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# CHARACTERISTICS OF SHORT INTERVAL RAINFALL FOR PUNPUN BASIN


**NATIONAL INSTITUTE OF HYDROLOGY**
**JALVIGYAN BHAWAN**
**ROORKEE - 247 667 (UTTARANCHAL)**


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# PREFACE

Estimation of design flood is one of the important components of planning and design of water resources projects. Based on the design criteria applicable for a hydraulic structure, design flood is generally estimated based on two types of approaches viz. (i) deterministic approach, and (ii) probabilistic approach. The deterministic approach is based on the hydrometeorological technique, which requires design storm and the unit hydrograph for a catchment. The probabilistic approach is based on the flood frequency analysis of the observed annual maximum peak flood data. Another alternative of estimating the frequency based floods is to carryout frequency analysis of rainfall data and convolute the design rainfall excess that is excess rainfall of the desired frequency with the unit hydrograph or some rainfall-runoff model appropriate to the catchment. After recent advances in the hydrometeorological techniques during last two decades, methods of flood estimation for design of the various types of the hydraulic structures have been considerably improved. These techniques use meteorological theories and concepts for estimation of probable maximum precipitation or the standard project storm from which losses are deducted for computation of the rainfall excess. The rainfall excess is then transformed into the probable maximum flood or standard project flood. In the absence of the observed records of stream flow data, especially for the moderate to small size catchments, the methodology involving the use of design rainfall that is excess-rainfall of desired frequency and the unit hydrograph is generally adopted. Considerable work has been done relating to depth area duration analysis of the storms of large durations (one day or more). However, relatively little work is reported in literature for the storms of durations shorter than one day.

In this report, an attempt has been made to develop the depth area duration (DAD) and depth duration frequency (DDF) curves for various short durations i.e. less than one day rainfall for the Punpun basin upto Hamidnagar gauging site of Bihar. The excess-rainfall values of the various return periods pertaining to the design storm duration have been convolute ' with the unit hydrograph of the catchment and floods for the various return periods have been computed. Further, a comparative study has been carried out for examining the relationship between frequency of rainfall and frequency of floods. The study has been carried out by Mr. N.G. Pandey, Scientist 'B', Mr. Rakesh Kumar, Scientist `E1' and Mr. B. Chakravorty, Scientist `C' of the Institute. Technical assistance has been provided by Mr. A.K. Sivadas, Technician.

**(K.S. Ramasastri)**
**DIRECTOR**

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# ABSTRACT

Estimation of flood peaks for small catchments are required for water resources planning, flood forecasting, design of various drainage systems, flood control and design of hydraulic structures etc. For this purpose, one of the simple and widely used techniques is based on design excess-rainfall and unit hydrograph approach. In this technique, the design excess-rainfall is convoluted with the unit hydrograph appropriate to the catchment for estimation of flood hydrograph resulting from the design excess-rainfall. For small catchments which have time of concentration in a few hours, long term short interval rainfall data are required for estimation of design storms.

In this study, an attempt has been made to study the characteristics of short interval rainfall data of the Punpun river basin lying in Bihar, which has been identified as the representative basin for the NIH-Centre for Flood Management Studies, Patna. The areal extent of the Punpun basin is 8,630 $km^2$. There are 11 raingauge stations located in the Punpun river basin, out of which only Gaya station is having self recording raingauge. Using the hourly rainfall data of Gaya raingauge station, the daily rainfall data of rest of the 10 ordinary raingauge stations have been converted into the hourly data. Thiessen polygon method has been used for computing the average areal rainfall for the study area. Depth area duration (DAD) and depth duration frequency (DDF) curves have been prepared for the study area. The DAD curves have been developed for various durations viz. 1 h, 2 h, 3 h, 6 h, 9 h, 12 h, 18 h and 24 h. For the aforesaid durations DDF curves have also been prepared for different return periods i.e. 5, 10, 25, 50 and100 years using the L-moment based general extreme value (GEV) distribution. Using the excess-rainfall of the design storm duration, floods of the various return periods have been estimated for the sub-basin defined by the bridge number 462 of the Punpun basin. For this purpose, the DDF curves developed for the Punpun basin upto Hamidnagar gauging site, based on the data of five raingauge stations and the unit hydrograph of the sub-basin defined by the bridge number 462 have been utilized. The average annual maximum rainfall excess data of the design storm duration for the various years have also been convoluted with the unit hydrograph and the annual maximum flood peaks have been calculated for the respective years. The annual maximum peak flood series derived based on this procedure has been used for estimating the floods of various return periods. The floods of various return periods computed by frequency analysis of rainfall and frequency analysis of annual maximum peak floods have been compared.

# Chapter 1

# INTRODUCTION

The analysis of rainstorms is of prime importance for planning and design of water resources projects. In the absence of actual records of stream flow data, analyses of rainstorms are useful for estimation of floods as well as the yields of river catchments. Estimation of design storm is very important for estimation of design floods for various types of hydraulic structures. Rainfall is expressed in terms of depth and its analyses give depth duration (DD), depth area duration (DAD), depth duration frequency (DDF) curves which may be used to estimate the design storm depth. Frequency analysis of rainstorms provides the depth of rainfall for assigned return periods. Enormous amount of work has been done relating to DAD analysis for large durations (one day or more). However, little work is reported for storms having shorter duration (less than one day). Short interval rainfall analysis is essential for estimation of floods for smaller catchments viz. less than 5000 $km^2$ in size since the time of concentration for such catchments would be a few hours. Moreover, for smaller catchments, discharge data are rarely available which are required for design of small dams, culverts, drop spillways etc. After advances in the hydrometeorological techniques during last three decades, methods of flood estimation for the safety of dams have greatly changed. These techniques use meteorological theories and concepts to estimate probable maximum precipitation (PMP) and probable maximum storm (PMS) from which losses are deducted to compute the rainfall excess. The rainfall excess values are then transformed into probable maximum flood (PMF) hydrograph, Rakhecha et al. (1996).

For designing a structure at a site where observed flood data are available, a choice must be made between some form of flood frequency analysis or using method based on design rainfall. Flood frequency analysis gives a direct estimate of flood for desired return period, but rainfall records are generally available for longer period than flow records, and are having greater spatial consistency in the immediately surrounding region. As per the Indian design criteria, frequency based floods find their applications in estimation of design floods for almost all the types of hydraulic structures viz. small size dams, barrages, weirs, road and railway bridges, cross drainage structures, flood control structures and small size dams. For designing of large and intermediate size dams, probable maximum flood (PMF) and standard project flood are adopted respectively. 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 probability 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 lesser accuracy.

The design criteria for various structures like spillway capacity, design flood estimation for barrages and weirs (ungated headworks), flood estimation for road and railway bridges, cross drainage structures on irrigation networks and flood control schemes have been developed by India. The details of the Indian Standards are given in Appendix-E. The design flood estimation is based whether on flood frequency or standard project flood (SPF) or probable maximum flood (PMF) has been specified in the design criteria.

Flood plain management and designs for flood control works, reservoirs, bridges and other investigations need to reflect the likelihood or probability 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 the first and vital step in designing process for a large variety of water resources development works. It is of course a hypothetical event that represents rare occurrence. 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 rainfall runoff process. The design rainfall comprises of three components viz. design rainfall magnitude, its time distribution and the spatial distribution.

While trying to estimate the design flood, a balance regarding relative merits of frequency studies of observed floods versus use of design storm is to be made. 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 may not be applied widely especially to small catchments because most of the streams and rivers generally happen to be ungauged.

The present study deals with analysis of short duration rainfall that would be used for estimation of design flood for small catchments. The study has been carried out for Punpun river basin, which has been identified as the representative basin by NIH centre for Flood Management Studies, Patna. The basin is having 10 ordinary raingauge stations (ORG). One self recording raingauge (SRRG) station located at Gaya lies outside the basin and it is having hourly rainfall data for15 years from 1975 to 1989. The daily rainfall data of the ORG stations have been converted into hourly data following the pattern of the SRRG station located at Gaya. The objectives of the present study are:

i. Evaluation of the characteristics of short duration rainfall i.e. derivation of DDF, DD and DAD curves.
ii. Computation of floods using the excess-rainfall values of the various return periods for the Punpun sub-basin upto Hamidnagar using the unit hydrograph method.
iii. Comparing the relationship between frequency of rainfall and frequency of flood.

# Chapter 2

# REVIEW OF LITERATURE

A design storm is an estimate of rainfall amount and its distribution over a given project catchment accepted for use in determining the design flood. Design storm is commonly used for estimating the spillway design flood. Magnitude of the storms are based on the probable maximum storm (PMS) or the standard project storm (SPS). However, the design storm used for other hydraulic structures such as bridges and culverts, flood plain zoaning, urban drainage systems and economic evaluation of flood protection structures etc. pertaining to small catchments (<5000 $km^2$.) is generally of specific frequency/return periods.

The probable maximum precipitation (PMP) is defined as theoretically the greatest depth of precipitation for a given duration physically possible over a given size of area at a particular geographical location and at a certain time of the year (WMO 332-1986). However the probable maximum flood (PMF) is defined as the flood that would result from the most severe combination of critical meteorological and hydrological conditions physically possible over the catchment. The PMP provides an estimate of the upper limit of precipitation (rainfall including hailstorm, snow etc.) whereas the PMS is the upper limit of rainfall only. The SPS is the most severe rainstorm which has actually occurred over the catchment during the period of available records. The PMP is used where the failure of structures will lead to significant economic loss or loss of life. The SPS is used where the economic investment in the structure is small and risk in loss is either very small or does not exist. The SPS is subjected to moisture maximization and transformed into PMS which is further analysed for design storm estimation.

## 2.1 Significance of Short Interval Rainfall

The three important rainfall characteristics are rainfall depth, duration and its areal spread over the basin. For small catchment having basin lag of few hours, short duration ( < 24 h) analysis is necessary. In India due to limited availability of autographic rainfall records, short duration rainfall data are rarely available. Hourly rainfall data from self recording raingauge stations are analysed to estimate storm parameters. To design culvert, bridges, drop spillways etc. for small catchments where time to peak of hydrograph is in few hours, frequency analysis of raindepths observed from the severe annual maximum storms actually occurred of that particular duration gives more accurate estimation of design storm. Central Water Commission (CWC) has prepared flood estimation reports for small catchments for different subzones of India.

### 2.2 Estimation of Design Storm Depth

Severe rain storms in a meteorologically homogeneous region surrounding a project basin form much needed historical evidences on which the design storm depths (SPS/PMS) are based. This is done by surveying the long series of daily weather charts, weekly and annual reports for storm tracts and details of storms. The daily annual maximum rainfall and corresponding SRRG data pertaining to the specific region of interest aid in selecting the severemost rain depths. The depth duration curves are the indicator of average rain depths in different durations. This pertains to the catchment characteristics. Curves joining the maximum depths of different durations make the enveloping curve which helps in selecting severemost storm. This severemost storm is further analysed. For short duration analysis isohyetal maps are prepared for different hourly durations ( e.g. 1, 2, 3, 6, 9, 12, 18, 24 h ) which have the primary function of locating the areal extent of a storm with maximum real raindepths and its duration. The depth area duration (DAD) curves prepared are the indicators of actual storm characteristics in situ.

#### 2.2.1 Depth area duration (DAD) relationships

The relationship among the three parameters is stated below

- In general the depth decreases as the area increases.
- Depth increases as the duration increases.
- In storm centered relationship, depth decreases moving outwards from the storm centre.

As per CWC (1993) the existing practice is to use observed maximum rainfalls in and around the catchment for the catchment areas upto 50.km$^2$. Design storm depths from DAD curves for 50-500 km$^2$ and transposed depths beyond 500 km$^2$. These are however subject to the individual judgement depending upon the shape and size of the catchments). It is also recommended that:

- Point PMP for catchment area upto 50 km$^2$.with basin lag less than 2 h and nearly circular. However for SPS point rainfall is recommended even upto 100 km$^2$ where elongation ratio (length/breath) is less than 1.5.
- Apply DAD curves for catchments ranging 50-500 km$^2$.with elongation ratio not more than 1.5.
- DAD can be further applicable upto 1000 km$^2$ area if the project shape matches with the isohyetal pattern.
- For conditions other than those specified like elongated catchments storm transposition is recommended.

## 2.3 Storm Transposition

The transfer of storm parameters identified at their places of occurrences to the places where they could occur is known as storm transposition. The storm transposition is limited to meteorologically homogeneous region. There are many areas that have not experienced severe storms observed in adjacent areas and hence transposition of severe storms is done to supplement the inadequate records.

Fixing limits to storms for their transposition is one of the most important aspects in a design storm study. The guide to hydrometeorological practices (WMO-332) suggests that:

- Areas within the transposable limits may have similar but not identical topographic and climatic characteristics throughout (Para 2.5.1)
- It is essential to determine maximum limits of seasonal transposition alongwith geographical limits since the storm mechanism may be changing beyond 15 days on either side of the storm period (Para 2.3.1).
- In temperate latitudes several lakhs $km^2$ can be meteorologically homogeneous. Contiguous homogeneity of such large areas are not possible in tropical regions (Para 6.1.5).
- Series of depth duration values over a catchment for a long period may form part of the historical evidences to avoid unrealistic exposure to certain parts of the catchments and help in obtaining realistic estimates of SPS/PMS for large catchments.
- Transposition involving elevation differences more than 800 m is generally avoided regardless an elevation adjustment is used (Para 2.6.2 and 2.6.3) because of their dynamic influence on storms.
- Limitation is placed on the rotation of displacement of isohyetal pattern (Para 2.11.2).

The study area for transposition is considered as $2^0$ latitude * $2^0$ longitude for nearly flat region and $1/2^0 * 1/2^0$ to $1^0 * 1^0$ in mountainous region WMO(1986). Mohile, *et al.* (1983) while studying the 1982 storms of coastal Orissa had also verified it to be adequate.

Coastal storms (storms having centre west of western ghat and east of eastern ghat) are not transposed to catchments located far inland. Storms that occurred in orographic region are not transposed to the plain area and vice versa.

Transposition to catchments larger than 50,000 $km^2$ may results in unrealistic excessive PMP estimates (para 1.3.2). This may result severe unrealistic situation of flooding that is not experienced by the catchment.

## 2.4 Correction and Adjustment Factors

The various correction and adjustment factors applicable to the rainfall data are described below.

### 2.4.1 Clock-hour correction

Rainfalls are recorded at fixed clock hours at 8.30 AM of each observational day. This rainfall represents the period 8.30 A.M of previous day to 8.30 A.M of recording day. This may not be the indicator of maximum rainfalls of 24 h., 48 h. etc. IMD has recommended a multiplicative factor of 1.15 based on the studies of Harihar Ayyar on the basis of SRRG data. Recent studies (Rao, 1991) based on the SRRG data of 171 storms indicated that the average correction needed is addition of 35 mm rainfall to the daily observational records.The lower bound expressed as R1DAY + 35 mm and the upper bound as R2DAY * $(0.5)^m$ .

| | | |
|---|---|---|
| where, R1DAY | = | Observed one day rainfall recorded at 8.30 A.M |
| R2DAY | = | Observed two day rainfall recorded at 8.30 A.M |
| m | = | 0.175+ 0.0004*R2DAY |

It is further recommended in CWC (1993) that:

- Correction for point rainfall conversion from observational day to 24 h. for PMP rainfall value 50 mm may be added.
- Where the design depth is obtained for catchments having areas 100-5000 $km^2$. from DAD analysis or from storm transposition 35 mm may be added to 1 day areal rainfall depth is adequate. No such correction is necessary for 2 day and 3 day areal depths.
- No clock hour correction is required for catchment areas above 5000 $km^2$.

### 2.4.2 Areal correction factor (ACF)

Areal correction factors (ACF) are applied to point PMP values when point rainfall are made use of in the design storm study to get areal rainfall over the catchment. ACF's are applied to percentage of time distribution obtained from point rainfall data. It has been also recommended in CWC (1993) that:

- Point rainfall value need no reduction upto 50 $km^2$ area of catchments whose basin lags are less than two hours. Application of point rainfall as SPS is also recommended for catchments upto 100 $km^2$ if the elongation ratio ( L/B )is less than 1.5.

- In India record of SRRG data being limited it is not possible to derive storm centered relationship within storm duration less than 24 h DAD curves of severe storms in the region may be used for 1 day areal rainfall from 1-day point SPS/PMP.

### 2.4.3 Moisture maximization factor (MMF)

Moisture maximization factor (MMF) is used to maximize the standard project storm to PMS. IMD recommends MMF of 1.2 to 1.4 for various states of India based on persisting dew point data during monsoon season, which is to be multiplied with SPS to arrive at PMS. The MMF also has been found to be highly variable due to the subjectivity in selecting the representative station. The network of dew point data observation stations in the country is extremely wide and inadequate, resulting in significantly differing results from the selected representative stations. It is essential that the stations located in the storm path between the moisture source and the project catchment are only selected for MMF.

In the light of the above observations a better alternative seems to adopt a value of 1.25 as the moisture replenishment factor (MRF) to make the moisture adjustment needed to transform the SPS values into PMS values CWC (1993).

## 2.5 Temporal Distribution Pattern of Design Storm Depth

This is to specify how the rainfall is distributed in successive time increments. To arrive at realistic time distribution, WMO (1986) stipulates the following:

- Sequence should be in accordance with the storm characteristics of the project region.
- The maximum summation of increments for any given duration shall be less than or equal to the design storm depth for the same duration.
- The increments of rainfall should be areal rainfall value.

The procedure involves to examine the number of rainbursts, their durations and intensities within each of mass curves obtained from autographic rainfall records. Since the mass curves exhibit a great variety of temporal patterns, it is customary to obtain the average/ envelop distribution pattern of consecutive maximum within storm duration of 24/48/72 h so as to obtain maximum probable short duration depths.

In India usually the SRRG data for short duration storms (< 24 h) are inadequate. Whenever, available it is not possible to study the patterns of areal rainfall distribution. Secondly, storms of the level of PMS are uncommon. Further point rainfall distribution is independent of catchment size in which the raingauges are located.

Central Water Commission based on various studies made on storm time distribution pattern, the depths within storm durations ( t ) of T hour storm has been approximated to Lth order distribution. After an in-depth study the following model known as Lth order storm model ( LORDS- I model) for SPS/PMS is formulated and recommended by CWC.

$$Rt = R24\ ( t / 24 )^{L}$$
$$L=0.8^{*}\ (R24 / R24\ MAX)^{0.5}$$

Where, R24 = the 24 hour areal design rainfall depth for any day within the design storm duration, R24 MAX = maximum 1 - day rainfall ever recorded in and around the catchment. This point rainfall needs to be converted to 24 hours point PMP using clock hour correction and necessary correction by moisture replenishment factor. This point PMP is further corrected to areal PMP by using regional curves. In the absence of regional curves, 1-day depth area curve of storms for the project catchment with minor adjustment for transposition may be applied. Since DAD values are applied for small catchments (< 1000 $km^2$), the value of L may be restricted to a maximum of 0.8.

Whenever, SRRG data are available temporal design storm depth may be based on average distribution of maximum consecutive hourly rainfalls. For PMP estimates of a basin the problem may be referred to IMD. India Meteorological Department provides PMP alongwith its temporal distribution for design flood calculations as per the recommendations of WMO.

## 2.6 Hyetograph of Design Rainfall

The SPS/PMS obtained are in the form of accumulated depths of the given duration. These are to be brought in chronological sequencing of rainfall increments. Such critical sequencing of rainfall blocks with t-h duration (the unit duration assigned to the unit hydrograph ordinates) are essential to convolute for the purpose of SPF/PMF assessment. The critical sequence of rainfall increments can be obtained by trial and error. Alternatively, increments of precipitation are first arranged in a table of relevant unit hydrograph ordinates such that:

- The maximum rainfall increment is against the maximum UH ordinate.
- The second highest rainfall increment is against the second largest UH ordinate and so on.
- The sequence of rainfall increments arranged above is reversed, with the last item first and first item last. The new sequence gives critical sequence of the design storm.

Generally duration of severe spell of rainfall possible within severemost long duration storms is 10 - 15 h. It is essential to verify the maximum rainfall intensities within 24 h duration before critical sequencing. It is recommended to represent the design hyetograph in two bells per day CWC (1993).

## 2.7 Design Loss Rate

The $\phi$- index approach for loss rate is a very simple tool for runoff estimation. $\phi$ -index is the average rainfall above which the rainfall volume is equal to the runoff volume. This value is found by treating it as a constant infiltration capacity. If the rainfall intensity is larger than the $\phi$-index the difference between rainfall and infiltration in an interval of time represents the runoff volume or rainfall excess (ER). As per CWC (1969) a minimum loss rate of 1 mm/h throughout the storm or 1 mm/h for the first 12 h, 0.75 mm/h in the next 12 h and 0.5 mm/h thereafter may be adopted. It is sometimes observed that while applying the above loss rate, the overall runoff ratio (after subtracting the loss rate) exceeds 95%. Such a situation in the field is highly unrealistic for medium and large catchments. It is therefore further recommended by CWC (1993) to apply loss rate of 1-2 mm/h depending upon catchment characteristics and nature of vegetation. Flood estimation report for Sone (Subzone- 1d) by CWC (1987) has recommended loss rate of 2.5 mm/ h for subzone 1 (d).

## 2.8 Application of Storm Analysis

The estimation of design storm parameters are used as the basic input for design flood estimation. This is subsequently used for determination of spillway capacity of dams, bridges, aqueducts and various water resources projects etc. 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 hydrograph 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. (1985).

The approach of flood estimation using design rainfall has some advantages over the frequency analysis of observed floods. The different parameters affecting the flood 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 rainfall are generally available at a larger number of rain gauges. Also, extreme rainfall values are more easily defined from physical consideration. The various methods of flood estimation are mentioned below.

## 2.9 Methods of Flood Estimation

The following approaches are used for estimation of design floods depending upon data availability, importance of the study, computation facilities and the design criteria applicable.

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.

The details of methods of flood estimation are available in literature. In this report, flood frequency analysis and unit hydrograph approaches have been used and the same are briefly reviewed below.

### 2.9.1 Frequency analysis

The frequency is the estimate how often a particular event will occur. Our aim is to assign the return period and the estimate the rainfall depth or flood that may be equaled or exceeded. 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.

*Methods of parameter estimation*

Some of the commonly used parameter estimation methods for most of the frequency distributions are:

(i) Method of least squares
(ii) Method of moments
(iii) Method of maximum likelihood
(iv) Method of probability weighted moments
(v) Method based on principle of maximum entropy
(vi) Method based on L-moments

The method of moments has been one of the simplest and conventional parameter estimation techniques. In this method, while fitting a probability distribution to a sample, the parameters are estimated by equating the sample moments to those of the theoretical moments of the distributions. It is found that the numerical values of the sample moments can be very different from those of the population from which the sample has been drawn, especially when sample size is small and the skewness of the sample is considerable. Further, estimated parameters of distributions fitted by method of moments, are not very accurate.

There have been quite a number of attempts in literature to develop unbiased estimates of skewness for various distributions. However, these attempts do not yield exactly unbiased estimates. In addition the variance of these estimates is found to increase. Further, a notable drawback with conventional moment ratios such as skewness and coefficient of variation is that, for finite samples they are bounded, and will not be able to attend the full range of values available to population moment ratios. Probability Weighted Moments (PWM), a generalization of the usual moments of a probability distribution, were introduced by Greenwood. There are several distributions whose parameters can be conveniently estimated from their PWM's. Landwehr *et al.* (1979) investigated the small sample properties of PWM estimators of parameters for Gumbel distribution and found them superior in many respects to the conventional moments and maximum likelihood estimators.

Hosking *et al.* (1990) have defined L-moments, which are analogous to conventional moments, and can be expressed in terms of linear combinations of order statistics . L- moments are capable of characterizing a wider range of distributions. A distribution may be specified by its L-moments, even if some of its conventional moments do not exist ( Hosking, *et al.*, 1990). The L –moments are estimated by linear combinations of order statistics. Practically, L- moment estimators and its ratios are less biased in estimation of population parameters from sample statistics and have nearly normal distribution characteristics. Main advantage of L-moments is that they suffer less from the sample variability, they are more robust to outliers in data. Moreover, for product moment log transformation of sample values can overemphasize small values. This defect can be avoided in L- moment. It is also to be noted that sample estimators of L-moments are always linear combinations of the ranked observations, while the conventional sample moment estimators require squaring and cubing the observations respectively, which in turn, increases the weightages to the observations away from the mean, thus resulting in considerable bias.

The first L-moment is the sample mean, a measure of location. The second L-moment is a measure of dispersion of data about the mean. By dividing the higher order L-moments by the dispersion measures, L-moment ratios are obtained. Measure of L- skewness is third/second moment. Similarly, measure of L- Kurtosis is fourth/second moments. The L-moment statistics of a sample reflect every information about the data. L-moment ratio diagram (L-skewness *vs*. L-kurtosis) can be used to identify the distribution. The statistics is plotted on the L-moment ratio diagram and the distribution nearest to the plotted point is selected as the best fit distribution for the data. On choosing the distribution, parameters estimation of the said distribution is undertaken for further analyses. Major advantage of L-moment ratio diagram is that one can easily compare the fit of several other distributions using a single graphical representation.

There are certain text book distributions like Lognormal, Pearson (Type III) and Gumbel extreme value distribution etc.that can be fitted to the data sets for peak flood discharge, rainfall for design storm estimation etc. This performs fairly well on availability of data for longer periods which are difficult to get. This also severely underestimate the extreme quantiles (design storms, design floods). On the other hand General Extreme Value, Generalized Pareto, Generalized Logistics etc. frequency distributions are robust and best suited in hydrological problems. L- moment ratios are calculated and plotted in the L-moment ratio diagram to select the probability distribution suited best for the sample data. The L-moments are employed to find out the parameters of the various frequency distributions. The General extreme value distribution (GEV) has been very widely adopted and found to be the robust distribution in a number of frequency analysis studies and the same has been used in this study also. The parameters of the GEV distribution have been estimated using L-moments approach.

***General extreme value distribution (GEV)***

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

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

where, u, $\alpha$ and k are location, scale and shape parameters of GEV distribution respectively. The value of shape parameter 'k' ranges between -0.6 to +0.6. Depending on the value of shape parameter (k), the GEV distribution is classified as EV-I(Gumbel) when k=0, EV-II(Log Gumbel or Frechet distribution) when 'k' value is negative and EV-III when 'k' value is positive. EV-II distribution has a lower bound equal to u + $\alpha$ /$\kappa$ and is used for flood frequency analysis.

### 2.9.2 Unit hydrograph analysis

The unit hydrograph can be derived by analysing the excess rainfall and direct surface runoff of various storms for the gauged catchments. Generally two types of approaches viz. non-parametric system analysis approach e.g. Collin's method and matrix method based on trial and error procedure and parametric system synthesis approach e.g. Clark IUH model (Clark, 1945) and Nash IUH model (Nash, 1957) are adopted for derivation of unit hydrograph. Sometimes, non-parametric methods do not converge and provide estimates of unit hydrographs with unrealistic shape and negative ordinates. In such a situation, one has to make subjective judgments for preserving the shape of the unit hydrograph with required unit volume. Further, methods based on non-parametric system analysis approach require development of more than five regional unit hydrograph relationships in order to derive the synthetic unit hydrograph for an ungauged catchment. While, the parametric system synthesis approaches viz. Clark IUH model and Nash IUH model require development of only two regional unit hydrograph relationships and there is no subjectivity involved in preserving the unit volume of the unit hydrograph. Thus, the methods based on parametric system synthesis approach overcome the deficiencies associated with the non-parametric methods. Small Catchment Directorate of Central Water Commission (CWC), Research Dèsigns and Standards Organizations (RDSO), Roads Wing of Ministry of Transport and Indian Meteorological Department (IMD) have jointly carried out regional unit hydrograph studies for various Indian hydrometeorologically homogeneous subzones. Specific subzones have been identified by dividing the whole of India into 26 hydrometeorological homogeneous subzones. 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 (IUH) models such as Nash and Clark IUH models etc. ( e.g. Singh & Kumar, 1991).

# Chapter 3

## DESCRIPTION OF STUDY AREA

The river Punpun originates from Chhotanagar hills of the Palamau district in Bihar at an elevation of about 300 m and at North latitude of $24^0$ 11' and East longitude $84^0$ 9'. It joins the river Ganga near Fatwa about 25 km downstream of Patna covering total distance of 232 km. The river has a number of tributaries joining it mostly from the right bank. Important among these are the Batane, Madar, Mohar and Dardha. All these rivers rise in the Chhotanagpur plateau and are rainfed. They flow only during Monsoon and remain dry for the rest of the year.

The Punpun basin lies between latitudes $24^0$ 11′ to $25^0$ 00′ N and longitudes 84 $^0$10′ to $85^0$ 20′ E. It is located on the right bank of the river Ganga and is bounded by the Sone river system on its west and Kiul-Harohar-Falgu river system in the east. On its northern side it is the river Ganga and on its southern side, it is bounded by Chotonagpur hills. The drainage map of Punpun river basin alongwith the locations of rainaguge stations is shown in Fig. 3.1.

The Punpun basin is roughly trapezoidal in shape. The length of the catchment is about 180 km and average widths in the upper and lower reaches are 60 km and 25 km respectively. The total catchment area of the basin is about 8,630 $km^2$. The general drainage direction of the basin is from south-west to north east. The catchment falls in the state of Bihar and Jharkhand that covers the districts of Patna, Gaya, Aurangabad, Hazaribagh and Palamau. The percentage area of each district falling in the basin is 11.25%, 35.25%, 27.08%, 9.39% and 16.41% respectively.The uppermost catchment of Punpun basin falls in the districts of Palamau and Hazaribagh in Chotanagpur hills is mostly covered under forest. The lower part lies in the districts of Aurangabad, Gaya and Patna are having mild slopes. The elevation varies from 300 m near origin and about 50 m at its outfall into the river Ganga.

A sub–catchment of Punpun basin upto Hamidnagar gauging site having drainage area of approximately 3,800 $km^2$ has been identified for studying the characteristics of the short interval rainfall data. As an application of rainfall analysis for flood estimation, synthetic unit hydrograph of a smaller sub-catchment of the Punpun basin upto Hamidnagar represented by the railway bridge number 462 having an areal extent of 516.5 $km^2$ has been considered. The study area falls in the Sone subzone 1(d) (CWC, 1987).

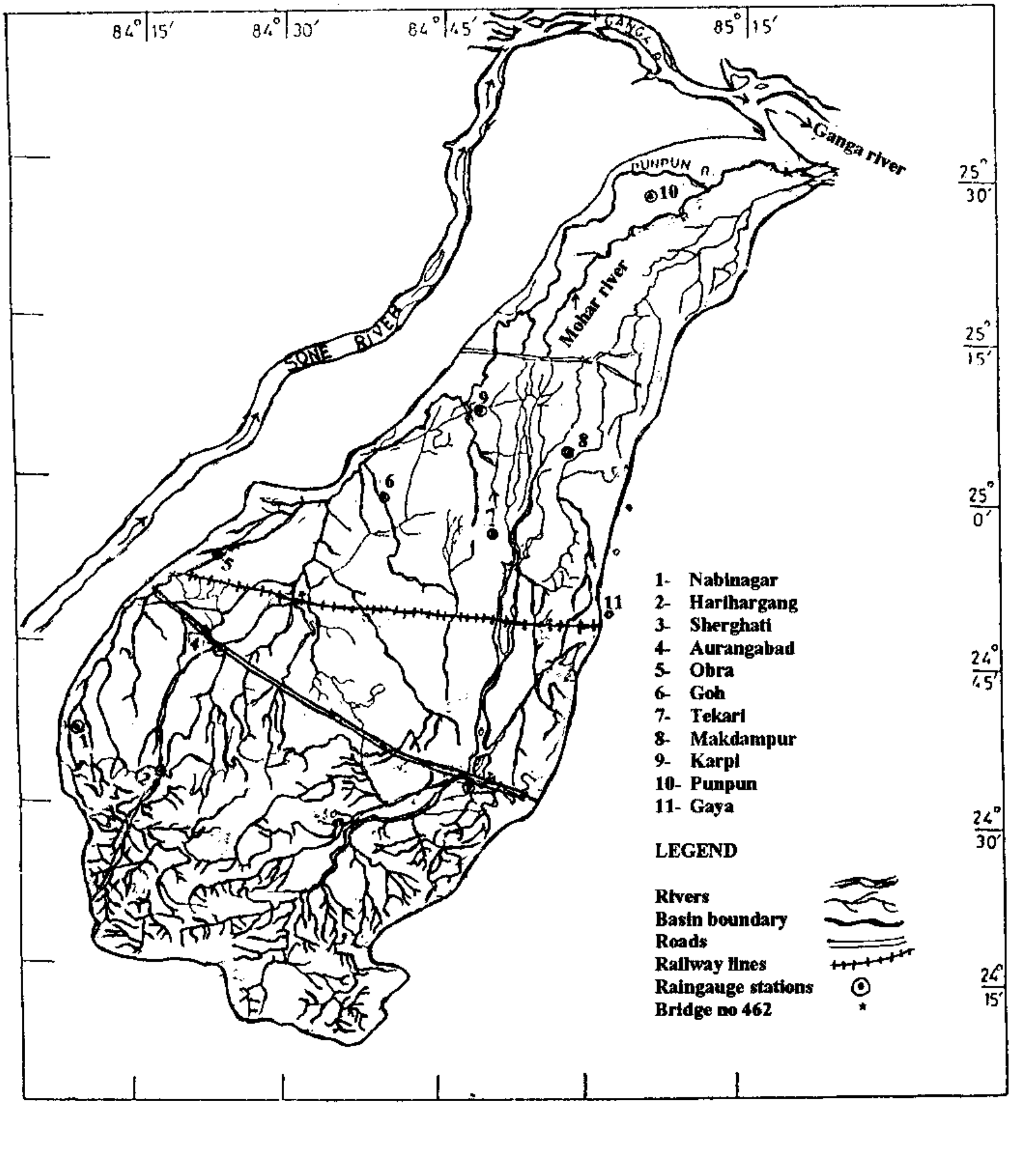


**Fig. 3.1 Location map of raingauge stations for Punpun basin**

## 3.1 Geology

The geology of the basin varies from granite, gneiss, charnokitess in the hills to the recent alluvium in the plains. The board soil groups are calcareous and non-calcareous, recent and old alluviums. Brown forest soils, red soils and laterite soils with deep cover in plains and deep to shallow in hills are available.

## 3.2 Rainfall Distribution

The basin receives about 85-87 % of its annual rainfall during the south-west monsoon period occuring from June to September. The average annual rainfall is about 992 mm

## 3.3 Landuse and Geomorphology

The land use pattern of the Punpun basin shows that out of the total area of 8,630 $km^2$. about 5000 $km^2$ is under agriculture, 2500 $km^2$ is under forest, and the remaining area of 1130 $km^2$ is under other uses.

The length of the main channel of Punpun basin is 255 km.. Drainage density (total length of channels of all order to the total area of the basin) is 0.377 km/ $km^2$. The shape parameters such as elongation ratio of 0.5486, watershed shape factor of 1.7267 and unity shape factor of 1.9480 indicate that the basin does not have evenly distributed areas NIH (1994).

# Chapter 4

## DATA AVAILABILITY

The daily rainfall data of 10 ordinary raingauge (ORG) stations viz. Hariharganj, Nabinagar, Sherghati, Aurangabad, Obra, Goh, Tekari, Makdampur, Karpi and Punpun were available in the Technical Report of National Institute of Hydrology ( TR-174 ) entitled 'Hydrological data book for Punpun basin (1974-90)'. Daily rainfall data of Gaya raingauge station were procured from IMD, Pune. The SRRG (hourly) data of Gaya raingauge for the selected storms periods were obtained from IMD, Patna. The SRRG charts of the respective dates were referred and noted. Some old hourly records were obtained from the IMD, Pune.

# Chapter 5

# METHODOLOGY

The missing data of raingauge station in the basin was filled in using the data of neighboring stations. In these calculations normal rainfall was used as a standard of comparison. The normal rainfall is the average value of rainfall at a particular date, month or year over a specified period of 30 years. If the normal annual rainfall at various stations are within 10% deviation of normal annual rainfall of a particular station then simple arithmetic average is followed. Deviations more than 10% are estimated by weighing the rainfall at various stations by the ratio of normal annual rainfall. This is called normal ratio method (Subramanya, 1990).

The normal annual rainfall of the 10 stations are Nabinagar(880mm), Hariharganj(924mm), Sherghati(1061mm), Aurangabad(1080mm), Obra(923mm), Goh(915mm), Tekari(916mm), Makdampur(1020mm), Karpi(942mm), and Punpun(955mm).The study area (Punpun basin) lies in Sone subzone – 1(d). The area is flat, having a mild slope. The maximum elevation difference is 250 m. There is no rain shadow areas. The area therefore may be considered meteorologically homogeneous. Gaya SRRG station is very close to the basin boundary. Normal annual rainfall of Gaya is within 10% deviation of other stations. Since Gaya is the only station having SRRG records it has been considered for hourly rainfall distribution for other 10 raingauge stations in the basin. This temporal distribution of Gaya station was adopted for other stations and converted the daily rainfall of all the stations into hourly rainfall.

Data consistency using double mass curve method was also checked by plotting cumulative rainfall of one station against cumulative rainfall of other 10 stations in a reversed chronological order. A break of slope in the plot indicates change in the rainfall regime of the station in question. This is then corrected multiplying by a slope correction factor.

## 5.1 Storm Selection

Selection of severe storms in a year may be done by selecting qualifying storms. Qualifying storms should have rainfall more than the average depth of all selected storms plus 10-20% of average depth. Annual maximum rainfall of each station among 10 ORG stations spread over 15 years (1975-1989) of the study area was sorted out and the corresponding dates (10*15 storms) were noted. The SRRG records of Gaya station corresponding to the storm dates were checked. The preceding and succeeding days of these 150 storm dates were also verified in order to note the severity of storms. In this connection annual daily maximum rainfall of Gaya station was also incorporated. Thus the annual severest storm out of 11 selected storm dates including Gaya station was identified. This exercise was repeated for 15 years (1975-1989) and finally 15 severe most storms (annual maximum) that had actually occurred over the basin were selected for further analysis

Thiessen polygon method was applied to convert the hourly point rainfall values into areal values. This value gave the average depth of rainfall corresponding to different durations over the basin. The hourly maximum values of 1, 2, 3, 6, 9, 12, 18 and 24 of the candidate storms were found out. The historic data of 15 rainstorms of 15 years were resorted to the same treatment.

### 5.2 Depth Duration Envelope Curve

The selected 15 storms (annual maximum for 15 years from 1975 to 1989) dates were 7-8/10/75, 15-16/09/76, 7-8/07/77, 29-30/07/78, 17-18/08/79, 01-02/08/80, 16-17/07/81, 21-22/10/82, 05-06/07/83, 28-29/08/84, 29-30/07/85, 05-06/07/86, 06-07/07/87, 10-11/08/88, 15-16/07/89.

The hourly maximum values of durations 1, 2, 3, 6, 9, 12, 18 and 24 hours for the above 15 storms were plotted in graphs duration on X-axis and depth on Y-axis.. The storm which formed the envelope curve (design storm dated 15-16/ 09 / 76 ) was selected for depth- area-duration (DAD) analysis.

### 5.3 Depth Area Duration Curves (DAD)

Mass curves of rainfall for various durations of each station corresponding to the severe most storm date of Gaya were drawn. The point depths of various durations were noted for each station. Isohyetal map of the hourly point rainfalls of all the stations of the basin was prepared and the areas encompassed in storm centered relationship (areal spread outwards from the eye of storm) by each isohyet were noted. The procedures were repeated for 2, 3, 6, 9, 12, 18, and 24 hour durations. Now the average depths were plotted against area for various durations producing DAD curves.

### 5.4 Depth Duration Frequency Curves (DDF)

Frequency analysis of rainfall for design storms and subsequently frequency analysis of design floods has been made by using L-moments for parameter estimation out of sample statistics and adopting the GEV distribution.

### 5.4.1 Frequency analysis

Frequency analysis is the estimation of how often a specified event will occur. Estimation of the frequency of extreme event is often a particular importance. Various distributions are available to model annual maximum streamflow. Moment statistics were widely used as the basis for identifying and fitting frequency distribution but these are associated with higher degree of biasness and algebraic boundedness. Hoskings(1990) found that certain linear combination of Probability Weighted Moments (PWM), called as L-moments could be interpreted as a measure of location, scale , and shape parameter of a probability distribution as the basis to identify, describe, and estimate the distribution. The steps involved in frequency analysis approach are mentioned below.

i. Estimation of L-moment statistics viz. first, second, third and fourth moment, and L-moment ratios (L- skewness and L- kurtosis).
ii. Choosing the best fit distribution by using L-moment ratio diagram and other criteria.
iii. Parameter estimation of the selected distribution.
iv. Estimation of quantile function.

Frequency analysis based on the L-moments approach is robust and less biased in quantile estimation. Estimation of L-moments statistics involves computation of $\lambda_1$, $\lambda_2$, $\lambda_3$, $\lambda_4$, and their ratios $\tau_3$, $\tau_4$ by the following relationships.

$$\lambda_1 \text{ (mean)} = \alpha_0$$
$$\lambda_2 \text{ (standard deviation)} = \alpha_0 - 2\alpha_1$$
$$\lambda_3 = \alpha_0 - 6\alpha_1 + 6\alpha_2$$
$$\lambda_4 = \alpha_0 - 12\alpha_1 + 30\alpha_2 - 20\,\alpha_3$$
$$\tau_3 \text{ (L- skewness)} = \lambda_3/\lambda_2$$
$$\tau_4 \text{ (L-kurtosis)} = \lambda_4/\lambda_2$$

where, $\lambda_1$, $\lambda_2$, $\lambda_3$, $\lambda_4$ are the first, second , third and fourth moment respectively and $\alpha_r$ (r = 0, 1, 2, 3 ) are the probability weighted moments. The relationship between $\lambda$ and $\alpha$ are expressed as under.

$$\lambda_{r+1} = (-1)^r \sum_{k=0}^{r} p^*_{r,k}\ \alpha_k = \sum_{k=0}^{r} p^*_{r,k}\ \beta_k$$

where, $p^r_{r,k}$ is an orthogonal polynomial expressed as:

$$p^*_{r,k} = (-1)^{r-k}\ {}^rC_k\ {}^{r+k}C_k = \frac{(-1)^{r-k}\ (r+k)!}{(k!)^2\ (r-k)!}$$

The parameters of the adopted distribution (for example in case of GEV distribution are u, $\alpha$ and k that is location, scale, and shape parameters) are found out. Once, these parameters are determined, the form of probability density function (pdf), cumulative distribution function (cdf), and quantile function can be found out. For GEV distribution pdf, cdf, and quantile functions are as under.

$$\text{pdf} = f(x) = \alpha^{-1} e^{-(1-k)y - e^{-y}}, y = -k^{-1} \log\{1 - k(x-u)/\alpha\}, \quad k \neq 0$$

$$y = (x-u)/\alpha, \quad k = 0$$

$$\text{cdf} = F(x) = e^{-e^{-y}}, \text{and} \quad x(F) = u + \alpha\{1 - (-\log F)^k\}/k, \quad k \neq 0$$

$$= u - \alpha \log(-\log F), \quad k = 0$$

Range of x varies between u+$\alpha$/k and infinity when value of k is negative indicating that it has a lower bound. This case is applicable for EV-II distribution for peak flow or rainfall data. 'F' denotes the probability of non exceedance that equals to (1-1/T), where T stands for return period.

# Chapter 6

# ANALYSIS AND DISCUSSION

## 6.1 Rainfall Distribution

Selection of severe rainstorms for the study area has been explained in the Chapter 5. The severe rainstorms for 15 years (1975-1989) are given in the Table 6.1. Thiessen polygons have been drawn for the Punpun basin for the 11 raingauges and the same are shown in Fig. 6.1. The Thiessen weights for these 11 raingauges have been computed and are given in Table 6.2.

**Table 6.1: Severe rainstorms (1975-1989) and their hourly distribution for Punpun basin (in mm)**

| Time | Durations / dates | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 07-08 Oct,75 | 15-16 Sept,76 | 07-08 July,77 | 29-30 July,78 | 17-18 Aug,79 | 01-02 Aug,80 | 16-17 July, 81 | 21-22 Oct,82 | 05-06 July, 83 | 28-29 Aug, 84 | 29-30 July, 85 | 05-06 July, 86 | 06-07 July, 87 | 10-11 Aug, 88 | 15-16 July, 89 |
| 9AM | 0.0 | 4.0 | 0.0 | 0.0 | 1.0 | 0.0 | 0.0 | 9.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 10AM | 0.0 | 2.0 | 0.0 | 1.0 | 5.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 11AM | 0.0 | 0.0 | 0.0 | 0.0 | 6.0 | 0.0 | 0.5 | 0.0 | 0.0 | 0.0 | 0.0 | 11.0 | 33.0 | 0.0 | 0.0 |
| 12Noon | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 1PM | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 20.0 | 0.0 | 0.0 | 3.0 | 17.0 | 0.0 | 0.0 |
| 2PM | 0.0 | 6.0 | 0.0 | 0.0 | 24.0 | 0.0 | 0.0 | 0.0 | 50.0 | 0.0 | 0.0 | 0.0 | 9.0 | 0.0 | 0.0 |
| 3PM | 0.0 | 2.0 | 0.0 | 0.0 | 22.0 | 0.0 | 0.0 | 0.0 | 2.0 | 4.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 4PM | 0.0 | 6.0 | 0.0 | 0.0 | 8.0 | 0.0 | 0.0 | 0.0 | 2.0 | 3.0 | 50.0 | 0.0 | 1.0 | 0.0 | 20.0 |
| 5PM | 0.0 | 10.0 | 2.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.8 | 0.0 | 24.0 | 0.0 | 1.0 | 0.5 | 12.5 |
| 6PM | 0.0 | 3.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 6.0 | 0.0 | 0.0 | 0.0 | 1.0 |
| 7PM | 0.0 | 11.0 | 0.0 | 0.0 | 0.0 | 0.0 | 17.5 | 0.0 | 0.0 | 0.0 | 5.0 | 0.0 | 0.0 | 0.0 | 0.5 |
| 8PM | 0.0 | 26.0 | 0.0 | 0.0 | 0.0 | 0.0 | 10.0 | 0.0 | 0.0 | 0.0 | 3.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 9PM | 0.3 | 24.0 | 0.0 | 0.0 | 0.0 | 0.0 | 2.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 10PM | 0.0 | 30.0 | 0.0 | 0.0 | 0.0 | 10.0 | 4.0 | 0.0 | 0.0 | 0.0 | 1.0 | 2.0 | 0.0 | 0.0 | 0.0 |
| 11PM | 0.0 | 16.0 | 0.0 | 0.0 | 0.0 | 40.0 | 20.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 12Midnight | 0.0 | 30.0 | 0.0 | 29.0 | 2.0 | 18.0 | 8.0 | 0.0 | 0.0 | 43.0 | 0.0 | 0.0 | 0.0 | 6.0 | 0.0 |
| 1AM | 0.0 | 40.0 | 0.0 | 30.0 | 0.0 | 4.0 | 6.0 | 0.0 | 0.0 | 60.0 | 0.0 | 10.0 | 1.0 | 0.0 | 0.0 |
| 2AM | 20.0 | 30.0 | 0.0 | 8.0 | 0.0 | 2.0 | 0.5 | 0.0 | 0.0 | 3.0 | 0.0 | 20.0 | 2.0 | 0.0 | 0.0 |
| 3AM | 30.0 | 40.0 | 0.0 | 6.5 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 1.0 | 0.0 | 30.0 | 16.0 | 3.5 | 0.0 |
| 4AM | 2.0 | 6.0 | 0.0 | 0.0 | 4.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 0.0 | 10.0 | 40.0 | 20.0 | 0.0 |
| 5AM | 0.0 | 4.0 | 17.0 | 0.0 | 0.0 | 0.0 | 0.0 | 16.0 | 0.0 | 2.0 | 0.0 | 28.0 | 30.0 | 16.0 | 0.0 |
| 6AM | 0.0 | 0.8 | 10.0 | 0.0 | 0.0 | 0.0 | 0.0 | 8.0 | 0.0 | 0.0 | 0.0 | 12.0 | 20.0 | 5.0 | 0.0 |
| 7AM | 0.0 | 0.0 | 30.0 | 0.0 | 0.0 | 0.0 | 0.0 | 2.0 | 0.0 | 0.0 | 0.0 | 28.0 | 10.0 | 39.0 | 0.0 |
| 8AM | 0.0 | 0.0 | 14.0 | 0.0 | 0.0 | 0.0 | 0.0 | 4.0 | 0.0 | 2.0 | 0.0 | 1.0 | 3.0 | 25.0 | 9.0 |
| Total | 52.3 | 290.8 | 73.0 | 74.5 | 72.0 | 74.0 | 68.5 | 39.0 | 75.8 | 118.0 | 90.0 | 155.0 | 183.0 | 115.0 | 43.0 |

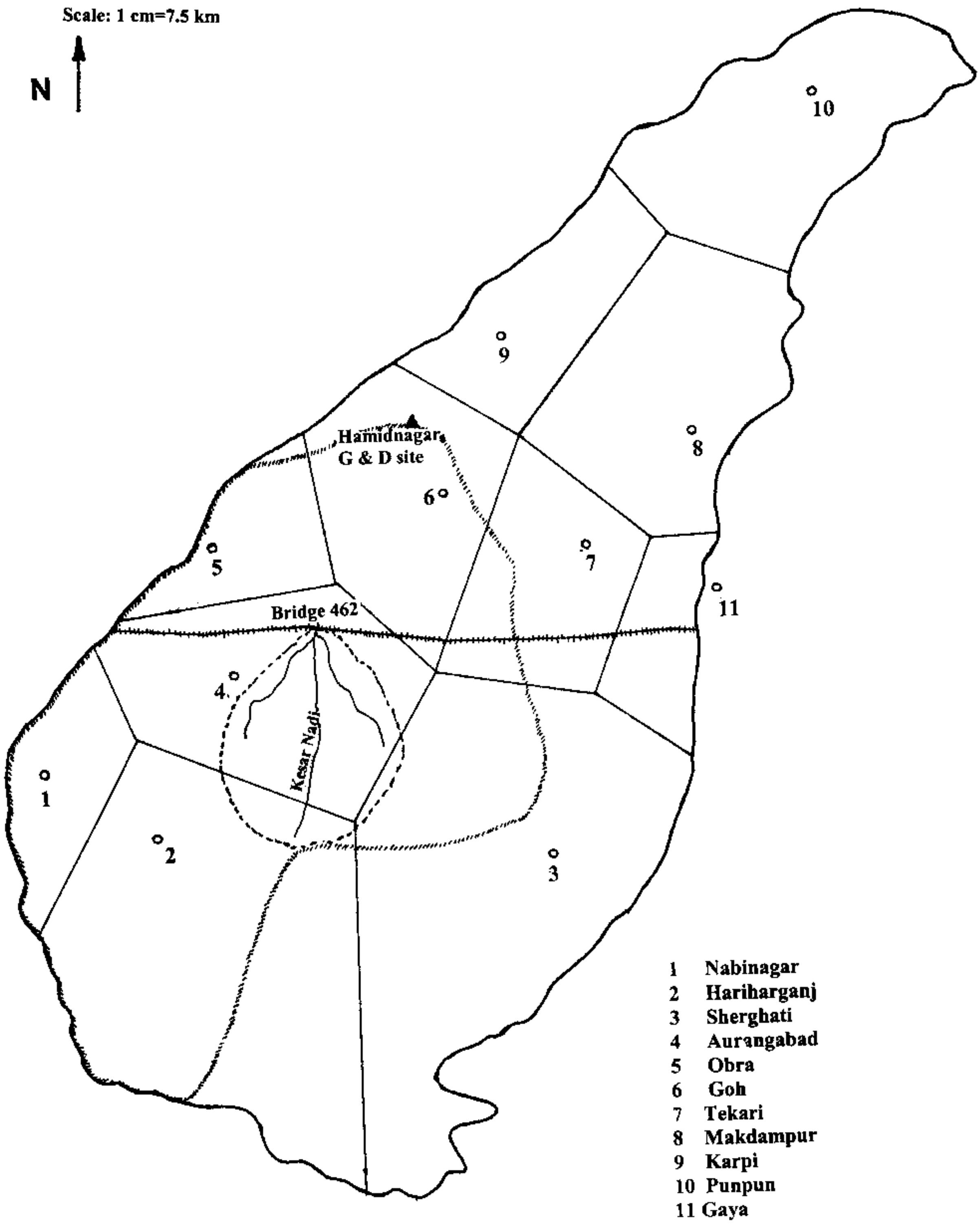


Fig. 6.1 Thiessen polygon for Punpun basin

**Table 6.2: Thiessen weights of raingauge stations for Punpun basin.**

| Station | Number code | Area covered ($km^2$) | Thiessen weight |
|---|---|---|---|
| Nabinagar | 01 | 348 | 0.040 |
| Hariharganj | 02 | 1640 | 0.190 |
| Sherghati | 03 | 1732 | 0.200 |
| Aurangabad | 04 | 877 | 0.101 |
| Obra | 05 | 360 | 0.041 |
| Goh | 06 | 688 | 0.080 |
| Tekari | 07 | 624 | 0.072 |
| Makdampur | 08 | 635 | 0.073 |
| Karpi | 09 | 616 | 0.071 |
| Punpun | 10 | 850 | 0.098 |
| Gaya | 11 | 260 | 0.030 |
| | Total | 8630 | 0.996 |

These Thiessen weights were used to estimate the average areal rain depths. Since all rainfall values were on hourly basis, the areal rainfall becomes hourly rainfall depth.

### 6.1.1 Depth-duration frequency (DDF) curves

From the average areal rainfall distribution, maximum 1, 2, 3, 6, 9, 12, 18 and 24 hourly values of each yearly storm were calculated. The values of these durations were subjected to frequency analysis by applying General Extreme Value (GEV) distribution and parameter estimation using L-moments. The output of this exercise gives the areal depths for specified short durations of rainfall for different return periods *viz.*5, 10, 25, 50 and 100 years for Punpun basin. Curves were drawn between return periods and the computed rain depths for specified durations (Fig. 6.2). These curves are called Depth duration frequency (DDF) curves. The rainfall frequency can be directly applied to estimate design flood using unit hydrograph approach. Synthetic unit hydrograph (SUH) approach is one of the methods to estimate flood for small catchments less than 5000 $km^2$. Since the Punpun basin is more than 5000 $km^2$, Punpun basin upto Hamidnagar G&D site was selected having drainage area of 3800 $km^2$ for the purpose. Maximum rainfall depths for various durations (1, 2, 3, 6, 9, 12, 18 and 24 hourly) for 15 years for Hamidnagar catchment are given in Appendix – B. Depth duration frequency curves

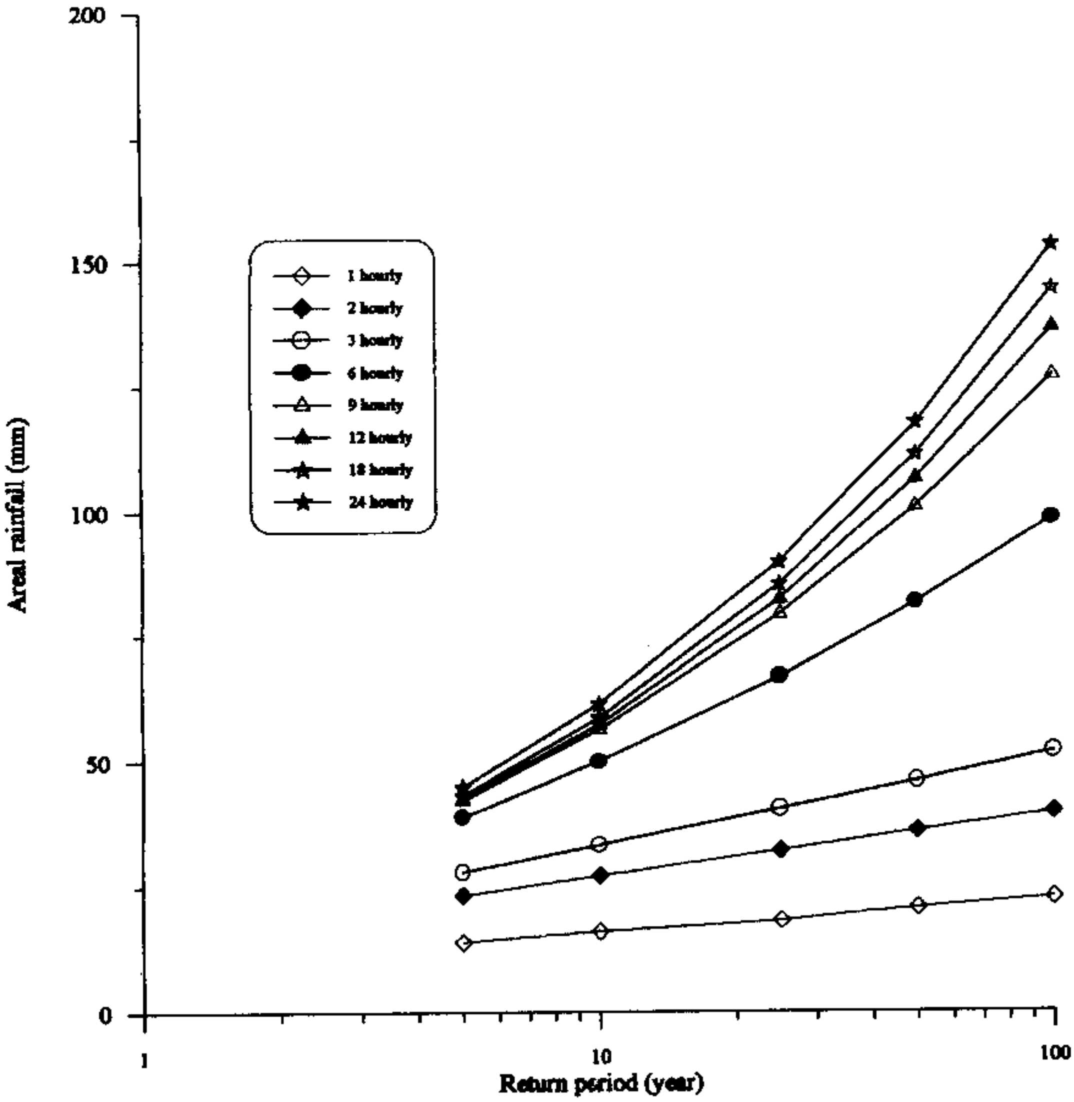


**Fig. 6.2 : Depth duration frequency curves for Punpun basin.**

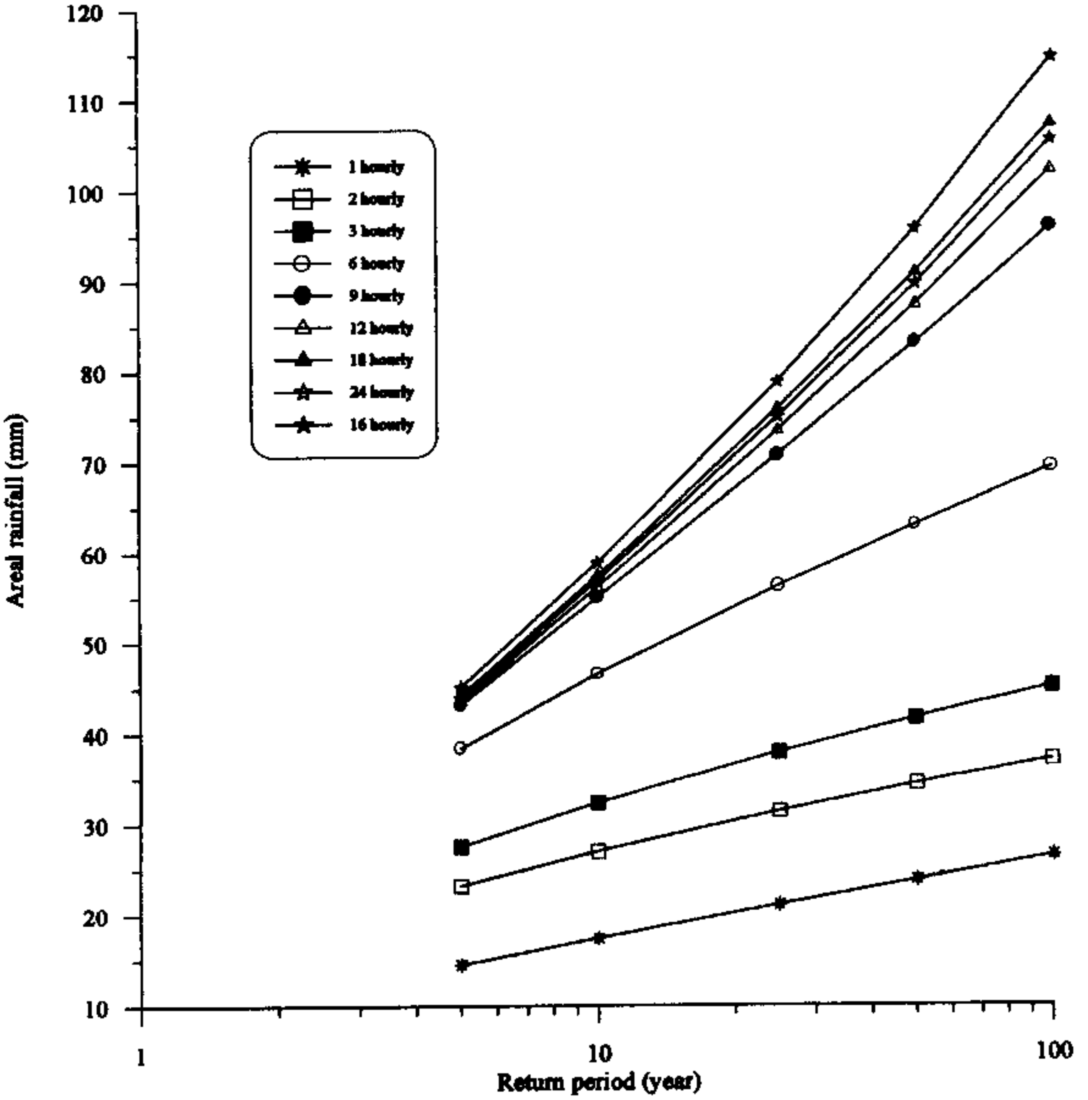


**Fig.6.3 : Depth duration frequency curves for Punpun basin upto Hamidnagar**

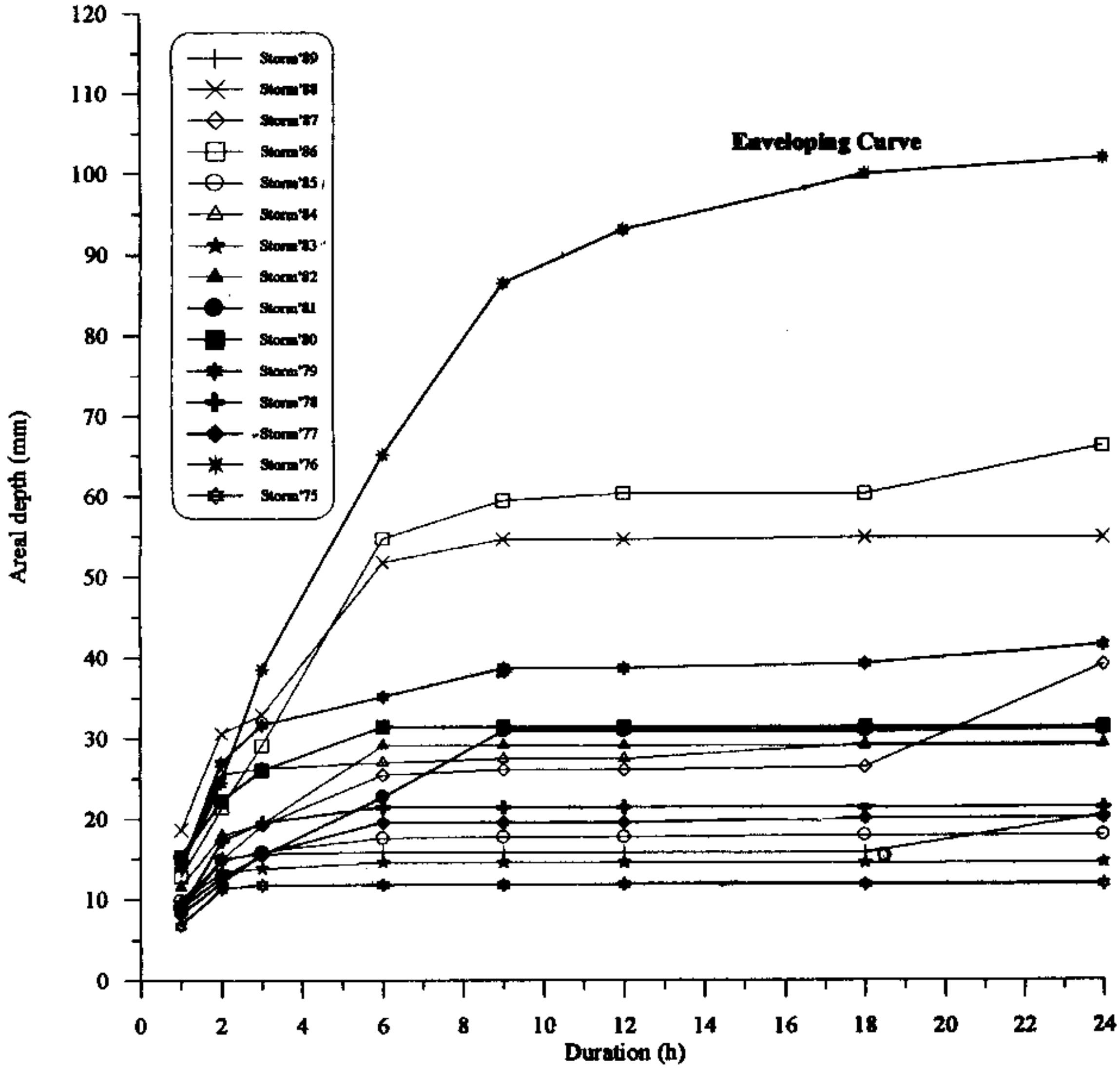


**Fig. 6.4 : Depth duration envelope curves of 15 storms for Punpun basin.**

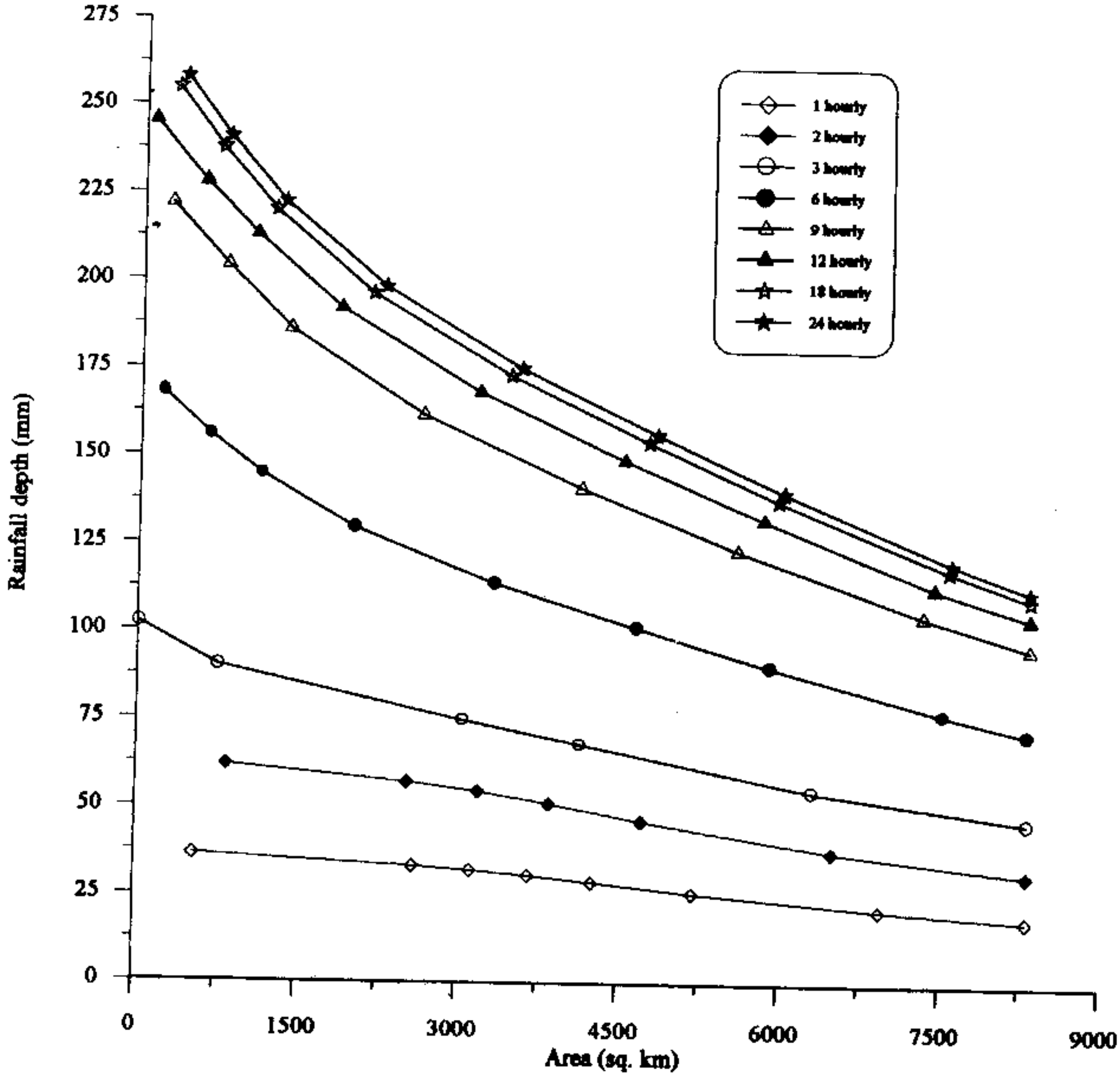


**Fig. 6.5: Depth area duration curves of Punpun basin.**

DDF curves were drawn for catchment upto Hamidnagar G&D site adopting the same procedure as was done for Punpun basin (Fig. 6.3) for same return periods (5, 10, 25, 50, and 100 years). The purpose of DDF curves is to estimate floods of various return periods using the design depths of the desired duration of storm.

### 6.1.2 Depth-duration envelope (DDE) curve

Depth duration (DD) curves are a representation of rainfall for various short durations (< 24 h). DD curves were plotted between eight short durations (1, 2, 3, 6, .....,24 h) and the depths corresponding to these durations for 15 storm events (selected from 15 years) for Punpun basin (Figure 6.4). The Depth Duration Envelope (DDE) curves represent the severe most storm amongst these 15 storms selected for analysis. This exercise is done to find the year in which the severe most storm has occurred. For Punpun basin, the severe most storm has occurred in 1976 (Sept. 15-16) as apparent from DDE curve. This enveloping DD curve would be used for depth area duration (DAD) analysis.

### 6.1.3 Depth-area duration (DAD) curves

The severemost rainstorm that occurred in 1976 in Punpun basin was considered. For DAD analysis we go back to the individual raingauge station for the particular year of severe most storm found from DD curve. Doing so, there would be severe most rainfall depths for eight defined durations for 11 raingauge stations (point data) for 1976. Isohyetal maps for these durations were drawn by following the storm centred relationship (outward spreading of area from the eye of the storm) as shown in figures in appendix – C and the area representing a particular depth of rainfall was found. The DAD curves were drawn between these areas and the corresponding depths for eight defined durations (Fig. 6.5). Generally, DAD curves are drawn to develop the design storm for use in computing design flood of major structures such as dams.

## 6.2 Application of Short Duration Rainfall Analysis

As an application of short duration storm analysis, an attempt was made to use unit hydrograph (UH) approach for flood estimation of small catchments. Sufficient at site discharge data was not available for Hamidnagar sub-basin for derivation of UH. Therefore, sub-catchment comprising railway bridge no. 462 (drainage area 516.5 $km^2$) of Hamidnagar sub-basin was considered for flood estimation for which the catchment characteristics were known CWC (1987) as given in Table-6.3.

**Table 6.3 : Catchment characteristics for railway bridge No. 462.**

| Sl. | Particulars | Descriptions |
|---|---|---|
| 1. | Name of stream | Kasar Nadi |
| 2. | Name of Railway Zone | Gaya-Sone East Bank |
| 3. | Lattitude | $24^0\ 51''\ 00'$ |
| 4. | Longitude | $81^0\ 52''\ 45'$ |
| 5. | Catchment Area | 516.5 $km^2$ |
| 6. | Longest length of main river | 56.35 km |
| 7. | Longest length of river from centre of gravity | 23.51 km |
| 8. | Equivalent stream slope | 2.35 m/km |

Flood estimation was done using synthetic unit hydrograph approach. For such estimation, UH ordinates are required to be convoluted with the rainfall excess. The parameters for drawing the Synthetic UH for bridge no. 462 were available in CWC (1987) (Table 6.4). From these parameters the SUH was drawn and necessary adjustments were made in the rising and recession limb of the SUH by taking its ordinates at 1-h interval to see the depth of runoff as unity. Now, this synthetic UH becomes the 1-h UH of 1cm depth.

**Table 6.4: Parameters of synthetic unit hydrograph for railway bridge No. 462**

| Sl. | Parameters | Magnitude |
|---|---|---|
| 1. | Time to peak ($t_p$) | 15.5 h |
| 2. | Storm duration ($T_d$) | 17.05 h |
| 3. | Unit rainfall duration (rainfall block) | 1 h |
| 4. | Peak discharge of unit hydrograph | 63.0 $m^3$/s |
| 5. | Width of the UH measured at 50% of peak discharge ordinate | 18.8 h |
| 6. | Width of the UH measured at 75% of peak discharge ordinate | 6.0 h |
| 7. | Width of the rising limb of UH measured at 50% of peak discharge ordinate | 3.4 h |
| 8. | Width of the rising limb of UH measured at 75% of peak discharge ordinate | 1.8 h |
| 9. | Base width of UH | 66.0 hours |
| 10. | Runoff depth represented by UH | 1.0 cm |

This 1-h UH is converted to 2-h UH for convenience. The storm duration $T_D = 1.1 * T_p = 17.05$ h (where $T_p$ the time to peak of 1-h UH for bridge no. 462 = 15.5 h) has been approximated as 16 h to make 8 blocks of 2 hour duration each.

The annual maximum daily rainfall available for the catchment was converted using clock-hour correction by adding 35 mm CWC (1993) and subsequently, the same was converted for 16 hour storm duration by multiplying with a factor of 0.955 CWC (1987). One more correction for moisture replenishment was also applied by multiplying the 16-hour storm so obtained with a factor of 1.25 CWC (1993). The $\phi$-index of 5.0 mm for 2 h block (2.5 mm/h as per CWC, 1987) was then applied to the average 2-h rainfall block for the catchment to compute the rainfall excess. This rainfall excess alongwith the 2-h UH ordinates are convoluted to generate the yearly peak flood for 15 years (1975-89) and also yearly direct runoff hydrograph (DRH). These yearly peak floods were subjected to frequency analysis using L-moments based GEV distribution to generate peak floods for different return periods for the catchment defined by the bridge No. 462. The GEV parameters for rainfall data, annual maximum peak flood data and rainfall as well as flood estimates for various return periods are given in Table 6.5.

**Table 6.5: GEV parameters, rainfall and flood estimates for various return periods**

| Rainfall over Punpun basin upto Hamidnagar GD site | | Estimated flood at railway bridge No. 462 | |
|---|---|---|---|
| **GEV Parameters** | | | |
| Location (u) = 0.6807<br>Scale ($\alpha$) = 0.4162<br>Shape (k) = -0.1627 | | Location (u) = 0.8740<br>Scale ($\alpha$) = 0.2089<br>Shape (k) = -0.0257 | |
| Return Periods (year) | Rainfall (mm) | Return periods (year) | Flood ($m^3/s$) |
| 5 | 45 | 5 | 439 |
| 10 | 59 | 10 | 499 |
| 25 | 79 | 25 | 577 |
| 50 | 96 | 50 | 637 |
| 100 | 115 | 100 | 696 |

The 16 hours storm of 2-h duration obtained after applying corrections were critically sequenced (i.e. maximum rainfall against maximum 2-h UH ordinate, the second highest rainfall increment against second largest UH ordinates and so on. This sequence is reversed by putting the last item first and first item last) and $\phi$-index is applied to compute the rainfall excess. This rainfall excess alongwith the 2-h UH ordinates were again convoluted to generate the yearly peak flood for 15 years (1975-89). These yearly peak floods were also subjected to frequency analysis to generate peak floods for same return periods for critically sequenced rainfall. The purpose of computing the peak flood for critically sequenced rainfall is to know the maximum possible peak flood for different return periods.

The DDF of Hamidnagar sub-basin obtained in rainfall analysis for T=5, 10, 25, 50 and 100 years was subjected to different corrections (clock hour, storm duration for 16 h and moisture replenishment) to know the storm depths for these return periods. These depths were distributed for 2-h duration rainfall blocks and the distribution pattern was obtained from `mean average time distribution curves of storm of various durations' (Appendix – C). The outcome of this exercise is the time distribution of rainfall frequency for these return periods. The usual $\phi$-index is applied to compute the rainfall excess. These rainfall excess alongwith the 2-h UH ordinates were again convoluted to generate the annual peak flood for these return periods (Table 6.6). The peak floods were also computed using critically sequenced rainfall frequencies (Table 6.6).

An attempt was also made to use the severe most rain depth obtained in the DAD analysis from 24 hour duration curve. The depth corresponding to the catchment area of 516.5 $km^2$ of bridge no. 462 was noted to be equal to 255 mm (1 day areal rainfall). After necessary corrections (clock hour, duration and moisture replenishment) equal to [(255+35)*0.955*1.25] i.e. 346 mm becomes design storm depth. This storm depth was distributed in eight blocks and $\phi$-index applied as explained above to get the rainfall excess distribution. These rainfall excess alongwith the 2 h UH ordinates were convoluted to get the severe most flood of magnitude 1481 $m^3$/s and 1685 $m^3$/s when the rainfall excess is critically sequenced.

The three groups of actual rainfall or rainfall frequencies from DDF used to estimate flood are summarised below.

AR: Actual rainfall, corrected (clock-hour, storm duration, and moisture replenishment), distributed among 8 rain blocks as rain actually occurred. On deducting losses($\phi$-index), the rainfall excess of each of the 15 years was convoluted with the SUH ordinates and annual maximum peak flood values were obtained for the period of 15 years viz. 1975 to 1989. Using this annual maximum peak flood series of 15 years record length, flood frequency analysis was carried out employing the L-moment based GEV distribution and flood of various return periods viz. 5, 10, 25, 50 and 100 years were estimated.

DDF: Rainfall depth obtained directly from DDF curves of Hamidnagar sub-basin, necessary corrections as in AR above were made. Distributed in 8 rain blocks as per time distribution curves. The rainfall excess was convoluted with SUH ordinates and obtained flood directly as per assigned return period of rainfall.

DDF(CS): Same as DDF above but the rainfall blocks were critically sequenced.

**Table 6.6: Comparison of peak floods for various return periods**

| Return Period(T) (year) | AR ($m^3/s$) | DDF ($m^3/s$) | % Deviation | AR ($m^3/s$) | DDF(CS) ($m^3/s$) | % Deviation |
|---|---|---|---|---|---|---|
| 5 | 439 | 344 | 21.6 | 439 | 361 | 17.7 |
| 10 | 499 | 419 | 16.0 | 499 | 449 | 10.0 |
| 25 | 577 | 516 | 10.5 | 577 | 562 | 2.6 |
| 50 | 637 | 586 | 8.0 | 637 | 639 | -0.3 |
| 100 | 696 | 658 | 5.4 | 696 | 722 | -3.7 |

For easy comparison, the tabular result is represented in the form of bar graph (Fig. 6.6).

Frequency analysis has also been carried out for long duration (1-3 day) storms. These daily data pertain to annual maximum point rainfall of individual stations spreading over Punpun basin. The 1, 2 and 3 day annual maximum rainfall (1974 – 1989) have been shown in Table 6.7. Application of L-moment based GEV distribution has been used and the rainfall with respective return periods are given in Table 6.8.

## Table 6.7: Rainfall for annual severe storms (1-3 day) for Punpun basin (in mm)

| Year | Day | Raingauge stations | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | H'gan | N'naga | S'ghati | A'bad | Obra | Goh | Tekari | M'pur | Karpi | Punpun |
| 1974 | 1 | 43.2 | 50.0 | 74.5 | 77.8 | 75.0 | 62.5 | 117.0 | 70.0 | 83.2 | - |
| | 2 | 79.4 | 75.0 | 86.0 | 128.0 | 112.0 | 92.5 | 121.0 | 128.0 | 127.8 | - |
| | 3 | 105.9 | 75.0 | 91.5 | 128.8 | 117.0 | 112.5 | 121.0 | 135.2 | 147.6 | - |
| 1975 | 1 | 127.0 | 50.0 | 50.0 | 102.0 | 37.0 | 25.0 | 53.0 | 73.0 | 30.0 | 102.0 |
| | 2 | 127.0 | 75.0 | 89.0 | 125.4 | 61.0 | 45.0 | 59.8 | 78.4 | 59.2 | 138.0 |
| | 3 | 127.0 | 85.0 | 109.5 | 130.8 | 61.0 | 61.0 | 61.4 | 83.2 | 73.2 | 150.4 |
| 1976 | 1 | 74.0 | 105.0 | 243.0 | 193.0 | 100.0 | 195.0 | 202.0 | 175.0 | 147.0 | 56.0 |
| | 2 | 89.0 | 200.0 | 415.0 | 381.0 | 119.0 | 346.0 | 382.0 | 345.0 | 194.5 | 109.5 |
| | 3 | 89.0 | 265.0 | 464.0 | 472.4 | 122.0 | 411.1 | 436.7 | 363.4 | 241.5 | 113.7 |
| 1977 | 1 | 139.0 | 53.0 | 72.0 | 177.0 | 102.8 | 94.0 | 88.0 | 127.0 | 135.5 | 106.6 |
| | 2 | 211.0 | 64.0 | 135.0 | 216.0 | 145.0 | 136.0 | 159.3 | 184.0 | 191.9 | 146.2 |
| | 3 | 232.0 | 64.0 | 145.0 | 237.0 | 182.1 | 146.2 | 219.5 | 198.6 | 191.9 | 150.4 |
| 1978 | 1 | 180.0 | 201.0 | 190.0 | 86.0 | 83.6 | 31.5 | 52.5 | 72.0 | 50.0 | 103.6 |
| | 2 | 256.0 | 237.0 | 204.0 | 140.8 | 140.0 | 54.5 | 87.5 | 132.3 | 98.0 | 199.6 |
| | 3 | 281.0 | 289.0 | 204.0 | 175.0 | 195.4 | 72.0 | 110.5 | 175.3 | 134.0 | 294.2 |
| 1979 | 1 | 145.4 | 75.0 | 165.5 | 42.0 | 87.7 | 78.0 | 36.5 | 44.0 | 100.0 | 60.0 |
| | 2 | 172.8 | 113.0 | 179.5 | 68.0 | 128.3 | 97.0 | 49.5 | 88.0 | 178.0 | 107.5 |
| | 3 | 189.6 | 145.0 | 183.5 | 86.0 | 128.8 | 115.5 | 60.1 | 99.0 | 180.0 | 114.7 |
| 1980 | 1 | 66.0 | 50.0 | 71.0 | 131.5 | 91.0 | 84.0 | 68.0 | 108.0 | 41.0 | 112.8 |
| | 2 | 81.0 | 80.0 | 107.0 | 135.8 | 91.0 | 94.0 | 114.0 | 198.0 | 76.0 | 151.2 |
| | 3 | 81.0 | 80.0 | 117.0 | 135.8 | 91.0 | 101.1 | 114.0 | 198.0 | 76.0 | 151.2 |
| 1981 | 1 | 70.0 | 41.0 | 74.4 | 87.0 | 65.2 | 110.0 | 25.3 | 57.2 | - | - |
| | 2 | 88.2 | 67.0 | 78.7 | 127.8 | 122.3 | 145.0 | 48.3 | 108.4 | - | - |
| | 3 | 100.2 | 91.0 | 78.7 | 150.6 | 127.8 | 145.0 | 60.0 | 120.4 | - | - |
| 1982 | 1 | 94.0 | 40.3 | 60.2 | 50.4 | 92.8 | 79.8 | - | 162.5 | 39.0 | 75.5 |
| | 2 | 127.2 | 67.3 | 120.0 | 84.6 | 124.8 | 92.8 | - | 171.0 | 68.0 | 103.0 |
| | 3 | 147.5 | 67.3 | 120.0 | 118.1 | 132.8 | 98.8 | - | 175.0 | 96.0 | 120.0 |
| 1983 | 1 | 65.4 | 82.0 | 80.0 | 108.0 | 57.0 | 75.0 | 80.0 | 90.0 | 120.0 | 67.0 |
| | 2 | 100.0 | 137.4 | 150.5 | 112.4 | 91.5 | 99.0 | 129.2 | 176.0 | 202.0 | 111.9 |
| | 3 | 102.0 | 165.0 | 169.5 | 112.8 | 111.5 | 99.0 | 164.4 | 227.2 | 271.0 | 115.9 |
| 1984 | 1 | 101.1 | 60.0 | 188.0 | 150.0 | 72.5 | 102.0 | 40.0 | 60.0 | 44.0 | 89.8 |
| | 2 | 169.4 | 101.0 | 209.0 | 200.0 | 87.5 | 117.0 | 76.0 | 111.0 | 84.0 | 155.1 |
| | 3 | 180.6 | 104.0 | 217.0 | 240.0 | 94.5 | 121.8 | 111.0 | 141.0 | 113.0 | 190.3 |
| 1985 | 1 | 86.4 | 97.0 | 133.0 | 88.0 | 72.0 | 180.0 | 50.0 | 60.0 | 136.0 | 60.0 |
| | 2 | 100.8 | 137.0 | 188.0 | 148.0 | 78.0 | 190.0 | 87.0 | 88.0 | 196.0 | 88.0 |
| | 3 | 104.0 | 149.0 | 196.0 | 186.0 | 82.0 | 190.0 | 105.0 | 100.0 | 246.0 | 89.0 |
| 1986 | 1 | 58.0 | 124.0 | 141.6 | 121.5 | 150.0 | 175.0 | 76.0 | 125.0 | 152.0 | 135.3 |
| | 2 | 100.3 | 174.0 | 201.8 | 157.5 | 162.0 | 305.0 | 121.0 | 206.0 | 292.0 | 195.3 |
| | 3 | 112.3 | 204.2 | 264.4 | 164.5 | 172.0 | 339.0 | 149.6 | 247.0 | 304.0 | 209.0 |
| 1987 | 1 | 195.3 | - | 82.0 | 110.0 | - | - | - | 140.0 | - | - |
| | 2 | 320.3 | - | 143.0 | 207.5 | - | - | - | 210.0 | - | - |
| | 3 | 428.3 | - | 173.0 | 239.5 | - | - | - | 250.0 | - | - |
| 1988 | 1 | 70.1 | - | 100.0 | 83.0 | - | - | - | 105.0 | - | - |
| | 2 | 121.1 | - | 130.0 | 153.0 | - | - | - | 147.0 | - | - |
| | 3 | 133.5 | - | 155.0 | 177.0 | - | - | - | 182.0 | - | - |
| 1989 | 1 | 85.3 | - | 138.0 | 128.0 | - | - | - | 115.0 | - | - |
| | 2 | 85.3 | - | 158.0 | 140.2 | - | - | - | 145.0 | - | - |
| | 3 | 85.3 | - | 174.0 | 141.2 | - | - | - | 166.0 | - | - |

**Table 6.8: Rainfall of individual stations for various return periods for Punpun basin (in mm)**

| Station | Day | Return Period (years) | | | | |
|---|---|---|---|---|---|---|
| | | 5 | 10 | 25 | 50 | 100 |
| Hariharganj | 1 | 128 | 159 | 204 | 242 | 284 |
| | 2 | 168 | 217 | 302 | 387 | 495 |
| | 3 | 185 | 247 | 358 | 476 | 634 |
| Nabinagar | 1 | 97 | 128 | 183 | 237 | 307 |
| | 2 | 151 | 197 | 271 | 342 | 428 |
| | 3 | 181 | 235 | 317 | 391 | 477 |
| Sherghati | 1 | 154 | 193 | 247 | 292 | 342 |
| | 2 | 205 | 255 | 330 | 395 | 468 |
| | 3 | 266 | 283 | 370 | 446 | 534 |
| Aurangabad | 1 | 142 | 165 | 193 | 213 | 231 |
| | 2 | 194 | 239 | 310 | 373 | 447 |
| | 3 | 214 | 272 | 371 | 469 | 593 |
| Obra | 1 | 100 | 108 | 114 | 117 | 120 |
| | 2 | 139 | 152 | 163 | 170 | 175 |
| | 3 | 157 | 179 | 205 | 222 | 237 |
| Goh | 1 | 140 | 173 | 215 | 246 | 276 |
| | 2 | 179 | 243 | 349 | 452 | 582 |
| | 3 | 187 | 256 | 383 | 518 | 698 |
| Tekari | 1 | 95 | 128 | 182 | 233 | 297 |
| | 2 | 148 | 204 | 304 | 407 | 544 |
| | 3 | 174 | 241 | 362 | 490 | 660 |
| Makdampur | 1 | 139 | 167 | 201 | 266 | 251 |
| | 2 | 212 | 255 | 313 | 358 | 405 |
| | 3 | 240 | 282 | 333 | 369 | 404 |
| Karpi | 1 | 132 | 155 | 178 | 193 | 205 |
| | 2 | 207 | 247 | 291 | 321 | 348 |
| | 3 | 241 | 285 | 335 | 369 | 400 |
| Punpun | 1 | 110 | 125 | 141 | 152 | 162 |
| | 2 | 166 | 189 | 218 | 239 | 260 |
| | 3 | 184 | 226 | 291 | 352 | 424 |

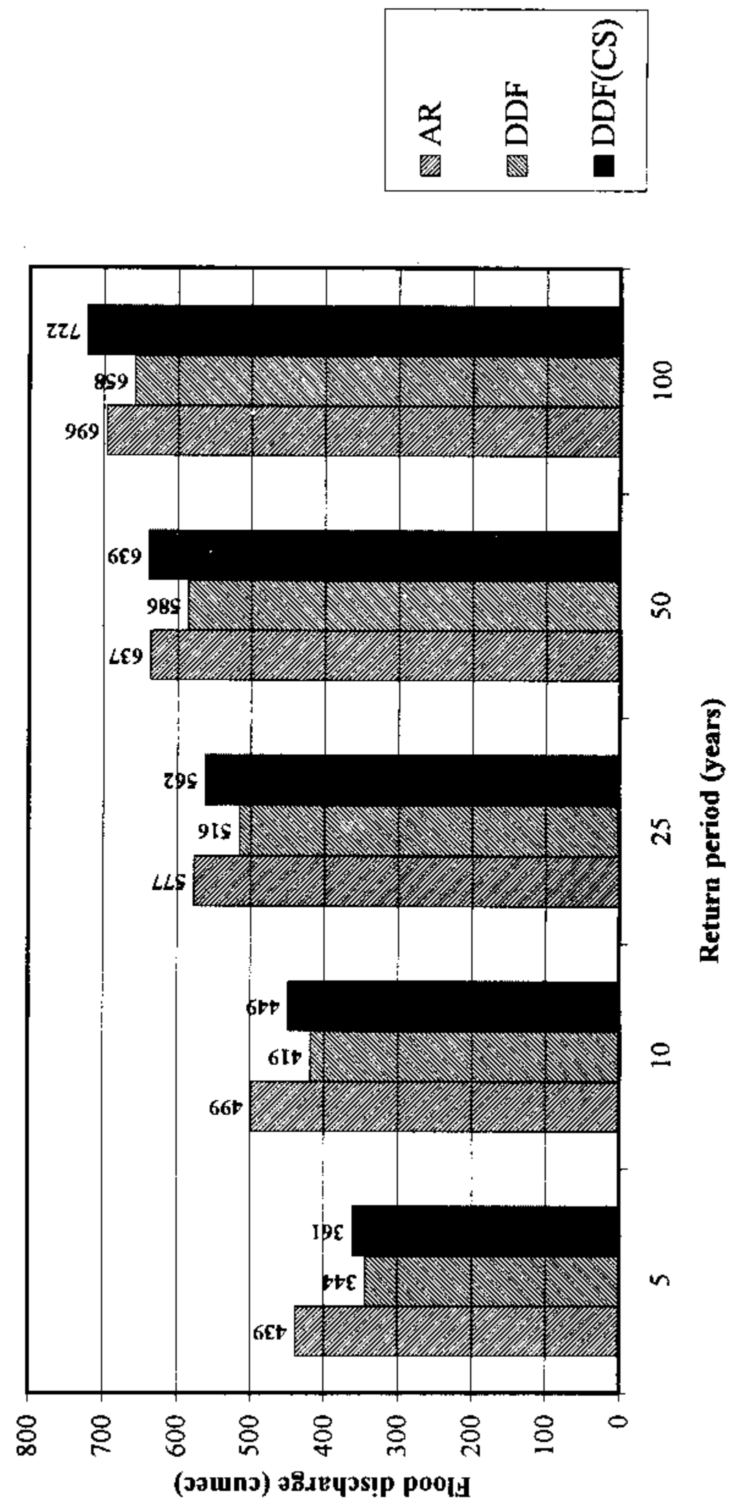


Fig.6.6 Comparison of flood peaks for Punpun basin

# Chapter 7

## CONCLUSIONS

In this study, the characteristics of short interval rainfall data have been studied using the rainfall data of the Punpun river basin of Bihar. The rainfall data of 11 raingauge stations located in the Punpun river basin have been used. Out of these raingauges only Gaya raingauge station is the self recording type, and the hourly data of this raingauge have been utilized for converting the daily data of the remaining 10 ordinary raingauge stations into the hourly data. Thiessen polygon method has been used for computing the average areal rainfall for the study area. Depth-area-duration (DAD) and depth-duration-frequency (DDF) curves have been prepared for the Punpun basin. The DAD curves have been developed for various durations. The DDF curves have also been prepared for the above mentioned durations using the general extreme value (GEV) distribution based on the L-moments approach. The flood frequency estimates computed by frequency analysis of rainfall and frequency analysis of annual maximum peak floods have been compared. As the length of the rainfall data used in the study is only fifteen years; hence, the results of the study may be treated as indicative only. A summary of the work carried out and the conclusions drawn from the study are mentioned below.

(i) The DAD curves have been developed for various short durations such as 1 h, 2 h, 3 h, 6 h, 9 h, 12 h, 18 h and 24 h for the Punpun river basin upto Hamidnagar.

(ii) The DDF curves of the various return periods viz. 5, 10, 25, 50 and 100-years have also been prepared for the Punpun river basin upto Hamidnagar using the L-moment based general extreme value (GEV) distribution for the above mentioned durations.

(iii) In addition to the above mentioned characteristics of the short interval rainfall, values of the 1-day, 2-day and 3-day rainfall for the various return periods viz. 5, 10, 25, 50 and 100-years have also been computed for all the ten ordinary raingauge stations.

(iv) Using the excess-rainfall of the design storm duration, floods of the various return periods have been estimated for the sub-basin defined by the bridge number 462 of the Punpun basin upto Hamidnagar. For this purpose, the DDF curves developed for the Punpun basin upto Hamidnagar gauging site, based on the data of five raingauge stations and the unit hydrograph of the sub-basin defined by the bridge number 462 have been utilized. The comparison of flood frequency estimates computed by frequency analysis of rainfall and frequency analysis of annual maximum peak floods shows that the percentage deviation between the frequency of rainfall and frequency of floods are 0.30% for the 50-year return period and 3.7% for the 100-year return period.

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**Table A-1: Yearly maximum daily rainfall (mm) of Gaya station and corresponding daily rainfall of other stations.**

| Storm Dates | Gaya | N'nagar | H'ganj | S'ghati | A'bad | Obra | Goh | Tekari | M'pur | Karpi | Punpun |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 7-8 Oct, 1975 | 52.3 | 0.0 | 26.4 | 0.0 | 0.0 | 24.0 | 12.7 | 27.2 | 0.0 | 19.0 | 0.0 |
| 15-16 Sept, 1976 | 290.8 | 65.0 | 0.0 | 243.0 | 65.0 | 0.0 | 195.0 | 54.7 | 175.0 | 47.0 | 0.0 |
| 7-8 July, 1977 | 73.0 | 6.0 | 0.0 | 13.0 | 71.4 | 11.0 | 9.8 | 18.0 | 13.0 | 58.4 | 2.3 |
| 29-30 July, 1978 | 74.5 | 22.0 | 64.0 | 5.5 | 0.2 | 9.2 | 0.0 | 6.2 | 33.5 | 20.2 | 6.6 |
| 17-18, Aug, 1979 | 72.0 | 18.0 | 30.0 | 135.0 | 11.0 | 28.0 | 24.6 | 0.0 | 0.0 | 26.0 | 0.0 |
| 01-02 Aug, 1980 | 74.0 | 48.0 | 43.0 | 5.5 | 70.2 | 53.0 | 24.0 | 0.0 | 11.0 | 40.2 | 0.8 |
| 16-17 July, 1981 | 68.5 | 41.0 | 70.0 | 20.6 | 62.4 | 0.0 | 10.0 | 12.4* | 16.5 | 11.5* | 0.0 |
| 21-22 Oct, 1982 | 39.0 | 40.3 | 0.0 | 60.2 | 57.0 | 18.0 | 56.0 | 25.3* | 0.0 | 20.3 | 0.8 |
| 5-6 July, 1983 | 75.8 | 16.4 | 0.0 | 7.0 | 0.0 | 14.0 | 50.0 | 5.0 | 45.0 | 23.5 | 3.9 |
| 28-29 Aug, 1984 | 118.0 | 4.0 | 68.3 | 33.4 | 4.0 | 6.0 | 18.5 | 18.0 | 0.0 | 28.0 | 5.0 |
| 29-30 July, 1985 | 90.0 | 10.0 | 35.3 | 15.4 | 30.0 | 2.5 | 10.5 | 0.0 | 2.0 | 4.0 | 7.0 |
| 5-6 July, 1986 | 155.0 | 38.2 | 82.2 | 89.0 | 121.5 | 0.0 | 4.2 | 41.0* | 30.0 | 95.0 | 23.2 |
| 6-7 July, 1987 | 183.0 | 0.0 | 6.4 | 20.0 | 25.2 | 21.5* | 61.3 | 61.3* | 68.3 | 63.0* | 64.0* |
| 10-11 Aug, 1988 | 115.0 | 5.0 | 10.1 | 38.0 | 24.0 | 20.5* | 94.2 | 94.3* | 105.0 | 97.0* | 98.3* |
| 15-16 July, 1989 | 43.0 | 0.0 | 11.2 | 0.0 | 4.1 | 3.5* | 40.3 | 40.4* | 45.0 | 41.5* | 42.0* |

* = Data not available. Missing data has been filled in by using normal ratio method of surrounding stations.

N'nagar = Nabinagar, H'ganj = Hariharganj, S'ghati = Sherghati, A'bad = Aurangabad.

**Table B-1: Hourly areal depth of rainfall (<24 hours) in Hamidnagar sub-basin of Punpun basin (mm).**

| Year | 1-hr | 2-hr | 3-hr | 6-hr | 9-hr | 12-hr | 16-hr | 18-hr | 24-hr |
|---|---|---|---|---|---|---|---|---|---|
| 1975 | 6.44 | 10.74 | 11.16 | 11.16 | 11.16 | 11.22 | 11.22 | 11.22 | 11.22 |
| 1976 | 10.63 | 18.6 | 29.23 | 49.42 | 65.54 | 70.08 | 73.2 | 75.21 | 77.51 |
| 1977 | 10.91 | 16.01 | 19.35 | 25.84 | 25.84 | 25.84 | 26.6 | 26.6 | 26.6 |
| 1978 | 7.79 | 15.32 | 17.4 | 19.08 | 19.08 | 19.08 | 19.08 | 19.08 | 19.35 |
| 1979 | 10.7 | 20.51 | 24.08 | 26.76 | 29.44 | 29.44 | 29.44 | 29.88 | 31.67 |
| 1980 | 24.48 | 35.5 | 41.61 | 45.29 | 45.29 | 45.29 | 45.29 | 45.29 | 45.29 |
| 1981 | 12.8 | 17.92 | 21.76 | 31.98 | 43.51 | 43.51 | 43.51 | 43.83 | 43.83 |
| 1982 | 15.21 | 22.82 | 24.72 | 37.07 | 37.07 | 37.07 | 37.07 | 37.07 | 37.07 |
| 1983 | 6.8 | 9.52 | 9.79 | 10.31 | 10.32 | 10.32 | 10.32 | 10.32 | 10.32 |
| 1984 | 12.81 | 21.99 | 22.63 | 22.84 | 23.69 | 23.69 | 23.69 | 25.19 | 25.19 |
| 1985 | 12.2 | 18 | 19.46 | 21.46 | 21.71 | 21.71 | 21.95 | 21.95 | 21.95 |
| 1986 | 13.095 | 23.25 | 27.9 | 51.14 | 64.15 | 65.08 | 65.08 | 65.08 | 71.59 |
| 1987 | 5.04 | 8.81 | 11.33 | 14.97 | 15.35 | 15.6 | 15.73 | 15.86 | 23.28 |
| 1988 | 10.61 | 17.41 | 18.81 | 29.55 | 31.15 | 31.15 | 32.1 | 31.3 | 31.3 |
| 1989 | 5.06 | 8.22 | 8.22 | 8.34 | 8.34 | 8.34 | 8.34 | 8.34 | 10.87 |

**Table B-2: Short duration rainfall frequency of Hamidnagar sub-basin of Punpun basin (mm).**

| T | 1-hr | 2-hr | 3-hr | 6-hr | 9-hr | 12-hr | 16-hr | 18-hr | 24-hr |
|---|---|---|---|---|---|---|---|---|---|
| 5 | 14.53 | 23.23 | 27.55 | 38.37 | 43.13 | 43.51 | 43.90 | 44.17 | 45.0 |
| 10 | 17.40 | 27.02 | 32.34 | 46.54 | 54.98 | 56.12 | 56.88 | 57.36 | 58.8 |
| 25 | 21.06 | 31.40 | 37.88 | 56.23 | 70.78 | 73.45 | 74.94 | 75.83 | 78.8 |
| 50 | 23.79 | 34.38 | 41.65 | 62.99 | 83.14 | 87.40 | 89.64 | 90.94 | 95.7 |
| 100 | 26.51 | 37.13 | 45.13 | 69.39 | 95.96 | 102.23 | 105.43 | 107.26 | 114.5 |
| 200 | 29.25 | 39.67 | 48.35 | 75.42 | 109.31 | 118.07 | 122.44 | 124.93 | 135.6 |
| 500 | 32.86 | 42.75 | 52.27 | 82.94 | 127.85 | 140.71 | 147.03 | 150.61 | 167.2 |
| 1000 | 35.62 | 44.89 | 54.98 | 88.29 | 142.60 | 159.24 | 167.37 | 171.98 | 194.5 |

**Isohyetal maps of 15-16 September 1976 for various short durations**

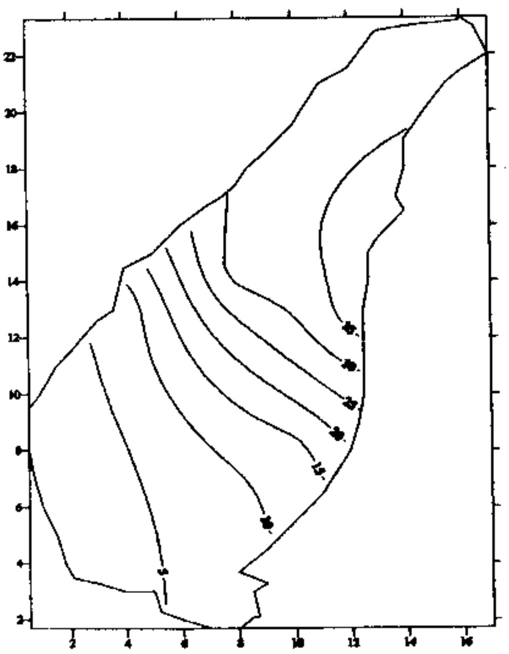


Fig.C-1: 1 hourly Isohyetal map of Punpun Basin

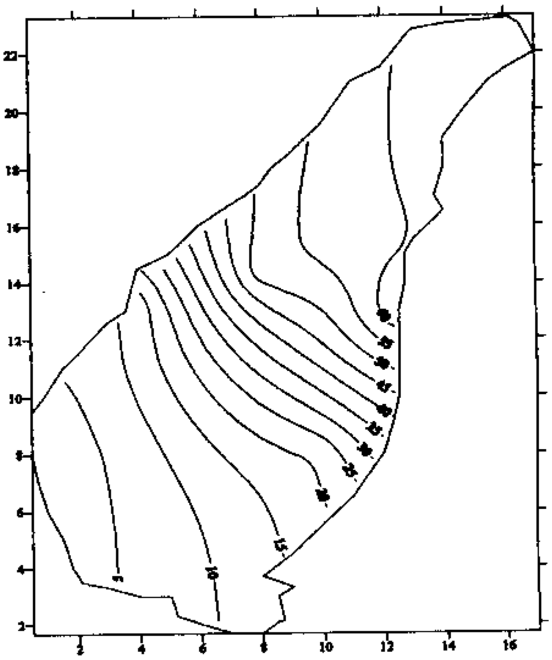


Fig.C-2: 2-hourly isohyetal map of Punpun basin

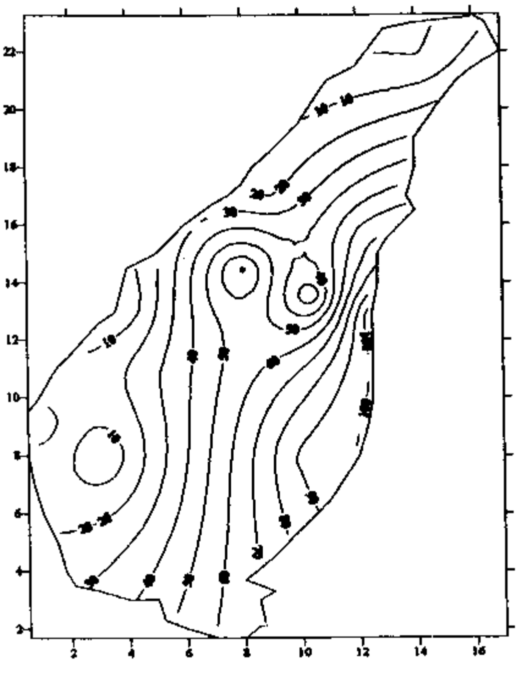


Fig.C-3: 3-hourly isohyetal map of Punpun Basin

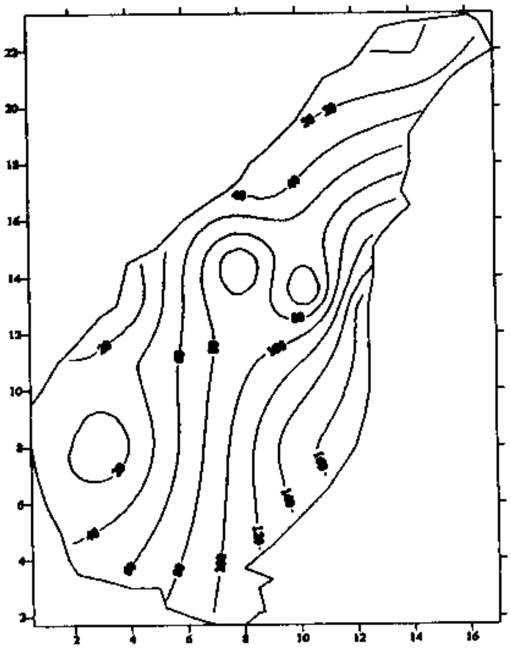


Fig.C-4: 6-hourly isohyetal map of Punpun basin

## Isohyetal maps of 15-16 September 1976 for various short durations

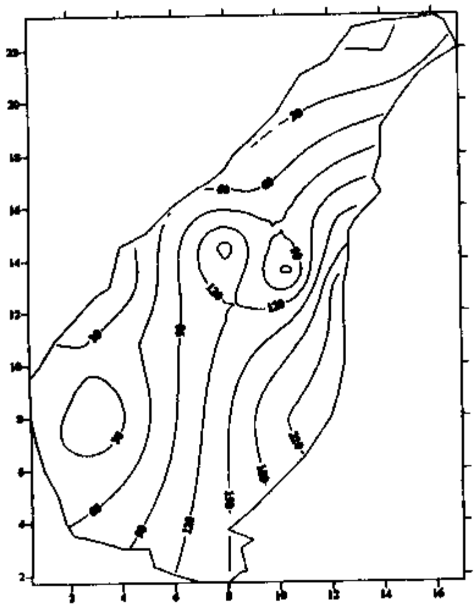


Fig.C-5: 9 hourly Isohyetal map of Punpun Basin

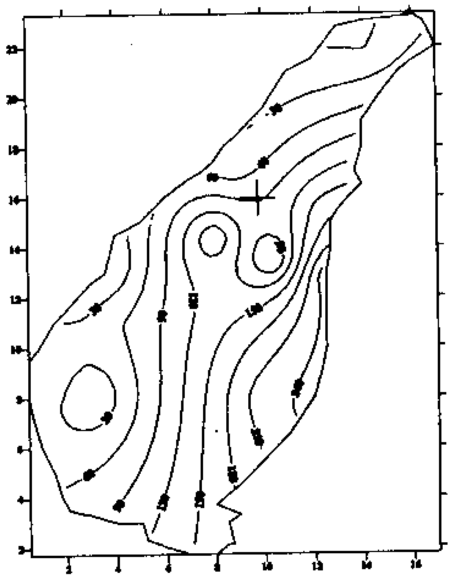


Fig.C-6: 12-hourly isohyetal map of Punpun basin

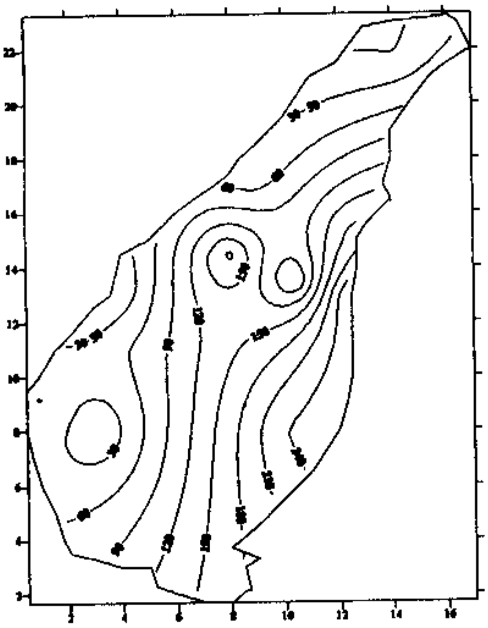


Fig.C-7: 18-hourly isohyetal map of Punpun Basin

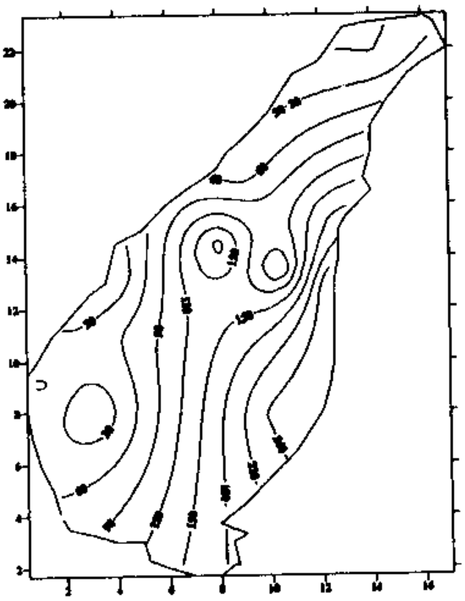
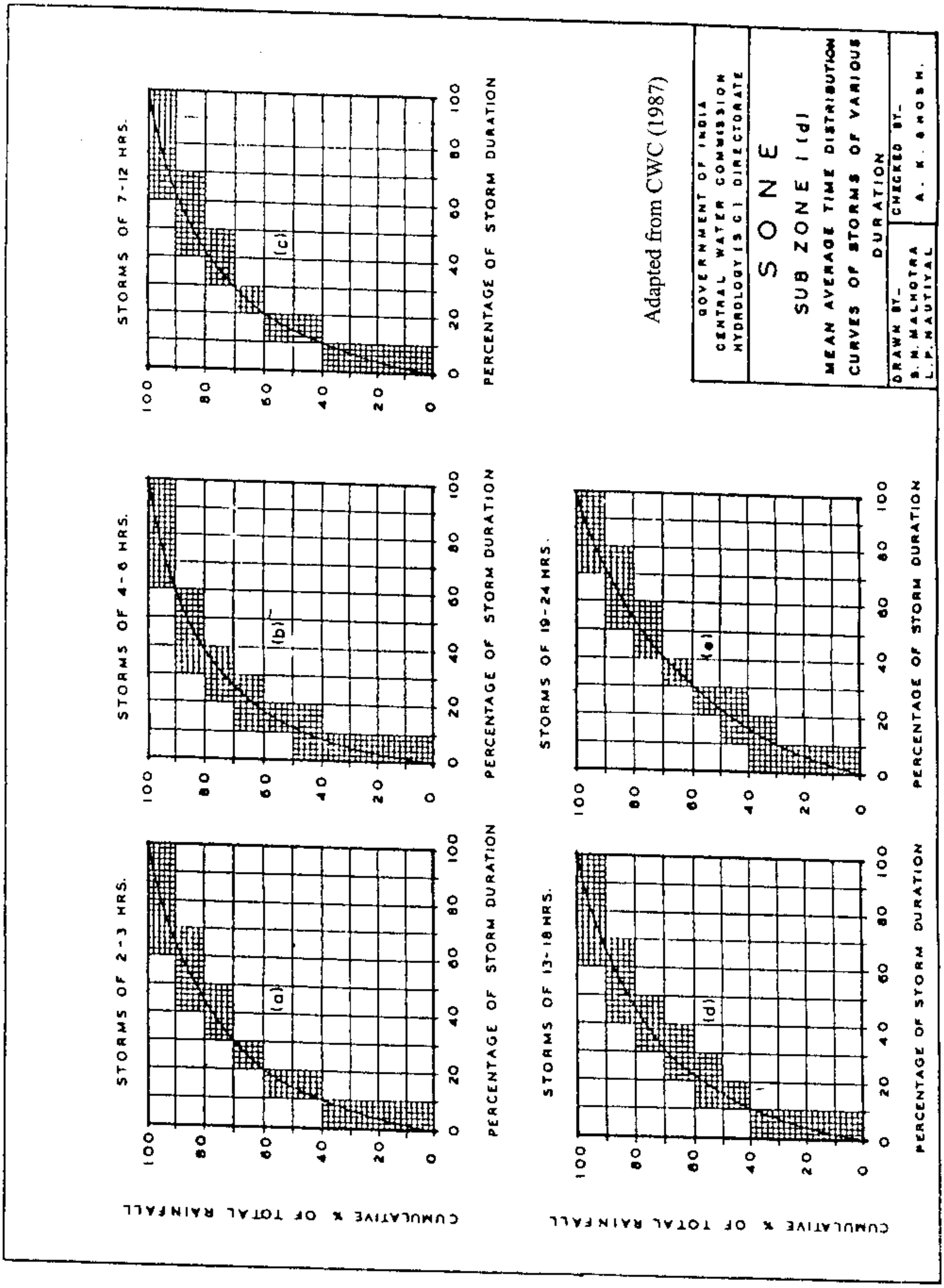


Fig.C-8: 24-hourly isohyetal map of Punpun basin

# APPENDIX- D




Adapted from CWC (1987)

**(i) 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.

**(ii) 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.

**(iii) 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).

**(iv) 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 freeboard 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 unforeseen 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 freeboard design.

| **Catchment area ($km^2$)** | **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 freeboards should be for a flood of not less than 100-year return period.

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

| **Catchment area ($km^2$)** | **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. |

**(v) Design Criteria for Flood Control Schemes**

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

| | |
|---|---|
| | 25 year return period flood on small tributaries and |
| Town protection works | 100 year return period flood |
| Important industrial complexes, 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 cumec, 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.

# Characteristics of Short Interval Rainfall for Punpun Basin


| | |
|---|---|
| **Director** | **K.S. Ramasastry** |
| **Coordinator** | **R.D. Singh** |

## STUDY GROUP

**N.G. Pandey**
**Rakesh Kumar**
**B. Chakravorty**

## SUPPORTING STAFF

**A.K. Sivadas**