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RELATIONSHIP BETWEEN FREQUENCY OF RAINFALL AND FREQUENCY OF FLOOD FOR A CATCHMENT OF UPPER NARMADA AND TAPI SUBZONE - 3 (C)

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PREFACE

Estimation of design flood is one of the important components of planning and design of any water resources project. Flood estimates of the various return periods are required for design of a variety of engineering works. The design flood is generally derived based on two types of approaches viz. (i) flood frequency analysis using the observed annual maximum peak flood data and (ii) the design storm approach using some rainfall runoff procedure.

Flood frequency analysis for those gauging sites, where the historical peak discharges are available for sufficiently long period, may be carried out using 'at-site data'. However, for the ungauged sites or sites with short record lengths, such analysis may not be able to provide consistent and reliable flood estimates. In such a situation, flood frequency analysis may be performed using regional approaches with 'regional and at-site data' or 'regional data' alone.

The design rainfall comprises of three components viz. design rainfall magnitude, its time distribution and areal pattern. In the design storm approach, it is presumed that the frequency of the flood peak is same as the frequency of the design rainfall.

For estimation of design flood, at a site where observed flood data are available, a choice must be made between some form of flood frequency analysis or one of the methods based on design rainfall. Flood frequency analysis gives a direct estimate of the flood of the desired return period, but rainfall records are generally longer than flow records, are less variable over time, are available at more locations, and have greater spatial consistency in the surrounding region.

While trying to estimate the design flood a matter of conjecture is the relative merits of frequency studies of observed floods versus use of design storm. Both the methods are in reality complementary and are not competitive. It is desirable that design flood is estimated directly from observed stream flow data wherever possible. The main advantage of the flood frequency approach is that it allows a direct estimate of the flood peak discharge of a given probability. In practice, this method could not be applied widely especially to small catchments because most of the streams and rivers generally happen to be ungauged.

In this study, an attempt has been made to study the relationship between frequency of rainfall and frequency of flood for a catchment of Upper Narmada and Tapi Subzone 3(c). The study has been carried out by Shri Rakesh Kumar, Scientist of the Institute.

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CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

Floods of various return periods have been estimated for the catchment defined by Bridge No. 253 of the Upper Narmada and Tapi Subzone 3(c) using various methods involving frequency analysis of rainfall and frequency analysis of annual maximum peak floods computed from the annual maximum excess rainfall of the design storm duration. The floods of various return periods have also been computed using the regional flood frequency analysis approach based on the observed annual maximum peak flood record for 13 gauging sites of the Subzone 3(c). Sensitivity analysis has also been conducted by increasing and decreasing the peak of the unit hydrograph, which has been used to convert the excess rainfall hyetographs into direct surface runoff hydrographs for identifying the peak values of floods.

The analysis carried out based on rainfall data shows that the floods are under estimated by 4.3% to 5.7% for the return periods of 2 to 200 years, by frequency analysis of floods as compared to the frequency analysis of rainfall. The rainfall data used in the study is of the limited record length of 19 years of one raingauge station only; hence the results of the study may be considered as indicative only, and detailed studies with long term record for a large number of catchments should be carried out for drawing more realistic conclusions.

The regional flood frequency methods used in the study, viz. SREV1, SRGEV and SRWAKE are based on `at site and regional data'; whereas, RGEV method is based on `regional data' alone. Flood estimates obtained by these methods show a deviation of -5.4% for SREV1, 0.4% for SRGEV, -1.4% for SRWAKE and 7.5% for RGEV methods for the return period of 50 years. The deviation varies from -3.1% to 13.5% for the return period of 100 years. For the return period of 200 years the deviation is -1.4% for SREV1, 11.3% for SRGEV, 2.8% for SRWAKE and 19.1% for RGEV method. Percentage deviations for flood estimates by the SRWAKE method, for the return periods of 50, 100 and 200 years are -1.4%, 1.2% and 2.8% respectively; which show that the flood frequency estimates obtained by SRWAKE method are very close to the flood frequency estimates obtained by the method based on frequency of rainfall [RAIN(CS)].

While conducting sensitivity analysis, when peak of the regional unit hydrograph is increased by 20%, it is observed that the flood estimates for the various return periods increase with respect to flood estimates computed by the respective methods, considered with the actual peak of the regional unit hydrograph by about 11.5% in case of the RAIN(CS) method, by about 6.5% for FLOD(CS) method, by about 14.5% for RAIN method and by about 10.5% for FLOD method. When peak of the regional unit hydrograph is decreased by 20%, it is observed that the flood estimates for the various return periods decrease with respect to flood estimates computed by the respective methods, considered with the actual peak of the regional unit hydrograph by about 14% in case of the RAIN(CS) method, by about 8.5% for FLOD(CS), by about 13% for RAIN method and by about 13% for FLOD method.

1.0 INTRODUCTION

Extreme rainfall events and the resulting floods can take thousands of lives and cause heavy losses in terms of property as well. Flood runoff results from short-duration highly intense rainfall, long-duration, low intensity rainfall, snowmelt, failure of a dam or levee systems, or combinations of these conditions. The best information on flood magnitudes that are likely to occur in the future can be obtained from observed flow records. The nature of flow producing system - the interaction of atmosphere, land geology and geomorphology, vegetation and soils, and the activities of people is so complex that sole use of theoretical or modelling approaches can provide only generalized estimates of flood regime of a stream or region.

Pilgrim and Cordery (1992) state that the choice of the method to be used is the first step in flood estimation. Unfortunately, the choice is made on a largely subjective and intuitive basis. While some subjectivity is always involved, the following considerations provide a sound basis for choice of a flood estimation method:

(i) The form and structure of available methods, the factors they consider, their theoretical basis, and their relative accuracies,

(ii) Whether a deterministic or probablistic estimate is required, and whether a particular method and its parameter values are suited to this application,

(iii) whether a method is capable of calibration with data recorded at the site or, if it is a regional method, whether it has been derived from data recorded in the region,

(iv) The type and importance of the work for which estimate is required, the effects of inaccuracy and exceedance of the estimate, and whether a peak flow or complete hydrograph is required.

(v) The time that that can be spent in estimating the flood,

(vi) The available expertize, as more complex methods generally require greater expertize in their use and interpretation, without which results may be poorer than for simpler methods.

For design at a site where observed flood data are available, a choice must be made between some form of flood frequency analysis or one of the methods based on design rainfall. Flood frequency analysis gives a direct estimate of the flood of the desired return period, but rainfall records are generally longer than flow records, are less variable over time, are available at more locations, and have greater spatial consistency in the surrounding region.

Very little quantitative guidance is available on the choice of flood estimation methods, and rather arbitrary rules are recommended for most of the regions. Bulletin 17 B of the Interagency Advisory Committee on Water Data (1982) of U.S.A. recommends that flood estimates from precipitation should be used only as an alternative method of estimating floods with exceedance probablity of 1 percent (i.e. 100 year flood) or less if the length of available streamflow record is less than 25 years. The U.S. Bureau of Reclamation (1981) gives similar recommendation. The U.K. Flood studies report (1975) recommends that a flood frequency curve should be extrapolated to a return period of 2N years only, where N years is the length of record. Beyond a return period of 4N years, a regional frequency curve is recommended upto a return period of 200 years, and even upto a return period of 500 years with lower accuracy.

Flood plain management and designs for flood control works, reservoirs, bridges and other investigations need to reflect the likelyhood or probablity of such events. Engineers and planners involved in the design of dams, spillways, river channel improvements, storm sewers, bridges, culverts etc. need information on flood magnitudes and their frequencies. Design flood estimation is a first and vital step in the design process for a large variety of water resources development works. It is ia hypothetical event that represents rare occurrence. It need not correspond with any specific event or time as it is essentially a maximum value which could be expected over a long period of time. However, for assigning a magnitude it could be expressed in terms of a probability or some return period. The design flood is generally derived based on two types of approaches viz. (i) flood frequency analysis using the observed annual maximum peak flood data and (ii) from the design storm using some rainfall runoff procedure. The design rainfall comprises of three components viz. design rainfall magnitude, its time distribution and areal pattern.

While trying to estimate the design flood a matter of conjecture is the relative merits of frequency studies of observed floods versus use of design storm. Both the methods are in reality complementary and are not competitive. It is desirable that design flood is estimated directly from observed stream flow data wherever possible. The main advantage of the flood frequency approach is that it allows a direct estimate of the flood peak discharge of a given probability. In practice, this method could not be applied widely especially to small catchments because most of the streams and rivers generally happen to be ungauged.

2.0 REVIEW OF LITERATURE

For estimation of floods of various return periods, approaches based on frequency analysis of peak floods and application of one of the methods based on design rainfall e.g. unit hydrogrraph techniques for converting the excess rainfall of desired frequency to the design direct surface runoff, or watershed modelling are adopted. The methodology for estimation of design storm is discussed in N.I.H. (1984-85). The various methods of design flood estimation and some of the studies relating to frequency of rainfall and frequency of floods as well as the criteria adopted for design flood estimation for various types of hydraulic/water resources structures are briefly reviewed below.

2.1 Methods of Flood Estimation

The following approaches may be used for estimation of design floods depending upon data availability, importance of the study and computation-facilities.

(i) Empirical Formulae and Envelope Curves
(ii) Rational Method
(iii) Flood Frequency Analysis
(iv) Unit Hydrograph Analysis
(v) Geomorphological Instantaneous Unit Hydrograph Approach, and
(vi) Watershed Modelling

2.1.1 Empirical formulae and envelope curves

Whenever, hydrological records are inadequate for flood frequency or unit hydrograph analysis the empirical formulae are only alternative approach for estimation of floods. Empirical formulae used for estimation of the flood peak are regional formulae based on statistical correlation of the observed peaks and important catchment characteristics. However, most of the formulae neglect the flood frequency as a parameter. The empirical formulae are usually based on data obtained for the larger streams because relatively few small streams are gauged in any region. Consequently, the empirical equations are usually applied in computing peak discharges for rivers having small catchment areas where stream flow data are inadequate. Some of the commonly used empirical formulae are Dicken's, Ryve's, Graig, Lillie, Inglis, Ali Nawaz Jung, formulae etc.

In regions having similar climatological characteristics, if the available flood data are scanty, the enveloping curve technique may be used to develop a relationship between the maximum flood flow and catchment area. This method is definitely better than the empirical formulae in the sense that it does not require the selection of coefficients on the basis of judgment as required in empirical formulae. The limitation of these curves lies in the fact that they are based on past records available up to the time such curves are drawn. Such curves, should, therefore be revised from time to time as more and more data become available.

2.1.2 Rational method

The Rational method is applied for estimation of peak floods for small catchments, normally less than 50 square kilometers. This method is based on the principle that if a rainfall of uniform intensity occurs over a catchment for a duration equal to or more than the time of concentration (T_c) of the catchment; then peak flood using this method is computed by multiplying the rainfall intensity for T_c hour duration by catchment area and the runoff coefficient which depends on the land use, soil types and antecedent moisture conditions etc. of the catchment. The runoff coefficients recommended for use in Rational method are given elsewhere (Chow, 1964). This method has been widely used to design the surface drainage systems. Its popularity may be attributed to its simplicity and limited data requirement; although reasonable care is necessary for selecting the runoff coefficient in order to use the method correctly.

2.1.3 Flood frequency analysis

Flood frequency analysis for those gauging sites, where the historical peak discharges are available for sufficiently long period, may be carried out using at-site data. For at-site flood frequency analysis, generally various theoretical frequency distributions are fitted to historical flood records. The parameters of the distributions are estimated using one or more parameter estimation techniques. The best fit distribution is selected on the basis of some goodness of fit criteria. The floods of different return periods are computed using the estimated parameters of the best fit distribution. However, for the ungauged sites or sites with short record lengths, such analysis may not be able to provide consistent and reliable flood estimates. In such a situation, flood frequency analysis may be performed using regional approaches with 'regional and at-site data' or 'regional data' alone. Farquharson et al.(1992) assembled GEV (PWM) based regional flood frequency curves for a number of semi-arid and arid areas of some parts of the world.

Various issues involved in regional flood frequency analysis are testing regional homogeneity, development of frequency curves and derivation of relationship between mean annual peak flood (MAF) and the catchment characteristics. Some of the comparative studies have been conducted by Kuczera (1983), Gries and Wood (1983), Lettenmaier and potter (1985) and Singh (1989). A procedure for estimating flood magnitudes for return period of T years Q_T is robust if it yields estimates of Q_T which are good (low bias and high efficiency) even if the procedure is based on an assumption which is not true (Cunnane, 1989).

Naghavi and Yu(1995) carried out regional frequency analysis if precipitation in Lousiana. A total of 92 raingauges were used to generate 25 synthesized stations with long periods of records. Annual maximum series of rainfall durations of 1, 3, 6, 12 and 24 hour from the 25 synthesized stations were used for various statistical analyses. The mean annual precipitation, geographical locations, and synoptic generating mechanisms were used to identify the three climatological homogeneous in Lousiana. Using the L-moment ratios, the underlying regional probablity distribution was identified to be the generalized extreme value (GEV) distribution. The regional parameters of the GEV distribution were estimated by the indexed probablity weighted moments (PWM). The regional analysis was tested by Monte Carlo simulation. Relative root mean square error and relative bias were computed and compared with those resulting from at-site Monte Carlo simulation. The results show that the regional procedure can substantially reduce the relative root mean square error and relative bias in quantile prediction.

Kumar et al. (1996) developed regional flood frequency curves by fitting the probability weighted moment(PWM) based General Extreme Value(GEV) distribution to the station-year data of annual maximum peak floods of various small catchments of Mahanadi Subzone-3(d). A relationship between mean annual peak floods and catchment area is also developed for the subzone. This relationship is coupled with the developed regional flood frequency curves for derivation of the regional flood formula for the subzone.

2.1.4 Unit hydrograph analysis

Whenever adequate and reliable records on stream flow and rainfall are available for any catchment, the unit hydrograph can be derived from the rainfall-runoff data of storm events. However, most of the small catchments are generally not gauged and many water resources projects are being planned in these catchments. Therefore, it becomes necessary to have estimates of floods at the proposed sites in small ungauged catchments or the catchments with limited data. The unit hydrographs for such catchments have to be derived by using data on climatological, physiographical and other factors of these catchments. This approach for unit hydrograph derivation is popularly known as regional unit hydrograph technique. The procedure involved in developing the regional unit hydrograph requires evaluation of representative unit hydrograph parameters and pertinent physiographic characteristics for the gauged catchments in the region. Then multiple linear regression is performed considering the unit hydrograph parameters one at a time as a dependent variable and various catchment characteristics as independent variables in order to develop the regional relationships for the unit hydrograph. Further, knowing the catchment characteristics for an ungauged catchment in the region from the available toposheets the unit hydrograph for the ungauged catchment can be derived using the relationships developed between unit hydrograph parameters and catchment characteristics for the region.

Small Catchment Directorate of Central Water Commission (C.W.C.), Research Designs and Standards Organizations(R.D.S.O.), Roads Wing of Ministry of Transport and Indaia Meteorological Deaprtment(I.M.D.) have jointly carried out regional unit hydrograph studies for various Indian basins. Specific regions have been identified by dividing the whole of India into 26 hydrometeorological homogeneous sub-zones. Regional unit hydrograph relationships have been developed relating the various unit hydrograph parameters of the gauged catchments with their pertinent physiographic characteristics. Apart from these, various regional unit hydrograph relationships have been developed for some of the regions in India relating the parameters of some well known instantaneous unit hydrograph models such as Nash and Clark models etc. (e.g. Singh and Kumar, 1991).

2.1.5 Geomorphological instantaneous unit hydrograph approach

The concept of Geomorphological Instantaneous Unit Hydrograph (G.I.U.H.) has been introduced in the literature (Rodriguez et al., 1979) for the derivation of unit hydrograph for the ungauged catchments. The geomorphologic approach has many advantages over the regionalization techniques as it avoids the requirement of data and computations for the neighbouring gauged catchments in the region. A hybrid approach by integrating the Clark model and the G.I.U.H. approach for estimation of Clark model parameters using the geaomorphological characteristics and storm pattern has been developed and discussed in N.I.H. (1993-94). This approach avoids the use of extensive rainfall runoff records, which are often not available for calibration of Clark model parameters. The developed approach has been applied for simulation of historical events of Kolar-sub basin of river Narmada.

2.1.6 Watershed modelling

With the advent of high speed computers and improvements in hydrological data base, the mathematical modelling of the hydrological processes becomes an useful tool for accurate estimation of the water resources in space and time. These models can estimate the flood with reasonable accuracy provided the required input data are available for their applications. Many types of models have been developed for estimation of flood hydrographs from excess rainfall. The characteristic that distinguishes these models from unit hydrographs and other transfer function procedures is that they attempt to represent the runoff processes in more detail. The hydraulics of runoff, the routing of runoff through temporary storage within the drainage basin, and the arrangement or topology of the stream network. Computer programmes are available for most of models and are required for practical application. Models that fall into this category include the Corps of Engineers HEC-1 model, the Soil Conservation Service TR-20 model, and similar models. Calculations proceed from upstream to downstream in the basin, and the general modeling sequence is the following.

(i) Subbasin average precipitation,
(ii) Determination of precipitation excess from time-varying losses,
(iii) Generation of the direct surface runoff hydrograph from precipitation excess,
(iv) Addition of a simplified base flow to the surface runoff hydrograph,
(v) Routing of stream flow,
(vi) Reservoir routing
(vii) Combination of hydrographs

In these models, the primary interest is the flood hydrograph, so it is not necessary to calculate evapotranspiration, soil moisture changes during and between storms, or detailed base flow processes.

HEC-1 is a computer model for rainfall-runoff analysis developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. This program develops discharge hydrographs for either historical or hypothetical events for one or more locations in a basin. The basin can be subdivided into many subbasins. Uncontrolled reservoirs and diversions can also be accommodated. The available program options include: calibration of unit hydrograph and loss rate parameters, calibration of routing parameters, generation of hypothetical storm data, simulation of snowpack processes and snowmelt runoff, dam safety applications multiplan/multiflood analysis, flood damage analysis and optimization of flood control system components.

The U.S. Soil Conservation Service TR-20 computer model is a single event rainfall runoff model that is normally used with a design storm as rainfall input. The program computes runoff hydrographs, routes flows through channel reaches and reservoirs, and combines hydrographs at confluences of the watershed stream system. Runoff hydrographs are computed by using the SCS runoff equation and the SCS dimensionless unit hydrograph. Computed flows are routed through channel reaches and reservoirs.

Although, in India, there has been considerable improvement in the data network, their collection and management; even then the data base is usually not adequate for the application of the complex hydrological models for design flood estimation. Thus the conventional methods are still being used for such applications.

Fontaine (1995) states that relatively little is known about the accuracy of conceptual rainfall-runoff model simulations of extreme floods. The author carried a case study to evaluate the accuracy of runoff model simulations of the 100 year flood on the Kickapoo river in sothwest Wisconsin. The accuracy of a simple and quick analysis is compared to that of an eloborate, labor-intensive anasysis. It has been stated by the authors that the more eloborate modelling approach produces more accurate results, although significant errors for the peak discharge and runoff volume are observed in both the approaches. The potential sources of uncertainty in the results are evaluated. Error in the precipitation data used for calibrating the model appears to be the primary source of uncertainty.

The approach of flood estimation using design rainfall has some advantages over the frequency analysis of observed floods. The different parameters affecting the flood runoff could be considered in a more realistic and explicit way and the catchment characteristics of different sub basins contributing to the flood flow in the main river could be determined more thoroughly and added appropriately. The necessary parameters (unit hydrograph and routing) could be estimated even from a short length of record and the parameters thus derived could be extended to the other ungauged subbasins. The design storm approach also allows for maintenance of consistency in a given geographical area.

Rainfall frequency studies are more advantageous than flood frequency studies because longer records of precipitation are generally available at a larger number of rain gauges more so in case of daily rainfall. Extreme rainfall values are more easily defined from physical consideration.

A number of limitations were noticed in prcatice in spite of the wide spread and continued use of the design storm approach by design engineers. These relate to almost all aspects of the design storm strarting from the approach and risk criteria to the time distribution and others. While some related to inadequacy, others were regarding the inconsistency and inapropriateness of the method.

Eagleson (1978) represented point precipitation by Poisson arrivals of rectangular intensity pulse that have random depth and duration. By assuming the storm depths to be independent and identically gamma distributed, the cumulative distributin function for normalized annual precipitation is derived in terms of two parameters of storm sequence, the mean number of storms per year and the order of the gamma distribution. In comparison with long-term observations in subhumid and an arid climate it is demonstrated that when working with only 5 years of storm observations this method tends to improve the estimate of the variance of the distribution of the normalized values over that obtained by conventional hydrologic methods which utilize only the observed totals.

The major criticism of the design rainfall approach is that in the process of deriving design flood from design storm a series of steps are involved which would introduce some error and, therefore, may not provide the expected results. Thus, a design rainfall of a given frequency might not produce flood of the desired frequency. Besides, some of the limitations of fitting a frequency distribution to the flood data apply equally well to the extreme rainfall values too.

Studies carried out by Bell (1968), Larson and Reich (1972) and Niemczynowicz (1982) using concurrent data of storm events and associated rainfall have shown a wide scatter of the recurrence intervals of rainfall versus recurrence intervals of corresponding peak flows. As shown in Fig. 1, although the scatter is very broad, it may be seen that approximately the same number of points fall on each side of the 45 degree line for the full range of values, indicating that, on the average, the probability of a particular design rainfall and the associated floods would be the same. The average 100 year flood for example, corresponds with the average 100 year rainfall for the watersheds considered (as quoted in N.I.H., 1984-85)

2.2 Hydrologic Design Criteria

Design criteria refers to standards and practices laid down for judging whether a project has been properly designed to deliver the anticipated outputs. If different criteria are adopted, different engineering decisions may result. The need for criteria and standardisation arises whenever choices between alternatives are to be made in a systematic and scientific manner. Further, the areas of activity which address the problems involved with complexities of nature, have to necessarily depend on experience and judgment, and thus need adequate standards and criteria to guide the practicing engineers and decision makers. The design criteria for some of the hydraulic/water resources structures as mentioned by Sharma(1991) are briefly summarized in Appendix-I.

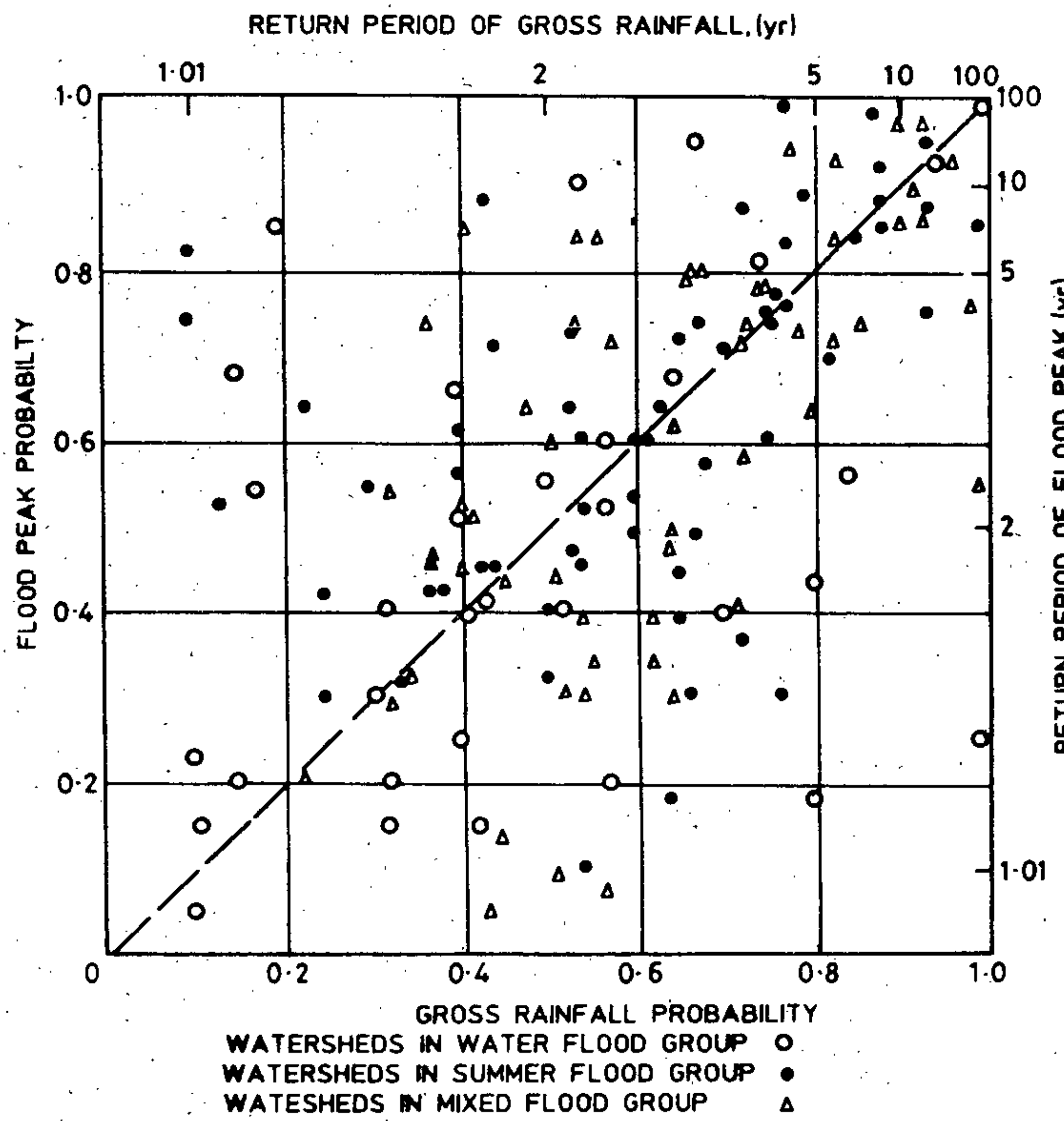


FIG. 1 PROBABILITY OF RAINFALL AND PROBABILITY OF ASSOCIATED FLOOD PEAK (REPRODUCED FROM BELL, 1968)

3.0 STATEMENT OF THE PROBLEM

Estimation of design flood is one of the important components of planning and design of any water resources project. Flood estimates of the various return period are required for design of a variety of engineering works, as indicated by the design criteria for different types of hydraulic structures given in Appendix-I.

For the design of many types of hydraulic structures different design criteria have been evolved. For example, design floods of return periods such as 50 year, 100 years, 200 years return period may be computed by the flood frequency approach for the design of small bridges, culverts and other hydrailic structures.

For estimation of floods of various return periods, two types of approaches viz. (i) frequency analysis of peak floods and (ii) application of one of the methods based on design rainfall e.g. unit hydrogrraph techniques for converting the excess rainfall of desired frequency to the design direct surface runoff, or watershed modelling are adopted. In the second approach it is presumed that the frequency of the flood peak is same as the frequency of the design rainfall.

The objectives of this study are:

(a) Computation of excess rainfall of various return periods for the design storm duration, and estimation of floods of the different return periods using the above rainfall and the unit hydrograph.

(b) Computation of excess rainfall for design storm duration for each year of rainfall record; estimation of annual maximum peak floods for each year of rainfall record using the unit hydrograph, and carrying flood frequency analysis using the above computed series of annual maximum peak floods.

(c) Carrying out regional flood frequency analysis using the available record of annual maximum peak floods.

(d) Comparison of flood estimates of various return periods obtained above viz. (a), (b) and (c).

(e) Examining the effect of sensitivity analysis of unit hydrograph used for converting excess rainfall into flood.

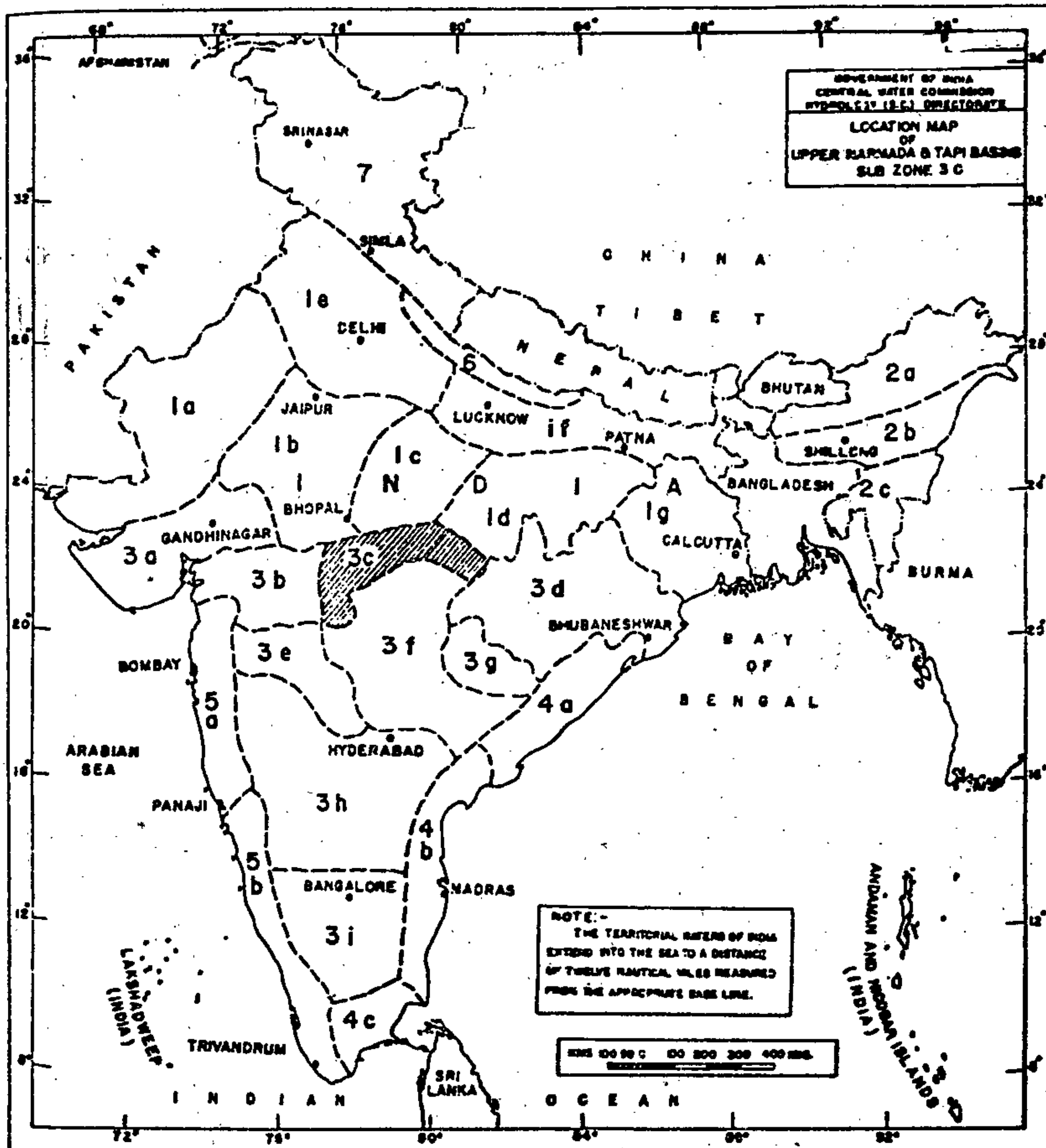
4.0 DESCRIPTION OF STUDY AREA

The catchment defined by the Bridge No. 253 lies in the hydrometeorologically homogeneous region of Upper Narmada and Tapi Subzone-3(c). Its catchment area is about 114.22 square kilometers. The description of the study area is given below.

The Narmada is a west flowing river which rises near Amarkantak in the Mailkala range in the Shahdol district of Madhya Pradesh at an elevation of about 1000 metres above sea level. It flows for a length of about 1300 kilometers before it outfalls into the gulf of Cambay in the Arabian Sea. The river Tapi rises near Multai in the Betwa district of Madhya Pradesh and like Narmada it flows westward for a length of about 725 Km. before outfalling into the gulf of Cambay. The Subzone-3(c) comprises of upper portion of Narmada and Tapi basins combined and constitutes about 50% of the entire area of the combined Narmada and Tapi basins. The lengths of main Narmada and Tapi rivers in the upper sub-zone are 813 km. and 229 km. respectively. The important tributaries of Upper Narmada are Burhnar, Banjar, Sher, Shakkar, Dudha, Tawa, Ganjal and Chhota Tawa along left bank and Hiran, Tendori, Barna, Kolar, Jamner and Datuni along right bank. Purna is the main tributary of Tapi, its upper part falls in the upper Subzone-3(c).

The Upper Narmada and Tapi Subzone-3(c) extends over an area of about 86,353 square kilometers and lies between east longitudes 76° 12′ to 81° 45′ and north latitudes of 20° 10′ to 23° 45′, lying in the northern extremity of the Deccan plateau. Location of the subzone 3(c) is shown in Fig. 2. This Subzone extends over the states of Madhya Pradesh and Maharashtra. Important cities and towns located in this Subzone are Mandla, Jabalpur, Narsinghpur, Itarsi, Betul, Hoshangabad, Akola and Amravati. Index map of the upper Narmada and Tapi subzone-3(c) given in Fig. 3 shows locations of bridge sites. The Upper Narmada and Tapi subzone-3(c) has a complex relief. High ranges of above 900 m exist over a small area near the source of Narmada river at Amarkantak. Areas varying in height between 600 m to 900 m lie along the eastern and middle portions of the boundary. About 60 percent of the sub-zone varies in height from 300 m to 600 m. Areas varying in height from 150 m to 300 m lie in patches near the western boundary.

The subzone has a continental type of climate. It is very hot in summer and cold in winter. It receives most of the rainfall from the south west monsoon during June to October. Mean annual rainfall of the subzone varies approximately from 800 to 1600 mm. About 50% of the subzone on eastern side is having mean annual temperature of 22.5° C to 25° C, while the western side is having mean annual



THE EXTERNAL BOUNDARY AND COAST LINE OF INDIA ON THE MAPS AGREE WITH THE RECORD COPY CERTIFIED BY THE SURVEY OF INDIA.

FIG. 2 LOCATION MAP OF UPPER NARMADA AND TAPI SUBZONE 3C

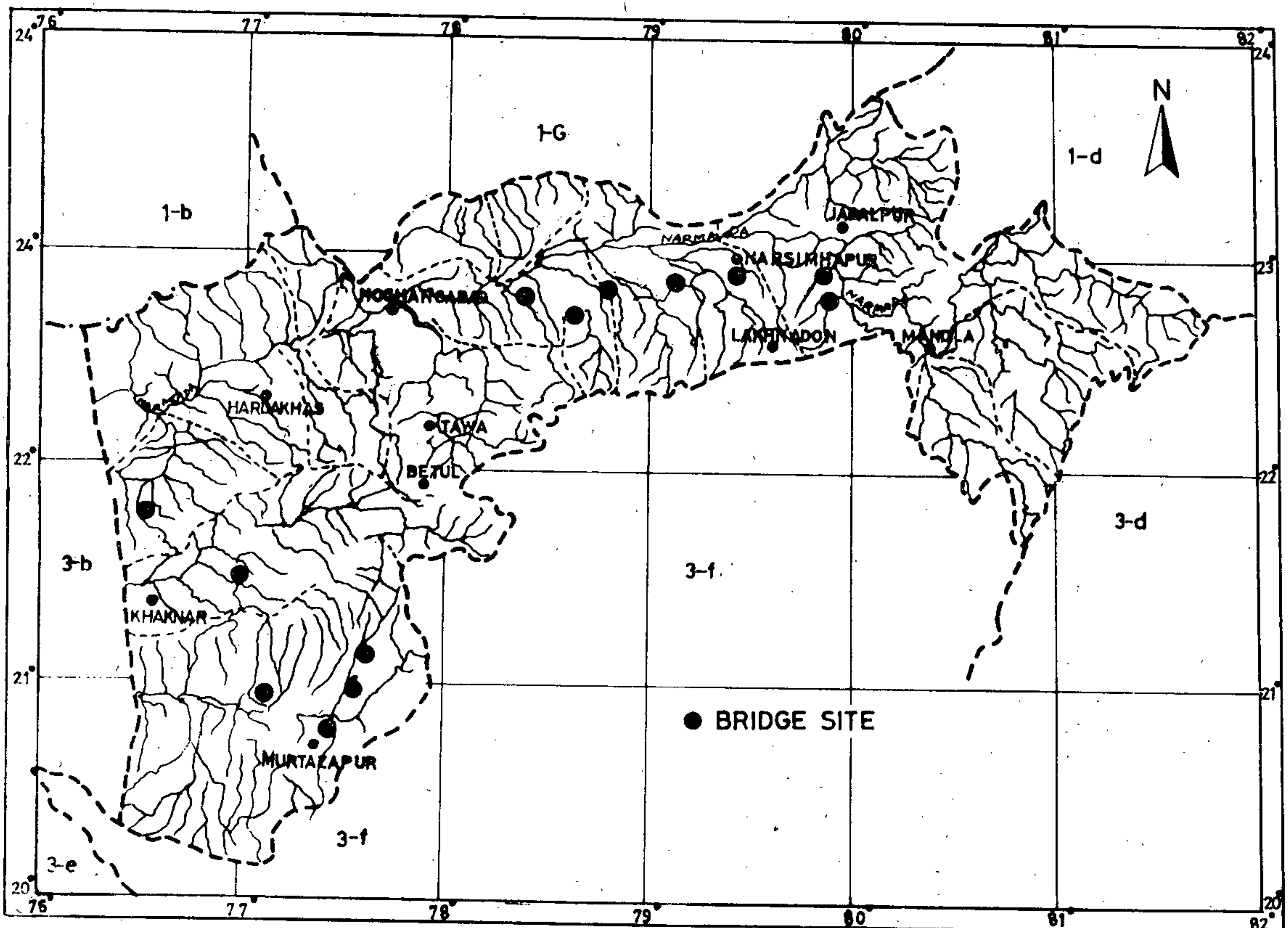


FIG.3 - INDEX MAP OF UPPER NARMADA AND TAPI SUB ZONE 3(c)

temperature of 25° C to 27° C except over a pocket in southern side where temperature is of the order of 25° C to 27° C. The maximum temperature has been recorded in the month of May and minimum temperature has been recorded in the month of December.

The main soil group of the subzone is black soil comprising of different varieties viz. deep black soil, medium black soil and shallow black soil. In addition, mixed red and black soil, red and yellow soil and skeletal soil are also observed in pockets. Of these, deep black soil covers the major portion of the subzone. At micro level (i.e. when small and medium catchments are considered), the soil type may vary considerably from the above indicated group. The subzone is having extensive area of about 55% under arable land, 40% of area under forest and remaining under wasteland, grassland etc.

5.0 DATA AVAILABILITY FOR THE STUDY

The description of rainfall and annual maximum peak flood data used in the study is given below.

(a) Hourly rainfall data of Jabalpur self recording raingauge have been used. Nineteen years of data covering the period 1961 to 1989 were available for the study. The Jabalpur self recording raingauge lies outside the catchment defined by the Bridge No. 253. The regional unit hydrograph for Bridge No. 253, available in the Report No. UNT/7/1983 of CWC(1983) has been used.

(b) The annual maximu peak flood data collected by Indian railways for 13 bridge sites have been used in this study (R.D.S.O., 1991). The record lengths vary from 14 to 30 years over the period of 1957 to 1990. The catchment areas of these bridge sites vary from 41.80 square kilometers to 2110.85 square kilometers. The observed annual mean peak floods vary from 111.95 cumec to 1730.53 cumec. (Details of catchment area, sample statistics and sample size are given in Table 3)

6.0 METHODOLOGY

The various frequency distributions used in the study along with the methodology adopted are described here under.

6.1 Frequency Distributions Used

Methods used in the study to carry out flood frequency analysis involved fitting of Extreme Value Type I(EV1), General Extreme Value (GEV) and Wakeby (WAKE) distributions, which are briefly discussed here under.

6.1.1 Extreme value type-I distribution (EV1)

This is a two parameter distribution and it is popularly known as Gumbel Distribution. The cummulative density function for EV1 distribution is given by:

$$F(x) = e^{-e^{\frac{-(x-u)}{\alpha}}} \tag{1}$$

where, F(x) is the probability of nonexceedence and equal to 1-1/T ; T is the recurrence interval in years, u and α are the location and shape parameters respectively. These parameters can be estimated from the sample of annual maximum peak floods using the parameters estimation techiniques available in literature. Method of probability weighted moments (PWM) is one of the parameter estimation techniques which has been successfully applied by Landerwehr et al.(1979) for estimating the parametres of EV1 distribution more efficiently with less bias. The method of probability weighted moments which has been discussed in subsquent section was, therefore, used for estimating the EV1 distribution parameters, in case of the regional flood frequency methods. Whereas, the parameters in case of the analysis carried out using rainfall data have been estimated using the method of moments.

6.1.2 General extreme value distribution(GEV)

GEV distribution is a generalised three parameter extreme value distribution proposed by Jenkinson (1955). Its theory and practical applications are reviewed in the Flood Studies (NERC,1975). The cummulative density function F(x) for GEV distribution is expressed as:

$$F(x) = e^{-(1-k\frac{(x-u)}{\alpha})^{1/k}} \quad (2)$$

where u, α and K are location, scale and shape parameters of GEV distribution respectively. For estimating these parameters, a procedure based on method of probability weighted moments has been used.

6.1.3 Wakeby Distribution

A random variable x is said to be distributed as Wakeby if:

$$x = m+a\,[1-(1-F)^{b}\,] - c\,[1-(1-F)^{-d}\,] \quad (3)$$

where F = F(x) = 1-1/T, and a, b, c, d and m are the parameters of Wakeby distribution which can be estimated using a special algorithm proposed by Landwehr et al.(1979) based on method of probability weighted moments.

6.2 Analysis Using Rainfall Data

Following four methods have been considered for analysis using rainfall data.

(i) Estimation of floods by frequency analysis of rainfall [RAIN(CS)]

(ii) Flood frequency analysis using annual maximum peak flood series computed from excess rainfall [FLOD(CS)]

(iii) Estimation of floods by frequency analysis of rainfall (RAIN)

(iv) Flood frequency analysis using annual maximum peak flood series computed from excess rainfall (FLOD)

6.2.1 Estimation of floods by frequency analysis of rainfall [RAIN(CS)]

The consecutive hourly annual maximum rainfall values for the design storm duration are identified for each year. The hourly rainfall values are multiplied by the areal reduction factor for converting the point rainfall values into areal rainfall. The constant loss rate as applicable for the study area is deducted from hourly rainfall increments for computing the excess rainfall hyetograph.

Frequency analysis is carried out using the sum of annual maximum excess rainfall values of design storm duration and rainfall of various return periods is computed using the Extreme Value Type -I (EV1) distribution. The sum of excess rainfall values of various return periods are distributed into hourly excess rainfall values by using the average time distribution curve. The critical sequencing of these hourly excess rainfall values is carried out using the approach described elsewhere (N.I.H., 1988). These critically sequenced excess rainfall values are convoluted with the regional unit hydrograph for estimation of direct surface runoff, and the direct surface runoff hydrographs for various return periods are computed. The values of peak direct surface runoff for different return periods are identified from the respective direct surface runoff hydrographs.

6.2.2 Flood frequency analysis using annual maximum peak flood series computed from excess rainfall [FLOD(CS)]

The consecutive hourly annual maximum rainfall values for the design storm duration are identified for each year. The hourly rainfall values are multiplied by the areal reduction factor for converting the point rainfall values into areal rainfall. The constant loss rate as applicable for the study area is deducted from hourly rainfall increments for computing the excess rainfall hyetograph.

The critical sequencing of the hourly annual maximum rainfall values for design storm duration for each year are carried out. The excess rainfall hyetographs thus obtained are convoluted with the regional unit hydrograph for estimation of direct surface runoff. For each year, the flood peaks are identified from the hydrographs of direct surface runoff of the respective years. Flood frequency analysis is carried out using this series of direct surface runoff. The values of direct surface runoff for various return periods are computed using the EV1 distribution.

6.2.3 Estimation of floods by frequency analysis of rainfall (RAIN)

The consecutive hourly annual maximum rainfall values for the design storm duration are identified for each year. The hourly rainfall values are multiplied by the areal reduction factor for converting the point rainfall values into areal rainfall. The constant loss rate as applicable for the study area is deducted from hourly rainfall increments for computing the excess rainfall hyetograph.

Frequency analysis is carried out using the sum of annual maximum excess rainfall values of design storm duration. Excess rainfall of various return periods is computed using the EV1 distribution. The sum of excess rainfall values of various return periods are distributed into hourly excess rainfall values by using the time distribution curve. These hourly excess rainfall values are convoluted with the regional unit hydrograph into direct surface runoff, without adopting the critical sequencing of the excess rainfall, and the direct surface runoff hydrographs for various return periods are computed. The values of peak direct surface runoff for the different return periods are identified from the respective direct surface runoff hydrographs.

6.2.4 Flood frequency analysis using annual maximum peak flood series computed from excess rainfall (FLOD)

The consecutive hourly annual maximum rainfall values for the design storm duration are identified for each year. The hourly rainfall values are multiplied by the areal reduction factor for converting the point rainfall values into areal rainfall. The constant loss rate as applicable for the study area is deducted from hourly rainfall increments for computing the excess rainfall hyetograph.

The hourly annual maximum rainfall values of design storm duration for each year are converted into direct surface runoff using the regional unit hydrograph, without adopting the critical sequencing of the excess rainfall rainfall values. Flood frequency analysis is carried out using this series of direct surface runoff and direct surface runoff values of various return periods are computed using the EV1 distribution.

6.3 Analysis Using Annual Maximum Peak Flood Data

The methodology adopted for testing the regional homogeneity and carrying out regional flood frequency analysis is discussed below.

6.3.1 Regional homogeneity test

In this study regional homogeneity has been tested by the U.S.G.S. homogeneity test. This test has widely been used for testing homogeneity of a region. The steps involved in U.S.G.S. homogeneity test are:

(i) Compute the EV1 reduced variate corresponding to 10 year return period flood using the relation:

$$Y_T = -\ln(-\ln(1-1/T)) \tag{4}$$

for example,

$$Y_{10} = -\ln(-\ln(1-1/10)) = 2.25 \tag{5}$$

(ii) Compute the 10 year flood putting $Y_{10} = 2.25$ in the following equation developed for the different catchments using least square approach :

$$X_{10} = u + \alpha\, Y_{10} \tag{6}$$

$$= u + 2.25\, \alpha \tag{7}$$

(iii) Repeat step (i) and (ii) to compute 2.33 year flood, which is the mean flood for EV1 distribution, for the different catchments.

(iv) Compute the ratio of 10 year flood to annual mean flood ($Q_{2.33}$) at each gauging sites. This ratio is known as the 10 year frequency ratio.

(v) Average the 10 year frequency ratios of all the gauging sites to obtain the mean 10 year frequency ratio for the region as a whole.

(vi) Determine the EV1 reduced variate corresponding to the product of annual mean annual flood and the average 10 year frequency ratio from the linear regression equations developed for each catchment . Thus :

$$Y_T = (X_T - u)/\ \alpha \tag{8}$$

(vii) Plot the EV1 reduced variates obtained from step (vi) against the effective length of records for that station on a test graph, where upper and lower regional limits of 95 % confidence are already plotted using the following coordinate pairs :

Sample size (n)	**Lower Limit** (Y)	**Upper Limit** (Y)
5	-0.59	5.09
10	0.25	4.25
20	0.83	3.67
50	1.35	3.15
100	1.52	2.88
200	1.80	2.70

(viii) If the plotted points for all the gauging sites lie between the 95 % confidence limits, then they are considered to be homogeneous.

6.3.2 Regional flood frequency analysis methods used in the study

The following four methods of regional flood frequency analysis have been used in the study.

(i) PWM based EV1 method applied to single sample of normalized data (SREV1)

(ii) PWM based GEV method applied to single sample of normalized data (SRGEV)

(iii) PWM based Wakeby method applied to single sample of normalized data (SRWAKE)

(iv) PWM based GEV method using regional data (RGEV)

6.3.2.1 PWM based EV1 method applied to single sample of normalized data (SREV1)

The steps involved in the regional flood frequency analysis using this method are given below:

(i) Select gauged catchments within the hydrologically homogeneous region.

(ii) Test for homogeneity of data obtained from various gauging stations, as explained in Section 6.3.1.

(iii) Discard those catchments from the anlaysis which are not homogeneous.

(iv) Scale the data by dividing the 'at site' data by 'at site' mean so that the regional flood curve will have a mean equal to unity.

(v) Pool the data from each selected site.

vi) Combine the scaled data obtained from step (v) for each site together to form a sample of scaled data having mean equal to unity. Hence, $\bar{m}_o = 1.0$

(vii) Compute $\bar{m}_1$ for the region by using the sample data obtained from step (vi). Thus,

$$\bar{m}_1 = \frac{1}{L} \sum_{i=1}^{L} Z_i \ (1-F_i) \qquad (9)$$

where,

$$L = \sum_{j=1}^{ns} n(j) \tag{10}$$

Z_i = Normalised data obtained from step (vi);

F_i = Plotting positions to be computed using the eq. given below:

$$F_i = \frac{i-0.35}{n} \tag{11}$$

(viii) Compute the regional EV1 parameters u and α using the PWM relations given by the following eqs:

$$\alpha = \frac{\bar{m}_0 - 2\,\bar{m}_1}{\ln 2} \tag{12}$$

$$u = \bar{m}_0 - 0.5772\,\alpha \tag{13}$$

(ix) Estimate the quantiles x_T using the relation:

$$x_T = u + \alpha\,(-\ln(-\ln(1 - 1/T))) \tag{14}$$

(x) Scale the quanitles x_T by at site mean (same as $m_{100,j}$) in order to estimate T year flood for any particular site:

$$Q_{T,j} = m_{100,j}\,x_T \tag{15}$$

where, $Q_{T,j}$ is T-year flood at jth gauging site.

6.3.2.2 PWM based GEV method applied to single sample of normalized data (SRGEV)

The regional flood frequency analysis may be carried out by this method in the following steps:

(i) Repeat step (i) to (vi) described in Section 6.3.2.1

(ii) Compute $\bar{m}_1$ *and* $\bar{m}_2$ for the region from the sample:

$$\bar{m}_1 = \frac{1}{L} \sum_{i=1}^{L} Z_i \; (1-F_i) \tag{16}$$

$$\bar{m}_2 = \frac{1}{L} \sum_{i=1}^{L} Z_i \; (1-F_i)^2 \tag{17}$$

(iii) Estimate the regional parameters, k, u and α by following the procedure described in Singh(1989)

(iv) Estimate the quantiles of T-year recurrence interval for any site using the relation:

$$X_T = u + \alpha \; (1-(-\ln(1-1/T))^k/k \tag{18}$$

(v) Follow step (x) of Section 6.3.2.1 for estimation of T year flood for any site.

6.3.2.3 PWM based Wakeby method applied to single sample of normalized data (SRWAKE)

The steps followed for carrying out the regional flood frequency analysis by this method are:

(i) Repeat step (i) to (vi) described in Section 6.3.3.1

(ii) Compute regional probability weighted moments from the sample using the equation:

$$\bar{m}_r = \frac{1}{L} \sum_{i=1}^{L} Z_i \; (1-F_i)^r \tag{19}$$

where r = 0, 1, 2, 3, 4.

(iii) Estimate the regional Wakeby parameters based on the regional probability weighted moments obtained from step (ii) using the special algorithm suggested by Landwehr et al.(1979 c).

(iv) Estimate the regional quantiles x_T using the relation:

$$X_T = m + a[1-(1/T)^b] - c[1-(1/T)^{-d}] \tag{20}$$

(v) Compute the T year flood for any gauging site by scaling the quantiles x_T obtained in step (v) by the as site mean following the Step (x) of 6.3.2.1

6.3.2.4 PWM based GEV method using regional data (RGEV)

The procedure mentioned in Section 6.3.3.2 is followed, except that the regional mean computed from the regional relationship between mean annual peak flood (MAF) and catchment area (CA), developed as discussed below is used for scaling the quantiles X_T , in place of the at site mean annual peak flood of the respective site.

For the ungauged catchments at site mean cannot be computed in absence of the flow data. In such a situation, a relationship between mean annual peak floods of the gauged catchments and their pertinent physiographic and climatological characteristics is needed for estimation of mean annual peal flood for the ungauged of the region. The form of such a relationship is mentioned below.

$$MAF = a\,(CA)^b\,S^c\,L^d\,D^e \tag{21}$$

Here, MAF is the mean annual peak flood, CA is the catchment area, S is the slope of the catchment, L is the length of main channel and D is the drainage density (or any other relevant physiographic and climatological characteristics may be adopted), a, b, c, d and e are the coefficients to be estimated using the least square approach.

7.0 ANALYSIS AND DISCUSSION OF RESULTS

The analysis carried out as well as results of the study are presented hereunder.

7.1 Analysis Using Rainfall Data

The following four methods have been considered for analysis using rainfall data.

(i) Estimation of floods by frequency analysis of annual maximum excess rainfall, considering its critical sequencing for convolution with unit hydrograph [RAIN(CS)]

(ii) Flood frequency analysis using annual maximum peak flood series computed from excess rainfall for each year considering critical sequencing of excess rainfall hyetograph for convolution with unit hydrograph [FLOD(CS)]

(iii) Estimation of floods by frequency analysis of annual maximum excess rainfall (RAIN)

(iv) Flood frequency analysis using annual maximum peak flood series computed from excess rainfall (FLOD)

7.1.1 Estimation of floods by frequency analysis of rainfall [RAIN(CS)]

The following steps are followed.

(i) The design storm duration T_D is computed as:

$$T_D = 1.1\ t_p, \text{ i.e. } T_D = 1.1*4.5 = 5 \text{ hours.}$$

(ii) The consecutive values of annual maximum rainfall for design storm duration T_D = 5 hours are identified from the available 19 years of record, for each year.

(iii) These values identified in the above Step are converted from point rainfall values to areal rainfall values by multiplying each hourly rainfall value by the point to area ratio. This ratio is worked out to be 0.91 for the duration of 5 hours and area of the catchment defined by the Bridge No. 253.

(iv) The design loss at rate of 3 mm/hour is considered, and it is subtrcted from the each block of hourly rainfall hyetograph and the excess rainfall for each year is computed.

(v) Using the above series of annual maximum excess rainfall values, frequency analysis is carried out and excess rainfall values corresponding to return periods of 2, 10 20, 50, 100 and 200 years are computed using the EV-1 distribution.

(vi) The excess rainfall values corresponding to the various return periods obtained in the Step (v) are distributed into hourly values using the mean average distribution curve for design storm duration.

(vii) The hourly values of excess rainfall of various return periods are critically arranged, and convoluted with the regional unit hydrograph for estimation of direct surface runoff hydrographs for various return periods.

(viii) The peak values of direct surface runoff are identified for the various return periods from the respective direct surface runoff hydrographs. These values are given in Table 1 for return period of 2, 10 and 20. For return periods of 50, 100 and 200 years these are given in Table 2. It is seen from the Table 1 that for the RAIN(CS) method, the values of direct surface runoff for the return period of 2, 10 and 20 years are 326, 540 and 622 cumec respectively. It is observed from the Table 2 that the values of direct surface runoff for the return period of 50, 100 and 200 years are 729, 808 and 888 cumec respectively.

Table 1 Comparison of flood estimates and percent deviations [with respect to RAIN(CS)] computed by rainfall frequency and flood frequency methods for return periods of 2, 10 and 20 years

S. No.	Method	2 year		10 year		20 year	
		Flood (Cumec)	% Deviation	Flood (Cumec)	% Deviation	Flood (Cumec)	% Deviation
1.	RAIN(CS)	326	---	540	---	622	---
2.	FLOD(CS)	384	17.8	669	24.0	777	24.9
3.	RAIN	289	-11.3	480	-11.1	553	-11.1
4.	FLOD	311	-4.6	516	-4.4	595	-4.3

Table 2 Comparison of flood estimates and percent deviations [with respect to RAIN(CS)] computed by rainfall frequency and flood frequency methods for return periods of 50, 100 and 200 years

S. No.	Method	50 year		100 year		200 year	
		Flood (Cumec)	% Dev-iation	Flood (Cumec)	% Dev-iation	Flood (Cumec)	%Dev-iation
1.	RAIN(CS)	729	---	808	---	888	---
2.	FLOD(CS)	918	25.9	1023	26.6	1128	27.0
3.	RAIN	648	-11.1	718	-11.1	789	-11.1
4.	FLOD	697	-5.7	773	-4.3	849	-4.4

7.1.2 Flood frequency analysis using annual maximum peak flood series computed from excess rainfall [FLOD(CS)]

(i) The Stpes (i) to (iv) of Section 7.1.1 are repeated.

(ii) The annual maximum excess rainfall values for of design storm duration of 5 hours for each year is arranged in critical sequence. These hourly excess rainfall values are convoluted with the regional unit hydrograph for estimation of direct surface runoff hydrographs for each year.

(iii) The annual maximum peak values of direct surface runoff are identified from the respective direct surface runoff hydrograph of each year.

(iv) Flood frequency analysis is carried out considering the series of annual maximum peak direct surface runoff values obtained in Step (iii) above, using the EV-1 distribution and direct surface runoff values for various return periods are estimated . The values are given in Table 1 and Table 2. It is observed from Table 1 that for the FLOD(CS) method, the direct surface runoff values for return periods of 2, 10 and 20 years are 384, 669 and 777 cumec respectively. It is seen from Table 2 that the direct surface runoff values for return periods of 50, 100 and 200 years are 918, 1023 and 1128 cumec respectively.

The percentage deviations with respect to the direct surface runoff computed by the method RAIN(CS), discussed in Section 7.1.1 above, and this method FLOD(CS) (Section 7.1.2) for return periods of 2, 10 and 20 years are given in Table 1. For return periods of 50, 100 and 200 years these are given in Table 2. It is seen from Table 1 and Table 2 that the percentage deviations between the estimates of direct surface runoff by the RAIN(CS) and FLOD(CS) vary from 17.8% for a return period of 2 years to 27% for the return period of 200 years.

7.1.3 Estimation of floods by frequency analysis of rainfall (RAIN)

(i) The Steps (i) to (vi) of Section 7.1.3 are repeated.

(ii) The hourly values of excess rainfall of various return periods of each year are convoluted with the regional unit hydrograph for estimation of direct surface hydrographs for various return periods. There is no critically arranging of the hourly excess rainfall values.

(iii) The peak values of direct surface runoff are identified for the various return periods from the respective direct surface runoff hydrographs, and the same are given in Table 1 and Table 2. It is observed from Table 1 that the direct surface runoff values for this method for return periods of 2, 10 and 20 years are 289, 480 and 553 cumec respectively. It is observed from Table 2 that the direct surface runoff values for return periods of 50, 100 and 200 years are 648, 718 and 789 cumec respectively.

The percentage deviations with respect to the peak value of direct surface runoff computed by the method RAIN(CS), discussed in Section 7.1.1 above, and this method RAIN (Section 7.1.3) for return periods of 2, 10 and 20 years are given in Table 1. For return periods of 50, 100 and 200 years these are given in Table 2. It is seen from Table 1 and Table 2 that the percentage deviations between the estimates of peak value of direct surface runoff by the RAIN(CS) and RAIN vary from -11.3% for a return period of 2 years to -11.1% for the return period of 200 years.

7.1.4 Flood frequency analysis using annual maximum peak flood series computed from excess rainfall (FLOD)

(i) The Stpes (i) to (iv) of Section 7.1.1 are repeated.

(ii) The annual maximum excess rainfall values for of design storm duration of 5 hours for each year are convoluted with the regional unit hydrograph for estimation of direct surface runoff hydrographs for each year, without adopting the critical sequencing of the excess rainfall values.

(iii) The annual maximum peak values of direct surface runoff are identified from the respective direct surface runoff hydrograph of each year.

(iv) Flood frequency analysis is carried out using the series of annual maximum peak direct surface runoff values obtained in Step (iii) above; and floods of various return periods are estimated using the EV-1 distribution, and the same are given in Table 1 and Table 2. It is seen from Table 1 that the direct surface runoff values for return periods of 2, 10 and 20 years are 311, 516 and 595 cumec respectively. It is observed from Table 2 that the direct surface runoff values for return periods of 50, 100 and 200 years are 697, 773 and 849 cumec respectively.

The percentage deviations with respect to the direct surface runoff computed by the method, RAIN(CS), discussed in Section 7.1.1 above, and this method, FLOD (Section 7.1.4) for return periods of 2, 10 and 20 years are given in Table 1. For return periods of 50, 100 and 200 years these are given in Table 2. It is seen from Table 1 and Table 2 that the percentage deviations between the estimates of direct surface runoff by the RAIN(CS) and RAIN vary from -4.3% for a return period of 2 years to -5.7% for the return period of 200 years.

7.2 Analysis Using Annual Maximum Peak Flood Data

The sample statistics computed from the observed annual maximum peak flood record of the 13 gauging sites located in the Upper Narmada and Tapr Subzone-3(c) are given in Table 3, along with the catchment area and sample size.

7.2.1 Testing homogeneity of the region

The homogeneity of the region has been tested using the U.S.G.S. homogeneity test, discussed in Section 6.3.1. Data for all the 13 gauging sites are regionally homogeneous as per the U.S.G.S. homogeneity test. The homogeneity test graph is shown in Fig. 4. After testing the regional homogeneity, regional flood frequency analysis has been carried out as discussed below.

7.2.2 Regional flood frequency analysis

The annual maximum peak flood data of 13 bridge sites have been used for estimation of the regional parameters required for the regional methods viz. SREV1, SRGEV, SRWAKE and RGEV, as well as for development of the regional relationship between mean annual peak flood (MAF) and catchment area (CA) used for the regional method RGEV, as discussed in Section 6.

Table 3 Catchment area, sample statistics and sample size

S.NO.	Br.No.	Catchment Area (Sq Km)	Mean Flood (Cumec)	Standard Deviation (Cumec)	Coff. of Variation	Coff. of Skewness	Sample Size (Years)
1	731/6	115.90	252.87	130.05	.514	.603	30
2	294	518.67	919.60	561.88	.611	.635	30
3	897/1	341.88	856.46	665.22	.777	1.222	26
4	634/2	348.92	380.10	249.40	.656	1.661	29
5	813/1	70.18	211.79	112.87	.533	.274	24
6	863/1	2110.85	1687.27	1481.13	.878	1.404	22
7	253	114.22	216.90	135.35	.624	.417	20
8	584/1	139.08	248.78	203.32	.817	1.252	23
9	512/3	142.97	219.95	154.69	.703	1.066	22
10	710/1	41.80	111.95	122.69	1.096	1.152	21
11	776/1	179.90	572.78	279.18	.487	.826	18
12	625/1	535.4	1730.53	711.90	.411	-.617	19
13	787.2	321.16	811.79	854.59	1.053	2.876	14

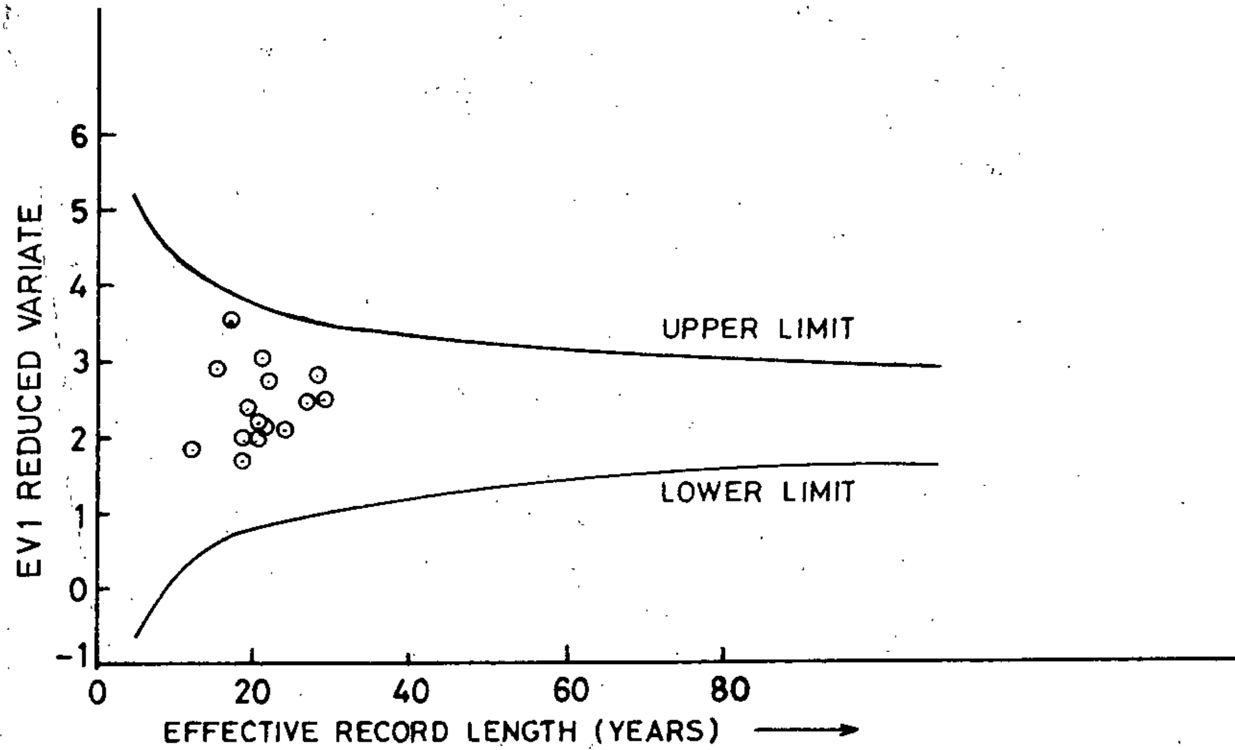


FIG. 4 HOMOGENEITY TEST GRAPH FOR SUB ZONE 3(c)

The regional values of parameters for the 3 frequency distributions considered in the study are:

(i) EV1 distribution	: $u = 0.689$,	$\alpha = 0.539$	
(ii) GEV distribution	: $K = -0.087$,	$u = 0.668$,	$\alpha = 0.494$
(iii) Wakeby distribution	: $a = 0.320$,	$b = 3.880$,	$c = -13.850$
	$d = -0.051$,	$m = 0.067$	

The relationship between MAF and CA developed for the region in log domain using least square approach is:

$$MAF = 6.619\ (CA)^{0.78} \tag{22}$$

for which correlation coefficient is, $r = 0.913$. Hypothesis $H_o : \beta_i = 0$ versus $H_a : \beta_i \neq 0$ is tested by computing T values corresponding to each β_i value, where β_i are regression coefficients. The hypothesis H_o is rejected if absolute value of computed value of T is greater than critical value of $T_{(1-\alpha/2),\,(n-2)}$, where α is the significance level and (n-2) is degree of freedom. The computed value of T are 3.275, and 7.433 respectively for the two regression coefficients in the log domain of this equation. The critical value of $T_{(1-\alpha/2),\,(n-2)}$ is 2.20 for 11 degree of freedom at 5% significance level. Since the computed values of T for the regression coefficients are greater than the critical value of T, hence the the null hypothesis H_o is rejected for both the regression coefficients. It indicates that the regression coefficients significantly contribute to the above equation.

Floods for different return periods viz. 2, 10, 20, 50, 100 and 200 have been computed using the above mentioned four methods of regional flood frequency analysis following the procedure discussed in Sections 6.3.2.1 to 6.3.2.4.

The peak values of direct surface runoff have been computed for the four regional flood frequency analysis methods by subtracting the baseflow from the estimated floods of various return periods. The baseflow of 5.71 cumec has been considered at the rate of 0.05 cumec per square kilometer(CWC, 1983) for the catchment area of 114.22 square kilometers. These peak values of direct surface runoff along with the percentage deviations with respect to direct surface runoff values estimated by the method RAIN(CS), discussed in Section 7.1.1 are given in Table 4 for the return periods of 2, 10 and 20 years; and these are given in Table 5 for the return periods of 50, 100 and 200 years.

Table 4 Flood estimates computed by regional flood frequency methods and percent deviations with respect to rainfall frequency method [RAIN(CS)] for return periods of 2, 10 and 20 years

S. No.	Method	2 year		10 year		20 year	
		Flood (Cumec)	% Deviation	Flood (Cumec)	% Deviation	Flood (Cumec)	%Deviation
1.	SREV1	215	-34.0	467	-13.7	564	-9.3
2.	SRGEV	206	-36.8	466	-13.7	577	-7.2
3.	SRWAKE	206	-36.8	476	-11.9	583	-6.2
4.	RGEV	221	-32.2	499	-7.6	618	-6.4

Table 5 Flood estimates computed by regional flood frequency methods and percent deviations with respect to rainfall frequency method [RAIN(CS)] for return periods of 50, 100 and 200 years

S. No.	Method	50 year		100 year		200 year	
		Flood (Cumec)	% Deviation	Flood (Cumec)	% Deviation	Flood (Cumec)	%Deviation
1.	SREV1	689	-5.4	783	-3.1	876	-1.4
2.	SRGEV	732	0.4	856	5.9	988	11.3
3.	SRWAKE	719	-1.4	818	1.2	913	2.8
4.	RGEV	784	7.5	917	13.5	1058	19.1

Fig. 5 to Fig. 7 show comparison of the direct surface runoff values for the return periods of 50, 100 and 200 years, for the various methods.

Variations of floods of 2, 10, 20, 50, 100 and 200 year return periods computed by frequency analysis of rainfall [RAIN(CS)] and frequency analysis of floods computed by the various methods viz. FLOD, SREV1, SRGEV, SRWAKE and RGEV are shown in Fig. 8.

FIG. 5 COMPARISON OF FLOOD ESTIMATES FOR 50 YEAR FLOOD FOR VARIOUS METHODS

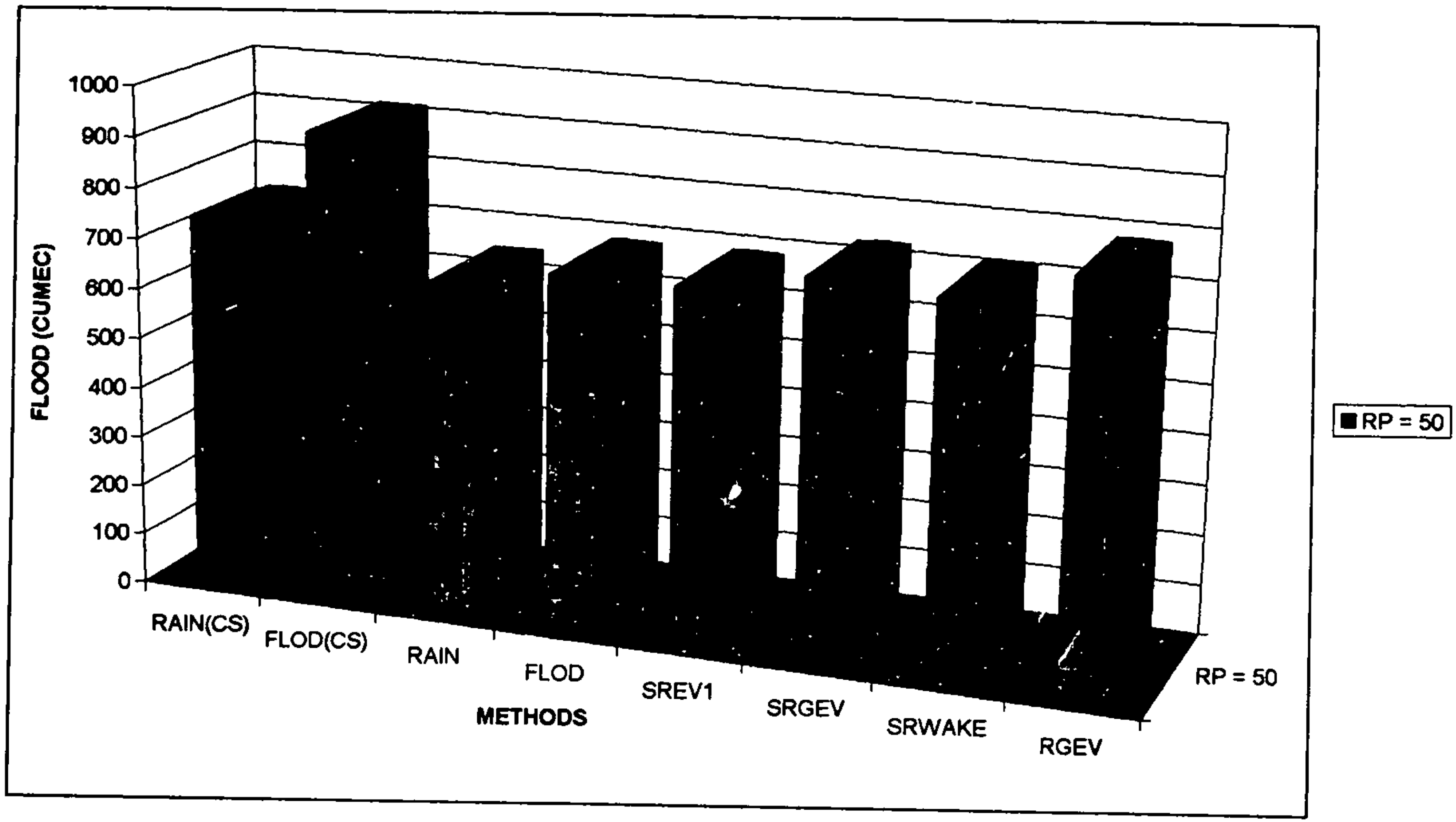


FIG. 6 COMPARISON OF FLOOD ESTIMATES FOR 100 YEAR FLOOD FOR VARIOUS METHODS

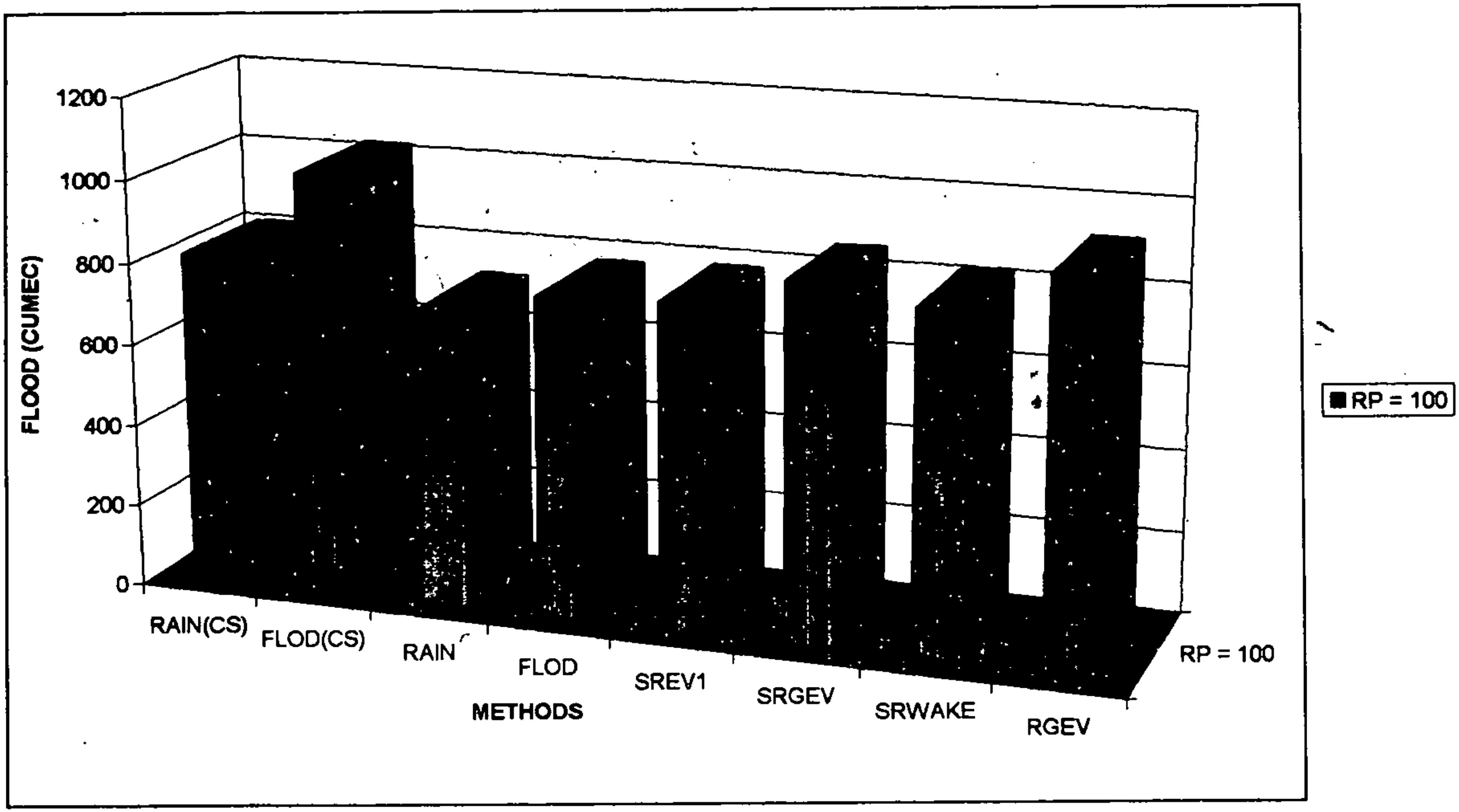
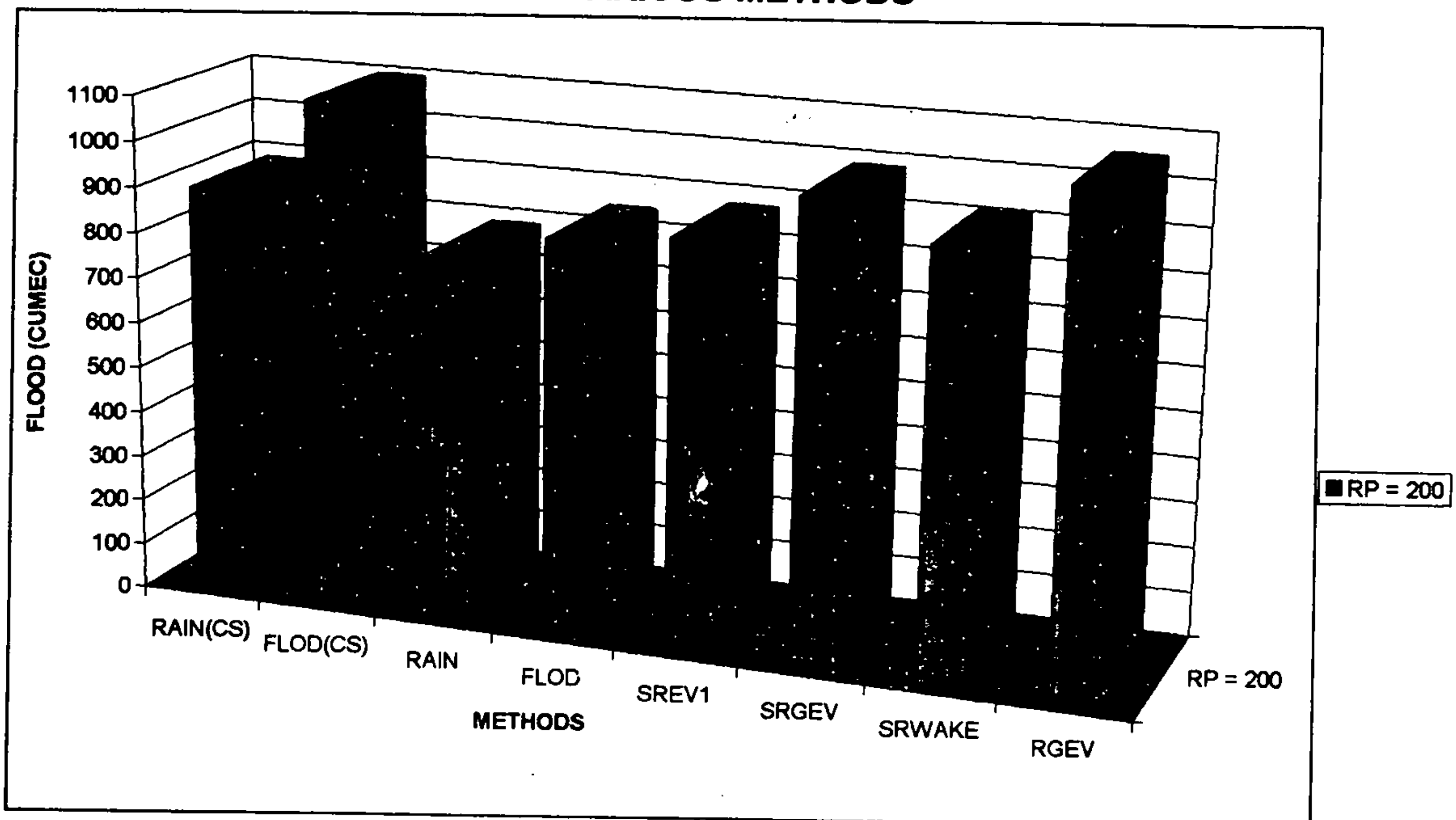


FIG.7 COMPARISON OF FLOOD ESTIMATES FOR 200 YEAR FLOOD FOR VARIOUS METHODS



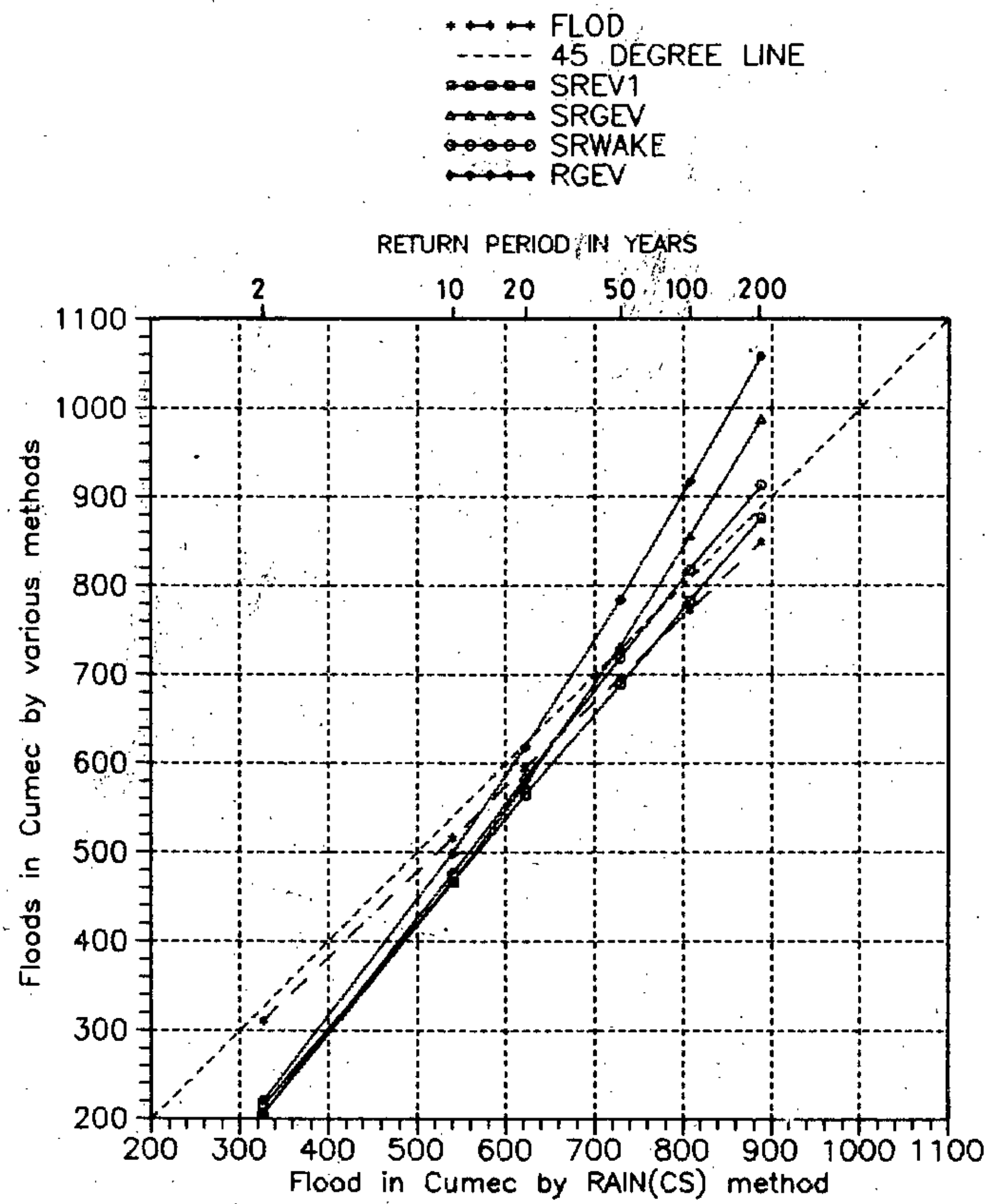


FIG. 8 VARIATION OF FLOODS OF 2, 10, 20, 50, 100 AND 200 YEAR RETURN PERIODS COMPUTED BY RAIN(CS) AND VARIOUS OTHER METHODS

7.3 Sensitivity Analysis for Regional Unit Hydrograph Parameters

In order to study the effect of change in unit hydrograph peak, sensitivity analysis has been conducted by increasing and decreasing the peak of the adopted regional unit hydrograph and the following cases have been considered.

7.3.1 CASE A

In this case, the regional unit hydrograph paramers as available in the CWC(1983) report have been considered. The excess rainfall values computed from the available data for the four methods viz. RAIN(CS), FLOD(CS), RAIN and FLOD as discussed in Section 7.1 have been convoluted with the regional unit hydrograph and the peak values of direct surface runoff of various return periods have been identified, as discussed in Sections 7.1.1 to 7.1.4 respectively.

7.3.2 CASE B

In this case, the peak of the regional unit hydrograph has been increased by 20%. The excess rainfall values computed from the available data for the four methods viz. RAIN(CS), FLOD(CS), RAIN and FLOD as discussed in Section 7.1 have been convoluted with the unit hydrograph, obtained by increasing the peak of the regional unit hydrograph by 20% and the peak values of direct surface runoff of various return periods have been computed, following the procedure given in Section 7.1.1 to Section 7.1.4 respectively. These values, along with their percentage deviations with respect to direct surface runoff values obtained in CASE A are given in Table 6 through Table 11 for the return period of 2, 10, 20, 50, 100 and 200 years respectively. From the Table 6 to Table 8, it is observed that percentage deviations of the direct surface runoff of CASE B, with respect to CASE A vary from 6% to 14.8% for the return periods of 2, 10 and 20 years for all the four methods. Table 9 to Table 11 show that percentage deviations of the direct surface runoff of CASE B, with respect to CASE A vary from 6.4% to 14.3% for the return periods of 50, 100 and 200 years for all the four methods.

7.3.3 CASE C

In this case, the peak of the regional unit hydrograph has been decreased by 20%. The excess rainfall values computed from the available data for the four methods viz. RAIN(CS), FLOD(CS), RAIN and FLOD as discussed in Sections 7.1.1 to 7.1.4 respectively have been convoluted with the unit hydrograph, obtained by decreasing the peak of the regional unit hydrograph by 20%, and the peak values of direct surface runoff of various return periods have been identified. These values, along with their deviations with respect to peak values of direct surface runoff obtained in CASE A are given in Table 8 through Table 11 for the return periods of 2, 10, 20, 50, 100 and 200 years respectively. From the Table 6 to Table 8, it is obseved that percentage of deviations of peak values of direct surface runoff obtained for CASE C with respect to CASE A vary from -8.5% to -14.2% for all the four methods, for return periods of 2, 10 and 20 years. From the Table 8 to Table 11, it is obseved that percentage of deviations of direct surface runoff obtained for CASE C with respect to CASE A vary from -7.3% to -13.5% for all the four methods, for return periods of 50, 100 and 200 years.

Table 6 Flood estimates and percent deviations for sensitivity analysis for return of period of 2 years

S. No.	Method	CASE A	CASE B		CASE C	
		Flood (Cumec)	Flood (Cumec)	% Dev-iation	Flood (Cumec)	%Dev-iation
1.	RAIN(CS)	326	363	11.3	271	-14.1
2.	FLOD(CS)	384	406	6.0	344	-10.4
3.	RAIN	289	332	14.8	248	-14.2
4.	FLOD	311	342	10.0	270	-13.2

Table 7 Flood estimates and percent deviations for sensitivity analysis for return of period of 10 years

S. No.	Method	CASE A	CASE B		CASE C	
		Flood (Cumec)	Flood (Cumec)	% Dev-iation	Flood (Cumec)	%Dev-iation
1.	RAIN(CS)	540	602	11.5	464	-14.1
2.	FLOD(CS)	669	711	6.2	612	-8.5
3.	RAIN	480	549	14.4	418	-12.9
4.	FLOD	517	571	10.4	450	-12.9

Table 8 Flood estimates and percent deviations for sensitivity analysis for return of period of 20 years

S. No.	Method	CASE A	CASE B		CASE C	
		Flood (Cumec)	Flood (Cumec)	% Dev-iation	Flood (Cumec)	%Dev-iation
1.	RAIN(CS)	623	694	11.4	536	-13.9
2.	FLOD(CS)	777	827	6.4	714	-8.8
3.	RAIN	553	632	14.3	483	-12.6
4.	FLOD	595	658	10.6	519	-12.7

Table 9 Flood estimates and percent deviations for sensitivity analysis for return of period of 50 years

S. No.	Method	CASE A	CASE B		CASE C	
		Flood (Cumec)	Flood (Cumec)	% Dev-iation	Flood (Cumec)	%Dev-iation
1.	RAIN(CS)	729	811	11.2	630	-13.5
2.	FLOD(CS)	918	977	6.4	847	-7.7
3.	RAIN	648	739	14.0	567	-12.5
4.	FLOD	697	771	10.6	608	-12.8

Table 10 Flood estimates and percent deviations for sensitivity analysis for return of period of 100 years

S. No.	Method	CASE A	CASE B		CASE C	
		Flood (Cumec)	Flood (Cumec)	% Deviation	Flood (Cumec)	%Deviation
1.	RAIN(CS)	808	901	11.5	700	-13.4
2.	FLOD(CS)	1023	1090	6.5	946	-7.5
3.	RAIN	718	821	14.3	630	-12.3
4.	FLOD	773	856	10.7	675	-12.6

Table 11 Flood estimates and percent deviations for sensitivity analysis for return of period of 200 years

S. No.	Method	CASE A	CASE B		CASE C	
		Flood (Cumec)	Flood (Cumec)	% Deviation	Flood (Cumec)	%Deviation
1.	RAIN(CS)	887	989	11.5	770	-13.2
2.	FLOD(CS)	1128	1202	6.5	1045	-7.3
3.	RAIN	789	902	14.3	693	-12.2
4.	FLOD	849	940	10.7	742	-12.6

Fig. 9 to Fig. 12 show comparison of the peak value of direct surface runoff estimated for the CASE A, CASE B and CASE C for the return period of 100 years.

Fig.9 COMPARISON OF FLOOD ESTIMATES FOR SENSITIVITY RUNS FOR RAIN(CS) METHOD

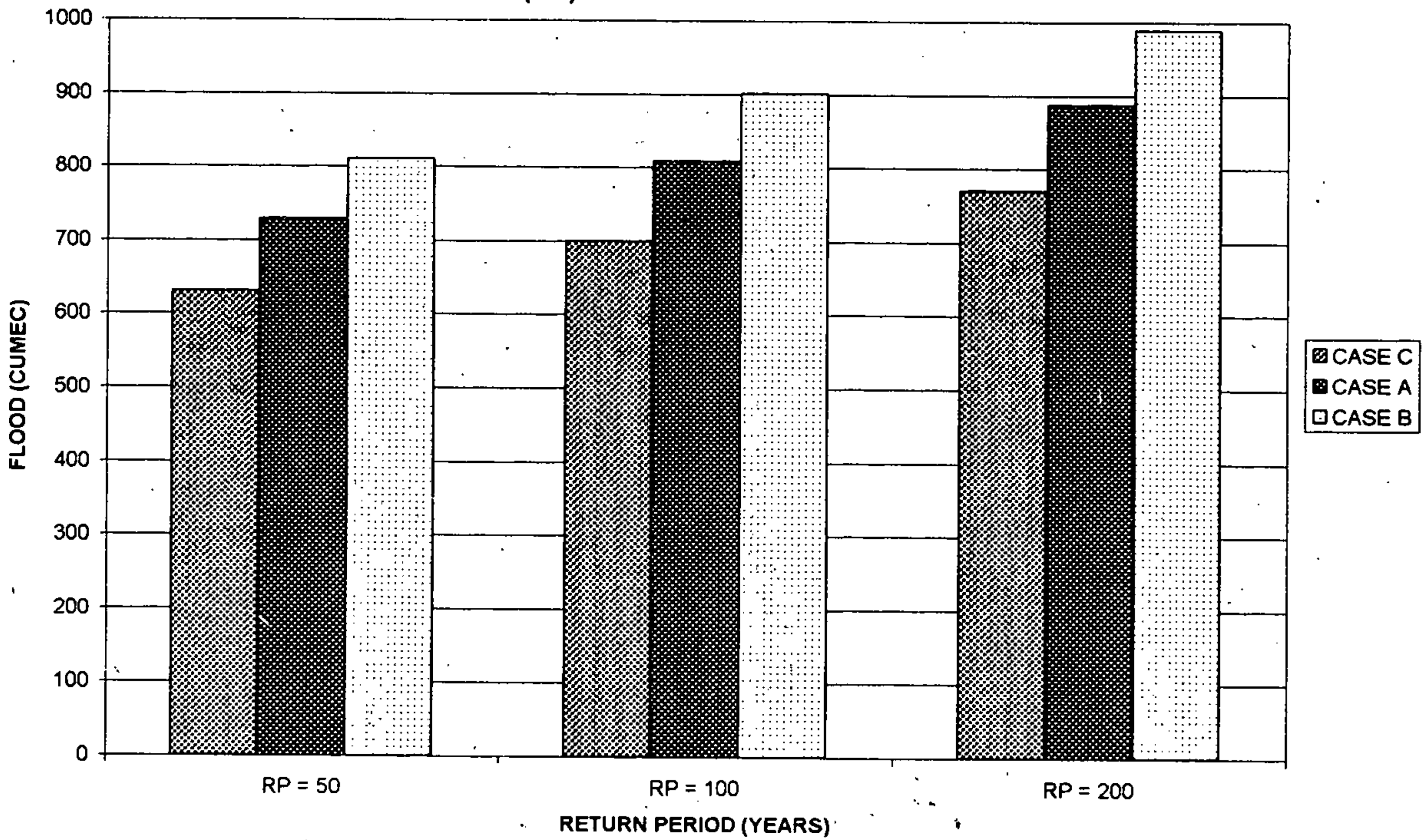


FIG.10 **COMPARISON OF FLOOD ESTIMATES FOR SENSITIVITY RUNS FOR FLOD(CS) METHOD**

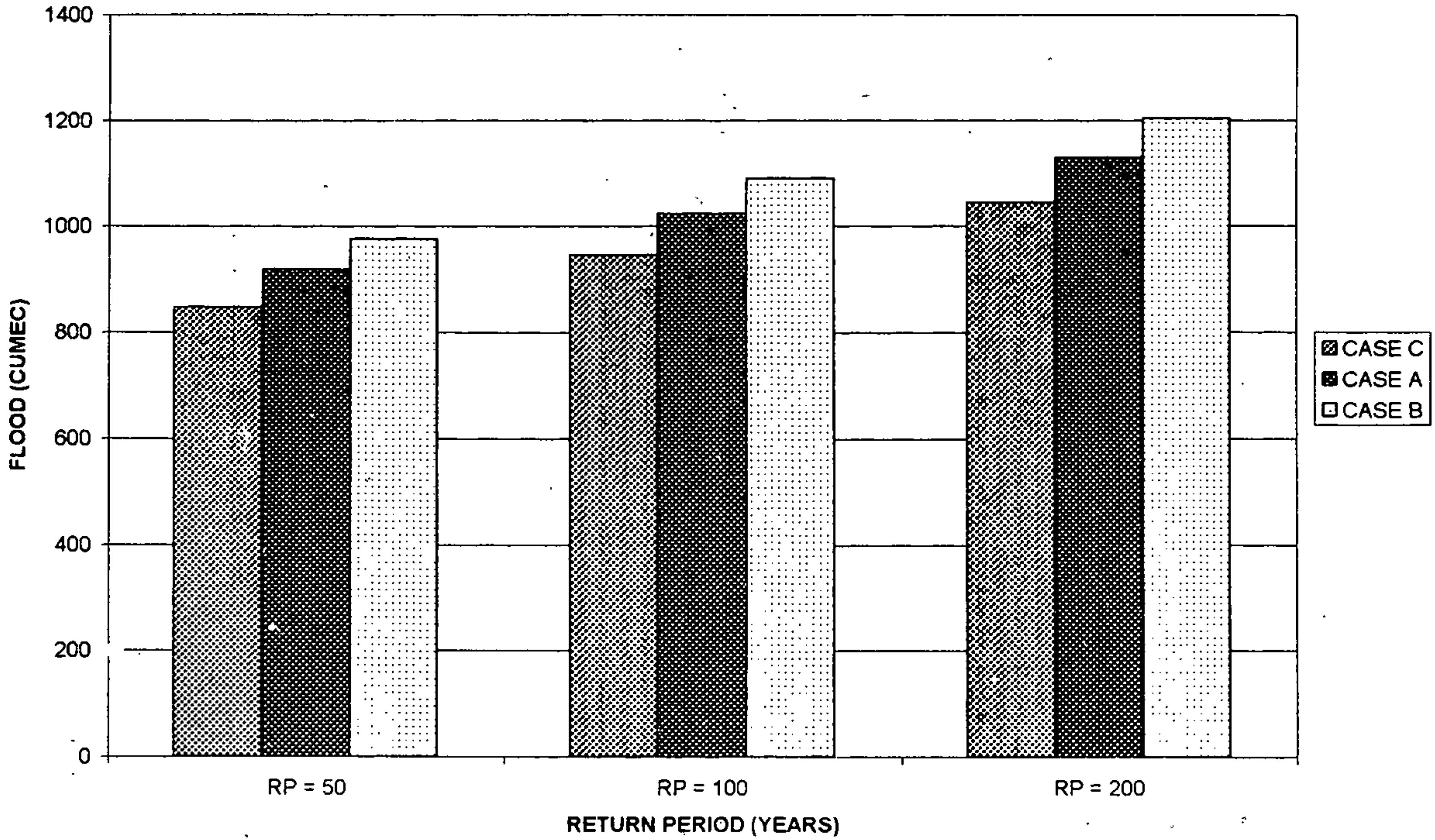


FIG.11 COMPARISON OF FLOOD ESTIMATES FOR SENSITIVITY RUNS FOR RAIN METHOD

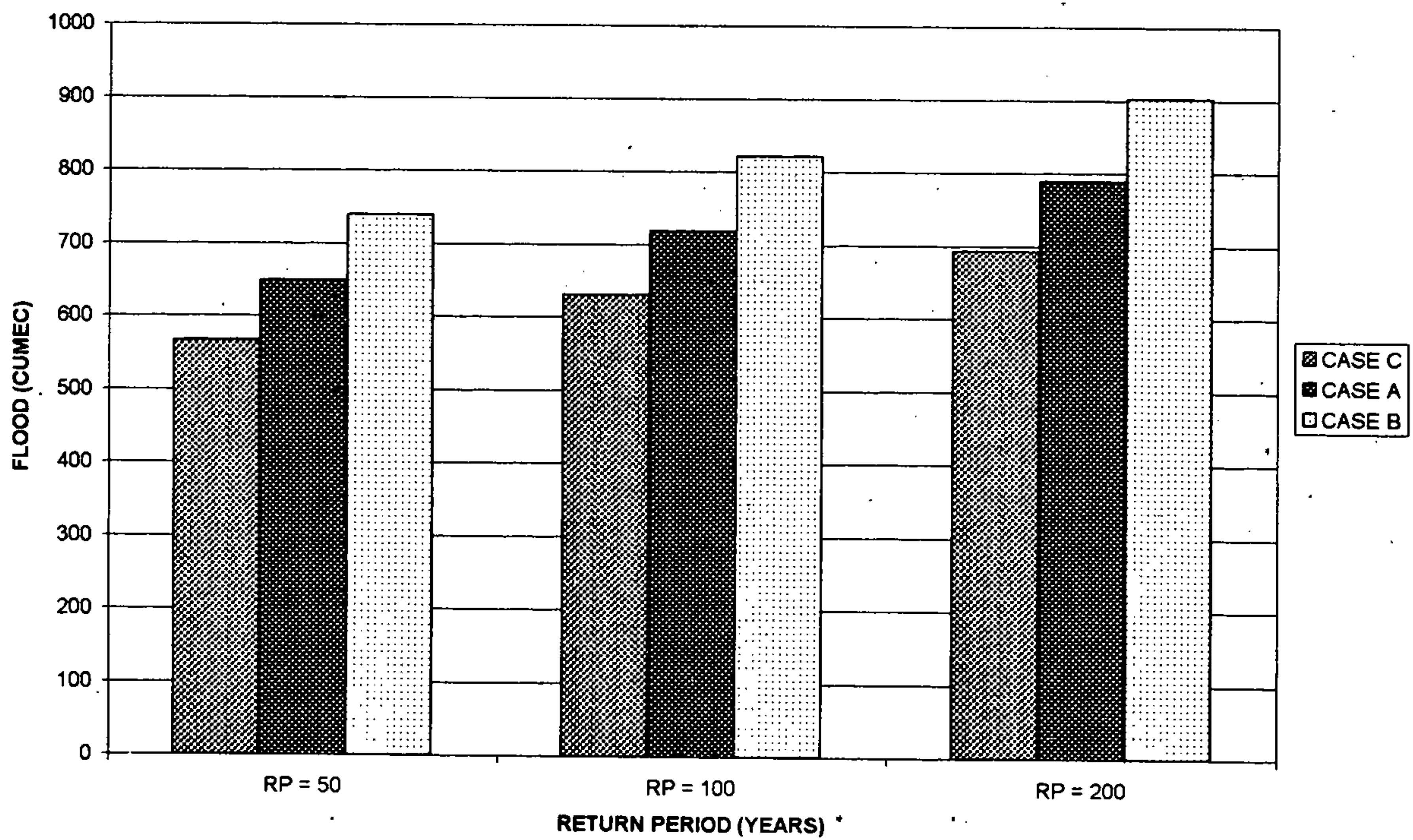
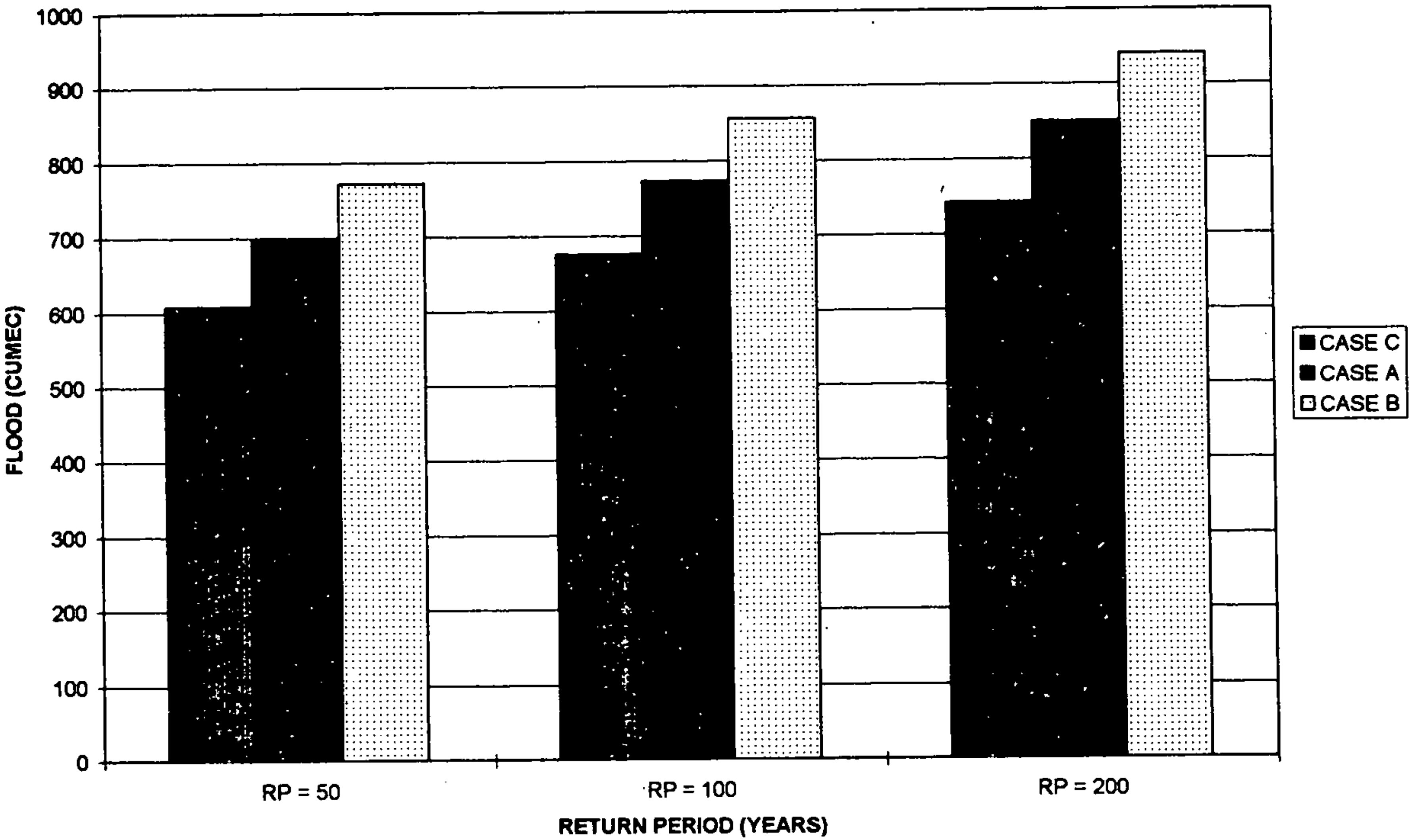


FIG.12 COMPARISON OF FLOOD ESTIMATES FOR SENSITIVITY RUNS FOR FLOD METHOD



8.0 CONCLUSIONS

The floods of various return periods have been estimated for the catchment defined by the Bridge No. 253 of the Upper Narmada and Tapi Subzone 3(c) using various methods involving frequency analysis of rainfall and frequency analysis of annual maximum peak floods computed from the annual maximum excess rainfall of design storm duration. The floods of various return periods have also been computed using the regional flood frequency analysis approach based on the observed annual maximum peak flood record for 13 gauging sites of the Subzone 3(c). Sensitivity analysis has also been conducted by increasing and decreasing the peak of the unit hydrograph, which has been used to convert the excess rainfall hyetographs into direct surface runoff hydrographs for identifying the peak values of floods.

On the basis of the study following conclusions are drawn.

(a) The flood estimates computed by the method based on frequency analysis of rainfall [RAIN(CS)] and the method based on frequency analysis of annual maximum peak floods computed from the annual maximum excess rainfall (FLOD) show a variation of -4.6%, -4.4%, -4.3%, -5.7%, -4.3% and -4.4% for the return periods of 2, 10, 20, 50, 100 and 200 years respectively. It shows that the floods are under estimated by 4.3% to 5.7% for the return periods of 2 to 200 years, by frequency analysis of floods as compared to the frequency analysis of rainfall.

(b) The FLOD(CS) method, in which critical sequencing of excess rainfall hyetograph is adopted when the annual maximum rainfall hyetograph of each year for the design storm duration is convoluted with the regional unit hydrograph, over estimates the floods by 17.8%, 24% 24.9%, 25.9%, 26.6% and 27% for the return periods of 2, 10, 20, 50, 100 and 200 years.

(c) The RAIN method, in which critical sequencing is not adopted for converting the excess rainfall values of various return periods, while convoluting the excess rainfall hyetograph with the regional unit hydrograph, under estimates the floods of return periods of 2 to 200 years by about 11%.

(d) The regional flood frequency methods used in the study, viz. SREV1, SRGEV and SRWAKE are based on `at site and regional data'; whereas, RGEV method is based on regional data' alone. For the return period of 50 years, flood estimates obtained by these methods show a deviation of -5.4% to 7.5% with respect to rainfall frequency method[RAIN(CS)]; percentage deviation is only 0.4% for SRGEV method and -1.4% in case of SRWAKE method. The deviation varies from -3.1% to 13.5% for return period of 100 years. For the return period of 200 years the deviation is -1.4% for SREV1, 11.3% for SRGEV, 2.8% for SRWAKE and 19.1% for RGEV method. The flood estimates by these methods show relatively higher deviation of about -36% with respect to RAIN(CS) method for the lowest return period of 2 years. Percentage deviation for flood estimates by the SRWAKE method, for the return periods of 50, 100 and 200 years are -1.4%, 1.2% and 2.8% respectively; which are very close to the method based on frequency of rainfall[RAIN(CS)].

(e) While conducting sensitivity analysis, when peak of the regional unit hydrograph is increased by 20%; it is observed that the flood estimates for the various return periods increase with respect to the flood estimates computed by the respective methods, considered with the actual peak of the regional unit hydrograph by about 11.5% in case of the RAIN(CS) method, by about 6.5% for FLOD(CS), by about 14.5% for RAIN method and by about 10.5% for FLOD method.

(f) For sensitivity analysis, when peak of the regional unit hydrograph is decreased by 20%; it is observed that the flood estimates for the various return periods decrease with respect to flood estimates computed by the respective methods, considered with the actual peak of the regional unit hydrograph by about 14% in case of the RAIN(CS) method, by about 8.5% for FLOD(CS), by about 13% for RAIN method and by about 13% for FLOD method.

The rainfall data used in the study is of the limited record length of 19 years of one raingauge station only; hence the results of the study may be considered as indicative only, and detailed studies with long term data for a large number of catchments should be carried out for drawing more realistic conclusions.

ACKNOWLEDGEMENT

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(i) Criteria for Fixing Spillway Capacity

The hydrologic design criteria for fixing spillway capacity as prevalent in India are mentioned in IS 11223-1985, "Guidelines for fixing Spillway Capacity". According to these guidelines various inflow design floods that need to be considered for various functions of spillways are:

(i) Inflow design flood for the safety of the dams:

It is the flood for which the performance of the dam should be safe against overtopping, structural failure and its energy dissipation arrangements, if provided for a lower flood, should function reasonably well.

(ii) Inflow design flood for efficient operation of energy dissipation works.

This flood could be lower than the flood for safety of dam and for this the dissipation arrangements, are expected to work most efficiently.

(iii) Inflow design flood for checking extent of upstream submergence.

(iv) Inflow design flood for extent of downstream damage in the valley.

The criteria for classification of dams is based on size of the dam and the hydraulic head (MWL - average flood level on downstream). The classification for the dam is greater of the two indicated by the two parameters:

Classification	Gross storage (in million cubic meters)	Hydraulic head (in meters)
Small	Between 0.5 and 10	Between 7.5 and 12
Intermediate	Between 10 and 60	Between 12 and 30
Large	Greater than 60	Greater than 30

The inflow design flood for safety of the dam would be as follows:

Size as determined above	Inflow design flood for safety of Dam
Small	100 year flood
Intermediate	Standard Project Flood (SPF)
Large	Probable Maximum Flood (PMF)

Floods of larger or smaller magnitudes may be used if the hazard involved is high or low. The relevant parameters to be considered in judging the hazard in addition to the size would be :

(i) distance to and location of the human habitations on the downstream after considering the likely future developments.

(ii) maximum hydraulic capacity of the downstream channel at a level at which catastrophic damage is not expected.

For more important projects dam break studies may be done as an aid to the judgment in deciding whether PMF needs to be used. Where the studies or judgment indicate an imminent danger to present or future human settlements, the PMF should be used. Any departure from the general criteria as above on account of larger or smaller hazard should be clearly brought out and recorded.

(ii) Criteria for Design Flood Estimation for Barrages

For barrages, the CWC 1968 criteria are applicable. Diversion dams or weirs and barrages have usually small storage capacities, and the risk of loss of life and property down stream would rarely be enhanced by failure of the structure. Apart from the loss of the structures by its failure, this would bring about disruption of irrigation and communications that are dependent on the barrage. In consideration of these risks involved the CWC criteria redesigned for floods of frequency 50 to 100 years. For barrages, it requires the use of a 100 year return period flood or standard project flood whichever is higher.

(iii) Criteria for Design Flood Estimation for Weirs (Ungated Headworks)

In the case of small reservoirs where the release of stored water due to the failure of the dam would not appreciably enhance the flood hazard downstream, the spillway capacity may be designed for a design flood of specified frequency, say 50 to 100 years as recommended by the Central Water Commission.

(iv) Criteria for Design Flood Estimation of Road and Railway Bridges

For road bridges, the Indian Road Congress IRC: 5-1970, Section-I General Features of Design applies. According to this, the design discharge for which the waterway of a bridge is to be designed shall be the maximum flood observed for a period of not less than 50 years; shall be discharge from an another recognised method applicable for that area; shall be the discharge found by the area velocity method; by unit-hydrograph method; and the maximum discharge fixed by the judgment of the engineers responsible for the design with comparison of above mentioned methods is to be adopted. For railway bridges, a 50-year flood is to be used for smaller bridges carrying railways of lesser importance like minor lines and branch lines. In the case of larger bridges i.e. those carrying main lines and very important rail lines, a 100-year return period flood is to be adopted as per the railway codes (Indian Railway Standards - 1963).

(v) Criteria for Design Flood Estimation for Cross Drainage Structures on Irrigation Networks

The BIS Code of practice for design of cross drainage works [IS:7784(part-I)1975] recommends that the design (of waterway) in such cases may be based on 10 to 25-year frequency flood with increased afflux. However, the foundations and free-board etc., should be checked to be safe for the increased afflux and velocities due to a 50 year or 100 year return period flood.

For very large cross drainage works, damage to which is likely to affect the canal supplies over a long period the design should be based on maximum probable flood. It is quite probable that a flood of higher magnitude than the design flood may pass through the structure posing great danger to the stability of foundation and the structure. Return period to take care of this unprecedented and unforseen nature of flood intensities in cases of important structures, an adequate margin of safety is envisaged in the estimation of design discharge. For this purpose, the design discharge may be increased by the percentages given below for obtaining the foundation and free-board design.

Catchment area (in square kilometers)	Increase in design discharge
upto 500	30% to 25% decreasing with increase in area
500 to 5000	25% to 20% decreasing with increase in area
5000 to 25000	20% to 10% decreasing with increase in area
above 25000	upto 10%

As per Central Water Commission criteria, waterways for canal aqueducts should be provided to pass a 50-100 year return period flood, but their foundations and free-boards should be for a flood of not less than 100-year return period.

The Government of Gujarat has adopted a still severer criteria for cross drainage works of Sardar Sarovar Narmada Canal, which is given below.

Catchment area (in square miles)	Design flood to be adopted	
	For design	for checking
0 to 10	100 year flood	100 year flood + 30%
10 to 50	- do -	- do -
50 to 200	- do -	P.M.F.
200 and above	- do - (or S.P.F.)	P.M.F.

(vi) Design Criteria for Flood Control Schemes

The following broad criteria are recommended and adopted in the country.

Predominantly agricultural	25 year return period flood on small tributaries and 50 year flood on major rivers
Town protection works	100 year return period flood
Important industrial comp- lexes, assets and lines of communications	100 year return period flood

According to Ganga Flood Control Commission, subject to availability of observed hydrological data, the design HFL may be fixed on the basis of flood frequency analysis. In no case, the design HFL should be lower than the maximum on record. For small rivers carrying discharge upto 3000 cumecs, the design HFL shall correspond to 25 years return period flood. For the river carrying peak flood above 3000 cumecs, the design HFL shall correspond to 50 years return period. However, if the embankments concerned are to protect big township, industrial area or other places of strategic importance the design HFL shall generally correspond to 100 year return period flood.

The Rashtriya Barh Ayog recommends that benefit-cost criterion should be properly adopted. But since the relevant data for such an analysis may not be available the Ayog recommends (i) for predominantly agricultural areas: 25-year flood frequency (in special cases, where the damage potential justifies, adopted); (ii) for town protection works, important industrial complexes etc: 100-year flood frequency (for large cities like Delhi, the maximum observed flood, or even the maximum probable flood should be considered for adoption).

Each site is individual in its local conditions, and evaluation of causes, and effects. While, therefore, the above mentioned norms, may be taken as the general guidelines, the hydrologist, and, the designer would have the discretion to vary the norms, and the criteria in special cases, where the same are justifiable on account of assessable and acceptable local conditions; these should be recorded, and, have the acceptance of the competent authority.

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