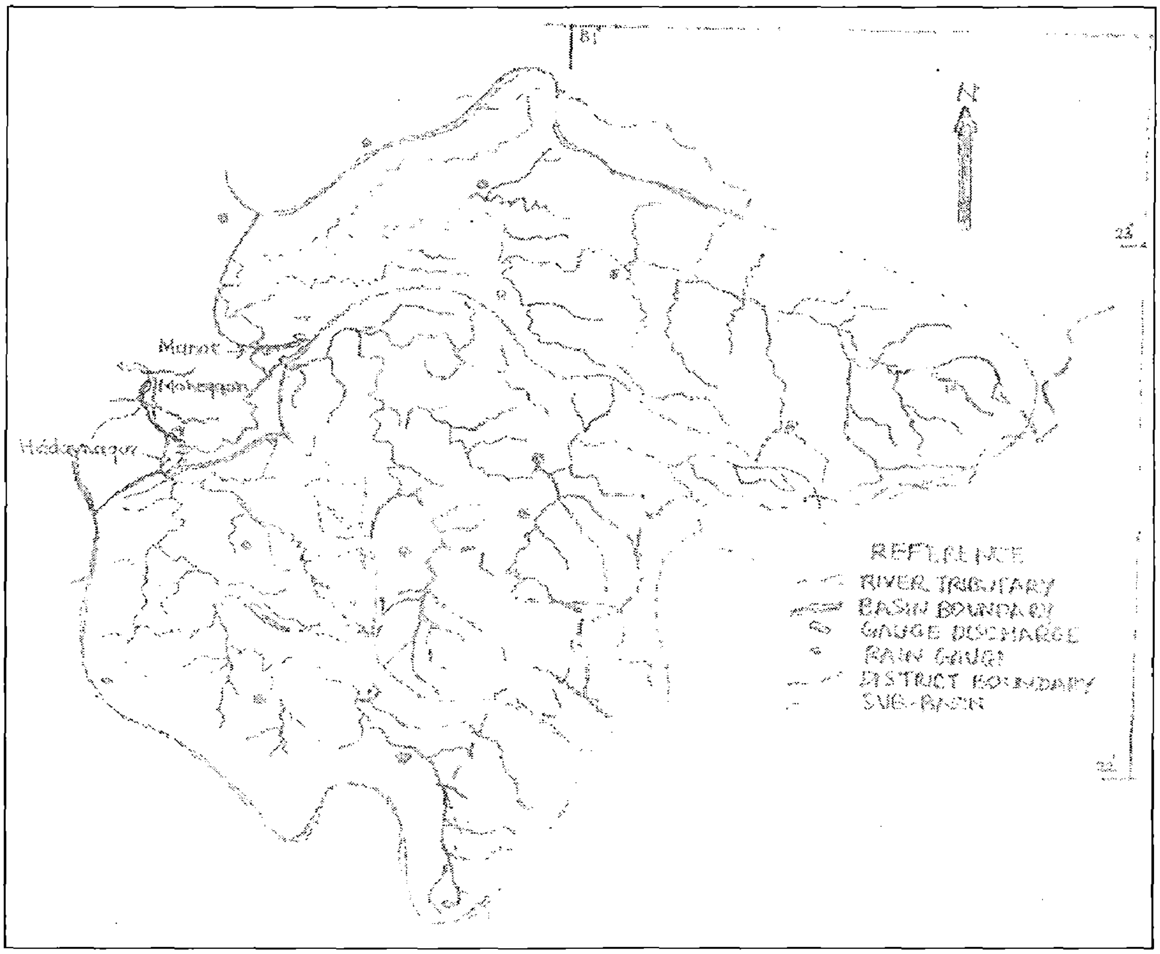


Fig 4.3 : Narmada Basin

4.2.3 Kalu catchment

The River Kalu (Fig. 4.4) is a tributary of the Ulhas River in the Thane district of Konkan region in Maharashtra. It originates near Harichandragad in Murbad Taluka of Thane district at an elevation of 1200 m above mean sea level and extends between east longitude 73° 36′ to 73° 49′ and north latitude 19° 17′ to 19° 26′. The steep terrain watershed (area = 224 km^2) experiences an average annual rainfall of 2450 mm, which varies from 2794 to 5080 mm in different parts of the watershed. Most of the rains are received during June to October. The existing crop pattern of the cultivation covers 46% paddy, 16% nanchani vari, 3% pulses and 35% grass. The catchment is covered with 50% thickly wooded forest, and 50% is cultivated area. A dam is proposed across the Kalu River near the village of Khapri about 31 km downstream of the origin to serve for irrigation as well as water supply.

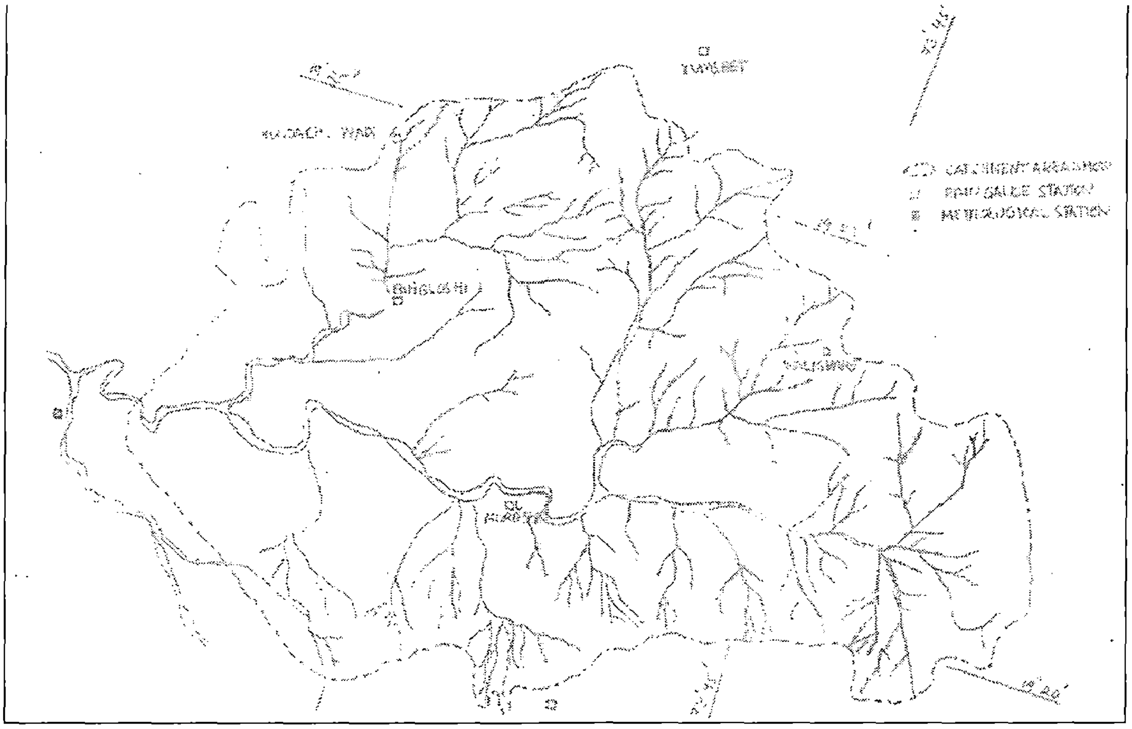


Fig 4.4 : Kalu watershed

4.2.4 Ghodahado catchment

Rushikulya is one of the major river in Orissa (Fig. 4.5). It originates from Rushamala hills of the eastern ghats in Phullabani district. It is 165 km long with 8900sq.km of catchment area. Ghodahado is a tributary of Rushikulya in Ganjam district near Degapahandi block. It extends between east longitude 84° 27' to 84° 40' and north latitude 19° 17' to 19° 28'. The watershed having area of 138 km^2 experiences an average annual rainfall of 1476 mm, having mean maximum summer temperature of 37°C and 10.3°C in winter. Most of the rainfall occurs during June to October. The watershed is situated in the East and South Eastern coastal Plain with hot and moist sub- humid climatic condition. The broad soil group of this area is Red soils, has blocky structures of either granular or sub granular geometry, and it is dominated by

Kaolinites and illites. The land use pattern of the watershed is 40% of forest area, permanent pastue is 3%, culturable waste is about 2%, non-agril land use is 5% and 50% of area is under net sown area.

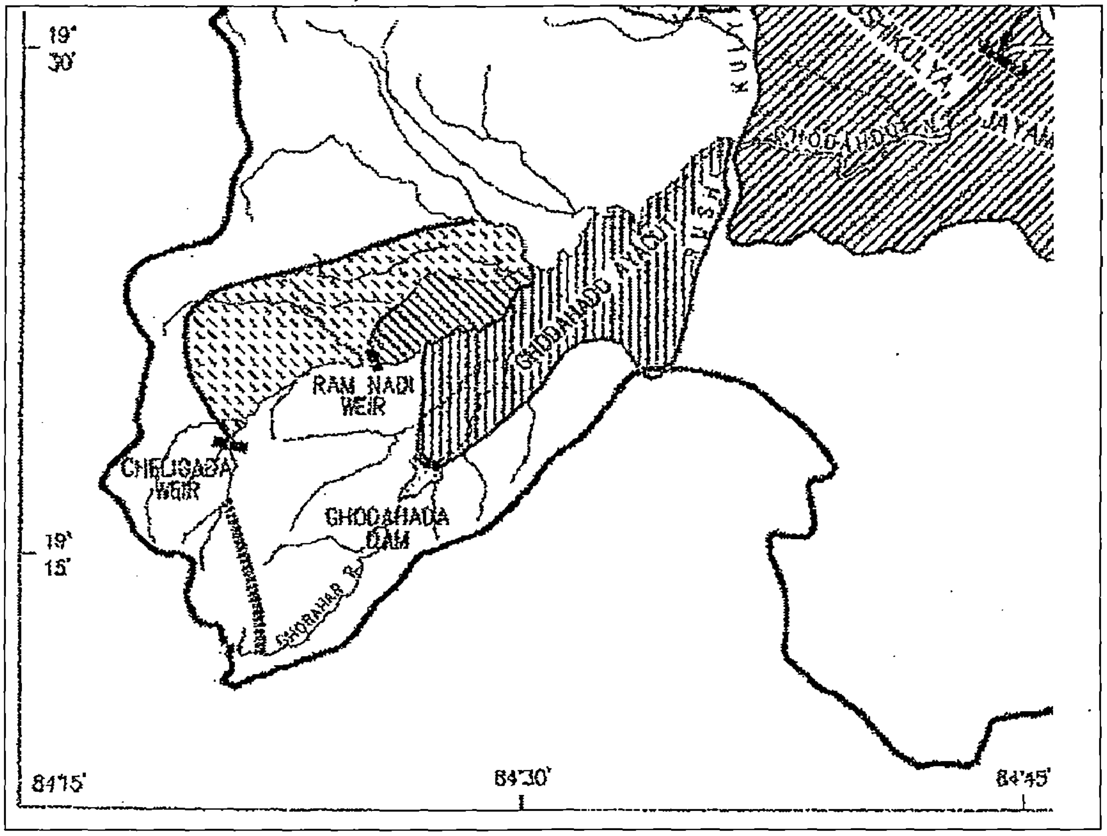


Fig 4.5 : Rushikulya basin showing Ghodahado catcment

4.3 DATA AVAILABILITY FOR OTHER CATCHMENTS

The data used in the study include daily rainfall, evaporation, and runoff for a total length ranging from 5 to 10 years. For Hemavati catchment, data of five years (1974 June –1979 May) were collected and for Kalu catchment, daily rainfall, evaporation, and stream gauge records of four years (1990–1993) were used, but only during monsoon period (June to November). Daily rainfall, evaporation, and runoff data for nine years (June 1980 –May 1990) were available for Manot catchments. For Mohegan catchments, the data were available for eight years (June 1981 – May 1989), and for Hirdaynagar catchments, these were for eight years (June 1981 – May 1989).

These are used in the study. For Ghodahado, the data available for June 1993–May 1996 and June 1987–May 1989 are used in the analysis. A time step of one day is used in simulation. Table 4.1 presents the area and the data length used in model calibration and validation for each catchment.

Table 4.1 : Data used in model calibration and validation

Catchment	Area (sq.km)	Data length	
		Calibration	Validation
Hemavati	600	1974 - 1977 (3 years)	1977 - 1979 (2 years).
Manot	5032	1981 - 1986 (5 years)	1986 - 1990 (4 years)
Hridaynagar	3370	1981 - 1986 (5 years)	1986 - 1990 (4 years)
Mohegaon	4661	1981 - 1985 (5 years)	1986 - 1989 (3 years)
Kalu	224	1990 - 1992 (3 years) Mansoon period	1993 (1 year) Mansoon period
Ghodahado	138	1993 - 1996 (3 years)	1987 - 1989 (2 year)

CHAPTER V: APPLICATION OF THE PROPOSED MODEL

The application of the proposed SCS – CN based long term hydrological simulation model (discussed in Chapter 3) requires daily data of rainfall, runoff and evaporation of the above described watersheds. The details of study watersheds and the records of hydrological data used in this analysis are presented in Chapter 4. The proposed model has four parameters CN, CN_d, K [T] and K_b[T]. The optimal estimates of model parameter were obtained by using the non- linear Marquardt algorithm (Mishra and Singh, 2003) coupled with trial and error.

5.1 PARAMETER ESTIMATION

Model parameters CN, CN_d, K [T] and K_b [T] can be determined using the Marquardt algorithm of constrained least squares. Although these parameters can be determined by trial and error for obtaining the maximum efficiency, it is also possible to derive these parameters physically or from rainfall– runoff data. As also discussed earlier in Chapter 4, the parameter CN represents the curve number in runoff simulation, assuming that the maximum pore space is available in the soil for water storage or retention on that day of simulation. The curve number can vary from 0 to 100. CN_d is the curve number for the drainage flow and it depends on the soil moisture storage. It also varies from 0 to 100. Parameter K represents the storage coefficient of the surface runoff hydrograph and is analogous to the time lag of the watershed. It can also be derived from the rainfall–runoff data by plotting them on a semi-logarithmic paper. The slope of the fit represents K. The rainfall–runoff data set selected for the derivation should correspond to high rainfall–runoff events excluding base flow. Similarly, parameter K_b, which represents the storage coefficient of the base flow appearing at the outlet of the watershed, can be derived for low rainfall–runoff events or using other methods. For describing the range of variation of these parameters, the lower bound is taken as zero, because all the parameters are physically non-negative. The upper bound values are, however, decided from the trial runs whether the estimated parameter values are well within the supplied range. If the estimated parameter value corresponds to the upper bound of the described range, the upper bound is increased to the extent that the estimate falls in the prescribed range. The ranges/values of parameters selected for trials and optimization are given in Table 5.1.

Table 5.1 Ranges and initial estimates of model parameter

Parameters		CN	CN_d	K	K_b
Range		0.001-99.999	0.001-99.999	0.001 - 5.0	1 - 360
Initial value	Ramganga	99	80	0.01	20
	Hemavati	98.9	95	0.1	30.5
	Manot	85	76.9	0.91	15
	Hridaynagar	65	60	0.9	21
	Mohegaon	80	70	0.80	20
	Kalu	85	70	0.05	10.5
	Ghodahado	90	80	0.1	10

5.2 MODEL EFFICIENCY

The efficiency (Nash and Sutcliffe, 1970) of both the models is computed using

$$\text{Efficiency} = \left[1-\left(\frac{RV}{IV}\right)\right]\times 100 \tag{5.1}$$

where

$$RV = \sum_{i=1}^{n}\left(Q_i - \hat{Q}_i\right)^2 \tag{5.2a}$$

$$IV = \sum_{i=1}^{n}\left(Q_i - \bar{Q}_i\right)^2 \tag{5.2b}$$

where RV=remaining variance; IV=initial variance; Q_i =observed runoff for *i*th day; Q_i=computed runoff for *i*th day; n =total number of observations; and Q = overall mean daily runoff. Efficiency is used for evaluating the model performance. Efficiency varies at the scale of 0 to 100. It can also assume a negative value if RV > IV, implying that the variance in the observed and computed values is greater than the model variance. The efficiency of 100 implies that the computed values are the same as the observed ones, which is the perfect fit.

The relative error (RE) is also computed to see the deviation between the observed and simulated runoff, with respect to the observed runoff and it is determined as:

$$\text{Relative error RE (\%)} = \left(\frac{Q_{obs} - Q_{comp}}{Q_{obs}}\right) \times 100 \qquad (5.3)$$

Here, Q_{obs} = observed runoff and Q_{comp} = simulated runoff. The higher RE is indicative of greater deviation from the observed, and vice versa.

5.3 MODEL CALIBRATION AND VALIDATION FOR RAMGANGA CATCHMENT

For model calibration and validation, the available five years data set of Ramganga catchment is split into two parts. For calibration three years (1985 – 1988) of data have been considered. The estimated values of the four parameters (CN, CN_d, K and K_b) along with their initial values (Table 5.1) and model efficiencies in calibration are given in the Table 5.2. It is apparent from the table that the values of the parameters CN, CN_d, and K decrease as the number of years data increase from 1- 3 years, and vice versa holds for K_b. From the results due to 3-year dataset, it is also seen that the CN value of the watershed is of the order of 80, indicating a good runoff producing watershed; CN_d of the order of 74, which is lower than CN indicating less baseflow production potential than the runoff generation from rainfall; K of the order of 2 days, a reasonable value of the lag in runoff hydrograph for Ramganga catchment (Cattchment area = 3134 sq. km); and K_b of the order of 30 days, which is also reasonable for mid-size watershed.

The resulting efficiencies are seen to vary from 81.82 to 73.62%, as the number of years of data varies from 1- 3 years. Though the efficiencies show a decreasing trend with the increase in the data length, these are indicative of adequate and satisfactory performance of the proposed model in calibration. Then taking the initial and final parameter values corresponding to three years of data in model calibration (Tables 5.1 and 5.2), the model was tested on the remaining two years (1988- 90) data. The resulting efficiency of two year data is 75.46% which indicates a satisfactory model performance. The daily variation of observed and computed runoff along with the rainfall is depicted in Figs. 5.1 – 5.3 for calibration and in Figs. 5.4 – 5.5 for validation.

As seen from Figs. 5.1-5.3, the computed runoff fairly simulated the observed runoff, except for a few peaks in years 1985 (Fig. 5.1) and 1986 (Fig. 5.2). From these three figures, it is interesting to note that the computed non-monsoon flows closely follow the trend exhibited by the observed runoff, indicating the satisfactory performance of the proposed SCS-CN-based baseflow model (Chapter 3).

Similar to the above trends can be observed from the Figs. 5.4 and 5.5 that show the validation results of the proposed model. Except for two largest runoff peaks in both the years 1988 (Fig. 5.4) and 1989 (Fig. 5.5), the computed runoff closely matches the observed runoff values, indicating again that the model performs satisfactorily on the data of Ramganga catchment, and there, is suitable for this catchment.

Table 5.2 Parameters from simulation of different time periods

No of years of data used in simulation	Parameters (Calibration)				
	CN	CN_d	K	K_b	Efficiency (%)
	Initial Estimate				
	99	80	0.01	20	
	Final Estimate				
1	83.88	76.82	2.29	24.32	81.82
2	81.73	76.62	2.5	30.55	76.95
3*	80.38	74.4	2.25	30.99	73.62

*used as calibration dataset

To further test the proposed model applicability to Ramganga catchment, the percent relative errors (equation 5.3) were computed and these are shown in Table 5.3. In this table, the resulting positive and negative values of the relative errors, respectively, show the underestimation and over-estimation of the yearly runoff by the proposed model. Apparently, except for 1987-88, the model has under-estimated the yearly runoff in all the years. The relative error values are seen to vary from 8.92 to ±29.66%, which indicate a reasonably satisfactory performance of the proposed model in yearly runoff computation.

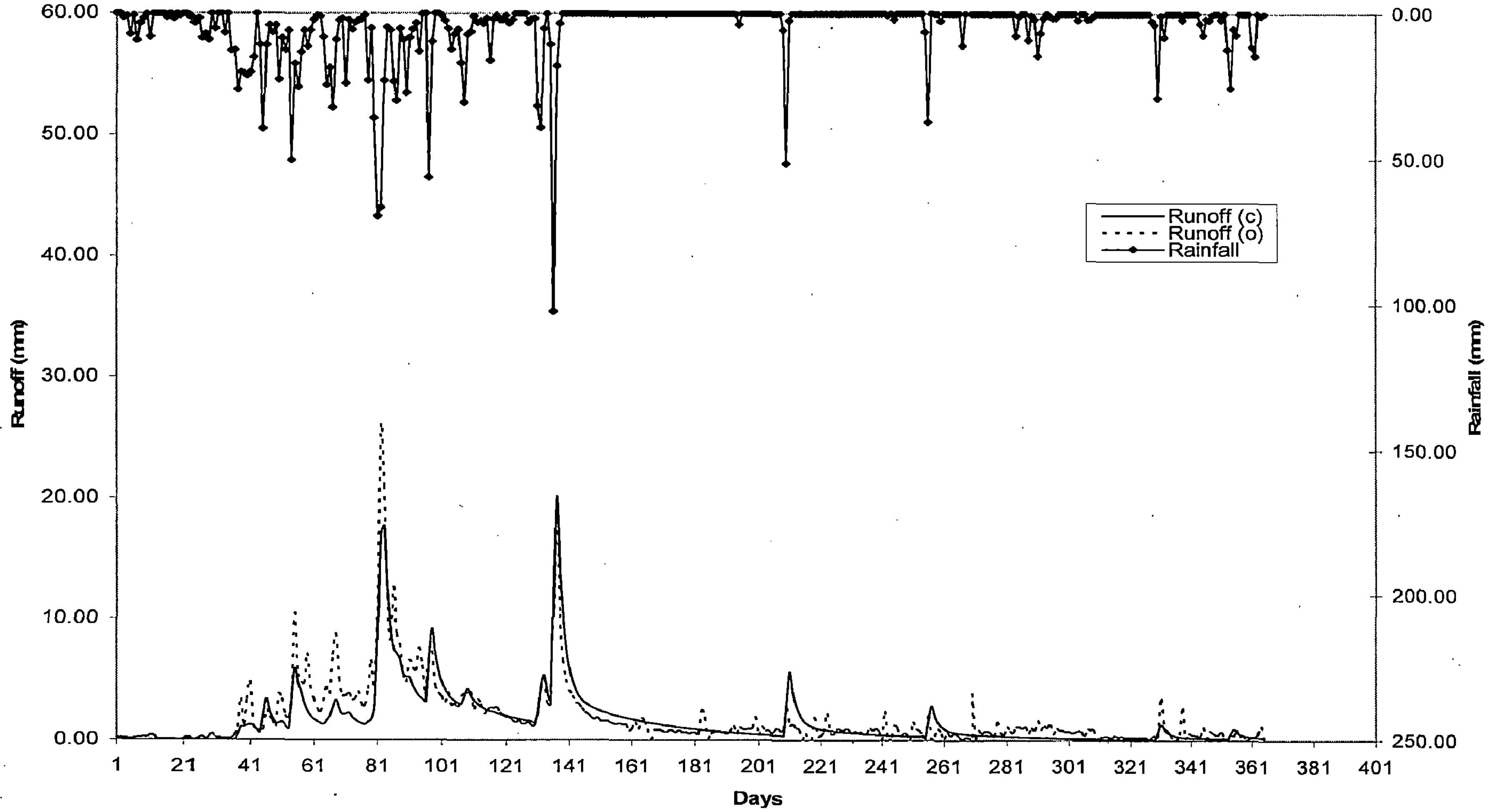


Fig : 5.1 Model calibration for Ramganga catchment (June 1985- May 1986) Day 1 represents June 1, 1985

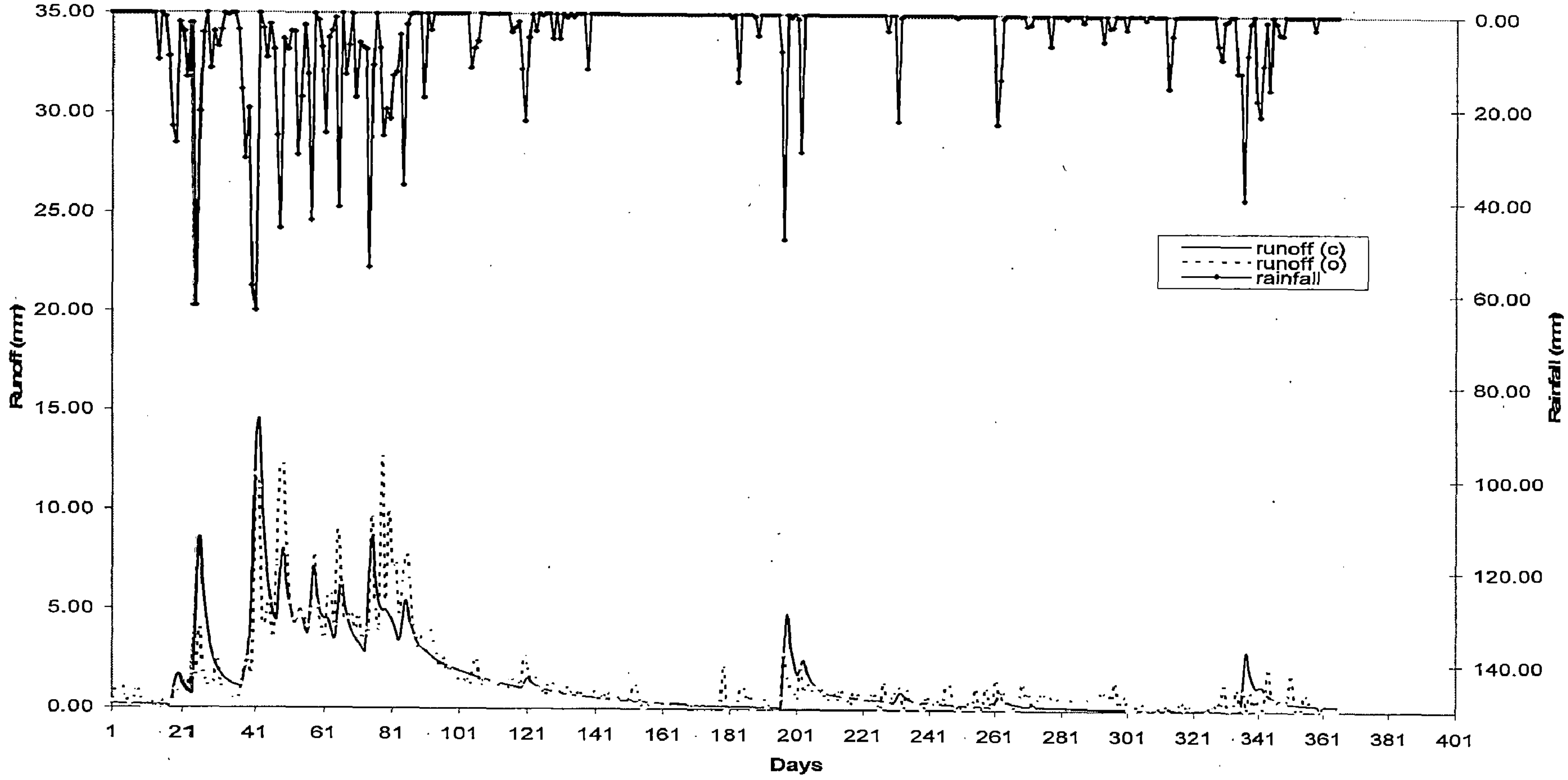


Fig : 5.2 Model calibration for Ramganga catchment (June 1986- May 1987) Day 1 represents June 1, 1986

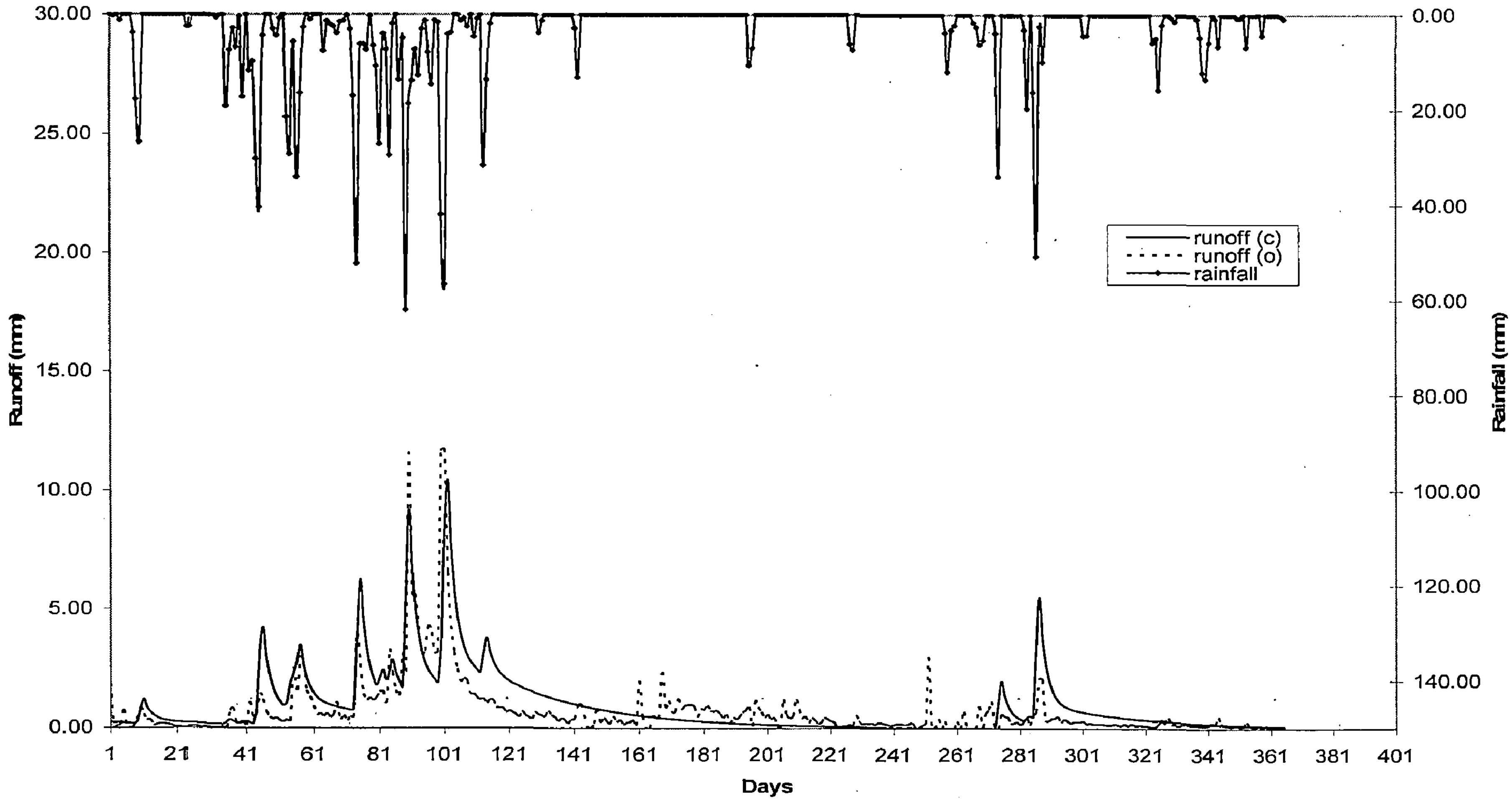


Fig : 5.3 Model calibration for Ramganga catchment (June 1987- May 1988) Day 1 represents June 1, 1987

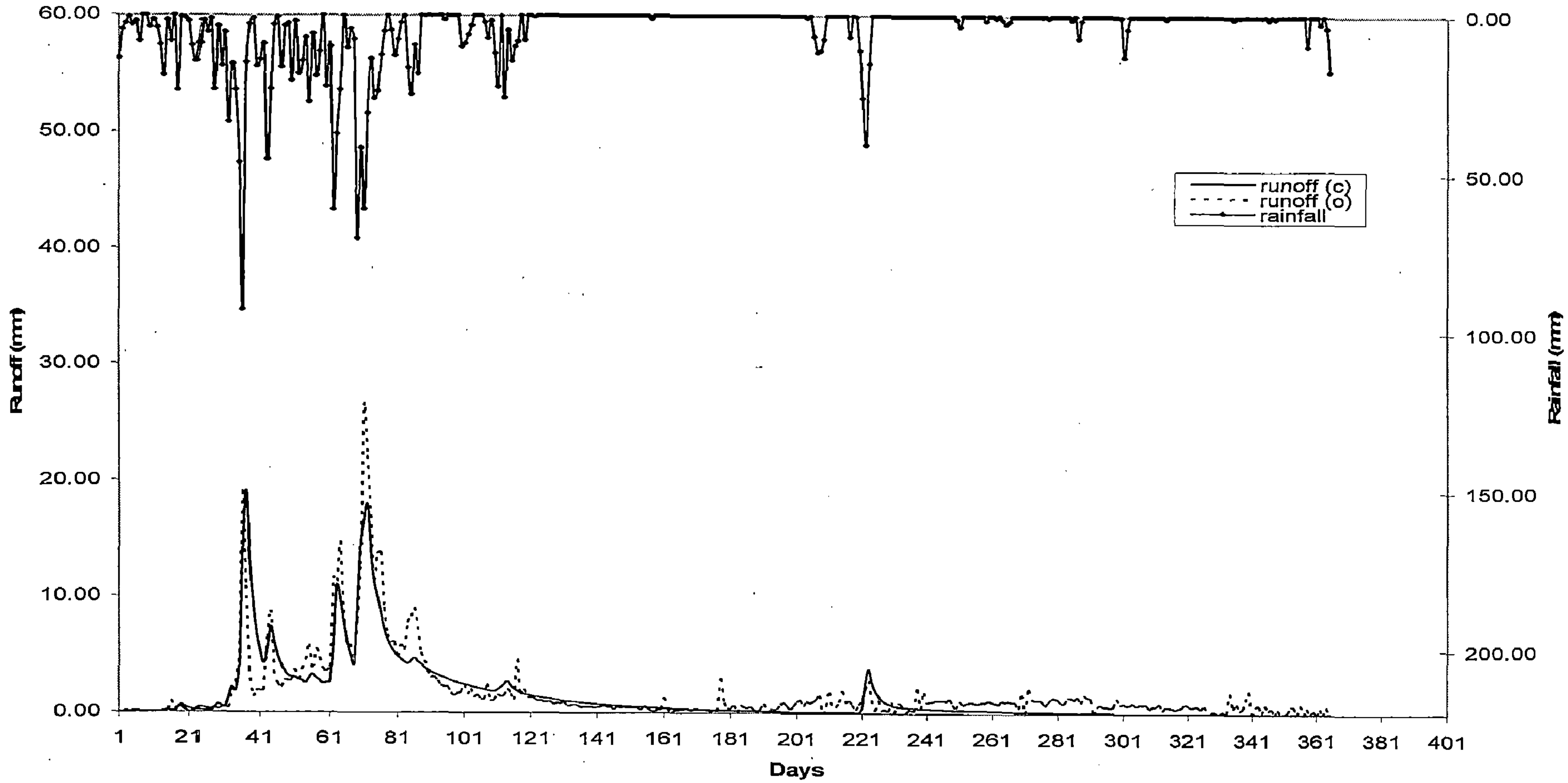


Fig : 5.4 Model validation for Ramganga catchment (June 1988- May 1989) Day 1 represents June 1, 1988

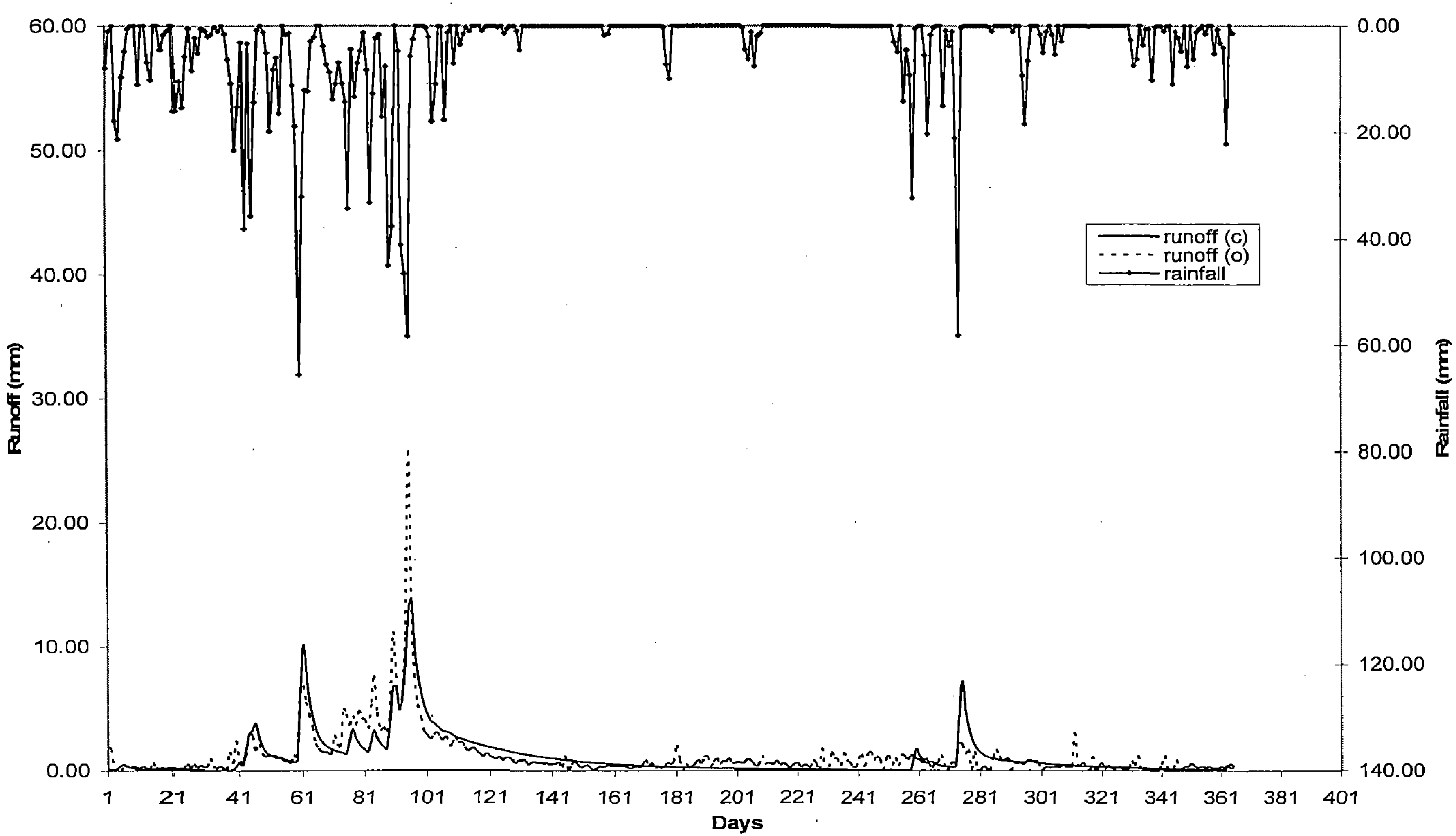


Fig : 5.5 Model validation for Ramganga catchment (June 1989- May 1990) Day 1 represents June 1, 1989

Table 5.3 Observed and simulated runoff and computed relative error

Year	Observed runoff (mm)	Simulated runoff (mm)	Relative Error (%)
1985 – 86	675.63	585.75	13.30
1986 – 87	539.36	491.26	8.92
1987 – 88	270.54	350.78	-29.66
1988 – 89	641.42	524.66	18.20
1989 – 90	443.40	397.71	10.30
Average	514.07	470.03	8.57

5.4 SENSITIVITY ANALYSIS

The purpose of such an analysis lies in distinguishing the parameters that are more sensitive, for their cautious and judicious derivation and employment in the field. Therefore, to assess the sensitivity of the above described four parameters of the model, a sensitivity analysis is carried out. To this end, the parameters calibrated for the year 1985 – 88 were varied for evaluating the impact of their variation on the model performance described above in terms of efficiency resulting from the model application to calibration dataset. For sensitivity, all the parameters were varied from ±5% to ±30%, and the corresponding efficiency computed. The changes in efficiency due to variation in the four parameters are shown in Figures. 5.6 to 5.9, and these are discussed below.

An increase in the value of parameter CN by ±30% of the calibrated value (Table 5.2) results into efficiencies varying in the range of 73.678% to 66.717% (Fig. 5.6). Notably, there is not much change in the efficiency as the parameter (CN) value increased upto 20%, but further increasing in the value shows a sudden drop in the efficiency (73.66 to 66.72%). On the other hand, the second parameter CN_d appears to be less sensitive than CN (Fig.5.7), for the efficiency varies a little with the change in the parameter value by the same extent. It can be seen that the variation of CN_d from ±5 to ±30% leads to change in efficiencies in the range 73.68 – 73.66%, which exhibits a much lesser range than that due to CN. From Figure 5.8, it is apparent that the efficiency does not change significantly, as these change from 73.68% to 73.56% only, with an increase or decreased in the K values by the same (as above) extent.

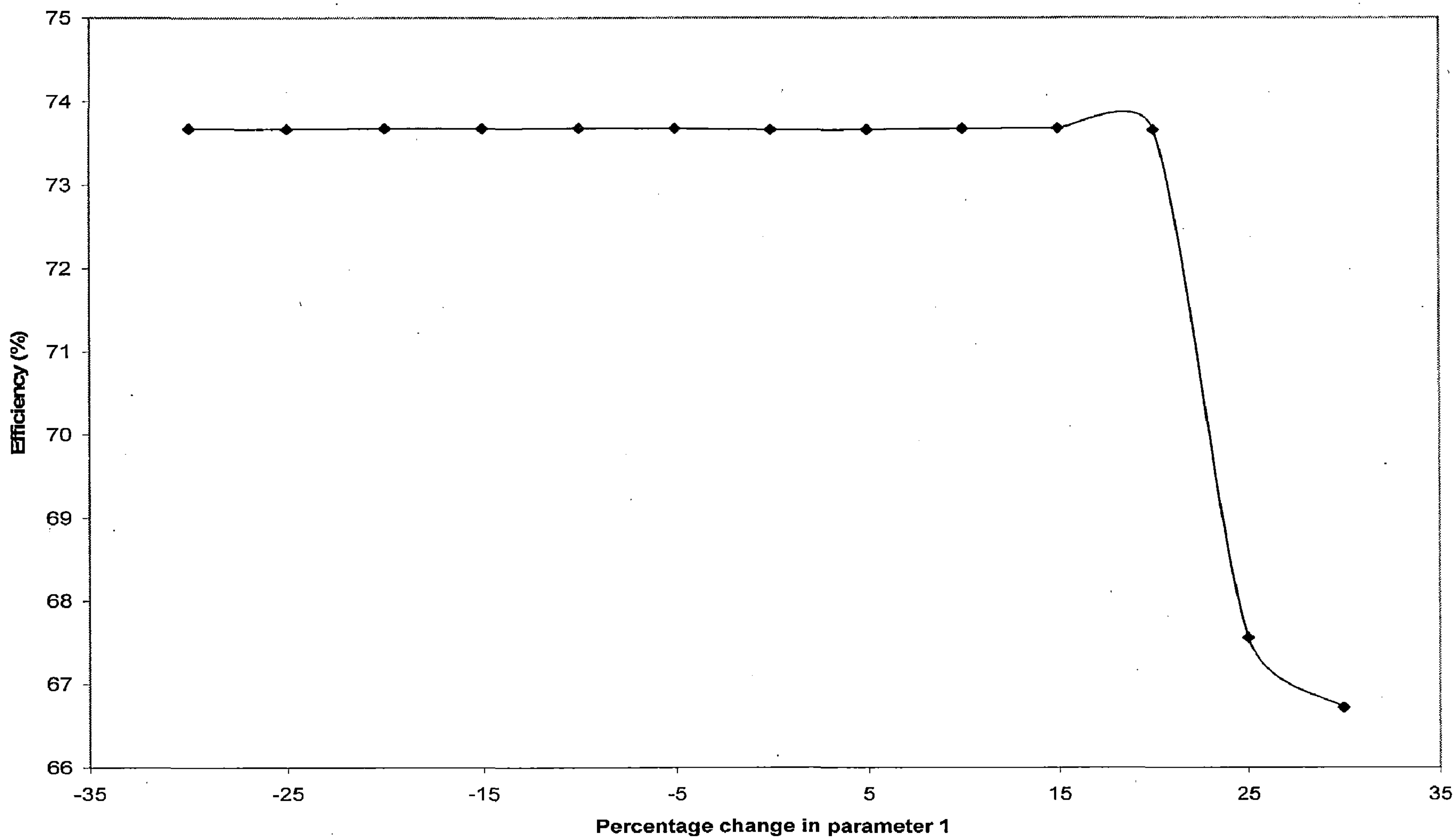


Fig: 5. 6 Sensitivity of model parameter CN (Ramganga catchment)

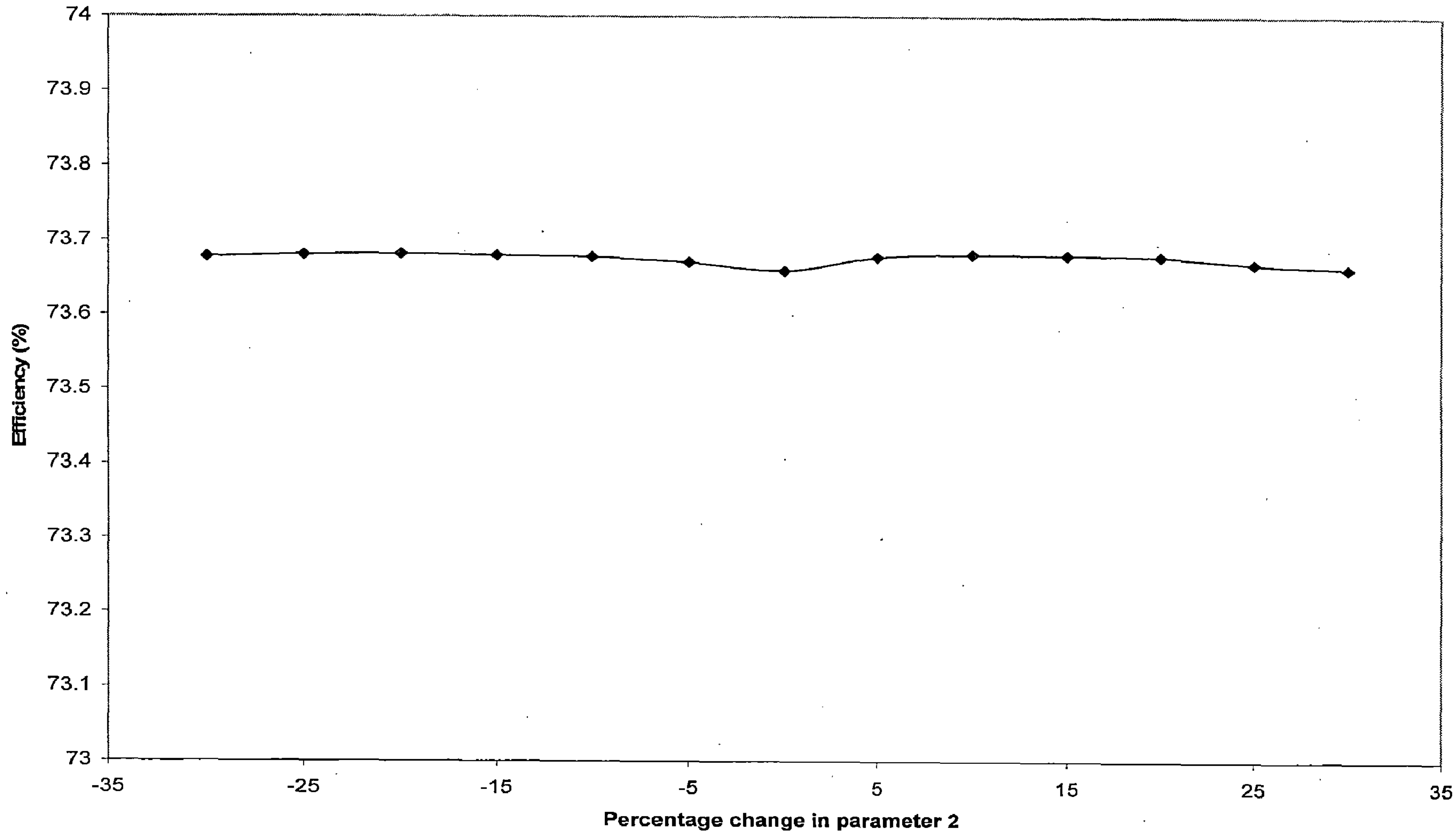


Fig: 5.7 Sensitivity of model parameter CN_d (Ramganga catchment)

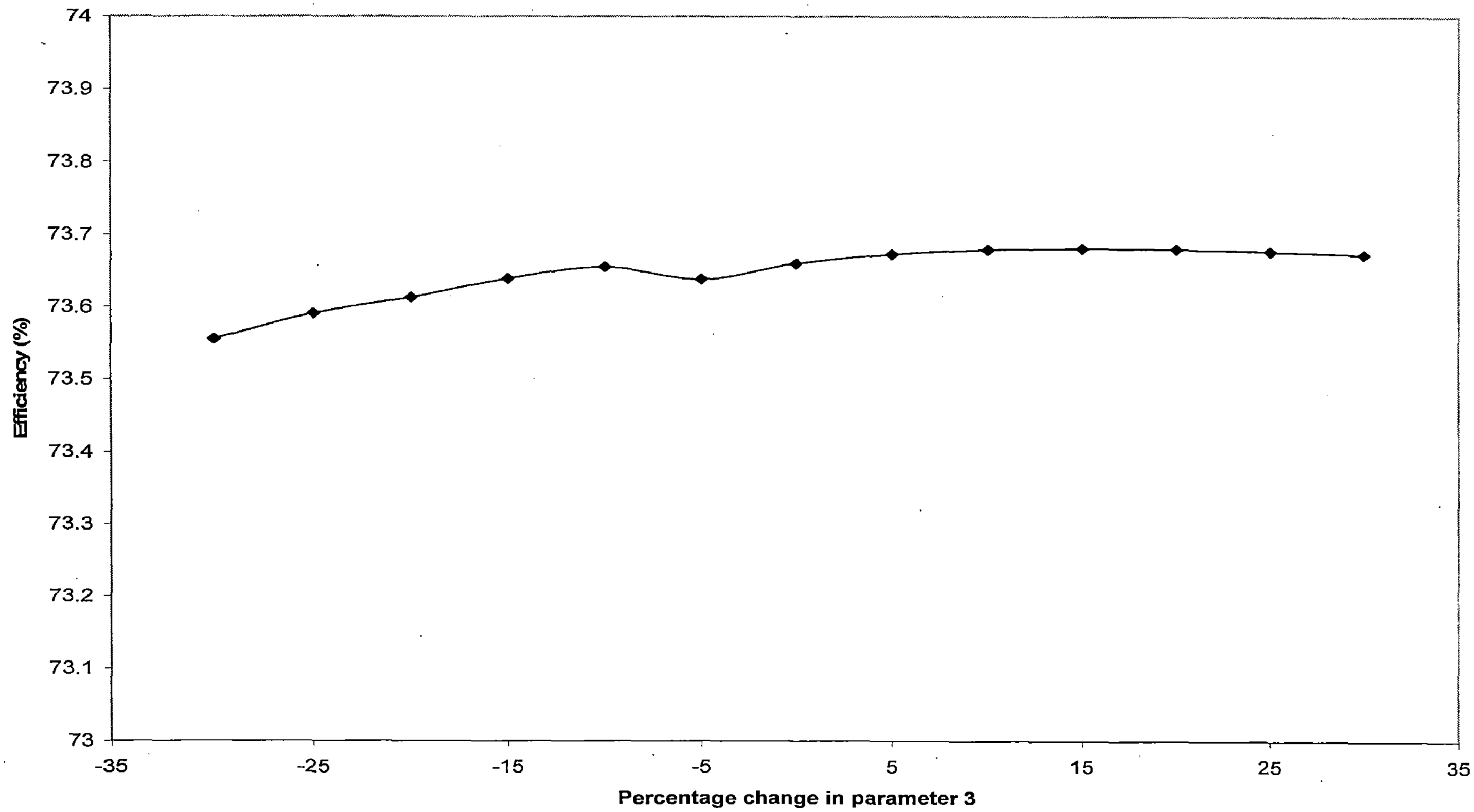


Fig: 5.8 Sensitivity of model parameter K (Ramganga catchment)

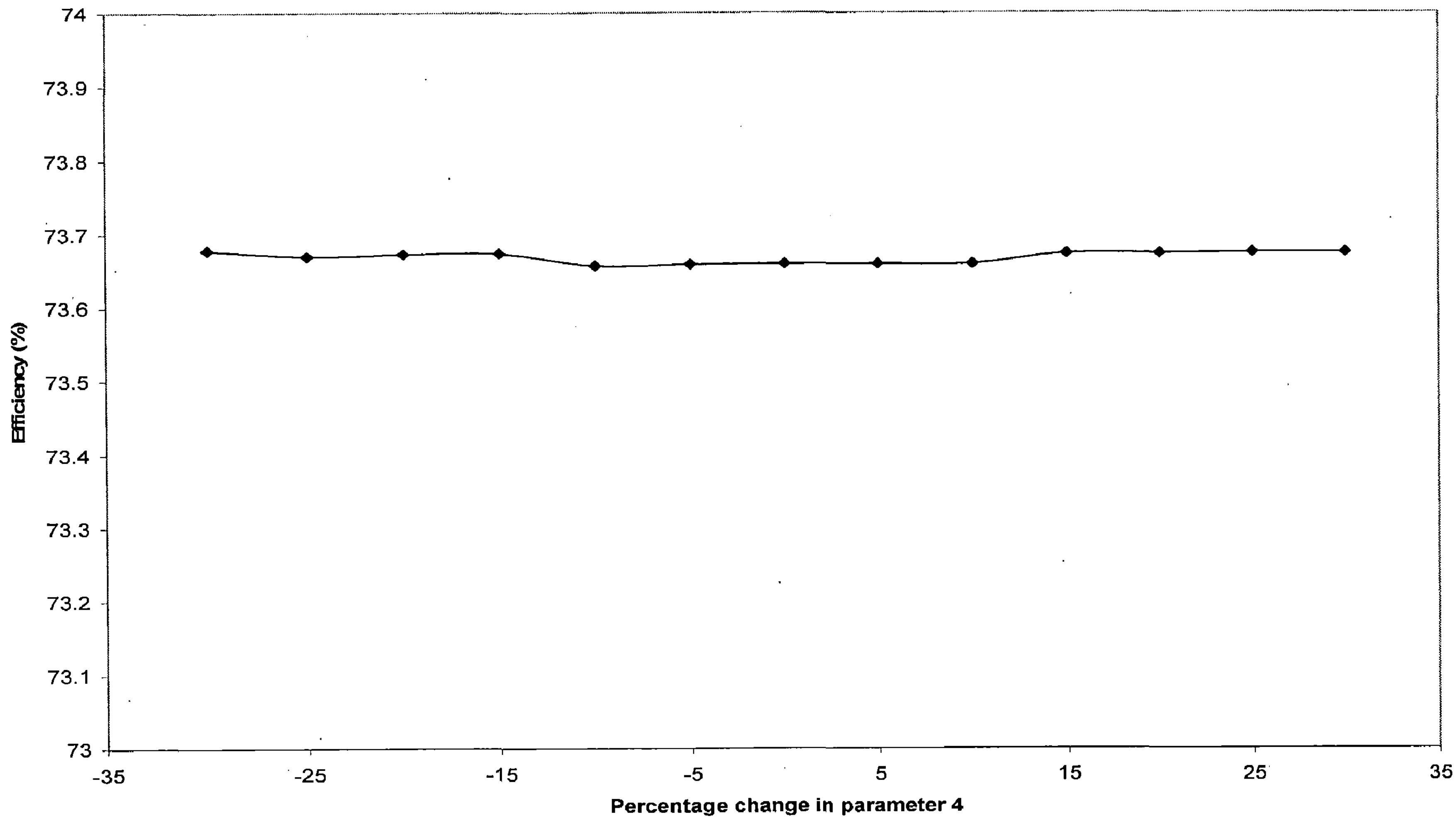


Fig: 5.9 Sensitivity of model parameter K_b (Ramganga catchment)

Thus the parameter is less sensitive than CN. A reduction or increase in K implies that the computed rainfall- excess is allowed to reach the outlet earlier or later than required and it results in the decrease in the model efficiency. Figure 5.9 shows a variation in efficiency (from 73.678% to 73.66%) with an increase in the value of parameter K_b by ±30. Here also the graph shows the less sensitivity of the parameter K_b than CN and K. From all the figures (5.6- 5.9) it is clear that the four parameters are sensitive in the following order: CN > K > CN_d > K_b.

5.5 MODEL TESTING ON OTHER WATERSHEDS

5.5.1 Model Calibration

The proposed model is calibrated using rainfall, evaporation, and runoff data of the other watersheds located in different hydro-meteorological conditions (Chapter 4). Three years of data were used for Hemavati (1974-1977), Kalu (1990-1992) and Ghodahado (1993- 1996) catchments. For Narmada basin nine years of data were available, out of which five years of data of Manot, Hridaynagar and Mohegaon sites were used in calibration. The values of parameters computed in calibration for all the watersheds are given in Table 5.4, and discussed below.

Table 5.4 Estimates of model parameters

Parameters	CN	CN_d	K	K_b
Hemavati	91.65	85.57	2.24	50.35
Manot	88.26	74.03	0.57	21.23
Hridaynagar	72.31	37.59	2.06	29.31
Mohegaon	76.6	44.71	0.1	21.73
Kalu	85.49	72.85	0.66	28.89
Ghodahado	82.71	78.07	3.20	37.16

The optimal estimates of model parameter were obtained by using non-linear Marquardt algorithm (Mishra and Singh, 2003) coupled with trial and error. The estimated values of the four parameters (CN, CN_d, K and K_b) along with their initial values (Table 5.1) and the estimated values of model parameters in calibration for other watersheds are given in Table 5.4. It is apparent from the table that the values of the parameter CN varies in the range of 70 to 90 for all the six catchments. This demarcates a maximum value of 91.65 for Hemavati which interprets it to be a good runoff producing watershed, whereas the minimum value occurs at Hridaynagar with a value of 72.31 representing it to be a less runoff producing watershed. Similarly, the CN_d value of the watershed varies in the range from 35 to 85 for all the considered catchments. The maximum value of 85.57 for Hemavati indicates good base flow production potential, and on the other hand, the Catchment which produces the lowest CN_d value of 37.59 is Hridaynagar shows less baseflow production potential. K-values are seen to vary from 3 hrs to 3day, whereas the time of travel for base flow, K_b varies in the order of 20 days to 50 days, which is also reasonable for mid-size watersheds.

Table 5.5 shows the resulting efficiencies along with runoff coefficients for each catchment. It is seen that Hridayanagar and Mohegaon catchments show the least runoff coefficient of 0.25 and 0.29, indicating them to be dry catchments (Gen et. al.;1997) whereas Hemavati and Kalu can be classified as high runoff producing catchments with their coefficient values of 0.8 and 0.91, respectively. The runoff coefficients for Manot and Ghodahado are 0.45 and 0.47 respectively, describing them to lie in the intermediate category of dry and wet. The model yields maximum efficiency of 83.27% in Hemavati cathment whereas Hridayanagar catchment produces the least efficiency of 42.08%. The other catchments like Manot, Mohegaon, Kalu and Ghodahado exhibit 60.75, 62.72, 62.85 and 59.35% efficiencies. For each catchment the simulated hydrographs (for calibration) depicting rainfall, runoff computed and observed are shown in Figs 5.10- 5.15.

Table 5.5 Model efficiencies in calibration and runoff co-efficient

Catchment	Data length	Efficiency (%)	Runoff coefficient
Hemavati	1974 - 1977 (3 years)	83.27	0.80
Manot	1981 - 1986 (5 years)	60.75	0.45
Hridaynagar	1981 - 1986 (5 years)	42.08	0.25
Mohegaon	1981 - 1986 (5 years)	62.72	0.29
Kalu	1990 - 1992 (3 years) Monsoon period	62.85	0.91
Ghodahado	1993 - 1996 (3 years)	59.35	0.47

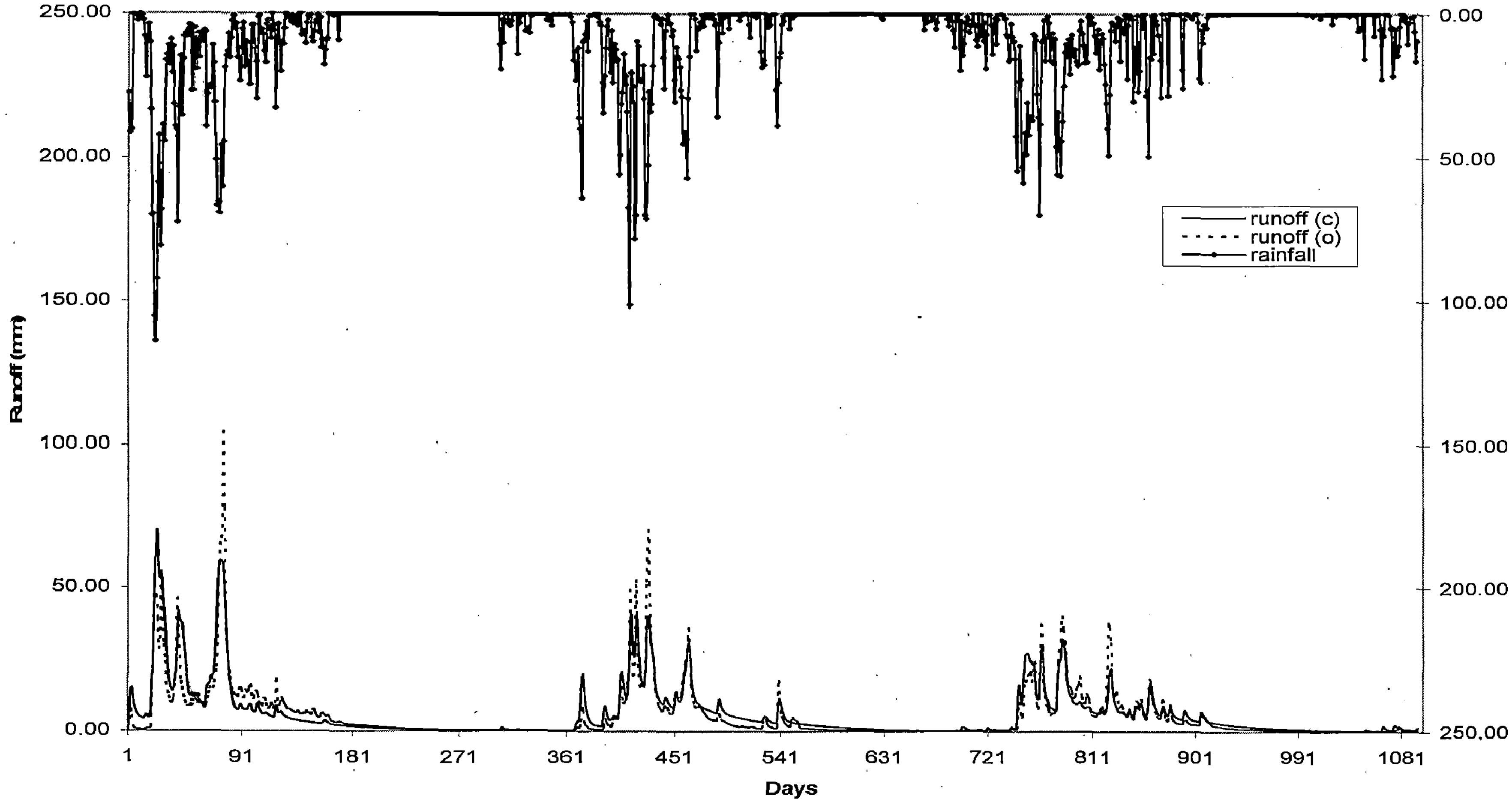


Fig: 5.10 Daily variation of Rainfall, observed runoff and simulated runoff in Hemavati catchment (calibration)

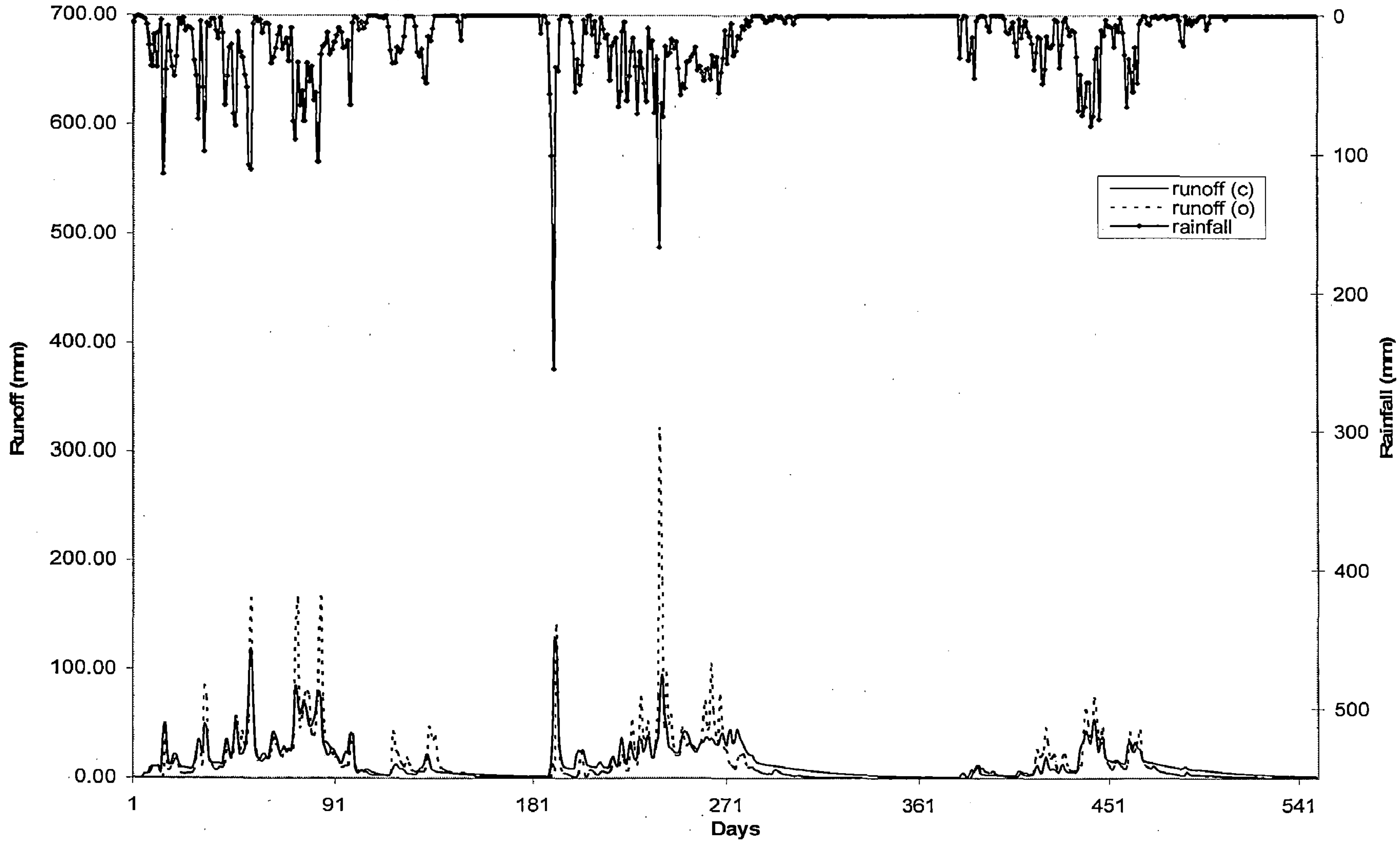


Fig 5.11 Daily variation of Rainfall, observed runoff and simulated runoff in Kalu catchment (calibration)

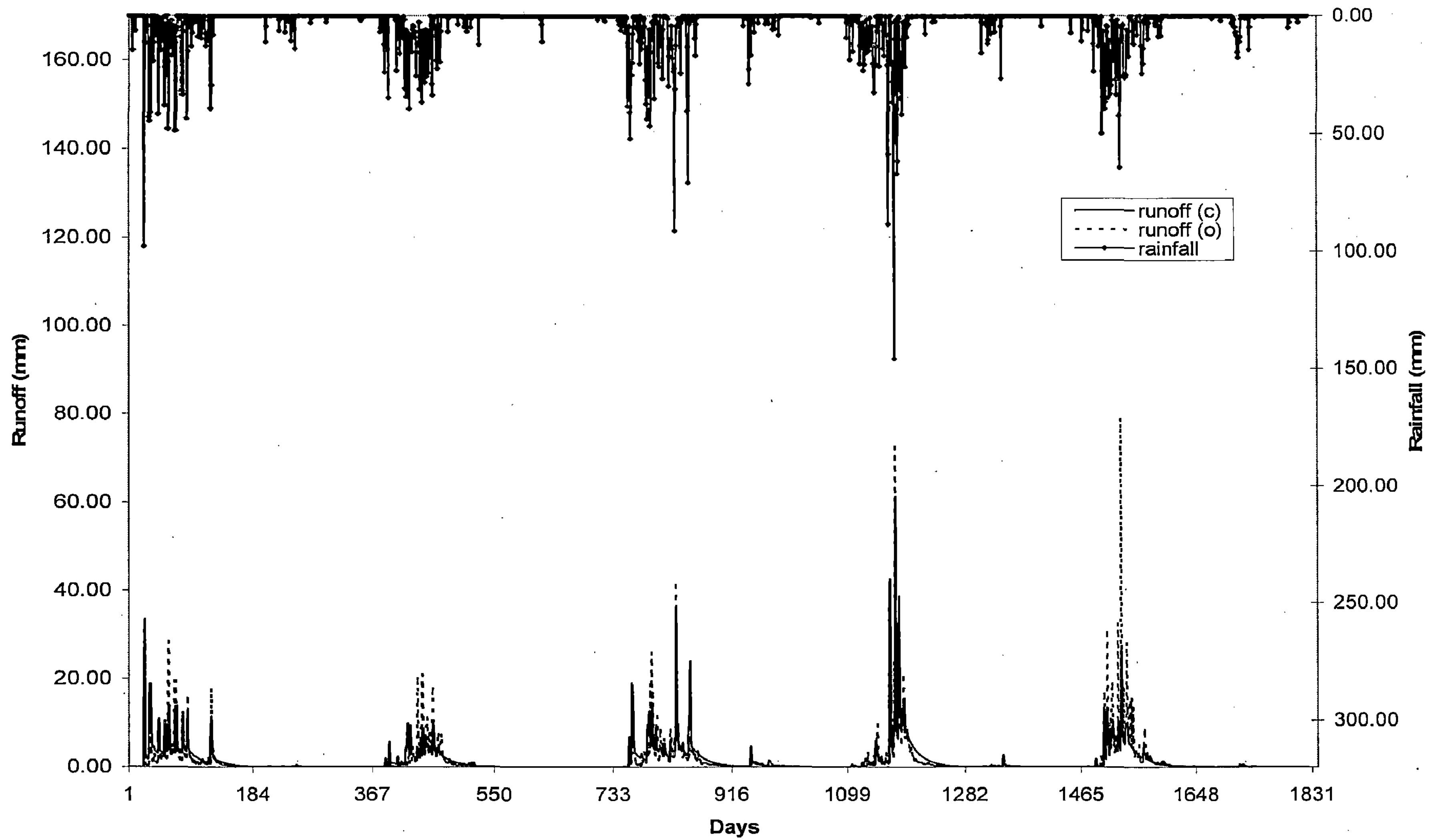


Fig : 5.12 Daily variation of Rainfall, observed runoff and simulated runoff in Manot catchment (calibration)

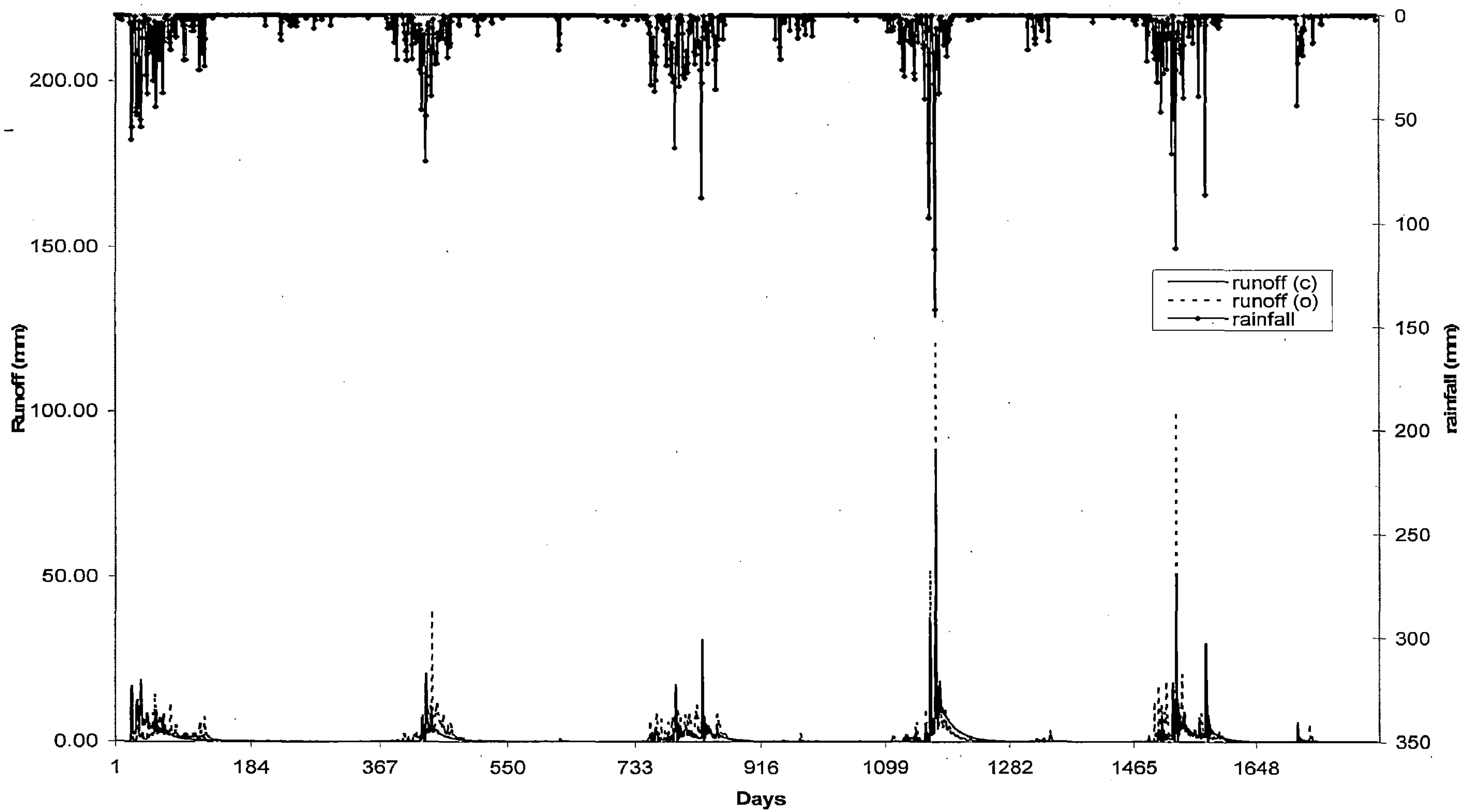


Fig : 5.13 Daily variation of Rainfall, observed runoff and simulated runoff in Mohegaon catchment (calibration)

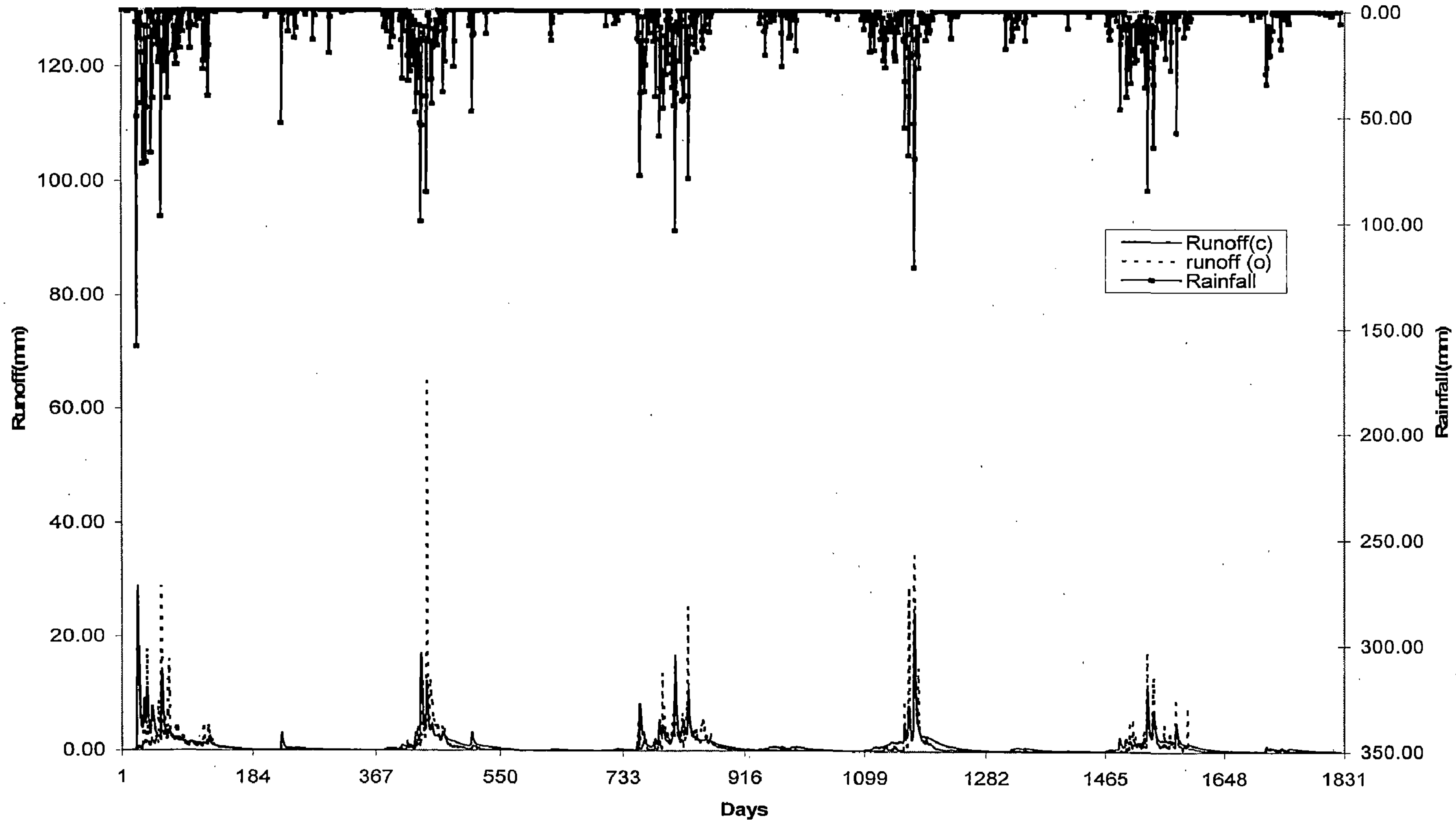


Fig 5.14 Daily variation of Rainfall, observed runoff and simulated runoff in Hridayanagar (calibration)

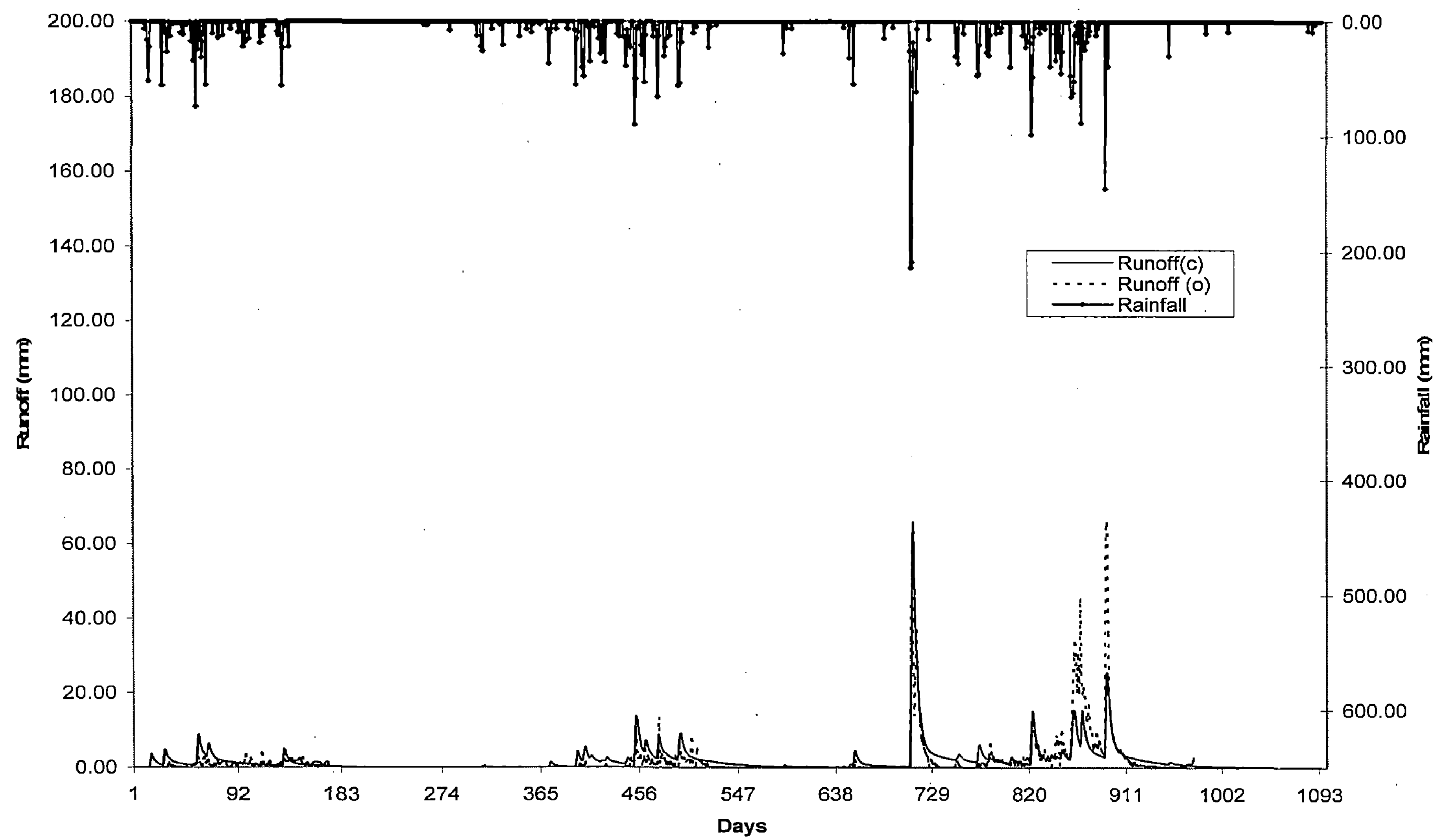


Fig : 5.15 Daily variation of Rainfall, observed runoff and simulated runoff in Ghodahado catchment (calibration)

Fig. 5.10 shows the daily variation of observed and computed runoff along with the rainfall for the Hemavati catchment from 1974-77. From this figure it can be interpreted that the observed runoff and the computed runoff follow a close trend except for the deviation at peaks of the hydrographs. Similar inferences can be drawn from the graphs for other catchments. From the figures, it is noted that the model underestimates the peaks, like for Hemavati in the year 1974, for Kalu in the year 1991, for Manot in year 1985 and for Mohegaon in year 1985. Since the proposed model accounts for baseflow, it appears that the model is not suitable for prediction of high flows.

5.5.2 Model Validation

Taking the parameter values corresponding to the data available for different catchments (Table 5.4) in model calibration, the model was tested on the remaining years data for the corresponding catchments. Two years of data were used for Hemavati and Ghodahado catchments for validation. From the available nine-year dataset for Narmada River at Manot, Hridaynagar and Mohegaon the remaining four years of data were used for validation. For Kalu catchments, four years data for monsoon period only were available for the analysis, out of which three years ware used for calibration and one year for validation. The efficiencies resulting from model application to the remaining data of different catchments are given in Table 5.6, which indicates a satisfactory model performance only on those watersheds for which runoff coefficient value is high, such as Hemawati and Kalu watersheds. On other watersheds, the model performs poorly.

Table 5.6 Model efficiencies in validation and runoff co-efficient

Catchment	Data length	Efficiency (%)	Runoff coefficient
Hemavati	1977 - 1979 (2 years)	84.82	0.83
Manot	1986 - 1990 (4 years)	54.06	0.39
Hridaynagar	1986 - 1990 (4 years)	34.88	0.20
Mohegaon	1986 - 1989 (3 years)	42.17	0.23
Kalu	1993 (1 year) Monsoon period	80.77	0.97
Ghodahado	1987 - 1989 (2 year)	32.31	0.52

Like in calibration, the catchments having higher runoff coefficient produces higher efficiencies in validation also. Hemavati catchment shows a better efficiency (=84.82%) than others. Kalu stands next to Hemavati in efficiency which is 80.77%, indiacting satisfactory model performance. But in case of Manot, Mohegaon, Hridaynagar and Ghodahado catchments, the efficiencies are low, viz., 54.06, 42.17, 34.88 and 32.31%, respectively, as shown in Table 5.6. The model efficiencies being too low on these catchments indicates its non-suitability to these watersheds. Alternatively, the proposed model is not applicable to watersheds of low runoff production potential.

For each catchment, the validation results are shown in Figs. 5.16 – 5.21 for the respective datasets (Table 5.6). It is seen that even on the Hemavati and Kalu watersheds, the model underestimated the peaks. Fig. 5.16 shows the daily variation of observed and computed runoff along with the rainfall for the Hemavati catchment from 1977-79. From this figure it is seen that the observed runoff and the computed runoff follow a close trend except for the deviation at peaks of the hydrographs in year 1977 and 1979. Similar inferences can be drawn from the graphs for other catchments. From the Figures 5.17- 5.19 it is noted that the model underestimates the peaks, such as for Hemavati and Kalu, but for Manot and Mohegaon, there exists much deviation in peak estimation in years 1986- 1988.

To further test the proposed model applicability, the percent relative errors (equation 5.3) for six watersheds were computed and these are shown in Table 5.7. The table shows yearly rainfall along with the observed and calculated runoff for the described watersheds. The average annual value of relative error ranges from 17.71% to ±24.61%, with an average value of ±3.57% for Hemavati catchment over the study period of five years. This watershed experienced an average annual rainfall of 2854 mm which varies from 2651 mm to 3064 mm in different years. The average annual runoff calculated is 2312 mm whereas the observed runoff is 2233 mm. Apparently, except for 1977-78, the model has over-estimated the yearly runoff in all the years. For Kalu watershed the average annual rainfall is 2944 mm which varies from 1903 mm to 3355 mm in these three years and the average annual runoff calculated is 2694 mm and observed runoff is 2930 mm annually. The relative error values are seen to vary from 13.5 to ±7.09% with an average value of 8.06%, which indicate a reasonably

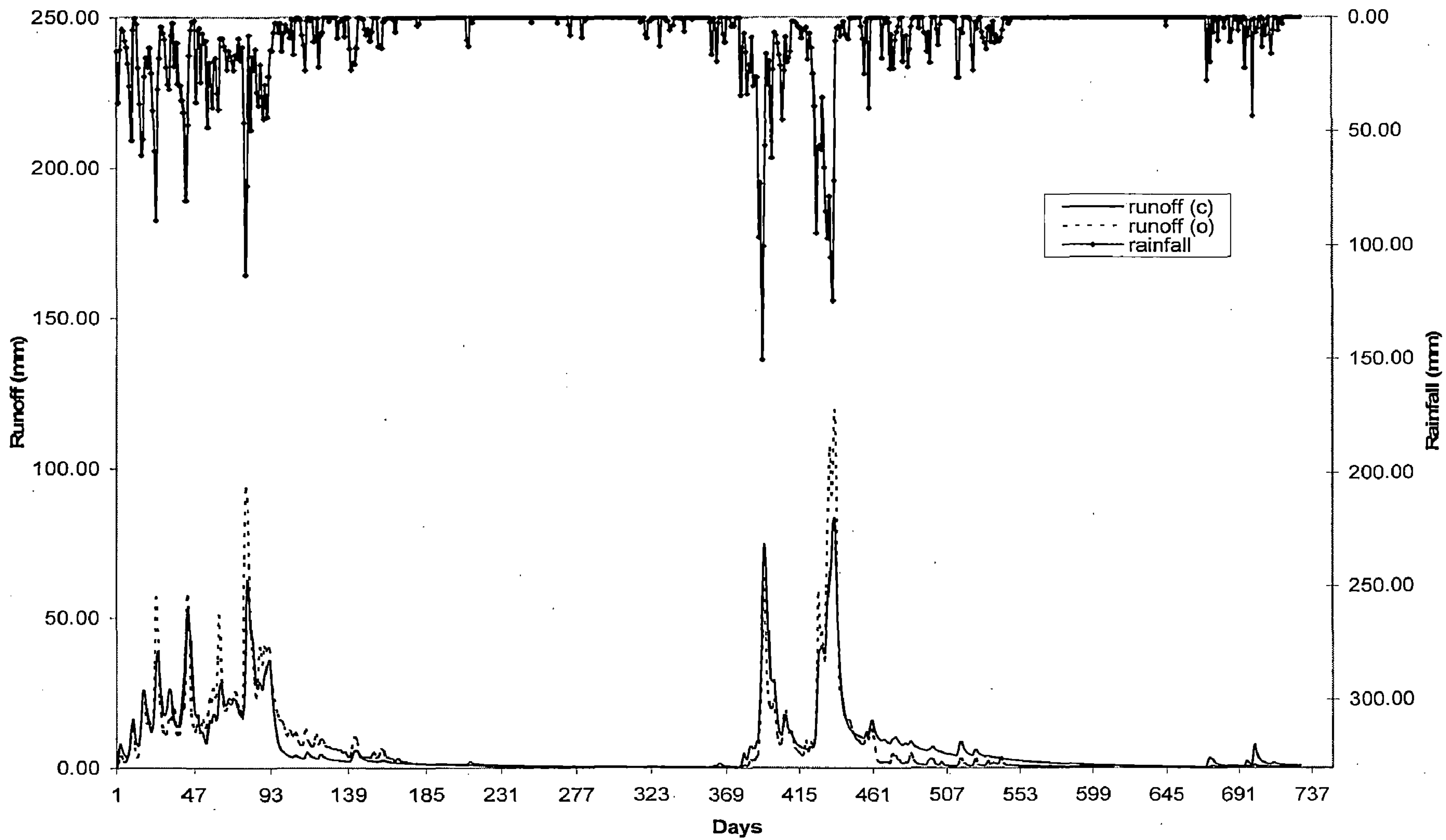


Fig 5.16 Daily variation of Rainfall, observed runoff and simulated runoff in Hemavati catchment (validation)

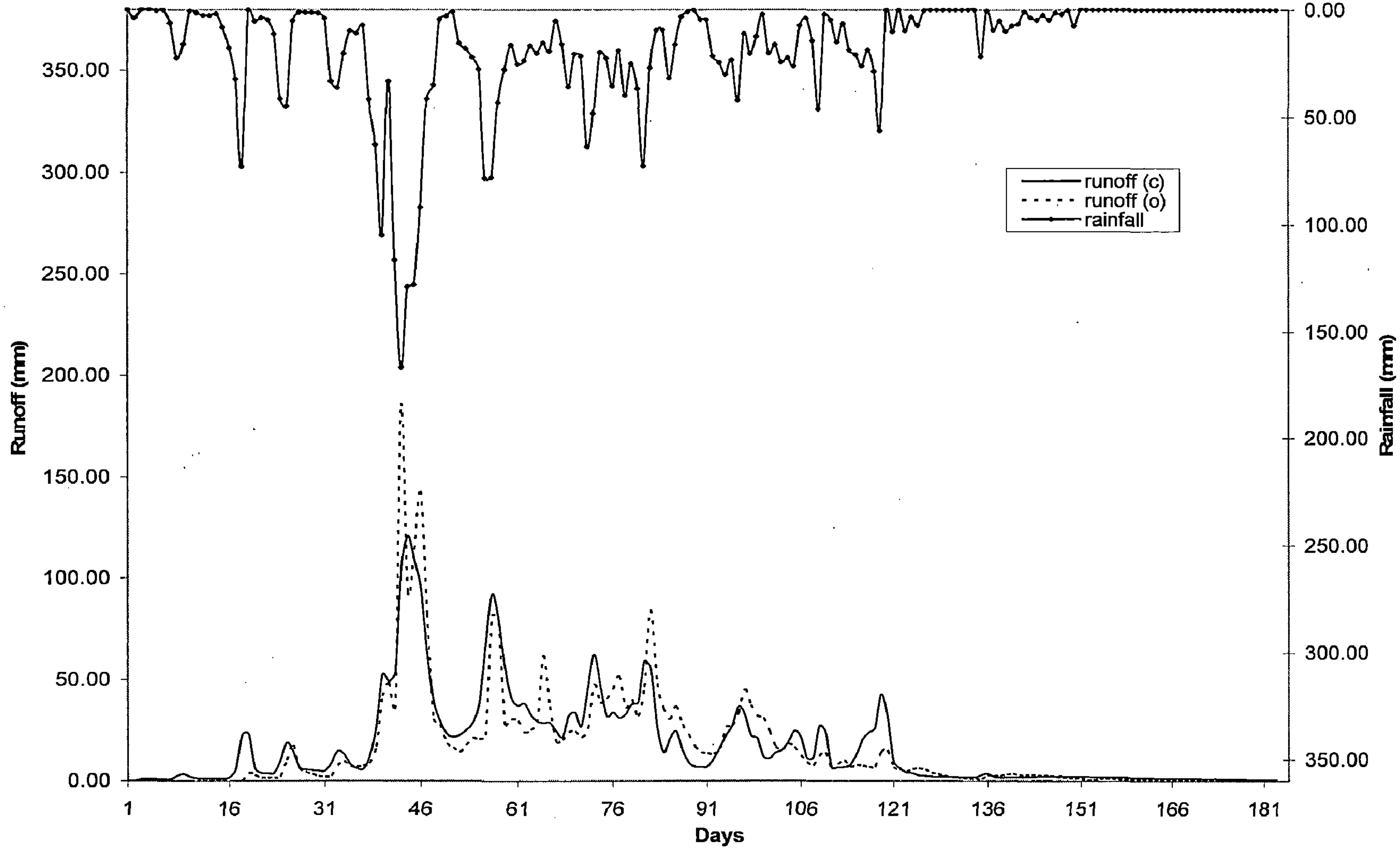


Fig: 5.17 Daily variation of Rainfall, observed runoff and simulated runoff in Kalu catchment (validation)

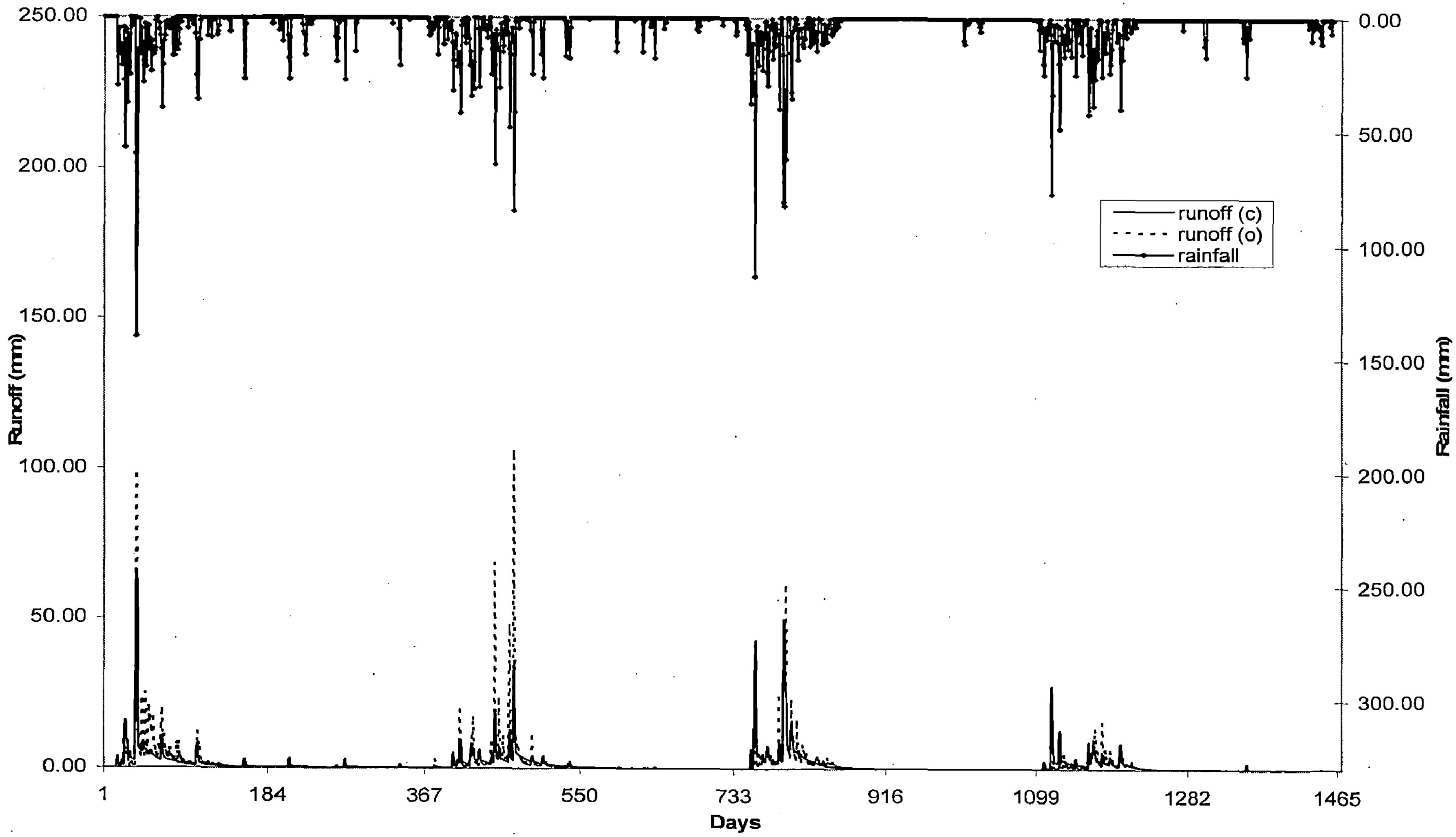


Fig: 5.18 Daily variation of Rainfall, observed runoff and simulated runoff in Manot catchment (validation)

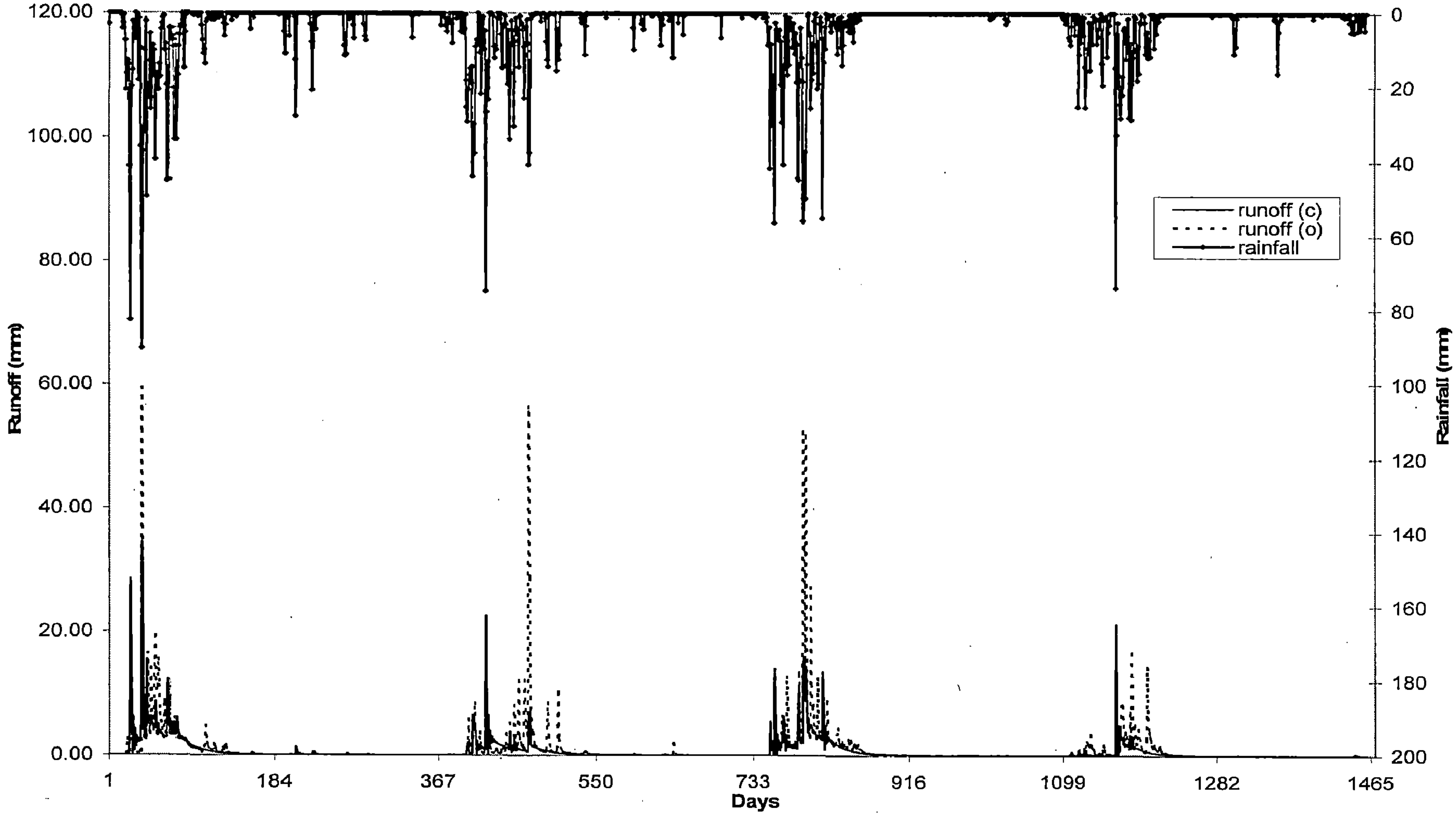


Fig: 5.19 Daily variation of Rainfall, observed runoff and simulated runoff in Mohegaon catchment (validation)

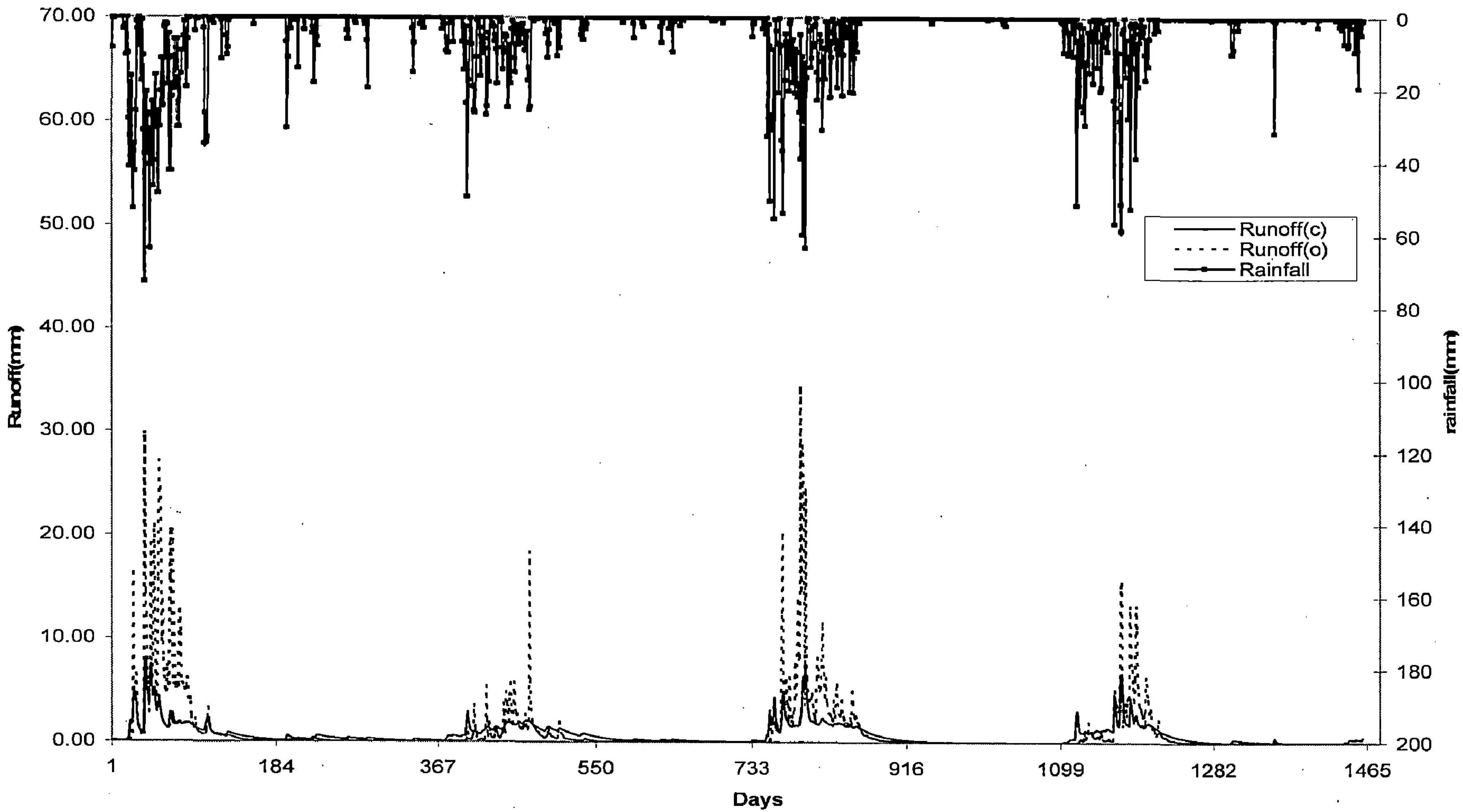


Fig : 5.20 Daily variation of Rainfall, observed runoff and simulated runoff in Hridayanagar (validation)

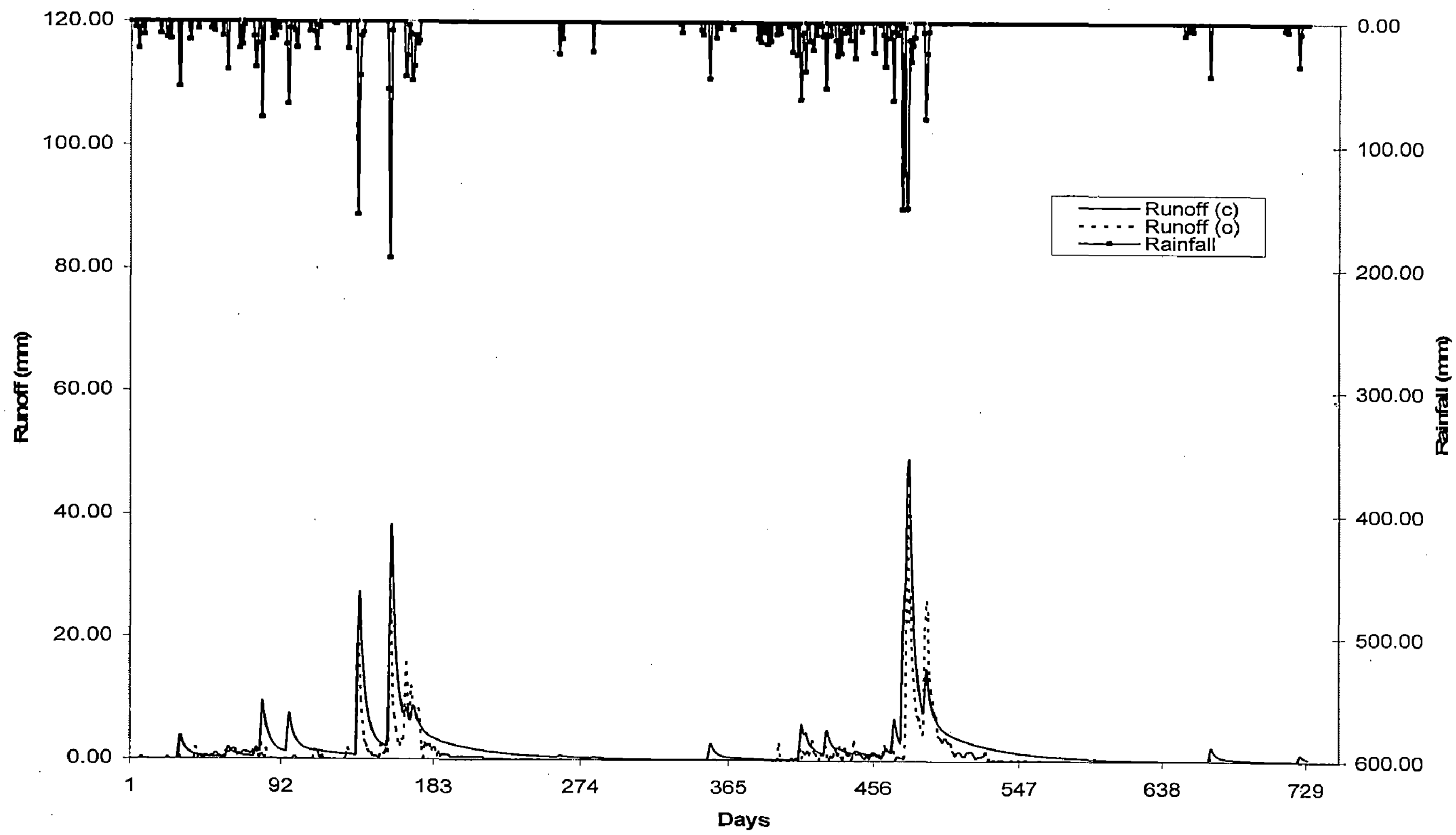


Fig: 5.21 Daily variation of Rainfall, observed runoff and simulated runoff in Ghodahado catchment (validation) satisfactory performance of the proposed model in yearly runoff computation. Similarly for Manot, the relative error varies in the range of 34.88% to ±44.54% with average value of 4.59. Average annual rainfall for the catchment is 1264 mm whereas the calculated and observed runoff are 559 mm and 533 mm respectively. The proposed model underestimated the yearly runoff in years 1981-82, 1983-84, 1984-85 and 1989-90, but in other years it overestimated.

Similarly for other catchments, such as Mohegaon and Hridayanagar deviation in annual runoff, in both calibration and validation are in the range of 46.93% to ±13.08 (average value of 25.03%) and 51.46% to ±40.53% (average value of 8.10%), respectively for both the catchment (Table 5.7). The average annual rainfall, runoff calculated and observed for Mohegaon are 1231 mm, 431mm and 323 mm and for Hirdayanagar are 1443 mm, 357mm and 328mm respectively. It is also observed that in some cases, the relative errors are negative, implying that the model overestimates the runoff. But in case of Ghodahado catchment, it shows maximum deviation in the observed and simulated one as the relative error is more, varying in the range of 8.46% to ±93.25%. This shows a poor model fit to the data of this catchment. But for other catchments with high runoff coefficient, the proposed model shows satisfactory results.

Table 5.7 Observed and simulated runoff and relative error

Sl. No	Year	Rainfall	observed runoff	Simulated runoff	Relative Error
		(mm)	(mm)	(mm)	%
HEMAVATI					
1	1974 -75	2938	2553	2551	0.06
2	1975 - 76	2651	1718	2066	-20.26
3	1976 - 77	2676	1894	1959	-3.43
4	1977 - 78	2942	2937	2417	17.71
5	1978 - 79	3064	2062	2569	-24.61
Average		2854	2233	2312	-3.57
KALU					
1	1990	3347	3529	3113	11.8
2	1991	3169	3060	2947	3.69

3	1992	1903	1602	1663	-3.81
4	1993	3355	3529	3053	13.5
Average		2944	2930	2694	8.06
MANOT					
1	1981 - 82	1136	383	553	-44.54
2	1982 - 83	1024	371	327	11.89
3	1983 - 84	1391	567	671	-18.36
4	1984 - 85	1303	616	703	-14.15
5	1985 - 86	1264	713	483	32.24
6	1986 - 87	1379	708	564	20.37
7	1987 - 88	1347	767	500	34.88
8	1988 - 89	1309	631	623	1.23
9	1989 - 90	1220	277	377	-36.39
Average		1264	559	533	4.59
MOHEGAON					
1	1981 - 82	1240	334	357	-6.93
2	1982 - 83	1113	339	183	46.06
3	1983 - 84	1533	486	301	37.99
4	1984 - 85	1295	518	586	-13.08
5	1985 - 86	1329	579	453	21.79
6	1986 - 87	1356	471	387	17.75
7	1987 - 88	1125	377	200	46.93
8	1988 - 89	1166	550	332	39.75
9	1989 - 90	926	227	111	51.21
Average		1231	431	323	25.03
HRIDAYANAGAR					
1	1981 - 82	1587	319	448	-40.53
2	1982 - 83	1462	314	333	-6.07
3	1983 - 84	1937	380	430	-13.07
4	1984 - 85	1300	299	368	-23.27
5	1985 - 86	1457	258	326	-26.03
6	1986 - 87	1815	601	292	51.46
7	1987 - 88	881	182	229	-26.06
8	1988 - 89	1377	568	284	50.06
9	1989 - 90	1171	289	241	16.73
Average		1443	357	328	8.10

GHODAHADO					
1	1993 - 94	947	145	263	-81.24
2	1994 - 95	1983	486	939	-93.25
3	1995 - 96	1483	1080	879	18.57
4	1987 - 88	1493	826	756	8.46
5	1988 - 89	1475	434	786	-81.21
Average		1476	594	725	-21.97

5.6 COMPARISON WITH AN EXISTING MODEL

This section compares the application of two models, viz., the proposed SCS – CN based model accounting on base flow computation and an available lumped-conceptual model (K. Geetha, 2007) on different watersheds.

Table 5.8 : Data length and model efficiency (%) with runoff coefficient

Catchment	Area Km^2	Efficiency (%)				Runoff coefficient
		Proposed model		Existing model		
		Calibration	Validation	Calibration	Validation	
Ramganga	3134	73.62	75.46	54.26	-18.79	0.33
Hemavati	600	83.27	84.82	83.50	87.72	0.80
Manot	5032	60.75	54.06	60.65	43.91	0.45
Kalu	224	63.89	82.01	63.33	76.15	0.91

Table 5.9 : Annual average rainfall, observed and relative error (%)

Catchment			**Proposed model**		**Existing model**	
	Average rainfall (mm)	Average observed runoff (mm)	Average simulated runoff (mm)	Average Relative Error (%)	Average simulated runoff (mm)	Average Relative Error (%)
Ramganga	1493	514.07	470.03	8.57	633.13	35.89
Hemavati	2854	2233	2312	-3.57	1986.87	9.78
Manot	1264	559	533	4.59	304.38	43.13
Kalu	2944	2930	2694	8.06	2546.73	7.42

Tables 5.8 and 5.9 compare the model efficiencies and average relative error values due to the above models. Both the models show a satisfactory performance on higher runoff producing catchments, like Hemavati and Kalu catchments. The catchment like Manot shows a low efficiency, as the runoff coefficient is low. The Ramganga catchment is however an exception.

The comparison of model efficiencies reveals that, the proposed model yields a maximum efficiency of 83.27% in calibration and 84.82% in validation in Hemavati catchment, whereas the existing model yields 83.5% and 87.72%, respectively in calibration and validation for the same catchment. For Ramganga catchment the proposed model yields efficiencies of 73.62% and 75.46% in calibration and validation, respectively and the existing model shows efficiencies of 65.48 and 41.64%, respectively in calibration and validation. Similarly for other catchments like Manot and Kalu catchments the proposed model shows respective efficiencies of 60.75 and 63.895% in calibration, and in validation these are 54.06 and 82.014%, whereas the existing method shows higher efficiencies in case of Kalu catchment.

It is apparent from Table 5.9 that the proposed model performs with the average relative error ranging from -3.57 to 8.57%, whereas the existing model ranges from 7.42 to 43.13%. Thus, the comparison based on average relative error indicates the proposed model to perform much better than the existing one in majority of the watersheds considered in this study.

CHAPTER VI: SUMMARY AND CONCLUSIONS

Information regarding flow rates at any point of interest along a stream is necessary in the analysis and design of many types of water resources projects. Although many streams have been gauged to provide continuous records of stream flow, planners and engineers are sometimes faced with little or no available stream flow information and must rely on synthesis and simulation as tools to generate artificial flow sequences for use in rationalizing decisions regarding structure size, the effect of land use, flood control measures, water supplies, and the effect of natural or induced watershed or climatic change.

The long-term hydrologic simulation plays an important role in water resources planning and watershed management, specifically for analysis of water availability; computation of daily, fortnightly, and monthly flows for reservoir operation and drought analysis. In this dissertation, a Soil Conservation Service Curve Number (SCS – CN) based long term rainfall runoff model was proposed, and it was tested on the data of Ramganga catchment (area = 3134 Km^2) using split sampling. The proposed model has four parameters, CN, CN_d, K and K_b. The first two parameters are the curve number for surface flow and drainage flow respectively, K is the catchment storage coefficient (day), and K_b is the ground water storage coefficient (day).

To check the versatility of the proposed model, this model was further applied to different watersheds located in different hydrometerologic conditions. These are catchments of Hemavati, Manot, Hridaynagar, Mohegaon, Kalu and Ghodahado. The following conclusions can be derived from the study:

(1) The model generally performed well in both calibration and validation on the data of Ramganga catchment. The resulting efficiencies for all the years varied in the range of 81.82 to 73.62%, showing a satisfactory fit and, in turn satisfactory model performance.

(2) The model exhibited a poorer performance as the length of data increased from 1 to 3 years in calibration. In calibration with three years of data, the efficiency

was 73.62% and in validation on the remaining two years of data, it was 75.46%, indicating a satisfactory model performance in both calibration and validation.

(3) The comparison of model efficiencies resulting from model application to other catchments, reveals that Hemavati yields maximum efficiency of 83.27% in calibration and 84.82% in validation. The other catchments like Manot, Kalu and Ghodahado exhibit 60.75, 63.895 and 59.35% efficiencies, respectively in calibration and 54.06, 82.014 and 32.31% in validation. The efficiencies of all catchments, except Hemavati and Ramganga, are higher in calibration than in validation, but reverse holds for the others.

(4) It is seen that the catchment of Hemavati and Kalu can be classified as high runoff producing catchments with runoff coefficient values of 0.83 and 0.91 respectively. Hridaynagar, Mohegaon catchments with low runoff coefficients of 0.25 and 0.27 respectively behave as dry catchments.

(5) The model simulated the yearly runoff values with relative error in the range (-29.66 to 18.20%) for Ramganga catchment. For other catchments it falls within the range of (-21.97 to 8.10%). These significantly low values indicate a satisfactory model performance. The negative (-) values of relative error indicate that the model overestimates the runoff values.

(6) The satisfactory model performance on the high runoff producing watersheds is further appreciable in view of the limited number of model parameters (only four) and its simplicity.

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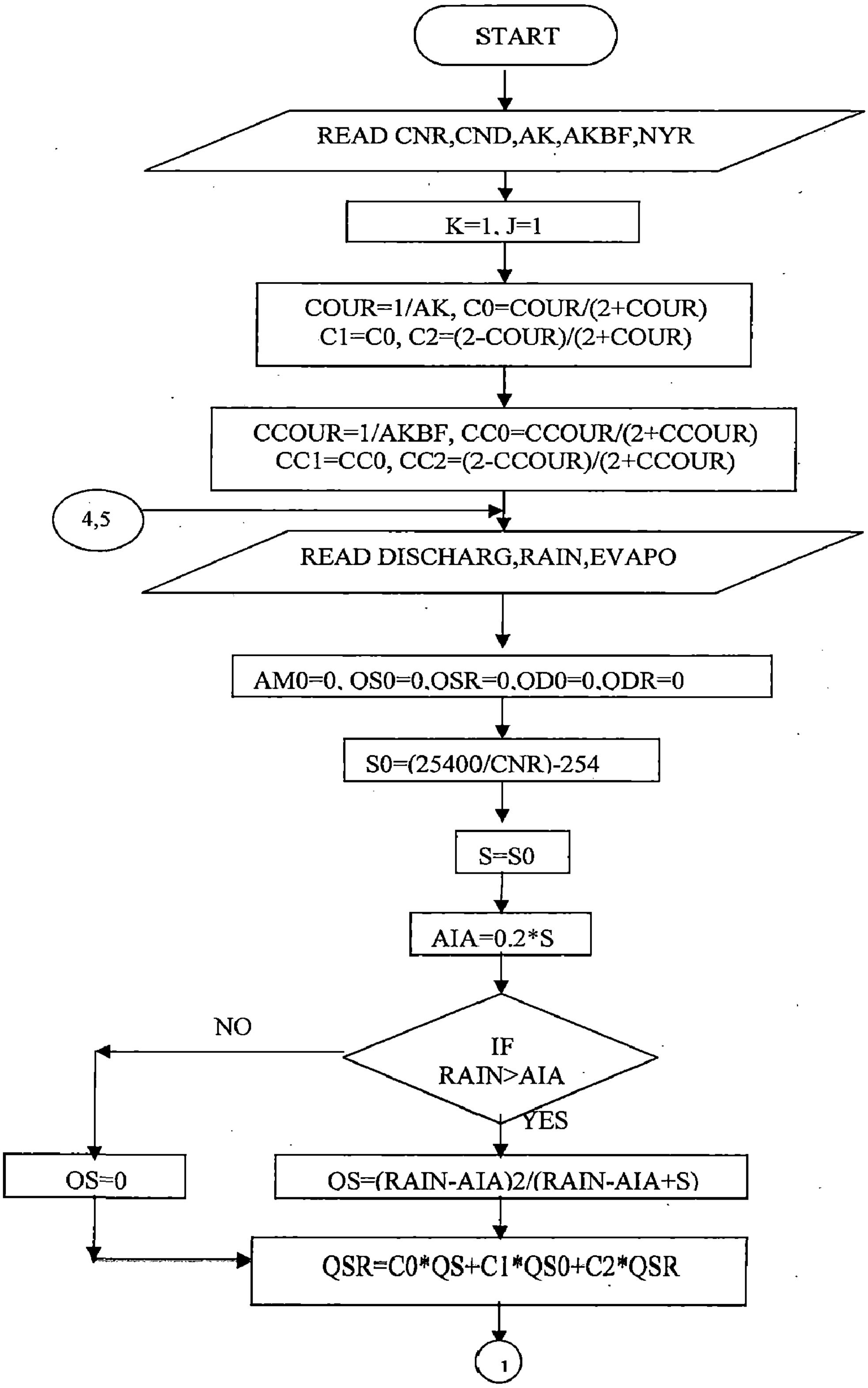
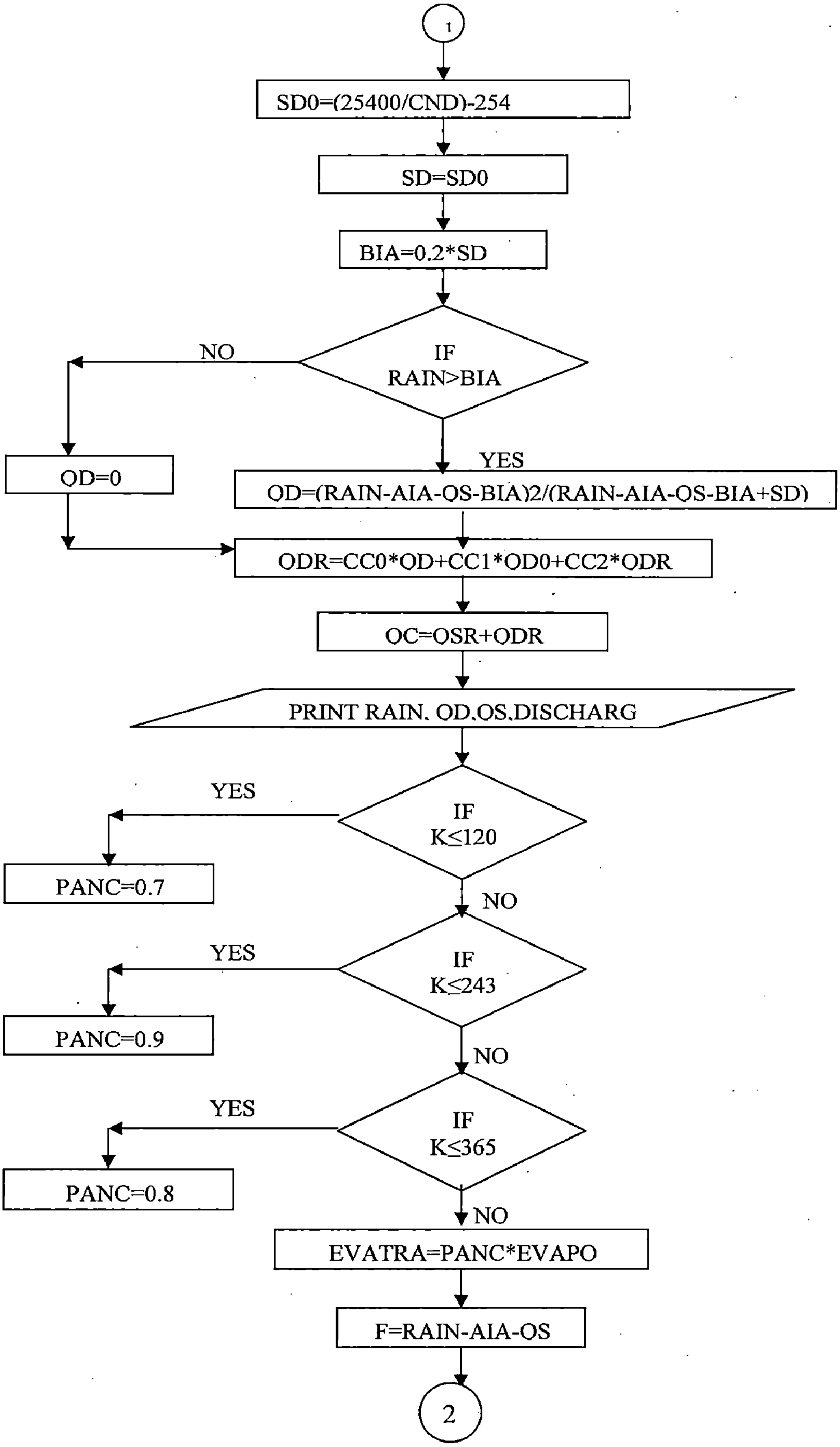
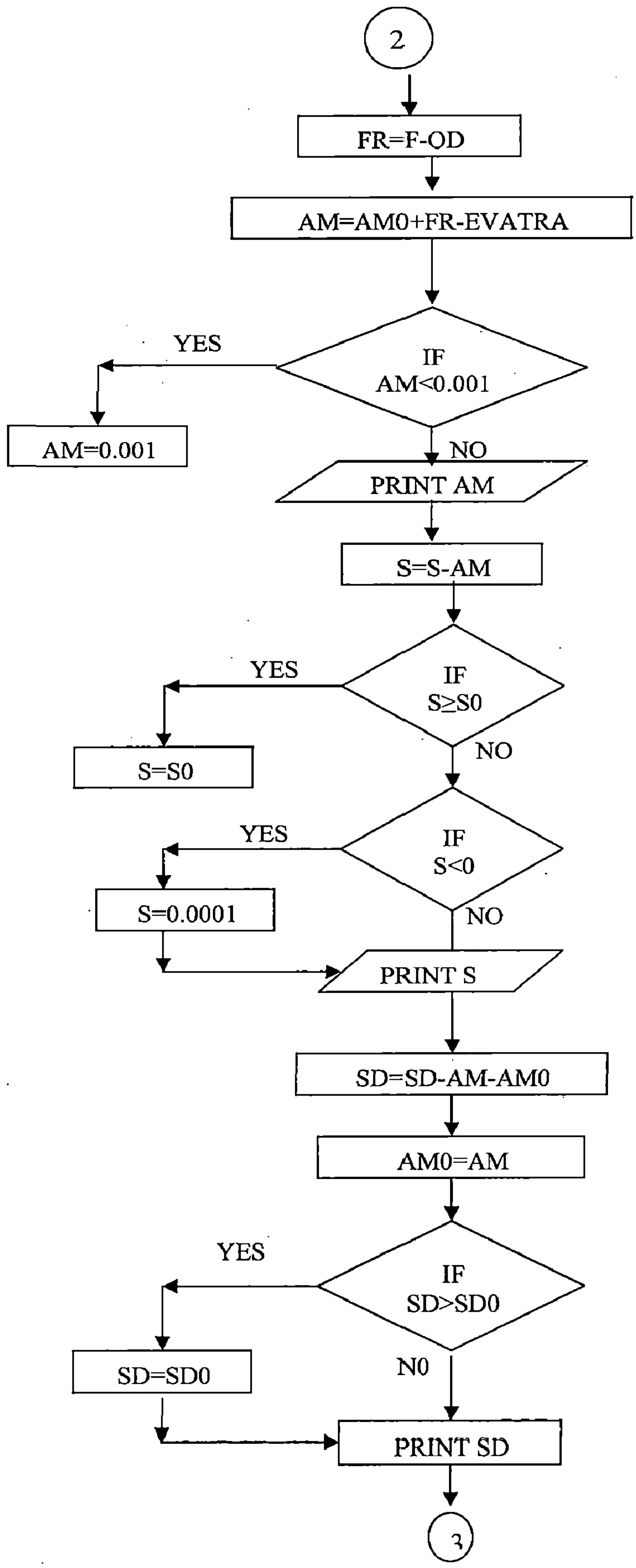
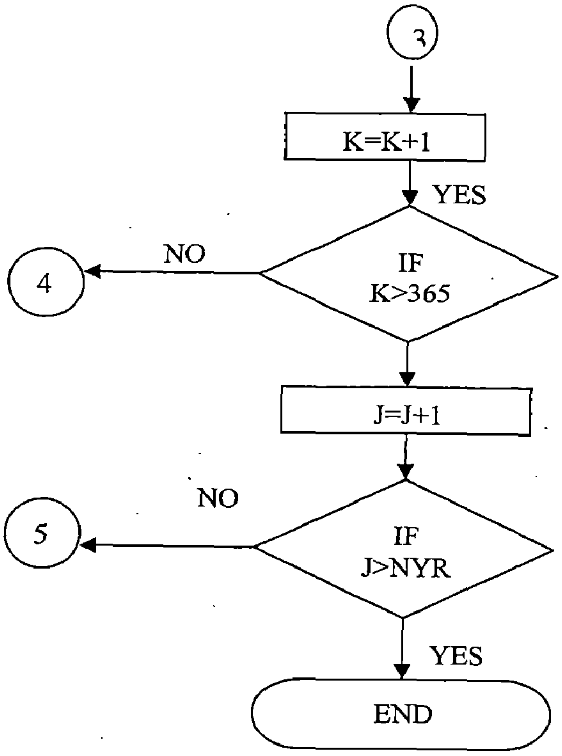
APPENDIX – I


START
READ CNR,CND,AK,AKBF,NYR
K=1, J=1
COUR=1/AK, C0=COUR/(2+COUR)
C1=C0, C2=(2-COUR)/(2+COUR)
CCOUR=1/AKBF, CC0=CCOUR/(2+CCOUR)
CC1=CC0, CC2=(2-CCOUR)/(2+CCOUR)
4,5
READ DISCHARG,RAIN,EVAPO
AM0=0, QS0=0,QSR=0,QD0=0,QDR=0
S0=(25400/CNR)-254
S=S0
AIA=0.2*S
NO
IF
RAIN>AIA
YES
QS=0
QS=(RAIN-AIA)2/(RAIN-AIA+S)
QSR=C0*QS+C1*QS0+C2*QSR
1


1
SD0=(25400/CND)-254
SD=SD0
BIA=0.2*SD
IF
RAIN>BIA
NO
YES
OD=0
OD=(RAIN-AIA-OS-BIA)2/(RAIN-AIA-OS-BIA+SD)
ODR=CC0*OD+CC1*OD0+CC2*ODR
OC=OSR+ODR
PRINT RAIN. OD.OS.DISCHARG
IF
K≤120
YES
PANC=0.7
NO
IF
K≤243
YES
PANC=0.9
NO
IF
K≤365
YES
PANC=0.8
NO
EVATRA=PANC*EVAPO
F=RAIN-AIA-OS
2


2
FR=F-OD
AM=AM0+FR-EVATRA
IF
AM<0.001
YES
AM=0.001
NO
PRINT AM
S=S-AM
IF
S≥S0
YES
S=S0
NO
IF
S<0
YES
S=0.0001
NO
PRINT S
SD=SD-AM-AM0
AM0=AM
IF
SD>SD0
YES
SD=SD0
N0
PRINT SD
3


3
K=K+1
YES
NO
4
IF
K>365
J=J+1
NO
5
IF
J>NYR
YES
END

APPENDIX – II

Program for daily rainfall – runoff using modified scs – cn method

```
$DEBUG
C     LONG-TERM SIMULATION USING EXISTING SCS-CN METHOD WITH
BASEFLOW CONSIDERATION
C
      CHARACTER*80 TITLE
      DIMENSION B(10),Y(1826),BV(10),BMIN(10),BMAX(10),P(12000)
      DIMENSION A(10,10),AC(10,10)
      DIMENSION DIS(365),RF(365),DISCHARG(1825),EVAPO(2000)
      DIMENSION QCOMP(2000),QBF(2000),EVATRA(2000),S(2000)
      DIMENSION RR(365),QQ(365),QQC(365)
      DIMENSION BBF(365),EEVTRA(365),QQCOMP(365),SS(365)
      DIMENSION X(2000),Z(2000)
      COMMON /A/ X
      COMMON /B/ QBF,EVAPO,EVATRA,S
      COMMON /C/ NYR

C
      EXTERNAL FUNC,DERIV
      OPEN(UNIT=1,FILE='DATA.TXT',STATUS='UNKNOWN')
      OPEN(UNIT=3,FILE='SCS.TXT',STATUS='UNKNOWN')
      OPEN(UNIT=4,FILE='SCS.OUT',STATUS='UNKNOWN')
C
      READ(1,1293) TITLE
      WRITE (*,*)TITLE
      READ(1,1293) TITLE
      WRITE (*,*)TITLE
      READ(1,1293) TITLE
      WRITE (*,*)TITLE

1293  FORMAT(80A)
C
      READ(1,*) NYR
      WRITE(*,*) NYR
      K=0
      SUMQ=0.0
      SUMRF=0
      SUMEVA=0
      DO I=1,NYR
      DO J=1,365
      K=K+1
      READ(1,*) NNN, DIS(J), RF(J), EVAPO(J)
      DISCHARG(K)=DIS(J)/3.134
      Y(K)=DISCHARG(K)
      X(K)=RF(J)
```

```
      EVAPO(K)=EVAPO(J)
      SUMQ=SUMQ+DISCHARG(K)
      SUMRF=SUMRF+X(K)
      SUMEVA=SUMEVA+EVAPO(K)
      ENDDO
      ENDDO
      NN=NYR*365
     AMQ=SUMQ/FLOAT(NN)
C
      WRITE(4,4444)SUMQ,SUMRF,SUMEVA
4444 FORMAT(1X,'SUMQ= ',F12.2,2X,'SUMRF= ',F12.2,2X,
    1'SUMEVA= ',F12.2/)
C
C     READ IN INITIAL GUESSES.
C     READ(3,*)CN0,BCOEF,AK,NLAG
      READ (3,*) KK
      READ(3,*) (B(J),J=1,KK)
C     OPTION FOR CALIBRATION AND VALIDATION
      READ(3,*) NOPT
      IF(NOPT.EQ.2) GO TO 300
C
C     READ IN LIMIT ON VARIABLE.
      READ(3,*) (BMIN(J),J=1,KK)
      READ(3,*) (BMAX(J),J=1,KK)
C
      FNU=0.
      FLA=0.
      TAU=0.
      EPS=0.
      PHMIN=0.
      I=0
      KD=KK
      DO 100 J=1,KK
      BV(J)=1
100   CONTINUE
      ICON=KK
      ITER=0
      NO=6
      WRITE(NO,1511)
1511  FORMAT(1H1,10X,27HBSOLVE REGRESSION ALGORITHM )
C
      PH2=0.0
200   CALL BSOLVE(KK,B,NN,Z,Y,PH,FNU,FLA,TAU,EPS,PHMIN,I,ICON,BV
     1,BMIN,BMAX,P,FUNC,DERIV,KD,A,AC,GAMM)
C
      ITER=ITER+1
      PH1=PH
      WRITE(NO,001) ICON,PH,ITER
      IF(ABS((PH1-PH2)/PH1).LE.0.01)GO TO 300
      PH2=PH1
```

```
      IF(PH.LE.0.0001)GO TO 300
001   FORMAT(/,2X,'ICON = ',I3,4X,5HPH = ,E15.8,4X'ITERATION
    1 NO. =',I3)
      IF(ICON) 10,300,200
10    IF(ICON+1) 20,60,200
20    IF(ICON+2) 30,70,200
30    IF(ICON+3) 40,80,200
40    IF(ICON+4) 50,90,200
50    GO TO 95
60    WRITE(NO,004)
004   FORMAT(//,2X,32HNO FUNCTION IMPROVEMENT POSSIBLE )
      GOTO 300
70    WRITE(NO,005)
005    FORMAT(//,2X,28HMORE UNKNOWNS THAN FUNCTIONS)
      GOTO 300
80    WRITE(NO,006)
006   FORMAT(//,2X,24HTOTAL VARIABLES ARE ZERO)
      GOTO 300
90    WRITE(NO,007)
007   FORMAT(//,2X,'HCORRECTIONS SATISFY CONVERGENCE
REQUIREMENTS',
    1' BUT LAMDA FACTOR (FLA) STILL LARGE')
      GOTO 300
95    WRITE(NO,008)
008   FORMAT(//,2X,20HTHIS IS NOT POSSIBLE)
      GOTO 300
300   WRITE(4,002)
002   FORMAT(//,2X,24HSOLUTION OF THE EQUATION)
      DO 400 J=1,KK
      WRITE(4,003) J,B(J)
      WRITE(*,003) J,B(J)
003   FORMAT(/,2X,2HB(,I2,4H)= ,E16.8)
400   CONTINUE
      CALL FUNC(KK,B,NN,Z)
C
     WRITE(4,11)
11   FORMAT(1X,' DAY  RAINFALL  EVAPTRA   BASEFLOW  RUNOFF(C) ',
    1'RUNOFF(O)  S'/)
     SUMFO=0.0
     SUMF1=0.0
     K=0
     DO I=1,NYR
     K1=0
     DO J=1,365
     K=K+1
     K1=K1+1
     SUMFO=SUMFO+(DISCHARG(K)-AMQ)**2
     SUMF1=SUMF1+(DISCHARG(K)-Z(K))**2
     RR(K1)=X(K)
     QQ(K1)=DISCHARG(K)
```

```
      QQC(K1)=Z(K)
       BBF(K1)=QBF(K)
       EEVTRA(K1)=EVATRA(K)
       QQCOMP(K1)=QCOMP(K)
       SS(K1)   =S(K)
       WRITE(4,1294)K1,RR(K1),EEVTRA(K1),BBF(K1),QQC(K1),QQ(K1),SS(K1)
1294  FORMAT(1X,I5,1X,6F10.2)
       ENDDO
       ENDDO
       EFF=(1-SUMF1/SUMFO)*100.
       WRITE(4,1295)EFF
       WRITE(*,1295)EFF
1295   FORMAT(1X,'EFFICIENCY= ',1X,F10.3)
        STOP
        END
C
      SUBROUTINE LEAP(NYEAR,ND)
      IF(AMOD(FLOAT(NYEAR),4.).EQ.0.)THEN
      ND=29
      ELSE
      ND=28
      ENDIF
      RETURN
      END
C*************************************************************
C   THIS IS SUBROUTIUNE FOR FUNCTION
      SUBROUTINE FUNC(KK,B,NN,Z)
C
       DIMENSION QCOMP(2000),QD(2000),QBF(2000),EVAPO(2000)
       DIMENSION X(2000),Z(2000),QC(2000),B(10),S(2000),SD(2000)
       DIMENSION EVATRA(2000),AM(2000)
       COMMON /A/ X
       COMMON /B/ QBF,EVAPO,EVATRA,S,SD
       COMMON /C/ NYR
C
C HERE, X IS RAINFALL, Z IS THE COMPUTED OUTFLOW (mm)
C DEFINE HERE WHICH B() PARAMETER REFERS TO THE REAL
PARAMETER.
       CNR = B(1)
       CND = B(2)
       AK = B(3)
       AKBF = B(4)
C AM0 IS ASSUMED EQUAL TO ZERO
C COMPUTATION OF COURANT NUMBER AND C0, C1, AND C2 FOR
SURFACE RUNOFF
       COUR=1./AK
       C0=COUR/(2.+COUR)
       C1=C0
       C2=(2.-COUR)/(2.+COUR)
```

```
C COMPUTATION OF COURANT NUMBER AND CC0, CC1, AND CC2 FOR
DRAINAGE
      COUR=1./AKBF
      CC0=COUR/(2.+COUR)
      CC1=C0
      CC2=(2.-COUR)/(2.+COUR)

C     COMPUTATIONS BEGIN
      K=0
      S0 = 25400./CNR - 254.
      AIA=0.2*S0
      SD0 = 25400./CND - 254.
      BIA = 0.03*SD0
      DO 2 I=1,NYR
      DO 3 J=1,365
      K=K+1
      IF(K.Eq.1)AM(K)=0.0
      IF(K.Eq.1)S(K)=S0

CC COMPUTATION OF DIRECT SURFACE RUNOFF AND DRAINAGE FLOW
101   IF(X(K).GT.AIA)THEN
      QCOMP(K)=(X(K)-AIA)*(X(K)-AIA)/(X(K)-AIA+S(K))
      ELSE
      QCOMP(K)= 0.0
      ENDIF
C
      IF(X(K)-AIA-QCOMP(K).GT.BIA)THEN
      QD(K)=(X(K)-AIA-QCOMP(K)-BIA)**2/(X(K)-AIA-QCOMP(K)-
BIA+SD(K))
      ELSE
      QD(K)= 0.0
      ENDIF
C
      IF(K.EQ.1)THEN
      QC(K)=0.0
      QBF(K)=0.0
      ELSE
      QC(K)=QCOMP(K)
      QBF(K)=QD(K)
C  ROUTING OF COMPUTED SURFACE RUNOFF
       QC(K)=C0*QCOMP(K)+C1*QCOMP(K-1)+C2*(QC(K-1)-QBF(K-1))
C ROUTING OF DRAINAGE FLOW
       QBF(K)=CC0*QD(K)+CC1*QD(K-1)+CC2*QBF(K-1)
        ENDIF
C TOTAL RUNOFF
       QC(K)=QC(K)+QBF(K)
       Z(K)=QC(K)
C COMPUTATION OF EVAPOTRANSPIRATION
       IF(J.GE.1.AND.J.LE.120)PANC=0.7
       IF(J.GT.121.AND.J.LE.243)PANC=0.6
```

```
      IF(J.GT.244.AND.J.LE.365)PANC=0.8
      EVATRA(K)=PANC*EVAPO(K)
C NEXT DAY PARAMETERS
      F=X(K)-AIA-QCOMP(K)
      FR=F-QD(K)
      AM(K+1)=AM(K)+FR-EVATRA(K)
      IF(AM(K+1).LE.0.0001) AM(K)=0.0001
      S(K+1)=S(K) - AM(K+1)
      IF(S(K+1).GE.S0) S(K+1)=S0
      IF(S(K+1).LT.0.0) S(K+1)=0.0001
      AIA = 0.2*S(K+1)
      SD(K+1)=SD(K)-(AM(K+1)-AM(K))
      IF(SD(K+1).GT.SD0)SD(K+1)=SD0
      BIA = 0.03*SD(K+1)
3     CONTINUE
2     CONTINUE
      RETURN
      END
C*****************************************************************
      SUBROUTINE
BSOLVE(KK,B,NN,Z,Y,PH,FNU,FLA,TAU,EPS,PHMIN,I,ICON,
   1BV,BMIN,BMAX,P,FUNC,DERIV,KD,A,AC,GAMM)
      DIMENSION B(10),Z(2000),Y(2000),BV(10),BMIN(10),BMAX(10)
      DIMENSION P(12000)
      DIMENSION A(10,10),AC(10,10),X(2000)
      COMMON /A/ X
      K=KK
      N=NN
      KP1=K+1
      KP2=KP1+1
      KBI1=K*N
      KBI2=KBI1+K
      KZI=KBI2+K
      IF(FNU.LE.0.) FNU=10.0
      IF(FLA.LE.0.) FLA=0.01
      IF(TAU.LE.0.) TAU=0.001
      IF( EPS.LE.0.) EPS=0.00002
      IF(PHMIN.LE.0.) PHMIN=0.0
120   KE=0
130   DO 160 I1=1,K
160   IF(BV(I1).NE.0.0) KE=KE+1
      IF(KE.GT.0) GOTO 170
162   ICON=-3
163   GOTO 2120
170   IF(N.GE.KE) GOTO 500
180   ICON=-2
190   GOTO 2120
500   I1=1
530   IF(I.GT.0) GOTO 1530
550   DO 560 J1=1,K
```

```
      J2=KBI1+J1
      P(J2)=B(J1)
      J3=KBI2+J1
560   P(J3)=ABS(B(J1))+1.0E-02
      GOTO 1030
590   IF(PHMIN.GT.PH.AND.I.GT.1) GOTO 625
      DO 620 J1=1,K
      N1=(J1-1)*N
      IF(BV(J1)) 601,620,605
601   CALL DERIV(K,B,N,Z,P(N1+1),J1,JTEST)
      IF(JTEST.NE.(-1)) GOTO 620
      BV(J1)=1
605   DO 606 J2=1,K
      J3=KBI1+J2
606   P(J3)=B(J2)
      J3=KBI1+J1
      J4=KBI2+J1
      DEN=0.001*AMAX1(P(J4),ABS(P(J3)))
      IF(P(J3)+DEN.LE.BMAX(J1)) GOTO 55
      P(J3)=P(J3)-DEN
      DEN=-DEN
      GOTO 56
55    P(J3)=P(J3)+DEN
56    CALL FUNC(K,P(KBI1+1),N,P(N1+1))
      DO 610 J2=1,N
      JB=J2+N1
610   P(JB)=(P(JB)-Z(J2))/DEN
620   CONTINUE
625   DO 725 J1=1,K
      N1=(J1-1)*N
      A(J1,KP1)=0.
      IF(BV(J1)) 630,692,630
630   DO 640 J2=1,N
      N2=N1+J2
640   A(J1,KP1)=A(J1,KP1)+P(N2)*(Y(J2)-Z(J2))
650   DO 680 J2=1,K
660   A(J1,J2)=0.0
665   N2=(J2-1)*N
670   DO 680 J3=1,N
672   N3=N1+J3
674   N4=N2+J3
680   A(J1,J2)=A(J1,J2)+P(N3)*P(N4)
      IF(A(J1,J1).GT.1.E-20) GOTO 725
692   DO 694 J2=1,KP1
694   A(J1,J2)=0.0
695   A(J1,J1)=1.0
725   CONTINUE
      GN=0.
      DO 729 J1=1,K
729   GN=GN+A(J1,KP1)**2
```

```
      DO 726 J1=1,K
726   A(J1,KP2)=SQRT(A(J1,J1))
      DO 727 J1=1,K
      A(J1,KP1)=A(J1,KP1)/A(J1,KP2)
      DO 727 J2=1,K
727   A(J1,J2)=A(J1,J2)/(A(J1,KP2)*A(J2,KP2))
730   FL=FLA/FNU
      GOTO 810
800   FL=FNU*FL
810   DO 840 J1=1,K
820   DO 830 J2=1,KP1
830   AC(J1,J2)=A(J1,J2)
840   AC(J1,J1)=AC(J1,J1)+FL
      DO 930 L1=1,K
      L2=L1+1
      DO 910 L3=L2,KP1
910   AC(L1,L3)=AC(L1,L3)/AC(L1,L1)
      DO 930 L3=1,K
      IF(L1-L3) 920,930,920
920   DO 925 L4=L2,KP1
925   AC(L3,L4)=AC(L3,L4)-AC(L1,L4)*AC(L3,L1)
930   CONTINUE
      DN=0.
      DG=0.
      DO 1028 J1=1,K
      AC(J1,KP2)=AC(J1,KP1)/A(J1,KP2)
      J2=KBI1+J1
      P(J2)=AMAX1(BMIN(J1),AMIN1(BMAX(J1),B(J1)+AC(J1,KP2)))
      DG=DG+AC(J1,KP2)*A(J1,KP1)*A(J1,KP2)
      DN=DN+AC(J1,KP2)*AC(J1,KP2)
1028  AC(J1,KP2)=P(J2)-B(J1)
      COSG=DG/SQRT(DN*GN)
      JGAM=0
      IF(COSG) 1100,1110,1110
1100  JGAM=2
      COSG=-COSG
1110  CONTINUE
      COSG=AMIN1(COSG,1.0)
      GAMM=ARCOS(COSG)*180./(3.14159265)
      IF(JGAM.GT.0)GAMM=180.-GAMM
1030  CALL FUNC(K,P(KBI1+1),N,P(KZI+1))
1500  PHI=0.
      DO 1520 J1=1,N
      J2=KZI+J1
1520  PHI=PHI+(P(J2)-Y(J1))**2
      IF(PHI.LT.1.E-10) GOTO 3000
      IF(I.GT.0) GOTO 1540
1521  ICON=K
      GOTO 2110
1540  IF(PHI.GE.PH) GOTO 1530
```

```
C    EPSILON TEST
1200  ICON=0
      DO 1220 J1=1,K
      J2=KBI1+J1
1220  IF(ABS(AC(J1,KP2))/(TAU+ABS(P(J2))).GT.EPS) ICON=ICON+1
      IF(ICON.EQ.0) GOTO 1400
C  GAMMA LAMDA TEST
      IF (FL.GT.1.0.AND.GAMM.GT.90.0) ICON=-1
      GOTO 2105
C  GAMMA EPSILON TEST
1400  IF(FL.GT.1.0.AND.GAMM.LE.45.0) ICON=-4
     GOTO 2105
1530  IF(I1-2) 1531,1531,2310
1531  I1=I1+1
     GOTO (530,590,800),I1
2310  IF(FL.LT.1.0E+8) GOTO 800
1320  ICON=-1
C
2105  FLA=FL
      DO 2091 J2=1,K
      J3=KBI1+J2
2091  B(J2)=P(J3)
2110  DO 2050 J2=1,N
      J3=KZI+J2
2050  Z(J2)=P(J3)
      PH=PHI
      I=I+1
2120  RETURN
3000  ICON=0
     GOTO 2105
     END
     FUNCTION ARCOS(Z)
     X=Z
     KEY=0
     IF(X.LT.(-1.)) X=-1.
     IF(X.GT.1.) X=1.
     IF(X.GE.(-1.) .AND.X.LT.0.) KEY=1
     IF(X.LT.0.)  X=ABS(X)
     IF(X.EQ.0.)  GO TO 10
     ARCOS=ATAN(SQRT(1.-X*X)/X)
     IF(KEY.EQ.1) ARCOS=3.14159265-ARCOS
     GOTO 999
10    ARCOS=1.5707963
C
999  RETURN
     END
C*****************************************************************
     SUBROUTINE DERIV(K,B,N,Z,P,J1,JTEST)
     RETURN
     END
```