FLOOD HYDROLOGY OF SOME CATCHMENTS IN WESTERN HIMALAYAN REGION

A DISSERTATION

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

of

MASTER OF TECHNOLOGY

in

WATER RESOURCES DEVELOPMENT (CIVIL)

By

KRISHNANUNNI N.M.

WATER RESOURCES DEVELOPMENT TRAINING CENTRE
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE-247 667 (INDIA)

JUNE, 2004

CANDIDATES DECLARATION

I hereby declare that the dissertation titiled, "FLOOD HYDROLOGY OF SOME CATCHMENTS IN WESTERN HIMALAYAN REGION", which is being submitted in partial fulfillment of the requirement for the award of the Degree of Master of Technology in Water Resources Development (Civil) at Water Resources Development and Training Centre (WRDTC), Indian Institute of Technology (IIT)-Roorkee, is an authentic record of my own work carried out during the period of 29-07-2003 to 30-06-2004 under the supervision and guidance of Dr. U.C.Chaube, Professor and Head, WRDTC, IIT-Roorkee, INDIA.

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

Krishnanunni 29/06/04

Roorkee,
Dated, 29 June 2004.

(Krishnanunni.N.M.)

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

U.C. Chaube 29.6.04

(Dr. U.C.Chaube)
Professor and Head,
WRDTC,
IIT-Roorkee,
Roorkee, INDIA.

ACKNOWLEDGEMENT

I take this opportunity to express my profound gratitude and sincere thanks to Dr. U.C. Chaube, Prof. and Head, WRDTC, Indian Institute of Technology-Roorkee, Roorkee, for his constant encouragement, inspiring guidance, persuasion and continued support for all aspects of this dissertation, without which this dissertation would have been incomplete.

I am also grateful to Dr. G.C. Mishra, Prof. and Chairman, DRC of WRDTC, for his kind cooperation, encouragement and guidance. I also extend my hearty thanks to all faculty members WRDTC, for their support and guidance. I also express my gratitude to all staff members of Water Resources Development Training Centre, especially those in Computer lab and departmental library, for their co-operation.

I wish to express my gratitude to Chairman, CE (HRM), CE (Hydrological Studies) and Director (Training), Central Water Commission, MoWR, for giving me an opportunity to undergo this course.

I will also like to express my gratitude to Mr. P.S.Kutyal, my colleague in CWC, who as Excecutive Engineer, Upper Ganga Division, gave some valuable advice and some information about the rivers of the area. I also like to express my gratitude to Mr. Anil Dhore, Research Scholar, WRDTC, for his constant encouragement.

It will be ungrateful, if I don't mention the co-operation and moral support from my wife, Reji and my children Rachana and Krishnapriya.

Krishnanunni 29/06/04

Date: June, 2004
Roorkee, INDIA.

(KRISHNANUNNI.N.M.)

ABSTRACT

The main objective of this study was to study the hydrological characteristics of some catchments in western Himalayan region and to investigate the criteria and procedures for selection and application of design flood for water resources structures with possible improvements. Towards this objective, practices related to both the design flood criteria and estimation procedures, currently being followed in India, in general, and in Western Himalayan region in particular, have been critically reviewed. A comparison has also been made with the practices being followed in some of the developed countries.

It is practically impossible to gauge every river at every location where a dam, barrage, bridge or similar hydraulic structure may be sited. Hence, a regional approach is inevitable for flood estimation, for a majority of such structures. In the past, there have been various attempts at regionalizing flood hydrological parameters. Due to the mountainous nature of the catchments in the Western Himalayan region, procedures developed for plain areas cannot be directly applied for flood estimation in this area. Snowmelt runoff is found to have very good correlation in log space with area under snow-cover.

Four major approaches, viz., Rational formula, empirical formula, regional flood frequency approach and regionalized synthetic UG approach have been critically reviewed. The Rational formula represents the response behaviour of only very small catchments. The other three approaches directly or indirectly use some form of regression relations, which represent the average behaviour. Wide confidence bands of the regression relations restrict the use of such methods. Disregard to this aspect leads to unreliable estimation. Synthetic UG approach proposed in CWC report (CWC, 1994) needs modification before applying to mountainous upper catchments. Flood estimation for nine project sites in the study area (Upper Yamuna, Upper Ganga and Sarju basins) using various methods have been carried out.

The UG approach assumes catchment linearity. In this dissertation, the linearity of some catchments in the region has been studied through study of volume-peak discharge relation in log-space. The applicability of UG method for catchments in the region has been examined in this study. Appropriate volume-peak discharge relation for the study area have been developed and used in flood estimation.

CONTENTS

CANDIDATES DECLARATION		i
ACKNOWLEDGEMENT		ii
ABSTRACT		iii
CONTENTS		v
LIST OF ANNEXURES		vii
LIST OF TABLES		vii
LIST OF FIGURES		viii
LIST OF NOTATIONS		ix
CHAPTER 1	**INTRODUCTION**	
1.1	BACKGROUND OF THE STUDY	1
1.2	STUDY AREA	2
1.3	OBJECTIVES	3
1.4	SCOPE OF STUDY	3
CHAPTER 2	**A REVIEW OF DESIGN FLOOD CRITERIA AND ESTIMATION PROCEDURES**	
2.1	CRITERIA FOR STORAGE DAMS	5
2.1.1	Criteria Existing In India	5
2.1.2	Criteria Existing in some other countries	6
2.2	CRITERIA FOR BARRAGES AND WEIRS	7
2.3	CRITERIA FOR CROSS DRAINAGE WORKS	8
2.4	CRITERIA FOR ROAD AND RAIL BRIDGES	9
2.5	CRITERIA FOR FLOOD CONTROL EMBANKMENTS	10
2.6	RISK BASED APPROACH FOR DESIGN FLOOD	10
2.6.1	Advantages Of A Risk Analysis Based Decision Making	11
2.7	RISK CLASSIFICATIONS AND DESIGN FLOOD SELECTION	11
2.8	A CRITICAL REVIEW OF DESIGN FLOOD - ESTIMATION PROCEDURES	12
2.8.1	Flood Frequency Approach	13
2.8.2	Hydrometeorological Approach	15
2.8.2.1	Basin Response Function	15
2.8.2.2	Design Storm Parameters	15
2.8.3	Regional Flood Frequency Approach	17
2.8.4	Regional UG Approach	17
2.8.5	Transposition Of UG From A Gauged Catchment	18
2.9	REVIEW OF DESIGN FLOOD ESTIMATES OF OLD DAMS	20
2.9.1	Hirakud Dam (Orissa)	20
2.9.2	Chambal Complex of Dams	21
2.9.3	Other Dams Recently Reviewed	22
CHAPTER 3	**ANALYSIS OF HYDROLOGIC CHARACTERISTICS**	
3.1	LOCATION	25
3.2	RIVER SYSTEM OF STUDY AREA	25
3.3	GENERAL CHARACTERISTICS	31
3.4	HYDROLOGIC AND OTHER CHARACTERISTICS	38
3.4.1	Snow Cover And Glacial Feature	39

3.5 CHARACTERISTICS OF THE SELECTED CATCHMENTS 39
3.5.1 Snowmelt and Baseflow Characteristics: 42
3.5.2 Assumptions And Limitations Of The SUG Approach 43

CHAPTER 4 FLOOD ESTIMATION FOR UNGAUGED CATCHMENTS -CASE STUDY
4.1 INTRODUCTION 45
4.1.1 Review of the Rational Formula 45
4.1.2 Review of Snyder's Method 46
4.1.3 Review of Modified Dicken's formula 49
4.2 PROCEDURE PRESCRIBED IN CWC REPORT FOR ZONE 7 50
4.3 FLOOD ESTIMATTION FOR SELECTED PROJECT SITES 52
4.3.1 Derivation of UG 54
4.3.2 Design Storm Depth and Distribution 54
4.4 DISCUSSSION 55
56

CHAPTER 5 CATCHMENT LINEARITY AND VOLUME-PEAK DISCHARGE RELATION
5.1 INTRODUCTION 58
5.2 GENERAL HYDROLOGIC SYSTEM MODEL 59
5.2.1 Linear System 61
5.3 VOLUME-PEAK DISCHARGE RELATION 62
5.3.1 Value of Standardisation 63
5.3.2 Hydrograph Separation Procedure 63
5.3.4 Theoretical Justification Based On Triangular UG Approach 64
5.4 STUDY FOR SOME CATCHMENTS IN WESTERN HIMALAYAN REGION 64
5.5 RELATIONSHIP BETWEEN INTERCEPT WITH CATCHMENT CHARACTERISTICS 71
5.6 FLOOD ESTIMATION USING SSPDD AND COMPARISON WITH UG METHOD 73
5.7 CONCLUSIONS 74
5.8 SCOPE FOR FURTHER STUDY 74

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS
6.1 CRITERIA FOR SELECTION OF DESIGN FLOOD 76
6.2 HYDROLOGIC CHARACTERISTICS OF STUDY AREA 77
6.2.1 General Features 77
6.2.2 Snowmelt and Baseflow characteristics 77
6.2.3 CWC Report for zone 7 77
6.2.4 Comparison of Catchments in CWC Report and in the Present Study 78
6.3 VOLUME-PEAK DISCHARGE RELATION 78
6.3.1 Conclusions 78
6.3.2 Scope for Further Study 79

REFERENCES 80

LIST OF ANNEXURES

3.1	COMPUTATION OF EQUIVALENT SLOPE	84
4.1	VARIOUS DEFINITIONS OF TIME LAG	87
4.2	SYNTHETIC UG DERIVATION FOR A TYPICAL CATCHMENT USING THE SUG APPROACH OF REPORT FOR ZONE 7	88
5.1	FLOOD EVENTS USED IN THE STUDY OF HYDROLOGIC LINEARITY	91

LIST OF TABLES

2.1	Inflow Design Flood for the Safety of the Dam	6
2.2	Criteria followed in USA (till late 90's)	7
2.3	Criteria for Cross Drainage Works	8
2.4	Criteria for Flood Embankments	10
2.5	Classification Of Dams In Terms Of Consequences Of Failure	12
2.6	Summary Results of Hydrologic Review of Dams	23
3.1	Climatic Regions in Study Area	33
3.2	Heaviest 24-Hour Rainfall At Stations In Study Area	38
3.3	Heaviest Short Duration (1-Hr, 3-Hr, 6-Hr) Rainfall At Stations In Study Area	38
3.4	Physiographic Characteristics Of Catchments Used In CWC Study	40
3.5	Physiographic Characteristics Of Catchments Used In This Study	40
3.6	Runoff Characteristics Of The Selected Project Catchments	41
3.7	Critical snowmelt used for Project Catchments Studied	43
4.1	UG Characteristics of Catchments	55
4.2	100-Year Flood Peaks Estimated at the Location of Sites by Various Methods*	56
5.1	Drainage Basin Characteristics	66
5.2	Summary Details Of The Flood Events Used In The Study	67
5.3	Relation Between Volume (cm) and Peak Discharge (m^3/s) in Log-space	68
5.4	Relation Between Volume (cm) and Peak Discharge Per Unit Area (cm/h) in Log Space	71
5.5	Comparison of Flood Peaks by UG Approach and SSPDD Approach	72
A 4.1	Some of the Definitions of Lag Time	87

LIST OF FIGURES

2.1	Envelope Curve for Estimates of PMF	14
3.1	Location of Zone-7 in the Map of India	26
3.2	Map of the River System	27
3.3	Schematic Diagram of the River System	28
3.4	Topographical Map of the Study Area	32
3.5	Soil Classification	32
3.6	Land Use	34
3.7	Normal Annual Rainfall (mm)	34
3.8	25-year Return Period Point Rainfall Depth for 24-hr Duration	35
3.9	50-year Return Period Point Rainfall Depth for 24-hr Duration	36
3.10	100-year Return Period Point Rainfall Depth for 24-hr Duration	37
3.11	Pattern Of 10-Daily Flows As Percentage Of Mean Annual Flow For A Typical Year At Various G&D Sites In The Region	42
3.12	Relation Between Snow Cover Area And Design Base Flow	43
4.1	Definition Sketch for Snyder's UG Method	47
4.2	Synthetic UG Parameters-Definition Sketch	53
5.1	Location of G&D Sites	65
5.2	Volume Peak Discharge Relation for Gola at Dam site	69
5.3	Volume Peak Discharge Relation for Alaknanda at Rudraprayag	69
5.4	Volume Peak Discharge Relation for Mandakini at Rudraprayag	69
5.5	Volume Peak Discharge Relation for Pinder at Karnaprayag	70
5.6	Volume Peak Discharge Relation for Alaknanda at Joshimath	70
5.7	Volume Peak Discharge Relation for Bhagirathi at Tehri	70
5.8	Regression Between Intercept And Catchment Area (excluding snow-covered area)	72
5.9	Regression Between Intercept And A/L Ratio	72
A.4.1	Definition Sketch –Various Definitions of Lag Time	87
A.4.2	SUG at Bagoli Dam	89

LIST OF NOTATIONS

A= Catchment Area in sq. km

Af = Forest Area in sq. km.

AMC = Antecedent Moisture Condition

b= Arithmetic Intercept

CN= Curve Number

df. = Degree of Freedom

dQ/dt = Change of outflow with Respect to Time

dS/dt = Change of Storage with Respect to Time

E = Mean Basin Elevation

Fa=Continuing abstraction

I_a = initial abstraction

L= Length of Main River in km

L_c = Length of river from centroid of catchment

m = slope of best fit line

P=Rainfall volume

P=Annual exceedence probability

Pe = Rainfall excess volume

Q =Discharge

Qp = Peak Discharge in m^3/s

r= Coefficient Correlation

r^2= Coefficient of Determination

S= maximum possible infiltration

s= Standard Deviation

SCS= Soil Conservation Service

S_e or S_{st} = Slope of main stream (m/km)

S_e= Standard Error

T = Return period (y)

Tb = Base period of UG (h)

Tc = Time of concentration (h)

Tp = Time to from start to peak (h).

UHG , UG =Unit Hydrograph

V= Runoff Volume as depth over catchment (cm)

W_{50} Width of UG at 50% of peak

W_{75} Width of UG at 75% of peak

WR_{50} Width of Rising limb of UG at 50% of peak

WR_{75} Width of Rising limb of UG at 75% of peak

t_r, D = duration of UG

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND OF THE STUDY

Hydrological studies form an important and basic input to planning and design of all water resources development projects. The three most important hydrological inputs, in general, are water availability, design flood and sediment pattern and distribution studies. Flood plain occupancy is becoming critical due to increasing population and industrial development in the flood plains. Thus, in Indian context, dam breaches are likely to lead to significantly large flood damages. Therefore, the design flood inputs for sizing of spillways of water resources structures being constructed across rivers need to be arrived at in a judicious manner, taking into account both economy in cost and safety of the structure itself and the people in the valley downstream. Globally, insufficient spillway capacity has been found to be one of the major causes of dam failures. Global warming and uncertainties about the possible climate change, especially, those related to extreme events, make the task of reliable hydrologic design all the more relevant. With many new projects being constructed or being planned in the Himalyan region of India, a critical review of the practices followed and the need for further studies to rectify shortcomings in the current methodology is necessary.

The approach to both selection and application of design flood in hydrologic design in India has been broadly in tune with international practices. However, in the recent past, there is a widening of technological gap between practices followed in India and in developed countries. A critical review of the criteria for selection and application of design flood has been advocated at various forums in India.

In chapter 4, a brief summary of the various techniques being followed for design flood estimation for the study area and a brief review of their performance has been presented.

Unit hydrograph (UG) method (Sherman, 1932) is the most common method for flood estimation in India. Unit hydrograph theory assumes linearity of catchment response, in the sense that the discharge values are considered to be proportional to the volume of rainfall excess occurring uniformly over space and time, in a given duration (called unit duration). The physiography of Upper Yamuna catchment is conducive to short duration, high magnitude flood peaks and the flows in channel phase in this part is super-critical in nature (Aggarwal, P.P. et al 1982). Because of the varying hydrological characteristics, formulae for derivation of UG based on observations in the plain area, cannot be directly utilized for the mountainous catchments. There has been number of studies covering catchments from many countries of the world, indicating non-linearity of catchments, thus raising doubts about the applicability of UG procedure. Linearity of upper catchments in Himalayan region needs to be checked before applying Unit Hydrograph method for estimation of design flood.

1.2 STUDY AREA

The study area is located in Himalayan region of India. The area has vast unutilized hydropower potential. The area is also geologically very young and seismically unstable. Uttaranchal state of India was carved out in the year 1998 from the then Uttar Pradesh state as an independent state. The study area consists of the catchments of Upper Yamuna, Upper Ganga and Kali basins. Most of the study area fall in the Uttaranchal state. A map of the study area is shown at Fig. 3.2, in chapter 3.

All the three rivers, Yamuna, Ganga and Kali are having substantial area under permanent snow cover, making them perennial rivers. Many of their tributaries rise from Himalayan glaciers. Yamuna, which is the most important tributary of river Ganga, joins Ganga nearly 1000 km outside the boundary of study area. Kali is a tributary of Gaggar, which in turn joins river Ganga, beyond the study area. Kali river is an international river, with India and Nepal, being the riparian countries.

Topographically the area can be roughly classified as Terai (plain), Sivaliks (sub-Himalayan tract), lower Himalayas and greater Himalayasm, in the order of increasing altitude. The normal annual rainfall has a maximum value near about the elevation band of 1200-2000m a.s.l. The rainfall pattern of the area is heavily influenced by the topography and can be gauged from Fig. 3.7 to 3.10 of chapter 3. The topographic, meteorological and drainage features of the study area has been described in detail in chapter 3.

1.3 OBJECTIVES

i) To examine the criteria and procedures for selection and application of design flood for water resources structures and to suggest improvements.

ii) To study hydrologic characteristics of some catchments in Himalayan region.

iii) To examine the various methods for design flood estimation being practiced in the region and to analyse their performance in predicting flood events.

iv) To examine the hydrologic linearity of the catchments in the area and applicability of unit hydrograph procedure in the Himalayan catchments.

v) To derive relation between runoff volume and peak discharge, and its application in flood estimation.

1.4 SCOPE OF STUDY

Background and objectives of this study are presented by explaining the need for review of hydrologic design practices and criteria for design flood selection. Country practices in selection of design flood for hydrologic design of hydraulic structures are critically reviewed and comments are offered in chapter 2. Hydrologic characteristics influencing flood formation in the study area are analysed. Catchments of nine hydropower projects in Upper Yamuna, Upper Ganga and Kali basins are analysed in chapter 3.

Conventional procedures for flood estimation (Rational formula, Snyder's method, Modified Dicken's formula, regional flood frequency studies, etc.) are reviewed and applied for flood estimation in the selected nine catchments (chapter 4).

Hydrologic linearity of catchments is analysed through study of volume-peak discharge relation. Relationship is derived for flood estimation in ungauged catchments (chapter 5).

Conclusions are drawn based on above mentioned study and recommendations are given for further study (chapter 6).

CHAPTER 2

A REVIEW OF DESIGN FLOOD CRITERIA AND ESTIMATION PROCEDURES

Different criteria are adopted for various types of hydraulic structures depending on magnitude of failure damages and prevailing socio-economic conditions. Brief details of practices followed in India and other countries are discussed in the following sections. In India, Central Water Commission (CWC), (earlier Central Water and Power Commission) and Bureau of Indian Standards (BIS) are the agencies at the federal level responsible for framing the hydrologic design practices.

2.1 CRITERIA FOR STORAGE DAMS

2.1.1 Criteria Existing In India:

Because of inadequacy and highly uneven spatial and temporal distribution of rainfall, water resources development in India depends heavily on creation of surface storages. There are about 3600 completed large dams and 700 are under construction (CWC, 1992). Majority of completed dams are more than three decades old. Spillways are necessary part of such structures to pass the excess water, which cannot be stored. Inadequate spillway capacity is found to be the most common cause of dam failure. There has also been the scenario of ever increasing population on the downstream flood plains giving rise to increase hazard with lapse of time.

Till 1985, recommendations from the Government of India report, "Estimation of Design Flood -Recommended Procedure" (CW&PC, 1972) served the purpose of national standard for selection of design flood for dams. The current national standard as reflected in "IS; 11223-1985-Guidelines for Fixing Spillway Capacity," (BIS, 1985) is an improvement to these recommendations. In this guideline, various inflow design floods that need to be considered for various functions of spillways have been listed:

a) Inflow design flood for the safety of the dam against overtopping structural failure: The spillway and its energy dissipation arrangements, if provided for a lower flood, should function reasonably well.
b) Inflow design flood for efficient operation of energy dissipation works: It can be lower than the inflow design flood for safety of the dam, provided no damage to main structure will occur.
c) Inflow design flood for checking acceptability of extent of upstream submergence.
d) Inflow design flood for checking acceptability of extent of downstream submergence.

For the purpose of deciding the level of inflow design flood for the safety of the dam, dams are classified as small, intermediate and large on the basis of gross storage and static head, as below. The overall size classification shall be the greater of that indicated by either of the two parameters. Further the code permits the use of floods of larger or smaller magnitude if the hazard involved in the eventuality of a failure is particularly high or low. For more important projects, dam break studies may be done as an aid to the judgment in deciding whether PMF needs to be used.

Table 2.1: Inflow Design Flood for the Safety of the Dam

Classi-fication	Gross storage (Mm^3)	Static Head (m)	Inflow Design Flood For Safety of The Structure
Small	Between 0.5 and 10.0	Between 7.5 and 12	100-year flood
Inter-mediate	Between 10 and 60	Between 12 and 30	SPF
Large	Greater than 60	Greater than 30	PMF

2.1.2 Criteria Existing in some other countries:

Murthy and Reddy (1990) reviewed the criteria and practices followed in other countries in regard to hydrologic safety of large dams.

In Australia, the definition of large dams is as given by International Commission on Large Dams (ICOLD) and selection of design flood is based on incremental flood hazard category. **In USA,** dams are classified into three categories depending upon the height and storage and the hazard potential due to dam failure. The size classification is more or less same as that adopted in India. (BIS, 1985). The recommended standards for the selection of design flood are given in Table below:

Table 2.2: Criteria followed in USA (till late 90's)

Hazard	Size	Safety standard in USA	Safety standard in Australia
Low	Small	50 to 100 yr flood	1 in 100 to 1 in 1000
	Intermediate	100 yr flood to SPF	
	Large	SPF to PMF	
Significant	Small	100 yr flood to SPF	1 in 1000 to 1 in 10,000
	Intermediate	SPF to PMF	
	Large	PMF	
High	Small	SPF to PMF	1 in 10,000 to PMF
	Intermediate	PMF	
	Large	PMF	

(Subsequently, agencies in USA have switched over to risk analysis as a basis for selection of the design flood in all future investments.)

2.2 CRITERIA FOR BARRAGES AND WEIRS

Weirs and barrages have usually small storage capacities, and the risk of loss of life and property downstream would rarely be enhanced by failure of the structure. Apart from the damage to the structure itself, this may bring about disruption of irrigation and communication that are dependent on the barrage /weir. Existing design practice are based on BIS code, IS: 6966 (Part I)-1989, "Hydraulic Design of Barrages and Weirs-Guidelines, Part I Alluvial reaches". The relevant provisions are:

"For purposes of design of items other than freeboard, a design flood of 50 year return-period may normally suffice. For designing the free board, a minimum of 500 year frequency flood or the SPF may be desirable."

It can be seen that in this IS code also, some flexibility has been given to the designer, to vary the criteria based on risks and hazard involved. It would be more appropriate to use the concept of incremental hazard and use consequence based classification. In incremental analysis, the potential hazard for the case of both before the dam (natural condition) and with the dam (regulated condition) are compared.

Comments: It would have been better to have a single BIS code to cover all types water retaining structures, above a particular level of height and storage. Such a code could apply to industrial structures as well, such as tailing dams, etc. This is the practice suggested by Canadian Dam Safety Association (CDA), vide their guidelines published in 1995 (CDA, 1995). Practice of having different codes for dams, barrages, weirs, embankments, etc. in India, creates confusion and inconsistency. In many cases, the terms "dam" and "barrage" have been used in a loosely interchangeable manner. In this context, the definition of dam as given in the CDA guideline (CDA, 1995), which is reproduced under can be considered:

> "These guidelines apply in general to dams, as defined in Section 1.2, that are at least 8 m high and which have at least 60,000 m^3 of reservoir capacity (see Definitions). They also apply to smaller dams where the consequences of failure would be "Low" or greater. Some examples of small dams requiring special consideration are; dams on erodible foundations if a breach could release a head in excess of 8 m, and dams retaining contaminated fluids."

2.3 CRITERIA FOR CROSS DRAINAGE WORKS

Existing practice are based on BIS code, IS: 7784 (Part I)-1993, "Code of Practice for Design of Cross Drainage Works, Part I General Features" (BIS, 1993). The structures are divided into four categories, A to D depending upon canal discharge and drainage discharge.

Table 2.3: Criteria for Cross Drainage Works

Category of structure	Canal discharge in m^3/s	Drainage discharge m^3/s	Frequency of design flood
A	0-0.5	All discharges	1 in 25 years
B	0.5 – 15	0-150	1 in 50 years
		Above 150	1 in 100 years
C	15 –30	0-100	1 in 50 years
		Above 100	1 in 100 years
D	Above 30	0-150	1 in 100 years
		Above 150	See note 2 below

Note 1: Design flood is the higher of value from the above and the observed flood peak.

Note 2: In this case, hydrology shall be investigated in detail and design flood not to be less than 1 in 100 year flood.

Further, the code says that the check flood for the foundation shall be 25% more than that given above.

Comment: Compared to the earlier version, these recommendations incorporate the concept of consequence based decision-making, though in an approximate manner. In the earlier code, proportionate increase was based not on canal discharge and drainage discharge, but rather on the catchment area. Earlier the standard was for 1 in 10 to 1in 25 year floods. Hence, standard has become harsher, which is but natural considering the improvement in economic status of the society and the increasing population density in flood plains.

There can be further improvement by incorporating the gross area under irrigation, which will be affected by disruption of supplies and also possible damage and loss of life (direct) due to breach in the structure. There is justification for assigning a higher level of design flood in the case of canals carrying water for municipal water supply and hydropower generation.

Notwithstanding the above provisions, more important CD structures in India, (such as those of the Narmada Main Canal in Gujarat) have been designed for floods of the order of SPF and PMF, considering the high level of consequence of failure of these structures.

2.4 CRITERIA FOR ROAD AND RAIL BRIDGES

For road bridges, the Indian Roads Congress code (IRC, 1970), 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 any other recognized 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 is to be adopted. Notwithstanding the above, generally, a 50 to 100 year flood is adopted. For railway bridges, Railway Design and Standards Organisation (RDSO) has recommended that 50-year flood is to be used for smaller bridges carrying 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.

2.5 CRITERIA FOR FLOOD CONTROL EMBANKMENTS

During the fifties, CWC had recommended that the highest water level, which the embankments should withstand, be assessed based on the data situation. Where G&D data for 40-50 years was available, a 100-year flood was recommended. For shorter data length situations, certain empirical relations were available connecting rainfall intensity and catchment characteristics. When no data was available, Dicken's formula or Ryve's formula were also used. The flood depth (measured above natural ground level), so obtained was further enhanced by a certain percentage. Later, the following broad criteria was recommended and is generally followed in the country.

Table 2.4: Criteria for Flood Embankments

Predominantly agricultural areas	25 years flood for small tributaries to 50-year flood on major rivers.
Town protection works	100-year flood
Important industrial complexes, assets and lines of communication	100-year flood

According to Ganga Flood Control Commission (GFCC, 1978), "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 year 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. In the case of double embankments, the design HFL shall be determined keeping in view the anticipated rise in the HFLs on account of jacketing of the river.

2.6 RISK BASED APPROACH FOR DESIGN FLOOD

Risk assessment is still a relatively new concept in the field of dam safety evaluation and decision making. When properly conducted it can provide valuable information which may not otherwise be available from conventional approaches.

2.6.1 Advantages Of A Risk Analysis Based Decision Making

From a decision-maker's perspective, the potential benefits of risk analysis in selection of design flood for dams and other hydraulic structures include systematic:

- identification and definition of potential hazards;
- identification of potential failure modes;
- qualitative and/or quantitative statements of risk;
- identification of the important contributors to risk and the vulnerabilities of the system;
- identification of strategies to reduce and control risk;
- provision of deeper understanding of the dam and its performance characteristics;
- improvement of emergency planning;
- identification of the uncertainty in the analysis and the description of the degree of confidence in the result of the analysis;
- comparison of risks posed by one dam with those of other similar dams and other facilities and activities;
- establishment of priorities for expenditures on safety improvements for individual dams and between dams;
- planning of monitoring and surveillance and identification of high risk structures.

All of the above play an important role in effective risk management of dams, whether the objective is improved public safety, prevention of economic loss, or compliance with government regulation.

2.7 RISK CLASSIFICATIONS AND DESIGN FLOOD SELECTION

Current national standards only give a broad suggestion for changes in criteria based on hazard potential and other factors; it does not give detailed procedure and guideline for conducting such studies. The broad frame work of classification given at Table 2.5 has been proposed for adoption in India. (CWC, 1999). However, a general consensus on this issue is yet to materialise.

Table 2.5: Classification Of Dams In Terms Of Consequences Of Failure

CONSEQUENCE CATEGORY	POTENTIAL INCREMENTAL CONSEQUENCES OF FAILURE [a]		MINIMUM CRITERIA FOR INFLOW DESIGN FLOOD
	LIFE SAFETY [b]	SOCIOECONOMIC FINANCIAL & ENVIRONMENTAL [b]	
VERY HIGH	Large number of fatalities >500	Extreme damages >Rs 500 m	PMF
HIGH	Some fatalities 10 to 500	Large damages Rs 10 m to 500 m	Between PMF and SPF (or AEP 1/1000)
LOW	Few fatalities anticipated 1 to 10	Moderate damages Rs 1 m to 10 m	AEP between 1/100 and 1/1000 (or SPF)
VERY LOW	No fatalities	Minor damages beyond owner's property <Rs 1m	AEP 1/100 or greater

****Note: Numbers in table are given as example only..***

[a] Incremental to the impacts which would occur under the same natural conditions (flood, earthquake or other event) but without failure of the dam. The consequence (i.e. loss of life or economic losses) with the higher rating determines which category is assigned to the structure. In the case of tailings dams, consequence categories should be assigned for each stage in the life cycle of the dam.

[b] The criteria which separate the Consequence Categories should be consistent with societal expectations. The criteria may be based on levels of risk including loss of life which are acceptable or tolerable to society. The incremental consequences shall include the effects of the loss of the reservoir.

2.8 A CRITICAL REVIEW OF DESIGN FLOOD - ESTIMATION PROCEDURES

Since, hydrology is still evolving from the empirical era, it has been found prudent by design engineers not to rush with "code of good practice" on estimation procedures. However, a BIS code is available for design storm estimation. (IS: 5442-1969, currently under revision). Procedures in vague in the country are briefly discussed here.

At the start of this century, organised and large scale observation of river discharges were limited to only a few sites in the country, primarily at existing barrages /weir locations. Based on this limited database, efforts were made to establish statistical correlation between observed maximum peak discharges at the sites and their catchment area. Empirical relations such as Dicken's formula, Ryve's formula, Englis Formula etc. are popular examples of this period.

Later on, envelope curves were developed using whatever observed peaks were available. Curve given by S/Sh. Kanwar Sain and Karpov (CW&PC, 1972) is an example. These and similar empirical formulae dominated the field of design flood estimation through as late as 1950's. Some of the main drawbacks of these methods are:

- the exceedence probability of the computed flood peak is unknown
- with the occurrence of more severe floods in the region, there is a need for revision of these formulae, which is rarely done,
- some coefficients etc. are to be subjectively chosen,
- catchment area is but one parameter influencing the flood peak; other important factors like rainfall intensity, catchment slope and shape, soil and vegetation cover, etc. are ignored in most formulae.

Subsequently, CWC has developed envelop curves based on estimates of PMF made during the 1980's, in which an average line, an upper envelop and a lower envelop have been given and is shown at Fig. 2.1. (CWC, 2001). These curves can be useful to get a preliminary estimate of PMF for design of major dams.

2.8.1 Flood Frequency Approach

With availability of more and more discharge data, the feasibility of conducting a statistical frequency study of flood peaks were explored. Many important dams, such as Hirakud dam in Orissa (catchment area of the order of 83,400 sq.km.) were designed on this basis. Presently, frequency analysis is being done using station data as also on a regional basis. Detailed analysis are to be carried out for checking a) consistency of the data, b) randomness, c) presence of features such as trend, jump, outlier, etc. Further, subjectivity in deciding the frequency distribution to be used, method of fitting etc. cannot be completely avoided, even though there are techniques to reduce the band of such subjective variations. Some of the common limitations of adopting this as the sole technique for estimation of design flood are:

- The method yields only peak discharge or level (therefore, unsuitable where complete hydrograph shape is required, as in the case of storage dams and other structures effecting significant flood moderation).

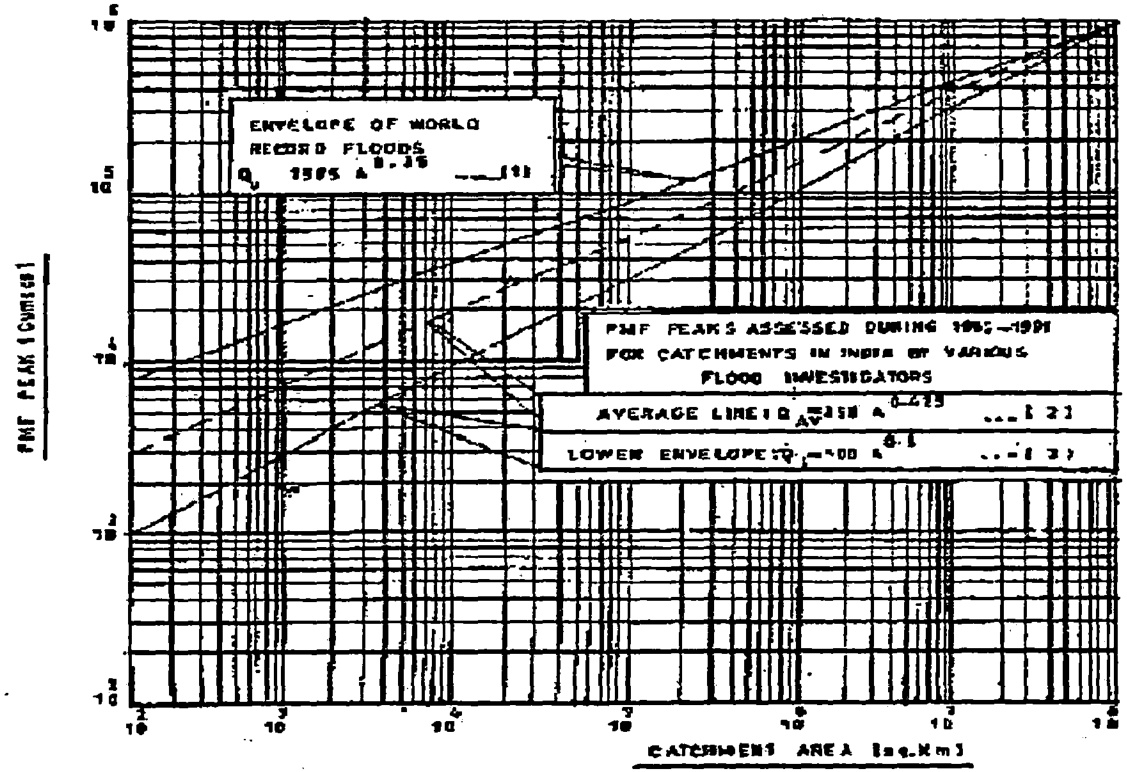


Fig. 2.1: Envelope Curves for Estimates of PMF (CWC, 2001)

- Larger values have a high influence on the estimated peaks. Unfortunately, these values are not systematically observed values, due to inherent technical and human limitations.
- High level of unreliability at higher ranges of extrapolation.

Selection Of Return Period For A Given Level Of Risk of exceedance

It can be seen that the probability of a T year flood being exceeded in a period of r-years is given by

$$P(X>X_T) = 1-(1-1/T)^r$$

Using this formula, for example, it can be seen that the probability of a 100-year flood being exceeded in a project life of 100-years is 63.4%, which is too high to be accepted in general. This is contrary to the popular notion that a 100-year flood has very little chance of being exceeded in100-years. Conversely, the return period which is to be used for design of a structure can be decided, if the acceptable degree of risk and the expected life of the project are known.

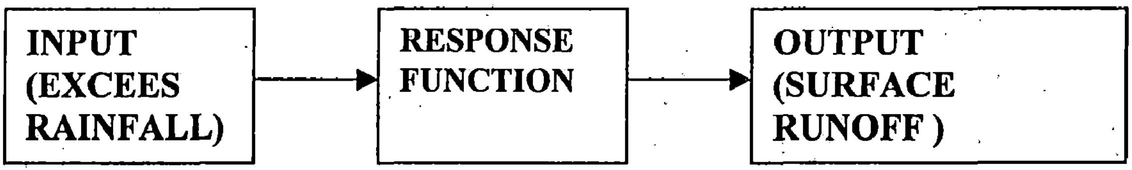
$T = \dfrac{1}{1-(1-P)^{1/r}}$ where, P represents the acceptable risk, in a project life of 'r' years. For example, it can be seen that the return period to be adopted for a structure having life of 100 years for an acceptable risk of 1% will be 9950 years (and not 100 years). There is need for better appreciation of these basic principles by designers, who actually uses the hydrologic inputs in their design.

In the above derivations, sampling errors are ignored. Since, this is not normally the case, there are further risks due to inaccuracies associated with estimation from limited samples. It is in this context, that, it is suggested to conduct a test for significance of estimates such as standard error, confidence band, etc.

2.8.2 Hydrometeorological Approach

This is by far the most commonly used approach today in estimation of design flood for water resources structures, despite its many limitations. In hydrometeorological approach, attempt is made to analyse the causative factors responsible for production of floods.

Generally, hydrograph of runoff is assumed to consist of a baseflow component and a direct runoff component. The direct runoff component is obtained by convoluting the excess rainfall hyetograph with basin response function. Even though many of these components elude precise definition, the method is found to be very convenient and sufficiently accurate for practical purposes (Chow V.T et.al., 1971, Singh, V.P., 1988) has also commented that the definitions of many of the components, such as baseflow, are not unique. Thus, in this method, basin response function, design rainfall excess graph (including its spatial and temporal distribution) and baseflow are required for estimation of flood hydrograph.



2.8.2.1 Basin Response Function

Development of a representative unit hydrograph at a particular site requires analysis of short interval discharge and concurrent rainfall data. Since, regular and systematic observation of discharges and short interval gauges is done only on a limited number of sites, numbering around 1000, development of unit hydrograph at the project site becomes a problem. Further, due to the topographical and meteorological features in India, some of the assumptions of the unit hydrograph theory are violated. Because of these reasons, many times, UG is developed using regional techniques or by transposition from adjacent hydrologically similar basins.

2.8.2.2 Design Storm Parameters

Design storm depth, duration, its spatial and temporal distribution have been found to be more important parameters, as compared to UG, baseflow and loss rate, in deciding the design flood peak, especially for lower level of exceedence probabilities.

Design storm duration: In the case of storage dams, duration of design storm to be equivalent to base of UG, rounded to multiples of 24 hours to be used for catchments upto 5000 sq. km. For larger catchments, duration for causing PMF is to be equivalent to 2.5 times the travel time from the farthest point to the site of the structure (CWC, 1993). Where peak and not volume is important, (WMO, 1995), a shorter duration equal to around 1.1 times the time to peak is likely to give largest flood peak. This later approach has been recommended for design of culverts, cross drainage works, road and rail bridges etc. A larger duration is likely to give more volume of hydrograph, but lesser peak, and hence, will be desirable in case of dams and other structures having significant flood moderation effect.

Design storm depth: The major steps involved in design storm studies are : study of meteorology of the region, identification of severe rain storm phenomena, fixation of the homogenous meteorological region, identification of candidate storms by scanning through the rainfall records of past 100 years or more (as is available), storm transposition, transposition adjustment, Depth Area Duration (DAD) and Depth Duration (DD) analysis, envelop adjusted DAD, storm maximization, adjustment for fixed observation cycle, etc. Design storm study is a highly specialized job and hence is usually done by an experienced hydrometeorologist.

Currently, the accepted practice in the country is to arrange the design storm hyetograph in the form of two bell shaped spells for each day, with the arrangement within each spell to represent the maximum flood producing characteristics, subject to the condition that for any within storm duration, the depth does not exceed PMP depth for that duration.

2.8.3 Regional Flood Frequency Approach

(For design of less important structures), economic considerations do not justify the detailed hydrological and meteorological investigations at every new site on a large scale and on a long-term basis. (Goel, N.K., et al, 2001). In such cases, regional analysis becomes very useful. Regional flood frequency analysis typically begins with the definition of regions. Regions are defined to be subsets of the entire collection of sites at which extreme flow information is available or required. A region can be considered to comprise a group of sites from which extreme flow information can be obtained for improving the estimation of extreme flows at any site in the region. (Ibid).

2.8.4 Regional UG Approach

This is one of the most popular approaches used in flood estimation for ungauged catchments. In this method, the more important UG characteristics such as time to peak, peak ordinate etc. are correlated with catchment physiographic parameters. There are numerous formulae available in literature for various regions of the world. However, caution has to be taken in their usage since, their applicability will be limited to the region for which they are derived and also for the range of catchment area sizes and other contributing factors like vegetation cover, land use, geology etc.

Central Water Commission had published a series of reports covering the various parts of India, for development of synthetic UG's using catchment physio-graphic data. Using these reports, the complete UG can be sketched for an ungauged catchment. Since, these UG parameters are obtained based on best-fit regression relations, and wide scatter have been observed, their usage for design of important structures should be with caution. These reports in addition also give necessary data for computation of 25-year, 50-year and 100-year return period rainfall values. These reports are increasingly being used by water resource engineers for design of bridges, culverts, barrages, small dams, etc., where consequences of hydrologic failure are not very severe. These reports have also been used in preliminary design flood estimation for other important structures.

Narain et. al (Hydrology Journal , 25 (1) 2002, pp 23-30) have described similar relations developed for Northern Himalayan region. In this, apart from the various physiographic parameters used by CWC, catchment elongation ratio, A/P ratio, shape factor, circularity ratio and fineness ratio have been used and hence, is expected to give better results. However, in the above paper, an inter-comparison has not been given.

2.8.5 Transposition Of UG From A Gauged Catchment

Investigators have used various methods for transfer of UG from a gauged catchment. Rastogi, et. al (2003) have described one such method using t/t_L and q/q_p ratios for a catchment in Uttaranchal. The brief details of the method are: i) The UG ordinates for all the storm events are first determined by dividing the direct runoff by effective rainfall. ii) The average distribution graph is then obtained by averaging these ordinates and then converting them into percentages with respect to sum of the ordinates. iii) If there are more than one gauged site in the homogenous region, average of these distribution graph can be considered as characteristic of the region. iv) this can be used at the ungauged site, if some formulae is available for computation of peak and lag time of the UG. A similar procedure was also described in CW&PC manual of 1972 (CW&PC, 1972).

Snyder's Method: In this well known method, a set if relations(refer to para 4.1.2 of chapter 4) are used at the gauged site, to obtain two parameters, Ct and Cp, which are considered uniform over the region.

These values are then used for the ungauged catchment to obtain the lag and peak of the UG. The UG is then shaped, using certain relations connecting W_{50}, W_{75}, W_{R50} and W_{R75} with the parameters already computed.

One major shortcoming of this method was pointed out by Linsley, et. al (1975) that the method does not take into account the slope of the river which is a major contributing factor. Accordingly, Linsley, et. al had suggested a modification to the above method, taking into account the slope of the main river. More detailed discussion is given at para 4.1.2 of chapter 4.

SCS Method For Small Ungauged Catchments: The Soil Conservation Service (SCS) of the US Department of Agriculture (1964) developed a model for predicting runoff volumes from small agricultural watersheds. This is a one parameter model in which the parameter defined by the so called curve number (CN) reflects the effect of soil type, vegetative cover, land use, and antecedent moisture condition. The values of CN have been computed based on field measurements of hydrologic variables. The SCS model hypothesizes that for a given rainfall the ratio of actual infiltration F to the maximum possible infiltration S (which is equal to the volume of voids, L) is equal to the ratio of actual runoff (Q) to the maximum possible runoff (R) (which is equal to the rainfall amount less abstractions, L). Put algebraically,

$$\frac{F}{S}=\frac{Q}{R}$$

The actual infiltration F is defined as: $F = P - I_a - Q$ and $R = P - I_a$

where, P denotes Precipitation, Ia initial abstraction and is normally taken as 0.2 times S, where

S = 1000/CN -10 (in inches).

Solving the above, Q can be computed from:

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

This runoff depth can be converted into a flood hydrograph using triangular UG approach. The method has been found to give better results as compared to the so called Rational Formulae, which was being used widely for very small catchments, typically urban watersheds. There are certain theoretical inconsistencies in the SCS model (Chow, Maidmont, et al,. 1987). These inconsistencies have been described in detail by Chen, S.J. and Singh V.P (1993) who have also given an improved SCS model. In this model, excess rainfall volume is considered inversely proportional to soil moisture deficiency.

2.9 REVIEW OF DESIGN FLOOD ESTIMATES OF OLD DAMS

2.9.1 Hirakud Dam (Orissa)

The dam was completed in the year 1956 for irrigation, power generation and flood moderation for downstream areas. The dam intercepts an area of 83,400 sq.km. of Mahanadi basin. It is a 4.8 km long composite dam, with the central concrete /masonry dam flanked with earthen dykes on either side. The dam blocks two arms of river Mahanadi by concrete spillways which are solid gravity type with ogee shaped crests. The maximum height over deepest foundation level is 60.96m and the gross storage capacity is 7189 MCM (at time of construction).

In 1947 the magnitude of maximum flood discharge (of unknown return period) was estimated as 32564 cumecs (11.5 lakh cusecs) and this was later revised to 51819 cumecs (18.9 lakh cusecs) with a volume of 35931 MCM (29 MAF). Further studies made in 1952 showed that the 500 year return period flood would have a peak of 42474 cumecs (15 lakh cusecs) which was adopted for design of the structure.

During the period of operation of the dam, many severe flood events have been reported. Some of these events are i) estimated inflow of 42475 cumecs (15 lakh cusecs) and a release of the order of 31148 cumecs (11 lakh cusecs) observed during July 1961, ii)estimated inflow of 37717 cumecs (13.32 lakh cusecs) and spillway discharge of 33385 cumecs (11.79 lakh cusecs) during 20th Sept.1980. Since these severe floods were observed during a short span of less than 30 years, a review of the earlier estimate was considered essential. CWC and Water Resources Department, Govt. of Orissa jointly conducted a study to arrive at PMF at Hirakud dam. Unit hydrographs at sub-catchment outlets and the design storm using daily rainfall data available for the region for the last 100 or so years were made use of. The PMF at Hirakud dam has been estimated to be having a peak of 69,632 cumecs (24.59 lakh cusecs) with a volume of 16,800 MCM.

Further studies were conducted for handling the revised PMF at the Hirakud dam by following the general principles contained in the "Report on Dam Safety Procedures", published by CWC in 1986. As per this report, there can be relaxation in initial reservoir level, if a flood forecasting system is in place and reservoir pre-depletion can be done based on such forecasts. Since, the flood forecasting system in Mahanadi basin has been in place for a long time and is said to be giving good results, and further modernization of the system was being implemented in 1996, it was considered advantageous to consider a reservoir routing study taking into account the possible pre-depletion in advance of the incoming severe flood.

As per the routing studies done with the above assumption and using the reservoir rule curves in practice by the state government, it was found that the PMF type of flood impinging at Hirakud dam during any period upto August can be safely routed through the existing spillways. Since, PMF is an extremely rare event, a proper view on the situation needs to be developed.

2.9.2 Chambal Complex Of Dams

Design floods for a cascade of four large dams, viz., Gandhi Sagar dam (GSD), Rana Pratap Sagar dam (RPS), Jawahar Sagar (JS) dam and Kota Barrage (KB), constructed across Chambal river were reviewed. Of the above, GSD is the upper most in the cascade and is in MP, whereas the other three are in Rajastan.

GSD was completed in the year 1960 and was designed for a flood peak of 21,237 cumecs (7.5 lakh cusecs). Higher floods of 7.55 lakh cusecs and 8.22 lakh cusecs were observed in the years 1961 and 1962 respectively. A review of the design flood was done in the year 1965 and the design flood peak was re-assessed as 14.05 lakh cusecs. Several high floods have been observed during the subsequent years as well, however, they were below this re-assessed value.

As per the review of design flood conducted in the year 1994, the peak of PMF hydrograph has been estimated as 54,390 cumecs (19.21 lakh cusecs).

2.9.3 Other Dams Recently Reviewed

Under a World Bank Assisted project, CWC and state governments have reviewed the hydrologic safety of nearly 62 large dams in the country. The results of the study in brief have been reported by Sharma et al (1999) and are given at Table 2.6. It is observed that in most of the cases the earlier estimates were too much on the lower side. One of the reasons could be that hydrology is a data based science and with occurrence of more severe events, earlier estimates are bound to be revised upwards. Further, the design floods of many of these old dams were arrived at by using the empirical formula derived in the 1800's, which need upward revision in view of occurrence of more severe events in the interregnum. Moreover, the list of 62 dams, appearing in the above paper by Sharma et al (1999), is not a random sample and represents perhaps the worst out of a population of nearly 3700 existing large dams in the country. Hence, no direct conclusion can be made right now, without reviewing the hydrologic safety of all large dams in the country in a phased manner.

Table 2.6 : Summary Results of Hydrologic Review of Dams

Sl. No.	Name of Dam	Site	Design flood used in design (cumecs)	Design flood as per review (cumecs)
(1)	(2)	(3)	(4)	(5)
LIST A				
1.	Pagara	M.P.	1337	4692
2.	Pillowa	"	1337	6731
3.	Kotwal	"	1247	5947
4.	Gandhi Sagar	"	21200	54390
5.	Tigra	"	1455	4067
6.	Kaketo	"	1811	5728
7.	Barna	"		
8.	Aoda	"	1168	3089
9.	Hirakud	Orissa	42474	69632
10.	Darjang	"	2831	4130
11.	Ghodahada	"	906	1900
12.	Saroda	"	656	915
13.	Bhanjanagar	"	175**	1250
14.	Behera	"	456	1030
15.	Ganianala	"	128	380
16.	Jharnai	"	104	274
17.	Alikuan	"	166	630
18.	Parbati	Rajasthan	NA	7150
19.	Matri Kundia	"	NA	8125
20	Alnia	"	NA	2605
21.	Galwa	"	NA	4010
22.	Sathnur	Tamil Nadu"	5664	*
23.	Pechiparai	"	1104	5238
24.	Manimuthar	"	1710	4965
25.	Uppar	"	708	5344
26.	Ponnaniar	"	199	846
27.	Gomukhinadhi	"	728	2834
28.	Vidur	"	1768	6167

* Flood peak discharges (under PMF conditions upto Sathnur) for
(a) Catchment upto Krishnagiri Dam 7883 cumecs
(b) Catchment below Krishnagiri dam and upto Sathnur 17942 cumecs
** Spillway capacity since original design flood value is not available

Table 2.6 : continued

Sl. No.	Name of Dam	Site	Design flood used in design (cumecs)	Design flood as per review (cumecs)
(1)	(2)	(3)	(4)	(5)
LIST B				
1.	Badjore	Orissa	329	542
2.	Banksal	"	420	976
3.	Damsal	"	436	1093
4.	Kalo	"	965	2235
5.	Kodigam	"	243	447
6.	Kumbbo	"	231	703
7.	Nesa	"	230	400
8.	Pilasalki	"	793	1785
9.	Sanmachhakandana	"	228	420
10.	Talkhol	"	157	333
11.	Chittar-I	"	235	944
12.	Chittar-II	"	265	1220
13.	Kodayar-I	"	257	1247
14.	Kodayar-II	"	787	2480
15.	Tambraparni	"	2549	5430
16.	Servalar	"	1820	4288
17.	Perunchani	"	878	6091
18.	Amravathy	"	4250	6542
19.	Willingdon	"	NA	1590
20.	Gunderipallam	"	NA	1418
21.	Sidhamalli	"	450	1920
22.	Jawai	Rajasthan	1900	6469
23.	Morel	"	NA	23457
24.	Gambhiri	"	NA	8144
25.	Sei	"	NA	1756
26.	Sampna	M.P.	600	158
27.	Chandora	M.P.	NA	1000
28.	Bundala	M.P.	NA	1200
29.	Manimuktha	Tamil Nadu	NA	4484

(Source, Sharma, et al, 1999).

CHAPTER 3

ANALYSIS OF HYDROLOGIC CHARACTERISTICS

3.1 LOCATION

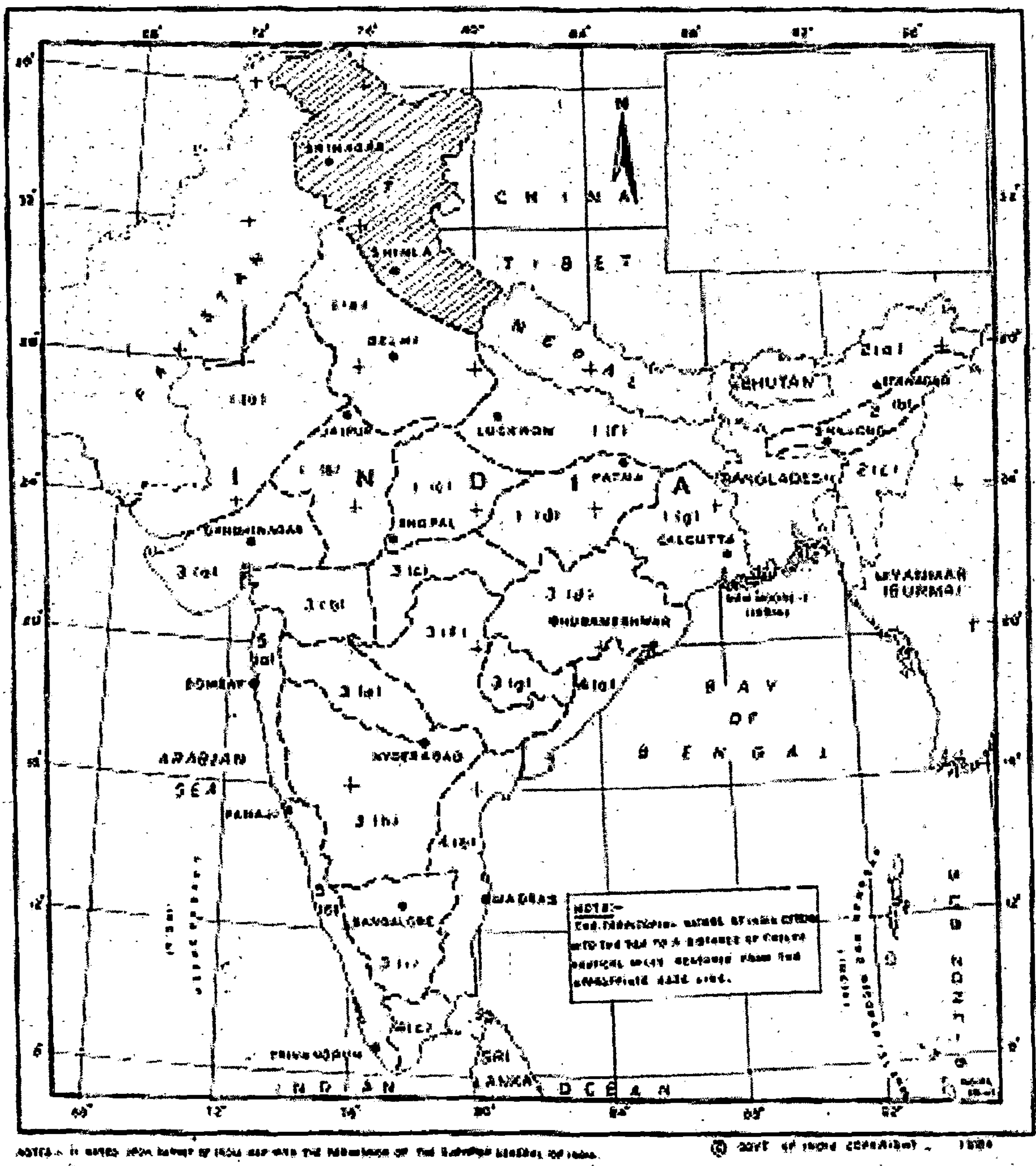
Western Himalayas zone covers the states of Jammu & Kashmir, Punjab, Himachal Pradesh and Uttaranchal in India and is located between longitudes 73^0 E to 80^0 E and latitude 29^0 N to 37^0 30' N. Fig. 3.1 shows location of the zone in map of India. The zone is bounded by international boundaries in the north and north-east (China), west (Pakistan) and in the east (Nepal). The study area for this dissertation consists of the Upper Yamuna, Upper Ganga and Kali basins in the Himalayan region, lying in the states of Uttaranchal and Himachal Pradesh. (Fig. 3.1)

3.2 RIVER SYSTEM OF STUDY AREA

A map showing the river system is at Fig. 3.2 and a schematic diagram showing the location of the river valley project sites chosen in the study is at Fig. 3.3. The individual component rivers of the system are described briefly in the following paragraphs.

Starting from the eastern slopes of the Shimla ridge to the eastern Nepal, Ganga River System is a major drainage of the entire Himalayan region. The important components of the system are:

a) River Yamuna and its tributaries: River Yamuna, which is the largest tributary of Ganga, originates from the Yamunotri glacier lying on the southwestern slope of the Bundar Punch peak. The Tons, which is the biggest tributary of the Yamuna, originates from a glacier near Rupin pass as Rupin river and is joined by another feeder stream, namely, Supin river, at Naitwar to form Tons. Supin river originates from the famous tourist spot of Har-ki-dun valley, which is a large glacial amphitheatre hemmed by snow-clad peaks



(Source: CWC,1994)

Fig. 3.1: Location of Zone 7 in map of India

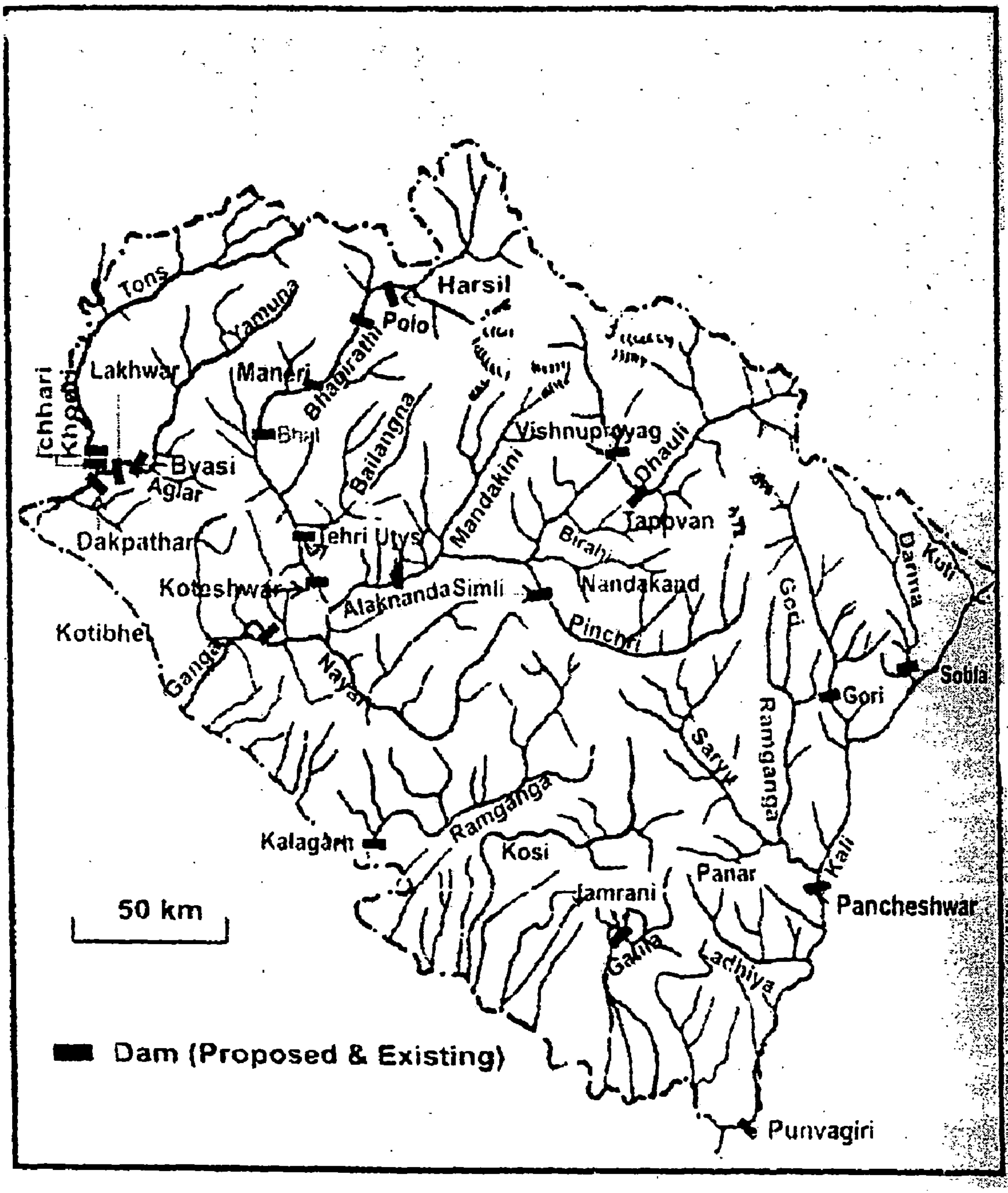
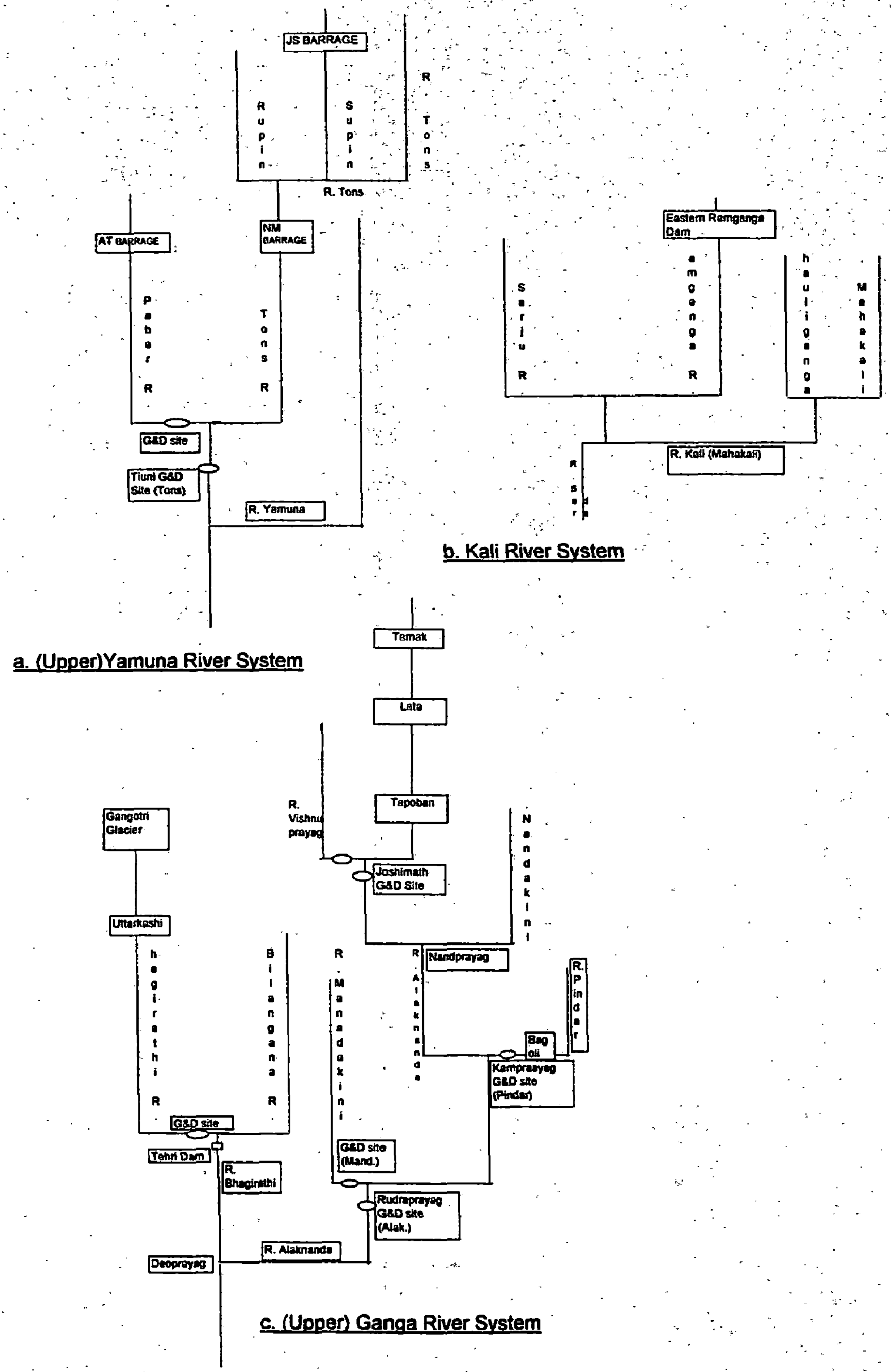


Fig. 3.2: River System of Study Area

(Source: Jag Mohan, et al,2003)



a. (Upper)Yamuna River System

b. Kali River System

c. (Upper) Ganga River System

Fig. 3.3: Schematic Diagram of the River System

of main Himalaya on all sides. Supin catchment is heavily forested and the Supin river flows through a series of falls and rapids before meeting Rupin river. Upper areas of the catchment of river Rupin are also covered with snow. River terraces are seen in the middle reaches of this river on both the banks. Further downstream, Tons river is joined by Pabar river at Tiuni, flowing from south-facing slopes of Dhauladhar range near the boder between the states of Uttaranchal and Himachal Pradesh. River Pabar flows in a south-westerly direction in its initial reaches and then takes a U-turn and then flows in a nearly south-easterly direction to meet Tons at Tiuni.

b) Tons joins the river Yamuna near Kalsi and brings in roughly double the amount of water as compared to the Yamuna at its junction. In between, Aglar river, which rises at the base of the Yamuna-Bhagirathi water divide, joins Yamuna, from the left, near Yamuna bridge. Giri river, which rises in east Central Himachal Pradesh merges with the Yamuna near Paonta from the right bank.

c) Ganga and its tributaries: The main tributary system of the Ganga (in the study reach) are:

- the river Yamuna and its tributaries as explained above
- the rivers Bhagirathi and Alaknanda
- the Kali river and its tributaries
- smaller rivers such as Song and Ramganga draining directly to Ganga
- other tributaries such as Gagra, Gandak, Kosi etc, which are outside the study area

These are briefly described below;

River Bhagirathi rises from near Gaumukh peak (Gangotri glacier) in Uttarkashi district and it meets tributary river Bilangana at Tehri where the Tehri dam has been constructed. Further downstream, it meets river Alaknanda at Deoprayag, beyond which the merged river is called river Ganga. Alaknanda rises from the glaciers to the north of the temple town of Badrinath and is joined by many tributaries such as river Vishuganga at Vishnuprayag, river Nandakini at Nandprayag, river Pindar at Karnprayag, river Mandakini at Rudraprayag. Catchments of all these rivers receive significant amount of precipitation in the form of snowfall.

River Nandakini originates as two streams from glaciers at the Trisul massif , which join together to form river Mandakini, and on its way to finally meet river Alaknanda at Nandprayag, it is joined by several small rivers.

River Mandakini originates as two streams from glaciers in the Kedarnath valley, which join together to form river Mandakini, and on its way to finally meet river Alaknanda at Rudraprayag, it is joined by several small rivers such as Kakra Gad, Chandrapuri Gad, Lastar Gad etc.

River Pindar is an important left bank tributary of river Alaknanda and rises from the Pindari glacier. Rishiganga river is an important tributary of this river.

Nayar river is the most important river draining the Kotdwara Satpuli area of Garhwal and originates from the southern slopes of the Pauri ridge.

River Ramganga rises from the south-eastern part of the water-divide with Alaknanda, is principally a river fed by underground springs and it flows through Korbett National Park and a major dam exists on this river in this region.

Kali River: It form border between India and Nepal flowing in a more or less SSWdirection along a narrow V-shaped valley.

Sarju river rises in the area to the north-west of Baijnath in central Kumaun and joines river Kali at Pancheshwar, where a dam has been proposed.

Ramgaganga (Sarju) river is a tributary of Sarju, and originates from a glacier on the south-east slopwes of the water-divide between Garhwal and Mumaun.

3.3 GENERAL CHARACTERISTICS

The Himalaya may aptly be divided into the following distinct physiographic divisions from South to North. They are,

i) Terai regions, which are plain areas, characterised by intensive agricultural activities and high population density.

ii) Siwaliks Hills, which form a series of low hills roughly parallel to the main Himalayas, and separated from the lower Himalayas by valleys, known as Duns, such as Dehradun, Patlidun etc. Siwaliks are profusely forested and receive abundant rainfall. This region also support high population density. The Siwaliks have a remarkable even crest between 750 - 1500 m and are profusely forested.

iii) Lower Himalayas, which lie to the north of Siwaliks and to the south of Higher Himalayas and have elevation range of 1500 - 2700m. There are several lakes in this zone, such as Naini Tal, Diuri Tal etc.

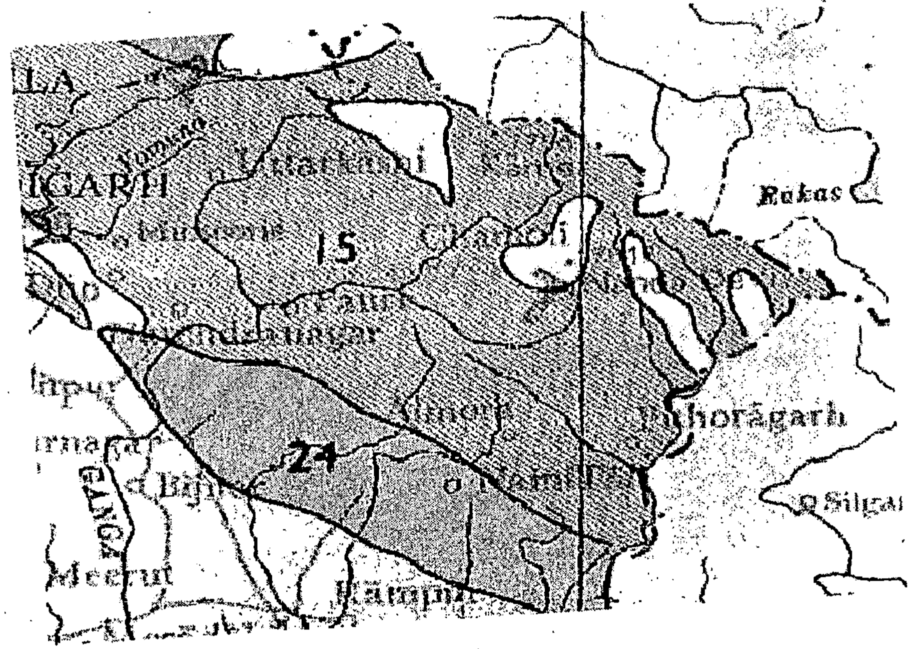
iv) Higher Himalayas, which lie broadly at the periphery of Indian sub-continent and have some of the highest peaks in this region. This region is home to many glaciers. This zone has an average width of around 50 km and elevation range of 4000 m to 6000 m.

v) Trans-Himalayas, is a vast table-land, with cold arid climate and sparse population. The area is unfit for cultivation and has very low rainfall.

The topography, soils and land-use pattern of the study area are shown in Fig. 3.4, 3.5 and 3.6 respectively. It can be seen from the map that in the study area, except for the

Note: Contours and spot levels in m.

Fig. 3.4: Topographical Map of Study Area



Mountain Meadow soils   Brown Hill soils

Fig. 3.5 : Soil Classification

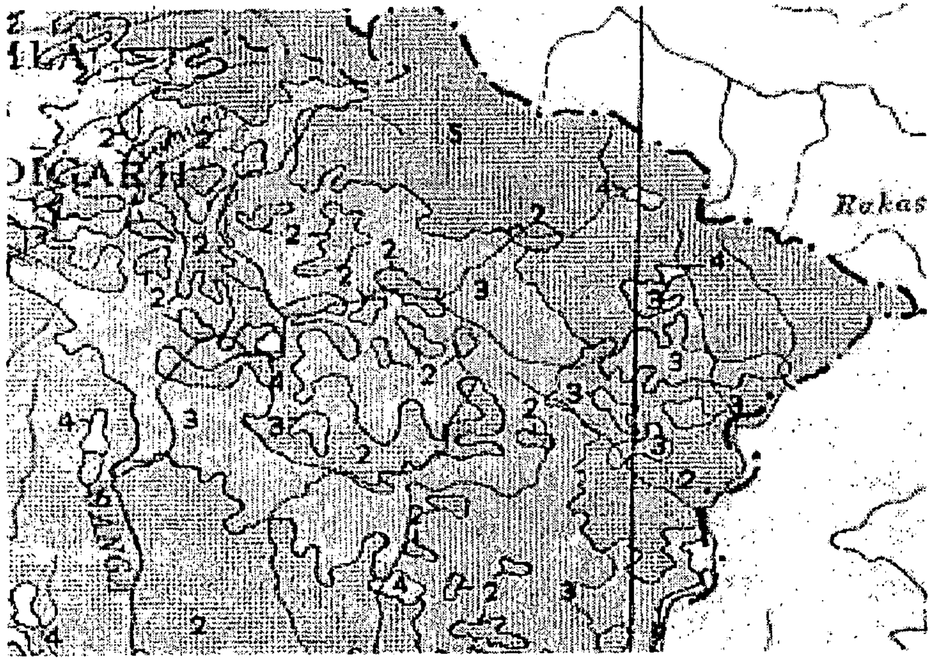
terai region, the whole area is having mostly brown hilly soils (from sandstone and shale) and except for some area under wheat and rice cultivation, most of the area is under forest and higher elevations are wasteland.

Because of the rapid changes in elevation and rain shadow effects, the climatic conditions are extremely varied and are strongly correlated with altitude and latitude. Since, latitude is not much changing in the study area, the following climatic regions can be identified based on altitude. (Negi 1982, as quoted by Negi, 1991).

Table 3.1: Climatic Regions in Study Area

Clima tic region	Elevation range (m)	Precipitation characteristics	Annual mean temperature (C)	Natue of Vegetation
Arctic	>4500	Most of the time snow covered	below 0	Grass (short period)
Sub-arctic	3500-4500	Receives substantial snowfall	0 - 4	Pine, juniper, deodar, chilgoza pine
Temperate	2000-3500	Cool zone	10-14	Oak, deodar, pine, fir, spruce
Sub-tropical	700-2000	Zone of maximum precipitation	19	Chir pine, dry and wet evergreen
Tropical	<700	Warm humid	20	Sal

D. S. Upadhyaya and others (1982) studied the variation of precipitation with altitude in the area and they have found a strong correlation between the two. The zone of maximum precipitation has been found to be between 1200m and 2000m. Analytical details of the distribution of rainfall with altitude have been studied by many authors, such as Singh (1995), Singh and Kumar (1997) etc. as per information given by Pratap Singh. (Singh and Singh, 2001). The normal annual rainfall and its typical monthly distribution pattern are shown in Fig. 3.7. The design rainfall depths of 25-year, 50-year and 100-year return periods for duration of 24-hour are shown in the maps at Fig. 3.8, 3.9 and 3.10 (CWC, 1994) respectively. Tables below show the heaviest 24-hour storm rainfall, mean annual rainfall and short duration (1-hr, 3-hr etc.) rainfalls at some of the



2: Arable Land   3: Forest
4: Grassland and shrubs   5: Wasteland

Fig. 3.6: Land Use

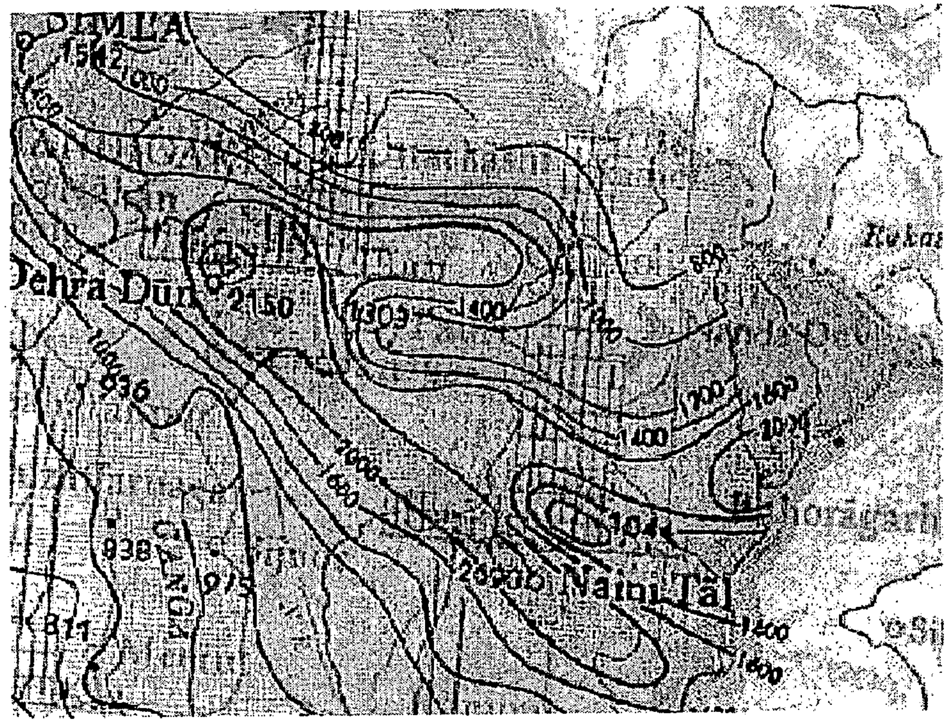
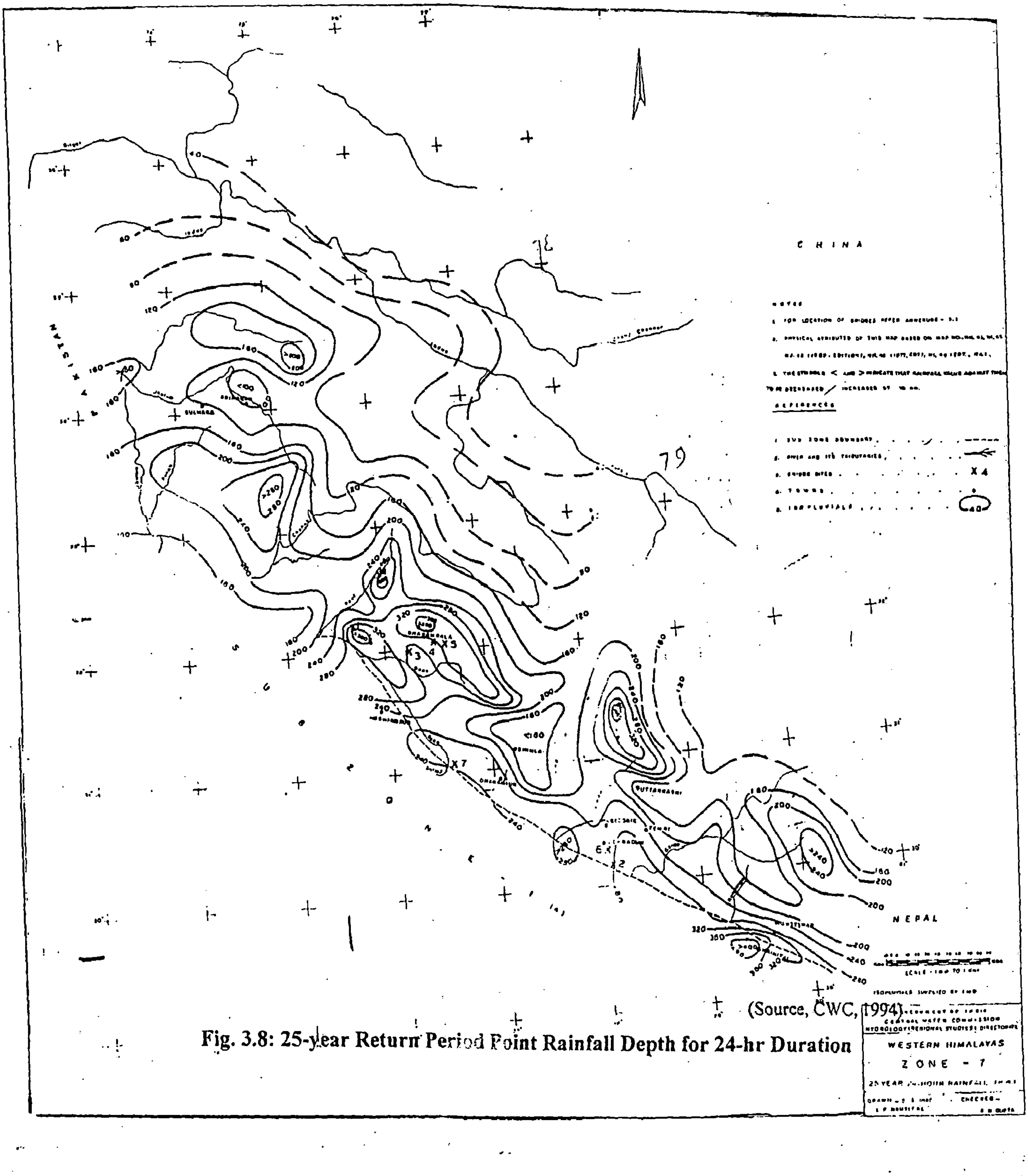
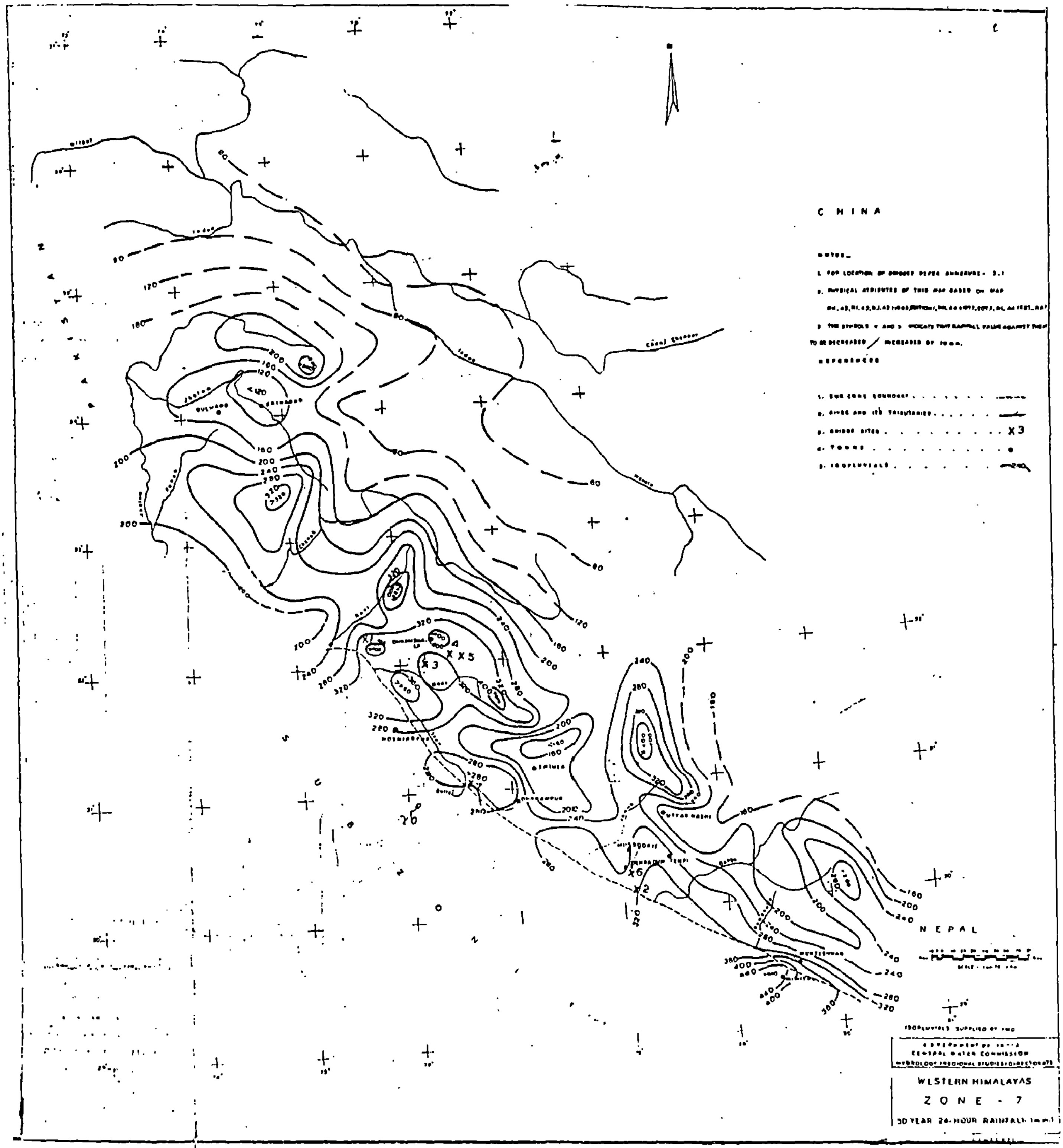


Fig. 3.7: Normal Annual Rainfall (mm)



(Source, CWC, 1994)

Fig. 3.8: 25-year Return Period Point Rainfall Depth for 24-hr Duration



(Source, CWC, 1994)

Fig. 39 :50-year Return Period Point Rainfall Depth for 24-hr Duration

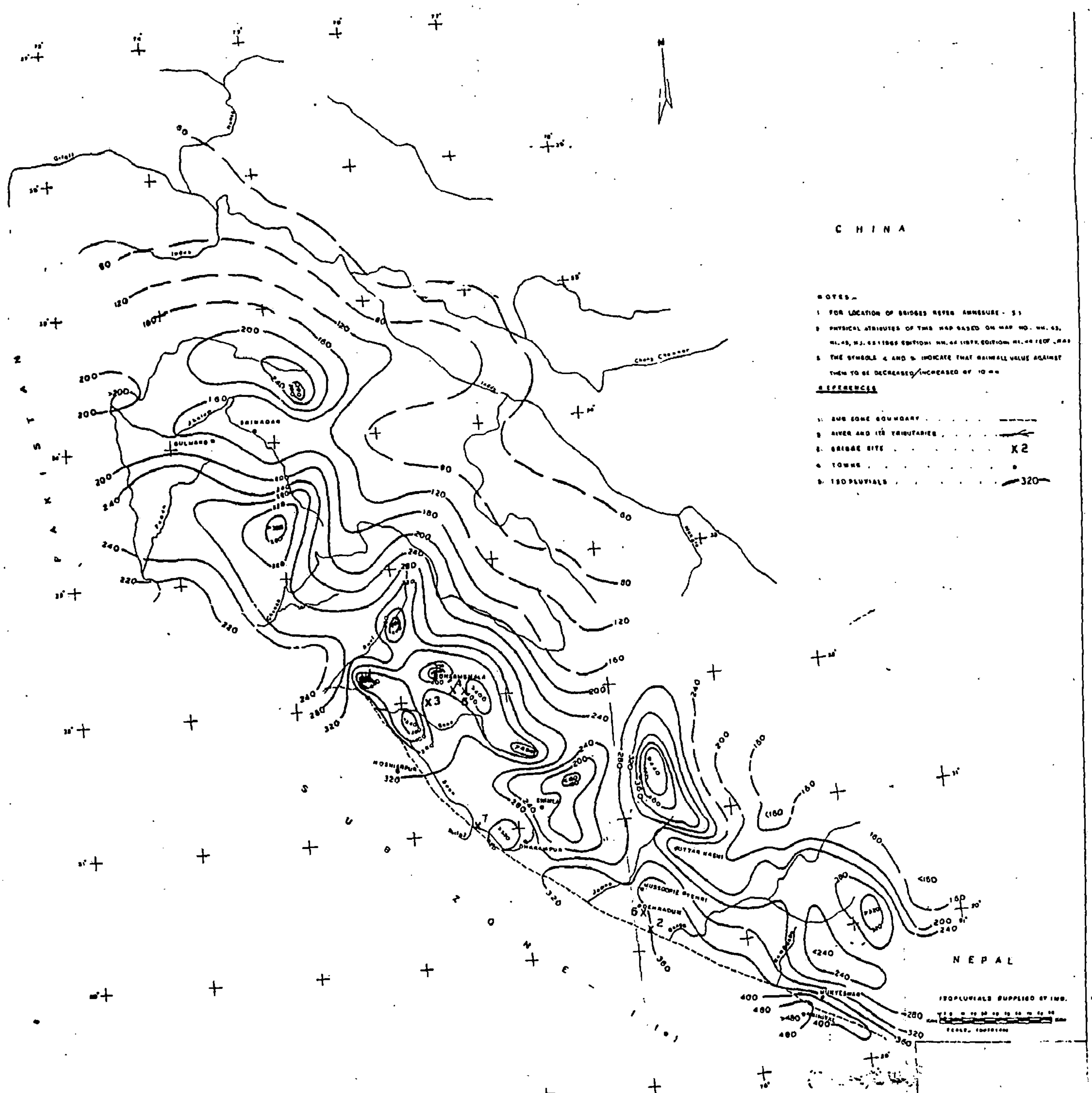


Fig. 3.10: 100-year Return Period Point Rainfall Depth for 24-hr Duration

stations. This information is very useful for design storm studies to ensure that the critical storm sequence utilized is reasonable based on observed rainfall data.

Table 3.2: Heaviest 24-Hour Rainfall At Stations In Study Area

District	Heaviest 24-hour Rainfall at stations in study area			Annual normal rainfall (mm)
	Station	Rainfall (mm)	Date of Occurrence	
Pithoragarh	Askote	450	05.09.1982	1853
Uttarkashi	Kharsali	400	15.09.1963	1934
	Uttarkashi	123	13.07.1979	1934
	Naitwar	210	10.02.1986	
Almora	Champavat	390	27.09.1897	1407
Garhwal	Kotdwara	349	27.08.1892	1756
Chamoli	Joshimath	273	21.07.1970	1247
	Chamoli	300	04.08.1991	
	Tapoban	190.5	27.07.1952	1247
Tehri Garhwal	Mukhim	121	16.09.1963	1742
	Tehri	91	21.07.1971	1742
Dehradun	Dehradun	487	25.07.1971	2168

Table 3.3: Heaviest Short Duration (1-Hr, 3-Hr, 6-Hr) Rainfall At Stations In Study Area

Heaviest short duration (1-hr, 3-hr, 6-hr) Rainfall and date of occurrence at stations in study area						
		For duration of				
District	Station	1-hr	3-hr	6-hr	12-hr	24-hr
Uttarkashi	Uttarkashi	**100** 12.07.1979	**103** 12.07.1979	**106** 12.07.1979	**108** 12.07.1979	**123** 12.07.1979
Tehri Garhwal	Tehri	**50** 21.07.1971	**90** 21.07.1971	**90** 21.07.1971	**90** 21.07.1971	**91** 21.07.1971
Dehradun	Dehradun	**98** 14.06.1970	**143** 14.06.1971	**189** 24.07.1973	**215** 24.07.1974	**331** 24.07.1975

3.4 HYDROLOGIC AND OTHER CHARACTERISTICS

All the rivers of the study area are marked by certain common features, which are described below:

- Some of these rivers originate from glaciers in Higher Himalayas.

- Upper areas of the catchments receive substantial amount of precipitation in the form of snowfall, especially during winter and also during the period of western disturbances, typically from January to May.
- These rivers have significant snow-melt runoff, making them perennial rivers
- Very steep descend in levels in the initial reaches, followed by steep bed gradients of the order of 15-30 m/km (0.015 to 0.030), before entering the Siwaliks.
- Very narrow gorges in the initial reaches, with river terraces at certain locations. Narrow channel, high channel slope and the absence of wide flood plains, result in short-duration high magnitude flood peaks. (Mathur, et al 1993).
- Heavy load of boulders and suspended particles, especially during periods of heavy rainfall. Sediment concentration, including bed load, of the order of 15-25 ha.m.per 100 sq.km. catchment area per annum.

3.4.1 Snow-Cover And Glacial Features: The snow-line is defined as a irregular line located along the ground surface where the accumulation of snow-fall equals the ablation, ie., melting and evaporation. (Jeyram, et al, 1982). In a study of adjacent Tos basin in Himachal pradesh, in which landsat imageries had been studied, (Jeyram, et al 1982), it was observed that the snowline comes down to the altitude of 2800m during winter and goes upto 4800m towards end of summer season (October).

3.5 CHARACTERISTICS OF THE SELECTED CATCHMENTS

The Central Water Commission of India has published a report (CWC, 1994) for developing synthetic unit hydrographs for ungauged catchments, in zone 7, Western Himalayan region in India. The report contains certain relations for development of synthetic unit hydrographs. The report in addition also contains design rainfall depths of return period 25years, 50 years & 100 years, design base flow rates, etc. Zone 7 covers the states of Jammu & Kashmir, Punjab, Himachal Pradesh and Uttaranchal (hilly areas).

In this report, data from 6 catchments were utilized for developing regression between UG parameters and catchment physiographic parameters. However, due to data constraints, all the catchments selected were from the fringe of the zone, from the plain areas, and hence, are not fully representative of the mountainous nature of the zone. The physiographic parameters of the catchments used in the CWC study are given in Table below.

Table 3.4: Physiographic Characteristics Of Catchments Used In CWC Study

	Site No.	C.A. (km^2)	L (km)	L_c (km)	S_e (m/km)
1	232	657.86	51.49	24.13	9.9
2	139	296.84	46.5	20.5	13.95
3	821	151.98	21.35	12	84.04
4	629	103.6	21.32	11.58	69.21
5	154	43.82	13.1	7.5	13.85
6	SEWA HEP	383	40.22	16	20.59

Since, all the above catchments are lying towards the plain areas, it was considered necessary in the present study to include catchments from the upper reaches also to have a fair representation of the catchment characteristics. The physiographic characteristics of the catchments studied in this dissertation are as below: Details of computation of the equitvalent slope is given in Annexure 3.1.

Table 3.5: Physiographic Characteristics Of Catchments Used In This Study

Catchment name	Catchment area (km^2)	Area under permanent snow-cover (%)	Non-snow covered area (%)	Slope of main stream (S_{st}) (m/km)
AT	1343	22.3	77.7	20
JS	281.41	21.3	78.7	28
NM	1105	13.6	86.4	31
TAP	1610	22.5	77.5	17
LATA	3100	61.3	38.7	25
TAMAK	2313	65.5	34.5	28(*)
NPL	6233	31.7	68.3	10(*)
BAGOLI	1610	22.55	77.45	17
E.Ramganga	1144	4	0.35	9.8

(*) Note: Approximate.

The runoff characteristics of these selected catchments in terms of mean annual runoff (based on 27 years of observed /computed discharge data) and mean 10-daily runoff are as given below:

Table 3.6: Runoff Characteristics Of The Selected Project Catchments

Project site	ANNUAL RUNOFF (mm)			ANNUAL RUNOFF (cumec-day/sq.km.)			10-daily runoff (cumec-day /sq.km.)		
	Minimum	Mean	Maximum	Minimum	Mean	Maximum	Minimum	Mean	Maximum
AT	498	948.5	1803	5.76	10.98	20.87	0.004075	0.030078	0.2566
JS	624	1568.4	3232.8	7.22	18.15	37.42	0.0030	0.0498	0.3526
NM	624	1568.4	3232.8	7.22	18.15	37.42	0.0030	0.0498	0.3526
TAP } LATA } TAMAK}	785	1193	1951.4	9.09	13.81	22.59	0.0056	0.0383	0.2055
NPL	838	1241	2201	9.70	14.36	25.47	0.0026	0.0269	0.1575
BAGOLI	809	1132.7	1444.68	9.36	13.11	16.72	0.0057	0.0539	0.5289
E.Ramganga	964.8	1714.3	2655.4	11.17	19.84	30.73	0.0045	0.0541	0.4853

It can be seen from the above that the region is homogeneous in terms of mean annual runoff, except for the two sites JS and NM. These two sites have lower percentage of snow cover area in their catchments as compared to other catchments. In terms of the mean, minimum and maximum of the 10-daily runoffs, it can be seen that the values are comparable and the range of variation is also not very large. This is because of the same broad climatic and hydrological features affecting all the catchments. The higher value of minimum flows at the four sites, TAP. LATA, TAMAK AND BAGOLI can be explained by referring to the earlier table, in which, it can be seen that these sites have higher percentage area under snow cover. The pattern of 10-daily flows as percentage of mean annual flows, for the year of mean flow is given in Fig. 3.11. The high values for site No. 5 is because of the high snowmelt at that particular site in the months of August and September and is found to be characteristic of the site in other years as well.

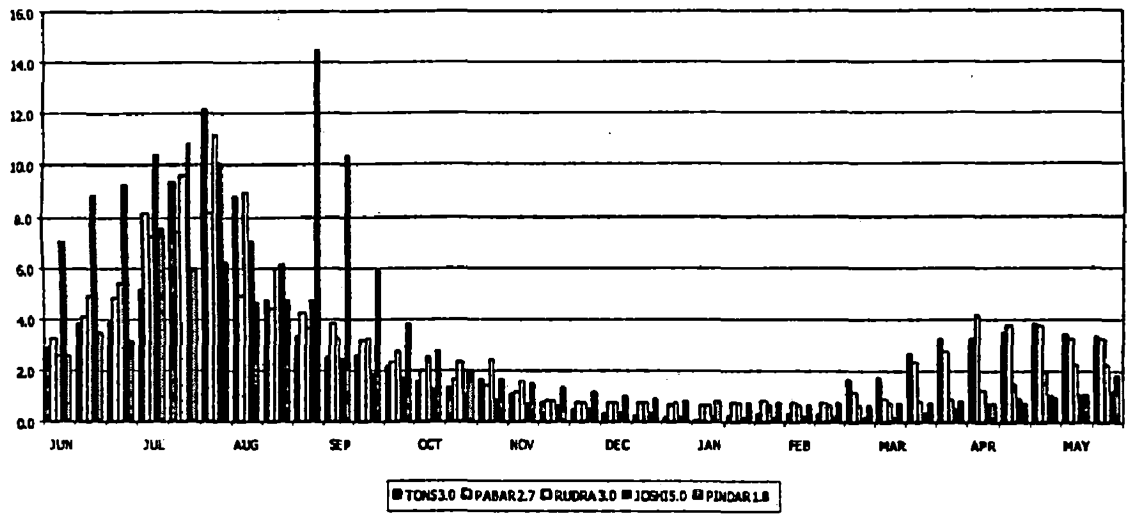


Fig. 3.11 Pattern Of 10-Daily Flows As Percentage Of Mean Annual Flow For A Typical Year At Various G&D Sites In The Region

3.5.1 Snow-melt and Baseflow Characteristics: In the case of catchments not having any area under permanent snow-cover, the baseflow obtained from flood events of summer months, will represent baseflow contribution from groundwater storage and delayed runoff only. The Himalayan region is not homogeneous from this point, since, several rivers are receiving significant amount of flows from springs. The catchments (Table 3.4) in zone 7, are not having major springs in their catchment. Based on analysis of flood events of these catchments, a median baseflow rate of 0.05 m3/s /sq.km. was obtained.

However, in the case of snowfed-rivers, the melting of snow-cover during the summer months may coincide with the flood flow from rainfed areas giving rise to critical combination of rainfed runoff and snowmelt runoff. The peak rates of runoff observed at the G&D sites in the summer months of April to Septmebr, for periods of no-rainfall have been taken to represent the sum of summer baseflow and critical snow-melt values. The G&D sites had observed data for around 27 to 28 years. The data of critical snowmelt values at G&D sites were transposed to the nearby project sites in proportion to snow-cover area. The snow-melt values adopted for the projects are given in Table below.

Table 3.7 Critical snowmelt used for Project Catchments Studied

Catchment name	Catchment area	Area under permanent snow-cover (%)	Critical Snowmelt used ($m^3/s/km^2$)
AT	1343	22.3	0.669
JS	281.41	21.3	2.667
NM	1105	13.6	1.333
TAP	1610	22.5	0.300
LATA	3100	61.3	0.300
TAMAK	2313	65.5	0.300
NPL	2000	68.6	0.405
BAGOLI	6233	31.7	0.551
E.Ramganga	1144	0.35	—

A regression was attempted on this data and it was found that the value used for the site JS is not fitting in the general trend and hence, was removed in regression. A linear regression did not give satisfactory result and hence, power regression was done. The result of regression study is shown in Fig. 3.12 below. The high value of coefficient of determination shows the significance of the regression.

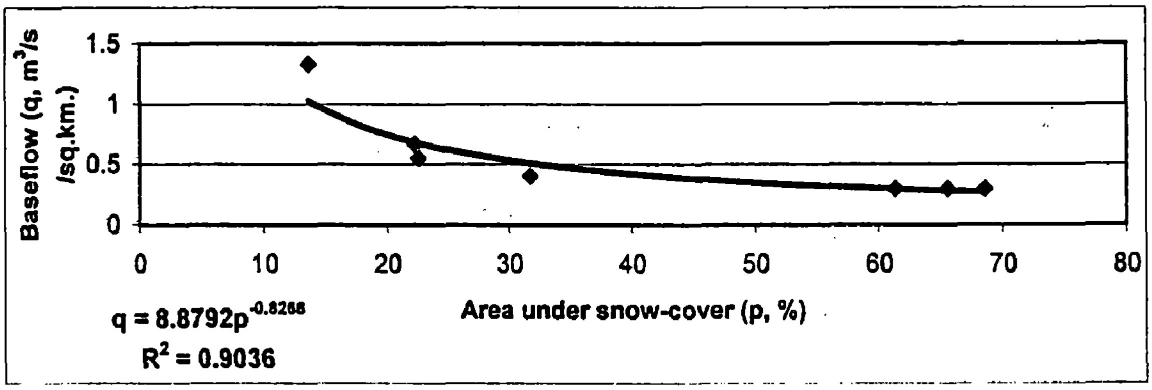


Fig. 3.12 Relation Between Snow Cover Area And Design Base Flow

3.5.2 Assumptions And Limitations Of The SUG Approach: The relations between catchment characteristics and UG parameters were derived from the data of nearly 45 flood events from 5 sites in the Western Himalayan river basin. As can be seen from Fig. 3.2, all these catchments are lying in the fringe of the basin, towards the plain areas and hence, are not fully representing the hydrological and meteorological characteristics of catchments in the basin.

Further, 5 data sets is too small a sample for regression analysis, since, the variance of the computed coefficients will be too large and hence, the 95% confidence limits of the estimated values of the dependent variable, i.e., UG parameters, will be too wide apart to be of any use in flood computations. These values can be used as a preliminary value to start with, which needs to be modified /firmed up based on data of G&D observations and short-interval rainfall data.

CHAPTER 4

FLOOD ESTIMATION FOR UNGAUGED CATCHMENTS –CASE STUDY

4.1 INTRODUTION

Flood estimation procedures for ungauged catchments using various formulae and unit hydrograph approach have been a matter of intense research in the field of hydrology. For regionalisation of the unit hydrograph, the attempt has been to correlate the catchment physiographic characteristics with the principal parameters defining the UG, such as, peak time, peak ordinate etc. The Rational formula and the Snyder's method were the most commonly used methods for flood estimation for ungauged catchments till recently. A critical review on the use of the Rational formula, the Snyder's method and the Modified Dicken's formula are given below.

4.1.1: Review of the Rational Formula: The origin of the Rational Formula is somewhat obscure. In the US, it is often referred to as the Kuichling method and in Great Britain, as the Lloyd-Davies method. (Singh, 1988, pp 120). The rational formula presents the concept of time of concentration and its relation to maximum runoff. This formula expressed as:

$$Q = C.P.A, \text{ where,}$$

Q is the peak discharge, C is the runoff coefficient depending upon the drainage characteristics, A is the area of the drainage basin and P is the average intensity of rainfall over the storm duration, which is generally taken as equal to the time of concentration (T_c) of the drainage area for estimation of maximum flood. Even though the equation is dimensionally balanced (L^3T^{-1}) on both sides, a conversion factor will be required, value of which will depend on the units used. Perhaps the word "rational" came from the fact that the equation was dimensionally balanced. Time of concentration (T_c) is "the time required for runoff to travel from the most distant point hydraulically (in time) to the outlet" (ASCE, 1996, p. 579, as quoted in http://www.hkh-friend.net.np/rhdc/training/lectures/HEGGEN/ Tc3.pdf). By such definition, T_c is distance traveled divided by mean water velocity, appropriately partitioned into reaches of reasonably uniform hydraulic characteristics. Traditional Rational Method usage employs this definition. As travel time also relates to rainfall intensity, McCuen (1998, p.140) suggests that T_c be associated with a 2-year 2-hour storm depth. The Rational Formula implicitly assumes that the whole of the catchment is contributing to the peak at a uniform rate. For large catchments, this is unrealistic. Although, Rational Formula has recently come under increasing criticism for its lack of physical realism, it continues to be used frequently in design of urban drainages and other small structures. This is not only true of US, but also of Australia, UK and India, to name but a few (UNESCO, 1977, as quoted by Singh, 1988, pp 125). Determination of time of concentration or time to peak, as the case be, becomes the most important step for flood estimation of ungauged catchments. Various formulae are available in literature. Singh (1988) has given an extensive list of formulae derived by various investigators for computation of peak time. The definition of peak time is also not unique and these have also been given in the sketch. The Table and sketch in Annexure 4.1 is an excerpt from the above list. Two of the popular formulae for computation of time of concentration for use in rational formula are given below:

Kirpich's formula: $\boxed{t_c = 0.01947 L^{0.77} S^{-0.385}}$

California formulae: $\boxed{T_c = \left[\frac{0.85 L^3}{H}\right]^{0.385}}$

L-Length of main stream (km),
S-slope of river ,
H-Height difference between outlet and highest point on the catchment (m)

4.1.2 Review of Snyder's Method:

Snyder (1938) was perhaps the first to have established a set of formulae relating the physical geometry of the watershed to three basic parameters of the UG. These formulae were derived from a study of 20 watersheds located mainly in the Appalachian

Highlands, which varied in size from 25 sq. km. to 25,000 sq. km. (Singh, 1988, pp 330). The basic parameter that Snyder defined is t_p, the time of lag to peak in hours taken as the time from the center of mass of the effective rainfall of unit duration to the peak of the UG or simply the watershed lag. (Fig. 4.1). He related all other important parameters of the UG, such as duration D, peak ordinate Q_p, and base time T_B as functions of t_p. Later on, US Army Corps of Engineers (1940, as quoted by Singh, 1988 at pp 331) developed a relation between Q_p and the width of the UG at values of 50% (W50) and 75% (W75). There had been many modifications to these formulae by individual researchers based on different data sets, and are applicable to the regions from which the data is derived.

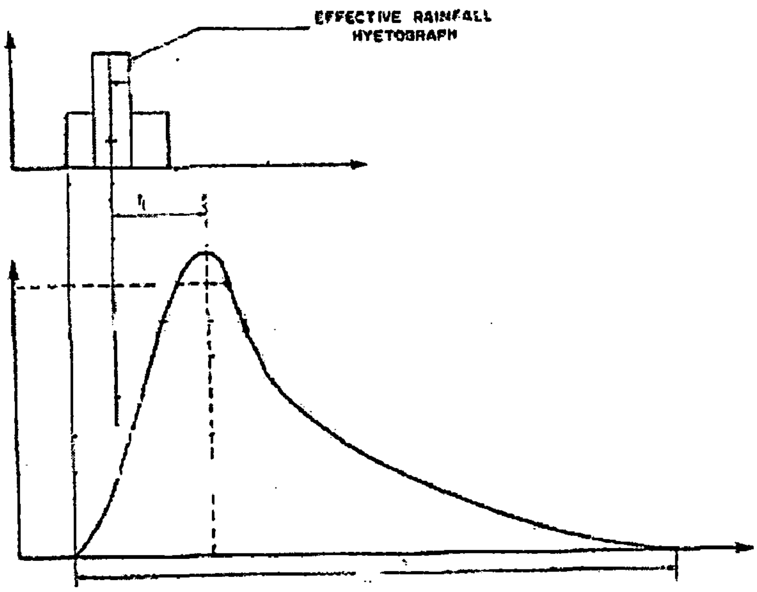


Fig. 4.1: Definition Sketch for Snyder's UG Method

$$t_p = C_t\,(L\,L_c)^{0.3},$$

where, L is the length of main stream and L_c is the length of the mainstream from the outlet to the point on the stream nearest to the centroid of the catchment area. C_t is a constant, depending on the topography and other features. Even within a region, substantial variations in value of this constant has been reported by many authors. Clark (1969, as quoted by Singh, 1988, pp 331) found the range to be 0.4 to 2.26 for Texas region, Linsley found the range to be 0.3 to 0.7 for NorthWest US and its range for Australia was found to be 0.4 to 2.24. Value of C_t will be lower for watersheds with higher slopes. Later on, Linsley incorporated catchment slope in computation of time to peak and, thereby improved the methodology. Linsley's formula for t_p is:

$$t_p = C_t * \left[\frac{LL_c}{\sqrt{S}}\right]^n$$

The unit duration of the effective rainfall (D) was defined as $\boxed{D = \frac{t_p}{5.5}}$

The peak Q_p of the UG is obtained as $\boxed{Q_p = C_p * \frac{A}{tp}}$

where, C_p is a constant, which again varies from region to region and within a region depending upon the physiographic characteristics of the catchment.

Further, the time base (T_b) of the UG is obtained as : $T_b = 3 + 3\left(\frac{t_p}{24}\right)$.

The rest of the points of the UG are so fixed as to get unit volume of runoff.

As can be seen, the determination of time to peak becomes the most important step for flood estimation of ungauged catchments. Various formulae are available in literature. Singh (1988) has given an extensive list of formulae derived by various investigators for computation of peak time. The definition of peak time is also not unique and these have also been given in the sketch and table in Annexure 4.1.

Snyder's Unit Hydrograph technique remained a popular tool for developing UG's for ungauged catchments for a long time. As stated above, since, the relations were developed based on data from Appalachian highlands region in US, direct adoption of the technique to India was not considered to be appropriate. With this in view, a Committee of Engineers headed by A. N. Khosla recommended that similar studies be done for various hydro-meteorologically homogenous regions of India and, in consequence, Central Water Commission (CWC) of India, published a report (CWC, 1994), for flood estimation for ungauged catchments in the Western Himalayan region.. This forms a part of a series of similar reports covering the various regions of India. The method prescribed in the report is being extensively used in India for design of road and rail bridges apart from the design of small hydraulic structures such as minor dams, small barrages, cross drainage structures, flood protection levees for rural areas etc. The report contains certain relations for development of synthetic unit hydrographs, broadly in the pattern of the technique suggested by Snyder. The report in addition also contains hydro meteorological inputs, such as, design rainfall depths of return period 25years, 50 years & 100 years, design base flow rates, design infiltration rates, etc.

4.1.3 Review of Modified Dicken's formula: The Dicken's formulae, evolved after a study by Col. Dicken in 1860's, is applicable to Northern Indian river catchments and is of the form:

$$Q = C.\ A^{3/4}$$

Where, Q is the peak flood in m^3/s, A is the catchment area in km^2 and C is a constant to be chosen based on the location, shape, slope of main river, etc. Col. Dicken derived this formula on the basis of his observations on four catchments of different sizes. He observed that the rainfall intensity is inversely proportional to the 4^{th} root of catchment area and hence, applying the Rational Formula, he could obtain the above relation. Apart from the shortcomings of the Rational Formula, this in addition, also has the disadvantage that the peak rate of runoff is modeled to depend on only one parameter, namely, catchment area; all other factors like, shape, slope, soil and vegetation, drainage pattern and density, location and rainfall intensity, etc. are ignored. To overcome this limitation, Uttar Pradesh Irrigation Department (UPID) brought out a technical memorandum, based on flood frequency study of 6 catchments in Uttar Pradesh state of India. Since, these catchments have significant area under snow cover, this was taken as a another parameter. The procedure in brief is as follows:

Constant 'C' of Dicken's formula is computed using the following relation:

$$C = 2.342*\log(0.6T)*\log(1185/p) + 4$$

Where, $P = \frac{a+6}{A} * 100$;

A is the catchment area (km^2) and a the area under permanent snow cover (km^2)

Further, C is to be corrected for the slope of the main river, if the value of C for T=1000 is less than the value given below, by multiplying by the ratio of C_{table}/C_{1000}

Slope of main stream (m/km)	Value of C_{table}
Upto 15	18
15 to 20	24
20 to 25	25

Some of the short-comings of the above formula are:

- The peak discharge data at some of the sites used in derivation of these formulae are not observed values, but computed by a mass-balance procedure at existing barrages. Since, the mass balance computations involve arithmetic operation of several variables, the expected variance of the computed peak will be too large to make the study unreliable.
- In the derivation of these formulae, 5 out of the 6 catchments used are in plain areas and hence, do not fully represent the hydrologic behaviour of mountainous rivers.
- The value of C_{table} jumps from 18 to 24 when the slope band change from <15 to 15-20. It would have been better to correlate the two variables by a smooth curve, to avoid such abrupt jumps.
- Many catchments in the upper areas have slopes well above 25 m/km.
- The derivation involved a prior assumption that the relation will be broadly of the form suggested by Col. Dicken. No attempt was made to verify the exponent ¾ in Dicken's formula.

4.1.4 Regional Frequency Approach:

Regional flood frequency approach can be used to estimate floods of different return periods at ungauged locations. Even for gauged locations, a regional approach can be applied, since; the effective data length gets increased. A detailed procedure for carrying out regional flood frequency analysis is described by USGS, in the USGS manual (USGS, 1960). This procedure is also called index flood method. Based on this procedure, a statistically homogeneous region is to be identified and thereafter, two sets of curves are developed, one relating the mean annual flood value with catchment characteristics and the other relating the return period (T) with the ratio of return period flood value (Q_T) to mean annual flood value, based on stations within the region.

Garde and Kothyari (1988, 1992) have reported a study of 93 stations located all over India, in which the mean annual flood ($Q_{2.33}$)was correlated with catchment area (A), slope, precipitation intensity (of a particular duration) and forest area ratio. They have reported 6 different sets of relations based on geographic classification. The combined relation had a coefficient of multiple regression of 0.96 and the maximum error of the computed values was of the order of ±25%. Agrawal et al (1982) reported a similar study using data of 8 catchments in the Western Himalayan region with total data length of 433 station years. In this case also, the fitted relation between $Q_{2.33}$ and A shows a maximum error of around 25%. The fitted relation between T and $Q_T/Q_{2.33}$ ratio also showed a maximum error of around 30% from the observed value. Thus, the combined effect of both the curves, will be that the flood estimate is likely to have large variance. In another study by Agrawal et al (1991), in which data of rivers of Uttaranchal region was used, index flood method of USGS has been found to give better results as compared to the five parameter Wakeby distribution. However, they have not reported the maximum percentage error of the $Q_{2.33}$ values from the values computed by the fitted relation. A comparison of the relations developed in the three studies are given below.

Garde and Kothyari, for Northern India: $Q_{2.33} = 13.78A^{0.16}p^{1.2}S^{0.01}Fv^{0.3}$
Agrawal P.P. et al (1982): $Q_{2.33} = 37.4\ A^{0.5775}$
Agrawal C.K. et al (1991): $Q_{2.33} = 1.53A^{0.80}$

Considering that the second and third relations above have been developed for more or less the same region, such a large variation in the intercept and slope of the relation is not expected.

4.2 PROCEDURE PRESCRIBED IN CWC REPORT FOR ZONE 7

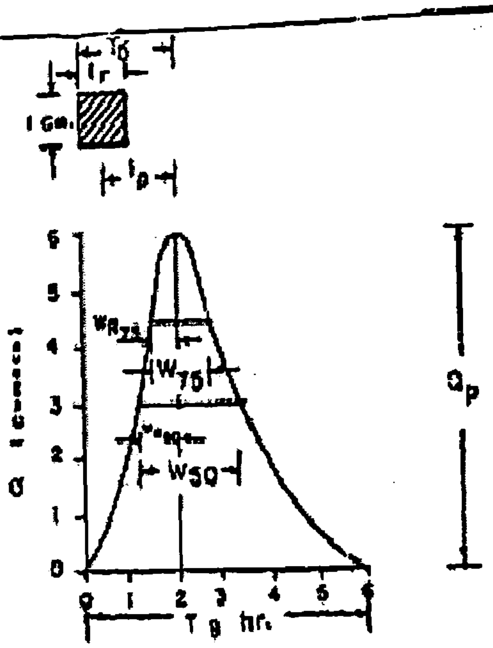
The following steps are involved for estimation of 25-year, 50-year or 100-year return period floods, for ungauged catchments. This report can be used only if the design storm duration, which depends on the base period of UG and the type of structure for which the design is being done, is equal to or less than 24 hours.

1. From relevant toposheets, catchment map, stream network, contours and important structures on the rivers (if any) are marked.
2. The catchment area, length of main-stream, length of main stream upto the centroid of the catchment (in plan) are measured.
3. L-section of the main river is drawn (or tabulated) and the equivalent stream slope, called, statistical slope (S_{st}) is computed using the following equation

$$S_{st} = \left[\frac{L}{\sum \frac{L_i}{\sqrt{S_i}}} \right]^2 \text{, where, } L = \sum_{1}^{n} L_i \text{ and}$$

L_i is the length and S_i is the slope of each segment to which the river (principal river) length is divided.

4. The parameters of the UG, such as, Tp, tr, Qp, etc. are computed. The meaning of these symbols is explained in Fig. 4.2
5. Sketch the UG and adjust the ordinates to make the volume unity.
6. The 25-year, 50-year or 100-year (as relevant) return period design storm depth of 24-hour duration can be read from the maps given in the report.
7. The duration of design storm is decided as below:
    a. For structures like bridges, culverts, cross drainage structures, small dams, barrages etc. for which peak and not volume is important, the duration generally used is equal to $1.1*t_p$, where t_p is the time to peak of UG from the center of rainfall excess hyetograph.



U.G : Unit Graph

t_r : Unit Rainfall Duration adopted in a specific study (hr.)

T_p : Time from the start of rise to the peak of the U.G (hr.)

Q_p : Peak Discharge of Unit Hydrograph (Cumecs.)

t_p : Time from the Centre of Effective Rainfall duration to the U.G Peak (hr.)

W_{50} : Width of the U.G measured at the 50% of peak discharge ordinate (hr.)

W_{75} : Width of the U G measured at 75% of peak discharge ordinate (hr.)

W_{R50} : Width of the rising limb of U.G measured at 50% of peak discharge ordinate (hr.)

W_{R75} : Width of the rising limb of U.G measured at 75% of peak discharge ordinate (hr.)

T_B : Base width of Unit Hydrograph (hr.)

A : Catchment Area (Sq.km.)

q_p : Q_p/A = Cumec per sq.km.

Fig. 4.2 Synthetic UG Parameters-DefinitionSketch

b. Flood control reservoirs and, generally for all cases where the effect of flood storage on flood moderation can be significant e.g., for design of spillway capacities and freeboards on large dams: In this case, the duration shall be such as to give maximum volume (WMO, 1994) and shall be at least equal to the base period of the UG.

c. In all other cases of design of dams, base period of UG, rounded off to the nearest 24 hours. (CWC, 1993).

8. The 24-hour storm depths obtained as above are converted to depths of desired duration using conversion factors given at Fig.3 of the CWC report.
9. The depth obtained as above is a point rainfall and needs to be reduced by multiplying by the areal correction factor given at Annexure 4.2 of the same report.
10. The hourly rainfall depths are obtained by multiplying the depth obtained at step 9, by the distribution coefficients obtained given at Annexure 4.1 of the report and design loss rate is applied. For 50-year return period, loss rate (phi-index) of 2mm /hr can be used.
11. By convolving the rainfall excess hyetograph with the UG, the DSRO is obtained, to which design baseflow (including any critical snowmelt, as applicable) is added to obtain the design flood hydrograph.

4.3 FLOOD ESTIMATTION FOR SELECTED PROJECT SITES

The salient features of the project sites for which the flood estimation by using the various approaches were done are described in para 3.5 of Chapter 3. The computation of slope of principal stream of the river is also described there.

4.3.1 Derivation of UG: Depending upon the availability of short duration rainfall and runoff data, or availability of UG at any sites in the vicinity, different methods have been employed for derivation of UG. These are shown in the Table 4.1.

4.3.2 Design Storm Depth And Distribution: Design storm depths for various return periods were obtained by using the charts given in the CWC report for zone 7. Based on the record of 24-h observed maximum storm depths in the region, the maximum 24-hr depth of 300mm has been obtained. This has also been used in flood computation. Uniform loss rate of 1.5 mm/hr has been used in all the cases, considering that the purpose is to compute a rare event, i.e., 100-year flood, and in such a case, not the average, but the critically low infiltration rates corresponding to saturated soil is to be adopted. (CW&PC, 1972). Considering the soil type, the annual rainfall pattern, vegetal cover, land slope etc. the adoption of this value is justified. The flood computations for a typical catchment by using the method given in this report are in Annexure 4.2.

Table 4.1 UG Characteristics of Catchments

<table>
<tr><th>Catch-ment name</th><th>Catch-ment area</th><th>Area under permanent snow-cover (%)</th><th>Method adopted</th><th>1-hr UG peak (m^3/s /km^2 /cm)</th><th>UG* Time to peak T_p (h)</th></tr>
<tr><td>AT</td><td>1343</td><td>22.3</td><td>SUG (*)</td><td>0.7807</td><td>6</td></tr>
<tr><td>JS</td><td>281.41</td><td>21.3</td><td>SUG(*)</td><td>0.8450</td><td>4</td></tr>
<tr><td>NM</td><td>1105</td><td>13.6</td><td>SUG(*)</td><td>0.7853</td><td>6</td></tr>
<tr><td>TAP</td><td>1610</td><td>22.5</td><td rowspan="3">Using observed discharge data at nearby site, UG is computed at G&D site. Flood computed at G&D site is transposed to project site in proportion to $A^{3/4}$.</td><td rowspan="3">0.2335 (3-hr UG)</td><td rowspan="3">12# (3-hr UG)</td></tr>
<tr><td>LATA</td><td>3100</td><td>61.3</td></tr>
<tr><td>TAMAK</td><td>2313</td><td>65.5</td></tr>
<tr><td>NPL</td><td>6233</td><td>31.7</td><td>By transposition of UG at nearby project site.</td><td>0.2336</td><td>12</td></tr>
<tr><td>BAGOLI</td><td>1610</td><td>22.55</td><td>Using observed discharge data at nearby site, UG is computed and transposed to project site</td><td>0.765</td><td>6</td></tr>
<tr><td>RAMGANGA</td><td>1144</td><td>0.4</td><td>SUG (*)</td><td>0.732</td><td>6</td></tr>
</table>

(*) Note: SUG means Synthetic UG by following the method of CWC report for zone 7.
(#) Note: UG for a nearby catchment with CA=1150 km^2 (excluding snow-cover area).

Flood peaks were also computed by certain other techniques, such as Modified Dicken's formula (UPIRI, 1979) and Regional Frequency Studies (Mutreja, 1992), for the purpose of comparison and for ensuring reliability of the estimates.

Table 4.2 100-Year Flood Peaks Estimated at the Location of Sites(*) by Various Methods

(Unit: Cumec /Sq. Km.)

S.N.	Method	AT	JS	NM	TAP	LATA	TAMAK	NPL	BAGO LI	E.Ram Ganga
1	Modified Dicken's Formula	2.573	3.852	2.833	1.197	1.184	1.148	1.131	2.306	2.953
2	Regional Unit Hydrograph Approach (*)									
	a) Alongwith storm rainfall charts in CWC Report	5.102	7.008	6.519	1.404	1.143	1.107	1.652	3.053	5.617
	b) Alongwith observed storm rainfall at Chamoli	3.948	4.836	4.456	1.050	0.935	0.853	1.500		
3	Regional Flood Frequency Approach	2.063	3.678	2.217	1.512	1.685	1.779	1.167	1.929	2.1966

* See figure 3.1 of chapter 3.

4.4 DISCUSSION

The details of the flood peak obtained are as given in Table 4.2 above. It can be observed from the table that the regional frequency approach and the Modified Dicken's formula gives lower values compared to the hydrometeorological approach. This may be due to the fact that in these two approaches, the basic formulae were derived from large catchments in the plain areas and hence, the effect of extremely high river slopes in the upper reaches is not fully reflected. In the method of CWC report, the slope of main-stream is an important input in derivation of UG. In the method based on CWC report, the variation in storm intensity has been accounted for; whereas, in both Modified Dicken's formula and in the regional frequency approach, this is not taken into account. Beacause of the mountainous nature of the catchment, there is considerable variation in the spatial distribution of design storm rainfall as shown by the IMD studies (Fig. 3.8 to 3.10 of chapter 3). The regional frequency approach also does not take catchment slope into account. As already discussed in para 4.1.4, these curves have a wide confidence band. Hence, all these three methods can be expected to give only a preliminary estimate of design flood, which needs to be confirmed through detailed study.

CHAPTER 5

CATCHMENT LINEARITY AND VOLUME-PEAK DISCHARGE RELATION

5.1 INTRODUCTION

Hydrologic linearity of drainage basin is defined as the condition that exists on a basin when runoff volumes are directly proportional to rainfall volumes. Serious error in hydrologic design can occur by over estimating or under estimating design discharge when a drainage basin is assumed to be linear while in fact it is nonlinear. The wide spread and long lasting usage of the unit hydrograph (UH) model (Sherman, 1932), which is based on the assumption of hydrologic linearity, makes more intensive the need for developing criteria for checking the applicability of the method and, thus, the linearity and non-linearity in the rainfall-runoff process. One of the most important attempts on this subject has been made in the USA, where a family of three peak discharge distributions has been developed and studied in detail (Rogers, 1980, 1982; Rogers & Zia, 1982). It was found that the slopes of these distributions are related to the drainage basin runoff characteristics. The slopes of these distributions were proposed as a criterion indicating the degree of drainage basin hydrologic non-linearity. Predicted peak discharges by the UH method for non-linear basins were found to be abnormally overestimated.

Mimikou (1983) tested the applicability of the peak discharge distributions in eight drainage basins in Greece and found that only the original peak discharge distribution (OPDD) is necessary for checking basin hydrologic linearity and accurately predicting peak discharges. Singh & Aminian (1986) proposed linear two parameters relation in log space between direct runoff volume per unit area and peak discharge of direct runoff per unit area. Since a basin is heavily damped system (Dooge, 1973), prediction of similar output by different linear models cannot be considered as a satisfactory criterion for validation of a proposed linear model without ascertaining hydrologic linearity of a basin. This chapter deals with development and application of various relationships between peak discharge and runoff volume for some drainage basins in India to identify degree of non-linearity of the basins.

The study area comprises of some mountainous catchments in Western Himalayan region.

5.2 GENERAL HYDROLOGIC SYSTEM MODEL

A hydrologic system is defined as a structure or volume in space, surrounded by a boundary, that accepts water and other inputs, operates on them internally, and produces them as outputs. The structure or volume in space is the totality of the flow paths through which the water may pass through from the point it enters the system to the point it leaves. The boundary is the continuous surface defined in three dimensions enclosing the volume or structure. In a systems approach, we are concerned with the system operation, not the nature of the system itself, (its components, their connection with one another and so on) or the physical laws governing its operation. The physical laws and the nature of the system are combined into a single concept of system operation. (Singh, V.P., 1988). A system approach is justified in flood hydrology, since, system behavior is primarily affected by a few major factors and others have insignificant effect.

A watershed, also known as basin or catchment, is an area of land draining into a stream at a given location. The watershed divide is a line dividing land whose drainage flows towards the given stream from land whose drainage flows away from the stream. The system boundary is drawn around the watershed by projecting the watershed divide vertically upwards and downwards to horizontal planes at the top and the bottom. For rainfall-runoff modeling, rainfall is input, distributed in space and time over the upper plane; stream flow is the output, concentrated in space at the catchment outlet. Evaporation, subsurface flow etc. can also be considered as outputs, depending upon the purpose of the study.

From the consideration of conservation of mass,

$\frac{dS(t)}{dt} = I(t) - Q(t)$ where, s is the water storage in the system, t is time, I is inflow and Q is outflow from the system. A general model explained by Chow et al (1988) is as follows:

$$S(t) = f\left(I, \frac{dI}{dt}, \frac{d^2I}{dt^2}, \ldots \frac{d^nI}{dt^n}, Q, \frac{dQ}{dt}, \frac{d^2Q}{dt^2}, \ldots\ldots \frac{d^nQ}{dt^n} \right)$$

The function, f, is determined by the nature of the hydrologic system being examined. For example, a linear reservoir will be represented as:

$$S = kQ$$

Nash (1960) had modeled catchment rainfall-runoff process through a system of linear reservoirs in series.

Let storage S be approximated by:

$$S(t) = a_1Q + a_2\frac{dQ}{dt} + a_3\frac{d^2Q}{dt^2} + \ldots\ldots\ldots + a_n\frac{d^{n-1}Q}{dt^{n-1}} + b_1I + b_2\frac{dI}{dt} + b_3\frac{d^2I}{dt^2} + \ldots\ldots\ldots + b_n\frac{d^{n-1}I}{dt^{n-1}}$$

in which a_1, a_2, ... are coefficients and derivates of higher order are neglected. If coefficients are not function of time, the system is time-invariant, i.e., the way the system processes the input into output does not change with time.

Differentiating the above equation with respect to time, and substituting (I-Q) for dS/dt and re-arranging yield:

$$a_n\frac{d^{n-1}Q}{dt^{n-1}} + \ldots\ldots + a_3\frac{d^2Q}{dt^2} + a_3\frac{d^2Q}{dt^2} + a_1\frac{dQ}{dt} + Q = I - b_1\frac{dI}{dt} - b_2\frac{dI}{dt} - b_3\frac{d^2I}{dt^2} - \ldots\ldots - b_n\frac{d^{n-1}I}{dt^{n-1}}$$

or

$$N(D)Q = M(D)I$$

where, D=d/dtand N(D)and M(D) are the differential operators as indicated above.

$$\text{e.g.}: N(D) = a_n\frac{d^{n-1}}{dt^{n-1}} + \ldots\ldots\ldots + a_3\frac{d^2}{dt^2} + a_3\frac{d^2}{dt^2} + a_1\frac{d}{dt} + 1$$

Thus we can write:

$$Q(t) = \frac{Q(D)}{N(D)} * I(t) = \Omega I(t)$$

The operator Ω is called the transfer function of the system; it defines the response of the output to a given input sequence. This equation was presented by Chow and Kulandaiswamy (1971, as mentioned in Chow et al.,1988) as a general hydrologic system model. It describes a lumped system because it contains derivatives with respect to time alone and not spatial dimensions. Chow and Kulandaiswamy showed that many of the previously proposed models of lumped hydrologic systems were special cases of this general model. For example, for a linear reservoir, the storage function has $a_1 = k$.

5.2.1 Linear System

A system, represented by the equation $Y = \Omega(X)$ is said to be linear if it satisfies:

$$\Omega(X_1 + X_2) = \Omega(X_1) + \Omega(X_2) \text{ and}$$
$$\Omega(cX) = c * \Omega(X).$$

In the unit hydrograph approach (Sherman, 1932), X is the rainfall excess and Y is the direct runoff at the catchment outlet and linearity is a basic assumption in this method. However, it is to be noted that Sherman's Unit Hydrograph hypothesis came with many conditions attached to it, such as i) the rainfall excess shall be of uniform intensity in space and time, ii) the duration of the rainfall excess shall be same. Two definitions of non-linearity of catchment response appear in literature (Sivapalan, et. al, 2002). The first definition is with respect to the rainfall-runoff response of a catchment and refers to a non-linear dependence of the storm response on the magnitude of the rainfall inputs. It is in this sense, the word has been used in this dissertation. Rogers and Zia (1982b) have outlined two major limitations to the application of the UG procedure to large drainage basins. These are: i) the largest drainage area for applicability of UG is that which can be covered by a storm producing uniform spatially distributed rainfall excess ii) UG procedure is not applicable to snow-melt hydrographs. There are other less important limitations as well. Based on study of isohyetal maps of major storm events in US, Rogers and Zia (1982b) have found the upper limit of area size for spatial uniformity of rainfall to be 1160 sq. km. beyond which the UG method is not applicable. In this paper, it was also shown that the method is applicable equally well to catchments having significant snow-melt runoff, so long as the snow melted in 3 to 5 days.

5.3 VOLUME-PEAK DISCHARGE RELATION

A relation between volume and peak of direct runoff is of fundamental importance in a wide variety of hydrologic analyses, especially where hydrologic data are scarce. It has immediate application in hydraulic design and water resources planning. (Singh and Aminian, 1986). Rogers (1980) was perhaps the first to study the relation between peak discharge and runoff volume in a systematic way. In this paper, data from 42 drainage basins lying in US and ranging in area from 4.5 sq.km. to 702 sq.km., were used to derive double log relation between volume (V) and peak (Q_p) of the hydrograph. This relation can be expressed as:

$$\log(Q_p) = b + m*\log(V)$$

in which Q_p is peak discharge per unit area (cm/h) and V is the runoff volume. Rogers called this Original Peak Discharge Distribution (OPDD). He showed that the procedure is equally applicable to both natural hydrographs as well as the hydrograph of direct runoff after separating the baseflow component, in line with the UG procedure of Sherman. He also found that the procedure becomes more meaningful if the peak is standardized by dividing it by the volume of runoff or the square of the volume of runoff, resulting in what he termed as First Order Standardised Peak Discharge Distribution (FSPDD) and Second Order Standardised Peak Discharge Distribution (SSPDD). The mathematical form of these distributions are as below:

OPDD: $\log_{10}(Q_p/V) = b + (m-1)*\log_{10}(V)$

SSPDD: $\log_{10}(Q_p/V^2) = b + (m-2)*\log_{10}(V)$

The regression coefficients in the case of FSPDD and SSPDD will be (m-1) and (m-2), where 'm' is the regression coefficient of OPDD. The intercept will remain unchanged. This is because the term $\log(Q_p/V)$ and $\log(Q_p/V^2)$ can be expanded as $\log(Q_p)-\log(V)$ and $\log(Q_p)-2\log(V)$ respectively and hence, the above two equations can be derived from the equation for OPDD stated earlier. The nature and significance of these equations have been discussed in detail by Rogers (1980, 1982), where it is shown that for hydrologically linear drainage basins, the absolute value of slopes m is 1.00 and non-linear basins are identified by smaller values for the slopes of these equations. There will be an apparent increase in coefficient of determination (r^2) as we go from OPDD to FSPDD and then to SSPDD. This can be explained by observing that there is correlation between V and 1/V or $1/V^2$ and hence, the increased correlation. This has also been corroborated by Mimikovu (1983), Singh et al(1986) etc.

5.3.1 Value of Standardisation: In unit hydrograph approach, duration of precipitation excess is a major consideration and uniformity in time and space of the precipitation excess during this duration is also a very important assumption in the theory. However, this is not the case with the peak discharge distribution suggested by Rogers. Hence, Rogers (1980) suggested the need to standardize the peak by dividing by the volume of hydrograph. Rogers (1980) and Rogers and Zia(1982a, b), had used units for volume and peak as cm and m^3/s. Later, Singh and Aminian (1986) have recommended that these two quantities be divided by catchment area to reduce the effect of catchment area. Hence, the units used will be cm and cm/h. In this dissertation, this approach has been used.

5.3.2 Hydrograph Separation Procedure: There are various methods used for separation of baseflow in the unit hydrograph approach, depending upon whether the definition of baseflow includes delayed runoff or not. Volume of direct runoff will accordingly be different. Since, runoff volume is the dependent variable in the present technique and hence, it is very important to have a uniform and relatively accurate procedure for hydrograph separation. Rogers and Zia (1982b) have recommended the following procedure:

$$Q_t = Q_0 + Q_p^{0.6}$$

where,

Q_t =discharge corresponding to the termination of the runoff hydrograph

Q_0 =base flow prior to hydrograph rise marking the beginning of the hydrograph

Since, this empirical relation was in FPS units (cubic feet per second), it needs to be converted before using in MKS units (i.e., cubic meter per second).The relation in MKS units will be:

$$Q_t = Q_0 + 0.2395 * Q_p^{0.6}$$

This relation has been used in this dissertation for separating the baseflow component.

5.3.4 Theoretical Justification Based On Triangular UG Approach : Rogers (1980), quoting personal correspondence from V.T.Chow, has given the following theoretical explanation for second order standardized peak discharge relation, using the triangular UG approach as the basis.

In triangular UG approach, $T_b = 2.67\ T_p$ and $Q_p = 2V/(2.67\ T_p)$. Hence,

$Q_p/V^2 = 1/(1.34*T_p*V)$

Taking logarithms,

$\log(Q_p/V^2) = -\log(1.34*T_p) - \log(V)$

This is in the format of SSPDD discussed earlier.

If we expand the LHS of the above equation, we get,

$\log(Q_p) - 2*\log(V) = -\log(1.34*T_p) - \log(V)$

or $\log(Q_p) = -\log(1.34*T_p) + \log(V)$

This is analogous to the OPDD.

However, it is to be noted that in OPDD and SSPDD, the volume is in cm and Qp in m3/s. Hence, a conversion factor will be required if direct comparison between the two sets of equations are being attempted. For example, if we denote v as the volume per unit area in cm, A is the catchment area in km^2, then, we can get,

$\mathrm{Log}(Q_p) = -\log(1.34*0.36*T_p/A) + \log(v)$

If T_p is more or less constant, this implies that OPDD and SSPD relations are linear in log space. Hence, the linearity of the unit hydrograph and linearity of the volume-peak discharge relation have some similarities, though they are distinct in detail.

5.4 STUDY FOR SOME CATCHMENTS IN WESTERN HIMALAYAN REGION

Seven catchments, (six in western Himalayan region and one in central India) were selected to study the linearity of peak discharge volume relationship. The catchments selected are Gola basin (near Nainital), Mandakini at Rudraprayag, Alaknanda at Rduraprayag, Pindar at Karnaprayag, Alaknanda at Joshimath, Bhagirathi at Tehri (all in

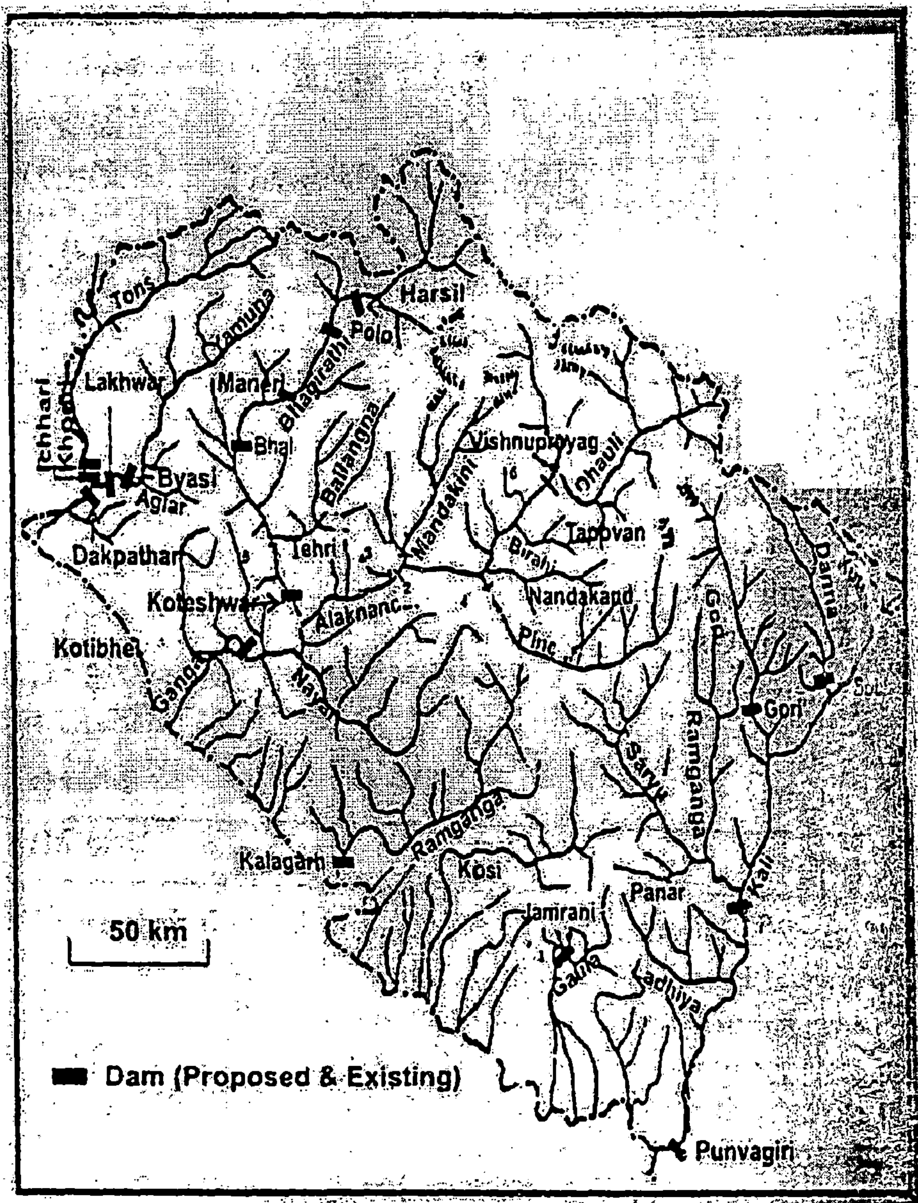


Fig. 5.1: LOCATION OF G&D SITES

western Himalayan region) and catchment of railway bridge No. 87 (lower Godavari basin in central India). The location of these sites are given at Fig. 5.1. The Table 5.1 below shows the physiographic characteristics and number of flood events used at the above sites.

Table 5.1: Drainage Basin Characteristics

Sl. No.	River	Location	Catch-ment Area (km^2)	Area of snow cover (%)	Length (L) (m)	Number of hydro-graphs	Source Of Data
1	Gola	Dam site	450	nil	23.5	10	WRDTC, 2002
2	Alaknanda	Rudraprayag	9045	30.30	138	5	UP Irr. Dept.
3	Mandakini	Rudraprayag	1644	20.92	63	11	CWC (unpublished)
4	Pinder	Karnaprayag	2294	15.80	103	10	
5	Bhagirathi	Tehri	7208	15.00	165	10	
6	Alaknanda	Joshimath	4508	40.00	63	8	
7	Lower Godavari	Br.No. 807	823.6	nil	61.1	13	Mishra, 1998 Kumar et al, 2001

Note: The site listed at sl.no.7 is lying outside the study area. This has been added only because none of the catchments selected for the study area were found to be hydrologically linear and this site which is linear is added as a case study for comparison with the remaining catchments.

The flood events used at these sites and their analysis are given in Annexure 5.1. The summary details of the flood events at the five sites are given in the table 5.2.

When the OPDD and SSPDD were developed using total runoff as the basis, it was found that both the regression coefficient (m or m'') and the coefficient of determination were very low. One of the main reason for the very low value of regression coefficient (slope) 'm' is due to the large contribution from snow-melt, which has relatively higher travel time and hence, low peaking characteristics. During the summer months, snow-melt runoff becomes significant and all the storm events considered in the study belong to the months of June, July

Table 5.2 Summary Details Of The Flood Events Used In The Study

Gola		Mandakini (Rudraprayag)		Alaknanda (Rudraprayag)		Pinder (Karnaprayag)		Bhagirathi (Tehri)		Alaknanda (Joshimath)		Br. No. 87 (Lower Godavari)	
V (cm)	q_p (cmh^{-1})	V (cm)	q_p (cmh^{-1})	V (cm)	q_p (cmh^{-1})	V (cm)	q_p (cmh^{-1})	V (cm)	q_p (cmh^{-1})	V (cm)	q_p (cmh^{-1})	V (cm)	q_p (cmh^{-1})
8.26	0.3600	0.14210	0.02838	0.7455	0.0417	0.08796	0.00782	0.1599	0.0849	0.2068	0.0188	1.0500	0.15735
7.60	0.3408	0.32900	0.05109	0.3859	0.0235	0.33898	0.02218	0.1828	0.0670	0.2335	0.0197	1.1600	0.23769
6.32	0.2288	0.58600	0.05566	0.3640	0.0271	0.27247	0.02691	0.1355	0.0673	0.2148	0.0181	1.2600	0.20906
2.00	0.1064	0.52900	0.03830	0.2625	0.0238	0.18889	0.02286	0.0685	0.0393	0.1182	0.0128	0.6100	0.14184
1.80	0.0936	1.06130	0.08225	0.7036	0.0360	0.45462	0.02739	0.1697	0.0807	0.2091	0.0170	0.8800	0.17576
4.90	0.4256	1.03400	0.08169			0.26910	0.02045	0.0245	0.0279	0.0962	0.0117	0.8500	0.10796
1.71	0.1960	0.97500	0.08446			0.72364	0.04930	0.0734	0.0503	0.2351	0.0194	4.2900	0.60800
5.13	0.4224	0.27550	0.03462			0.23103	0.01993	0.3852	0.1050	0.2472	0.0198	0.5900	0.14105
17.62	0.5592	0.87420	0.08236			0.26197	0.02147	0.0653	0.0512	0.2068	0.0188	1.5200	0.19888
2.60	0.2240	3.33800	0.13431			0.03145	0.00446	0.2285	0.1077			0.6400	0.09424
		3.04000	0.12738									4.9800	0.62592

Note: V -Direct Runoff Volume as depth over CA (unit: cm).

q_p --Peak discharge per unit area of catchment (unit: $cm.hr^{-1}$) $= \dfrac{Q_p * 0.36}{A}$,

(where, Q_p is in m^3/s and A is catchment area in km^2).

and August. Thus, it was decided to do the studies using direct runoff as the basis only. The table 5.3 below shows the results of the study using the direct runoff as the basis. The plots of the data and the best-fit line are shown in the Fig. 5.2 to 5.7 for the six sites. It can be seen that the slope of the fitted line for OPDD is in the range 0.52 to 0.75 only. The values obtained for these basins are also compared with the values obtained for basins in the Peninsular India in an earlier study by Chaube et al (2004), in which they had identified the catchment of Bridge No. 807 of Lower Godavari Basin to be linear.

Table 5.3: Relation Between Volume (cm) and Peak Discharge (m^3 /s) in Log-space

Sl. No.	River	Site	Intercept b	OPDD		SSPDD	
				Slope M	R^2	Slope m-2	R^2
1	Gola	Dam site	2.091	0.644	0.696	-1.356	0.910
2	Alaknanda	Rudraprayag	2.9233	0.5310	0.8671	-1.4690	0.980
3	Mandakini	Rudraprayag	2.4364	0.5162	0.9163	-1.4838	0.989
4	Pinder	Karnaprayag	2.4997	0.7519	0.9431	-1.2481	.9786
5	Bhagirathi	Tehri	2.7028	0.524	0.914	-1.476	0.988
6	Alaknanda	Joshimath	2.5228	0.5747	0.9753	-1.4253	0.996
7	Lower Godavari	Br.No. 807	2.588	0.800	0.894	-1.200	0.950
8	Study by Chaube et al (2004)	Peninsular India	1.82 to 2.73	0.64 to 0.81	0.7 to 0.97	-1.2 to 1.36	0.91 to 0.99

Singh et al (2001) had reported a study in which the peak discharge was expressed in cm/h units by dividing the peak discharge by catchment area and a suitable conversion factor depending on the units used. Such a double adjustment of both volume and peak, in his opinion, makes the relations more suitable for regionalisation. Accordingly, the OPDD and SSPDD have been worked out for all the above catchments and the results are given in the Table 5.4 below.

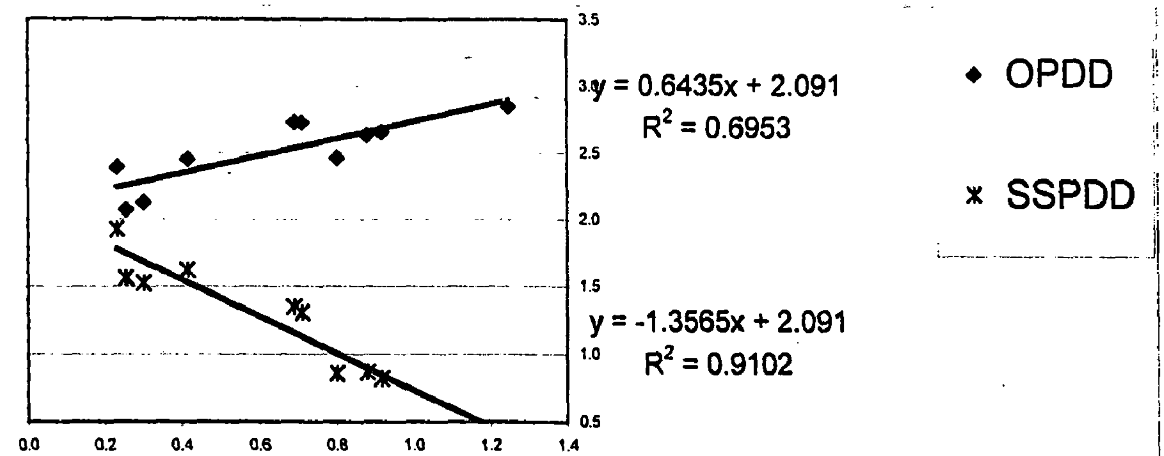


Fig. 5.2: Volume Peak Discharge Relation For Gola At Dam Site

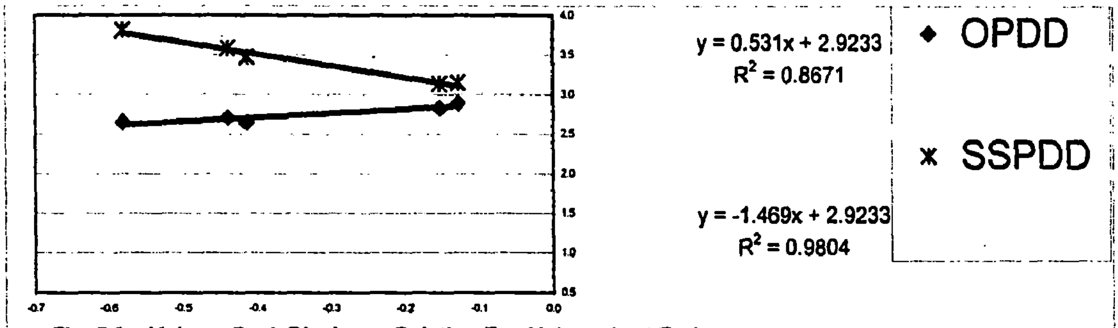


Fig. 5.3: Volume Peak Discharge Relation For Alaknanda at Rudraprayag

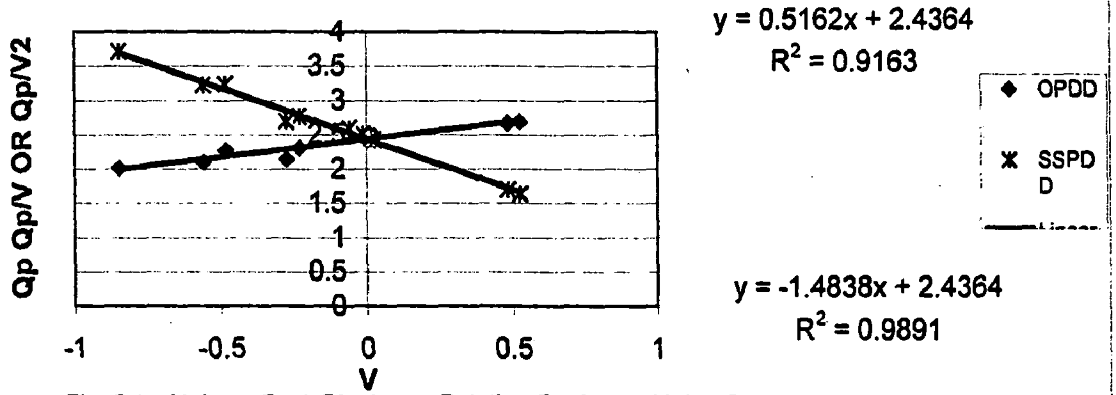


Fig. 5.4: Volume Peak Discharge Relation For Mandakini at Rudraprayag

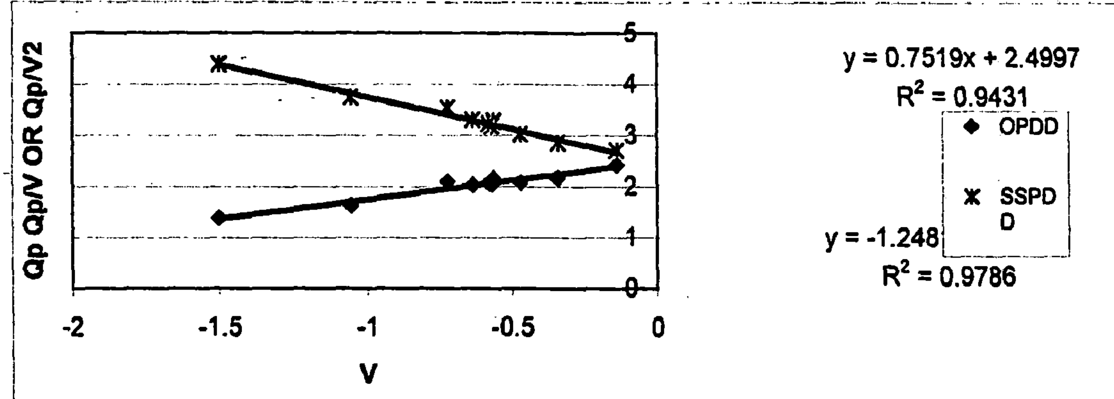


Fig. 5.5: Volume Peak Discharge Relation For Pinder at Karnaprayag

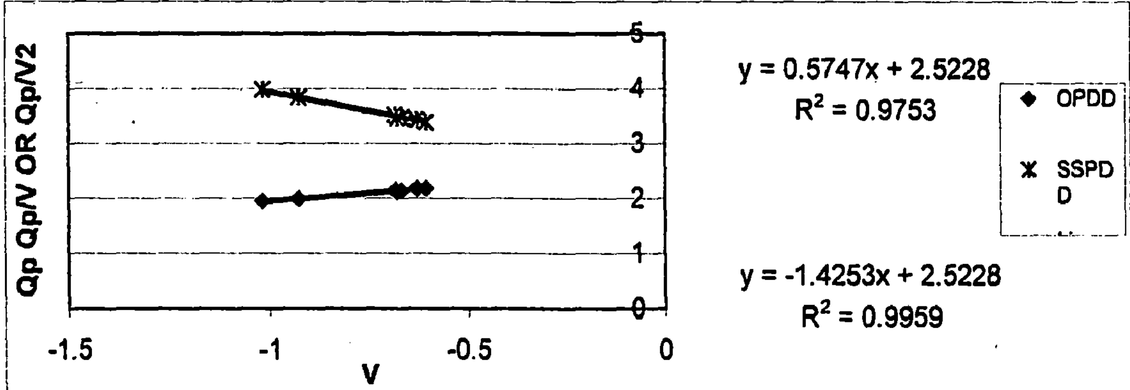


Fig. 5.6: Volume Peak Discharge Relation For Alaknanda at Joshimath

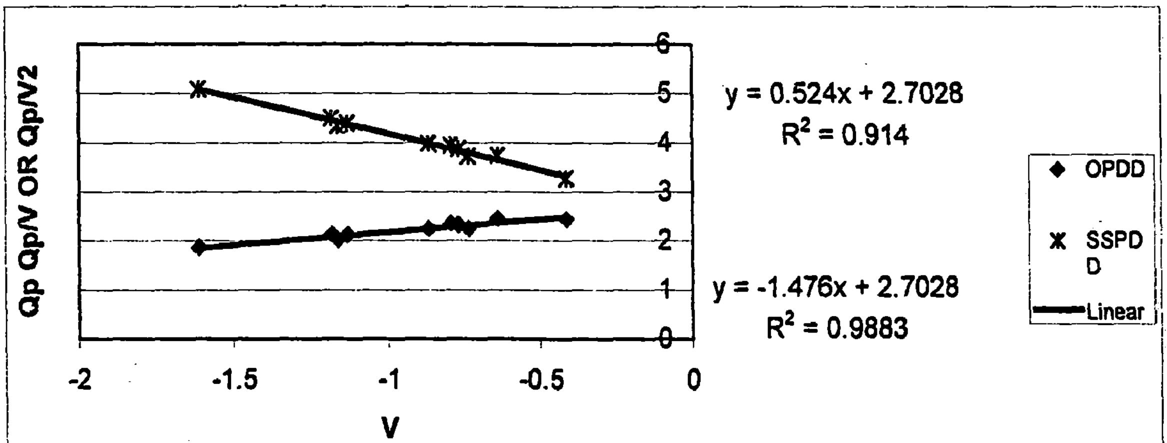


Fig. 5.7: Volume Peak Discharge Relation For Bhagirathi at Tehri

Table 5.4: Relation Between Volume (cm) and Peak Discharge Per Unit Area (cm/h) in Log Space

Sl. No.	River	Site	Intercept b	OPDD		SSPDD	
				Slope M	R^2	Slope m-2	R^2
1	Gola	Dam site	-1.0062	0.644	0.696	-1.356	0.9100
2	Mandakini	Rudraprayag	-1.1213	0.5162	0.9163	-1.4838	0.9891
3	Alaknanda	Rudraprayag	-1.3466	0.5310	0.8671	-1.4690	0.980
4	Pinder	Karnaprayag	-1.2297	0.7519	0.9431	-1.2481	0.9786
5	Bhagirathi	Tehri	-0.7121	0.524	0.914	-1.476	0.988
6	Alaknanda	Joshimath	-1.3531	0.5747	0.9753	-1.4253	0.996
7	Lower Godavari	Br.No. 807	-0.7665	0.808	0.900	-1.192	0.952
8	Study by Chaube et al (2004)	Peninsular India	-0.67 to –1.0	0.64 to 0.81		-1.19 to -1.36	

5.5 RELATIONSHIP BETWEEN INTERCEPT WITH CATCHMENT CHARACTERISTICS

If the slope and intercept of the OPDD /SSPDD can be related with the catchment characteristics, it will help in i) predicting peak discharge per unit of direct runoff and ii) developing peak discharge distribution for ungauged catchments. Intercept 'b' is equl to $\log(Q_p)$ when runoff volume V is equal to 1 cm.

Based on a study in Greece, Mimikoy (1983) found that the variation in b is significantly explained by the logarithm of any of the two basin morphological indices, AS/L and A/L. Singh and Aminian (1986) studied 134 drainage basins and found that basin area alone explains the variance of 'b' by more than 86% ($r^2 = 0.8610$). Inclusion of bed slope 'S' and stream length 'L' increased r^2 marginally to 0.869. They, therefore, concluded that A alone can explain 'b' satisfactorily.

In the present study, regional intercept equation has been developed by using catchment area (excluding the area under permanent snow-cover) as the independent variable. The plot of the best fit line and the equation developed are shown in Fig. 5.8. As can be seen from the figure, the relation explains nearly 95% of the variation in the intercept. When the relation was attempted between intercept 'b' and basin morphological index A/L, the coefficient of determination was weak (0.567, see Fig. 5.9) and hence, the relation is not recommended.

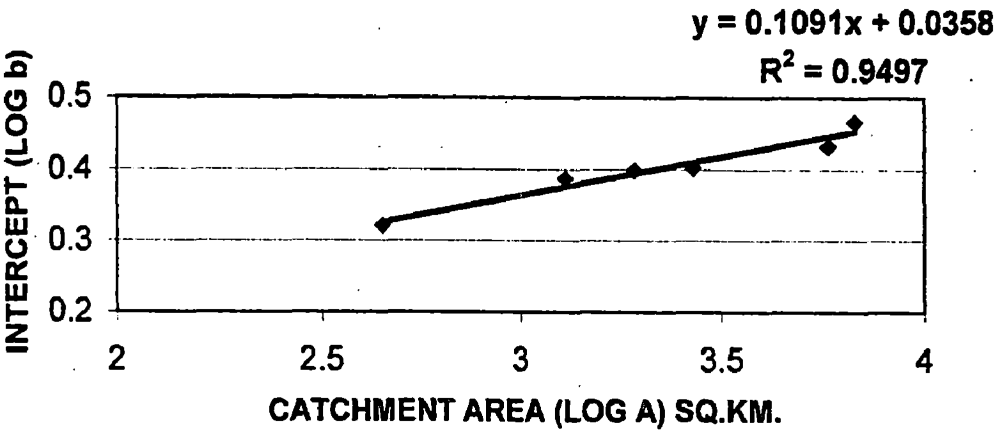


Fig. 5.8. Regression Between Intercept And Catchment Area (excluding snow-covered area)

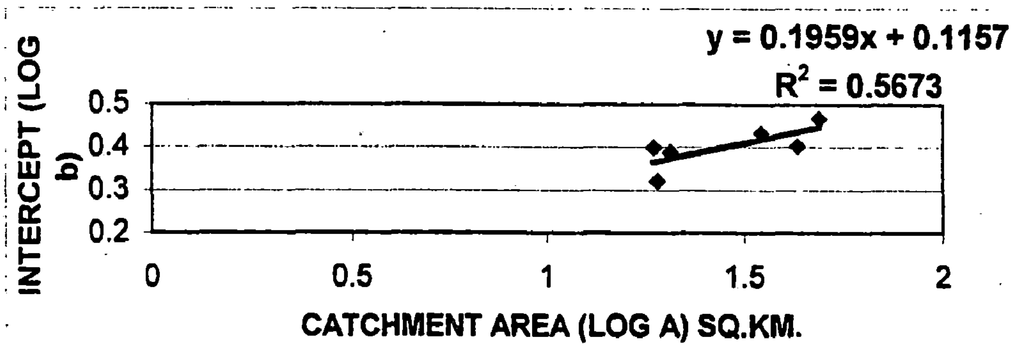


Fig. 5.9 Regression Between Intercept And A/L Ratio

Similar relation was attempted between the slope of OPDD and catchment characteristics; however, the correlation coefficient was too small and statistically insignificant. Hence, further studies will be required to establish a regional relation for finding the slope of the OPDD.

5.6 FLOOD ESTIMATION USING SSPDD AND COMPARISON WITH UG METHOD

Unit hydrograph method is, by far, the most popular method for conversion of a given rainfall hyetograph input into runoff hydrograph. However, the UG method is applicable only to linear basins. If the non-linearity of the basin is pronounced, then, alternatively, the SSPDD can be used to determine the flood peak from the given storm rainfall excess hyetograph and its distribution can be done using the triangular UG approach (Mockus, 1957 as quoted by Chow, 1988). Typically, this is a problem encountered in flood estimation for design purposes, where in the design storm depth can be estimated using standard procedures and using SSPDD, the corresponding peak discharge can be estimated. In this study, the peaks have been obtained using both the approaches and are compared in the Table 5.5. It can be seen from the table that the UG method overestimates the peak by as much as 2 to 3 times as compared to the SSPDD procedure, in the case of the basins with high level of non-linearity. For the sake of comparison, a drainage basin from Peninsular India, viz., Bridge site 287 on lower Godavari, has been used. This catchment has been studied in detail by Chaube et al (2004) and Suarbawa (2002) and they have reported that this site is linear and hence, UG method is applicable.

Table 5.5 Comparison of Flood Peaks by UG Approach and SSPDD Approach

Catchment	V (cm)	Peak by UG method (m^3/s)		Peak by using SSPDD	Remark
		Critical Time distribution and sequence	No critical sequencing		
Tehri	5.9	7260	3803	1279	UG method gives almost six times as much as compared to SSPDD.
"do"	7.8	9280	5153	1480	
"do"	8.5	9900	5660	1550	
Br.No. 807 (Lower Godavari)	12.22	2944		2909	Very close.

5.7 CONCLUSIONS

1. The log volume-log peak discharge relation can be used to explain non-linearity of a catchment. Use of direct runoff, in place of total runoff, has been found to give better relationship with peak discharge.
2. SSPDD has higher coefficient of determination as compared to OPDD. There is a consistent and systematic improvement in case of non-linear basins. Coefficient of determination is significant irrespective of non-linearity of catchment.
3. The degree of non-linearity can be gauged from the deviation of slope of the OPDD or SSPDD from 1 or –1 respectively. Original Peak Discharge Distribution is sufficient for identification of basin linearity and for predicting the peak discharges. Correlation between log Qp and log Tp needs to be further investigated.
4. Application of linear UG theory for estimation of peak flood in non-linear basins, as demonstrated, may result in serious error in hydrologic design.
5. Peak discharge distribution can be utilized for ungauged catchments in a variety of hydrologic studies such as computation of peak discharge from rainfall excess volume, etc.
6. Further studies are required to correlate the slope and intercept of OPDD with catchment physiographic parameters.

5.8 SCOPE FOR FURTHER STUDY

In this study, many flood events correspond to low order of magnitude of direct runoff volume. Further study is required using higher ranges of volume of direct runoff.

The distribution of rainfall in time and space has significant effect on the flood peak. Assuming uniform rainfall coverage in space, when it is actually not so, particularly for small catchments, can produce flood peaks of significantly different magnitude. But, the OPDD /SSPDD will estimate the same value of peak discharge as the volume of runoff is same.

Hence, the OPDD procedure needs further investigation to include the duration of rainfall /rainfall excess as an independent variable.

In conventional design flood synthesis techniques, using UG or other techniques, the hyetograph of rainfall is the one corresponding to the most severe hydrologic / meteorologic pattern considered as characteristic /possible for the catchment and hence, critical placement of storm isohyetal pattern, critical depth duration pattern, critical sequencing of rainfall ordinates, etc are done. Whereas, the OPDD relation represents an average of the pattern of storm rainfall (including its location, depth-duration characteristics, etc.) and hence, development of OPDD/SSPDD should be based on a large number of flood events covering all possible ranges of storm patterns.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CRITERIA FOR SELECTION OF DESIGN FLOOD

The criteria for selection of design flood for dams being followed in India are based on the provisions contained in the BIS Code IS:11223-1985. The provisions in the Code are broadly those which were followed in USA and other western countries during the eighties and were based on the height and storage volume involved. Though the Code has an enabling provision to take into account the potential downstream damages, no guidelines are given for consideration of magnitude of damages in the selection of design flood. Thus, there is a need to incorporate quantitative risk analysis in the criteria, as is now being increasingly felt in India and other countries. This is all the more important for India, because of the ever-increasing flood plain occupancy, caused by population pressure and developmental activities.

The BIS code for barrages in India allows the use of a lower design flood than for equivalent dams. There appears to be no logic, especially since, the demarcation between the two structures is not rigid in the mountainous regions.

With developments in technology, weather forecast, flood forecasting, dam breach analysis etc., flood damage evaluation has become more scientific and reliable. Traditionally, a conservative approach has been followed in hydrologic design of river valley projects. The above mentioned developments should also be considered in evolving design flood critieria. However, it will depend on the confidence of the decision makers in available technical expertise in the country.

Within-year and over-the-year storage schemes should be distinguished in fixing design flood criteria. At present no such distinction is made.

6.2 HYDROLOGIC CHARACTERISTICS OF STUDY AREA

6.2.1 General Features:

- Most rivers in western Himalayas have significant snow-melt runoff, making them perennial rivers
- Streams have very steep descend in levels in the initial reaches, followed by steep bed gradients of the order of 15-30 m/km (0.015 to 0.030), before entering the Siwaliks, foothills of Himalayan mountains.
- Very narrow gorges in the initial reaches, with river terraces at certain locations. Narrow channel, high channel slope and the absence of wide flood plains, result in short-duration high magnitude flood peaks.

6.2.2 Snowmelt and Baseflow characteristics: Typically, the snowline comes down to the altitude of 2800m during winter and goes upto 4800m towards end of summer season, though the level of permanent snow cover may change slightly from year to year. Average baseflow from area not having snow-cover has been found to be 0.05 cumec /km^2. The peak snow-melt runoff computed from daily discharge data of around 25 years, has been found to be well correlated with area under snow-cover in log-space, with a coefficient of determination of 0.9036.

6.2.3 CWC Report for zone 7: The relations between catchment characteristics and UG parameters presented in the CWC report (CWC, 1994), were derived from the data of nearly 45 flood events from 5 sites lying in the fringe of the basin, towards the plain areas and hence, are not fully representing the hydrological and meteorological characteristics of the mountainous catchments in the basin. Further, data sets for five sites only is too small a sample for regression analysis, since, the variance of the computed coefficients will be too large and hence, the 95% confidence limits of the estimated values of the dependent variable, i.e., UG parameters, will be too wide apart to be of any use in flood computations. These values can be used as a preliminary value to start with, which need to be modified /firmed up based on data of G&D observations and short-interval rainfall data.

6.2.4 Comparison of Catchments in CWC Report and in the Present Study: The project catchments studied in this dissertation have steep slopes of the order of 10-30 m/km. However, the slopes of the 5 catchments included in the CWC report (CWC, 1994) ranged between 10 and 84 m/km and hence, are steeper. Since, slope is an independent parameter in the CWC approach, its effect on UG is incorporated in the method.

The catchments in CWC fall in the terai (plain areas) region and here, the land use pattern is predominantly agriculture and forest area is less. Whereas, the former has tendency to reduce the peak, the latter has a tendency to increase the peak, especially for small catchments. The catchments of the project sites are predominantly forest areas, with little agricultural activity. However, the soil layer in the high altitudes being thin, the net effect is expected to be a higher runoff coefficient.

6.3 VOLUME-PEAK DISCHARGE RELATION

6.3.1 Conclusions

1. The log volume-log peak discharge relation can be used to explain non-linearity of a catchment.
2. SSPDD has higher coefficient of determination as compared to OPDD. Coefficient of determination is significant irrespective of non-linearity of catchment.
3. The degree of non-linearity can be gauged from the deviation of slope of the OPDD or SSPDD from 1 or -1 respectively.
4. Application of linear UG theory for estimation of peak flood in non-linear basins, as demonstrated, may result in serious error in hydrologic design.
5. Peak discharge distribution can be utilized for ungauged catchments in a variety of hydrologic studies.

6. The relation developed between intercept of OPDD and catchment area and slope can be used for ungauged catchments to obtain the intercept of OPDD. However, no statistically significant relation could be established for slope of OPDD.

6.3.2 Scope for Further Study

1 Correlation between log Qp and log Tp needs to be further investigated.

2 Further studies are required to correlate the slope and intercept of OPDD with catchment physiographic parameters.

3 In this study, many flood events correspond to low order of magnitude of direct runoff volume. Further study is required using higher ranges of volume of direct runoff.

4 OPDD procedure needs further investigation to include the duration of rainfall /rainfall excess as an independent variable.

5 The OPDD relation represents an average of the pattern of storm rainfall (including its location, depth-duration characteristics, etc.) and hence, development of OPDD /SSPDD should be based on a large number of flood events covering all possible ranges of storm patterns.

REFERENCES

1 Aggarwal C.K.et al, 1991, "*Computer Aided Frequency Analysis for Himalayan Region Rivers National Seminar on Use of Computers in Hydrology & Water Resources*", Dec. 1991 New Delhi

2 Aggarwal P.P.et al, 1982, "*Flood Computation for Kishau Dam Project by Regional Frequency Analysis*", Intl. Symposium on Hydrological Aspects of Mountainous Watershed, Nov. 4-6, 1982, Roorkee

3 BIS, 1985, "*IS: 11223, Guidelines for fixing spillway capacity of dams*", Bureau of Indian Standards, New Delhi

4 BIS, 1989, "*IS: 6966 (Part I)-1989, Hydraulic Design of Barrages and Weirs-Guidelines, Part I Alluvial reaches*", Bureau of Indian Standards, New Delhi

5 BIS, 1993 "*IS: 7784 (Part I)-1993, Code of Practice for Design of Cross Drainage Works, Part I General Features*", Bureau of Indian Standards, New Delhi

6 Chen S.J. & Singh V.P., 1993, "*Improved SCS Model for Computing Runoff Volume*", Journal of Hydrology, Vol. XVI, Jan-June 1993, pp. 14-18 IAH, Roorkee

7 Chow, V.T. (Ed.), 1964, "*Handbook of Applied Hydrology*", Mc Graw Hill Inc., NY

8 Chow, V.T., Maidment, D.R., and Mays, L.W, 1971, "*Applied Hydrology*",

9 Cruise,J.F., Singh, V.P., and Molfino,M.E., 1992, *"Some Aspect of The Log-Linear Relationship Between Runoff Peaks and Volumes* ", Hydrology Journal, XV,3 & 4, IAH, Roorkee.

10 CW&PC, 1972, "*Estimation of Design Flood -Recommended Procedure*", Central Water Commission, New Delhi.

11 CWC, 1993, "*Recommendations -National Workshop on Rationalization of Design Storm Parameters for Design Flood Estimation*", Central Water Commission, New Delhi

12 CWC, 1994, "*Flood estimation report for Western Himalayas – Zone 7*", Central Water Commission, New Delhi

13 CWC, 1995, *"Flood Estimation Report for Lower Godavari Sub-Zone - 3(f)* ", Revised, Directorate of Hydrology (Regional Studies), Central Water Commission, New Delhi

14 CWC, 1999, "*Guidelines for Risk Analysis in Dam Safety-Draft*" (unpublished, placed before the National Committee on Dam Safety for approval), Central Water Commission,

New Delhi

15 CWC, 2001, "*Manual on Estimation of Design Flood*", Central Water Commission, New Delhi

16 Dooge, J.C.I., 1973. *"Linear Theory of Hydrologic System "*, Technical Bulletin No. 1468, Agricultural Research Service, United States Department of Agriculture.

17 Garde, R.J. and Kothyari, U.C., 1988, "*Flood Estimation Using Regional Flood Frequency Analysis*", International Seminar on Hydrology of Extremes, 1-3 Dec. 1988, Roorkee, INDIA.

18 Garde, R.J. and Kothyari, U.C., 1992, "*Flood Estimation Using Regional Flood Frequency Analysis*", Institution of Engineers (India) Journal, Vol. 73, Nov. 1992.

19 GFCC, 1978, "*Design Criteria for flood protection embankment sections*", Ganga Flood Control Commission, Patna

20 Goel, N.K. et.al, 2001, "*Regionalisation of Hydrological Parameters*",Hydrology Journal, 24(3) 2001 25-32, IAH, Roorkee

21 IRC, 1970, "*IRC: 5-1970 Section I General Features of Design*", Indian Roads Congress, New Delhi

22 Jagmohan Balokhara & Navin Kumar Chittoria, 2003, "*Uttaranchal-An Information Manual*", H.G. Publications, New Delhi-62

23 Jeyaram et al, 1982, "*Snow Covered Area Estimation in Tos Basin Using Landsat Imageries*", Intl. Symposium on Hydrological Aspects of Mountainous Watershed, Nov. 4-6, 1982, Roorkee.

24 Kumar, R., Chatterjee, c., Kumar, S., Jain, S.K., Lohani, A. K. and Singh, R.D., 2001. *"Intercomparison of Responses of HEC - 1 Package and NWish Model"*, Hydrology Journal of IAH, 24 (3), 13 - 24.

25 Linsley R.K.et al, 1975, "*Applied Hydrology*", Tata Mc Graw Hill, Bombay

26 Mathur B.S. et al, 1982, "*Modelling of Flood Flows of Upper Yamuna Watershed*", Intl. Symposium on Hydrological Aspects of Mountainous Watershed, Nov. 4-6, 1982, Roorkee.

27 Mimikou, M., 1983. *"Study of Drainage Basins Linearity and Non-Linearity"*, Journal of Hydrology, 64: 113 - 134.

28 Mishra, S.K, and Garg, V., 1998. *"An SCS-CN Based Time-Distributed Runoff Model"*

Hydrology Journal of IAH, Volume XXI, No.1 - 4, 13 - 34.

29 Mockus, V., 1957. *"Use of Storm and Watershed Characteristic in Synthetic Hydrograph Analysis and Application"*, U.S. Soil Conscrv. Serv., Washington, D.C.

30 Murthy Y.K. and Reddy M.S., 1990, "*Sizing of Spillway for Large Dams in India*", Irrigation and Power", Vol-47, CBI&P, New Delhi

31 Mutreja K.N., 1986, "*Engineering Hydrology*", Tata Mc Graw Hill, Bombay

32 Mutreja, K.N., 1986. *"Applied Hydrology"*, Tata McGraw Hill Publishing Company Limited, New Delhi

33 N.I.H., 1995. *"Technical Report: Derivation of GIUH for Small Catchments of Upper Narmada and Tapi Sub-Basin (Sub-Zone 3e) - Part* I", National Institute of Hydrology, Jalvigyan Bhawan, Roorkee.

34 Narain et al, 2002, "*Derivation of Regional UG for Northern Himalayan Region by Mathematical Modelling approach*", Hydrology Journal, 25 (1) 2002, pp 23-30, IAH, Roorkee

35 Rastogi, et. al, 2003, "*Estimation of Hydrological Response of a Himalayan Watershed by Dimensionless UG*", Indian Journal for Soil Conservation, 31(2), 2003, pp 187-191

36 RBA /MoWR,1980, "*Report of the Rashtriya Barh Ayog*", National Flood Commission, Ministry of Water Resources, Govt. of India, New Delhi

37 Rogers, W.F. and Zia, H.A., 1982. *"Linear and Non-linear Runoff from Large Drainage Basins"*, Journal of Hydrology, 55 : 267 - 278.

38 Rogers, W.F., 1980. "A *Practical Model for Linear and Non-linear Runoff"*, Journal of Hydrology, 46:51-78.

39 Rogers, W.F., 1982. *"Some Characteristics and Implications of Drainage Basins Linearity and Non-Linearity"*, Journal of Hydrology, 55 : 247 - 265.

40 SCS, 1964, "*National Engg Handbook, Hydrology Section 4; Part I Watershed models*", Soil Conservation Service, USA

41 Sharma, R.K., Shiv V.P., and Krishnanunni. N.M., 1998, "*Hydrologic Safety of Dams in India-Practices Followed and Issues Involved*", ICOLD seminar on Dam Safety, Nov. 1998, New Delhi.

42 Sherman, L.K., 1932. *"Stream Flow from rainfall by the Unit Graph Method"*, Eng.

News-Rec., 108,501-505.

43 Singh V.P., 1988, "*Hydrologic Systems: Vol. 1 Rainfall Runoff Modelling*", Prentice Hall Inc., N.J. USA

44 Singh, V.P. and Chen, 1982. *"On The Relation Between Sediment Yield and Runoff Volume. in : Modeling Component of Hydrologic Cycle* ", V.P. Singh, Editor, Water Resources Publications, 555 - 570.

45 Singh, V.P., and Aminian, I-I., 1986. *"An Empirical Relation Between Volume and Peak of Direct Runoff*, Water Resources Bulletin, Journal of American Water Resources Association, 22 (5), 725 - 730.

46 Singh, V.P.and S.J.Chen, 1993, "*Improved SCS Models for Computing Runoff Volume*", J. of Hydrology (IAH), Vol. XVI, pp14-18, Roorkee.

47 Sivapalan, M., C. Jothityangkoon, and M. Menabde, 2002. *"Linearity and Nonlinearity of Basin Response as a Function of Scale: Discussion of Alternative Definitions* ", Journal of Water Resources Research, 38 (2), 4.1-4.5.

48 Soil Conservation Service, 1972. *"National Engineering Handbook"*, section 4, Hydrology, U S Dept. of Agriculture, Washington, D.C.

49 Tyagi, A., Singh, R.D. & Nemo, R., 1995. *"Marquardt Prameya Par Adharil Nash Model Dwara Akak Jalalekh Ka Nirdharan* ", in National Seminar on Hydrology and Water Resources 15 -. 16 December 1995, NIH, Roorkee.

50 UPIRI, 1980, "*Estimation of Peak Floods and Their Frequency for Himalayan Region of Uttar Pradesh*", Technical memorandum, UP Irrigation Research Institute, Roorkee, INDIA.

51 USGS, 1960, "*Manual of Hydrology Part 3-Flood Flow Techniques (Water Supply Papers 1543-4)*", United States Geological Survey, Colarado.

52 Wang, *C.T.*, V.K. Gupta, and E. Waymire, 1981. *"A Geomorphologic Synthesis of Nonlinearity in Surface Runoff"*, Water Resources Research, 17 (3), 545-554.

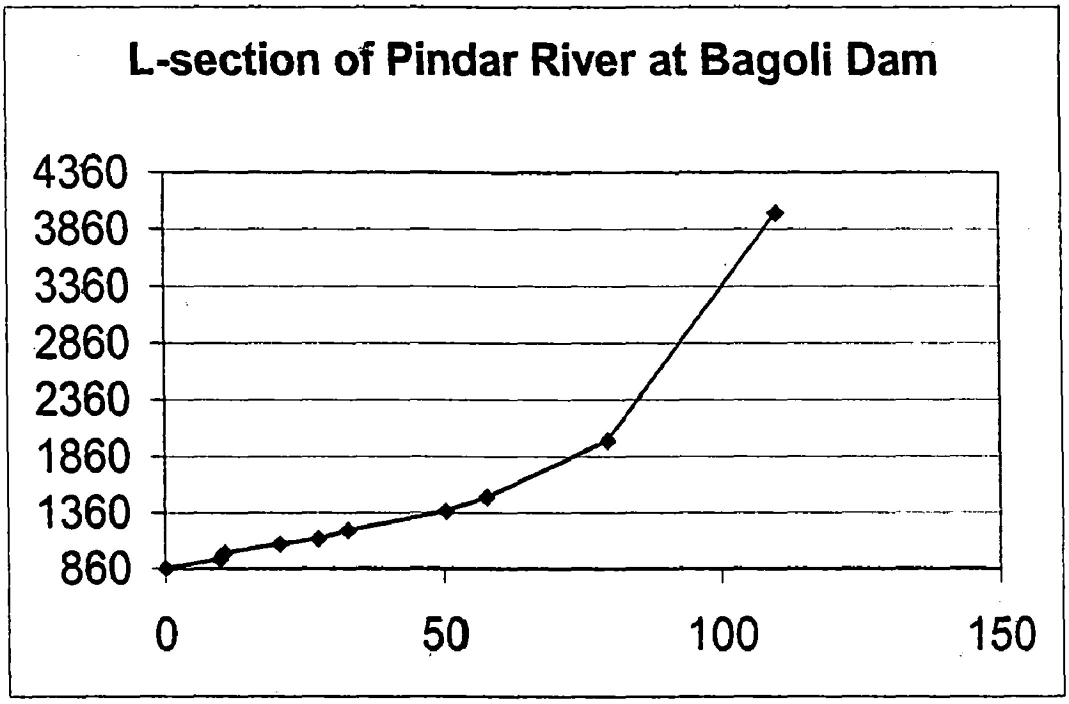
53 WMO 1994, WMO No. 168, "*Guide to Hydrometeorological Practices*", World Meteorological Organisation, Geneva

54 WRDTC, 2002. *"Project Paper of River Engineering Group M. Tech 2001-2002* ", Water Resources Development Training Centre - I.I.T. Roorkee.

COMPUTATION OF EQUIVALENT SLOPE

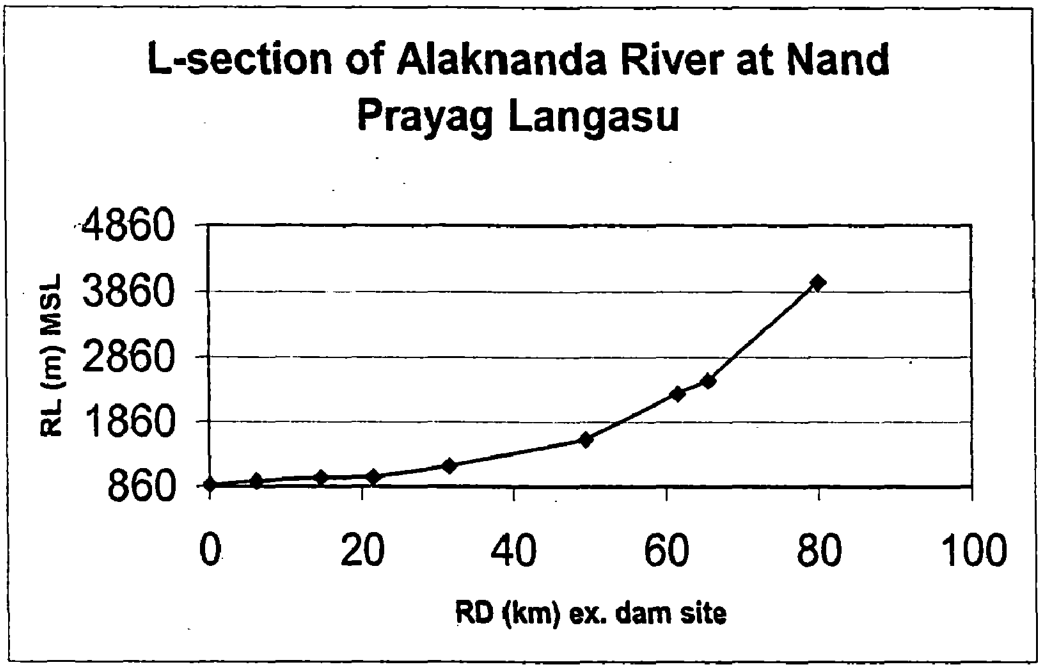
COMPUTATION OF SLOPE FOR RIVER PINDAR AT BAGOLI

Contour (m)	RD (km)	Li (km)	S_i	$L/S_i^{0.5}$
860	0.0	0.0		
943	9.6	9.6	8.7	3.2
1000	10.4	0.9	67.1	0.1
1082	20.2	9.8	8.3	3.4
1124	27.1	6.9	6.1	2.8
1200	32.5	5.4	14.1	1.4
1372	50.3	17.8	9.7	5.7
1500	57.7	7.4	17.3	1.8
2000	80.0	22.3	22.4	4.7
4000	110.0	30.0	66.7	3.7
4500	114.0	4.0	125.0	0.4
Sum		114.0		26.87
	S_{st} =		18.00	



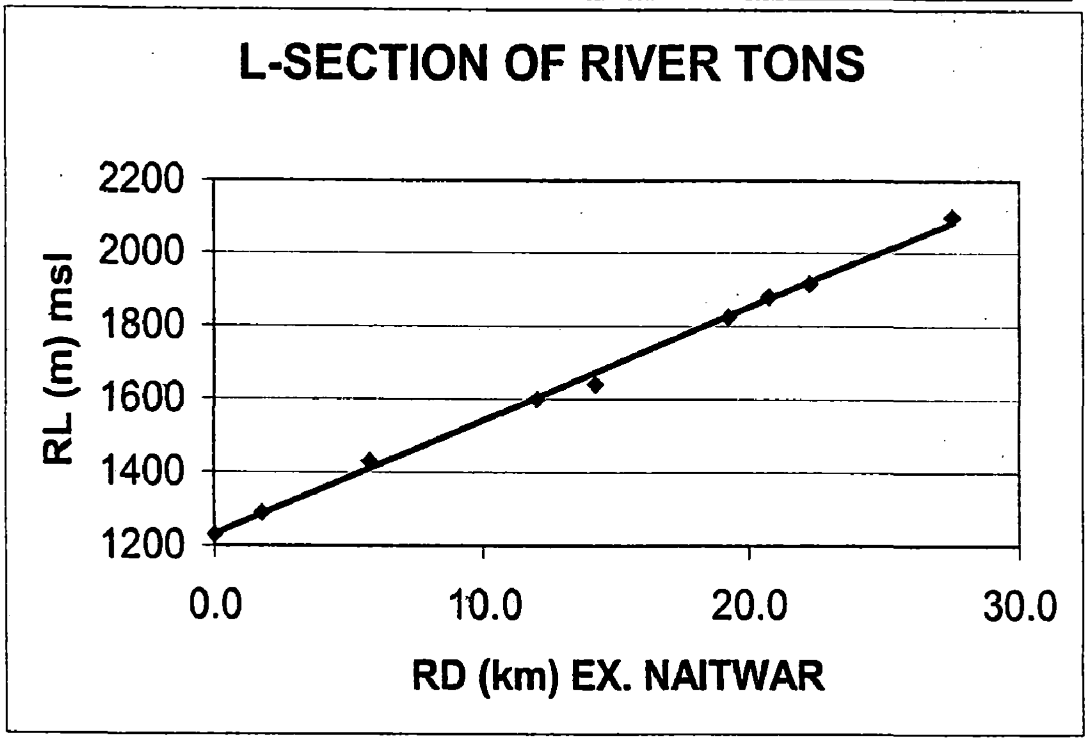
COMPUTATION OF SLOPE FOR RIVER ALAKNANDA AT NPL

contour (m)	RD (km)	Li (km)	Si	$L/S_i^{0.5}$
880	0.0	0.0		
939	6.0	6.0	9.8	1.9
992	14.5	8.5	6.2	3.4
1005	21.5	7.0	1.9	5.1
1172	31.5	10.0	16.7	2.4
1590	49.5	18.0	23.2	3.7
2300	61.5	12.0	59.2	1.6
2500	65.5	4.0	50.0	0.6
4000	80.0	4.0	375.0	0.2
Sum		114.0		26.87
	S_{st} =		18.00	



COMPUTATION OF SLOPE FOR RIVER TONS AT NAITWAR

contour (m)	RD (km)	Li (km)	S_i	$L/S_i^{0.5}$
1230	0.0	0		
1290	1.8	1.75	34.3	0.3
1430	5.8	4	35.0	0.7
1600	12.0	6.25	27.2	1.2
1640	14.3	2.25	17.8	0.5
1825	19.3	5	37.0	0.8
1880	20.8	1.5	36.7	0.2
1915	22.3	1.5	23.3	0.3
2100	27.5	5.25	35.2	0.9
		27.5		4.97
			S_{st} =	30.59



Annexure 4.1.

VARIOUS DEFINITIONS OF TIME LAG

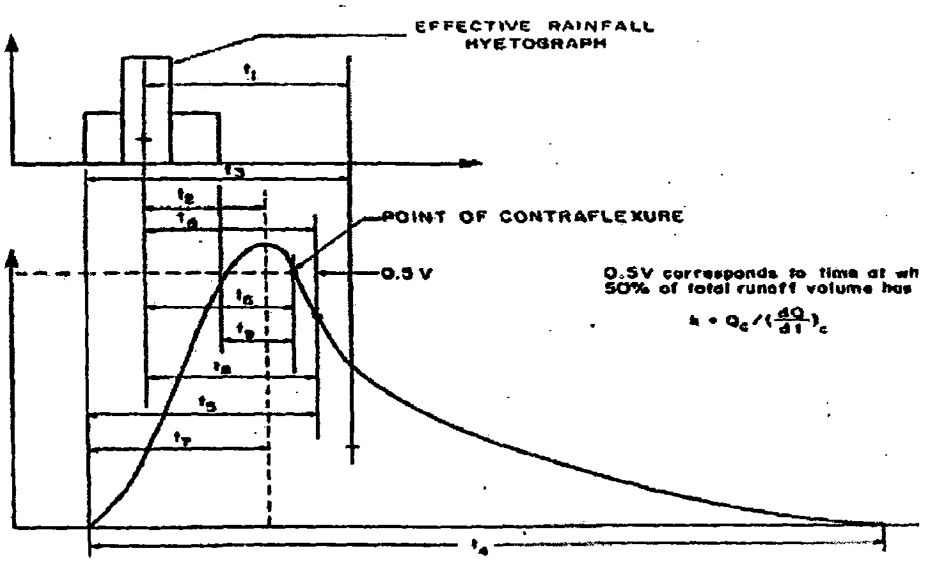


Fig. A. 4.1: Definition Sketch-Various Definitions of Time Lag (Singh, 1988)

Table A 1.1: Some of the Definitions of Lag Time

Symbol	Explanation
t_1	time interval between the centroid of effective rainfall and the centroid of direct runoff
t_2	time interval between the centroid of effective rainfall and peak of direct runoff
t_3	time interval between beginning of effective rainfall and the centroid of direct runoff
t_4	time interval between beginning of direct runoff and the end of direct runoff (time base)
t_5	time interval between beginning of direct runoff and the time when 50% of direct runoff has passed the gauging station
t_6	time interval between centroid of effective rainfall and the point of contraflexure on direct runoff recession
t_7	time interval between beginning of effective rainfall and peak of direct runoff

Annexure 4.2

SYNTHETIC UG DERIVATION FOR A TYPICAL CATCHMENT USING THE SUG APPROACH OF REPORT FOR ZONE 7.

Catchment map of river Pindar, a tributary of Alaknanda at Bagoli is shown below. The computation of slope of main river is given in Chapter 3 (Annexure 3.2).

Step 1: The following physiographic parameters are measured /computed from the map.

Gross CA	1610km^2	River bedlevel	860m
Snowfed CA	363km^2	L	100km
Rainfed CA	1247km^2	L_c	40km
Latitude (dam)	30 12 0 N	S_{st}	18m/km
Longitude (dam)	79 18 0 EE		

Only the catchment area of 1247 is considered in UG derivation. Critical snow-melt is added later to the flood hydrograph.

Step 2: The UG parameters, defined in the sketch in Chapter 3, are computed as below:

t_p	2.498*(L*Lc/S_{st})^0.156	5.80 hr
q_p	1.048/tp^0.178	0.77 cumec/sq.km
Q_p	qp*A	955.6 cumec
W_{50}	1.954*(L*Lc/Se)^0.099	3.34 hr
W_{75}	0.972*(L*Lc/Se)^0.124	1.90 hr
WR_{50}	0.189*(W50)^1.769	1.59 hr
WR_{75}	0.419*(W75)^1.246	0.93 hr
T_B	7.845*(tp)^0.453	17.40 hr
T_p		6.00 hr

Using the above values, a UG is sketched and its shape is further refined to make the volume 10mm. The UG is shown in the Fig. A.4.2.

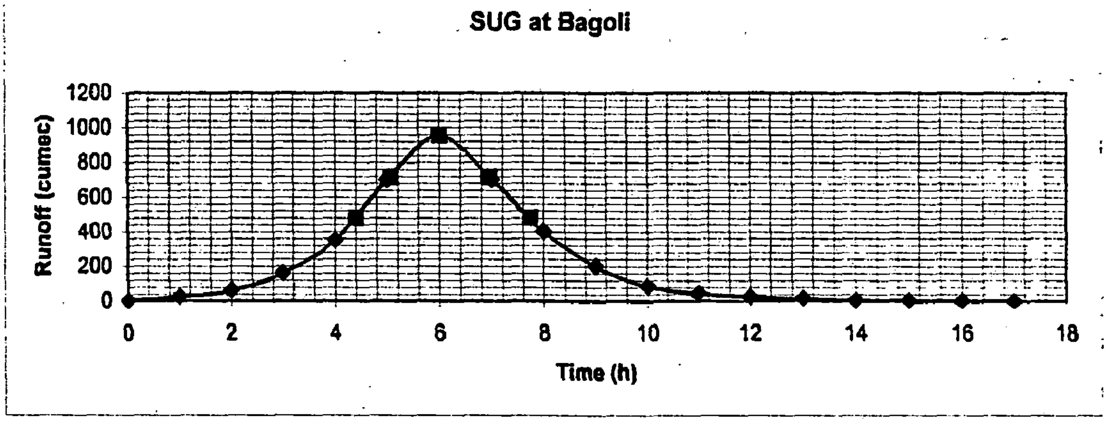


Fig. A.4.2 SUG AT BAGOLI (10mm, 1-hr)

Step 3: The 100-year point storm depth for 24-hr duration was obtained from the relevant fig. as 230mm. The areal correction factor for catchment area of 1247 sq.km. was obtained as 0.85. Hence, the 24-hr, 100-yr return period rainfall for the catchment is 195.5mm. As per the current design practice, the design rainfall is split into 12-hr spells. Using the 12-hr distribution coefficient of 0.76, the 12-hr spells are obtained as 46.9 and 148.6 mm respectively.

Step 4: These are further distributed to hourly increments using the distribution coefficients given in the relevant table. Uniform loss rate of 1.5 mm/hr is substrated and the resulting ordinates are arranged in a critical order (CW&PC, 1972) in two bell spaped spells each of 12-hr separately, to obtain the design excess hyetograph. This is then applied to the design UG obtained in step 2 and the Direct Runoff ordinates (DRO) is obtained. Baseflow at the rate of 0.05 cumec /sq.km. is added to obtain the flood hydrograph corresponding to the area of 1247 sq.km.

Step 5: Critical snowmelt corresponding to the monsoon period is obtained from G&D data of a site downstream of this site and the value is adjusted based on the snow-cover at the two stations. Critical snowmelt value of 150 cumec is used.

Details are given below.

Cumulative rainfall (mm)

10.5	16.7	22.2	26.5	29.6	32.7	35.8	38.9
41.4	43.2	45.1	46.9	33.2	52.8	70.4	84.1
93.8	103.6	113.4	123.2	131.0	136.9	142	148.6

Rainfall increments (mm)

9.0	4.7	4.1	2.8	1.6	1.6	1.6	1.6
1.0	0.4	0.4	0.4	31.7	18.1	16.1	12.2
8.3	8.3	8.3	8.3	6.3	4.4	4.4	4.4

Design Excess hyetograph (mm)

0.4	0.4	1.6	1.6	2.8	4.7	9.0	4.1
1.6	1.6	1.0	0.4	4.4	4.4	8.3	8.3
12.2	18.1	31.7	16.1	8.3	8.3	6.3	4.4

DRO (cumec)

0	1	2	7	21	55	124	254	448	700	1025
1369	1520	1300	967	737	658	828	1249	1828	2540	3385
4434	5535	6018	5288	4141	3139	2280	1527	880	459	225
111	59	28	14	6	3	1	0			

100-yr return period flood hydrograph ordinates (cumec), including critical snow-melt.

212	213	214	220	233	268	336	467	661	912	1237
1582	1732	1513	1179	950	870	1040	1461	2040	2752	3598
4647	5747	6230	5500	4354	3352	2492	1740	1093	671	437
324	271	241	226	218	215	213	212			

Flood events for: Pinder at Karnaprayag

Unit: m^3/s

Time (h)	Event 1	Event 2	Event 3	Event 4	Event 5	Event 6	Event 7	Event 8	Event 9	Event 10
0	83	101	171	218	211	319	85	154	202	237
1	86	107	172	226	218	324	112	155	209	248
2	89	115	175	233	225	341	175	160	216	259
3	94	121	187	236	230	358	237	164	222	261
4	105	133	205	237	236	376	266	166	227	262
5	112	144	219	237	240	397	280	168	237	263
6	126	154	232	236	241	415	337	169	258	265
7	123	164	234	233	244	427	327	173	294	259
8	118	169	297	230	255	430	352	176	306	255
9	112	197	316	262	269	431	352	178	315	252
10	108	200	318	300	286	426	338	180	320	250
11	117	201	308	334	313	416	313	183	312	244
12	115	216	280	344	348	405	288	186	305	
13	111	223	268	327	363	395	266	186	293	
14	108	223	255	316	366	388	236	202	283	
15	107	219	244	301	362	383	226	208	280	
16	111	209	236	288	355	377	214	247	272	
17	110	201	227	277	349	372	202	265	261	
18	105	194	220	268	336	366	194	258	254	
19	101	189	216	257	327	365	189	241	248	
20	99	182	214	251	320	360	180	229	245	
21	97	175	208	243	313	356	173	223	241	
22	92	168	202	236	311	352	166	218	237	
23	87	161	197	230	306	348	160	212	233	
24		154	191	225	301	344	153	205	229	
25		147	186		295	340	147	202	225	
26		133	180		290	337	141	198	222	
		126	178		284	334	136	197	219	
		119			280	327	130	196	216	
		112			276		126	194	213	
		108			272		122	187	210	
					268		118	182		
					265		114	175		
					262		110	168		
					259		105	162		
					255		101			
					252		99			
					250					
					247					
					244					
					241					
					239					
					236					
peak	126	223	318	344	366	431	352	265	320	265

Annexure 5.1 /Sheet 2

Flood events for: **Bhagirathi at Tehri**

Unit: m^3/s

Time (h)	Event 1	Event 2	Event 3	Event 4	Event 5	Event 6	Event 7	Event 8	Event 9	Event 10
0	267	338	424	424	568	564	506	509	700	545
1	267	351	432	431	575	572	513	513	726	564
2	278	370	444	447	587	583	523	522	752	583
3	293	388	457	471	602	593	534	542	779	602
4	314	408	479	497	622	606	545	543	793	642
5	341	431	498	513	638	622	564	587	815	681
6	378	441	527	529	663	642	575	618	827	726
7	432	457	564	529	694	632	589	652	839	770
8	474	466	587	518	719	616	598	696	822	802
9	491	486	602	518	748	602	612	715	804	829
10	483	504	595	506	781	593	642	737	797	815
11	461	516	583	500	770	583	622	752	793	793
12	441	511	555	495	757	575	612	770	770	774
13	432	500	536	491	752		598	781	748	763
14	424	491	527	484	748		587	786	739	748
15	418	491	527	478	743		573	765	726	737
16	413	481	520	469	737		564	757	719	726
17	405	469	514	464	694		555	750	713	715
18	393	462	514	457	663		542	737		704
19	378	454	509	444	654		531	722		683
20	355	441	500	432	648		523	711		667
21	337	436	493		642		518	702		650
22	320	431	486		634			704		632
23	306	423	474		626			702		612
24	293	415	464		618			683		600
25	286	408	457		612			679		583
26		402	451		606			673		570
		396	444		600			669		557
		390	441		595			663		
		385	434		589			656		
		378			583			650		
		366			579			644		
		363						638		
		351						632		
								626		
								612		
								602		
								595		
								587		
								579		
								572		
								564		
								557		
								551		
								545		
								540		
								534		
								531		
								527		

Flood events for: **Alaknanda at Joshimath**

Unit: m^3/s

Time (h)	Event 1	Event 2	Event 3	Event 4	Event 5	Event 6	Event 7	Event 8
0	572	590	629	642	595	624	572	590
1	580	598	634	652	601	632	580	593
2	590	606	645	660	614	642	593	595
3	601	614	655	673	626	655	603	603
4	611	616	668	691	637	668	611	614
5	621	624	678	699	647	675	624	626
6	637	634	694	712	660	688	642	637
7	655	655	709	725	678	701	657	652
8	675	673	722	735	694	712	675	668
9	694	691	737	740	706	719	688	686
10	709	709	750	745	714	714	706	681
11	719	719	761	737	722	699	717	725
12	714	730	771	730	730	688	725	735
13	704	745	766	714	725	673	719	745
14	691	735	755	694	719	663	709	740
15	673	732	737	675	709	650	696	732
16	660	722	725	663	696	637	681	719
17	647	709	714	668	681		668	704
18	637	694	706	655	668		657	688
19	626	673	696		657		645	678
20	616	660	688		647		632	668
21	606	652	678		639		624	660
22	595	642	670		629		616	655
23	590	626	657		619		598	647
24	585	611	650		608		590	639
25		606	642				585	631
26		603						624
								618
								612
								603

Annexure 5.1 sheet 4/6

FLOOD EVENTS STUDIED FOR MANDAKINI AT RUDRAPRAYAG

TIME (h)	RUNOFF (m^3/s)		TIME (h)	RUNOFF (m3/s)							
0	130.0	155.0	0.0	630.2	190.9	262.5	583.8	249.8	337.2	420.8	235.3
3	145.0	170.0	1.0	650.6	226.7	279.5	605.3	259.7	343.3	426.9	238.4
6	150.0	215.0	2.0	656.3	258.0	298.8	635.9	268.0	353.2	434.5	242.9
9	510.0	425.0	3.0	662.0	297.5	319.9	651.1	277.2	364.6	439.8	248.2
12	630.0	632.0	4.0	670.0	331.7	350.8	659.4	286.3	377.5	446.7	253.6
15	618.0	540.0	5.0	670.0	365.1	374.5	656.4	297.7	388.9	455.0	259.7
18	506.0	450.0	6.0	670.0	389.2	449.6	651.1	314.4	401.8	468.7	265.0
21	350.0	365.0	7.0	674.6	368.7	477.0	651.1	324.3	423.1	487.7	282.5
24	250.0	320.0	8.0	682.7	344.4	477.0	651.1	335.7	457.3	510.4	310.6
27	200.0	270.0	9.0	690.8	328.3	477.0	654.8	366.1	502.8	529.4	343.3
30	170.0	230.0	10.0	697.8	310.0	469.1	657.9	411.7	559.8	544.6	375.2
33		202.0	11.0	708.3	293.0	457.4	662.4	453.5	597.7	552.2	404.1
			12.0	721.2	280.0	457.4	688.7	518.0	624.3	540.8	457.3
			13.0	731.8	265.0	445.0	708.9	548.4	648.5	528.7	502.8
			14.0	743.7	250.0	407.7	726.1	525.6	631.8	495.3	552.9
			15.0	749.7	238.0	389.2	737.9	495.3	605.3	478.6	548.4
			16.0	810.3	226.0	357.2	748.3	476.3	586.3	464.0	533.2
			17.0	737.8		331.7	755.7	464.9	573.4	439.8	495.3
			18.0	717.0		324.8	750.5	450.5	559.8	431.0	464.9
			19.0	695		317.9	748.3	438.3	523.3		438.3
			20.0	685		309.7	729.8	420.8	480.1		404.1
			21.0			303.0	718.6	400.3	455.0		391.2
			22.0				696.2	381.3	436.0		373.7
			23.0				681.2	373.7	424.6		360.0
			24.0				666.2	366.1	405.0		350.9
			25.0				651.1	350.0	383.0		344.8
			26.0				639.7	335.0	365.0		339.5
			27.0				637.0	322.0	350.0		325.0
			28.0					310.0			310.0
			29.0					298.0			300.0
			30.0					288.0			290.0
			31.0					275.0			280.0
			32.0					260.0			
			33.0								

ALAKNANDA AT RUDRAPRAYAG

(RUNOFF IN m^3/s)

Time	Event 1	Event 2	Event 3	Event 4	Event 5
0	810.0	760.0	760	760.0	0.0
6	750.0	900.0	802	820.0	0.0
12	670.0	1060.0	1270	925.0	615.0
18	615.0	925.0	1150	1212.0	670.0
24	955.0	665.0	925	900.0	955.0
30	1219.0	630.0	825	800.0	1219.0
36	1400.0	0.0	777	777.0	1300.0
42	1044.0		0	0.0	1044.0
48	760.0				760.0
54	900.0				640.0
60	1060.0				
66	925.0				
72	665.0				
78	785.0				

Annexure 5.1 sheet 6/6

FLOOD EVENTS FOR GOLA AT DAM SITE

(RUNOFF IN m^3/s)

Event 1	Event 2	Event 3	Event 4	Event 5	Event 6	Event 7	Event 8	Event 9	Event 10
54.0	54.0	32.0	13.0	20.0	103.0	35.9	110.1	20.0	76.2
58.3	58.3	100.0	17.5	22.8	355.0	90.8	351.3	164.0	139.8
153.3	153.3	120.0	22.0	25.7	465.0	133.1	352.9	350.0	263.0
153.3	153.3	132.0	27.5	28.5	637.0	189.6	355.9	725.0	356.7
161.6	181.6	145.0	33.0	31.3	465.0	266.4	265.4	640.0	269.1
225.0	266.6	172.0	58.5	34.2	382.0	329.1	231.0	506.0	231.7
418.3	361.6	230.0	84.0	37.0	351.5	420.1	223.3	400.0	211.5
506.3	481.5	320.0	98.0	47.5	321.0	508.6	215.6	376.0	196.3
506.3	425.3	300.0	112.0	58.0	290.5	568.4	208.0	312.0	166.2
418.3	361.6	284.4	148.0	56.0	260.0	461.1	200.6	256.0	176.1
266.6	266.6	268.8	145.0	67.3	240.0	450.1	193.6	162.0	166.0
266.6	245.1	253.1	115.0	75.3	220.0	439.0	186.8	130.0	155.6
245.1	239.9	237.5	103.3	82.0	200.0	410.2	186.8	44.0	145.7
239.9	213.5	225.6	91.7	86.3	180.0	381.4	183.5		135.6
240.0	213.5	213.8	80.0	90.7	163.8	352.5	180.2		125.5
213.5	210.7	201.9	73.6	95.0	147.5	323.7	176.9		115.4
210.7	197.4	190.0	67.2	141.0	131.3	294.9	171.0		105.3
197.4	194.1	181.0	60.7	125.5	115.0	266.0	165.2		95.2
194.1	190.9	172.0	54.3	110.0		237.2	159.3		91.7
190.9	187.6	158.5	49.3	96.0		208.4	153.5		66.6
187.6	184.4	145.0	44.3	82.0		179.5	147.6		65.2
184.4	176.0	138.5	39.3	74.2		169.1	141.7		
176.0	167.6	132.0	34.3	66.5		158.7	135.9		
167.6	164.7	126.0	30.0	58.7		148.4	130.0		
164.7	161.7	120.0	25.0	50.9		138.0	125.0		
161.7	161.7	110.0	22.0	45.7		127.6	122.0		
161.7	161.7	100.0	18.5	43.0		117.2	120.2		
161.7	146.0	93.0		36.5		106.8	119.0		
146.0	130.2	86.0		30.0		96.4	118.0		
130.2	130.2	64.0		28.0		86.1	117.5		
130.2	130.2	42.0				75.7			
130.2	125.9					53.0			
125.9	121.5								
121.5	104.5								
104.5	87.4								
87.4	75.4								
75.4	63.3								
63.3									

Note: The runoff ordinates at 2-hourly intervals for flood events 1 to 6 and 10
The runoff ordinates at 1-hourly intervals for flood events 7 & 8
The runoff ordinates at 6-hourly intervals for flood event 9