SCS-CN-BASED LONG-TERM HYDROLOGIC SIMULATION

A DISSERTATION
Submitted in partial fulfillment of the
requirements for the award of the degree
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By
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CANDIDATE'S DECLARATION

I hereby certify that the work, which is being presented in this Dissertation entitled **"SCS-CN-Based Long-Term Hydrologic Simulation"** in partial fulfillment of the requirements for the award of the degree of MASTER OF TECHNOLOGY in Water Resources Development (Civil) and 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 July 2010 to June 2011 under the supervision and guidance of Dr. S.K. Mishra, Associate Professor, Department of Water Resources Development and Management, Indian Institute of Technology Roorkee, India.

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

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

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ABSTRACT

Runoff estimation is essential for planning and management of water resources projects. A number of models varying from the simplest empirical relations to the most complex physically based models have been suggested in literature to mimic the complex phenomenon of rainfall-runoff. The Soil Conservation Service Curve Number (SCS-CN) method is one of the simplest and most popular methods available and widely used world over for predicting direct surface runoff from given storm rainfall amount. Of late, the method has also been employed in long term hydrologic simulation.

In this study, an SCS-CN-based long-term simulation model is proposed and tested on the 10-year daily data of three watersheds namely Betwa catchment (area = 4122 sq. km), Ret catchment (area = 262sq. km), and Siul catchment (area = 360 sq. km). The available 10 years of data of each watershed was split into two parts. The first part of data was used for calibration, and the other for validation. Simulation was carried out using yearly data and the whole data. Besides, a yearly volumetric analysis, a sensitivity analysis of the three parameters of the proposed model was also carried out. It was seen that the model performance degraded with the increase in length of data. In both yearly simulations and in calibration as well as validation, the proposed model showed a satisfactory performance, i.e. with significantly low relative errors. The least sensitive and most significant parameter CN_0 of the SCS-CN model indicated its amenability to field applications employing the NEH-4 CN values or the CN values derived using remote sensing data. Over and above all, the model is simple, has three parameters, and is dependable for field applications.

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AMC I	AMC corresponding to dry condition;
AMC II	AMC corresponding to normal condition;
AMC III	AMC corresponding to wet condition;
AMC	Antecedent moisture condition;
AVP	Average annual rainfall;
AVQ	Average annual runoff;
b_c	Depletion coefficient;
CN	Curve number;
CNI	Curve number corresponding to AMC I;
CNII	Curve number corresponding to AMC II;
CNIII	Curve number corresponding to AMC III;
CN_t	Curve number at time t;
$C_0 C_1 C_2$	Routing coefficients ;
DP	Average daily depletion;
E	Lake evaporation;
E_t	Average monthly lake evaporation for day t;
F	Actual infiltration;
F^2	Index of disagreement;
F_c	Cumulative static portion of total infiltration;
F_d	Cumulative dynamic portion of total infiltration;
I_a	Initial abstraction;
K	Storage coefficient;
ALPHA	a coefficient for lateral inflow;

M	Soil moisture index at beginning of the first storm;
M_a	Average soil moisture index;
M_t	Soil moisture index at any time t;
N	Number of values (time steps) within the considered time period;
P	Total rainfall
PANC	Ratio of the potential maximum retention at a time t to the absolute potential maximum retention;
PET	Potential evapotranspiration;
Q	Direct surface runoff;
Q_b	Base flow;
$QOBS_i$	Recorded value at time step I;
$QSIM_i$	Simulated value at time step I;
Q_t	Total daily flow;
R^2	Coefficient of correlation;
S	Potential maximum retention;
CN_0	Potential storage space available in the soil column for water retention;
S_{abs}	Absolute potential maximum retention;
T	No. of days between storms;
t	Time;
λ	Initial abstraction coefficient;

CHAPTER 1
INTRODUCTION

1.1 GENERAL

The problem of transformation of rainfall into runoff has been subject of scientific investigations throughout the evolution of the subject of hydrology. Hydrologists are mainly concerned with evaluation of catchment response for planning, development and operation of various water resources schemes. A number of investigators have tried to relate runoff with different watershed characteristics affecting it. For simulation of the rainfall-runoff process and design flood estimation, conceptual and/or physically based models are widely used. Long-term hydrologic simulation is required for augmentation of hydrologic data; to delineate vulnerable areas of the watershed contributing to sediment yield, which is significantly related with the direct surface runoff generated by the watershed; for analysis of water availability; computation of daily, fortnightly, and monthly flows for reservoir operation; drought analyses; water quality analyses; and so on. Since the rainfall data are generally available for a much longer period than are the stream flow data, long-term hydrologic simulation helps extend the gauged data required for the above applications. Thus, it is useful for water resources planning and watershed management.

Stream flow representing the runoff phase of the hydrologic cycle is the most important basic 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 portion 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.

In addition to unit hydrograph based approach, hydrological modeling is another approach for runoff estimation. The key outcomes of hydrologic model's (or rainfall-runoff models) are flow hydrographs. Simulating the transformation of rainfall into runoff at the catchment scale using mathematical models has seen considerable developments since the early 1960s due to increasing computing capacities. Now there exist a large number of models in literature, among which are the spatially lumped conceptual or empirical types that represent the link between rainfall and stream flow by a series of interconnected storage elements. The available popular rainfall-runoff models are HEC-HMS, SHE, MIKE-II, SWMM etc. These models are useful for the hydrologic and hydraulic engineering planning and design as well as water resources management.

A hydrologic model can be defined as a mathematical representation of the flow of water and its constituents on some part of the land surface or subsurface environment. Hydrologists are mainly concerned with evaluation of catchment response for planning, development and operation of various water resources schemes. Computer models began to appear in the mid 1960s, first for surface water flow and sediment transport, then in the 1970s for surface water quality and ground water flow, then in 1980s for ground water transport. Conventional models require considerable hydrological, meteorological, and spatial data.

Rainfall-runoff methods developed during the early 1940's utilized infiltration data for computing the runoff amount. Andrews (1954) eventually developed a graphical rainfall runoff method taking into account the soil texture, type and amount of cover and conservation practices, combined into what is referred as soil cover complex or soil-vegetation-land use (SVL) complex (Miller and Cronshey, 1989). According to Rallison and Miller (1982), the methods given by Mockus (1949) and Andrews (1954) were transformed and generalized to yield the existing SCS-CN method so that it could generally be used universally and was also applicable to ungauged watersheds.

1.2 OBJECTIVE OF THE STUDY

The primary objective of the study is to propose a simple Soil Conservation Service Conservation Curve Number (SCS-CN) Method based long-term (daily) flow simulation model and test its workability using the data of three Indian watersheds located in different geo-climatic settings.

1.3 SCOPE OF THE STUDY

The study is organized as follows:

CHAPTER 1 provides brief introduction about different rainfall-runoff models.

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

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

CHAPTER 4 provides features of the Betwa river catchment in Madhya Pradesh (India), the Ret river catchment in Odisha, and the Siul river catchment in Himachal Pradesh, the data of which have been used for model testing.

CHAPTER 5 describes the results.

CHAPTER 6 summarizes and concludes the study.

CHAPTER 2
REVIEW OF LITERATURE

Rainfall-runoff modeling is widely used in flow simulation. Its origin 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 extend their application to larger catchments. In 1930's, the popular unit hydrograph and instantaneous 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 proving the generality of the available approaches. The subsequent era saw the development of a number of models and evoked the problem of classification (Dooge, 1973; Todini, 1988).

2.1 CLASSIFICATION OF HYDROLOGICAL MODELS

The available hydrological models can be broadly classified into deterministic Models, lumped conceptual models, and fully distributed, physically-based models, a brief description of which is provided as below.

2.1.1 Deterministic Models

Deterministic models can be classified according to whether the model utilizes a spatially lumped or distributed description of the catchment area, and whether the description of the hydrological processes is empirical, conceptual or fully physically-based. In practice, most conceptual models are also lumped and most fully physically based models are also distributed.

2.1.2 Lumped Conceptual Models

These occupy an intermediate position between the fully physically based approach and empirical black-box analysis. Such models are formulated on the basis of a relatively small number of components, each of which is a simplified representation of one process element in the system being modeled.

2.1.3 Fully Distributed, Physically-Based Models

These are based on our understanding of the physics of the hydrological processes, which control catchment response and use physically based equations to describe these processes. From their physical basis such models can simulate the complete runoff regime, providing multiple outputs (e.g. river discharge, phreatic surface level and evaporation loss) while black box models can offer only one output. Also, almost by definition, physically based models are spatially distributed since the equations from which they are formed generally involve one or more space coordinates. They can therefore simulate the spatial variation in hydrological conditions within a catchment as well as simple outflows and bulk storage volumes. On the other hand, such models require large computational time and data and are costly to develop and operate. 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, was used for long term hydrologic simulation; a brief review of such studies is in order.

2.2 LONG -TERM HYDROLOGIC SIMULATION

Long-term hydrologic simulation is required for augmentation of hydrologic data. It is useful for water resources planning and watershed management. Long-term hydrologic data are specifically required for analysis of water availability; computation of daily, fortnightly, and monthly flows for reservoir operation; and drought analyses. Since the rainfall data are generally available for a much longer period than are the stream flow data, long - term hydrologic simulation helps extend the gauged data required for the above applications.

There exist a multitude of models for hydrologic simulation. In 1991, the U.S. Bureau of Reclamation prepared an inventory of 64 watershed models into four categories and the inventory is 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 United States. Wurbs (1998) listed a number of generalized water resources simulation models in seven categories and discussed their dissemination.

The available models vary in description of the components of the hydrologic cycle, degree of complexity of inputs, number of parameters to be determined, time interval used, and output generated. Some models like Hydrologic Simulation Package Fortran (HSPF), USDAHL (Holtan and Lopez, 1971) and its variants, System Hydrologic European (SHE), hydrologic Engineering Center (HEC), Hydrologic Modeling System (HMS) (HEC, 2000) have a number of parameters, usually use a short time interval, and produce hydrographs as well as water yield. The HSPF and SHE models are not applicable to unsauged watersheds for the reason that their application requires priori 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 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 an I-hr interval is used during rains), has simple inputs, and only outputs runoff volume. In, testing, this model was reported to exp lain about 80% of the variation in 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 hydrologic 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 of Asia are ungauged and there is a little hydrologic data available. Second, these models contain too many parameters which are difficult to estimate in Practice and which 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 are needed in developing countries are simple models which can provide reasonable simulations and need little data. The 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 modeling 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 hydrologic 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 surface runoff hydrograph produced at the outlet of the basin. On the other hand, small watersheds require minimal routing in long-term hydrologic simulation utilizing a time interval of I-d or larger. Consequently, the SCS-CN method has been used in long-term hydrologic simulation and several models have been developed in the past two decades. The models of Williams and LaSeur (1976), Huber et al. (1976) and Krusel (1980) that have been applied with varying degrees of success (Woodward and Gburek, 1992) are notable among others. The model of Soni and Mishra (1985) is a variant of the Hawkins, (1978) model. The generally available and frequently cited models of Williams and LaSeur (1976) and Hawkins (1978) along with the recent models of Pandit and Gopalakrishnan (1996) and Mishra et al. (1998) are described to help better understand the mathematical treatment of hydrological processes by the SCS-CN method. Mishra and Singh(2002a) proposed a continuous variation of antecedent moisture directly within the runoff equation. Jain et al., 2006 incorporates storm duration, a non-linear Ia-S relation and simple continuous moisture content in runoff estimation. Sahu et al. Model, (2007) in an attempt is made to develop an expression for initial soil moisture store level (V_0) to make the model a continuous watershed model. Geetha et al. (2008) Model A new lumped conceptual model based on the Soil Conservation Service Curve Number (SCS-CN) concept has been proposed in this paper for long-term hydrologic simulation. LTHS MICHEL MODELS (2010) the Michel et al. (2005) model is modified to avoid the unrealistic sudden jump in initial moisture level V_0 by incorporating conceptual SMA procedure and variation of daily CN based on antecedent moisture amount instead of antecedent moisture condition. Durbude et al. (2011) Model the Michel et al. (2005) model is modified to avoid the unrealistic sudden jump in V_0 by incorporating conceptual SMA procedure and variation of daily CN based on antecedent moisture amount instead of antecedent moisture condition.

2.2.1 Williams-LaSeur Model (1976)

Williams LaSeur (1976) proposed a model based on the existing SCS-CN method which is based on the water balance equation and two fundamental hypotheses (Chapter 3). The SCS-CN parameter potential maximum retention S is linked with the soil moisture (M) according to equation (2.1) expressed as:

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

where, S_{abs} is the maximum potential maximum retention, which is taken as 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 and 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, be 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 dt 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:

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

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, is the average monthly lake evaporation for day t, and T is the number of days between 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 Tday period is added to M before equation (2.3) is solved. However, runoff is not abstracted from rainfall until the end of 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 (2.2) is modified for rainfall P as

$$M_T = \frac{M+P}{1.0+b_c M \sum_{t=1}^{T} Et} - Q \qquad (2.4)$$

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:

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

The procedure is repeated for each storm in the rainfall series. Thus, the William Laseur model can also be applied to the pre-identified rainstorm 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 estimate of b_c is derived from the average annual rainfall and runoff values as

$$DP = \frac{AVP-AVQ}{365} \qquad (2.6)$$

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 day; (b) M is the average soil moisture index, M_A (c) E is the average lake evaporation; and (d) P = Q = O for the day. For this situation, equation (2.4) can be recast as

$$M_t = \frac{M_A}{1.0+b_c M_A E_t} \qquad (2.7)$$

in which, M_A is computed from equations $S = \frac{1000}{CN} - 10$ 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

$$DP = M_A - M_T \qquad (2.8)$$

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

$$DP = M_A - \frac{M_A}{1.0+ b_c M_A E_t} \qquad (2.9)$$

From which b_c can be derived as

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

The simulation begins 1 year before the actual calibration Period because of a priori 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 ratio of the AMCII 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 of b_c lose the physical soundness of the model apart from the undesirable loss of I-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 $S_{abs} = S_0 = S$, 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, $M_t = S_0$ its substitution into equation (2.3) leads to

$$(S_0 - S_t)/S_0 = \frac{1}{(1 + b_c S_0 \bar{E}_t)} \qquad (2.11)$$

where E is the average rate of evapotranspiration. Here, $(S_0$ -S,)/ $S_0 = F/S_0$' 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 $I_0 = b\,c\,S_0^2E$), a substitution of these relationships into equation $P = I_a + F + Q$ yields $Q=P\,S_0 / (S_0 + P)$ or $Q=PS/(S+P)$, which holds for F in the existing SCS-CN approach, rather than Q, and therefore equation (2.3) is physically unrealizable.

2.2.2 Hawkins Model (1978)

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 $P \rightarrow \infty$, the maximum possible water is equal to S_t as below:

$$S_t= (1+\lambda)\ S \tag{2.13}$$

Assuming $\lambda = 0.2$, $S_t= 1.2S$.

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 ($t + \Delta t$), Δt is the storm duration, can be derived from the moisture balance as:

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

Here, 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 be written as

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

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 CN

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

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

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

Since ET, P arid Q in the equation (2.18) correspond to the time duration Δt and these are known quantities, Q can be computed from equation (1) for a given CN_t Input of these values along with the known value of ET yields CN at time level $(t, t + \Delta t)$.

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 $\varphi - \theta$ relationship, according to which S is directly proportional to the average φ which approaches infinity as $\theta \rightarrow 0$. Under the situation that the soil is fully saturated or), $\theta \rightarrow n$ (soil porosity), $\varphi \rightarrow 0$. 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 soil profile to the root zone of 1.2m for computing S.

The advantage of 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 WilliamLaSeur 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 (2.12), the term $(I_a + S)$ takes part in the dynamic infiltration process, rather than the S alone. 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
5. Hawkins model considers the maximum F amount equal to (I_a+ S).
6. Substitution of P = 0 in equation (2. 12) yields Q = 0.05 S, which is impossible. Although equation P = I_a+F +Q, where P is total rainfall, I_a is initial abstraction, F is actual infiltration, and Q is direct surface runoff. This equation is stated to be valid for $P \geq 0.2S$, 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 → O. Thus, at time t (= St) Corresponds to CN at time t (=CN_t) Equation (2.12), therefore needs modification by substitution of 1000 for 1200.

2.2.3 Pandit and & Gopalakrishnan Model (1996)

Pandit and Gopalakrishanan (1996) suggested a continuous simulation for computing the annual amount of runoff for computing annual pollutant loads. This model is specifically useful for urban areas characterized primarily by the percent imperviousness. It involves the following steps:

(i) Determine the pervious curve number for AMC II.

(ii) Determine the directly connected impervious area of the urban watershed.

(iii) Estimate daily runoff depth for both pervious and impervious areas separately using equation (2.5).

(iv) Determine the actual AMC based on the previous 5- day rainfall and modify CN using equations

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

$$CN_{III} = \frac{CN_{II}}{0.427+0.00573CN_{II}}; \; r^2 = 0.994 \text{ and } SE = 0.7CN \qquad (2.20)$$

such that CN does not exceed 98.

NEH-4 identified three antecedent moisture conditions (AMC): AMC I, AMC II, and

AMC III for dry, normal and wet conditions of the watershed, respectively. As shown in Fig. 2.1, AMC I correspond to the lower enveloping CN, AMC II the median 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.

(v) Calculate the yearly storm runoff depth by assuming the runoff for each day.

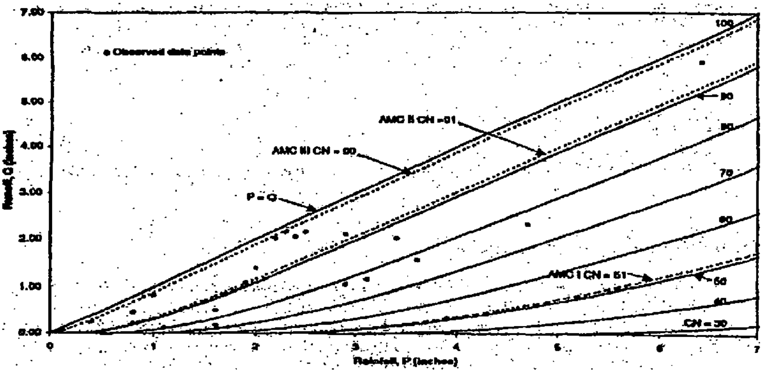


Fig 2.1 Determination of CN for AMC I through AMC III using existing SCS-CN model

In Summary, the method is very simple, allows sudden jumps in 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.

2.2.4 Mishra et al. (1998)Model

The Mishra et al. model assumes CN variation with time dependent on AMC (Ponce and Hawkins, 1996) only. The computed rainfall- excess Q (equation 2.5) is transformed to direct runoff DO_t using a linear regression approach, analogous to the unit hydro graph 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 as 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 Hawkins models.

(2) It allows sudden jumps in CN values when changing from one AMC to another AMC level.

(3) The use of linear regression equation invokes the problem of mass balance, for the sum of 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 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).

2.2.5 Mishra and Singh (2002a) Model

Since three AMC levels used in the existing SCS-CN methodology permit unreasonable sudden jumps in CN and hence a corresponding jump in estimated runoff, a continuous equation is needed to estimate the antecedent moisture. For achieving such a continuous equation a good attempt was made by Mishra and Singh (2002a). Using C=S_r concept. Where C is the runoff coefficient (= Q/ (P-I_a)) and S_r = degree of saturation, Mishra and Singh (2002a) modified equation ($\frac{Q}{P-I_a} = \frac{F}{S}$) for antecedent moisture M as:

$$\frac{Q}{P-I_a} = \frac{F+M}{S+M} \tag{2.21}$$

which upon substitution into eq. (P=I_a+F+Q) Leads to

$$Q = \frac{(P-I_a)(P-I_a+M)}{P-I_a+M+S} \tag{2.22}$$

Here, I_a is the same as in eq (I_a=λS). The following procedure was used for accounting antecedent moisture M. The procedure assumes that (a) the watershed is dry 5-days before the onset of rainfall. (b) The antecedent moisture (M) on the day of onset of rainfall is equal to the amount of water infiltrated (F) due to antecedent 5-day rainfall amount (P_5) at a time and (c) the runoff equation is valid for P=P_5.

Runoff produced byP_5 is given as

$$Q=\frac{(P_5.-\lambda S_1)^2}{P_5-\lambda S_1+S_1} \tag{2.23}$$

where S_1 is the potential maximum retention corresponding to AMC-I. Now antecedent moisture is given by

$$M=F=(P_5-Ia)-Q$$

Or

$$M=(P_5-\lambda S_1)-\frac{(P_5.-\lambda S_1)^2}{P_5-\lambda S_1+S_1} \tag{2.24}$$

which leads to

$$M=\frac{S_1(P_5.-\lambda S_1)}{P_5+(S_1-\lambda S_1)} \tag{2.25}$$

If λ=0.2, above equation gives

$$M=\frac{S_1(P_5.-0.2S_1)}{P_5+0.8S_1} \tag{2.26}$$

The above equation is used to determine M in Mishra and Singh (2002a)model. Here S_1 may be treated as absolute maximum retention capacity (S_0) since $S_0=S+M$, we get $S_1=S+M$

Using the above two equations one obtains (Mishra et al2003a):

$$M=0.5\left[-1.2S+\sqrt{0.64S^2+4P_5S}\right] \tag{2.27}$$

Here +sign before the square root is retained for M to be greater than or equal to Zero. However above equation can be generalized (Mishra et al., 2004a) by replacing 2.0 by λ and in this case M is expressed as:

$$M=0.5\left[-(1+\lambda)S+\sqrt{(1-\lambda)^2S^2+4P_5S}\right] \tag{2.28}$$

This model uses the above equation for determining M. Mishra et al. (2004a) evaluated the AMC dependent rainfall runoff models which are based on SCS-CN method.

2.2.6 Jain et al. (2006)Model

Jain et al. (2006) identified the existence of following issues in the conventional SCS-CN model: Implementation of AMC procedure, I_a-S relationship and effect of storm intensity of duration in runoff estimation. Based on these identified issues, Jain et al. suggested a new model formulation to enhance the SCS-CN model. This is expressed as follows:

$$\mathrm{Q} = \frac{(P_c - \mathrm{I_{ad}})(P_c - \mathrm{I_{ad}} + \mathrm{M})}{P_c - \mathrm{I_{ad}} + \mathrm{M} + \mathrm{S}} \qquad (2.29)$$

where $P_c > \mathrm{I_{ad}}$, otherwise Q=0.

A non-linear $\mathrm{I_a}$-S relation has also been given as below:

$$\mathrm{I_{ad}} = \lambda \mathrm{S} \left(\frac{P_c}{P_c + S}\right)^{a} \qquad (2.30)$$

M, the 5-day antecedent moisture, is computed using the equation ($\mathrm{M} = \gamma P_5$), as in Mishra and Singh model:

$$\mathrm{M} = \gamma P_5$$

P_c and S are calculated as follows:

$$P_c = P_0 \left(\frac{t_p}{\bar{t}_p}\right)^{\beta}$$

$$S = \frac{25400}{CN} - 254$$

In these equations P_0 = observed rainfall; P_c=adjusted rainfall; $\bar{t_p}$ = mean storm duration; t_p = storm duration; and P_5 = antecedent 5-day precipitation amount. All the equations represents an enhanced form of the run-off curve number model (Jain et al., 2006), which incorporates storm duration, a non-linear I_a-S relation and a simple continuous moisture content in run-off estimation. This model has five parameters.

2.2.7 Sahu et al. (2007) Model

The SCS-CN method has been a topic of much discussion, especially in the last three decades. Recently, Michel et al. (2005) pointed out several inconsistencies in the soil moisture accounting (SMA) procedure used in the SCS-CN method and developed a procedure that is more consistent from the SMA viewpoint. However, the model proposed by them does not have any expression for initial soil moisture store level (V0) and hence there is a scope for further improvement. Like the original method, there is sudden jump in V0 and therefore a quantum jump in computed runoff is possible. In the present study, an attempt is made to develop an expression for V0 to make the model a continuous watershed model. Then, the performance of the new model is compared with the model proposed by Michel et al. and the original SCS-CN model by applying them in a large number of small watersheds in the United States. The present model was found to perform significantly better than both the original SCS-CN model and the model proposed by Michel et al. (2005).

2.2.8 Geetha et al. (2008) Model

The proposed model differs from the original model, as in daily flow simulation, the original SCS-CN method is used to compute the direct surface runoff considering the rainfall of the current day utilizing the CN-values corresponding to antecedent 5-day AMC, allowing unrealistic sudden quantum jumps in CN-variation. Secondly, the value of initial abstraction coefficient is fixed as 0.2, which has shown to be varying in literature (For example, Mishra and Singh 2003). The proposed long term hydrologic model obviates these limitations and is capable of simulating, other than direct surface runoff, the total stream flow and its components such as surface runoff, through flow, and base flow which is conceptualized to have two different moisture stores, i.e. soil moisture store and ground water store. This continuous simulation model considers a daily time step interval for analysis. Thus, the present version is a significant enhancement over the previous ones utilizing original SCS-CN method.

This long term hydrologic model is capable of simulating stream flow and its components such as surface runoff, through flow, and base flow and is also conceptualized to have two different moisture stores, i.e. soil moisture store and ground water store.

2.2.9 LTHS Michel (2010)Models

In this model, total watershed runoff is quantified by incorporating sub-modules for surface run-off, lateral flow and base flow. Accounting for soil moisture and GWS is considered on daily basis. The basic difference between these models is in simulation of surface run-off only. The expressions for subsurface flow including lateral flow and base flow in both models are the same. As these models operate on daily time step, they require daily rainfall as input, and the observed runoff is used to calibrate model parameters and for their testing.

Michel *et al.* (2005) hypothesized the SCS-CN model to be valid not only at the end of the storm but also at any instant during a storm. Based on this hypothesis, they proposed a procedure more consistent from the SMA view point. However, the Michel *et al.* concept also needs refinement particularly in defining the initial moisture level (V0) and the SMA procedure used. In the generalized discrete form of Michel *et al.* model, V0 at the beginning of rainfall event is optimized, whereas it depends on the AMC in simplified form as pointed out by Sahu *et* *al.* (2007). This results in unrealistic sudden jump in V0 and further quantum jump in the computation of surface run-off. As V0 plays a vital role in the SMA procedure, Michel *et al.* did not provide mathematical expression for its computation. Incorporating the expression for V0, Sahu *et al.* (2007) modified the procedure of Michel *et al.* As similar expression for moisture threshold (Sa) also does not exist, there exists a need to modify the model of Michel *et al.* Thus, there exists a scope for recasting the SMA procedure by incorporating the variation of daily CN with respect to the variability of antecedent rainfall and moisture availability prior to the rainfall and with possibility of representing base flow using linear and exponential stores for continuous hydrologic simulation. The present study suggests two new/modified continuous LTHS models based on the SCS-CN method and concept given by Michel *et al.* for SMA procedure. The first model uses the expression for computing direct run-off based on SMS level prior to rainfall occurrence (V0) for various AMCs proposed by Michel *et al.*, and subsurface flow computation based on the conceptual behavior of SMS and ground water store (GWS) as given by Putty and Prasad (2000), whereas the second model re-conceptualizes the Michel *et al.* SMA procedure to preclude the unrealistic sudden jump in CN and initial SMS level (V0) by deriving an expression for V0 based on antecedent moisture amount (AMA), leading to the advance soil moisture accounting (ASMA) procedure. Here, the first model is referred as Long-Term Hydrologic Simulation MICHEL model (LTHS MICHEL model), whereas the second model as Long-Term Hydrologic Simulation Advance Soil Moisture Accounting model (LTHS ASMA model). The proposed models along with existing Lumped Conceptual Rainfall–Runoff (LCRR) model (Geetha *et al.*, 2008) and the generalized Michel *et al.* continuous surface run-off model using same formulations for subsurface flow computations as in proposed models are tested for their applicability to 17 small watersheds falling in different agro-climatic zones of India. Here, it is noted that all the proposed models differ from each other in simulation of surface run-off

2.2.10 Durbude et al. (2011) Model

This study suggests two new/modified continuous LTHS models based on the SCS-CN method and concept given by Michel et al. for SMA procedure. The first model uses the expression for computing direct run-off based on SMS level prior to rainfall occurrence (V0) for various AMCs proposed by Michel et al., and subsurface flow computation based on the conceptual behavior of SMS and ground water store (GWS) as given by Putty and Prasad (2000), whereas the second model re-conceptualizes the Michel et al. SMA procedure to preclude the unrealistic sudden jump in CN and initial SMS level (V0) by deriving an expression for V0 based on antecedent moisture amount (AMA), leading to the advance soil moisture accounting (ASMA) procedure. Here, the first model is referred as Long-Term Hydrologic Simulation MICHEL model (LTHS MICHEL model), whereas the second model as Long-Term Hydrologic Simulation Advance Soil Moisture Accounting model (LTHS ASMA model). The proposed models along with existing Lumped Conceptual Rainfall–Runoff (LCRR) model (Geetha et al., 2008) and the generalized Michel et al. continuous surface run-off model using same formulations for subsurface flow computations as in proposed models are tested for their applicability to 17 small watersheds falling in different agro-climatic zones of India. Here, it is noted that all the proposed models differ from each other in simulation of surface run-off only. The proposed model performed better in high run-off producing (wet) watersheds than in low run-off producing (dry) watersheds, and the base flow was more and less significant in high and low run-off producing watersheds, respectively, whereas evapotranspiration showed a reverse trend.

CHAPTER 3
METHODOLOGY

The Soil Conservation Service-Curve Number (SCS-CN) method is one of the most popular and simple methods used in rainfall-runoff modeling. Its simplicity lies in the fact that it requires less number of inputs, such as rainfall data and a single parameter called as Curve Number (CN) to estimate the runoff. The SCS-CN method was developed in 1954 and is documented in Section 4 of the National Engineering Handbook (NEH-4) published by the Soil Conservation Service (now called the Natural Resources Conservation Service),U.S. Department of Agriculture in 1956. The document has since been revised in 1964, 1965, 1971, 1972, 1985, and 1993. The SCS-CN method is the result of exhaustive field investigations carried out during the late 1930s and early 1940s and the works of several early investigators, including Mockus (1949), Sherman (1949), Andrews (1954), and Ogrosky (1956). The passage of Watershed Protection and Flood Prevention Act (Public Law 83-566) in August 1954 led to the recognition of the method at the Federal level and the method has since witnessed myriad applications all over the world. It is one of the most popular methods for computing the volume of surface runoff for a given rainfall event from small agricultural, forest, and urban watersheds. The method is simple, easy to understand and apply, stable, and useful for ungauged watersheds. The primary reason for its wide applicability and acceptability lies in the fact that it accounts for most runoff producing watershed characteristics: soil type, land use/treatment, surface condition, and antecedent moisture condition. This chapter describes the existing SCS-CN method, the concept of curve number and factors affecting it, and its advantages and limitations.

In mid 1930's, an acute need for hydrologic data for design of conservation practices was felt and eventually, the Soil Conservation Service (SCS) was established under the United States Department of Agriculture (USDA). The major objectives of SCS were to set up demonstration conservation projects and evaluate the design and construction of soil and water conservation practices. To that end, several experimental watersheds were set up at different locations for collecting data on rainfall, runoff and associated factors. According to the Flood Control Act of 1936 (Public Law 74-738), USDA carried out surveys and investigations for installing measures for retarding flows from watersheds, which is a classical hydrologic problem.

It eventually led to the evaluation of the effect of watershed treatment and/or conservation measures on the rainfall-runoff process. The data collected from experimental watersheds were, however, found to be scant and covering only a marginal fraction of the conditions affecting the rainfall-runoff process in watersheds (Andrews, 1954). Therefore, a need for collecting data for carrying out infiltration studies was felt.

Using the sprinkler-type infiltrometer, thousands of infiltration tests on field plots of 6 feet wide and multiples of 12 feet long were carried out during the late 1930's and early 1940's. For economic reasons, another FA infiltrometer (Rallison and Miller, 1982) was devised for plots of 12 x 30 inches and it was used extensively. Using these infiltration data, a rational method for estimating runoff under various cover conditions was developed. For that purpose, SCS hired three private consultants, W. W. Homer, R. E. Horton, and R. K. Sherman. Horton (1933) characterized the infiltration capacity curves and Homer (1940) concentrated on the development of infiltration capacity from small watershed data. Their studies resulted in the development of a series of rainfall retention rate curves and rainfall- excess and time-of-excess curves for computing runoff volume from field plots. This method, however, required time-distributed rainfall data, and therefore, its application was severely restricted in many areas.

3.1 SCS-CN THEORY

The SCS-CN method consists of

(a) Water balance equation:

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

(b) Proportional equality hypothesis:

$$\frac{Q}{P-I_a} = \frac{F}{S} \tag{3.2}$$

c) I_a -S hypothesis

$$I_a = \lambda S \tag{3.3}$$

where, P= total rainfall, I_a =initial abstraction, F=cumulative infiltration excludingI_a, Q=direct runoff, and S= potential maximum retention or infiltration, also described as the potential postinitial abstraction retention (McCuen, 2002). All quantities in above equation (A) through (C) are in depth or volumetric units

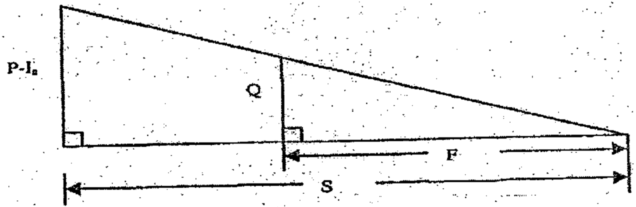


Figure 3.1: Proportionality concept

The fundamental hypothesis equation (3.2) is primarily a proportionality concept as shown in Fig.3.1. Apparently, as Q→ (P-I_a), F →S. This proportionality enables partitioning (or dividing) (P-I_a) into two surface water (Q) and subsurface water (F) for given watershed characteristics or S. This partitioning, however, undermines the saturated overland flow or source area concept that allows runoff generation from only saturated or wet portions of the watershed. Consequently, the statistical theory (Moore and Clarke, 1981; 1982; 1983; Moore, 1983; 1985) based on the runoff production from only saturated (independent or interacting) storage element is negated. According to the SCS-CN method, the extent of runoff contribution of a storage element depends on its capacity or, alternatively, the magnitude of S and, therefore, the whole watershed should contribute to runoff, if S is taken to be a definite quantity. Thus the ratio of the wet and total areas describing the contributing portion should be equal to one.

Parameter S of the SCS-CN method depends on the soil type, land use, hydrologic condition, and antecedent moisture condition (AMC). The initial abstraction accounts for the short- term losses, such as interception, surface storage, and infiltration. Parameter λ is frequently viewed as a regional parameter dependent on geologic and climatic factors (Bosznay, 1989; Ramasastri and Seth, 1985). The existing SCS-CN method assumes λ to be equal to 0.2 for practical applications. Many other studies carried out in the UnitedStates and other countries (SCD, 1972; Springer et al., 1980; Cazierand Hawkins, 1984; Ramasastri and Seth, 1985; Bosznay, 1989) report λ to vary in the range of (0, 0.3).

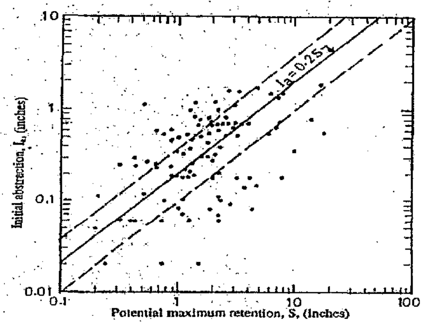


Figure 3.2: Relation between initial abstraction Ia and potential maximum retention, S.

The second hypothesis of the SCS-CN method ($I_a = \lambda S$) linearly relates the initial abstraction to the maximum potential retention. It is based on the results of Fig. 3.2 (SCS, 1971) depicting the plot between I_a and S. The data for S and I_a were derived from rainfall- runoff records of watersheds less than 10 acres in area.

The S values were derived from rainfall- runoff plots prepared for determining CN for AMC II. It is apparent from Fig.3.2 that more than 50% the data points lie within the limits of $0.095 \leq \lambda \leq 0.038$. Errors in S were largely attributed to the computation of the average rainfall of the watershed. The I_a - values were computed by accumulating the rainfall amount from the beginning to the time of start of runoff. The large scatter in the data points in Fig. 3.2 was attributed to the errors in the estimates of I_a due to (SCS, 1971):

(i) The difficulty in determining the actual time of the start of rainfall because of storm travel and lack of instrumentation.
(ii) The difficulty in determining the time of the start of runoff largely due to time lag in runoff from the watershed, and
(iii)Impossible determination of the amount of interception losses prior to runoff and its

delayed contribution to runoff. λs originally hypothesized (SCS, I97I), parameters includesI_a. For this condition, equation $I_a = \lambda s$ can be re-written as (Chen, 1982):

$$I_a = \frac{\lambda}{1-\lambda} S \tag{3.4}$$

For $\lambda = 0.2$ equation (3.4) is recast as $I_a = 0.25S$. Combining equation (3.1) and (3.2), the popular form of the SCS-CN method is obtained:

$$Q = \frac{(P-I_a)^2}{P-I_a+S} \tag{3.5}$$

This equation is valid for $P \geq I_a$; $Q = 0$ otherwise. For $\lambda = 0.2$, equation (3.5) can be written as

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \tag{3.6}$$

where Q = actual amount of direct surface runoff, I_a = initial abstraction, and S = potential maximum retention.

The existing SCS-CN method equation (3.6) is a one parameter model for computing surface runoff from daily storm rainfall, for the method was originally developed using daily rainfall-runoff data of annual extreme flows (Rallison and cronshey, 1979). Mockus (1964) [In: Rallison, 1980] described the physical significance of parameter S of equation (1) as follows: "... S is that constant and is the maximum difference of (P-Q) that can occur for the given storm and watershed conditions. S is limited by either the rate of infiltration at the soil surface or the amount of water storage available in the soil profile, whichever gives the smaller S value. Since infiltration rates at the soil surface are strongly affected by the rainfall impact, they are strongly affected by the rainfall intensity." This description, however, compares the magnitude of infiltration rate with the volume of water retention in the soil, which is unwarranted.

Since parameter S (mm) in equation (3.6) can vary in the range of $O \leq S \leq \infty$, it is mapped into a dimensionless curve number (CN), varying in a more appealing range $0 \leq CN \leq 100$, as follows:

$$S = \frac{25400}{CN} - 254 \tag{3.7}$$

The underlying difference between S and CN is that the former is a dimensional quantity (L) whereas the latter is a non-dimensional quantity. Although CN theoretically varies from 0 to 100, the practical design values validated by experience lie in the range (40, 98) (Van Mullem, 1989).

Since its inception, the method has been modified, restructured, strengthened based on its limitations and applications. In fact, it is renamed too as "Natural Resources Conservation Service - Curve Number (NRCS-CN) method" from 1994 onwards, primarily with objective to widen the scope. The only unknown parameter of this method is the CN, which is estimated in various ways by researchers. CN is varied as CN I, CN II or CN III according to the antecedent 5-d rainfall index. Each storm is assigned a CN-value based on the antecedent 5-d rainfall amount and the corresponding S-value from equation (3.7) is used in equation (3.6) for computing the rainfall-excess or direct surface runoff.

3.2 RUNOFF CN ESTIMATION

The basic parameter CN of the SCS-CN model requires the watershed characteristics such as, land use and treatment classes (Agricultural, Range, Forest, and more recently, Urban (SCS, 1986)), Antecedent Moisture Condition (AMC), Hydrologic Soil Group information (A, B, C, and D) and Hydrologic condition (Poor, Fair and Good) of a watershed. From the error analysis, Hawkins (1975) pointed out that the errors in CN may have much more serious consequences than errors of similar magnitude in P, but for a considerable precipitation range (up to about 9 inches). Thus, it is clearly understood that the accurate CN estimation is of significant importance in storm runoff calculation.

3.3 HYDROLOGIC SOIL-COVER COMPLEX NUMBER PROCEDURE

This method primarily needs the watershed characteristics such as land use and soil type. According to National Engineering Handbook-4 (NEH-4), these soil types were broadly classified into four Hydrologic Soil Groups: A, B, C, and D: 1) 'A' Soils having high infiltration rates, even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission. 2) 'B' Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission. 3) 'C' Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission. 4) 'D' Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material. These soils have a very slow rate of water transmission.

SCS developed soil classification system that consists of four groups, which are identified by the letters A, B, C and D. Soil characteristics that are associated with each group are:

Group A soils has a low runoff potential due to high infiltration rates even when saturated (7.6 mm/hr to 11.4 mm/hr). These soils primarily consist of deep sands, deep loess, and aggregated silts.

Group B soils have a moderately low runoff potential due to moderate infiltration rates when saturated (3.8 mm/hr to 7.6 mm/hr). These soils primarily consist of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures (shallow loess, sandy loam).

Group C has a moderately high runoff potential due to slow infiltration rates (1.3 mm/hr to 3.8 mm/hr if saturated). These soils primarily consist of soils in which a layer near the surface impedes the downward movement of water or soils with moderately fine to fine texture such as clay loams, shallow Sandy loams, soils low in organic content, and soils usually high in clay.

Group D soils have a high runoff potential due to very slow infiltration rates (less than 1.3 mm/hr if saturated). These soils primarily consist of clays with high swelling potential, soils with permanently high water tables, soils with a clay pan or clay layer at or near the surface, shallow soils over nearly impervious parent material such as soils that swell significantly when wet or heavy plastic clays or certain saline soils.

3.4 COVER TYPE

The most cover types are vegetation, bare soil and impervious surface. There are a number of methods for determining cover types. The most common are field reconnaissance, aerial photograph and land use map.

3.5 HYDROLOGIC CONDITION

Hydrologic condition indicates the effects of cover type and treatment for infiltration and runoff and is generally estimated from density of plant and residue cover on sample areas. Good hydrologic condition indicates that the soil usually has a low runoff potential for that specific hydrologic soil group, cover type, and treatment. Some factors considering the effect of cover on infiltration and runoff are (a) canopy or density oflawns, crops, or other vegetative areas; (b) cover; (c) amount of grass or close-seeded legumesin rotations; (d) percent of residue cover; and (e) degree of surface roughness.

3.6 ANTECEDENT MOISTURE CONDITION

The amount of rainfall in a period of 5 to 30 days preceding a particular storm referred to as antecedent rainfall and the resulting condition in regards to potential runoff is referred to as an antecedent condition. This condition, which is most often, called antecedent moisture condition influences the direct runoff that occurs from a given storm, the effect of antecedent rainfall may also be influenced by infiltration and evapotranspiration during the antecedent period, which in turn affects direct runoff.

To determine the antecedent moisture conditions from data normally available, SCS developed three conditions, which were labeled AMC I, AMC II and AMC III. The soil condition for each is as follows:

1. AMC I represent dry soil with a dormant season rainfall (5-day) of less than 12.7 mm and a growing season rainfall (5-day) of less than 35.56 mm,
2. AMC II represents average soil moisture conditions with dormant season rainfall averaging from 12.7 to 27.94 nun and growing season rainfall from 35.56 to 53.34 mm, and
3. AMC III conditions represent saturated soil with dormant season rainfall of over 27.94mm and growing season rainfall over 53.34 mm.

Later, depending on the 5-day precipitation amount, AMC II (CN II) is convertible to AMC I(CN I) or AMC III (CN III) using any of the relations (Table 3.1) given by Sobhani(1975), Chow et al. (1988), Hawkins et al. (1985), and Neitsch et al. (2002),and also directly from the NEH-4 table (SCS, 1972). Here, the subscripts I-III in Table 3.1, and elsewhere in the text refer to AMC I-AMC III, respectively.

Table 3.1. Popular AMC dependent CN-conversion formulae.

Method	AMC I	AMC III
Sobhani (1975)	$\mathrm{CN_I} = \frac{\mathrm{CN_{II}}}{2.334 - 0.01334\mathrm{CN_{II}}}$	$\mathrm{CN_{III}} = \frac{\mathrm{CN_{II}}}{0.4036 + 0.005964\mathrm{CN_{II}}}$
Hawkins et al. (1985)	$\mathrm{CN_I} = \frac{\mathrm{CN_{II}}}{2.281 - 0.01281\mathrm{CN_{II}}}$	$\mathrm{CN_{III}} = \frac{\mathrm{CN_{II}}}{0.427 + 0.00573\mathrm{CN_{II}}}$
Chow et al. (1988)	$\mathrm{CN_I} = \frac{4.2\mathrm{CN_{II}}}{10 - 0.058\mathrm{CN_{II}}}$	$\mathrm{CN_{III}} = \frac{23\mathrm{CN_{II}}}{10 + 0.13\mathrm{CN_{II}}}$
Neitsch et al. (2002)	$\mathrm{CN_I} = \mathrm{CN_{II}} - \frac{20(100 - \mathrm{CN_{II}})}{\{100 - \mathrm{CN_{II}} + \exp[2.533 - 0.0636(100 - \mathrm{CN_{II}})]\}}$	$\mathrm{CN_{III}} = \mathrm{CN_{II}}\exp\{0.00673(100 - \mathrm{CN_{II}})\}$

Chronologically, the AMC-dependent CN-values given by NEH-4 in tabular form were represented by mathematical expressions given independently by Sobhani (1975), Smith and Williams (1980), Hawkins et al. (1985), Chow et al. (1988), and Neitsch et al. (2002). Smith and Williams (1980) developed a relation only for CN I to CN II while others provided for both, viz., CN I to CN II or CN III. According to Mishra et al. (Under review),the Hawkins et al. (1985) CN conversion formulae perform better than others, though there was About 0.1 % difference among them all over the range of CNII values from 50 to 100 for either the CNI or CN III conversions. Here, it is noted that the CN-values obtained from most soil-cover-moisture complexes in the field are generally greater than 40 (SCS, 1972). Therefore, the Hawkins et al. (1985) formulae are recommended for CN-conversion.

3.7 ESTIMATION OF CN FOR A WATERSHED

A curve number is an index that represents the combination of hydrologic soil group and land use and land treatment classes. Empirical analysis suggests that the CN was a function of three factors: soil group, the cover complex, and antecedent moisture conditions. The CN-values for different land uses, treatment, and hydrologic conditions are available elsewhere (Singh, 1992; Ponce 1989).

The values of CN for various land use on the above Hydrological soils can be taken directly from the available standard CN tables (Hydrologic soil-cover complex number) of NEH-4 (SCS, 1993). These CN values of NEH-4 table represent the average median site-CN, which corresponds to the curve that separated half of the plotted P-Q data from the other half for the given site. This is denoted as CN II, where the subscript stands for AMC II, indicating average runoff potential under average wetting condition of the watershed. This CNII can be derived either by the weighted CN approach or weighted Q approach, as below:

3.7.1 Weighted CN approach

In this approach, the CN-values of the respective hydrological-soil cover complex are multiplied with the respective percent areal coverage of the complexes, as follows:

$$CN_{aw} = \frac{\sum_{i=1}^{n}(CN_i * Ai)}{\sum_{i=1}^{n} A_i} \tag{3.8}$$

where CN_{aw} = area-weighted curve number for the drainage basin; CN = curve number for each land use-soil group complex; A, = area for each land use-soil group complex; and n= number of land use-soil group complex in drainage basin. Then, using this weighted CN, the runoff is estimated from Eqs. (3.6) and (3.7).

3.7.2 Weighted Q approach

Here, direct surface runoff (Q) is computed for each sub-areas of a watershed from Eq. (3.6) and (3.7), utilizing the CN-value derived for respective hydrological-soil cover complex of the sub-area. Finally, the area-weighted Q is computed as below:

$$Q_{aw} = \frac{\sum_{i=1}^{n}(Q_i * Ai)}{\sum_{i=1}^{n} A_i} \tag{3.9}$$

where Q_{aw} = area-weighted runoff for the drainage basin; and Q_i = runoff at each land use- soil group complex.Obviously, the weighted-Q method is superior to the weighted-CN method, as the former is more rational than the latter for water balance reasons. However, the weighted-CN approach is easier to work with the watershed having many complexes or with a series of storms. Mishra and Singh (2003) - pointed out that the computed runoff by above two approaches would significantly deviate for a wide range of CNs for various complexes in a watershed. In general, the weighted-CN method is less time consuming, but tends to be less accurate when compared to the actual measured runoff depth. The difference between the two methods is however is insignificant for total CN difference less than 5 and if the rainfall is high in magnitude.

The following two problems are generally encountered IN case of the above "Hydrologic soil-cover complex number" procedure:

1. The calculation of "Hydrologic soil-cover complex number" approach is much more sensitive to the chosen CN than it is to the rainfall depths (Hawkins, 1975; Bondelid et al., 1982).
2. It is difficult to accurately select the CNs from the available CN tables (Hawkins, 1984).

This method is generally used for ungauged watersheds and its utility is enhanced with the aid of remote sensing and Geographical Information System (GIS) techniques in distributed watershed modeling. Here, it is worth emphasizing that CN determination from field data is better than that from hydrologic soil-cover complex number method, as the latter leads to variable, inconsistent or invalid results (Hawkins, 1984).

3.8 OTHER METHODS

Due to the SCS-CN method being sensitive to accurate CN estimation for accurate runoff estimation, some researchers tried entirely different approaches. For example, Bonta (1997) evaluated the derived frequency distribution approach for determining watershed CNs from measured data, treating P and Q data as separate frequency distributions. This method gives fewer variable estimates of CN for a wide range of sample sizes than do the methods of asymptotic and Median-CN for CN-estimation. It is advantageous in limited P-Q data situation, and does not require watershed response type to estimate CN, as needed in the asymptotic method. Mishra and Dwivedi (1998) presented an approach to determine the upper and lower bounds or enveloping CNs, which are useful in high and low flow studies, respectively. McCuen (2002) found the quantity (100-CN) to fit the gamma distribution, which he used for developing the confidence intervals for CNs ranging from 65 to 95, with parameter estimation by Method of Moments (MOM). Later, Bhunya et al. (2003) provided a more reliable procedure for estimation of confidence interval by employing the Method of Maximum Likelihood (MOML), and Method of L-moment in addition to MOM as parameter estimation. These methods however require testing on a large data set.

3.9 PROPOSED SCS-CN BASED LONG TERM SIMULATION MODEL

The model is described as a sum of a few components described as follows:

3.9.1 Rainfall excess computation

For rainfall-excess computation, Q is computed as

$$Q = \frac{(P-I_a)^2}{P-I_a+S} \tag{3.10}$$

It is valid if $P-I_a>0$, otherwise $Q=0$. The infiltration is derived from water balance equation as follows:

$$F = P - I_a - Q$$

Here, $I_a = 0.2S$ and

$$S = 25400/CN - 254.$$

3.9.2 Computation of Evapotranspiration:

The Potential Evapotranspiration (PET) is computed using the pan evaporation as:

$$PET = PANC \times ET \tag{3.11}$$

where PANC is the pan coefficient, considered as 0.8 for the period from 1 day (Jan. 1) to 121 days, 0.6 for the period from 122 days to 244 days, and 0.7 for the period from 245 days to 365 days.

3.9.3 Routing of flow

The routing is performed using storage method incorporating lateral inflow as below:

$$Q_{(t+\Delta t)} = C_0\, Q_{t+\Delta t} + C_1 Q_t + C_2 Q_t \tag{3.12}$$

where t and $t+\Delta t$ are the time steps at Δt interval (= 1 day in daily simulation) and C_0, C_1 *and* C_2 are the routing coefficients:

$$C_0 = (\text{cour}\ (1 + \text{ALPHA})/(2 + \text{cour})) \tag{3.13}$$

$$C_1 = C_0$$

$$C_2 = (2 - \text{cour})/(2+\text{cour})$$

cour = Courant number = 1/K

K = storage coefficient

ALPHA = a coefficient for lateral inflow

3.9.4 Total Runoff

Total Runoff is usually taken as the sum of direct surface runoff and base flow. In this approach however it is considered as equal to above Q for ALPHA considers all other types flows including base flow and intra-basin transfer flows. Apparently, daily simulation model contains three parameters including CN_0, K, and ALPHA.

3.10 MODEL CALIBRATION

Model calibration in general involves manipulation of a specific model to reproduce the response of the catchment under study within some range of accuracy. In a calibration procedure estimation is made of the parameters, which cannot be assessed directly from field data. All empirical (black box) models and all lumped, conceptual models contain parameters whose values have to be estimated through calibration. The fully distributed physically-based models contain only parameters which can be assessed from field data, so that in theory a calibration should not be necessary if sufficient data are available. However, for all practical purposes the distributed, physically-based models also require some kind of calibration, although the allowed parameter variations are restricted to relatively narrow intervals compared with those for the empirical parameters in empirical or lumped, conceptual models. In principle three different calibration methods can be applied:

a. 'Trial and Error', manual parameter assessment
b. Automatic, numerical parameter optimization
c. A combination of (a) and (b)

The trial and error method implies a manual parameter assessment through a number of simulation runs. This method is by far the most widely used and is the most recommended methods, especially for the more complicated models. A good graphical representation of the simulation results is a prerequisite for the trial and error method. An experienced hydrologist can usually achieve a calibration using visual hydrograph inspection within 5-15 simulation runs.

Combination of the trial and error and automatic parameter optimization method could involve, for example, initial adjustment of parameter values by trial and error to delineate rough orders of magnitude, followed by fine adjustment using automatic optimization within the delineated range of physically realistic values. The reverse procedure is also possible, first carrying out sensitivity tests by automatic optimization to identify the important parameters and then calibrating them by trial and error. The combined method can be very useful but does not yet appear to have been widely used in practice.

Finally, given the large number of parameters in a physically based distributed model like the SHE model, it is not realistic to obtain an accurate calibration by gradually varying all the parameters one by one or in combination. A more sensible approach is to attempt a coarser simulation using only the few parameters to which the simulation is most sensitive, which is derivable from sensitivity analysis. However, experience suggests that the soil parameters will usually require the most attention because of their role in determining the amount of precipitation which infiltrates and hence the amount which forms overland flow.

The above methods of calibration consider single objective function. In case multi objective function is required to be considered, then two types of approaches, viz. classical approach and Pareto approach may be utilized. In classical approach a combined objective function is desired assigning the weights to the various objective function depending upon the user requirement. In Pareto approach a set of parameter values is determined using a search algorithm in such a way that the global optima are achieved considering the multi objective function. In this study, the non-linear Marquardt algorithm is employed to estimate model parameters and, in turn, calibrate the model.

3.11 GOODNESS OF FIT AND ACCURACY CRITERIA

In calibration an accuracy criterion can be used to compare the simulated and measured outputs. This enables an objective measure of the goodness of fit associated with each set of parameters to be obtained and the optimum parameter values to be identified. However, selection of an appropriate criterion is greatly complicated by the variation in the sources of error discussed in the last section. It further depends on the objective of the simulation (e.g. to simulate flood peaks or hydrograph shape) and on the model output variable, e.g. phreatic surface level, soil moisture content, and stream discharge or stream water level. Not a single criterion is entirely suitable for all variables and even for a single variable it is not always easy to establish a satisfactory criterion. Hence a large number of different criteria have been developed. The most widely used criterion is the sum of the squares of the deviations between recorded and simulated F^2 -value of a variable:

$$F^2 = \sum_{i=1}^{n} (QOBS_i - QSIM_i)^2 \qquad (3.20)$$

$QOBS_i$ = recorded value at time step i

$QSIM_i$ = simulated value at time step i

n= number of values (time steps) within the considered time period

All values of $QOBS_i$ and $QSIM_i$ are based on a time step, which may be one hour, one day or one month. One disadvantage with this criterion is that F^2 is dimensional (e.g. $(m^3/s)^2$). Therefore, the following non-dimensional form is often used:

$$R^2 = \frac{\frac{1}{n}\sum_{i=1}^{n}(QOBS_i - \overline{QOBS})^2 - \frac{1}{n}\sum_{i=1}^{n}(QOBS_i - QSIM_i)^2}{\frac{1}{n}\sum_{i=1}^{n}(QOBS_i - \overline{QOBS})^2} \tag{3.21}$$

$$\overline{QOBS} = \frac{1}{n}\sum_{i=1}^{n} QOBS_i$$

where R^2 is often denoted the coefficient of determination, the explained variance or the model efficiency. R^2 can vary from 0 to 1, where $R^2 = 1$ represents a complete agreement between recorded and simulated values. It is noted that the simple one parameter model. $QSIM_i = QOBS$ will give $R^2 = 0$. Although the R^2-criterion is a dimensionless measure it depends heavily on the variance in the recorded series. Thus comparison of R^2 values for different catchments or even for different periods in the same catchment makes no sense.

Among the other numerical criteria often used are the following:

$$F = \sum_{i=1}^{n}|(QOBS_i - QSIM_i)| \tag{3.22}$$

which is a measure of the accumulated deviation (absolute) between recorded and simulated values;

$$F^{2\log} = \sum_{i=1}^{n}(\log QOBS_i - \log QSIM_i)^2 \tag{3.23}$$

$$R = \frac{\sum_{i=1}^{n}(QOBS_i - \overline{QOBS})(QOBS_i - \overline{QSIM})}{\sum_{i=1}^{n}(QOBS_i - \overline{QOBS})^2 \sum_{i=1}^{n}(QSIM_i - \overline{QSIM})^2} \tag{3.24}$$

which does not focus as much on peak matching as does the F^2 criterion; and it is the linear correlation coefficient between the simulated and the recorded series.

It is perfectly feasible to calibrate a model by optimizing just one of the available criteria. However, a calibration based on 'blind' optimization of single numerical criterion risks producing physically unrealistic parameter values, which, if applied to a different time period, will give poor simulation results. In the same vein it should be remembered that the criteria measure only the correctness of the estimates of the hydrological variables generated by the model and not the hydrological soundness of the model relative to the processes being simulated. It is therefore recommended that, in a calibration, numerical criteria be used for guidance only. In general it is recommended that a combination of the following four conditions be considered in determining goodness of fit:

1. A good match between average simulated and recorded flows and good water balance.
2. A good agreement for the peak flows, with respect to volume, rate and timing.
3. A good agreement for low peaks.
4. A good overall agreement for hydrograph shape with emphasis on a physically correct model simulation.

These four conditions can be optimized numerically or subjectively through interactive computer graphics. In cases where all four criteria cannot be optimized simultaneously the priority depends on the objective of the project in question. Finally, although the use of numerical criteria has been emphasized above, the value of graphical comparison of simulated and observed hydrograph should not be overlooked. Although analyzed more subjectively, a graphical plot provides a good overall impression of the model capabilities is easily assimilated and may yield more practical information than does a statistical function. Graphical comparison should always be included in any examination of the goodness of fit of a simulated hydrograph.

3.12 MODEL VALIDATION

If the model contains a large number of parameters it is nearly always possible to produce a combination of parameter values, which permits a good agreement between measured and simulated output data for a short calibration period. However, this does not guarantee an adequate model structure or optimal parameter values. The calibration may have been achieved purely by numerical curve fitting without considering whether the parameter values so obtained are physically reasonable. Further, it might be possible to achieve multiple calibrations or apparently equally satisfactory calibrations based on different combinations of parameter values. In order to find out whether a calibration is satisfactory, or which of several calibrations is the most correct, the calibration should therefore be tested (validated) against data different from those used for the calibration (e.g. Stephenson and Freeze, 1974). According to Klemes (1986), a simulation model should be tested to show how well it can perform the kind of task for which it is intended. Performance characteristics derived from the calibration dataset are insufficient as evidence of satisfactory model operation. Thus the validation data must not be the same as those used for calibration but must represent a situation similar to that to which the model is to be applied operationally.

CHAPTER 4
STUDY AREA AND DATA AVAILABILITY

The study areas selected for the present study include a part of Betwa sub-basin falling between Bhopal and Vidisha in Madhya Pradesh (India), Ret watershed which is situated in the Kalahandi district of Orissa, and Siul watershed located in Chamba district of Himachal Pradesh. A brief description of each is given below.

4.1 DESCRIPTION OF BETWA BASIN

Betwa river rises in the Raisen district of Madhya Pradesh near village Barkhare, south west of Bhopal at an elevation of about 576 m above mean sea level and flows in a north-easterly direction for 232 km through Madhya Pradesh and enters the Jhansi district of Uttar Pradesh. After running for a further distance of about 261 km. in Jhansi, Jalaun, and Hamirpur districts of Uttar Pradesh, it joins river Yamuna near Hamirpur at about 106 m above mean sea level. The total length of the river from its origin to its confluence with the Yamuna is about 590 km, about 232 km in Madhya Pradesh and the balance in Uttar Pradesh. During its course up to the Yamuna, the river receives many tributaries, the important among them being Bina, Narain, Jamni, Dhasan and Birma on the right bank and Kaliasote, Halali, Bah, Sagar, Naren and Kethan on the left bank. The total catchment area of Betwa river is 46580 sq. km, out of the total catchment area 4122 sq.km area is taken in the present study as shown in the Fig.4.4.

4.1.1 Topography

The Betwa basin lies between the east longitudes of $77^{0}10'$ to $80^{0}20'$ and the north latitudes of $23^{0}10'$ and $20^{0}00'$. It is of rectangular shape, the maximum width from south to north being about 430 km and the maximum width from west to east being 155 km. The basin covers the areas of Bundelkhand uplands, the Malwa Plateau and the Vindhyan scarp lands in the districts of Tikamgarh, Sagar, Vidisha, Raisen, Bhopal, Guna, Shivpuri and Chhatarpur of Madhya Pradesh and the districts of Hamirpur, Orai, Jhansi and Banda in Uttar Pradesh.

4.1.2 Hydrogeology

The Ken and Betwa sub-basins, which form part of the Ganga basin, are varied in its geological setting. As per the report on Ground Water Resources and Development Prospects of Madhya Pradesh, prepared by the Central Ground Water Board, North-Central Region, Bhopal in March, 1994, the following types of hydro-geological formations are found in the Ken and Betwa basins. The older metamorphies occur in Panna, Chhatarpur and Tikamgarh districts of Madhya Pradesh. Ground water occurs in them only in the weathered mantle and the fractured zone underlying them. The wells are recorded to be generally up to 25 to 30 m in depth with water levels in the lean part of the year exceeding 10 m below ground level. Specific capacity of the wells in these formations ranges from 20-100 lpm/m of drawdown, where the thickness of the aquifer is commendable. Hydraulic conductivity is generally less than 1 m/d and the specific yield is generally less than 5%.

The purana formations of both Vindhyan and Cuddapah age comprise of orthoquartzites, limestones and shale sequence are found in parts of Panna, Raisen, and Bhopal districts. The wells located in these areas are easily capable of yielding 100-500 m^3/d for a drawdown of 3 m. Specific capacity is in the range from 100 to 300 lpm/m of drawdown and the hydraulic conductivity varies from 5-15 m/d. Similarly, specific yield is generally in the range of 5 to 15%. The Deccan traps cover the Guna, Vidisha, Damoh, Sagar, Bhopal and Raisen districts. The flows are generally 10-20 m in thickness, of which 25 to 40 percent is generally vesicular. The characteristic red bole beds generally form the masker horizons between the successive flows. The wells of these areas are capable of yielding 250 to 750 m^3/d for a drawdown of 3 to 6 m. The specific capacity ranges from 50 to 150 lpm/m of drawdown. Hydraulic conductivity ranges from 5 to 15 m/day. The specific yield in the area is generally in the range of 5 to 10%.

4.2 DATA AVAILABILITY

4.2.1 Hydro-Meteorology

The study area falls in the semi-arid to dry sub-humid climatic region of India with a single rainy season (July–September) followed by dry winter, and then a very dry summer. An investigation of monthly rainfall records at various stations in the study area revealed the rainfall to be highly variable and unevenly distributed. The percentage annual rainfall departures were estimated to identify the occurrence of meteorological drought years during 1951–1998. The investigation further revealed the stream flows to vary significantly from season to season and even month to month. For nearly two-third of the time of the available data, the flow in Betwa at Basoda and Rajghat was zero after April, and persisted till the arrival of next monsoon. The flow in Betwa at Mohna and Shahjina was never zero, and the flow in river Dhasan was dry at Garauli in April–June for about three-fourth of the time during 1982–2001.

The climatic condition of the river basin is largely influenced by the orographic effects. The elevation 1525 m is the approximate boundary between areas receiving the majority of precipitation in the form of rain. There are three raingauge stations in the catchment area and using Theisen polygon approach, rainfall for the catchment area has been computed. The discharge data is available at Bhopal site on the river. The effective area between the two sites is 4122 km². The discharge data for this catchment area was collected at Basoda barrage and converted to runoff in mm. In this part of the Betwa catchment, there is no contribution from snowmelt. The daily rainfall and evaporation data are available from 1993-2002. The runoff data is available for the same period.

4.2.2 Rainfall

There are at present 11 raingauge stations in the catchment of Betwa river at which long term records are available. Out of these, rainfall data of 4 raingauge stations situated in the catchment of stream between Bhopal and Vidisha is considered. The annual precipitation pattern is dominated by the monsoon from June to September during which about 50% of the total annual rainfall occurred. The average rainfall of the 4 raingauge stations are given in Table 4.1.

Table 4.1 Average monthly rainfall data of Betwa basin

Year	Jan.	Feb.	Mar.	Apr.	May.	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1993	0	9	11	2	9	442	1003	1100	1436	56	0	6
1994	52	29	11	45	19	1151	1818	1941	302	19	15	0
1995	78	10	107	11	1	229	1676	1151	754	150	0	42
1996	32	35	14	0	0	142	2460	2489	611	113	0	0
1997	25	0	18	12	20	354	1989	1464	566	404	274	324
1998	0	0	139	61	0	739	1874	1107	1096	114	21	0
1999	7	301	0	62	211	810	1646	1818	1892	463	0	0
2000	0	2	2	0	145	394	2551	545	107	4	1	0
2001	4	2	3	2	301	994	1348	1170	116	303	0	0
2002	0	106	45	26	19	384	342	2128	388	2	98	5

4.2.3 Evaporation

Monthly evaporation data which are observed at Bhopal are given in the table no.4.2 from the year 1993-2002.

Table 4.2 Evaporation data recorded at Bhopal

YEAR	1993	1994	1995	1996	1997	1998	1999	2000	2001	2002
PET(mm)	658	524	763	664	657	693	734	658	578	740

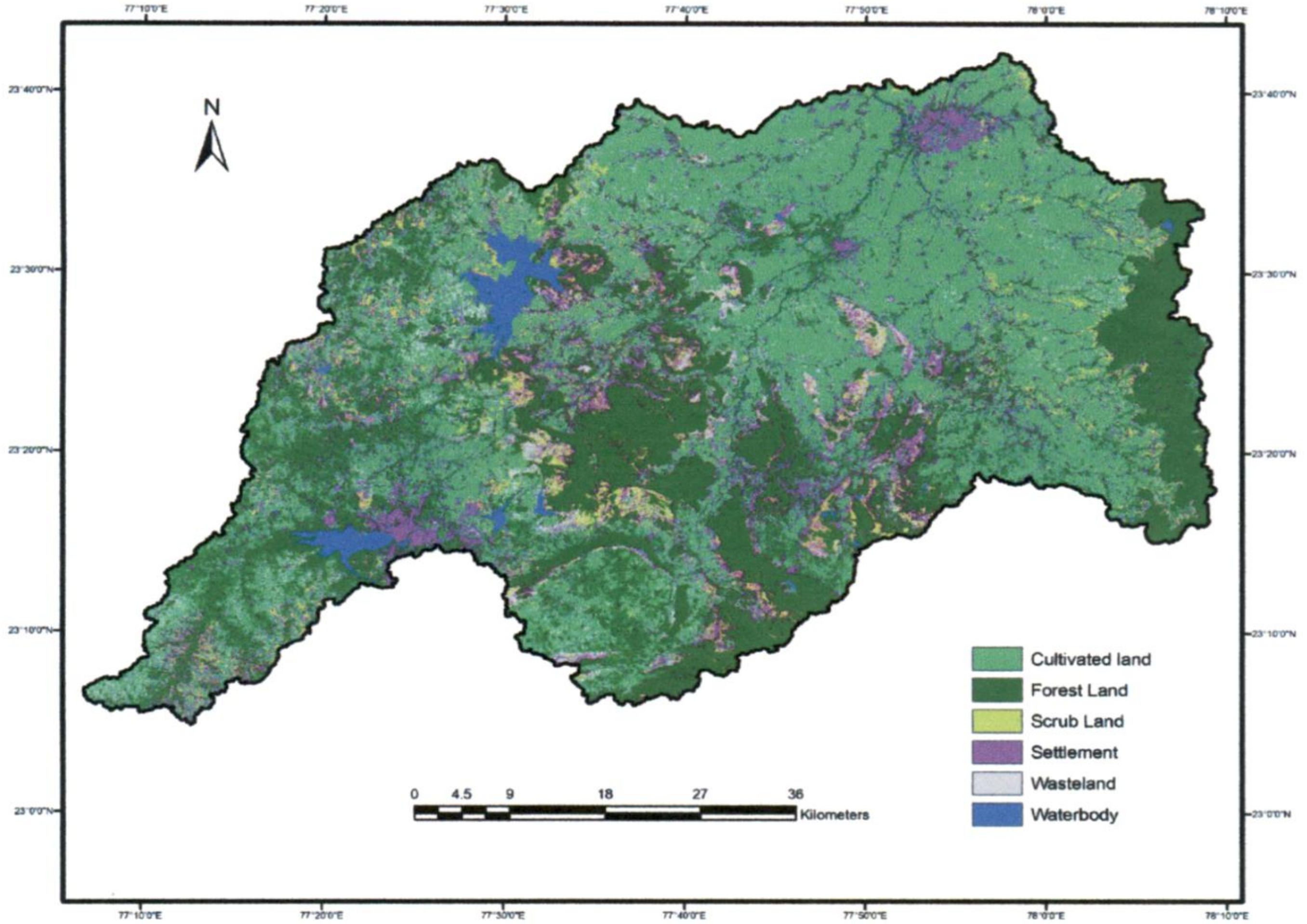
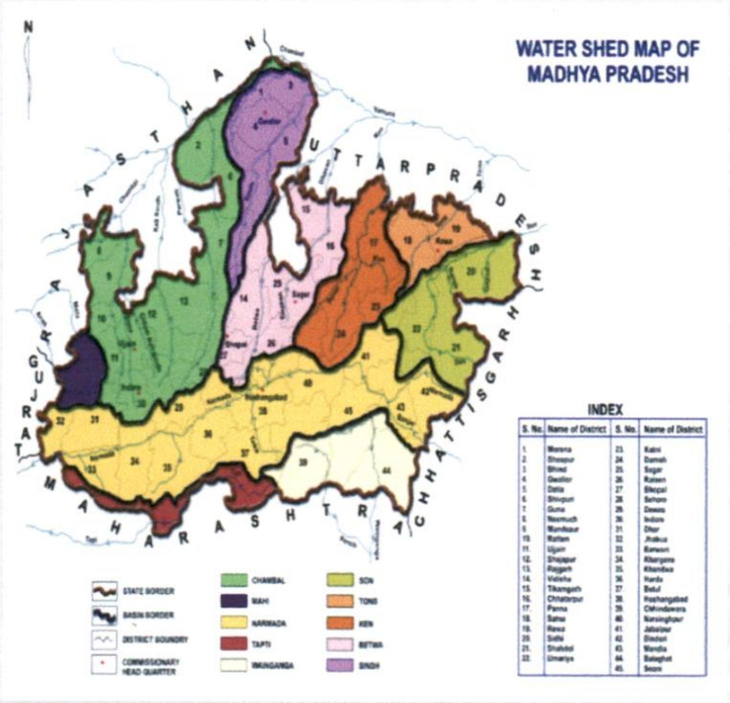


Fig.4.1 Landuse map of betwa watershed



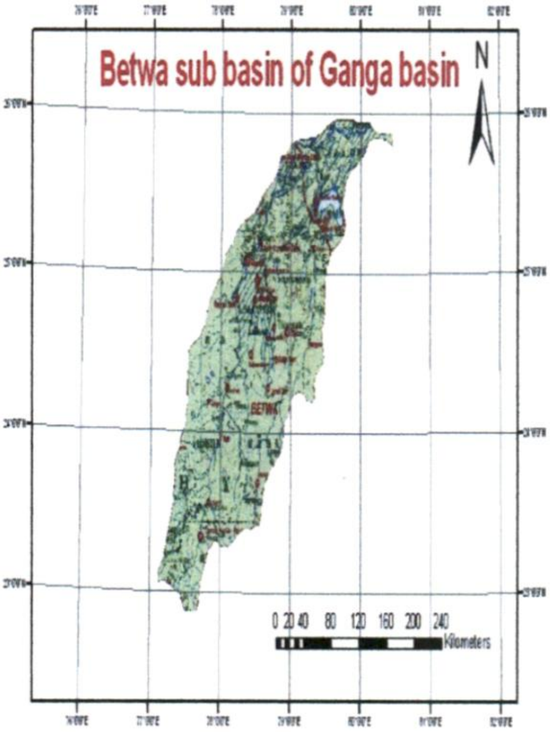


Fig. 4.2 Watershed map of M.P.

Fig. 4.3 Sub-Basin map of Betwa

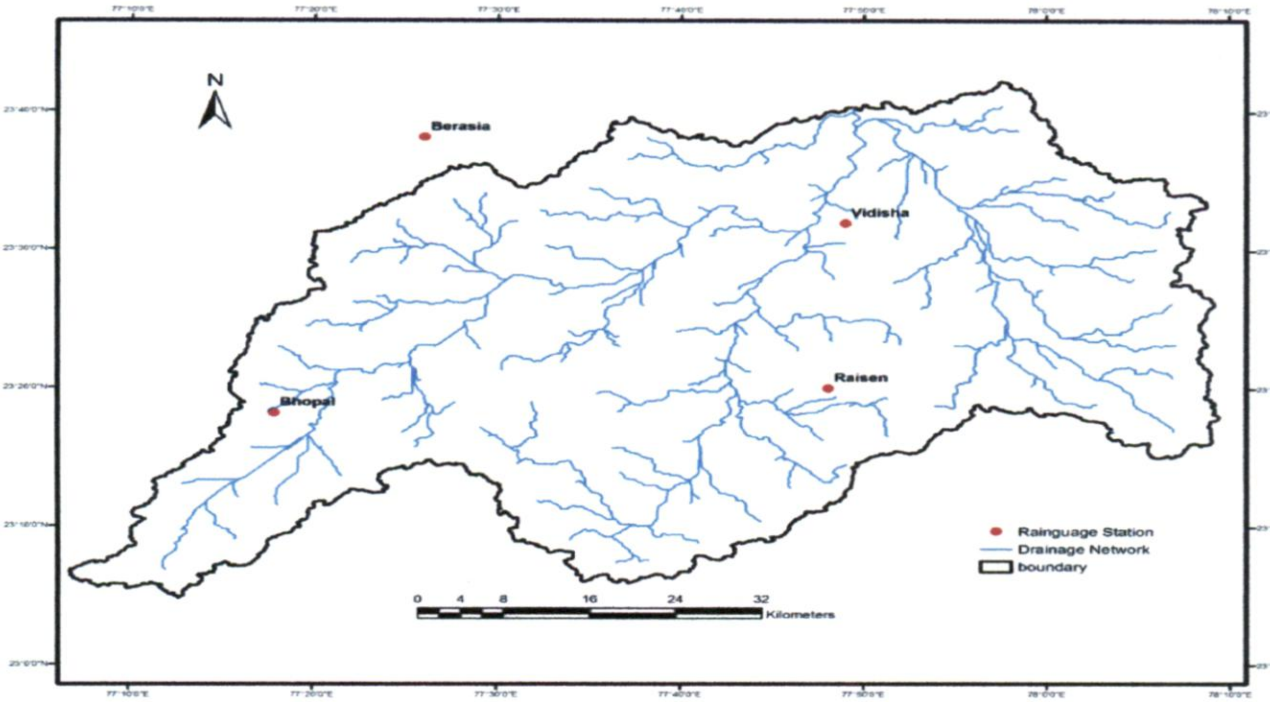


Fig.4.4 Drainage map of Betwa sub-basin.

4.3 DESCRIPTION OF RET WATERSHED

The Ret watershed is situated in the Kalahandi district of Orissa and is a part of Mahanadi river basin and encompasses an area of about 360 sq. km. Location map of the study area is shown in Fig. 4.6. It is located between 19'40' and 200 N latitudes and 83015' and 83025' E longitudes. The outlet point of the Ret river watershed is located 21 km from Bhawanipatna, the head quarter of Kalahandi district. The annual average rainfall of the study area is 1378.2 mm and 80% of the rainfall occurs during monsoon season. The temperature of the study area varies widely between 11° C (December) to 49° C (May). The topography of the watershed is moderately sloping. The climate of the area is sub-tropical, sub-humid monsoonic climate with three prominent seasons namely hot summer from March - mid June, rainy season from mid-June – September, and winter from October - February. Rice is the major crop grown in the region. The other crops are maize, finger millet, minor millets, niger, potato, brinjal and fruit tree such as mango, jack fruit, guava, papaya and sapota.

The District has two distinct physiographic regions, the plain lands and the hilly tracts. The plain region runs Southward up to Bhawanipatna and then westward through Junagarh and Dharmgarh and then further up to the boundary of the District. The plains cover about 59 percent of the total area of the District. The Hilly tracts are mostly located in the South western part of Bhawanipatna Subdivision. Some of the hilly regions are covered with dense forest.

4.3.1 Hydro-Meteorology

The climate of the District is of extreme type. It is dry except during monsoon. There is a large variation in day and night temperatures. The average annual rainfall of the district is 1378.20 mm. The variation in the rainfall from year to year is not large. The monsoon starts late in June and generally lasts up to September. 90% of the rainfall received from June to September. August is the month with more number of rainy days. About 28% of rainfall is received during this month. Drought is a normal feature of this district.

4.3.2 Temperature and Humidity

The summer seasons starts from the beginning of March. May is the hottest month when the maximum temperature is about 45 C (82 degree F). The temperature drops down with the onset of monsoon towards the second week of June and throughout the monsoon the weather remains cool. December is the coldest month, as the mean daily minimum temperature is recorded at 11 degree C. Relative humidity is generally higher from June to December. It is lower (27%) in the non-monsoon months. During August, it is the highest i.e. 70% and March is the month lowest when it is lowest 27%. Northern plateau (at 2150 msl) of Sunabeda in Komna blocks of Nawapara district has a colder climate, and so does the Rampur area (at 2700 ft above msl).

4.3.3 Soil and Land Classification

The district has five types of soils broadly classified as follows. The red laterite soil which is different in phosphorus and nitrogen is found all over the district. Mostly under the foothill and hillocks in Bhawanipatna and Dharmgarh tehsils. Occurrence of heavy soil is common. It is rich in potassium and nitrogen but poor in phosphorus. Sandyloan soil is seen in Lanjigarh and of the Bhawanipatna Tehsil. The area on the river bank of Udanti, Utei and Sagada are alluvial sandy and sandyloam spills. The fertility of soil in Dharmgarh and Jaipatna tehsil areas is high. The red soil, black clay, sandy loam, yellow soils occur in the district with following percentages:

1.	Red soil	31.68%
2.	Black clay (heavy)	13.90%
3.	Clay & sandy loam	54.44%

The soil map of the study area was obtained from the National Bureau of Soil Survey and Land use Planning (NBSSLUP), Nagpur. The soils of the study area are characterized by red soil, mixed red & black and medium black. The Ret watershed is occupied by Red soil and its texture is fine loamy. The soils of the Ret watersheds are strongly to moderate acidic with low to medium organic matter status and poor water retentive capacity.

Forest occupies 4,964 of the total geographical area of the district, i.e. not cultivated area of the District in the year 1993 is 375752 ha. In the same year, 11,602 ha was left as fallow lands or cultivable waste land.

4.4 DATA AVAILABILITY

4.4.1 Rainfall and Evaporation data:

The annual precipitation pattern is dominated by the monsoon from June to September during which about 50% of the total annual rainfall occurred. The average rainfall and evaporation data from year Jan. 2000 to Dec. 2009 are given in the Table 4.3.

Table 4.3 Rainfall and Evaporation Data observed at Bhawanipatna,Odisha.

YEAR	2000	2001	2002	2003	2004	2005	2006	2007	2008	2009
RAINFALL(mm)	1245	1159	1408	2157	1569	1508	1391	1348	1774	1427
PET(mm)	1038	1200	1224	751	844	1042	781	956	756	742

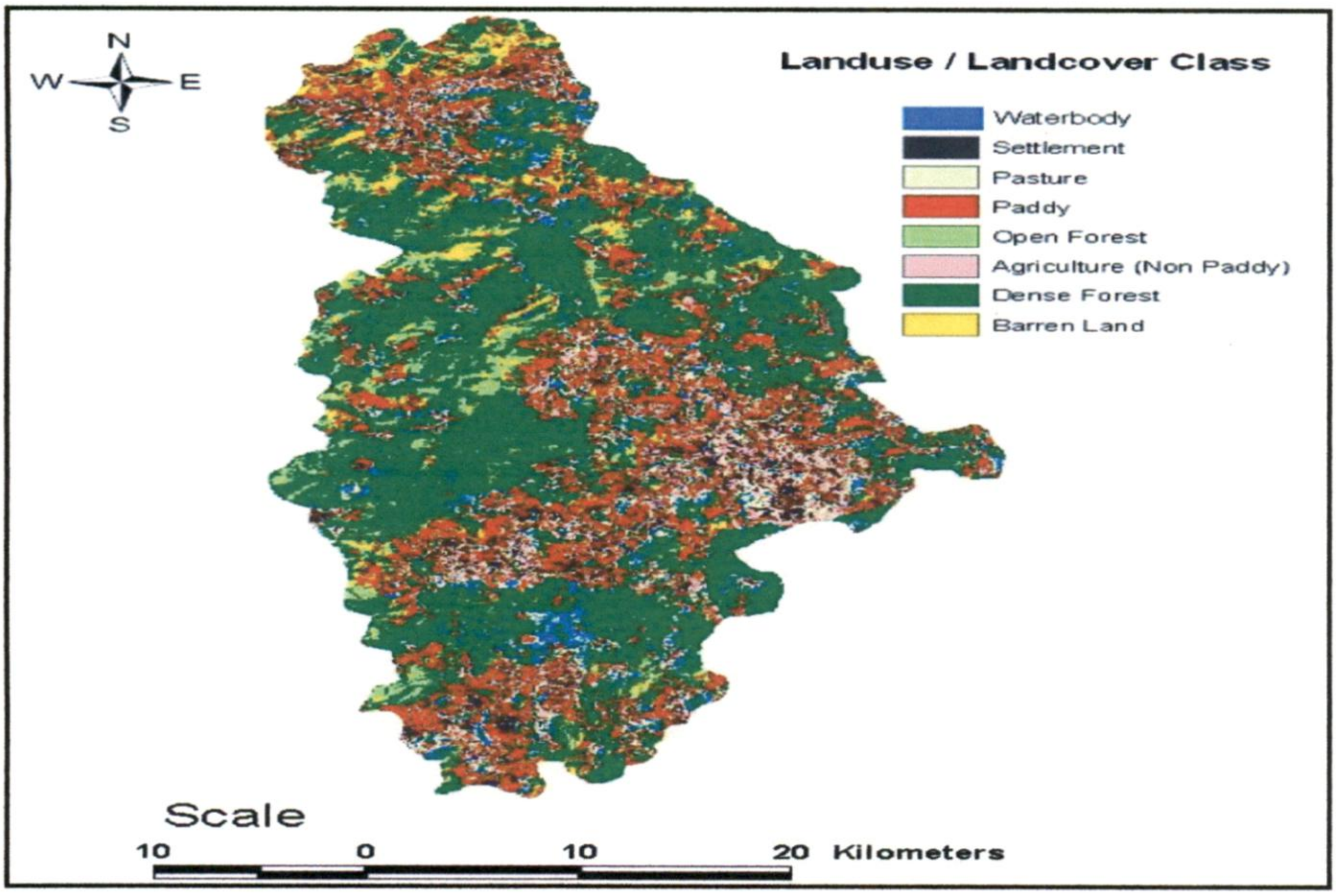


Fig. 4.5 Landuse map of Ret watershed

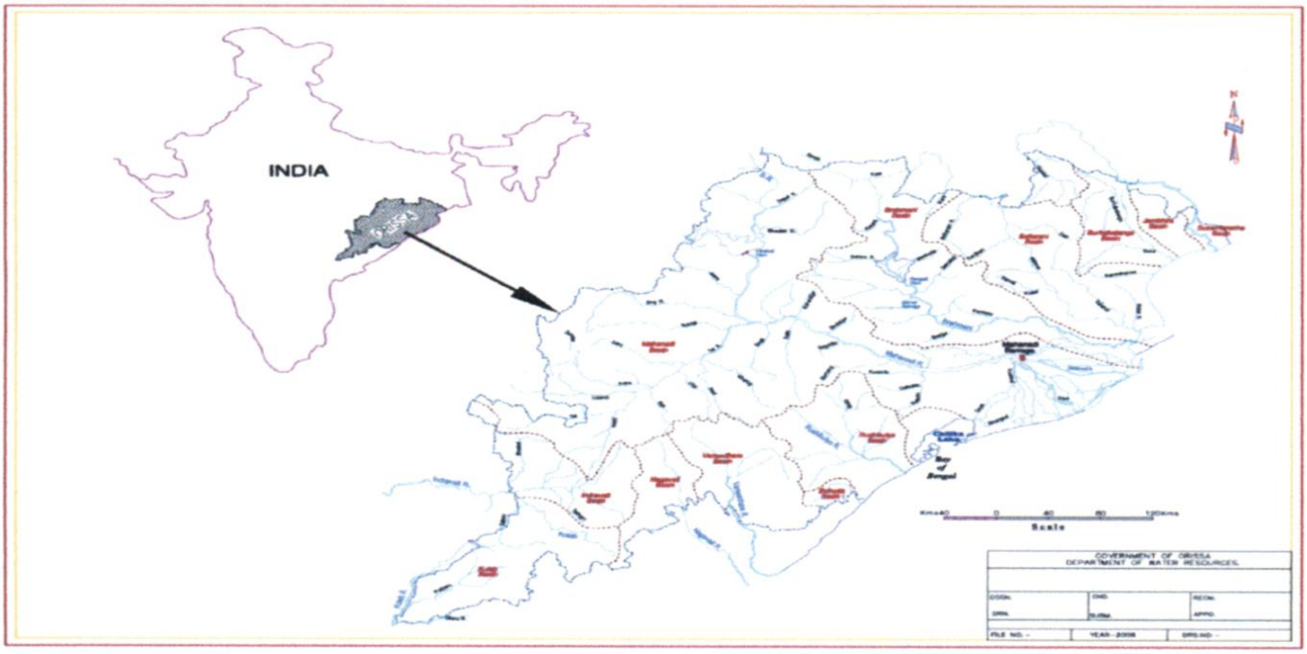


Fig.4.6 Drainage map of odisha

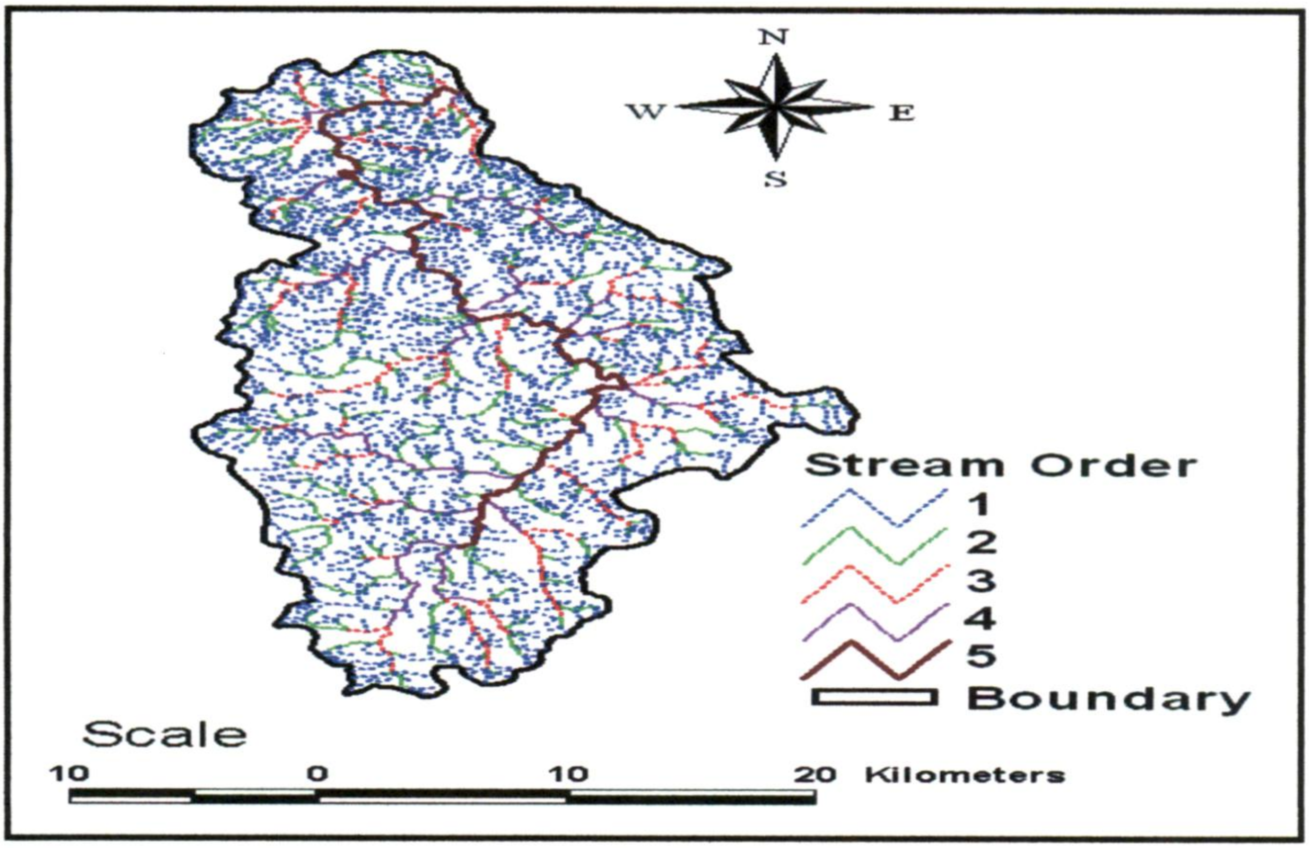


Fig. 4.7 Drianage map of Ret watershed

4.5 DESCRIPTION OF SIUL WATERSHED

The Siul River is a tributary river Ravi. The waters of the Ravi river drain into the Indian Ocean through the Indus River in Pakistan. The river rises in the Bara Bhangal area in district Kangra in Himachal Pradesh, India. The river drains a total catchment area of 14,442 sq. km (5,576 sq. mi) in India after flowing for a length of 720 km (450 mi). Flowing westward, it is hemmed by the Pir Panjal and Dhauladhar ranges, forming a triangular zone.

A major tributary that joins the Ravi River, just below Bharmour, the old capital of Chamba, is the Siul River from the northern direction. The valley formed by the river was also exploited for its rich timber trees. However, the valley has large terraces, which are very fertile and known as "the garden of Chamba". Crops grown here supply grains to the capital region and to Dalhousie town and its surrounding areas. One more major tributary that joins the Ravi River near Bissoli is the Sewa. This river was also exploited for its forest resources (controlled by the then Raja of Chamba) originating from the Jammu region. The valley is also formed by another major tributary that joins Siul River, the Baira-Nalla. Its sub-basin is in the Chamba district, located above Tissa. Baira drains the southern slopes of the Pir Panjal Range. The valley has an elevation variation between 5,321 m (17,457 ft) and 2,693 m (8,835 ft).

Tant Gari is another small tributary that raises from the subsidiary hill ranges of the Pir Panjal Range to the East of Bharmour. The valley formed by this stream is U-shaped with a river bed scattered with boulders and glacial morainic deposits. In the present study, the catchment area of 360 sq.km is taken into consideration as shown in Fig.4.10

4.5.1 Topography

Lying mostly in the range of Himalayas and touching the Shiwaliks on the southern fringe, the Ravi catchment area is rugged and covered with the spurs of the high ranges. The Dhauladhar range separating the basin of the Beas from that of the Ravi, the Pangi or Pir Panjal range dividing the watershed between the river Ravi and river Chenab and Zaskar range bifurcating the basins of the Chenab and the Indus are the three well defined snowy ranges, constituting the main topographical features of the area.The Dhauladhar range running in North-West direction forms the boundary between Mandi and Kullu Districts at the point where it gives off Bara

Bangahal branch to join the mid Himalayas. It makes a sudden bend west-ward and for the first time touches Chamba District on the southern border. From this point, it continues for about 50 kms. forming the boundary between Kangra and Chamba districts.

The Zaskar range is the direct continuation of the main Himalayan axis. It runs in north-west direction, dividing Ladakh from Lahaul- Spiti and then touches Chamba District, for a short distance along its northern border, separating Chamba and Lahaul-Spiti from Zaskar. The Pir Panjal range known as the Pangi range within the Chamba District, after separating Kullu from Lahaul-Spiti, enters Chamba district on the western border of the Bara Bangahal and traverses the district from South-East to North-West for more than 100 kms. On the North-Western border, where the Pangi range leaves the territory, it gives off a branch to the South-West called the Daganidhar which forms the boundary between Chamba and Bhadrawah of Jammu and Kashmir. At its western extremity, this branch is connected by a short ridge, in which the Padri and the Chatardhar passes. Topographically, the Dagnidhar and the Chatardhar are different sections of one continuous offshoot, forming with the Pangi range, the watershed between the Ravi and the Chander-Bhaga (Chenab).

4.6 DATA AVAILABILITY

4.6.1 Rainfall

There are in all thirteen non-recording type raingauge stations in the catchment area of the river Ravi. But the rainfall data given in the Table 5.4 are recorded at chamera dam site. The normal annual rainfall & annual rainfall has been recorded in mm at all this stations for the period 2000-2008.

Table 4.4 Rainfall data recorded at Chamera dam site.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2000	282	167	125	21	14	193	219	264	122	0	4	1
2001	28	13	41	146	34	215	295	280	68	1	0	0
2002	67	120	86	71	8	62	157	296	147	0	0	0
2003	45	123	180	0	0	59	114	357	86	0	45	0
2004	162	59	0	22	43	161	287	181	42	160	26	8
2005	236	473	204	20	33	37	473	265	111	3	0	0
2006	181	82	136	34	53	171	206	304	151	30	40	92
2007	0	147	327	1	39	134	165	142	70	0	1	49
2008	143	426	10	26	13	250	197	223	51	49	8	30

4.6.2 Temperature

There is no temperature record available at the proposed diversion site while some record is available at Chamba town, which is ±10 kms downstream of the proposed diversion site. The relative humidity is generally high in the monsoon season, being over 80%. In the post-monsoon and winter seasons, the humidity is less. The summer is generally the driest part of the year.

4.6.3 Evaporation

Monthly evaporation data which are observed at Chamera dam site are given in the table no-4.5 from the year 2000-2008.

Table 4.5 Evaporation data recorded at Chamera dam site.

YEAR	2000	2001	2002	2003	2004	2005	2006	2007	2008
PET(MM)	676	672	619	751	683	868	844	695	714

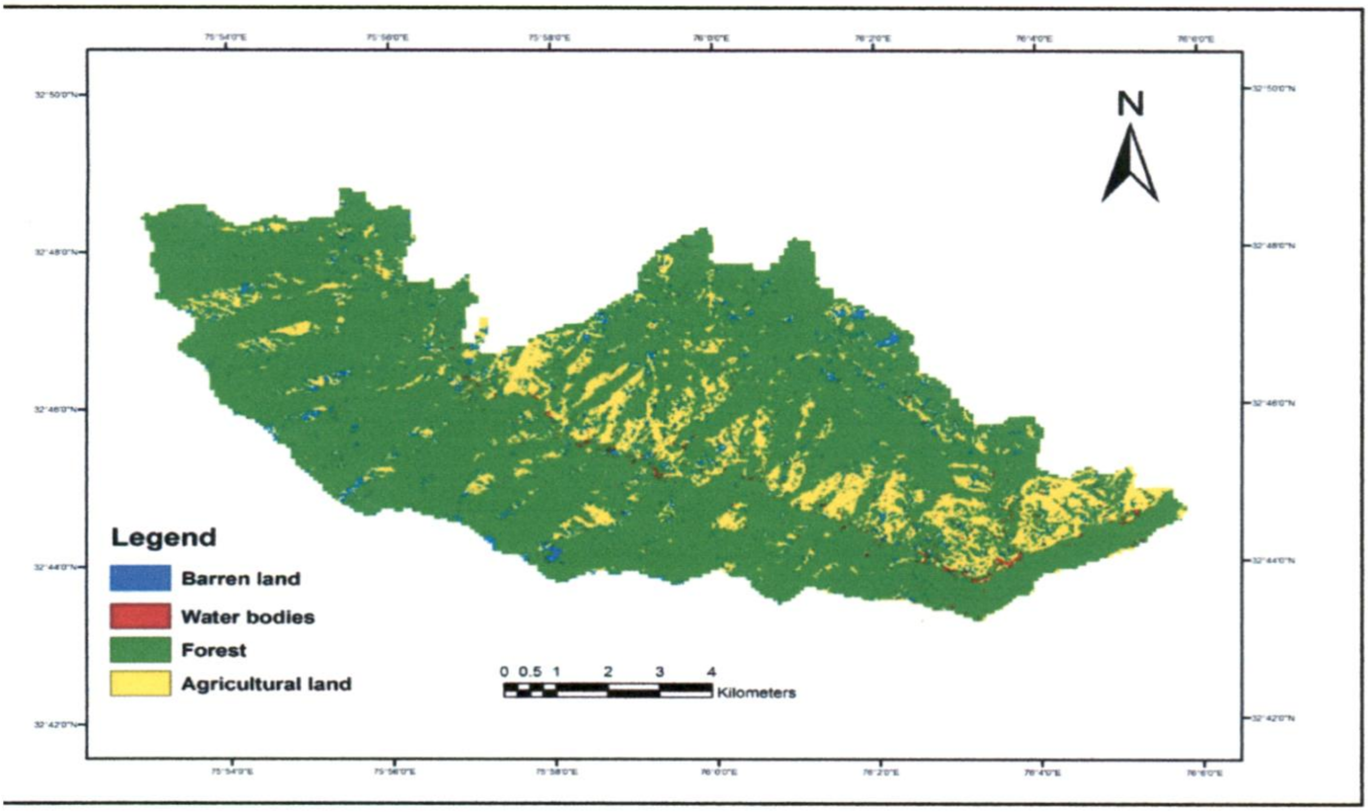


Fig. 4.8 landuse map of Siul watershed

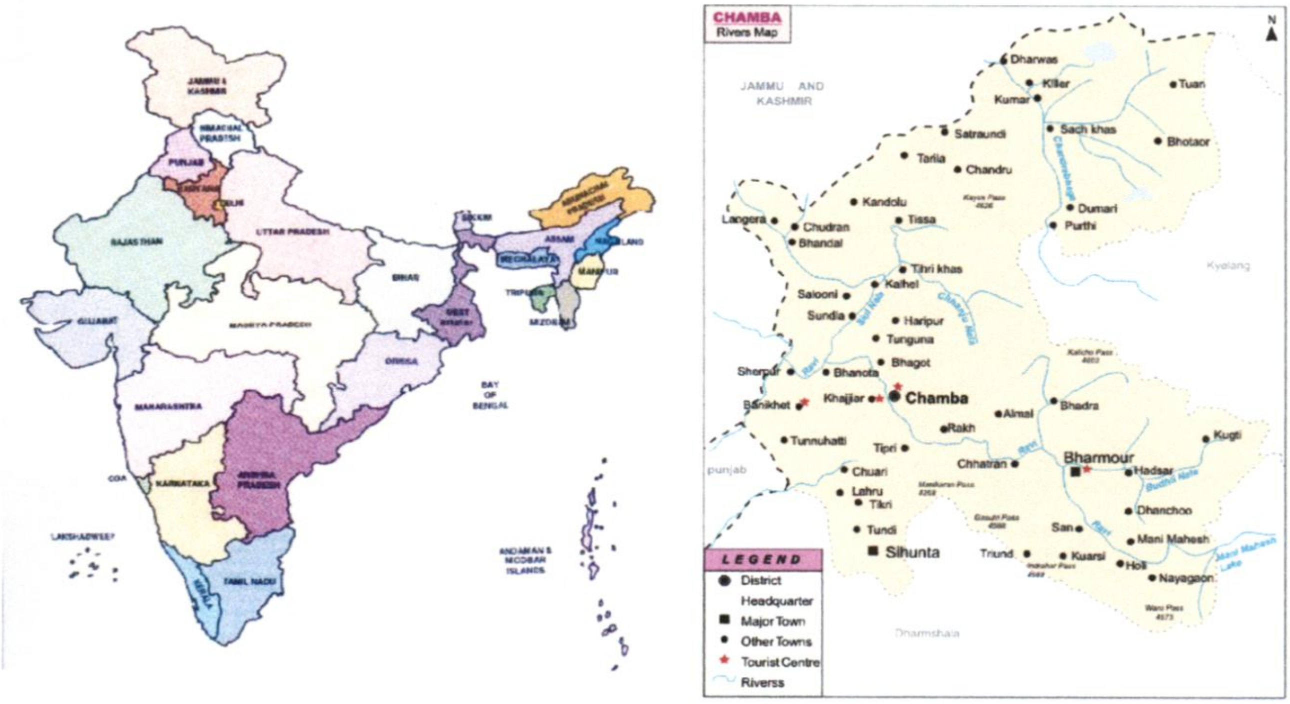


Fig 4.9 River map of district Chamba,H.P

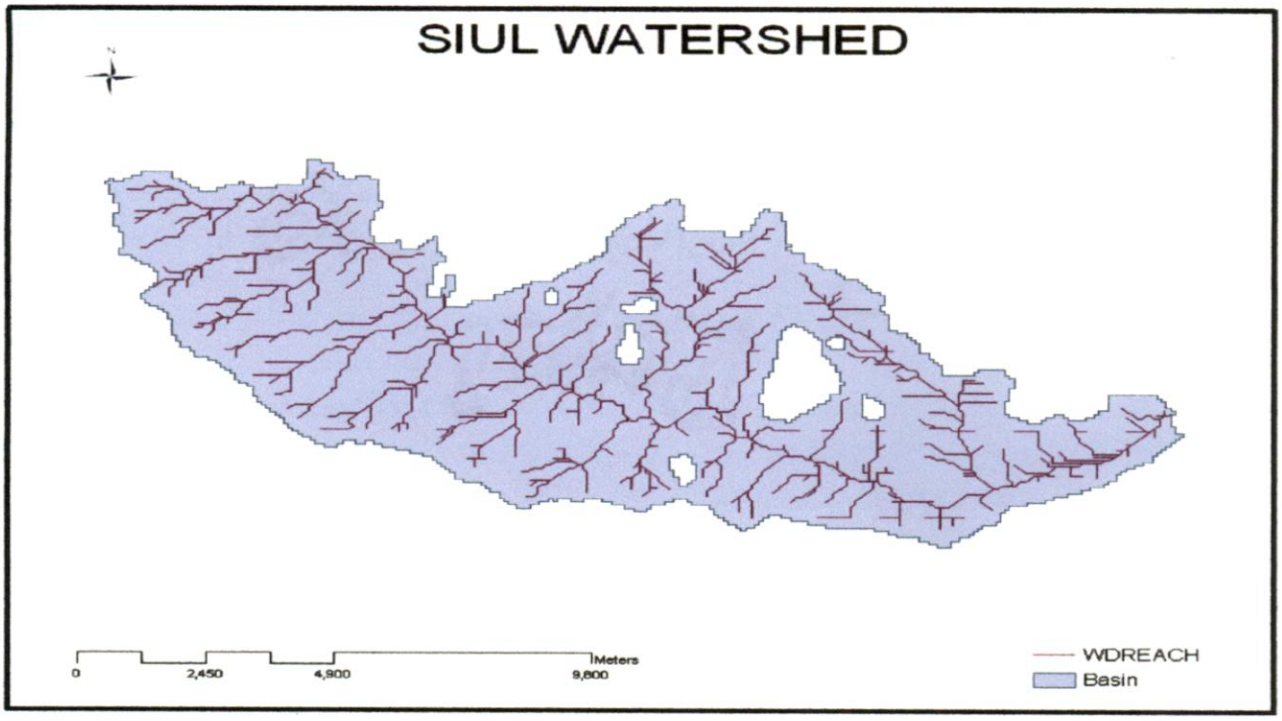


Fig 4.10 drainage map of Siul watershed

CHAPTER 5
RESULTS AND DISCUSSION

As described in previous chapters, an SCS-CN-based long term hydrologic simulation model was developed (Chapter 3) and it is tested on the data of Betwa, Ret, and Siul watersheds described in Chapter 4. The proposed model has three model parameters CN_0, K, ALPHA. The first parameter as such represents the initial value of curve number or, alternatively, the potential maximum retention, the second parameter represents the time of travel of flow, and the last one represents the lateral (base flow or intra-basin transfer) flow. Since the available data starts from Jan. 1993 for Betwa watershed, Jan. 2000 for Ret watershed and Jan. 2000 for Siul watershed, a value of CN_0 is required to be supplied as an initial guess for model run in calibration. Therefore, it is in order to begin the discussion with the computation of this value using remote sensing data. Here, it is worth indicating that the initial values of other parameters were fixed by trial and error considering the maximum efficiency criterion.

5.1 CATCHMENT CHARACTERISTICS

As discussed in Chapter 4, the study area of Betwa catchment has clayey, fine, fine loamy and loamy skeletal soil and these are well drained to moderately drained. These soils can be broadly classified as to generally fall in hydrologic soil groups B and C and the resulting curve numbers for different landuses in Betwa catchment are given in Table 5.1.

Table 5.1 Landuse pattern of Betwa catchment

Landuse	Area covered in (%)	Hydrologic Condition	Curve Numbers	
			B	C
Cultivated land	42.42	Good	75	82
Forest land	27.76	Fair	60	73
Shrubland	6.26	Good	59	75
Settlemnt	14.18	Poor	79	86
Waste land	7.52	Poor	86	91
Waterbody	1.86		99	99

On the basis of soil group and landuse map, Fig.4.1 (Chapter 4), curve number for the catchment was estimated and its areal weighted average was computed as 74, valid for normal antecedent moisture condition (AMC II), and its corresponding value of potential maximum retention under AMC I is 55 mm. This value was taken as initial estimate for model run in calibration.

As discussed in Chapter-4, the study area of Ret catchment has clay and sandy loam. The Ret watershed is occupied by red soil and its texture is fine loamy. The soils of the watershed are strongly to moderately acidic with low to medium organic matter content and has poor water holding capacity. These soils can be broadly classified as to generally fall in hydrologic soil groups C and D. The resulting curve numbers for different land uses in Ret catchment are given in Table 5.2.

Table 5.2. Landuse pattern of Ret catchment

Landuse	Area covered in (%)	Hydrologic Condition	Curve Numbers	
			C	D
Forest	51.74	Fair	73	79
Water	4.34	Good	99	99
Paddy	22.42	Poor	84	88
Agricultural land	7.12	Poor	86	89
Pasture land	5.92	Poor	86	89
Barren land	2.96	Poor	91	94
Settlement	5.50	Poor	86	89

On the basis of soil group and landuse map, Fig.4.5 (Chapter 4), a representative curve number for the catchment was estimated as an areal weighted average was computed as 83, valid for normal antecedent moisture condition (AMC II). The corresponding value of potential maximum retention under AMC I is 68 mm and it was taken as initial estimate for model run in calibration.

The study area of Siul catchment (Chapter 4) has loam, sandy loam and sandy clay loam shallow to medium deep depth, and these are well drained to excessively drained. These soils can be broadly classified as to generally fall in hydrologic soil groups A and B. The resulting curve numbers for different land uses in Ret catchment are given in Table 5.3.

Table 5.3 Landuse pattern of Siul catchment

Landuse	Area covered in (%)	Hydrologic Condition	Curve Numbers	
			A	B
Agricultural	15.61	Good	62	71
Forest	74.83	Fair	43	65
Barren land	7.82	Fair	49	69
Snow and Water bodies	1.74		99	99

On the basis of soil group and landuse map, Fig.4.8 (chapter 4), curve number for the catchment was estimated and its areal weighted average was computed as 72, valid for normal antecedent moisture condition (AMC II), and its corresponding value of potential maximum retention under AMC I is 42 mm and it was taken as initial estimate for model run in calibration.

5.2 MODEL CALIBRATION AND VALIDATION

For model calibration and validation, the available 10-year dataset of Betwa catchment, 10-year dataset of Ret catchment, and 9-year dataset of Siul catchment (Chapter 4) are split into two parts. For calibration, five years (1993-1997) data of Betwa catchment, five years (2000-2004) data of Ret catchment, and five years (2000-2004) data of Siul catchment have been considered. The simulated hydrographs depicting rainfall, runoff computed, and observed runoff are shown in Fig. II-1 to Fig. II-17 for calibration and validation results in Appendix-II. The derived parameters along with their initial estimates are given in Table 5.4. Using the derived parameters, the runoff was computed in years other than those used in calibration. In these figures, the results in calibration or validation begin from January 01 and the year ends on December 31 in each year. In this chapter Figs. 5.1 to 5.12 show best and worst fitting years of observed and computed runoff each from calibration and validation years of the three catchments.

Table 5.4a Calibration and validation results for Betwa watershed

DURATION (year)	PARAMETERS(CALIBRATION)			
	CN$_0$ (mm)	K (days)	ALPHA	EFFICIENCY (%)
	INITIAL ESTIMATE			
	55	1.0	1.0	
	FINAL ESTIMATE			
1993	75.10	1.44	-0.28	77.98
1993-1994	70.89	1.18	-0.10	73.73
1993-1995	72.46	1.24	-0.17	69.60
1993-1996	70.45	0.74	-0.12	62.91
1993-1997	71.64	1.08	-0.15	60.24
1993-1998	72.97	1.20	-0.20	57.00
1993-1999	73.65	1.05	-0.25	61.11
1993-2000	73.56	1.08	-0.24	59.72
1993-2001	73.82	1.07	-0.26	59.68
1993-2002	73.84	1.08	-0.26	60.08
VALIDATION				
1998-2002	68.08	1.11	-0.32	63.54
1999-2002	81.67	0.99	-0.39	66.48
2000-2002	99.99	1.46	-0.37	67.13
2001-2002	77.28	1.20	-0.39	64.65
2002	99.99	1.86	-0.32	78.81

Table 5.4b Calibration and validation results for Ret watershed

DURATION (year)	PARAMETERS(CALIBRATION)			
	CN$_0$ (mm)	K (days)	ALPHA	EFFICIENCY (%)
	INITIAL ESTIMATE			
	68	1.0	1.0	
	FINAL ESTIMATE			
2000	99.99	15.25	-0.48	78.14
2000-2001	67.70	4.67	-0.28	48.21
2000-2002	87.18	5.01	-0.44	44.93

2000-2003	99.99	4.32	-0.65	40.64
2000-2004	99.99	3.72	-0.65	47.40
2000-2005	99.99	3.03	-0.63	43.37
2000-2006	99.99	3.04	-0.62	46.96
2000-2007	99.99	3.46	-0.63	45.63
2000-2008	99.99	3.21	-0.62	51.77
2000-2009	99.99	3.44	-0.63	52.10
VALIDATION				
2005-2009	99.99	3.14	-0.62	62.28
2006-2009	94.83	3.98	-0.65	65.29
2007-2009	43.13	3.39	-0.63	69.16
2008-2009	64.51	3.01	-0.65	69.94
2009	82.81	4.54	-0.72	72.18

Table 5.4c Calibration and validation results for Siul watershed

DURATION (year)	**PARAMETERS(CALIBRATION)**			
	CN_0 (mm)	**K** (days)	**ALPHA**	**EFFICIENCY (%)**
	INITIAL ESTIMATE			
	42	**1.0**	**1.0**	
	FINAL ESTIMATE			
2000	99.99	17.76	-0.81	74.41
2000-2001	99.99	2.31	-0.85	45.59
2000-2002	99.99	2.95	-0.88	37.95
2000-2003	99.99	2.99	-0.87	38.64
2000-2004	99.99	2.42	-0.87	39.33
2000-2005	99.99	3.32	-0.86	33.86
2000-2006	99.99	3.99	-0.86	33.41
2000-2007	99.99	5.43	-0.85	34.34
2000-2008	99.99	5.65	-0.86	35.98
VALIDATION				
2005-2008	99.99	17.87	-0.832	44.04
2006-2008	89.77	16.15	-0.854	59.96
2007-2008	99.99	15.51	-0.861	61.09
2008	99.99	15.07	-0.866	62.18

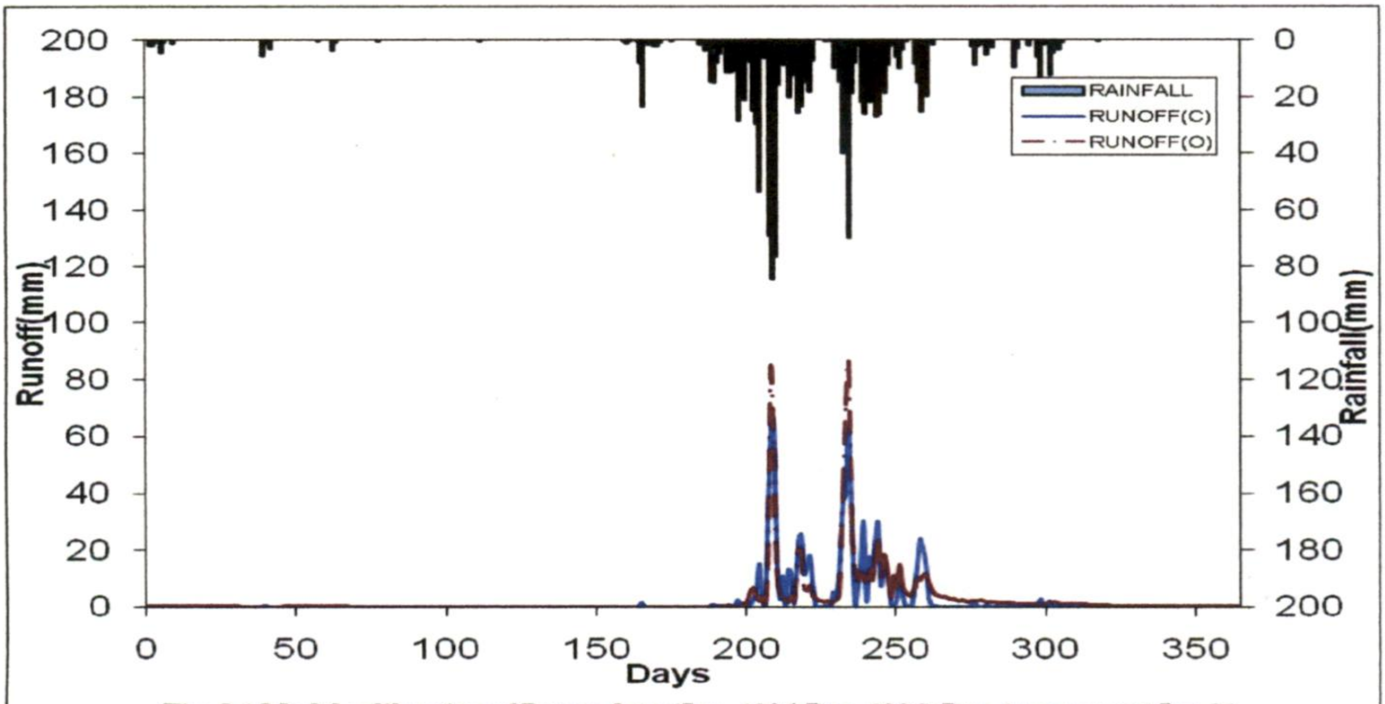


Fig. 5.1 Model calibration of Betwa data (Jan. 1996-Dec. 1996).Day 1 represents Jan.01

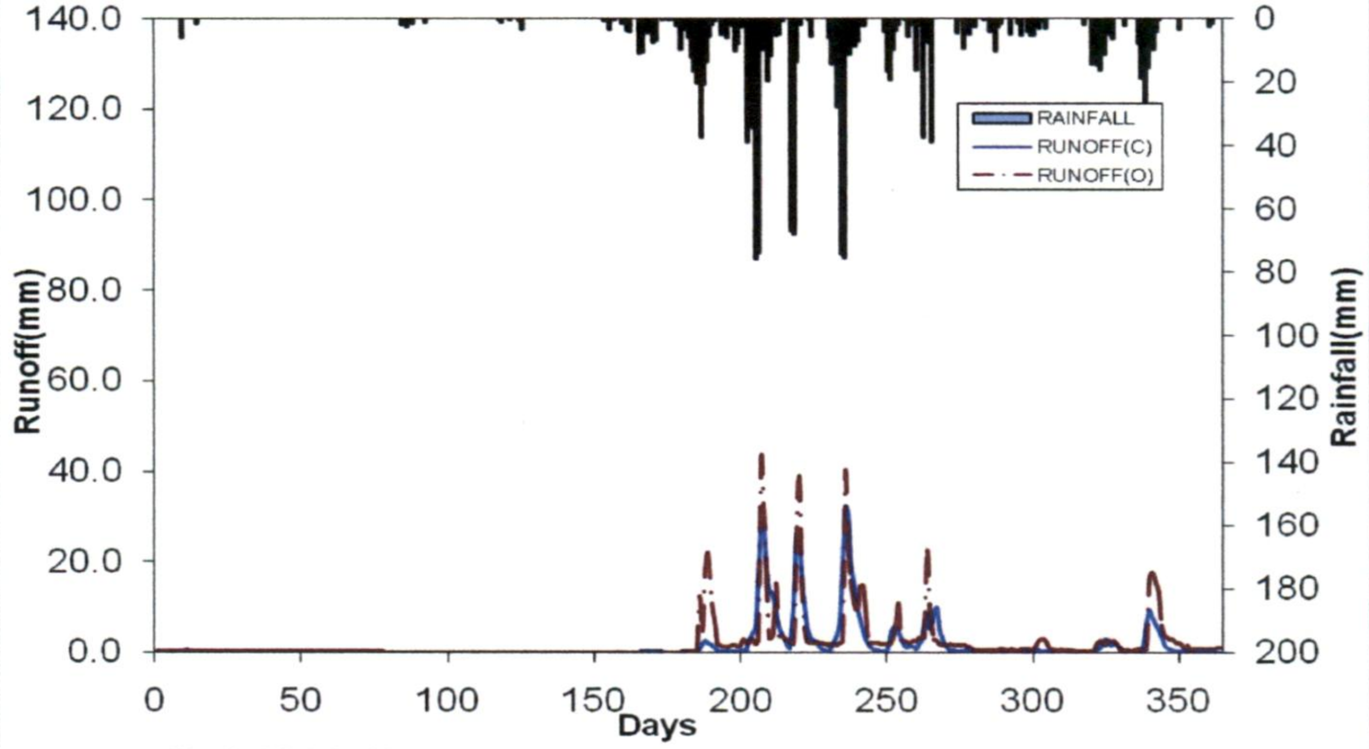


Fig. 5.2 Model calibration for Betwa data(Jan. 1997-Dec. 1997).Day 1 represents Jan.01

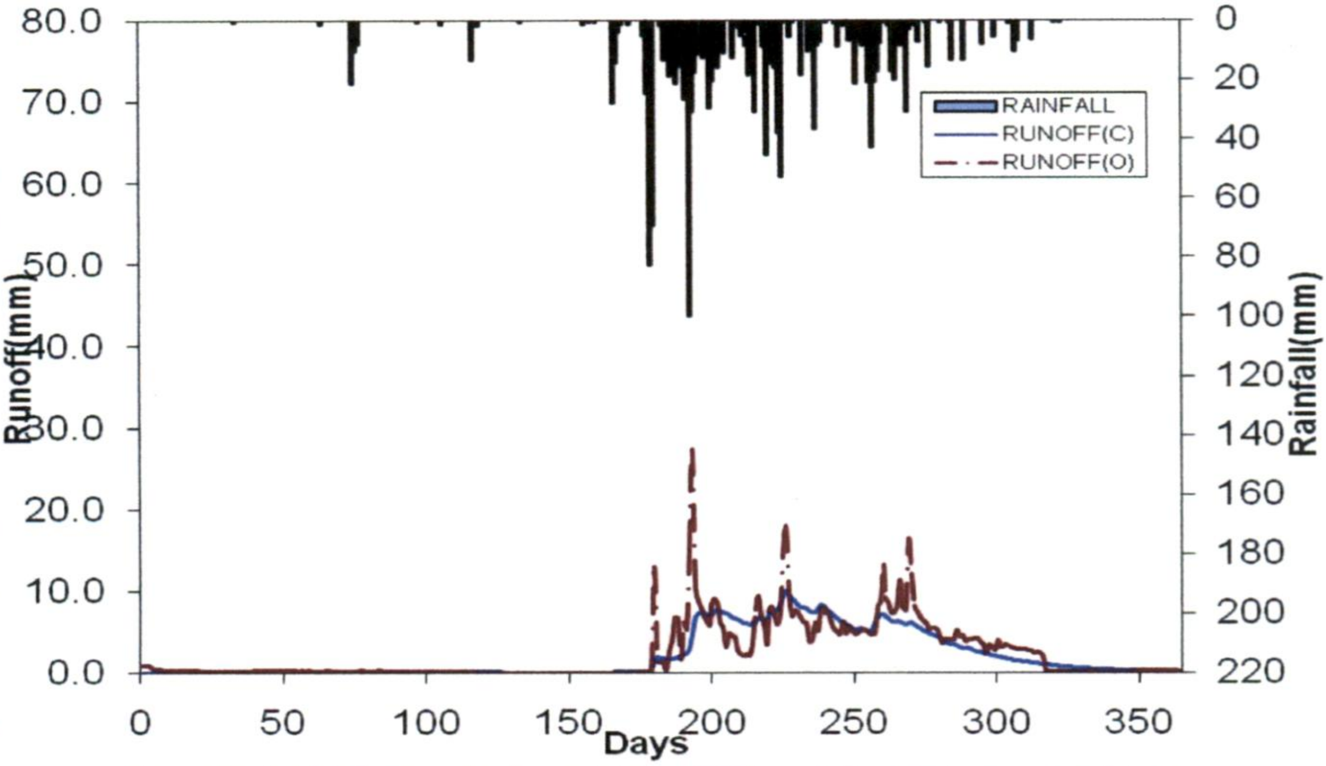


Fig. 5.3 Model Validation of Betwa data(Jan.1998-Dec. 1998).Day 1 represents Jan.01

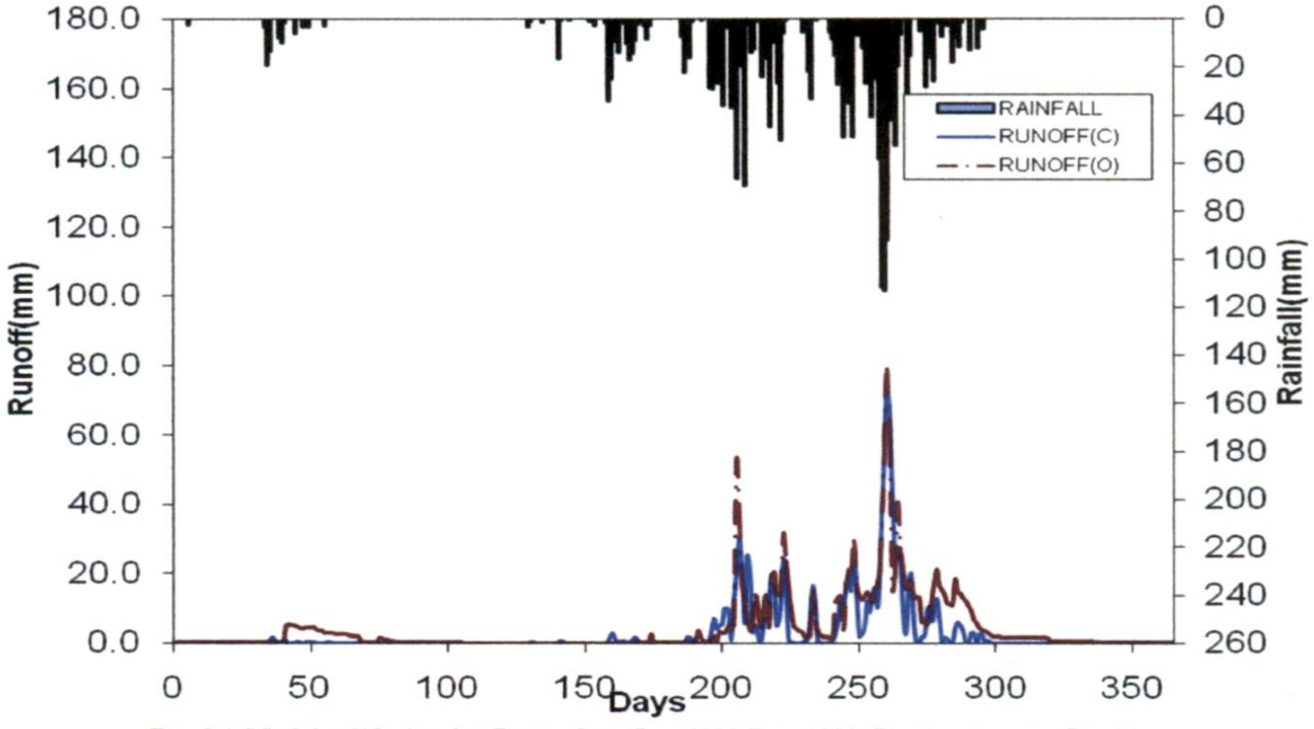


Fig. 5.4 Model validation for Betwa data(Jan. 1999-Dec. 1999).Day1 represents Jan.01

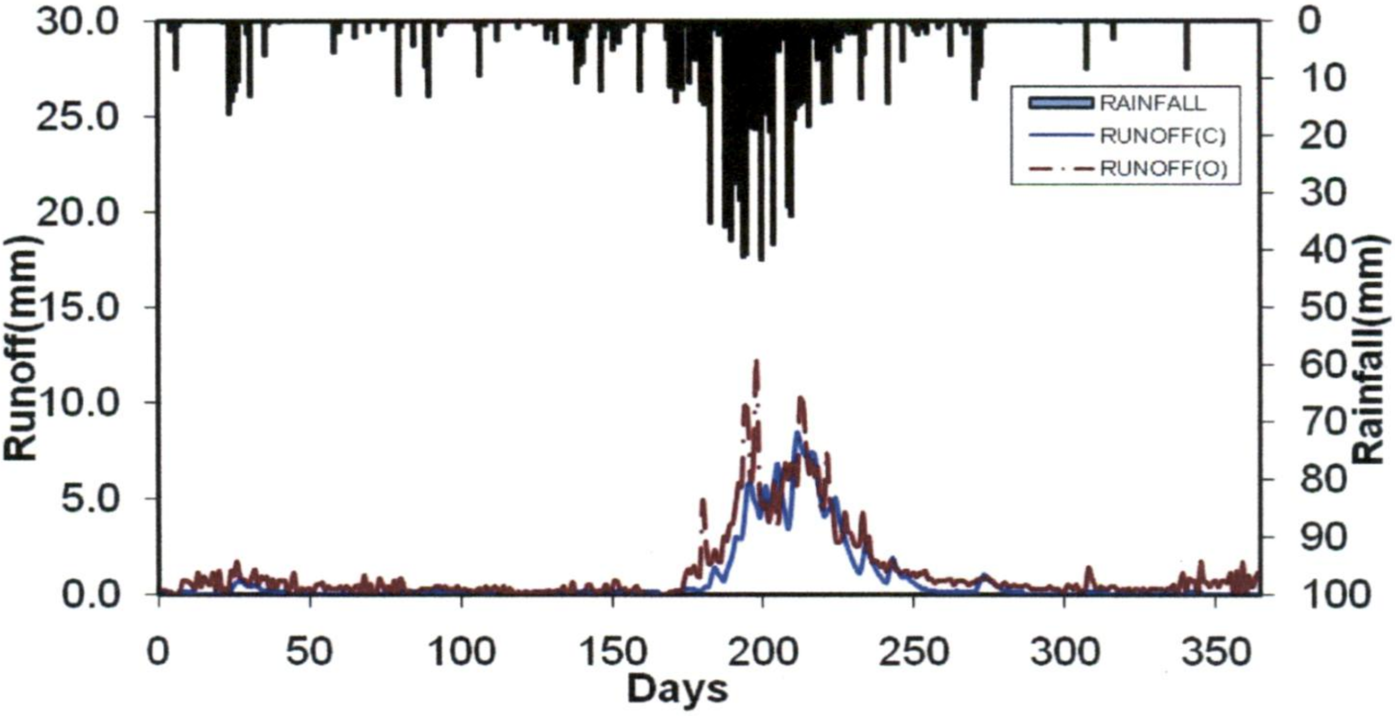


Fig.5.5 Model calibration for Ret data(Jan. 2001-Dec. 2001).Day1 represents Jan.01

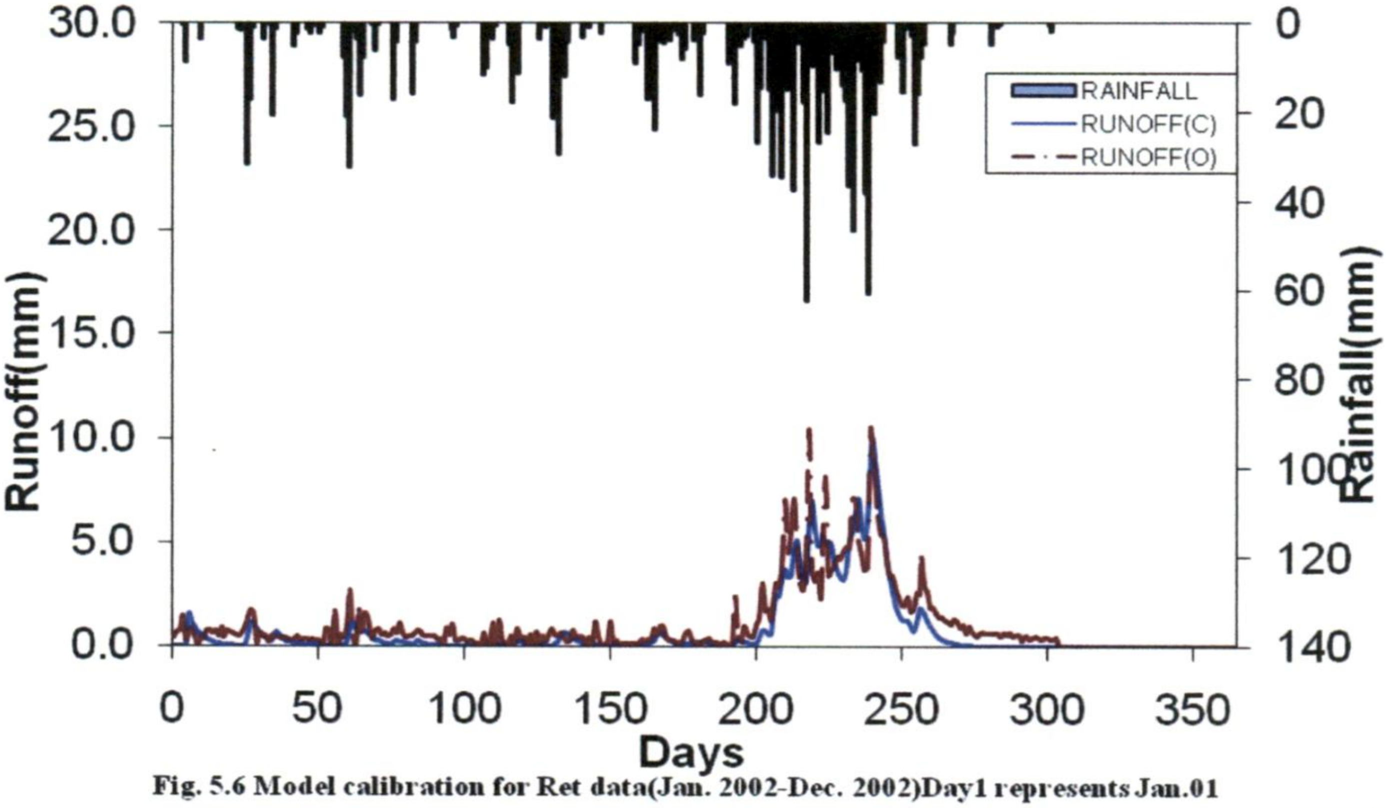


Fig. 5.6 Model calibration for Ret data(Jan. 2002-Dec. 2002)Day1 represents Jan.01

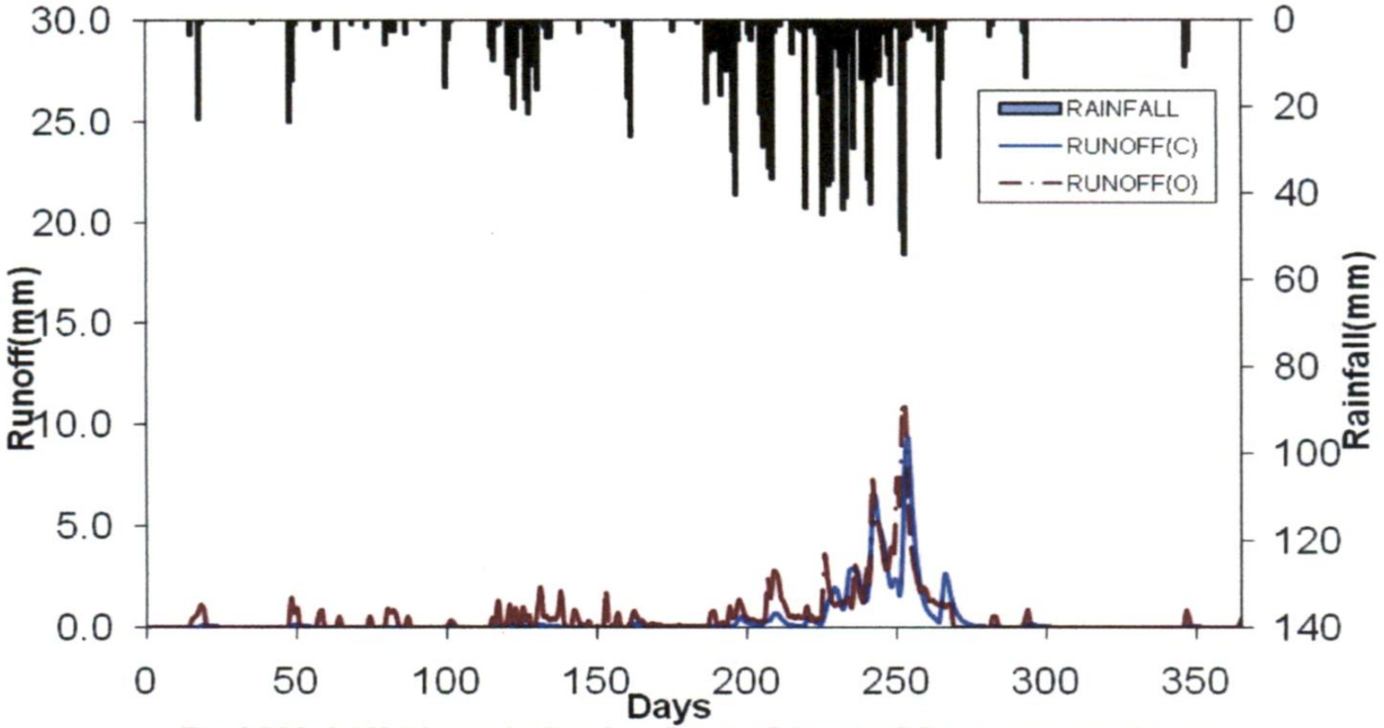


Fig. 5.7 Model Validation for Ret data(Jan. 2007-Dec. 2007)Day1 represents Jan.01

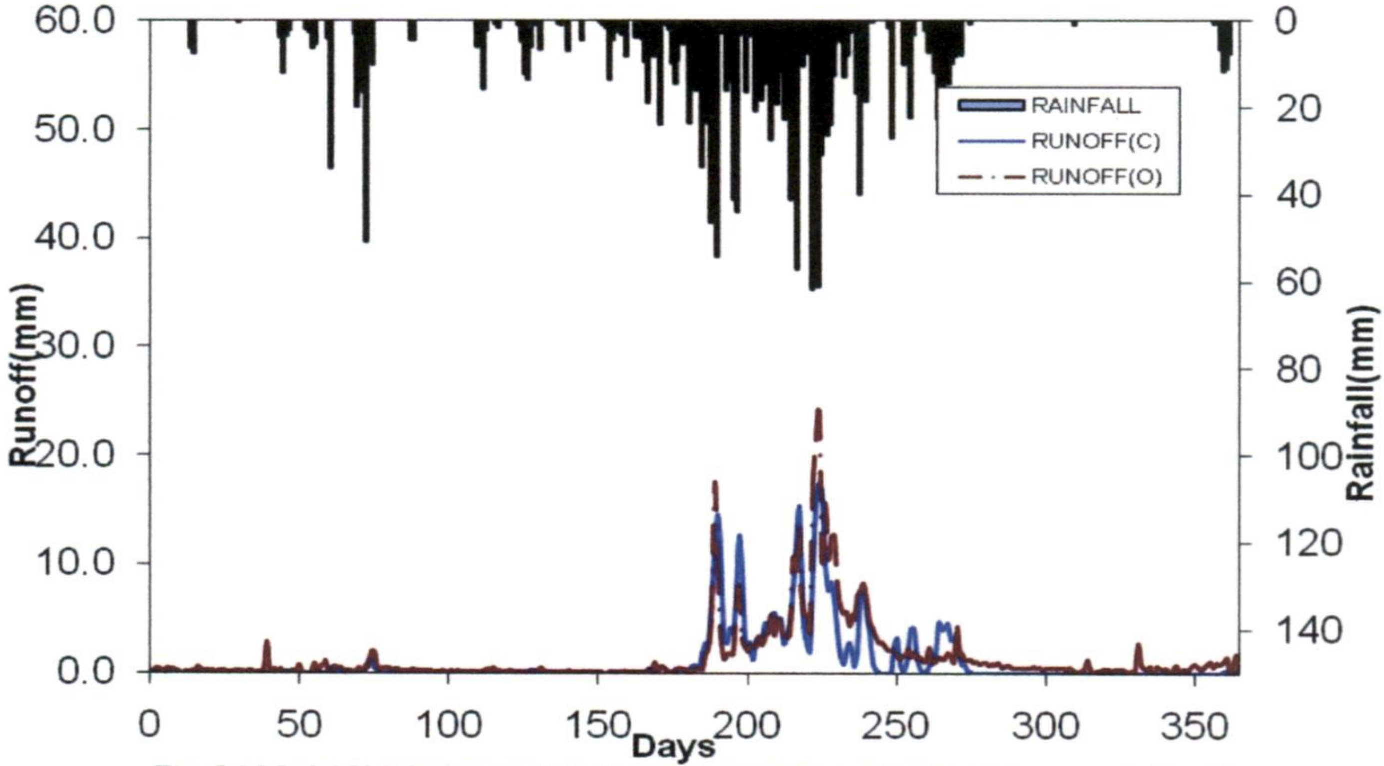


Fig. 5.8 Model Validation for Ret data(Jan. 2008-Dec. 2008)Day1 represents Jan.01

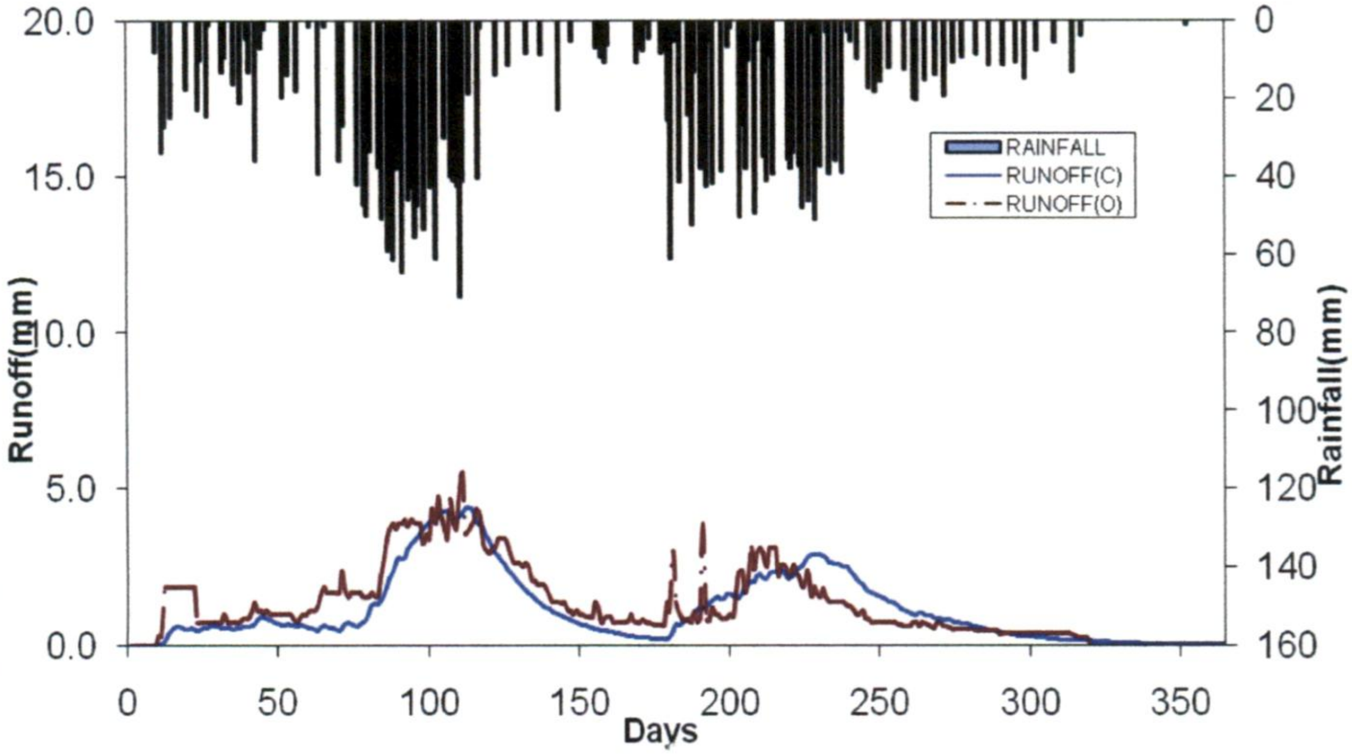


Fig. 5.9 Model calibration for Suil data(Jan. 2000-Dec. 2000)Day1 represents Jan.01

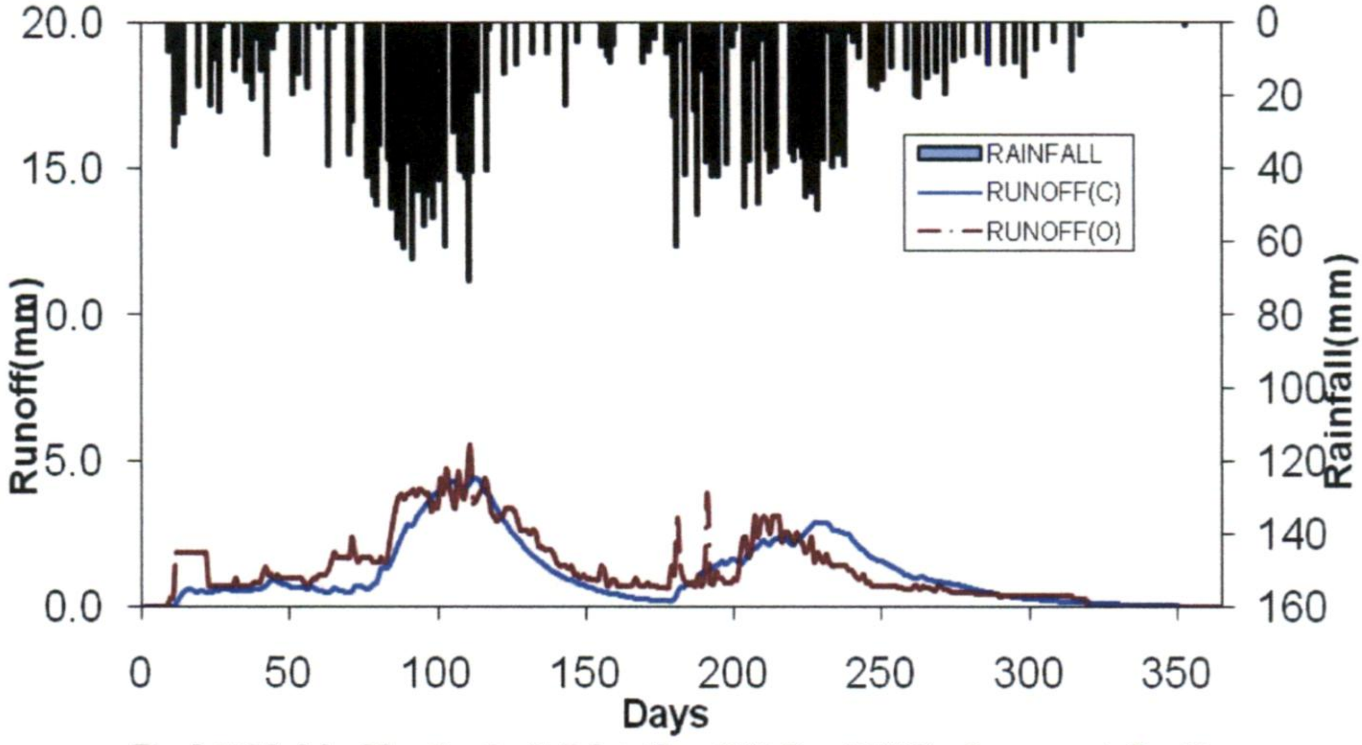


Fig. 5.10 Model calibration for Suil data(Jan. 2000-Dec. 2000)Day1 represents Jan.01

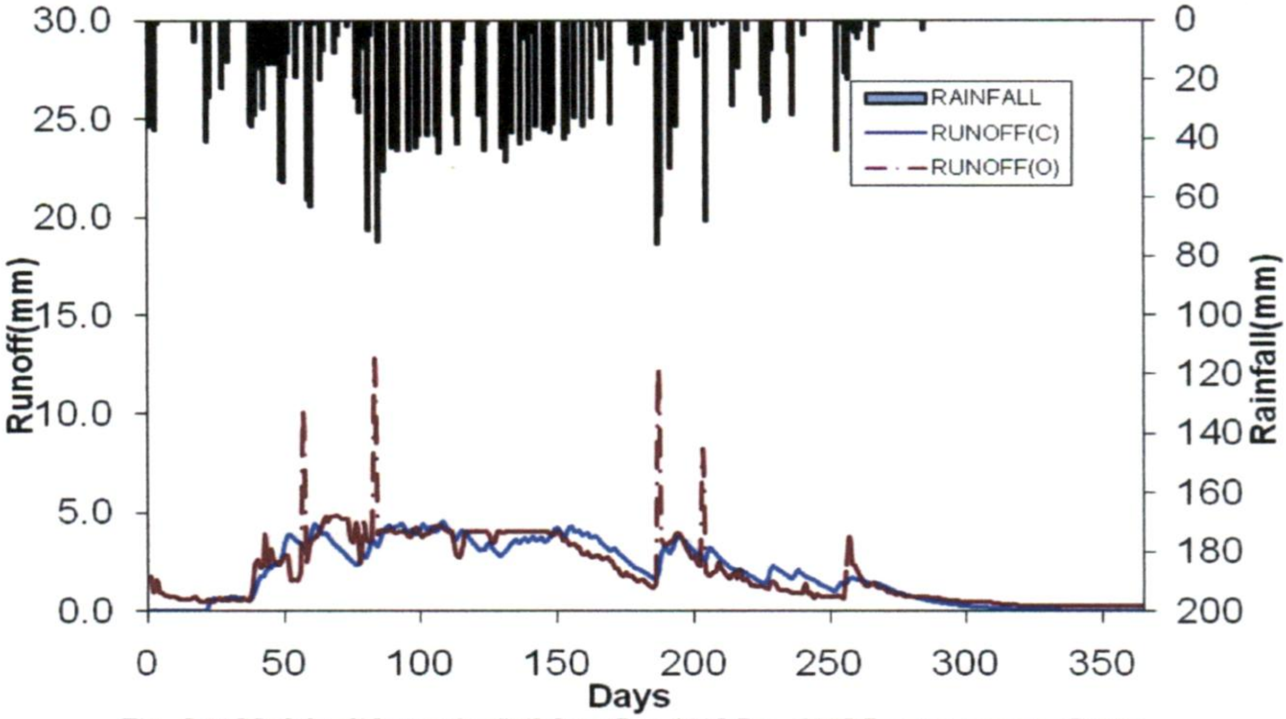


Fig. 5.11 Model validation for Suil data(Jan. 2005-Dec. 2005)Day1 represents Jan.01

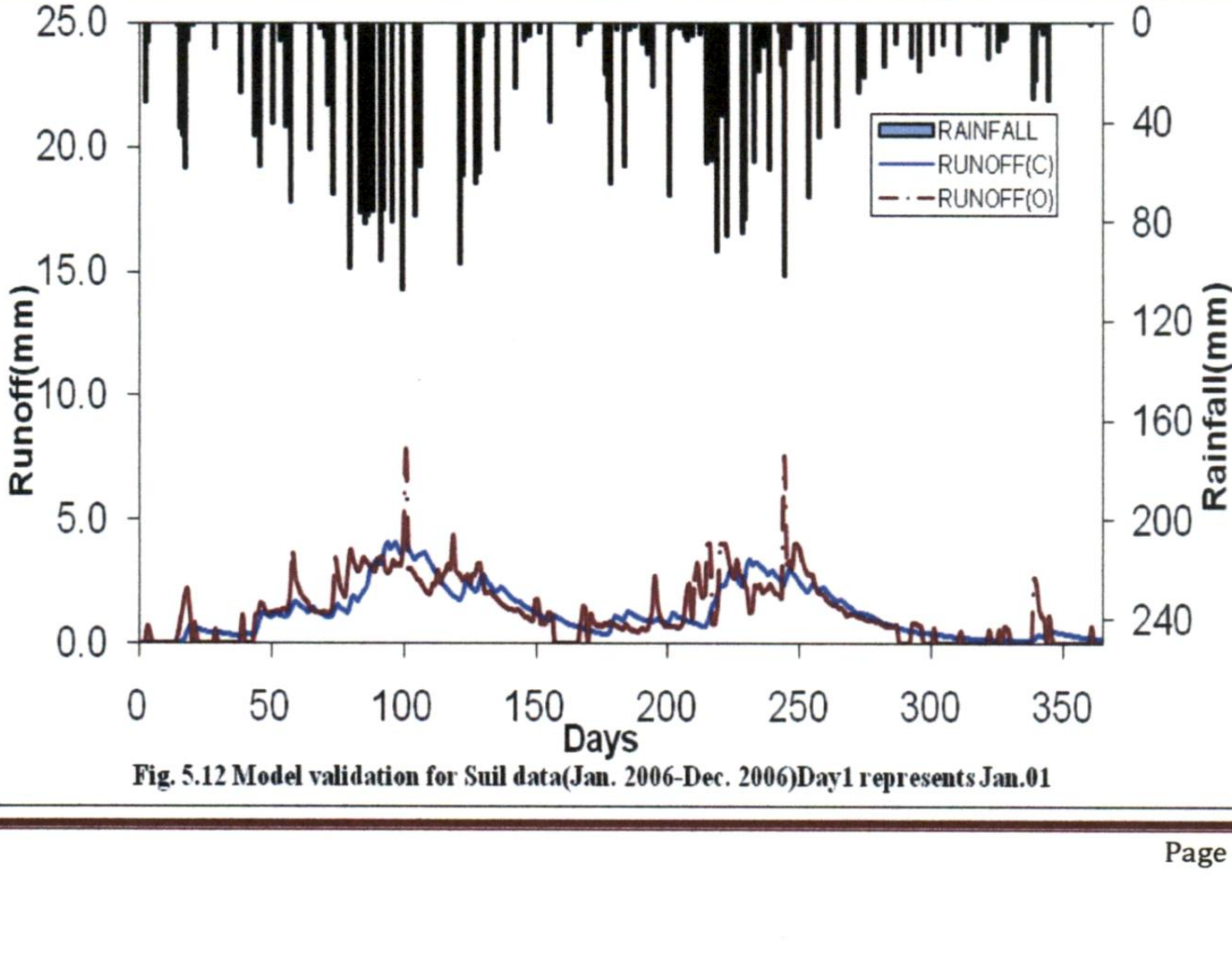


Fig. 5.12 Model validation for Suil data(Jan. 2006-Dec. 2006)Day1 represents Jan.01

In calibration, the estimated values of the three model parameters (CN_0, K, ALPHA) along with their initial values and model efficiencies are given in Tables 5.4a-c. It is apparent from this table that the resulting efficiencies in simulation vary from 57.01 to 77.98% in Betwa data, 40.64 to 78.14% in Ret data, 33.61 to 74.41% in Siul data as the number of years of data varies from 1 - 10 years. Though the efficiencies generally show a decreasing trend with the increase in data length, these are indicative of satisfactory to poor performance of the proposed model in simulation. Then, taking the parameter values corresponding to five years of data in model calibration (Table 5.4), the model was validated on the remaining five years data in the three catchments. The resulting efficiencies of five years of data as 63.54% in Betwa catchment, 62.28% in Ret catchment and 44.04 % in Siul catchment indicate a satisfactory model performance in Betwa and Ret catchments and poor performance in Siul catchment.

As also seen from Table 5.4, with the increasing length of data, the values of parameters also vary. For example, the parameter CN_0 varies in the range from 70.45 to 75.10 mm in Betwa catchment, 67.70 to 99.99 in Ret catchment and 99.99 in Siul catchment. K varies from 0.74 to 1.45 day in Betwa catchment, 3.21 to15.25 days in Ret catchment, 2.311 to 17.76 day in Siul catchment. Parameter ALPHA varies from-0.10 to -0.28 in Betwa , -0.28 to -0.65 in Ret and -0.81 to -0.88 in Siul watershed. Thus, the parameters CN_o and K show quite a wide range of variation while parameter ALPHA shows small range of variation. The negative value of parameter ALPHA shows a case of lateral outflow from the catchment. This water doesn't appear at the outlet, rather it is abstracted before it reaches the outlet. Here, an attempt is made to correlate these parameter values with the general features of the watersheds, as follows.

The parameter CN_0 , which represents the potential maximum retention or the space available for moisture storage. This value is comparable with that given by above estimation corresponding to CN value of 55, 68, and 42(AMC III) for Betwa, Ret, Siul catchment, respectively. Parameter K represents the lag time between the occurrence of rainfall and runoff. In other words, it represents the time lag between the rainfall and runoff. The parameter ALPHA represents the coefficient which when multiplied with the inflow yields baseflow and/or intra basin or any other form of lateral inflow.

After calibration, the model is evaluated for its performance on annual data of all the three watersheds, as shown in Table 5.5a-c. The table shows that the annual performance varies from 66.19% to 82.76 % for Betwa, 64.90% to 78.99% for Ret, and 60.17% to 74.40% for Siul catchments, indicating a satisfactory model performance on all watersheds. As also seen from Table 5.5, the parameter CN_0 varies in the range from 72.96 to 99.99 mm in Betwa catchment, 41.74 to 99.99 in Ret catchment and 87.58 to 99.99 in Siul catchment. K varies from 0.08 to 1.53 days in Betwa catchment, 0.88 to 3.83 days in Ret catchment, and 0.04 to 2.88 days in Siul catchment. Parameter ALPHA varies from-0.01 to -0.63 in Betwa, -0.41 to -0.74 in Ret and -0.68 to -0.91 in Siul catchment. Thus, parameters CN_0 shows quite a wide range of variation while parameter K and ALPHA varies in a small range.

In all the years, the computed peak runoffs were compared with the observed peak runoff and these are given in Table 5.6 for performance evaluation, the relative error (%) was computed as:

$$\text{Relative error (\%)} = (R_o - R_c)*100/R_o \qquad (5.1)$$

where R_o and R_c correspond to the observed and computed annual runoff values, respectively. Thus the relative error may take any value ranging from 0 to ∞ depending on the value of R_o. As R_o approaches zero, the relative error approaches infinity. It is apparent from Table 5.6 the relative error ranges from 22.72% to 33.7% in Betwa data, 3.72% to 45.6% in Ret data, and 8.7% to 50.3% in Siul data. Beside, in some years a generally satisfactory match is seen between the observed and simulated peak runoff rates. Thus, it can be inferred that, in calibration, the proposed model works well on the data of Betwa , Ret and Siul catchments. Here, it is noted the SCS-CN method was originally proposed for agricultural watersheds with flat to mild slopes (Neitsch et al., 2002), and not for hilly areas. However, in application of SWAT model, Neitsch et al. (2002) provided a slope reduction factor for CN conversion in such situations.

Table 5.5a Annual Performance of Model on Betwa data

YEAR	EFFICIENCY(%)	CN_0	K	ALPHA
1993	77.98	75.10	1.44	-0.28
1994	77.11	72.96	0.94	-0.01
1995	71.25	98.46	1.53	-0.63
1996	82.76	87.57	0.08	-0.04
1997	69.00	99.99	1.80	-0.32
1998	66.19	99.99	2.00	-0.24
1999	73.50	85.74	0.50	-0.34
2000	73.06	99.99	1.36	-0.35
2001	72.47	99.99	1.11	-0.52
2002	72.72	81.51	0.18	-0.41

Table 5.5b Annual Performance of Model on Ret data

YEAR	EFFICIENCY(%)	CN_0	K	ALPHA
2000	78.18	99.99	1.13	-0.48
2001	65.10	99.26	2.44	-0.41
2002	78.99	94.71	3.83	-0.60
2003	70.54	99.99	1.05	-0.74
2004	69.83	65.89	2.76	-0.67
2005	71.18	99.99	2.22	-0.44
2006	72.15	73.34	2.92	-0.57
2007	64.90	41.74	2.57	-0.60
2008	75.02	62.06	0.88	-0.64
2009	72.07	78.16	1.01	-0.72

Table 5.5c Annual Performance of Model on Siul data

YEAR	EFFICIENCY(%)	CN_0	K	ALPHA
2000	74.40	99.99	1.80	-0.81
2001	60.94	99.99	0.04	-0.87
2002	63.48	99.99	1.17	-0.91
2003	62.07	99.99	2.88	-0.78
2004	61.53	92.05	0.11	-0.89
2005	62.56	99.99	1.02	-0.68
2006	60.17	87.58	1.55	-0.83
2007	60.43	99.99	1.40	-0.85
2008	62.18	99.99	1.32	-0.86

Table 5.6a Relative error computation in Betwa application

Year	Observed Peak Runoff(mm/day)	Computed peak Runoff(mm/day)	Relative Error (%)
1993	48.0	31.8	33.7
1994	75.5	53.8	28.6
1995	17.6	13.5	22.9
1996	85.2	65.8	22.7
1997	42.4	32.2	24.0
1998	17.4	9.8	43.8
1999	79.0	70.7	10.5
2000	43.3	33.3	23.0
2001	33.3	23.4	29.7
2002	35.2	26.8	24.0

Table 5.6b Relative error computation in Ret application

Year	Observed Peak Runoff(mm/day)	Computed peak Runoff(mm/day)	Relative Error (%)
2000	7.9	6.2	21.9
2001	12.2	8.4	31.1
2002	10.5	10.1	3.7
2003	12.9	7.0	45.6
2004	15.2	10.5	30.7
2005	23.7	17.8	24.9
2006	11.8	9.5	19.9
2007	10.8	9.3	13.7
2008	24.2	17.4	28.1
2009	11.1	8.1	26.9

Table 5.6c Relative error computation in Siul application

Year	Observed Peak Runoff(mm/day)	Computed peak Runoff(mm/day)	Relative Error (%)
2000	5.5	4.4	20.8
2001	15.4	9.4	39.0
2002	8.5	4.2	50.3
2003	13.9	9.4	32.2
2004	11.9	10.9	8.7
2005	7.8	4.5	42.1
2006	7.8	4.1	47.8
2007	5.6	4.2	25.4
2008	7.6	4.1	46.0

5.3 VOLUMETRIC STATISTIC

To generally show the water balance of the considered watershed, a yearly volumetric analysis for all the components, such as precipitation, infiltration, evapotranspiration, and total surface runoff, was carried out and its statistics is given in Tables 5.7a-c for Betwa, Ret and Siul catchments respectively. To compare the model computed yearly runoff values with the observed ones, the relative errors (Eq. 5.1) were computed and these are shown in Tables 5.7a-c. These errors range from -7.7 to 37.9% for Betwa, from 22 to 39% for Ret, and from 3.7 to 34.2% for Siul catchments. Here, '+' values indicate that the computed values are lower (or under-estimated) than the observed ones, and vice versa for' -' values. It is also apparent from the table that the low runoff producing year yields relatively high relative errors, largely because of the SCS-CN applicability to high runoff magnitudes.

The above described model performance can be further appreciated in view of (i) the limited number of model parameters (only three), (ii) simplicity, and (iii) no constraints imposed for matching the observed annual runoff volumes. In addition, there is little information available on base flows and the lateral ground water interaction across the basin boundaries.

Table 5.7a Annual volumetric statistic for Betwa watershed

Year	Rainfall (mm)	Infiltration (mm)	Evapotranspiration (mm)	Runoff Computed(mm)	Runoff Observed(mm)	Relative Error(%)
1993	1354	412	455	605	662	8.6
1994	1281	326	381	870	1126	22.8
1995	903	334	538	180	291	37.9
1996	1297	373	460	832	902	7.7
1997	1383	427	458	583	752	22.4
1998	1420	385	476	662	834	20.5
1999	2044	374	503	1056	1522	30.6
2000	872	287	455	336	403	16.6
2001	955	237	364	331	436	24.1
2002	865	283	514	348	405	14.1

Table 5.7b Annual volumetric statistic for Ret watershed

Year	Rainfall (mm)	Infiltration (mm)	Evapotranspiration (mm)	Runoff Computed(mm)	Runoff Observed(mm)	Relative Error(%)
2000	1245	501	729	313	403	22
2001	1129	840	836	259	419	38
2002	1408	607	850	269	376	29
2003	2157	360	528	442	571	22
2004	1569	542	602	297	391	24
2005	1510	629	735	378	581	35
2006	1391	528	560	314	515	39
2007	1348	824	665	130	197	34
2008	1774	526	464	406	526	23
2009	1427	452	455	236	336	30

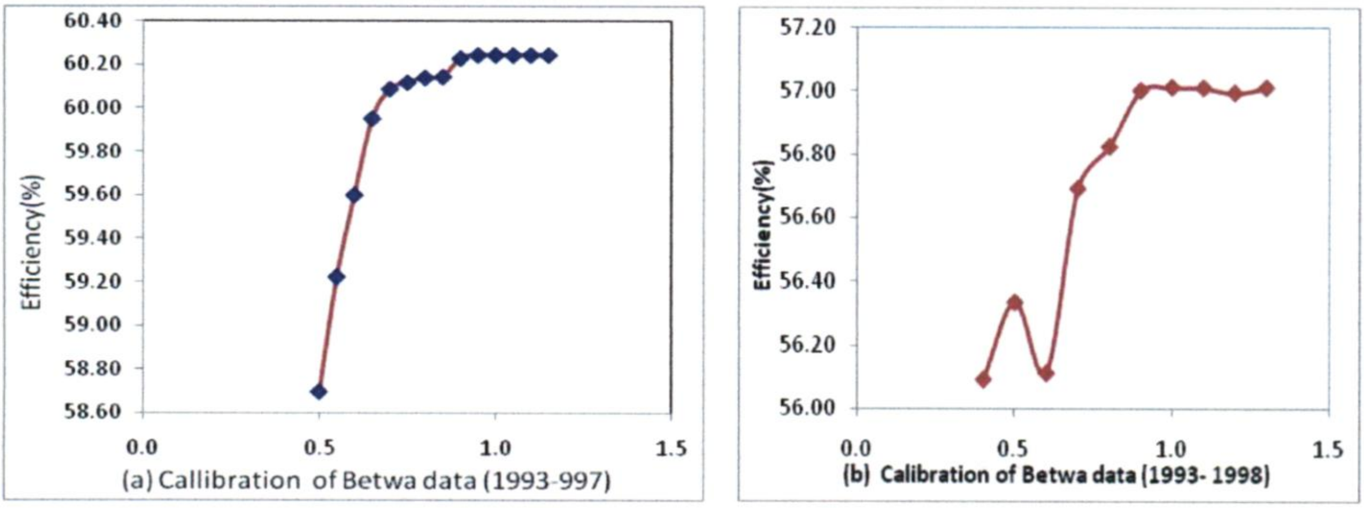
Table 5.7c Annual volumetric statistic for Siul watershed

Year	Rainfall (mm)	Infiltration (mm)	Evapotranspiration (mm)	Runoff Computed(mm)	Runoff Observed(mm)	Relative Error(%)
2000	2891	308	395	412	468	12.0
2001	2580	248	406	261	352	26.0
2002	2736	337	426	298	360	17.1
2003	2222	397	526	371	564	34.2
2004	2529	807	870	268	358	25.1
2005	2815	404	598	708	746	5.2
2006	2656	598	595	472	489	3.7
2007	2630	456	490	448	471	4.8
2008	2099	309	493	501	552	9.3

5.4 SENSITIVITY ANALYSIS

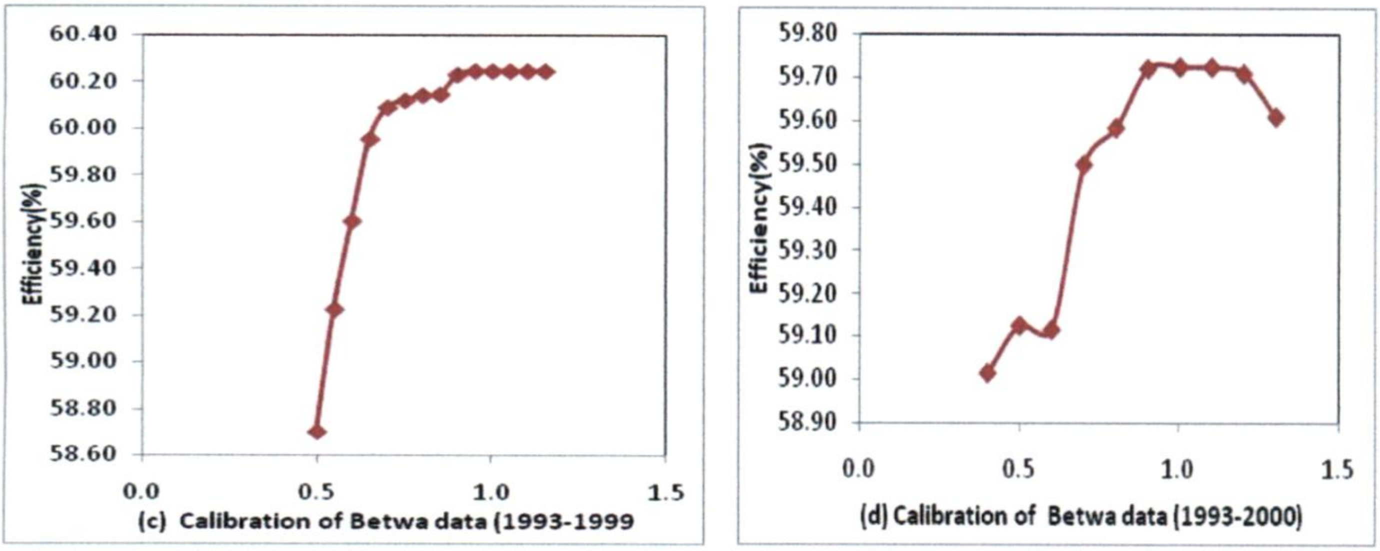
To assess the sensitivity of the model parameters, a sensitivity analysis was carried out. To this end, the parameters calibrated for 5-10 yrs of Betwa data and similar years of Ret and Siul data were varied for evaluating the impact of variation on the computed runoff values (or model performance in terms of efficiency) in calibration. If efficiency increases the computed values come closer to the observed ones, and vice versa if the efficiency decreases with the varying parameter values. However, the purpose of such an analysis lies in the distinguishing more sensitive parameters for their cautious and judicious employment in the field.

As shown in Figs. 5.13 to 5.15, as expected the efficiency in general decreases if the parameter is either drastically increased or decreased from the calibrated one. All the parameters were changed from 10% to 50%, and the corresponding efficiency computed. The change in efficiency in different years of calibration period (5-10 years) due to parameters CN_0, K and ALPHA is shown above mentioned figures of Betwa catchment and these are given below. The sensitivity figures of Ret and Siul catchments are given in Fig.III.1 to Fig.III.6 in Appendix-III. It is observed from the graphs that the percentage change in Parameter ALPHA shows small variance in efficiency of the model. The negative value of ALPHA indicates the occurrence of lateral outflow from the catchment in terms of any form including the intra-basin transfer.



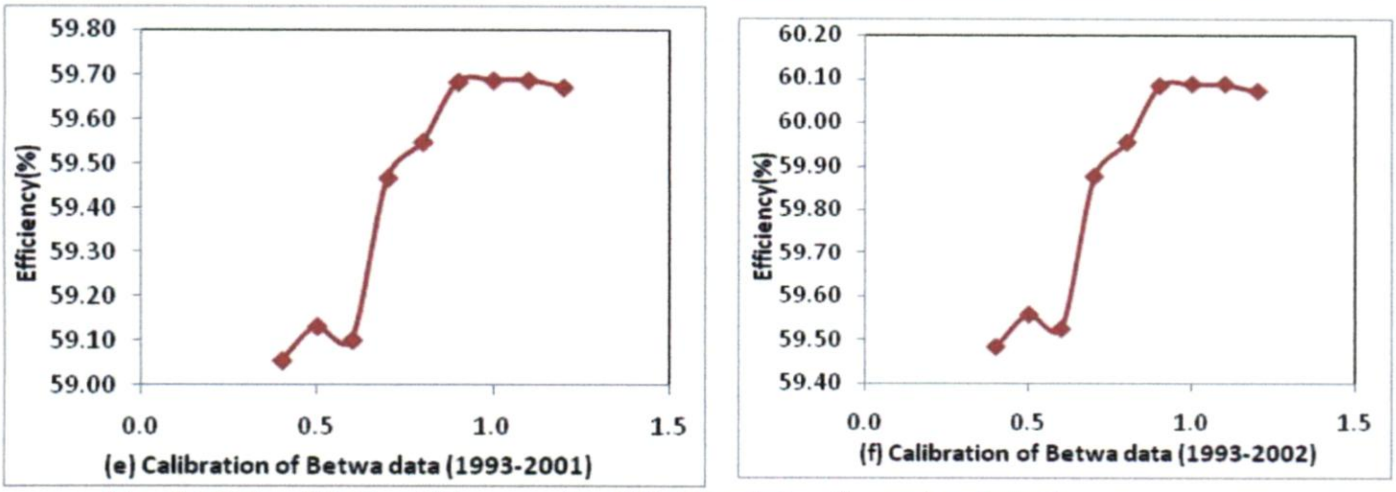
(a) Callibration of Betwa data (1993-997)

(b) Calibration of Betwa data (1993- 1998)



(c) Calibration of Betwa data (1993-1999

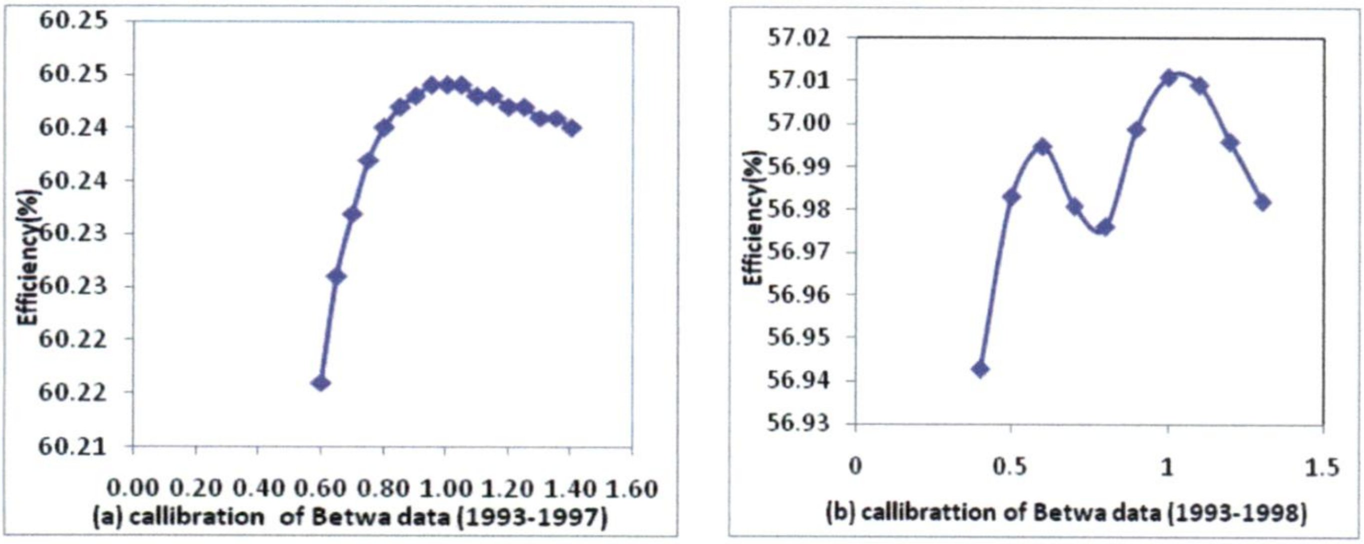
(d) Calibration of Betwa data (1993-2000)



(e) Calibration of Betwa data (1993-2001)

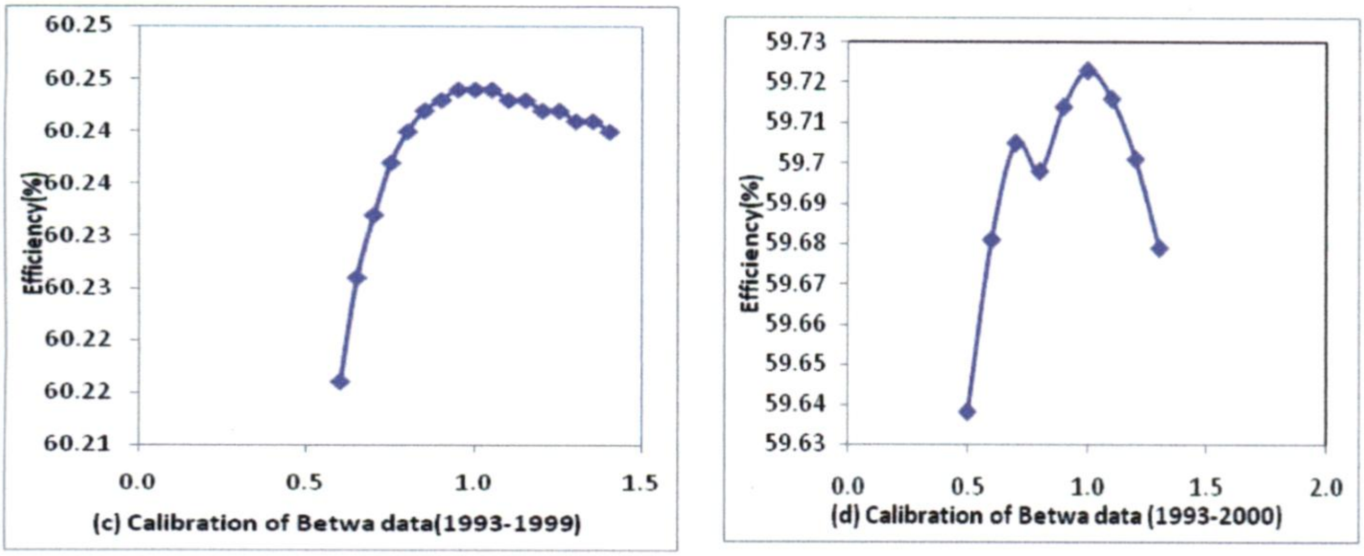
(f) Calibration of Betwa data (1993-2002)

Fig. 5.13 Sensitivity of Parameter CN_0 for data of varying lengths.



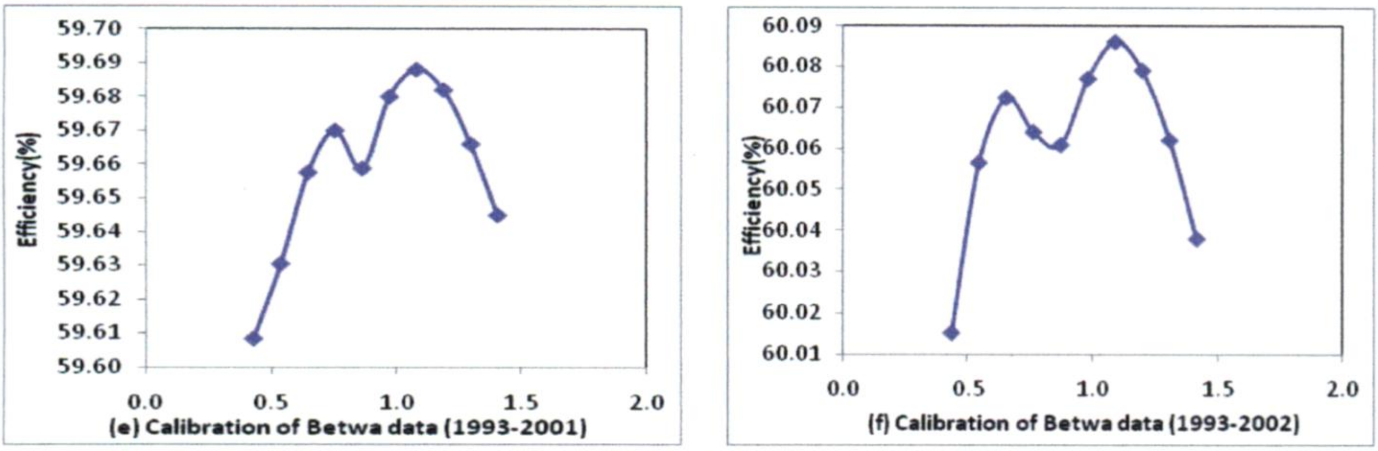
(a) callibration of Betwa data (1993-1997)

(b) callibrattion of Betwa data (1993-1998)



(c) Calibration of Betwa data(1993-1999)

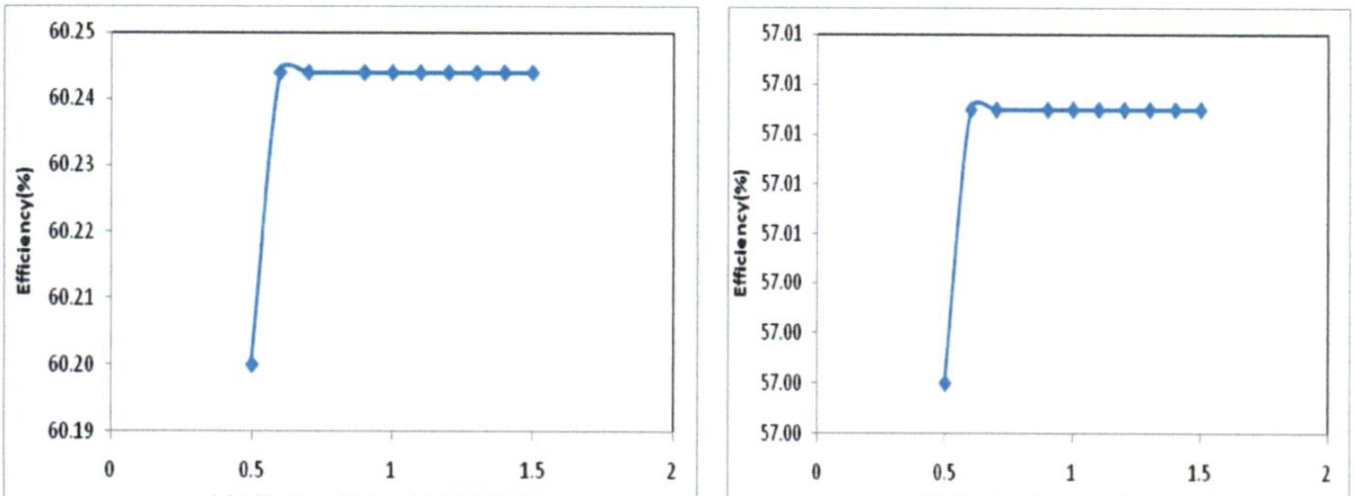
(d) Calibration of Betwa data (1993-2000)



(e) Calibration of Betwa data (1993-2001)

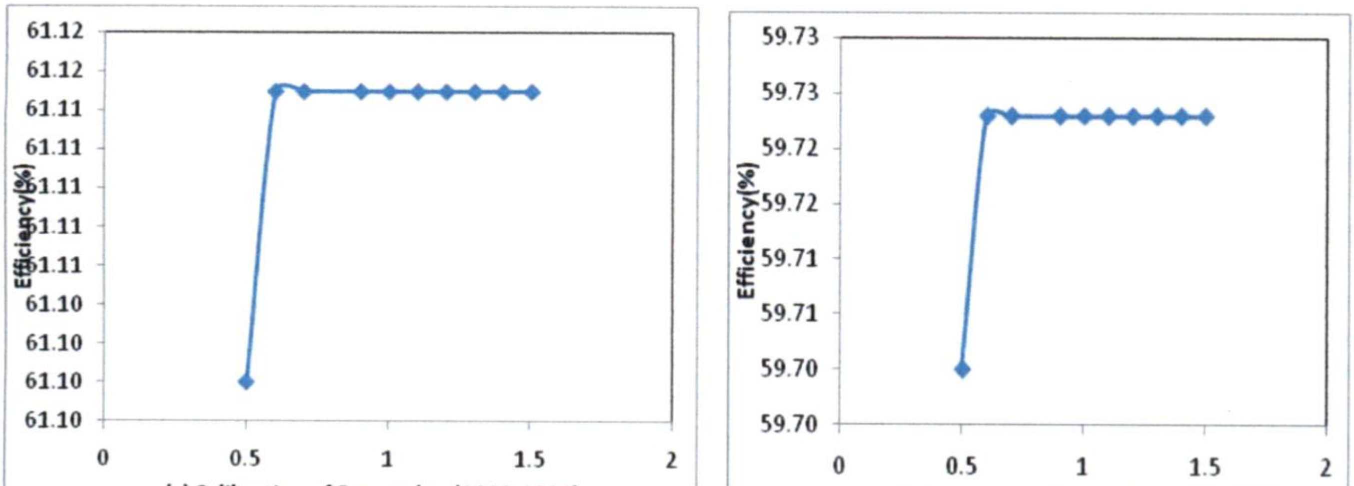
(f) Calibration of Betwa data (1993-2002)

Fig. 5.14 Sensitivity of Parameter K for data of varying lengths



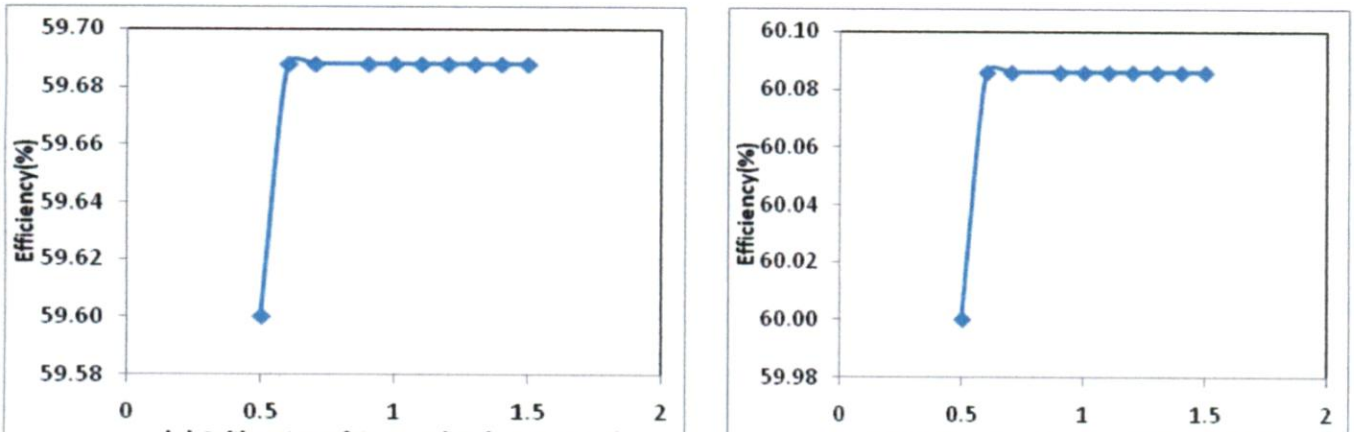
(a) Calibration of Betwa data(1993-1997)

(b) Calibration of Betwa data(1993-1998)



(c) Calibration of Betwa data(1993-1999)

(d) Calibration of Betwa data(1993-2000)



(e) Calibration of Betwa data(1993-2001)

(f) Calibration of Betwa data(1993-2001)

Fig. 5.15 Sensitivity of parameter ALPHA for data of varying lengths

CHAPTER 6
SUMMARY AND CONCLUSIONS

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 analyses. Since the rainfall data are generally available for a much longer period than are the stream flow data: long - term hydrologic simulation helps extend the gauged data required for the above field applications. In this study, Soil Conservation Service Curve Number (SCS-CN)-based long term hydrologic simulation model (Chapter 4) was developed, and tested using the data of three watershed namely Betwa (area = 4122 sq. km), Ret (area = 262 sq. km) and Siul (area = 360 sq. km). The proposed model has three model parameters, CN_0, K, ALPHA. The first parameter, when transformed, as such represents the initial value of curve number or, alternatively, the potential maximum retention and K is the catchment storage coefficient and ALPHA represents the coefficient for lateral inflow. For testing of the above SCS-CN based model, the daily rainfall, runoff and evaporation data of three catchments, Betwa (1993-2002), Ret (2000-2009), and Siul (2000-2008) were used. The first five years or more data was used for model calibration, and the remaining for validation. The following conclusions can be derived from the study:

(1) The model generally performed poorly as the length of data increased from 1 to 10 years. In yearly simulations, the resulting efficiencies for all the years vary in the range of 57.01 to 77.98% in Betwa watershed, 43.37 to78.14% in Ret watershed and 33.61 to 74.41 % in Siul Watershed. However, these values of efficiency show a satisfactory to less than satisfactory fit and, in turn, the model performance.

(2) In calibration with the first five years of data, the resulting efficiency was 60.24% for Betwa, 47.40% for Ret, and 39.33% for Siul watershed, and in validation, on the remaining five years of data, it was 63.35%, 62.01%, and 44.05% for respective watersheds. Thus the model as such performed satisfactorily only on Betwa, and poorly on the other two watersheds.

(3) The annual performance of the model varies from 66.19% to 82.76 % for Betwa, 64.90% to 78.99% for Ret, and 60.17% to 74.40% for Siul catchment, which indicates a satisfactory model performance in all catchments. Thus the model appears to have been performing better on annual data.

(4) The volumetric analysis was carried out for segregated components of the various components of hydrologic processes, such as precipitation, infiltration, evapotranspiration, and total runoff. The model simulated the yearly runoff values with relative errors in the range of 7.7 to 37.9% of Betwa data, 22 to 39%of Ret data, and 3.7 to 34.2% of Siul data. These appear to be in acceptable range.

(5) The above model performance is appreciable in view of the limited number of model parameters (only three), simplicity, and no constraints imposed for matching the observed annual runoff volumes.

REFERENCES

1. Bhunya, P.K., S.K. Mishra, and Ronny Bemdtsson. (2003),'Discussion on 'Estimation of confidence interval for curve numbers,' J. Hydrologic Engrg., ASCE, Vol. 8(4), 232-233.
2. Bonta, J.V. (1997)" Determination of watershed curve number using derived distributions", J. Irrigation and Drainage Engg., A.S.C.E., 123(1), pp 28-36.
3. Boszany, M., (1989) Generalization of SCS curve number method. J. Irrigation and Drainage Engineering, ASCE, 115 (1), 139-144.
4. Burton, J.S. Compiler (1993), Proceedings of the federal Interagency Workshop on hydrologic Modeling demands for the 90's, USGS Water resources Investigations report 93-4018, Federal center, Denver, Colrodo.
5. Chow, V.T., D.R Maidment, and L.W. Mays (1988). "Applied Hydrology."
6. Cazier, D.J., and Hawkins, R H., (1984) Regional application of Curve number method. Water today and tomorrow. Proc., ASCE, Irrigation and Drainage Division, Special Conference, ASCE, New York, N.Y.
7. Dooge, J. C. I. (1959) A general theory of the unit hydrograph. J.Geophys. Res., 64(2), 241-256.
8. Dooge, J.C.I. (1957) The ratjonal method for estimating flood peaks. *Engineering,* 184,311-313,
9. Dilip G. Durbude, Manoj K. Jain and Surendra K. Mishra(2011) “Long-term hydrologic simulation using SCS-CN-based improved soil moisture accounting procedure”, Hydrol. Process. 25, 561–579.
10. Dooge, J. C. I., (1973) Linear Theory of hydrologic systems, Tee. Bull. No. 1468, U. S. Deptt. of Agriculture, Agriculture Research Service, Washington D. C.
11. Hawkins, R. H., (1975) the importance of accurate curve numbers in the estimation of

storm runoff' Water Resources Bulletin, 11(5), pp 887-89.

12. Hann, C.T., (1975),'Evaluation of a model for simulating monthly water yields from small watersheds,' Southern Cooperative Series Bulletin 201, Agricultural Experiment tations of Kentucky, Mossissipi, North Carolina, South Carolina, Tennessee, and Virginia, July.
13. Hawkins, R. H., (1978) Runoff Curve Numbers with varying site moisture. J. Irrig. & Drainage Engg., Vol.104, No.IR4, 389-398.
14. Hawkins, R. H., (1984) Comparisions of predicted and observed runoff curve numbers" Proc. Spec. Conf., Irrigation and Drainage Div., A.S.C.E., New York, N.Y., pp 702-709.
15. Hawkins, R. H.AT. Hjelmfelt, Jr. and AW. Zevenbergen (1985) Runoff probability, storm depth, and curve numbers, 1. Irrig. and Drainage Engrg. A.S.C.E., III (4), pp 330-339.
16. Holtan, H. N., and Lopez, N. C., (1971) USDHAL-70 Model of Watershed Hydrology. Technical Bulletin 1435, U. S. Department of Agriculture.
17. Horton, R.I., (1933)." The role of infilteration in the hydrologic cycle", Trans., Am. Geophysical Union, 14, 446-460.
18. Huber, W.C., Heaney, J.P.,Bedient, RP:, Bender, J.P., (1976)." Environmental Resource Management studies in Kissimmee River Basin. Report Env-05-76-3, Deparment of Enviormental Engineering and Science, University ofFlorida:
19. Jain, M.K., Mishra, S.K., Babu, P.S., Venugopal, K., and Singh V.P.(2006). "Enhanced runoff curve number model incorporating storm duration and a non linear Ia-S relation". J. Hydrologic Engineering, ASCE, 11(6),631-635.
20. Knisel W.G., 1980, CREAMS, a Field- Scale Model. for Chemicals, Runoff, and Erosion of Agricultural Management Systems. Conservation Report 26, USDA

Agricultural Research Service: Washington, DC

21. K. Geetha , S. K. Mishra , T. I. Eldho, A. K. Rastogi, R. P. Pandey(2008). "SCS-CN-based Continuous Simulation Model for Hydrologic Forecasting", Water Resource Manage (2008) 22:165–190.
22. Mishra, S.K. and A.K. Dwivedi (1998),'Determination of curve numbers for a watershed,'9th Nat. Symp. on Hydrology, Indian National Committee on Hydrology (INCOH), Amritsar, India, Nov. 26-27.
23. Mishra, S. K., and Singh, V. P., (2002) Chapter 13, SCS-CN based hydrologic simulation package. In Mathematical Models in small Watershed Hydrology, Singh V.P, Frevert D. K. (eds). Water resources Publications:P.O.Box 2841, Littleton, CO; 80161.
24. Mishra, S. K., and Singh, V. P., (2002a) SCS-CN based hydrologic simulation package. In Mathematical Models in small Watershed Hydrology, Singh V.P, Frevert D. K. (eds). Water resources Publications: Littleton, CO; 391-464.
25. Mishra, S.K., and Singh, V.P., (2002 b) SCS-CN Method. Part 1: Derivation of SCS-CN based Models. Acta Geophysicapolonica, Vol. 50, No.3.
26. Mishra, S.K., and Singh, V.P., (2004) Long-term hydrologic simulation based on Soil Conservation Service curve number. Hydrological processes, 18, 1291-1313.
27. Mishra, S.K., and Singh, V:P., (2004) Validity and extension of the SCS-CN method for computing infiltration and rainfall-excess rates. Hydrological processes, 18, 3323-3345.
28. Mishra, S.K., and Singh, V.P., (1999) Another Look at SCS-CN Method. J. Hydrologic Engineering, VolA, No.3, 257-264.
29. 32. Mishra, S.K., Jain, M.K., and Singh, V.P., (2004) Evaluation of the SCS-CN based -

model incorporating Antecedent moisture. Water Resources Management; 18: 567589.

30. Mishra, S.K., Jain, M.K., Pandey, R.P., and. Singh, V.P., (2003) Evaluation of the AMC-Dependent SCS-CN based Models using large data of Small Watersheds. Water and Energy International, Vol. 60, No.3, 13-23.
31. Moore, R.J and Clarke, R.T., (1981) A distribution function approach to rainfall runoff modeling. Water Resources Res., 17 (5), pp. 1367-1382.
32. Moore, R.J and Clarke, R.T., (1982) A distribution function approach to modeling basin soil moisture deficit and stream flow, In: statistical analysis of Rainfall and Runoff (ed. V. P. Singh), Water Resources Publications, 173-190.
33. Moore, R.J and Clarke, R.T., (1983) A distribution function approach to modeling basin sediment yield. J. Hydrology, 65, 239-257.
34. Moore, R.J., (1985), The probability distributed principle and runoff production at point and basin scales, 'Hydrological Sciences Sciences J.30, pp. 273-257.
35. Moore, R.J., (1983), 'The probability- distributed approach to spatial conceptual rainfall-runoff modeling, 'report to Flood Protection Commission, Ministry of Agriculture, Fisheries and Food, Institute of Hydrology, Wallingford, Oxfordshire.
36. Neitsch, S.L., Arnold, J.G., Kiniry, J.R., Williams, J.R., and King, K.W. (2002). "Soil and water Assessment Tool (SWAT) Theoretical Documentation, Version 2000." Texas Water Resources Institute, College Station, Texas, TWRI Report TR-191.
37. Pandit, A., and Gopalkrishnan, G., (1996) Estimation of annual storm runoff coefficients by continuous simulation. J. Irrig. & Drainage Engrg., VoLl22, No.4, 211-220.
38. Ponce, V.M., and Hawkins, R.H.,(1996) Runoff Curve Number: Has. it reached maturity? J. Hydrologic Engineering, VaLl, No.1, 11-19.
39. R. K. Sahu, S. K. Mishra, T. I. Eldho and M. K. Jain (2007). " An advanced Soil

Moisture accounting procedure for SCS curve number method", *Hydrol. Process.* **21**, 2872–2881 (2007)

40. Rallison, R.E., N. Miller, (1982) 'Past, present, and future', Proc. Int. Symp. Rainfall-Runoff Relationship, (ed. V.P. Singh), water Resources Pub., P.O. Box. 2841, Littleton, Colorado 80161.
41. Rallison, R.E. and R.C. Cronshey, (1979), Discussion to 'Runoff curve numbers with varying soil moisture, 1. Irrig. and Drain. Div. A.S.C.E., 1 05(4),439-441.
42. Ramasastri, K.S. and Seth, S. M., (1985) Rainfall-runoff relationships, Rep. RN-20, National Institute of Hydrology, Roorkee-247667, Uttar Pradesh, India.
43. Schaake, J.C, V.I. Koren, Qing-Yun Duan, K. Mitchell, and F. Chen (1996),'Simple water balance model for estimating runoff at different spatial and temporal scales,' J. Geophysical Research, Vol. 101, No. 03, pp. 7461-7475.
44. SCS (1985) National Engineering Handbook, Hydrology, Section 4, Soil Conservation Service, US Department of Agriculture, Washington DC.
45. Sobhani, G. (1975) A review of selected small design methods for possible adoption to Iranian conditions, M.S. Thesis, Utah State Univ., Logan Utah.
46. Soil Conservation Service (SCS) (1956, 1971, 1993) Hydrology; National Engineering Handbook; Supplement A, Section 4, Chapter 10, Soil Conservation Service, U.S.D.A.,Washington, D.C.
47. Soni, B., and Mishra, G. C., (1985) Soil water accounting using SCS hydrologic soil classification. Case Study, National Institute of Hydrology, Roorkee, India.
48. Springer, E.P., B.J. McGurk, R.H Hawkins, and G.B. Goltharp (1980),'Curve numbers from watershed data,' Proc., A.S.C.E. Irrigation and Drainage Symp. on Watershed Management, AS.C.E., New York, N.Y., Vol. II, 938-950.

49. Stephenson, G.R. and R.A Freeze (1974),'Mathematical simulation of subsurface flow contribution to snow melt runoff, Reynolds Creek watershed, Idaho,' Water Resources research, 10, 284-294.

50. Smith, R. E., and J. R. Williams. (1980). Simulation of surface water hydrology. In: W. G. Knisel (ed.), CREAMS: A Field-Scale model for Chemicals, Runoff, and Erosion from Agricultural Management Systems, U. S. Department of Agriculture, Science and Education Administration, Conservation Research Report No. 26, Vol. I, Chapter 2. pp.

51. Todini, E. (1988),'Rainfall-runoff modeling: Past, present, and future,' J. Hydrology, 100, 341-352.

52. Van Mullem, J.A., (1989) Runoff and Peak discharges using Green-Ampt infiltration model. J. Hydraulic Engineering, ASCE, 117 (3), pp. 354-370.

53. Williams, J. R., and LaSeur, W. V., (1976) Water Yield model using SCS curve numbers.

54. Wurbs, R.A., (1998), Dissemination of Generalized water resources models in the United States,' Water International, 23,pp. 190-198.

55. Woodward, D.E., and W. J. Gburek (1992),' Progress report ARS/SCS curve number work group, 'Proc., ASCE, Water Forum'92, ASCE, New York, N.Y., 378-382.

APPENDIX I

Program for Daily Rainfall-Runoff Modeling using SCS-CN Method

```
$DEBUG
$LARGE
C     ORIGINAL SCS –CN METHOD with single linear reservoir routing
C     PROGRAM FOR DAILY RAINFALL-RUNOFF MODELLING USING SCS –CN
METHOD
C     PARAMETER 'S' MODIFIED BY EVAPOTRANSPIRATION
C
        CHARACTER*80 TITLE
        DIMENSION B(10),Y(5000),BV(10),BMIN(10),BMAX(10),P(20000)
        DIMENSION A(10,10),AC(10,10)
      DIMENSION DIS (365), RF (365), EV (365), DISCHARG (5000)
        DIMENSION EVAPO (5000), F (5000)
        DIMENSION QCOMP (5000), S (5000), BF (5000),EVATRA (5000)
      DIMENSION RR (365), QQ (365), QQC (365), FF (365), SS (365)
        DIMENSION BBF (365) ,EEVTRA (365), QQCOMP (365)
C       DIMENSION NYEAR(10)
        DIMENSION X(5000),Z(5000)
        COMMON /A/ X
        COMMON /B/ S,QCOMP,EVAPO,EVATRA,F,BF
        COMMON /C/ NYR
        EXTERNAL FUNC,DERIV
      OPEN(UNIT=1,FILE='betwa.TXT',STATUS='UNKNOWN')
      OPEN(UNIT=3,FILE='SCS.DAT',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),EV(J)
C     WRITE(*,*) NNN,DIS(J),RF(J),EV(J)
C CONVERT DISCHARGE TO mm (CA=*** SQ. KM)
      DISCHARG(K)= DIS(J)
      EVAPO(K)=EV(J)
      Y(K)=DISCHARG(K)
      X(K)=RF(J)
      SUMQ=SUMQ+DISCHARG(K)
      SUMRF=SUMRF+X(K)
      SUMEVA=SUMEVA+EVAPO(K)
      ENDDO
      ENDDO
C
      NN=NYR*365
    AMQ=SUMQ/FLOAT(NN)
C
C     READ IN INITIAL GUESSES.
C     READ (3,*) CN0, AK, ALPHA
      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
     I 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     S  RF-EXCESS   INFILT ',
   1'BASEFLOW RUNOFF(C) RUNOFF(O)  EVAPTRA'/)
     SUMFO=0.0
     SUMF1=0.0
     SUMQC=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
     SUMQC=SUMQC+Z(K)
        RR(K1)=X(K)
     FF(K1)=F(K)
     QQ(K1)=DISCHARG(K)
     QQC(K1)=Z(K)
       SS(K1)=S(K)
       BBF(K1)=BF(K)
       EEVTRA(K1)=EVATRA(K)
       QQCOMP(K1)=QCOMP(K)
       WRITE(4,1294) K1,RR(K1),SS(K1),QQCOMP(K1),FF(K1),
   1 BBF(K1),QQC(K1),QQ(K1),EEVTRA(K1)
1294   FORMAT(1X,I5,1X,8F10.2)
```

```
      ENDDO
      ENDDO
      EFF=(1-SUMF1/SUMFO)*100.
      WRITE(4,1295)EFF
      WRITE(*,1295)EFF
1295   FORMAT(1X,'EFFICIENCY= ',1X,F10.3)
        WRITE(4,4444)SUMQ,SUMRF,SUMEVA,SUMQC
4444   FORMAT(1X,'SUMQ= ',F12.2,2X,'SUMRF= ',F12.2,2X,
     1'SUMEVA= ',F12.2,'SUMQC= ',F12.2/)
      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 F(5000),BF(5000)
     DIMENSION QCOMP(5000),EVAPO(5000)
        DIMENSION X(5000),Z(5000),QC(5000),B(10)
        DIMENSION S(5000),EVATRA(5000)
        COMMON /A/ X
        COMMON /B/ S,QCOMP,EVAPO,EVATRA,F,BF
        COMMON /C/ NYR
C HERE, X IS RAINFALL, Z IS THE COMPUTED OUTFLOW (mm)
C DEFINE HERE THE WHICH B() PARAMETER REFERS TO WHICH REAL
PARAMETER.
        CN0 = B(1)
        AK = B(2)
        ALPHA = B(3)
        AL = 0.2
C COMPUTATION OF COURANT NUMBER AND C0, C1, AND C2
        COUR=1./AK
```

```
      C0=(COUR*(1.+ALPHA)/(2.+COUR))
      C1=C0
      C2=(2.-COUR)/(2.+COUR)
C     COMPUTATIONS BEGIN
      K=0
    CN=CN0
    DO 2 I=1,NYR
    DO 3 J=1,365
      K=K+1
    IF(K.LE.5)THEN
    S(K)=25.4*((1000.0/CN0)-10.)
      S0=S(K)
    SIA=0.2*S(K)
        IF(X(K).GT.SIA)THEN
        QCOMP(K)=(X(K)-SIA)**2/(X(K)-SIA+S(K))
        ELSE
        QCOMP(K)=0.0
        ENDIF
    QC(K)=QCOMP(K)
    ENDIF
    IF(K.GT.5)THEN
       SIA=AL*S(K)*(X(K)/(X(K)+S(K)))
            IF(X(K).GT.SIA)THEN
            QCOMP(K)=(X(K)-SIA)**2/(X(K)-SIA+S(K))
            F(K)=X(K)-SIA-QCOMP(K)
            ELSE
            QCOMP(K)=0.0
            F(K)=0.0
            ENDIF
C
C      ROUTING OF COMPUTED OUTFLOW
      QC(K)=C0*QCOMP(K)+C1*QCOMP(K-1)+C2*QC(K-1)
      IF(QC(K).LT.0.)QC(K)=0.0
C      BASEFLOW ROUTING
      BF(K+NLAG)=0.0
C      TOTAL RUNOFF
    Z(K)=QC(K)+BF(K)
      ENDIF
C COMPUTATION OF EVAPOTRANSPIRATION
      IF(J.GE.1.AND.J.LE.122)PANC=0.8
```

```
      IF(J.GT.122.AND.J.LE.245)PANC=0.6
      IF(J.GT.245.AND.J.LE.365)PANC=0.7
      EVATRA(K)=PANC*EVAPO(K)
      S(K+1)=S(K)-(1.-BCOEF)*F(K)+EVATRA(K)
      IF(S(K+1).LE.0.)S(K+1)=0.0001
      CN=1000./(S(K+1)/25.4 + 10.)

3     CONTINUE
2     CONTINUE
       WRITE(4,1296)CN0,AK,ALPHA
     WRITE(*,1296)CN0,AK,ALPHA
1296  FORMAT(1X,'CN0= ',1X,F10.3,1X,'AK= ',1X,F10.3,
      1'ALPHA= ',1X,F10.3)

        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(5000),Y(5000),BV(10),BMIN(10),BMAX(10)
      DIMENSION P(20000)
      DIMENSION A(10,10),AC(10,10),X(5000)
      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
C  EPSILON TEST
C
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
C  GAMMA LAMDA TEST
C
```

```
      IF (FL.GT.1.0.AND.GAMM.GT.90.0) ICON=-1
      GOTO 2105
C
C  GAMMA EPSILON TEST
C
1400  IF(FL.GT.1.0.AND.GAMM.LE.45.0) ICON=-4
      GOTO 2105
C
C
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
C
      END
C
      FUNCTION ARCOS(Z)
C
      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
```

APPENDIX-II

CALIBRATION AND VALIDATION FIGURES

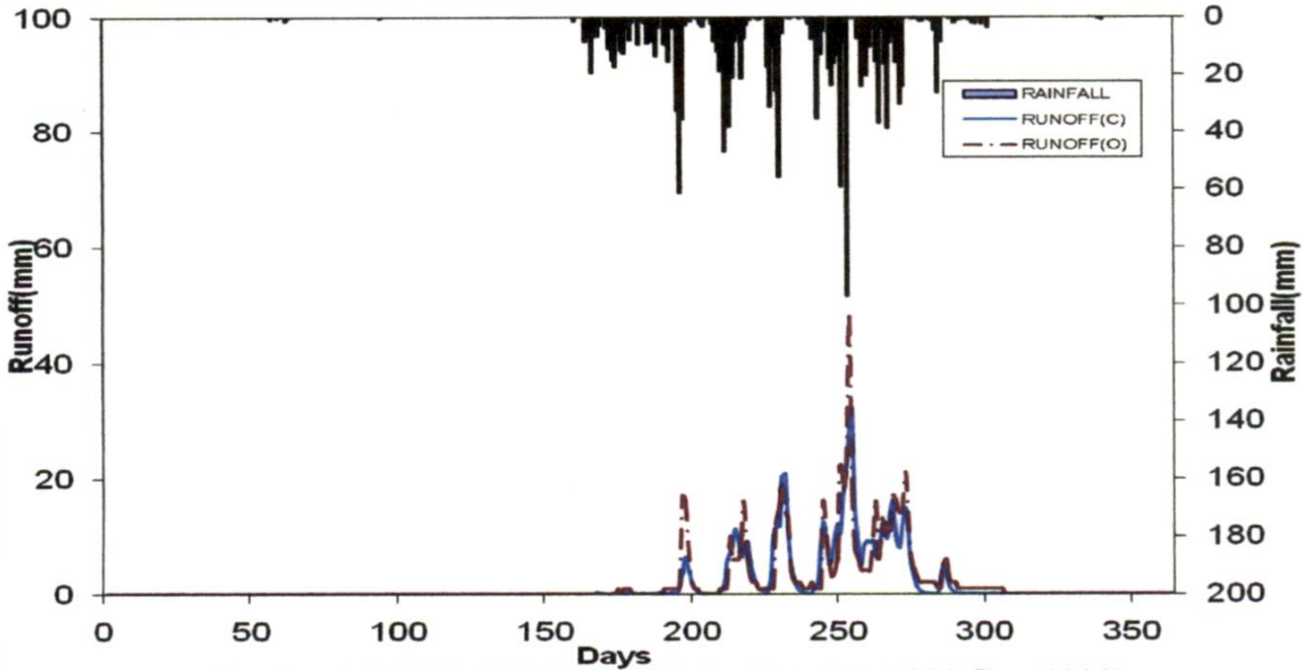


Fig. II.1 Model calibration for Betaw data(Jan.1993-Dec.1993)

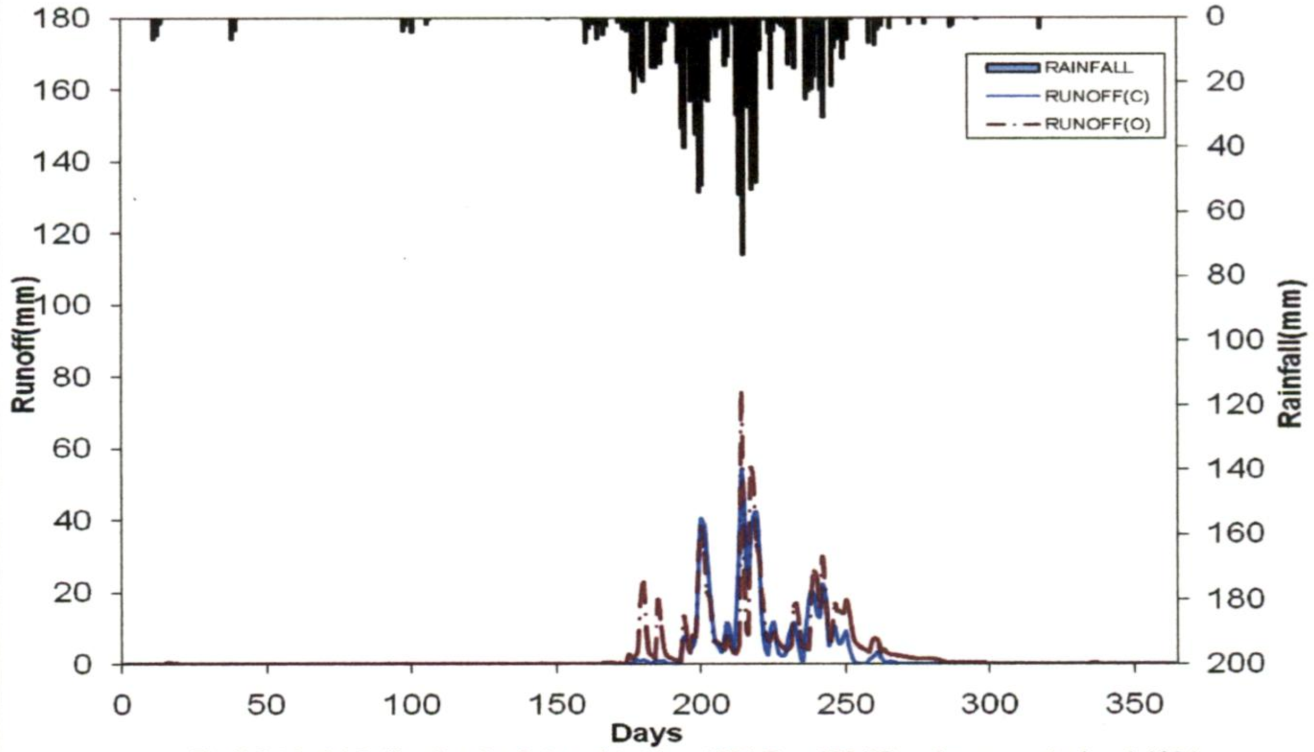


Fig.II.2 Model Calibration for Betwa data(Jan. 1994-Dec.1994)Day 1 represents Jan.1,1994

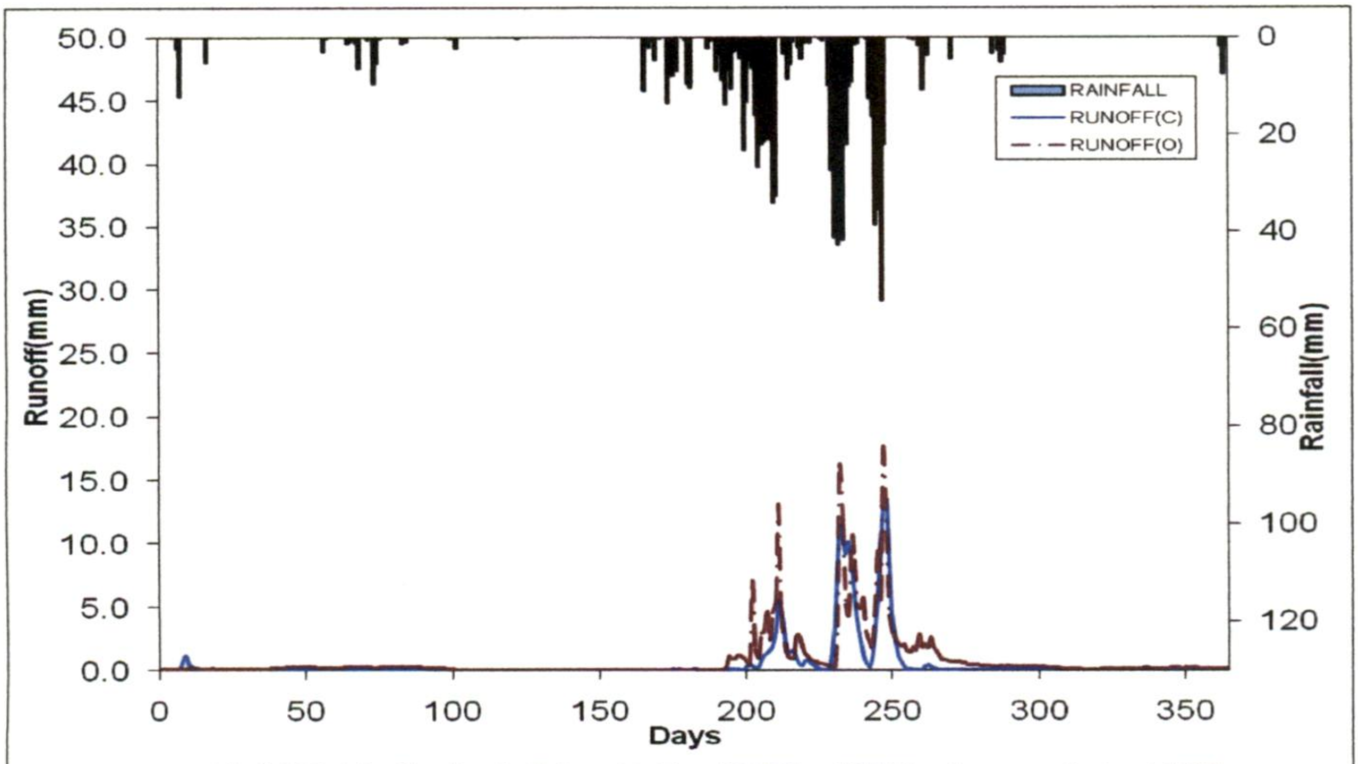


Fig.II.3 Model calibration for Betwa data(Jan. 1995-Dec.1995)Day 1 represents Jan.1,1995

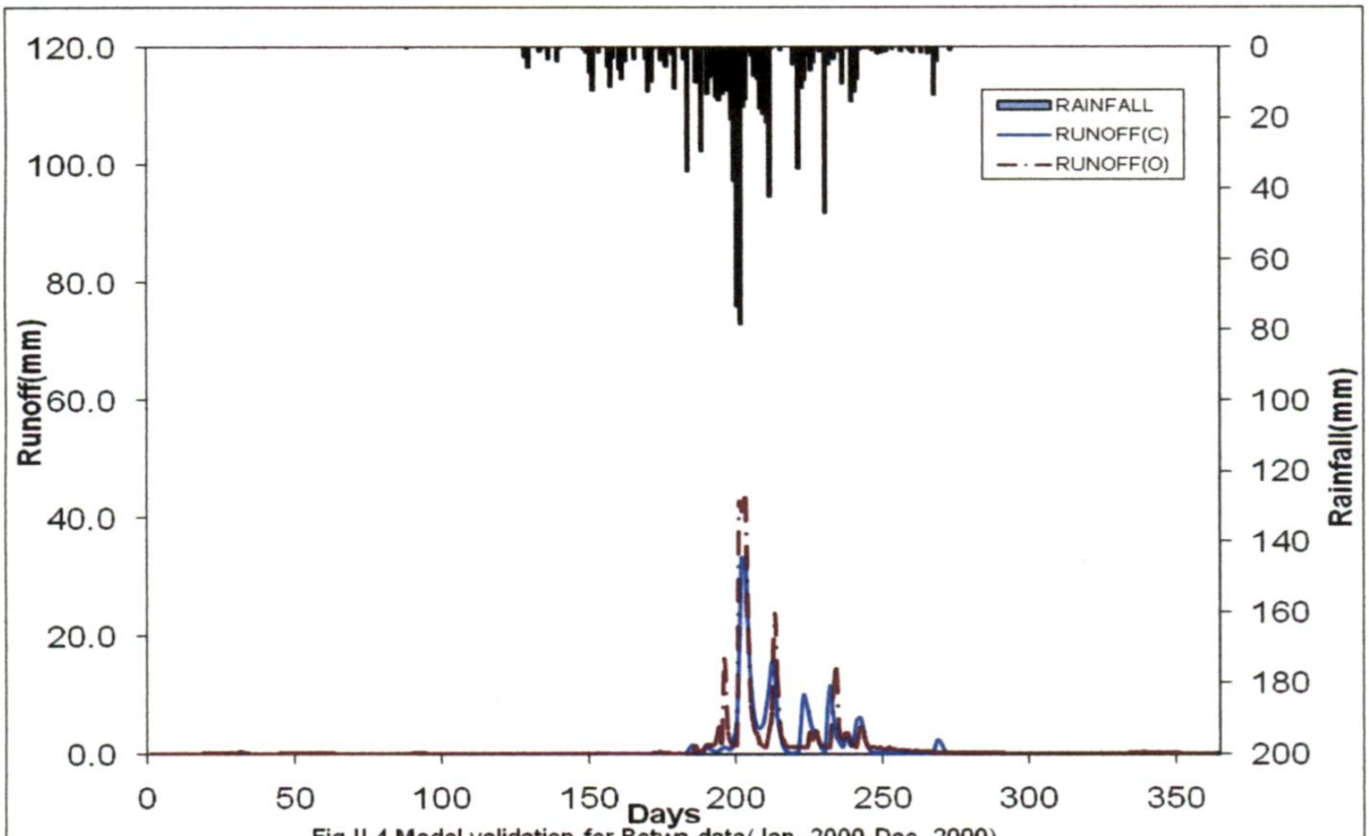


Fig.II.4 Model validation for Betwa data(Jan. 2000-Dec. 2000)

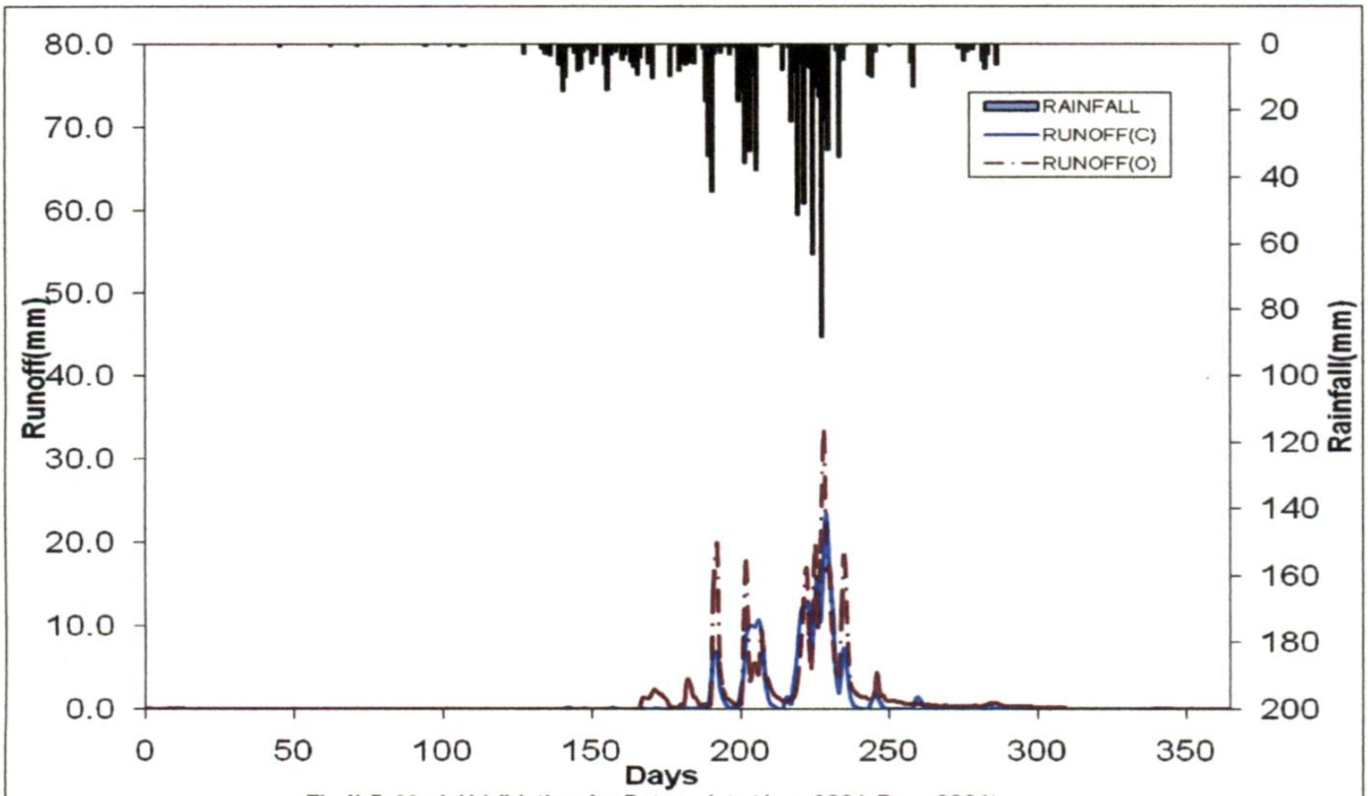


Fig.II.5 Model Validation for Betwa data(Jan. 2001-Dec. 2001)

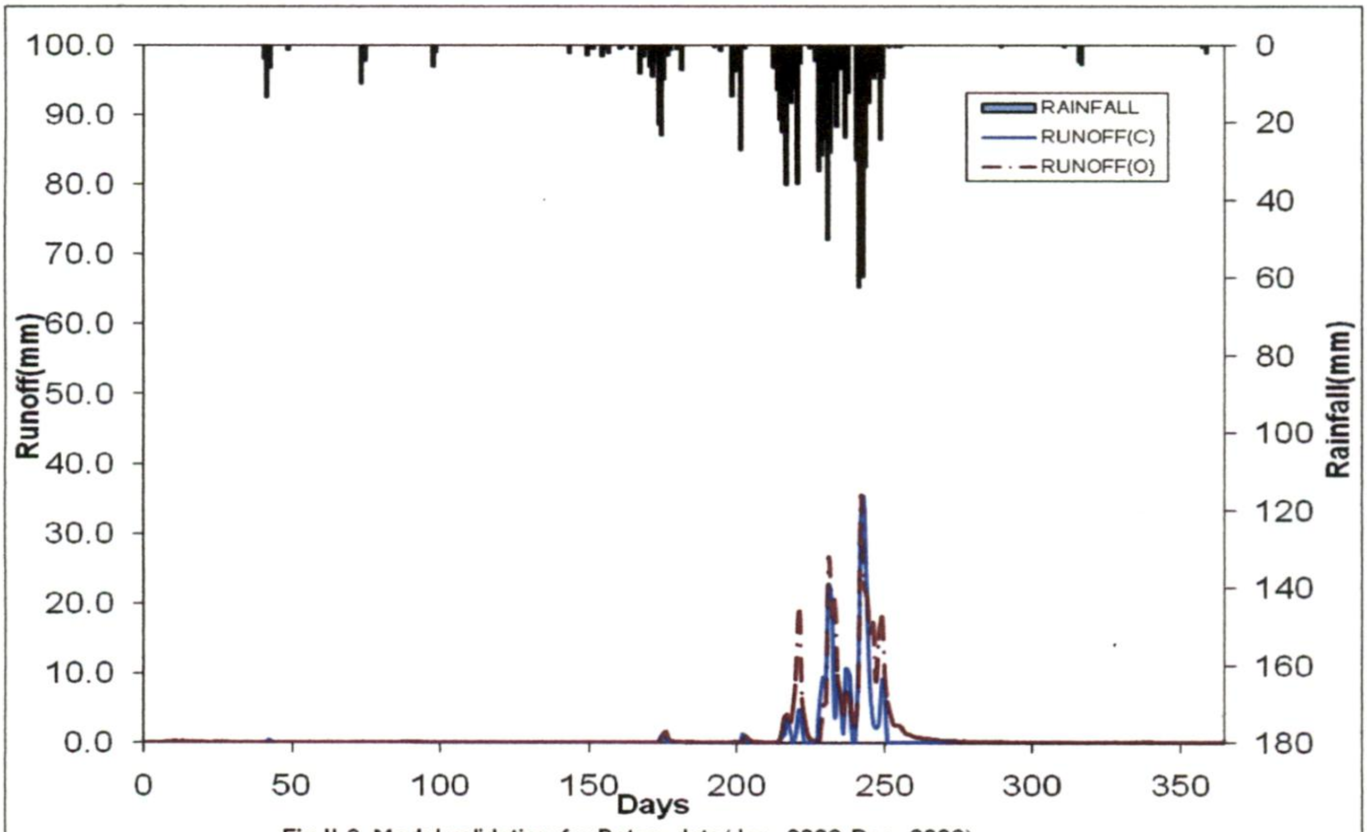


Fig.II.6 Model validation for Betwa data(Jan. 2002-Dec. 2002)

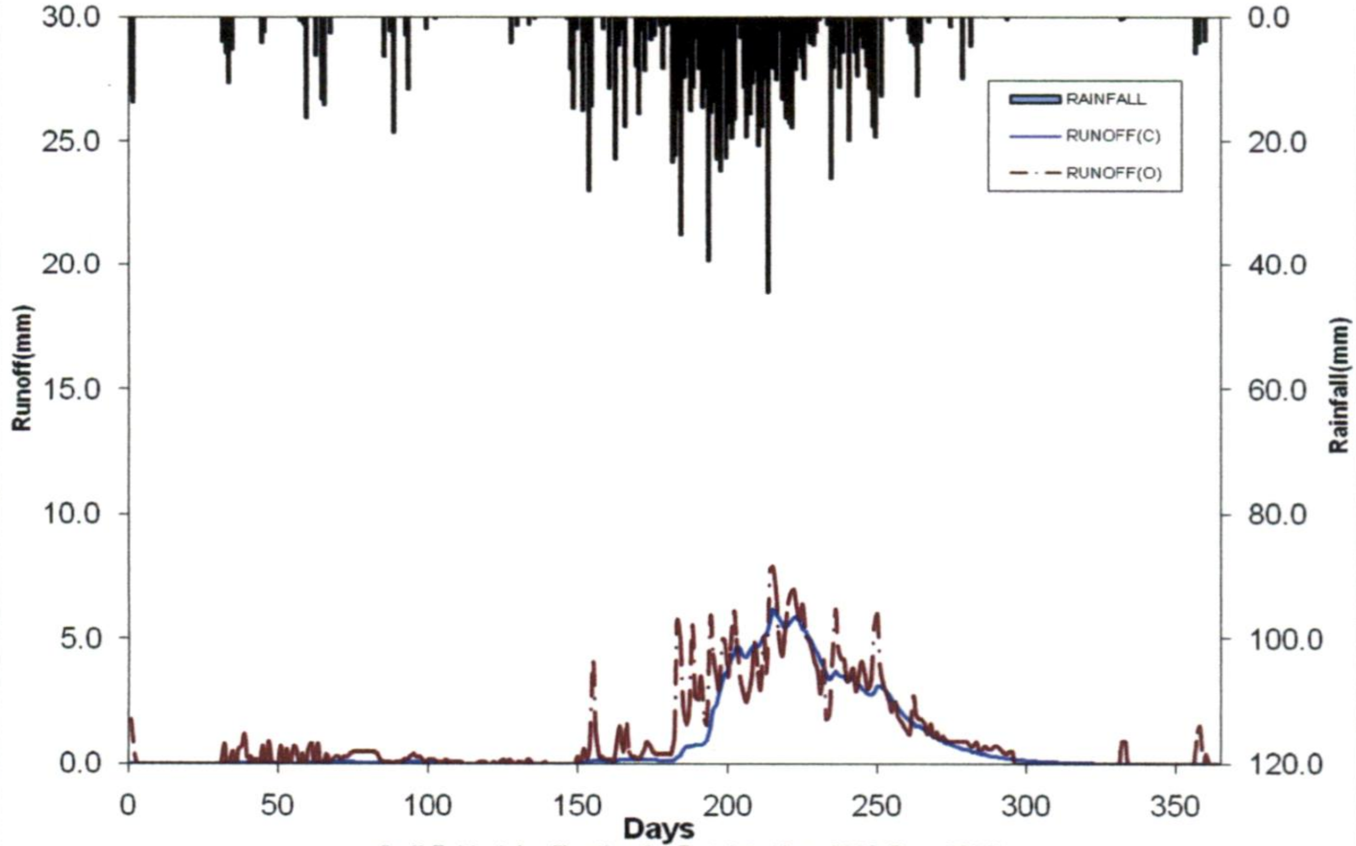


fig.II.7 Model calibration for Ret data(Jan. 2000-Dec. 2000)

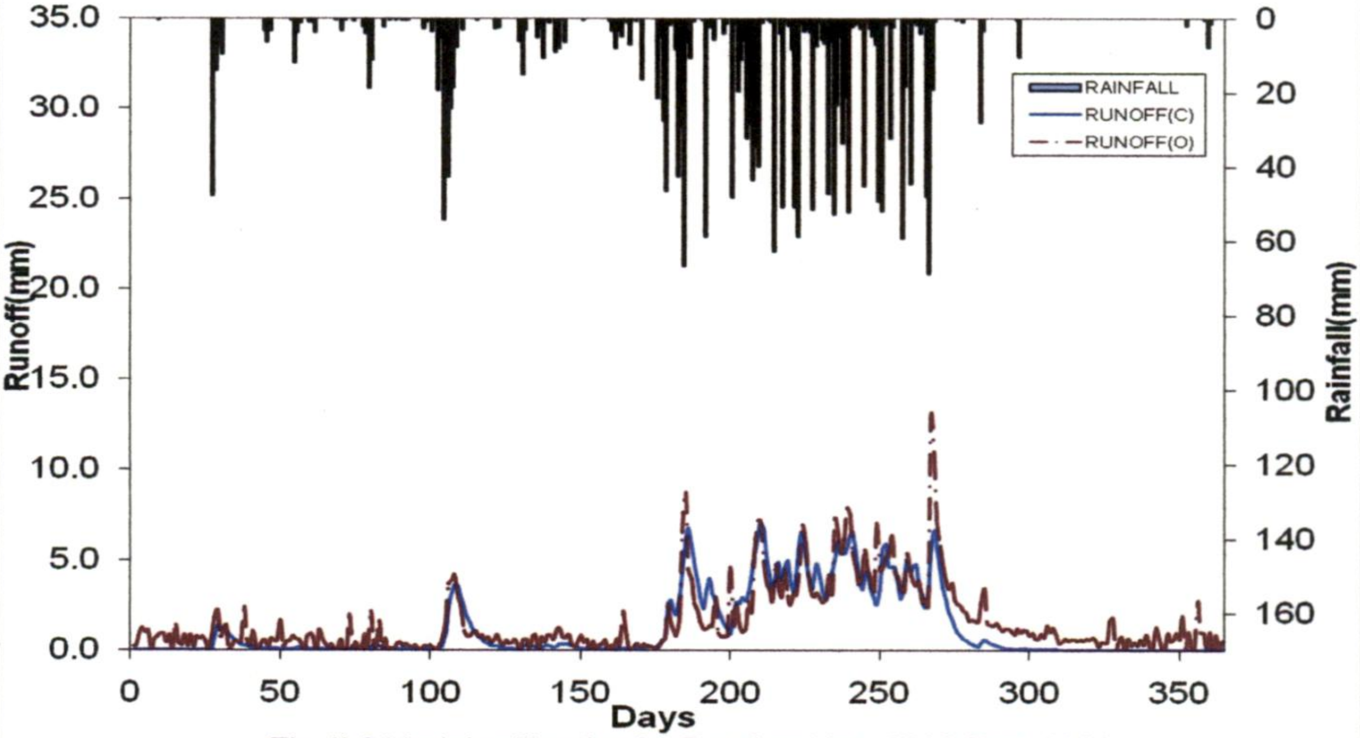


Fig. II.8 Model calibration for Ret data(Jan. 2003-Dec. 2003)

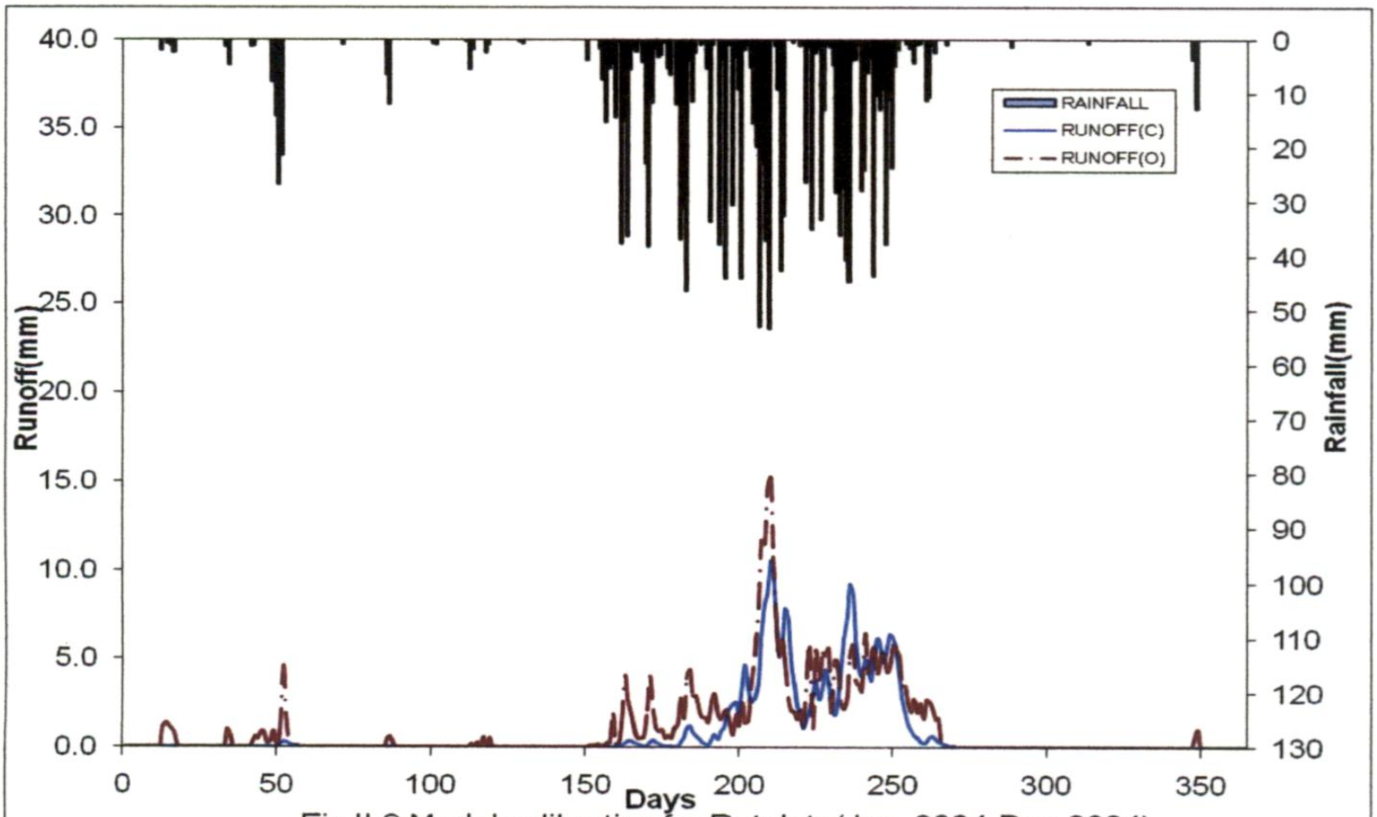


Fig II.9 Model calibration for Ret data(Jan. 2004-Dec.2004)

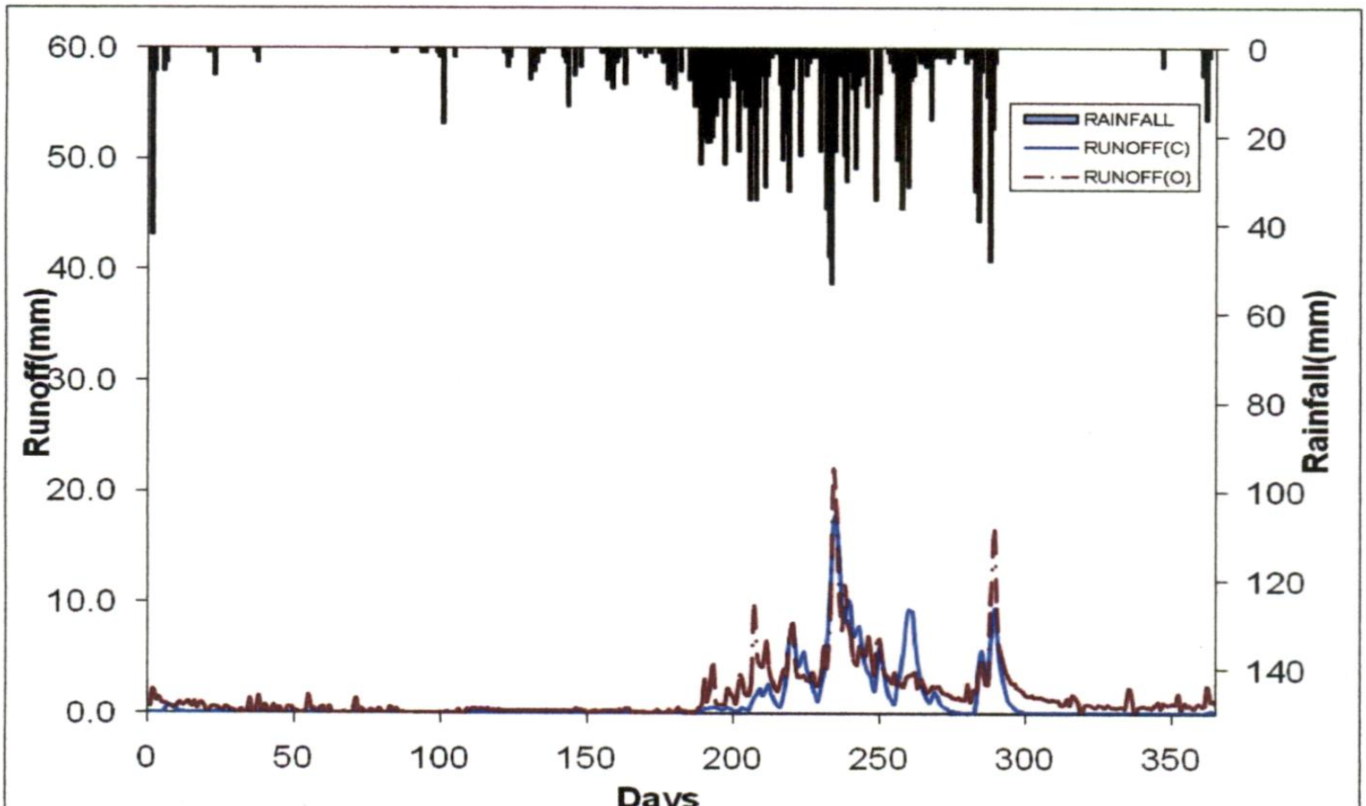


Fig.II.10 Model validation for Ret data(Jan. 2005-Dec.2005)

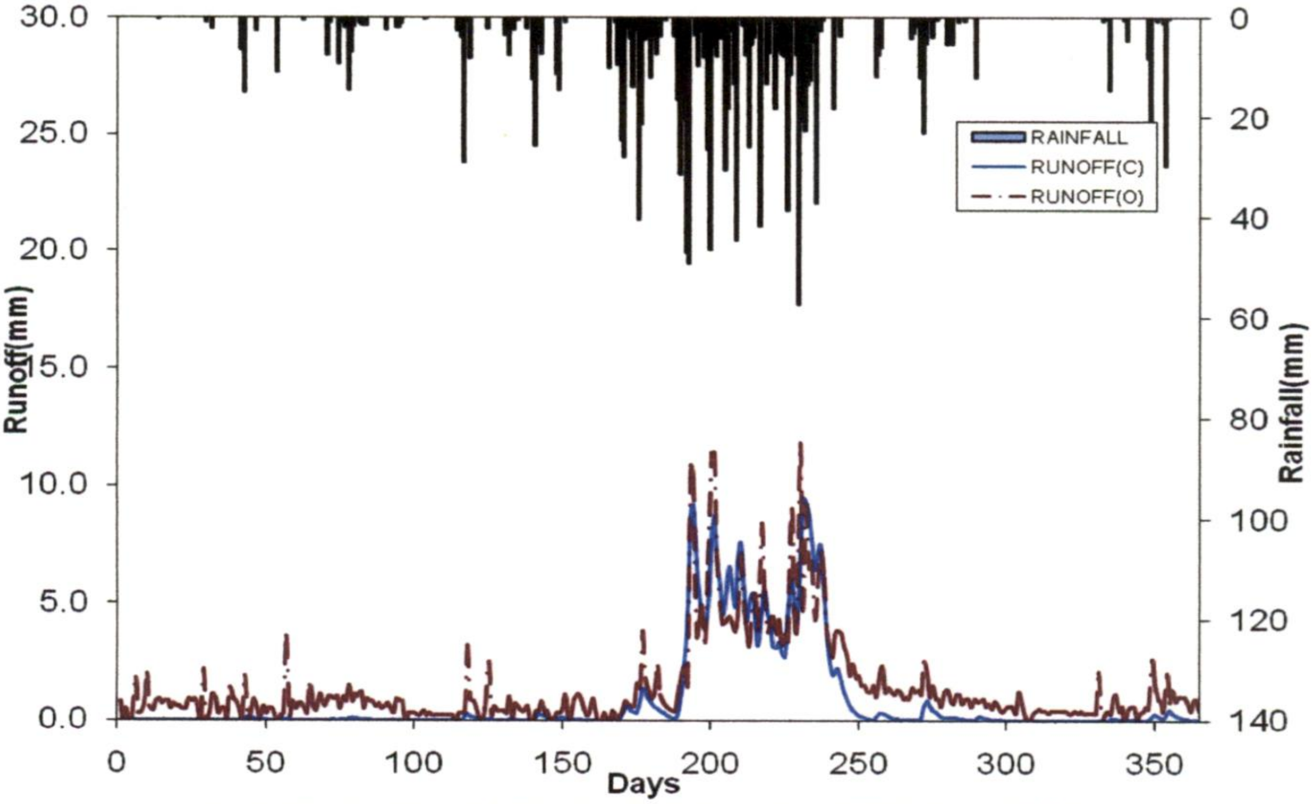


fig. II. 11 Model validation for Ret data(Jan. 2006-Dec. 2006)

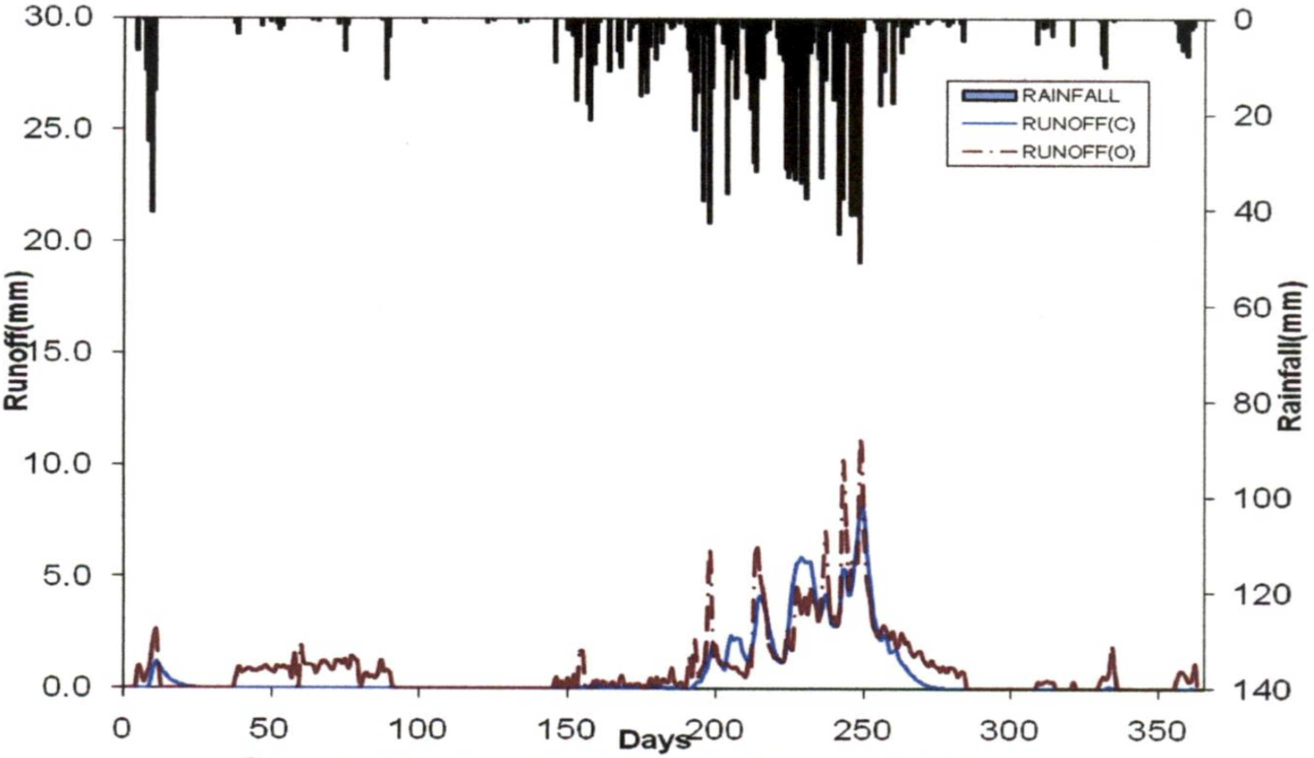


Fig. II. 12 Model validation for Ret data(Jan. 2009-Dec. 2009)

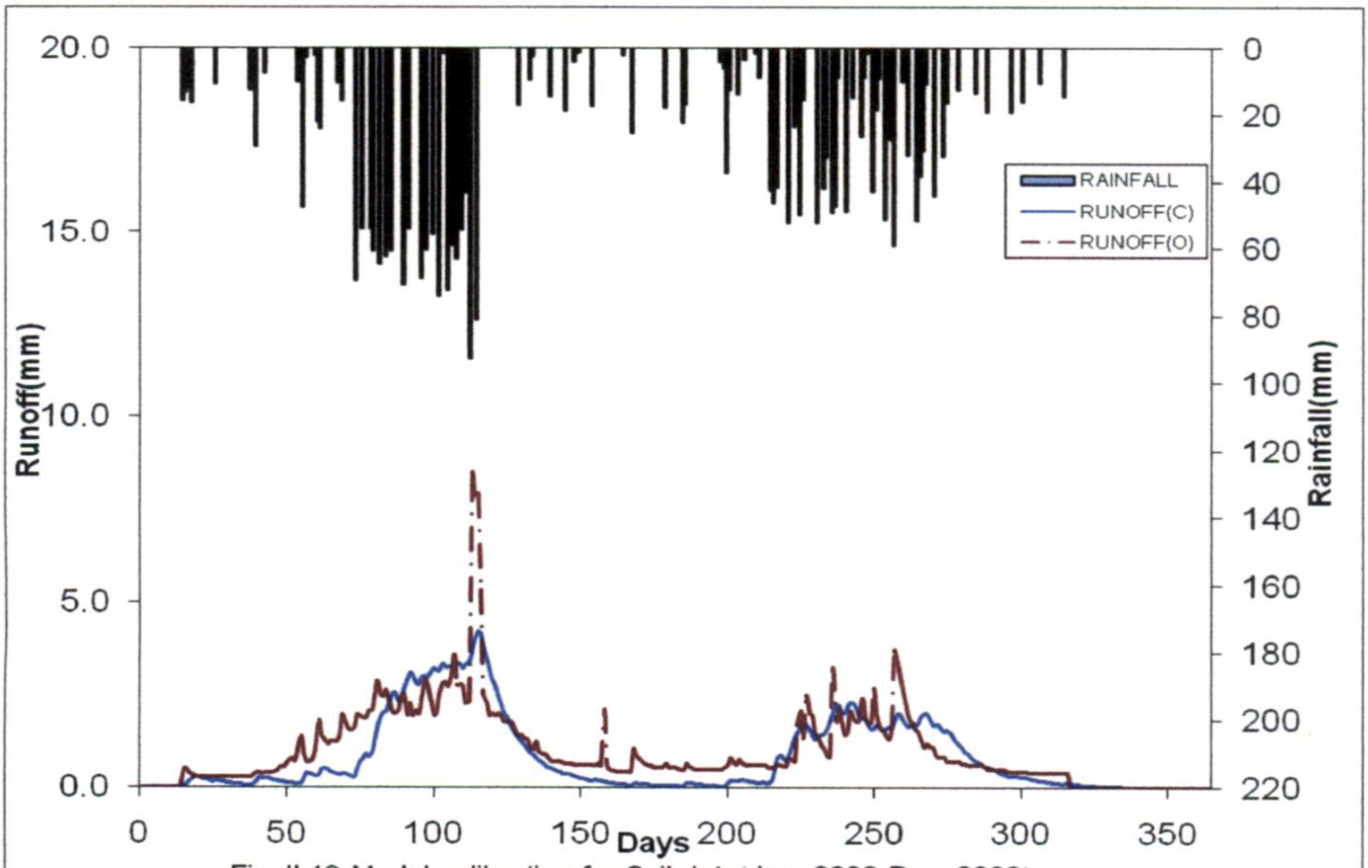


Fig. II.13 Model calibration for Suil data(Jan. 2002-Dec. 2002)

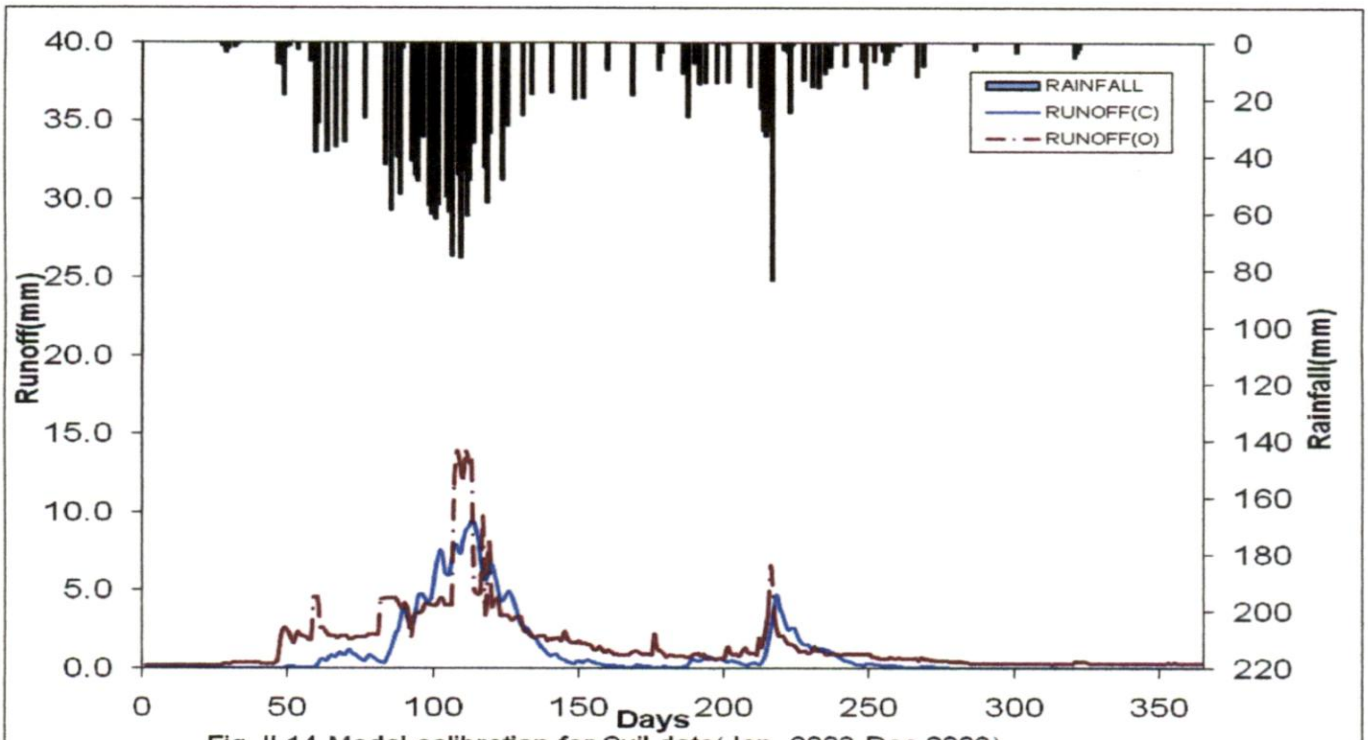


Fig. II.14 Model calibration for Suil data(Jan. 2003-Dec.2003)

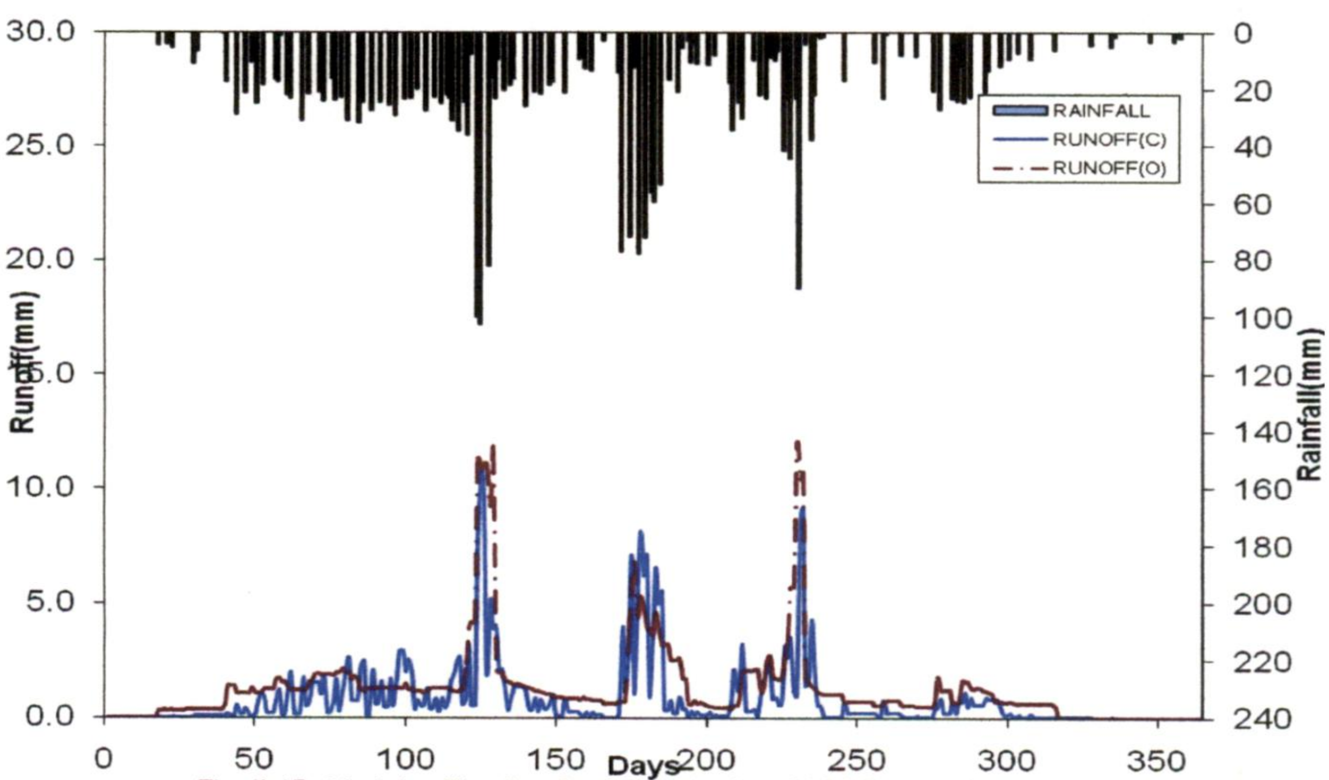


Fig. II.15 Model calibration for Suil data(Jan. 2004-Dec. 2004)

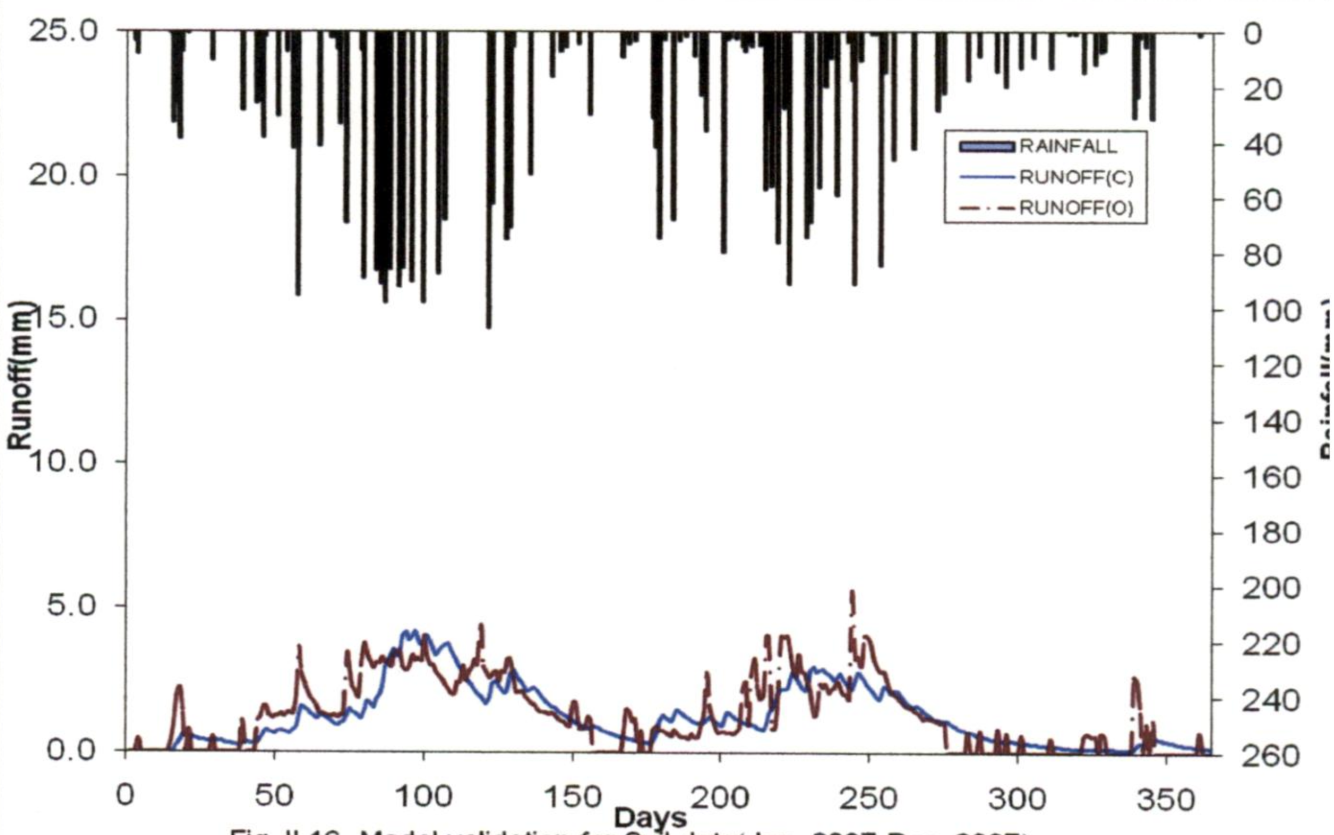


Fig. II.16 Model validation for Suil data(Jan. 2007-Dec. 2007)

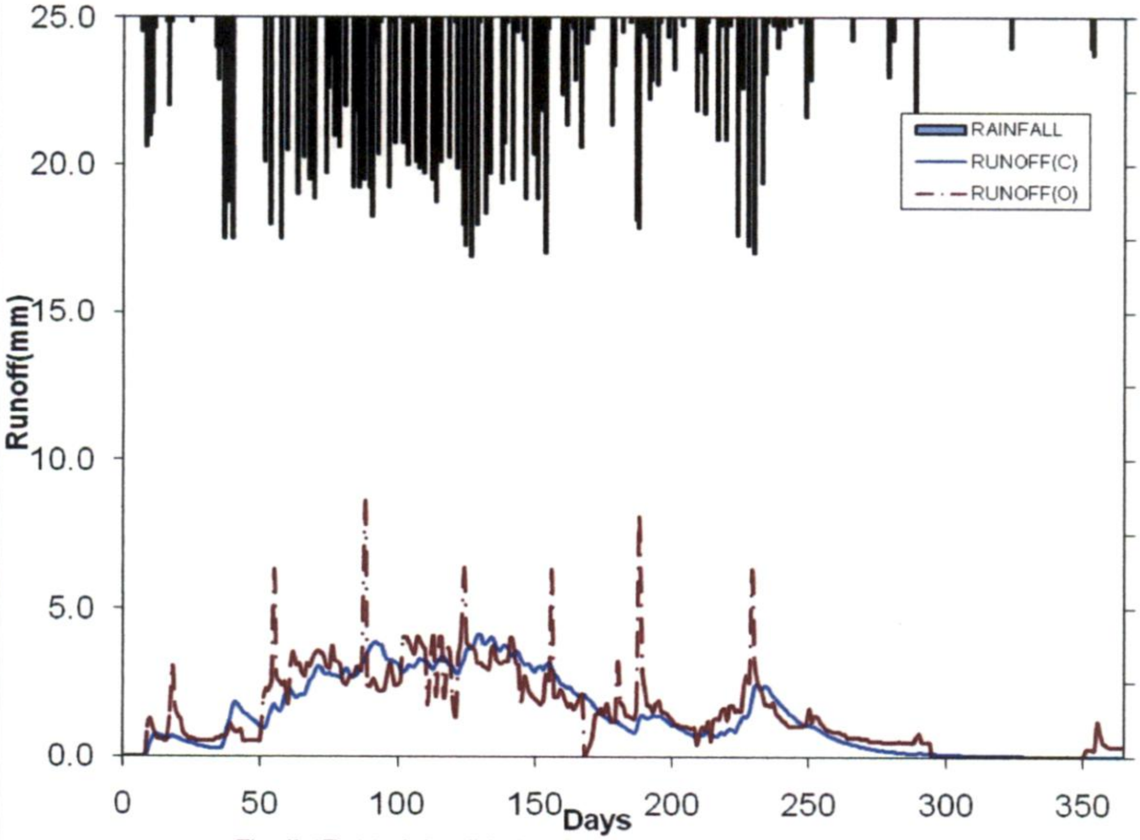


Fig. II.17 Model validation for Suil data(Jan. 2008-Dec. 2008)

APPENDIX III

SENSITIVITY FIGURES OF MODEL PARAMETERS FOR RET AND SIUL CATCHMENTS

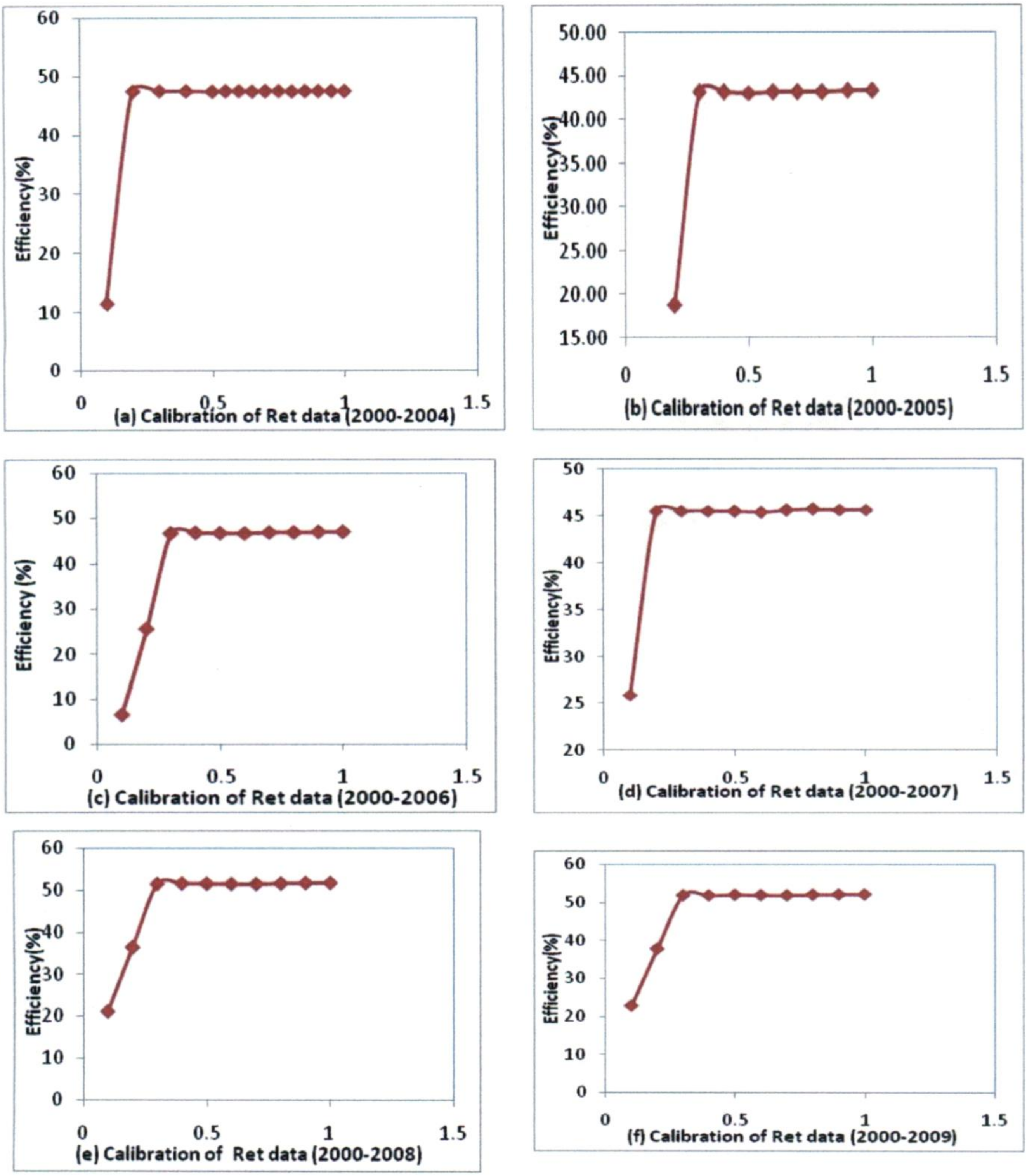
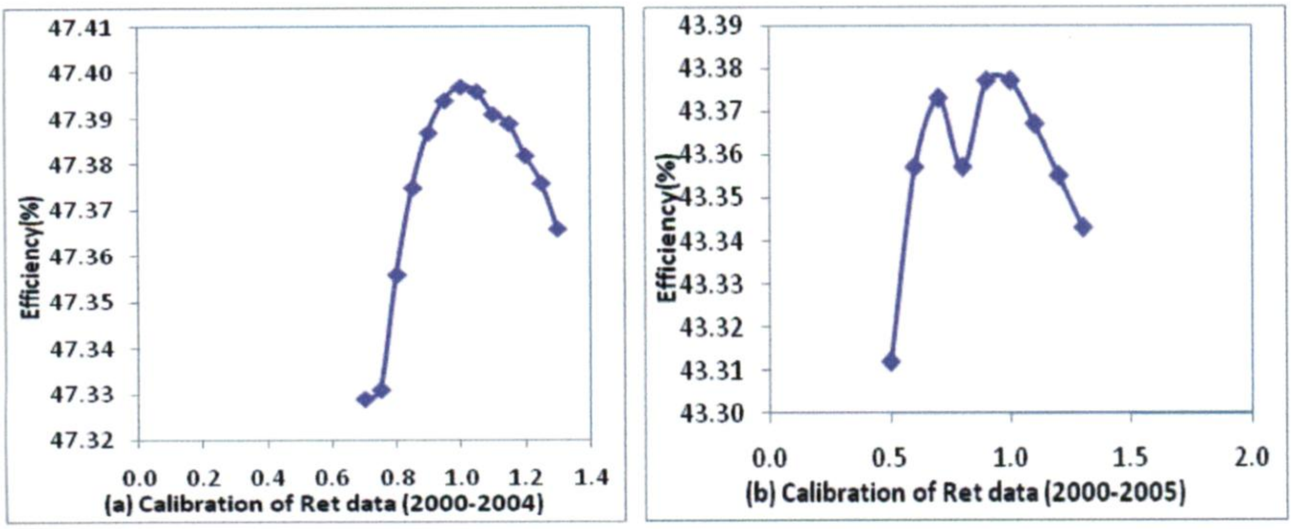
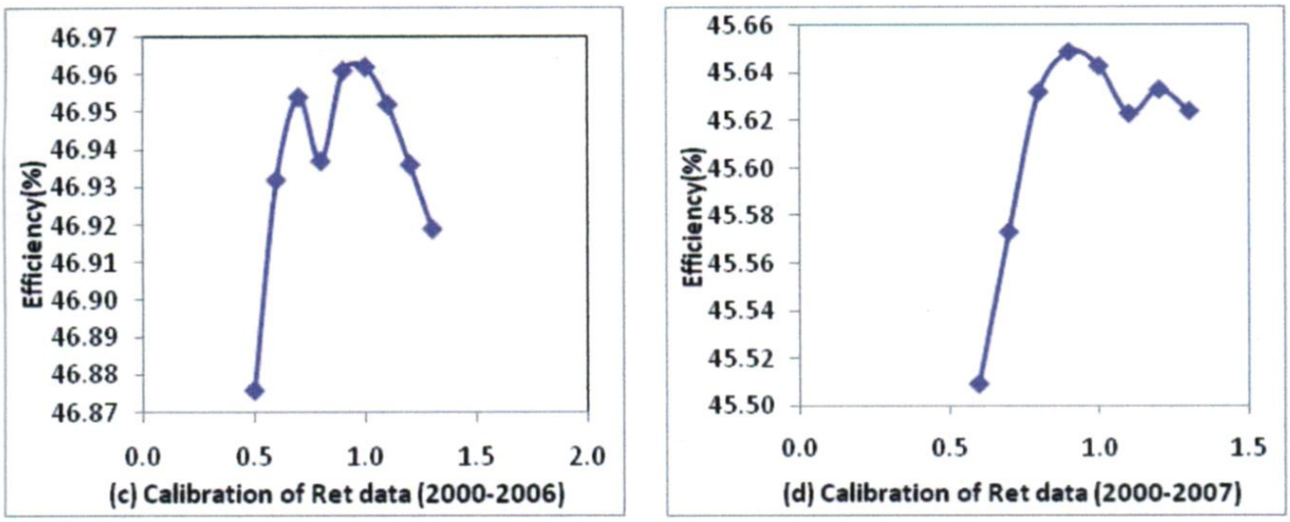


Fig.III.1 Sensitivity Parameter CN_0 for data of varying lengths



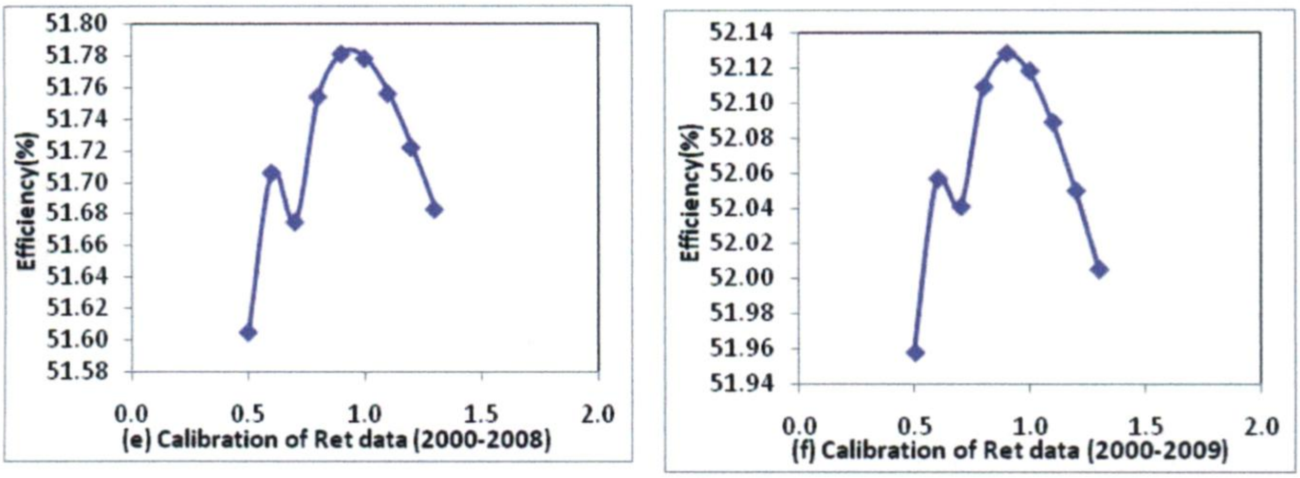
(a) Calibration of Ret data (2000-2004)

(b) Calibration of Ret data (2000-2005)



(c) Calibration of Ret data (2000-2006)

(d) Calibration of Ret data (2000-2007)



(e) Calibration of Ret data (2000-2008)

(f) Calibration of Ret data (2000-2009)

Fig. III.2 Sensitivity parameter K for data of varying lengths

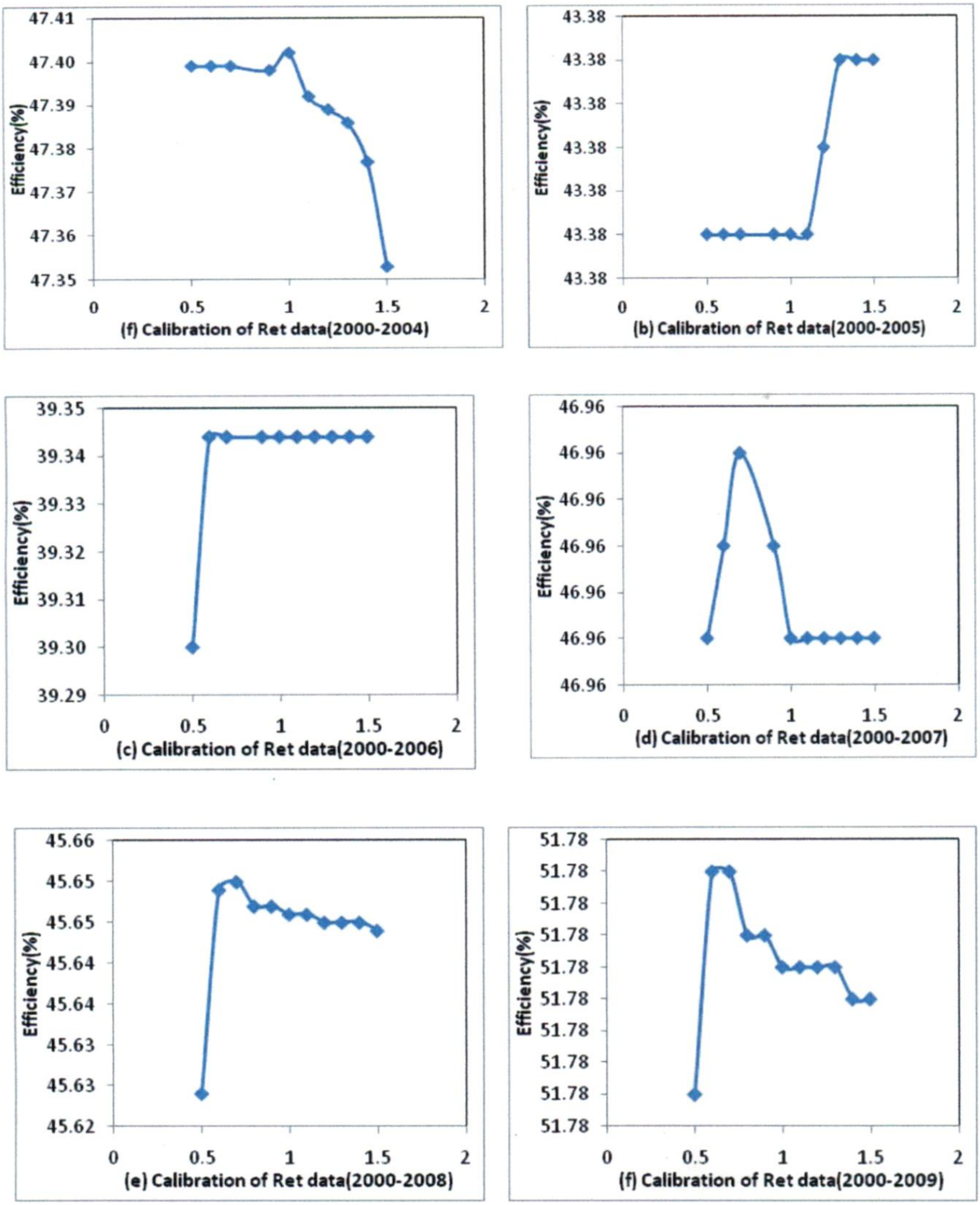
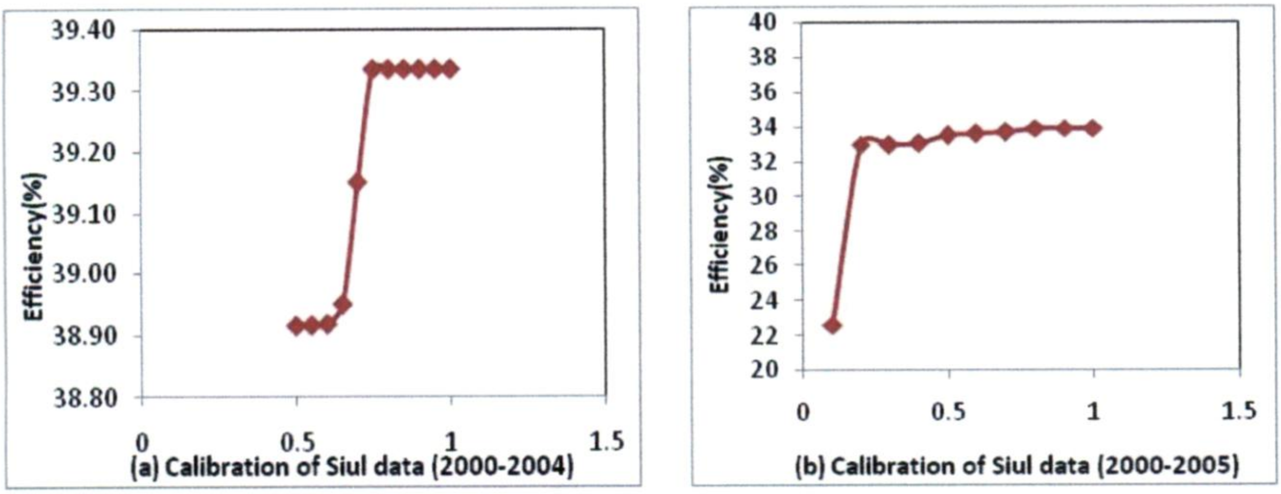
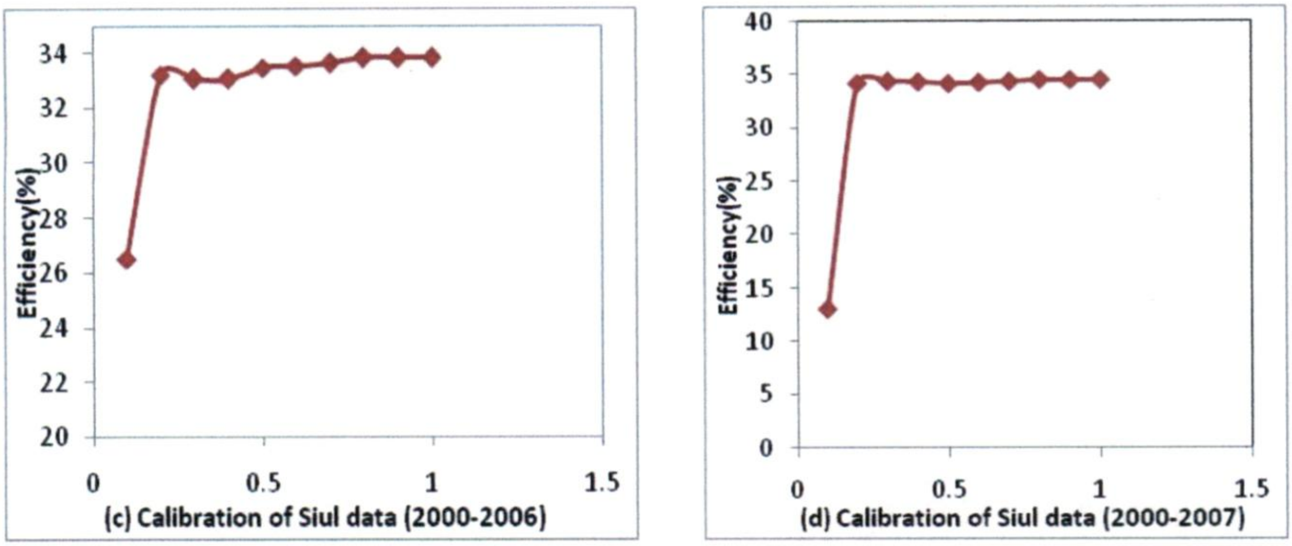


Fig.III.3 Sensitivity Parameter ALPHA for data of varying lengths



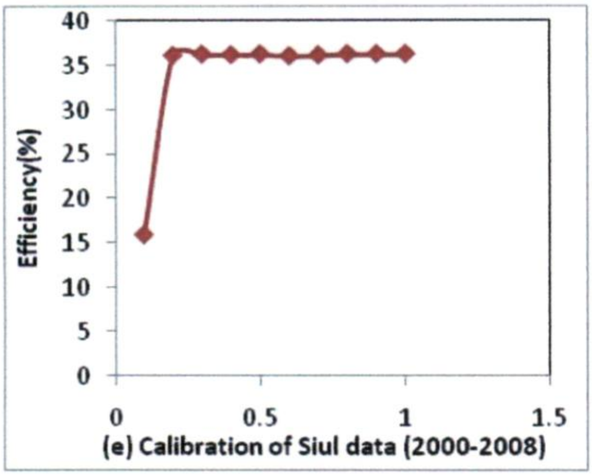
(a) Calibration of Siul data (2000-2004)

(b) Calibration of Siul data (2000-2005)



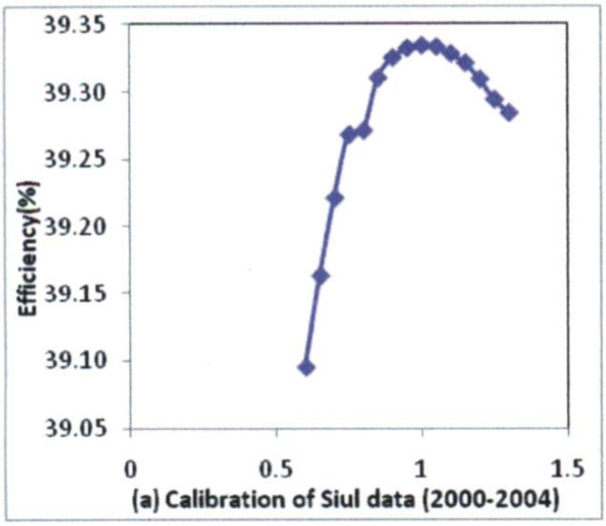
(c) Calibration of Siul data (2000-2006)

(d) Calibration of Siul data (2000-2007)

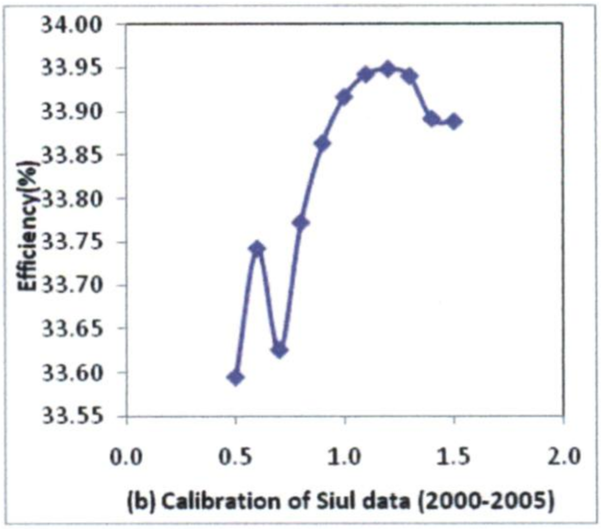


(e) Calibration of Siul data (2000-2008)

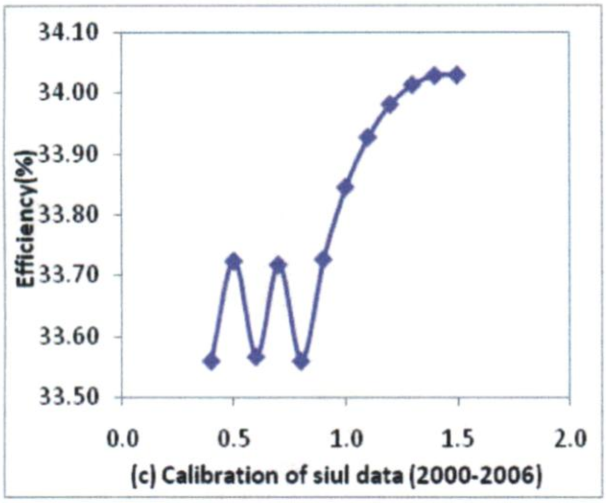
Fig.III.4 Sensitivity parameter CN_0 for data of varying lengths



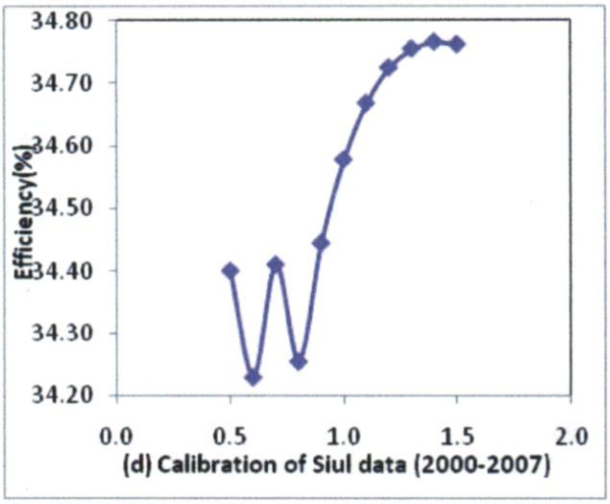
(a) Calibration of Siul data (2000-2004)



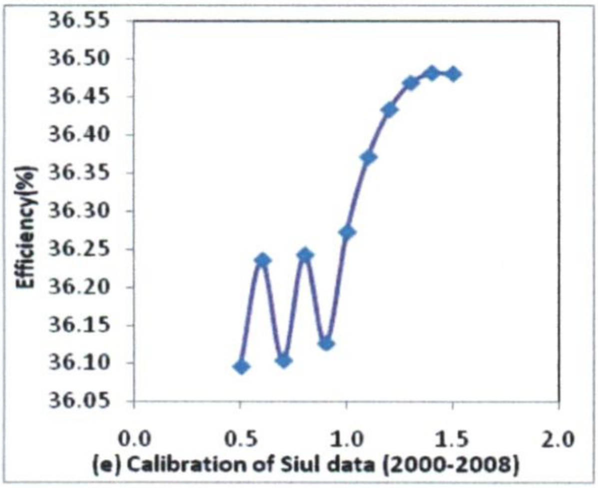
(b) Calibration of Siul data (2000-2005)



(c) Calibration of siul data (2000-2006)

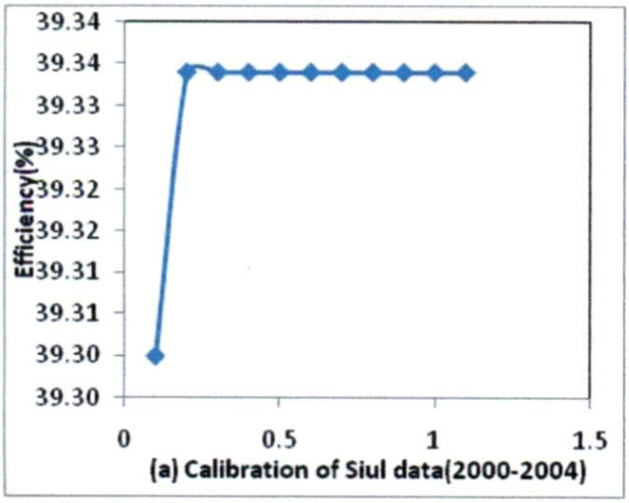


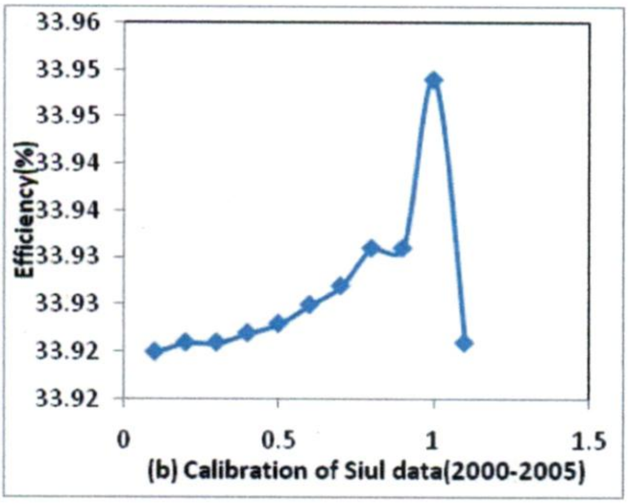
(d) Calibration of Siul data (2000-2007)

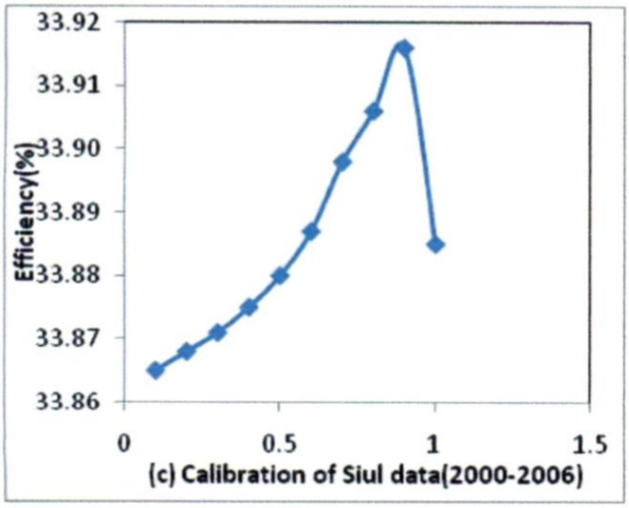


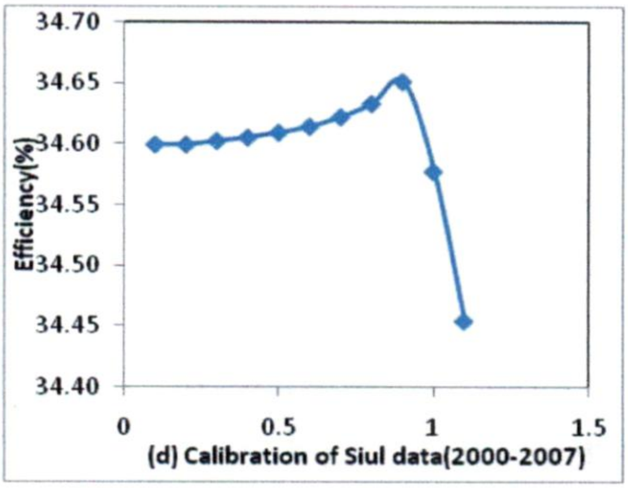
(e) Calibration of Siul data (2000-2008)

Fig. III.5 Sensitivity Parameter K for data of varying lengths









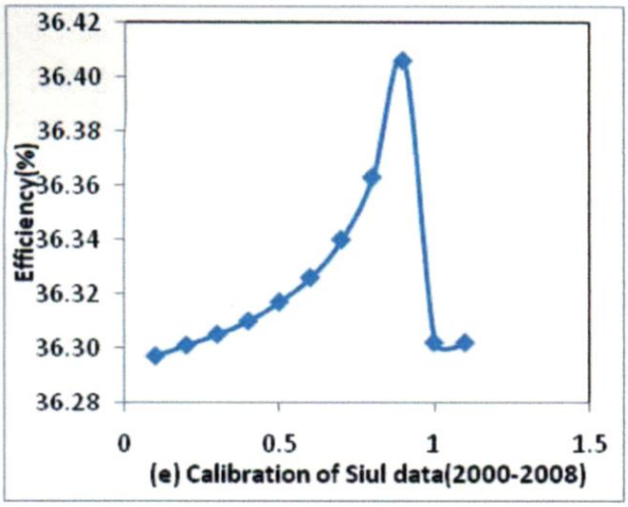


Fig. III.6 Sensitivity Parameter ALPHA for data of varying lengths.