HYDROLOGIC CONSIDERATIONS IN SAFETY OF DAM

A DISSERTATION

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

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By

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CANDIDATE'S DECLARATION

I hereby certify that the work, which is being presented in the dissertation entitled "**HYDROLOGIC CONSIDERATIONS IN SAFETY OF DAM**" in partial fulfillment of the requirement for the award of degree of **Master of Technology in Water Resources Development** in the Department of Water Resources Development and Management of Indian Institute of Technology Roorkee, is an authentic record of my own work carried out during a period from July 2009 to June 2010 under the guidance of **Prof. U.C. Chaube,** Professor Department of Water Resources Development and Management, Indian Institute of Technology Roorkee, India.

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

Dated : June 28, 2010

Place : Roorkee

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CERTIFICATE

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

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ABSTRACT

This dissertation report is based on review of guidelines for the hydrologic evaluation to determine the safety of dams and allied structures. Focus is on hydrological aspects only.

Many of the older dams are now characterized by increased hazard potential due to developments in flood plain and increased risk due to structural deterioration or inadequate spillway capacity of the dam. The Government of India has constituted Dam Safety Organization in the Central Water Commission during June 1979 Guidelines issued by Dam Safety Organization are reviewed.

There have been 26 major dam failures in India in post independence period. Further, literature review of flood estimates of 62 large dams shows that reassessed design floods are significantly larger than earlier estimates. With occurrence of more severe events in a large sample, earlier estimate of design flood are bound to be revised up ward as illustrated through flood reviews of dams in India. Subjectivity in estimation of PMP and PMF should be minimized by evolving consensus and codifying the criteria, and procedures for estimation of design flood.

When various storms are considered for development of Unit Hydrograph for the same catchment a marked variation is observed in the peak as well as the time of occurrence of the peak. Therefore average Unit Hydrograph needs to be derived giving higher weightage for the Unit Hydrograph derived from severe storms. Different unit hydrographs should be identified for the various conditions which have major influence on formation and time distribution of the runoff. These unit hydrograph may then be judiciously applied under different conditions.

Recent flood hydrographs should be used for derivation of UH. Changes in land use, land cover over the catchment area should be and should be duly considered in the analysis. Errors in forecasting floods in term of estimation of peak discharge and time to peak due to use of different UH when rainfall is not uniform and when rainfall is assumed to be uniform over the catchment are significant as shown by case study of Baitarni basin(India).

Effect of various factors on design flood estimation is analysed through case study of floods at Bhakra dam site on river Sutlej in India Different probability distributions provide significantly different estimates e.g. using 1909-1992 data series of Bhakra dam, 10000 year estimate by EVI is 21,036.26 m^3/s and by PIII is 26,154.53

m^3/s (24.33 % higher). Using same probability distribution but different samples from same population also result in significantly different estimates. 10000 year flood estimate using LN II probability distribution are 18,732.75 cumec (1909-92 data series), 15,064.06 cumec (1909-59 data series) and 24,588.81 m^3/s (1960-92 data series).

Case study of Wonogiri watershed, Indonesia, has been carried out to find the largest depth of PMP (mm) for the catchment estimated (using Hersfield equation). HEC-HMS (Hydrologic Modeling System) has been used to compute the PMF, using PMP depths as input. Likewise, dam and spillway performance can be simulated with the reservoir model included in HEC-HMS. For that reason, the analysis must derive and specify functions that describe how the reservoir will perform. The peak spillway discharge is 5678.5 m^3/sec as computed using HEC-HMS software whereas the spillway capacity is 5100 m^3/sec. The spillway capacity of 5100 m^3/s corresponds to 1.2 times 100 years probable flood (table 6.2) where as the probable maximum flood as given in project report (table 6.2) is 9600 m^3/sec. Therefore it is concluded that there is adequate spillway capacity in the Wonogiri dam reservoir, Indonesia.

Dam break flow analysis for Wonogiri dam has been performed assuming a hypothetical dam failure case. A mathematical model 'DAMBRK' has been used for this purpose. The peak discharge simulated from dam break analysis at the dam site is 19,289 m^3/sec and it gradually decreases to 19,203 m^3/sec at a distance of 3.1 kms. The dam break peak discharge at dam site (19,289 m^3/s) is significantly higher than the probable maximum flood (9600 m^3/sec).The maximum water level at the dam site is 136.06 m and at 3.1 km distance it is 121.89 m.

Keywords: dam failures, dam safety, flood hydrograph, peak floods, Flood frequency, PMP & PMF, hydrologic modeling, spillway capacity, dam break flow.

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LIST OF NOTATIONS

Flood forecasting:

A	=	Absolute Error
E	=	Efficiency
R	=	Relative Error
K_N	=	Outliers test K values for 10 percent significance level for a Normal distribution
X_m	=	Errors due to the Choice of Number of Verticals
X_d	=	Errors in Measuring Depth
X_f	=	Errors on account of duration of exposure of Current Meter
Xo	=	Errors due to the choice of number of point in a vertical
X_q	=	The overall Random Errors
Yo	=	Actual level at $(N+T)_{th}$ time
Yf	=	Forecast level at $(N+T)_{th}$ hour
Y	=	Nth hour level on the basis of which the forecast has been formulated
Mf	=	means of forecasted stream flow (m^3/s)
Mo	=	means of observed stream flow (m^3/s)
MSE	=	Mean squared error
RMSE	=	Root mean squared error
P	=	Probability
CA	=	Catchment area (km^2)
L	=	Length of main stream from farthest point up to outlet (km)
Lc	=	Length of main stream from a point on main stream nearest to centroid (km)
Q_f	=	forecasted stream flow (m^3/s)
Q_o	=	observed stream flow (m3/s)
R^2	=	Coefficient of determination
UH	=	Unit hydrograph
W_{50}	=	Width of UH at 50% peak discharge
W_{75}	=	Width of UH at 75% peak discharge
WR_{50}	=	Width of rising limb of UH at 50% peak discharge
WR_{75}	=	Width of rising limb of UH at 75% peak discharge

S	=	Slope of catchment (m/km)
Tp	=	time from start to peak (hr)
tr, D	=	duration of UH
T	=	Return period (year)
T_b	=	Base period of UH (hr)
Q	=	discharge (m3/s)
Q_p	=	peak discharge (m3/s)
V	=	runoff volume as depth of over catchment (cm)

Flood frequency approach:

S	=	Standard deviation of the log transformed series
X_H	=	High outlier threshold
X_L	=	Low outlier threshold
X	=	Mean of the log transformed series
Xb	=	Errors in measuring width
V	=	Variance

Hershfield statistical method:

ARF	=	Area reduction factor
PMP	=	Probable maximum precipitation
PMF	=	Probable maximum flood
X_{PMP}	=	Point value of PMP (mm)
$\overline{X}n$	=	Mean annual maximum rainfall (mm)
K_m	=	Frequency factor-Function of rainfall duration and mean annual maximum rainfall
σ_n	=	Standard deviation of of a series of n annual maximum rainfall in Hershfield statistical method
X_m	=	he highest value from the series.
$\overline{X}_{n-m}$	=	the mean excluding the X_m value from the series.
σ_{n-m}	=	the standard deviation excluding the X_m value from the series.

Dam break flow analysis:

b	=	width of the breach (m)
h	=	flow depth (m)
u	=	flow velocity (m/s)
x	=	distance (m)

h_0	=	dam height (m)
C_0	=	wave celerity,
T_b	=	failure time interval (hour)
h_{bm}	=	final elevation (m)
I	=	reservoir inflow (m^3/s)
Q	=	reservoir outflow (m^3/s)
ds/dt	=	rate of change of storage volume
A, A_0	=	active and inactive flow areas
x	=	distance along the channel (m)
t	=	time (hour)
q	=	lateral inflow or outflow /unit distance along the channel (m^3/s)
g	=	gravitational acceleration
Q	=	discharge (m^3/s)
h	=	water surface elevation
S_f	=	friction slope
S_c	=	expansion-contraction loss slope.

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CHAPTER I

INTRODUCTION

1.1. BACKGROUND

For centuries, dams have provided mankind with such essential benefits as water supply, flood control, recreation, hydropower, and irrigation. They are an integral part of society's infrastructure. However in the last few decades, several major dam failures have increased public awareness of the potential hazards caused by dams. In today's technical world, dam failures are rated as one of the major "low-probability, high-loss" events. There are very large numbers of dams all over the world that are 50 or more years old. Many of the older dams are now characterized by increased hazard potential due to development in flood plain in downstream and increased risk due to structural deterioration or inadequate spillway capacity of the dam. Several of the old dams were designed with inadequate hydrological investigations.

In developing countries such as in Indonesia, India, Bangladesh, there is pressure to live and work in flood-prone areas, which typically feature attractive rich soils, sources of abundant water supplies and ease of transport. At present about 1 billion people - the majority of them among the world's poorest inhabitants - are estimated to live in the potential path of a 100-year flood and, unless preventative efforts are stepped up worldwide, that number could double or more in two generation period. The number of people worldwide vulnerable to a devastating flood is expected to grow to 2 billion by 2050 due to climate change, deforestation, rising sea levels and population growth in flood-prone lands. (Sharma K.D. & P. Singh 2007).

1.1.1. Significant Dam Failures in the World

The total number of dams in the world which represents Hazards in the event of failure may exceed 150,000. As a rough estimate, there have been perhaps 2000 failures, including partial collapses, since 12th century A.D. Most of these were not major dams. There have been around 200 notable reservoir failures in the world. So far in the 20th century more than 8000 people died in these disaster. (Source: Dams & Public Safety USBR-1983).

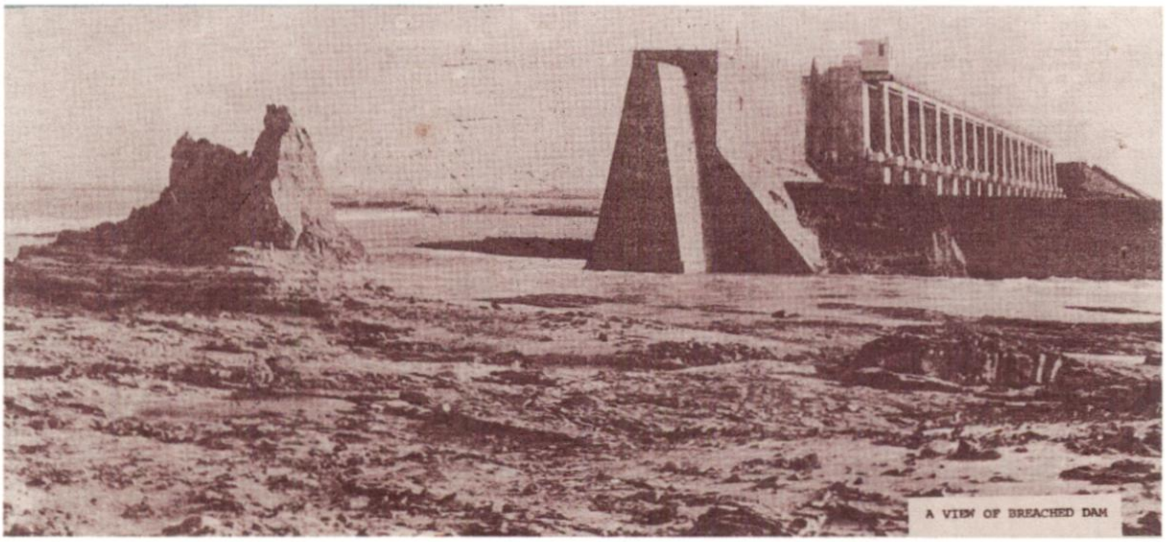


Figure 1.1: A view of breached dam

Table 1.1: Examples of Significant Dam Failures (Excluding India and China)

No.	Dam	Country	Year	Lives lost
1	San Ildefonso	Bolivia	1979	2000 +
2	Vaiont	Italy	1626	Unknown
3	South Fork	USA	1889	2209
4	Oros	Brazil	1960	Unknown
5	Puentes	Spain	1820	608
6	Kuala Lumpur	Malaysia	1961	600
7	Gleno	Italy	1923	600
8	St.Francis	USA	1928	450
9	Malpasset	France	1959	421
10	Hyokiri	S.Korea	1961	250
11	Quebrada la Chapa	Colombia	1963	250
12	Bradfield	England	1864	238
13	El Habra	Algeria	1881	209
14	Sempor	Indonesia	1967	200
15	Walnut Grove	USA	1890	150
16	Babii Yar	USSR	1961	145
17	Vega de Tera	Spai	1959	144
18	Mill River	USA	1874	144
19	Buffalo Creek	USA	1972	125
20	Valparaiso	Chile	1888	Over 100
21	Alia S. Zerbino	Italy	1935	Over 100
22	Bouzey	France	1895	Over 100
23	Zgorigard	Bulgaria	1966	100
24	Austin	USA	1911	80
25	Bila Desna	Czch.	1916	65
26	Fr ias	Argentina	1970	42 +
27	Lower Otay	USA	1916	30
28	Palagnedra	Switz.	1978	24
29	Eigiau-Coedty	Wales	1925	16
30	Teton	USA	1976	11
31	Baldwin Hills	USA	1963	5

Source: Dams & Public Safety USBR-1983

1.1.2. Causes of Failure

Dams may fail due to variety of reasons as given in table 1.2. Analysis of 1620 dams (Dams & Public Safety USBR-1983) shows that major causes are foundation failure and inadequate spillway capacity. 23% of dam failures have been examined due to inadequate spillway capacity.

Table 1.2: Cause of Dams Failure-Spanish Experience

No.	Cause	Percent of Failure
1	Foundation Failure	40
2	Inadequate Spillway	23
3	Poor Construction	12
4	Uneven Settlement	10
5	High Pore Pressure	5
6	Acts of War	3
7	Embankment Slips	2
8	Defective Materials	2
9	Incorrect Operation	2
10	Earthquake	1

Source: Dams & Public Safety USBR-1983

1.2. OBJECTIVE OF STUDY

This study has been taken up to critically analyse hydrologic considerations in dam design and consequent dam safety over its life. Objectives of this study are:

1. To review available literature on: Country practices on dam safety (India, Indonesia, USA), Design Flood criteria and estimation procedure.
2. To suggest improvement in flood forecasting for dam safely.
3. To analyse factors affecting flood estimation using hydrometeorological approach and flood frequency approach.
4. To analyze hydrologic safety of Wonogiri dam (PMP, PMF, spillway approach) using HEC-HMS software.
5. To carry out dam break analysis of Wonogiri dam.

1.3. ORGANIZATION OF DISSERTATION REPORT

The dissertation is arranged in six chapters as follows:

1. Chapter I: The first chapter provides background of the study and objectives which are proposed to be achieved in this study.
2. Chapter II: This chapter covers review of literature relevant this study and some guidelines which are useful for analysis in this study.

3. Chapter III: Flood forecasting for dam safety have been described in this chapter.
4. Chapter IV: Factors affecting flood estimation using hydrometeorological approach have been analysed.
5. Chapter V: In chapter fifth, factors affecting design flood using flood frequency approach are analysed.
6. Chapter VI: PMP estimation and PMF evaluation of spillway adequacy for Wonogiri dam reservoir have been described in this chapter.
7. Chapter VII: Literature on dam break analysis is reviewed and DAMBRK software is used in the dam break flow analysis of Wonogiri dam described in chapter seventh.
8. Chapter VIII: Chapter eighth covering the conclusion for the improvement of the safety of the dam.

CHAPTER II

LITERATURE REVIEW

Literature has been reviewed on a selective basis:

Dam Safety in India (dam failures, review of design estimates of some dams, dam safety service and procedure for hydraulic and hydrologic analysis)

Flood Estimation (effect of climate, design flood criteria in India and Indonesia, and uncertainty in PMF estimation).

2.1. DAM FAILURES IN INDIA

Since independence, a large number of dams have been constructed in India at great cost. Though most of them performed well, some did develop problems. In view of the fact that large number of dams have been and are being constructed in the country by different agencies under varying conditions, the failure of such structures is fraught with serious consequences involving extensive damage to the property and loss of life in the down stream of dams. Repairs and replacement require extensive financing and time. Therefore, it is of utmost importance to take every care so as to avoid defects and eliminate the possibility of failure either complete or partial of any of the structures.

There have already been a few major and partial failures of dams in the country. Table 2.1 provides brief details of the failures of 26 dams in India. Machhu II dam in Gujarat was completed in the year 1975 and the dam failed just after 4 years in the year 1979. Sampna dam in Madya Pradesh was completed in the year 1956. It has partially failed three times (in the year 1957, 1961 and 1964). Similarly Kaili Sindh dam in Rajasthan has failed in the year 1956, 1960 and 1961 (CWC 1986).

Table 2.1: Recent Failures of Dams in India

Sl.No.	Location	Name of the Project	Type	Max. ht. (M)	Year of Completion	Year of Failure
1	Andhra Pradesh	Kaddam	Composite	22.5	1957	1958
2	Gujarat	Dantiwada	Earth	41.1	1965	1973
3	Gujarat	Machhu II	Masonary-Earth	24.1	1975	1979
4	Karnataka	Chikkahole	Masonary	36.7	1968	1972
5	Madhya Pradesh	Sampna	Earth	21.3	1956	1957/61/64
6	Madhya Pradesh	Palakmati	Earth	14.6	1942	1953
7	Madhya Pradesh	Gopalapura	Earth	N.A.	1955	1955
8	Madhya Pradesh	Nawagaon	Earth	N.A.	1958	1959
9	Madhya Pradesh	Kedarnala	Earth	N.A.	1964	1964
10	Maharashtra	Ashti	Earth	17.7	1883	1883/1933
11	Maharashtra	Bandsura	Composite	21.6	N.A.	1962
12	Maharashtra	Panchet	Earth	53	1963	1963
13	Maharashtra	Khadakwasla	Masonary	60	1875	1963
14	Ra jasthan	Dakhya	Earth	N.A.	1953	1953
15	Ra j as than	Girinanda	Earth	N.A.	1954	1955
16	Rajasthan	Arwar	Earth	12.5	1956	1956/57
17	Ra jasthan	Guddah	Earth	28.3	1956	1956/57
18	Rajasthan	Kaili Sindh	Masonary	N.A.	N.A.	1956/60/61
19	Rajasthan	Sukri	Earth	N.A.	N.A.	1958
20	Rajasthan	Dervakheda	Earth	N.A.	N.A.	1959
21	Rajasthan	Galwania	Earth	N.A.	1960	1961
22	Rajasthan	Nawgaza	Earth	N.A.	1955	1961
23	Rajasthan	Kaneda	Earth	N.A.	N.A.	1962
24	Rajasthan	Bhimlat	Masonary	N.A.	N.A.	N.A.
25	Uttar Pradesh	Ahraura	Earth	22.8	1953	1953
26	Uttar Pradesh	Nanak Sagar	Earth	16	1962	1967

Source : CWC(1986)

2.2. REVIEW OF DESIGN FLOOD ESTIMATES OF DAM IN INDIA

Following is based on study of information available in Sharma et al (1999), Krishnaunni N. M. (2004) and other project documents available in the library of WRDM, IIT Roorkee.

2.2.1. 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 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 m^3/s and this was later revised to 51819 m^3/s with a volume of 35931 MCM. Further studies made in 1952 showed that the 500 year return period flood would have a peak of 42474 m^3/s (15 lakh cusecs) which was adopted for design of the structure. (Krishnaunni 2004).

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 m3/s (15 lakh cusecs) and a release of the order of 31148 m3/s (11 lakh cusecs) observed during July 1961. ii) Estimated inflow of 37717 m^3/s (13.32 lakh cusecs) and spillway discharge of 33385 m^3/s (11.79 lakh cusecs) during 20 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 daily rainfall data available for the region for the last 100 or so years were made : The PMF at Hirakud dam has been estimated to be having a peak of 69,632 m^3/s (24.59 lakh cusecs) with a volume of 16,800 MCM.

"Report on Dam Safety Procedures", published by CWC in 1986 states that, 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 Hirakud dam during any period up to 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.

2.2.2. Gandhi Sagar Dams (Madhya Pradesh)

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 Ihambal river were reviewed. Gandhi Sagar dam 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 21240 m^3/s (7.5 lakh cusecs). Higher floods of (21382 m^3/s) and (23279 m^3/s) 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 (39790 m^3/s). As per the review of design flood conducted in the year 1994, the peak of PMF hydrograph has been estimated as 54,390 m^3/s (19.21 lakh cusecs).

2.2.3. Other Dams

Central Water Commission has reviewed the hydrologic safety of nearly 62 large dams in the country. Table 2.2 provides magnitude of design flood used in design and estimated design flood as per review carried out by Central Water Commission after completion of dams (Sharma et al 1999). 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.

Table 2.2: Design Flood used in Design and as per Review

SI	Name of Dam	Site	Year of completion	River	Height above lowest foundation (m)	Design spillway capacity (m3/s)	Design flood used in design (cumecs)	Design flood as per review (cumecs)
1	Pagara	M.P	1927	Asan	23	1500	1337	4692
2	Gandhi Sagar	M.P	1960	Chambal	64	21240	21200	54390
3	Tigra	M.P	1917	Sank	24	1274	1455	4067
4	Kakcto	M.P	1935	Parvati	32	3028	1811	5728
5	Aoda	M.P	1934	Seep	22	1250	1168	3089
6	Hirakud	Orrisa	1957	Mahanadi	59	42459	42474	69632
7	Darjang	Orrisa	1977	Ningara & Matalia	26	2830	2831	4130
8	Ghodahada	Orrisa	1978	Ghodahado	27	906	906	1900
9	Ganianala	Orrisa	1975	Gania Nallah	15	129	128	380
10	Alikuan	Orrisa	1977	Ragandhnalla	12	201	166	630
11	Parbati	Rajasthan	1959	Parbati, Yamuna	29		1722*	7150
12	Alnia	Rajasthan	1961	Alnia, Chambal	14		2152*	2605
13	Galwa	Rajasthan	1960	Galwa, Banas	22		1014*	4010
14	Kumbbo	Orissa	1982	Kumbho Nallah	15	231	231	703
15	Talkhol	Orissa	1977	Sanjorinallah	11	158	157	333
16	Jawai	Rajasthan	1957	Jawai	35	4248	1900	6469
17	Morel	Rajasthan	1956	Morel, Banas	28		1642*	23457
18	Gambhiri	Rajasthan	1956	Gambheri, Banas	21		2039*	8144
19	Sampna	M.P.	1956	Kalar	22		158	600

** Design spillway capacity as design flood not available*

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 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.

2.3. DAM SAFETY PROCEDURE/PROGRAMME IN INDIA

Meeting of the State Ministers of Irrigation held on July 17-18, 1975 deliberated on the safety of dams and the conference recommended that "In view of the increasing number of large dams in India, the Govt. of India may constitute an advisory Dam Safety Service to be operated in Central Water Commission."(CWC 1979).

The Government of India constituted Dam Safety Organization in the Central Water Commission during June 1979 to assist the State Governments to locate causes of potential distress affecting safety of dams and allied structures and to advise / guide the State Governments in providing suitable remedial measure. It initiates action in response to specific requests from the State Governments or in consultation with the State Governments concerned. A consultative Committee is then set up comprising officers of Dam Safety Service of Central Water Commission, the Director of the Specialized Directorate concerned with the particular topic, and expert/experts from a panel of consultants and representatives of State Govts. concerned. After study of the problems in detail, the Chief Engineer (Dam Safety) communicates his suggestions for remedial measures to the concerned officers for further action.

The functions of the Dam Safety Organization are as under (CWC 1987):

1. to document the salient design features of the project and data on which it has been based;
2. to visit, examine and study conditions of dams and allied works to verify construction methods and specifications adopted;
3. to be of assistance to the State Governments, at the time of pre-commissioning of dams and allied hydraulic structures, specially with respect to functioning of gates, surplussing arrangements and overall behavior of structures;
4. to continue periodic visits of dams and allied structures during the post construction period; and
5. to review the structural behavior reports received from the Engineer in charge of the dam and or allied structures on the basis of the instruments embedded in the Dams.

2.3.1. Selection of Dams to be Investigated for Dam Safety

Dams are classified in accordance with size and hazard potential.

1. Size

The classification for size is based on the height of the dam and storage capacity (Table 2.3.). The height of the dam is established with respect to the maximum storage potential measured from the natural river bed at the downstream toe of the dam. For the purpose of determining dam size, the maximum storage elevation may be considered equal to the top of dam elevation. Size classification may be determined by either storage or height, whichever gives the large size category.

Table 2.3: Size Classification

Category	Storage (Hectare Metres)	Height (Metres)
Minor	< 125 and ≥ 6	< 12 and ≥ 8
Medium	≥ 125 and < 6250	≥ 12 and < 30
Major	≥ 6250	≥ 30

Source: Guidelines for Safety Inspection of Dams, Central Water Commission, Ministry of Water Resources, Government of India.

2. Hazard Potential

The classification for potential hazards should be in accordance with Table 2.2. The hazards pertain to potential loss of human life or property damage in the area downstream of the dams in the event of failure or misoperation of the dam or appurtenant facilities.

Table 2.4: Hazard Potential Classification

Category	Loss of Life (Extent of Development)	Economic Loss (Extent of Development)
Low	None expected (non-permanent structures for human habitation)	Minimal (undeveloped to occasional structures or agriculture)
Significant	Few (no developments more than number of structures) urban and no a small inhabitable	Appreciable (notable agriculture, industry or structures)
High	More than few	Excessive (extensive community, industry or agriculture)

Source: Guidelines for Safety Inspection of Dams, Central Water Commission, Ministry of Water Resources, Government of India (1987).

Those dams possessing a hazard potential classified high or significant as indicated in Table 2.4 should be given first and second priorities, respectively, in the inspection programme.

2.3.2. Evaluation of Hydraulic and Hydrologic Features

1. Design data

All constraints on water control such as block entrances, restrictions on operation of spillway and outlet gates, inadequate energy dissipater or restrictive channel conditions, significant reduction in reservoir capacity by sediment deposition and other factors should be considered in evaluating the validity of discharge ratings, storage capacity hydrographs, routings and regulation plan. The discharge capacity and/or storage capacity should be capable of safely handling the recommended spillway design flood.

The Indian Standard IS: 11223 – 1985 "Guidelines for fixing spillway capacity" (BIS 1985) gives the criteria for inflow design flood as under:

The dams may be classified according to size by using the hydraulic head (from normal or annual average flood level on the downstream to the maximum water level) and the gross storage behind the dam as given below. The overall size classification for the dam would be greater of that indicated by either of the following two parameters and the inflow design flood for safety of the dam would be as follows:

Table 2.5: Inflow Design Flood – BIS Criteria

Classification	**Gross Storage**	**Hydraulic Head**	**Inflow design flood for safety of dam**
Small	Between 0.5 & 10 million m3	Between 7.5 m & 12 m.	100 year flood
Intermediate	Between 10 & 60 million m3	Between 12 m & 30 m.	SPF
Large	Greater than 60 million m3	Greater than 30 m.	PMF

Source: BIS : 11223 – 1985 "Guidelines for fixing spillway capacity" (BIS 1985).

The relevant parameters to be considered in judging the hazard in addition to the size would be:

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

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

For more important projects, dam break studies may be done as an aid to the judgment in deciding whether PMF needs to be used.

2. Experience data

In some cases where design data are lacking, an evaluation of overtopping potential may be based on watershed characteristics and rainfall and reservoir records. An estimate of the probable maximum flood may also be developed from a conservative, generalized comparison of the drainage area, size and the magnitude of recently adopted probable maximum floods for dam sites in comparable hydrologic regions. Where the review of such experience data indicates that the recommended spillway design flood would not cause overtopping, additional hydraulic and hydrologic determinations will be unnecessary.

2.3.3. Hydraulic and Hydrologic Analysis

1. Hydraulic and hydrologic capabilities should be determined using the following criteria and procedures.

Depending on the project characteristics, either the spillway design flood peak inflow or the spillway design flood hydrograph should be the basis for determining the maximum water surface elevation and maximum outflow. If the operation or failure of upstream water control projects would have significant impact on peak flow or hydrograph analyses, the impact should be assessed.

2. Maximum water surface based on SDF peak inflow.

When the total project discharge capability at maximum pool exceeds the peak inflow of the spillway design flood (SDF) and operational constraints would not prevent such a release at controlled projects, a reservoir routing is not required. The maximum discharge should be assumed equal to the peak inflow of the spillway design flood. Flood volume is not controlling in this situation and surcharge storage is either absent or is significant only to the extent that it provides the head necessary to develop the release capability required.

2.3.4. Peak for Standard Project Flood (SPF)

When the SPF flood is applicable, and data are available, the spillway design flood peak inflow may be determined by usual conventional methods. Flow frequency information from regional analysis is generally preferred over a single station results when available and appropriate.

1. Peak for PMF

The unit hydrograph – infiltration loss technique is generally the most expeditious method computing the spillway design flood peak for most projects.

2. Maximum water surface based on SDF Hydrograph

Both peak and volume are required in this analysis. Where surcharge storage is significant, or where there is insufficient discharge capability at maximum pool to pass the peak inflow of the SDF, considering all possible operational constraints, a flood hydrograph is required. When there are upstream hazard areas that would be imperiled by fast rising reservoir levels, SDF hydrographs should be routed to ascertain available time for warning and escape. Determination of probable maximum precipitation of SPF or 100 year precipitation, whichever is applicable, and unit hydrographs or runoff models will be required, followed by the determination of the PMF or SPF or 100 year flood. When applicable, conservatively high snow melt runoff rates and appropriate releases from upstream project should be assumed. The maximum water surface elevation and spillway design flood outflow are then determined by routing the inflow hydrograph through the reservoir surcharge storage, assuming starting water surface at the bottom of surcharge storage, or lower when appropriate. For projects where the bottom of surcharge space is not distinct or the flood control storage space (exclusive of surcharge) is appreciable, it may be appropriate to select starting water surface elevations below the top of the flood control storage for routings. Necessary adjustment of reservoir storage capacity due to existing or future sediment or other encroachment may be approximated when accurate determination of deposition is not practicable.

3. Acceptable procedures

Whenever the acceptability of simple or recommended procedures is in question, the advice of competent experts should be sought. Such expertise is generally available in the Central Water Commission, India Meteorological Department and National Institute of Hydrology.

4. Freeboard allowances

The present day practice is to check the freeboard allowance in earth / rockfill dam by Savillie's method which takes into account the effective fetch, reservoir depths, wave generation, wind speed, wave run-up depending upon the roughness and slope of embankment face. For final selection of freeboard, the hazard potential of dam should also be taken into consideration.

2.4. METHODOLOGIES OF DESIGN FLOOD ESTIMATION

The methodologies of design flood estimation for storage projects have changed overtime with increased understanding of flood hydrology. The hydrometeorology approach of postulating a design storm and computing the resulting flood has become the accepted practice for defining the upper limit of expected flood from extreme, meteorological events. Design storm studies for different projects have been carried out by various investigators with widely varying results due to subjectivity involved in the procedures. Till such time the understanding of the physical processes improves, engineer" have to rely on the principle that "If there is a recognized professional standard of care, that standard will generally serve as the minimum legal duty" and they have to take: recourse to codify the procedures and methods based on consensus to achieve some sort of uniformity in the level of safety envisaged and to reduce individual preferences in solutions offered. This recourse is not exclusive to flood hydrology but very common to all spheres of human activity involving safety assurance.

The following approaches are available for estimating design flood.

1. Formula approach
2. Regional approach
3. Statistical approach, commonly known as Flood Frequency Approach.
4. Hydro meteorological approach, commonly known as the Unit Hydrograph Approach.

The detailed methodology to be adopted in a particular case depends upon the data availability. The procedures are indicated in the flow chart given in Figure 2.1.

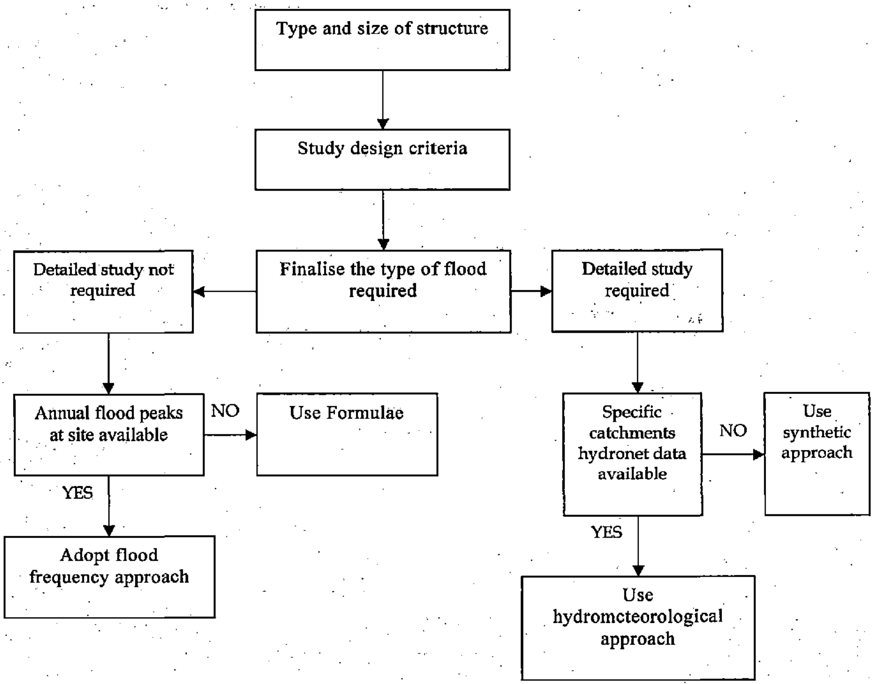


Figure 2.1: Selection of Method

2.5. CRITICAL FLOODS DESIGN CRITERIA-OHIO (USA)

Dam Safety Rules in Ohio State of USA require dams to pass floods through their spillways without endangering the safety of the dam. The magnitude of the design flood is directly related to the classification of the dam - which in turn is related to the dam's downstream hazard and/or the dam's height. Specific guidelines are available for preparing a critical flood engineering analysis. (Source: http://www. dnr.state.oh.us/ odnr/ water/ temp/dartrlsa.html).

The critical flood criteria were developed to make Ohio's Dam Safety Rules more flexible in recognizing that some dams fall outside of the typical parameters used in designing spillway capacity. Specifically, for those circumstances where the size of the dam, its downstream hazard, drainage area, and downstream topography are such that traditional flood design standards do not accurately account for the downstream hazard, critical flood criteria allow for a reduction of up to 60% of the design flood.

A critical flood analysis approved by office means that a reduced design flood is acceptable for the dam. It is important to note that a reduction in the design flood for the dam may increase the risk of failure or damage to the dam. This could result in an economical burden on the dam owner. This risk should be closely considered.

2.6. DESIGN STANDAR OF SPILLWAY CAPACITY IN INDONESIA

In 1994, Dewan Standarisasi Nasional (DSN) published 'Standar Nasional Indonesia Number of SNI 3-3432-1994' which is standard of design flood discharge and outflow capacity of spillway in Indonesia. The standard mentioned condition of constructed dam as follows:

2.6.1. Consequence in Downstream Area Due to Dam failure

Some consequences have to be considered due to lower area condition as follows:

1. High consequence, if there are people, settlements (villages, cities), estates and developing industry have to be protected when a dam gets failure
2. Low consequence, if there are not or only small settlement, nor industry in the lower area.

If there is any protected cultural reservation or natural reservation, than it has to be discussed with relating agency or people

2.6.2. Types and Height of Dam

1. Fill type
    - Low dam with dam height lower than 40 m
    - - Medium dam with dam height between 40 m – 80 m
    - - High dam with dam height higher than 80 m
2. Concrete type

Table 2.6 presents standard criteria for design flood and spillway capacity of dam.

Table 2.6: Criteria for Design Flood and for Spillway Capacity of Dam

Type and Height of Dam	High Consequence		Low Consequence	
	Design Flood	**Overflow Capacity**	**Design Flood**	**Overflow Capacity**
I. Fill Type Dam				
(1) < 40 m (low)	Q1000 and Maximum Allowable Flood (MAF) with standard freeboard	1) To be determined by flood routing. 2) Minimum: 15% of MAF	To be selected which is higher between Q1000 and 0.5 MAF.	1) To be determined by flood routing. 2) Minimum: 15% of Peak of Design Flood.
(2) 40-80 m (medium)	ditto	1) To be determined by flood routing. 2) Minimum: 25% of MAF	ditto	1) To be determined by flood routing. 2) Minimum: 25% of Peak of Design Flood.
(3) > 80 m (high)	ditto	1) To be determined by flood routing. 2) Minimum: 35% of MAF	ditto	1) To be determined by flood routing. 2) Minimum: 35% of Peak of Design Flood.
II. Concrete Type Dam	Q1000	Minimum: 125% Q100	0.5 Q1000	Minimum: 125% x 0.5 Q1000

Source: Dewan Standarisasi Nasional (DSN) published 'Standar Nasional Indonesia Number of SNI 3-3432-1994 (Indonesian National Standard, SNI SNI 3-3432-1994)

2.7. CLIMATE CHANGES AND FLOODS

Floods are the most significant natural hazard causing sufferings to a large number of people and damages to properties year after year. Changes in stream flow and floods have been observed in different parts of the world due to climate changes. Evidences of regional climate change shifting of peak stream flow has shifted back from spring to late winter in large part of eastern Europe, European, Russia, and North America in last decades. Increasing frequency of droughts and floods in some area is related to variations in climate - for example, droughts in Sahel and in northeast southern Brazil, and floods in Colombia and northwest Peru. (Sharma K.D. & P. Singh 2007).

IPCC Report (2001) indicated a likelihood of increased intensity of extreme precipitation over the south Asia region under changed climatic scenarios. The amplitude and frequency of extreme precipitation events is very likely to increase over many areas and the return period for extreme precipitation events are projected to decrease. This would lead to more frequent floods and landslides avalanches, and soil erosion with attendant loss of life, health impacts (e.g., epidemics, infectious diseases, and food poisoning), and property damage, loss to infrastructure and settlements, soil erosion, pollution loads, insurance and agriculture losses, amongst others. (Sharma K.D. & P. Singh 2007).

For the period 1871-1984, Parthasarathy et al. (1987) identified a range of 2-30 flood years (i.e., years when precipitation is at least 26% higher than normal) in the various meteorological sub-divisions in India. In the same period, the range of severe flood years (i.e., precipitation more than 51% higher than normal) was between 1 and 14. According to country's report to the United Nations Framework Convention on Climate Change (UNFCCC), the global climate change is likely to result in severe droughts and floods in India - and have major impacts on human health and food supplies (MOEF, 2004). High flood levels can cause substantial damage to key economic sectors: agriculture, infrastructure and housing. Although floods affect people of all socioeconomic status, the rural and urban poor are hardest hit.

Flash floods are likely to become more frequent in many regions of temperate and tropical Asia in the future. A decrease in return period for extreme precipitation events and the possibility of more frequent floods in parts of India, Nepal, and Bangladesh is projected. (Sharma K.D. & P. Singh 2007). Increased precipitation intensity, particularly during the summer monsoon, could increase flood-prone areas in temperate and tropical Asia. Flood plains, i.e., the lands bordering rivers and streams, are normally dry round the year but get covered with water during floods. Floods can damage buildings or other structures like levees and embankments placed within the flood plains. Climate variability and extreme climate events will generate increased flood, avalanche, landslide, and mud slides damage, soil erosion. Under such conditions there would be increased pressure on government and flood insurance system and disaster relief.

Ramasastri (2006) in the status of Art report on effect of climate change in water resources states that frequency of heavy rainfall (more than 70 mm in 24 hours) during south-west monsoon has shown increasing trend over Andaman and Nicobar islands l Lakshadweep, west coast and some pockets in control and north west India.

2.8. CONCLUSIONS

1. There have been 26 major dam failures in India in post independence period. Further, literature review of flood estimates of 62 large dams shows that reassessed design flood are significantly larger than earlier estimates.

2. Dam failures reported in literature are not a random sample and represent perhaps the worst out of all the existing dams. Hence it may be wrong to assume that all existing dams are unsafe.
3. Hydrology is a data based science. With occurrence of more severe events in a large sample, earlier estimate of design flood are bound to be revised up ward as illustrated through flood reviews study of Hirakud dam, Gandhi Sagar dam and other dams in India.
4. Due to intensive flood plain occupancy (economic development and increase in density of population) in downstream of dam, it is of almost importance to eliminate possibility of dam failure either completely or partially as there will be extensive damage to property and and heavy loss of life in case of dam failure.
5. A critical flood analysis based on dam break study is justified for the important dams. It should be possible to accept the cost of dam safety analysis if the same result in a) acceptance of reduced design flood and acceptance of higher risk of failure or b) acceptance of increased design flood for safety of property and life in downstream flood plain.
6. Physical process of flood formation is not fully understood therefore a degree of subjectivity is involved in modeling and estimation of design storm and design flood. Subjectivity can be minimised by evolving consensus and codifying the criteria, and procedures for estimation of design flood.

CHAPTER III

FLOOD FORECASTING FOR DAM SAFETY

The purpose of this chapter is to examine the factors influencing choice of forecasting method and sources of error. These two are important considerations for safety of existing dams.

3.1. METHOD FOR FLOOD FORECASTING

On the basis of the analytical approach for development of the forecasting model, the methods of flood forecasting can be classified as:

1) Methods based on statistical approach, and
2) Methods based on mechanism of formation and propagation of flood.

Based on the data used for formulation of forecast, the various methods of flood forecasting can be classified in three major groups:

1) Forecast on the basis of stage-discharge data at various points along river (correlation, channel routing);
2) Rainfall-runoff methods; and
3) Meteorological methods.

The detailed classification is illustrated in Figure 3.1.

In the sub-basin affected by flash floods, the only effective method of flood forecast will be rainfall- runoff method for which the basic data required is precipitation. This may also be helpful in increasing the warning time of forecasting for the lower reaches of the river as the forecast values of river stage in the upstream, could be used for forecasting down stream stages. Then, it is ideally suited for inflow forecasting into reservoirs and lakes.

For a small catchment where the concentration time is very less, even the use of rainfall data in forecast formulation will not help in getting sufficient warning time. In such cases, hydrometeorological methods are used.

3.1.1. Factors Governing Adoption of Method for Forecasting

The various factors which govern the adoption of a particular method of forecasting are as follows:

1) Physiographic factors;

2) Data availability;

3) Warning time required;

4) Purpose of the forecast.

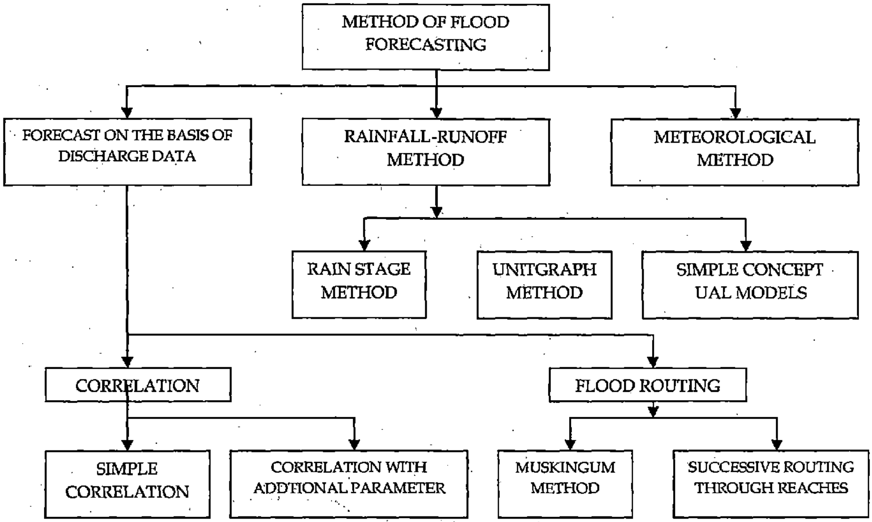


Figure 3.1: Flood Forecasting - Hydrometeorological Approach

1. Physiographic Factors

By physiographic factors, we mean basin and channel characteristics of a catchments. These characteristics will help in identifying the method which will be most suitable for the particular point. In general it may be concluded that gauge to gauge relations including simple correlations, multiple correlation and coaxial diagrams etc. are useful in long, slow-flowing rivers. Rainfall-runoff model is very useful for flood forecasting in head water reaches where use of gauge to gauge relation is very difficult, if not impracticable. This is also very effective tool in formulations of Flash Flood Forecasts for the flood prone tributaries as also for increasing the warning time in medium length rivers if appropriately used in conjunction with stream flow routing or gauge to gauge relations for lower reaches.

2. Data Availability

For the establishment of gauge to gauge relationship between two stations, only gauge data at different time for• the two stations are required. For development of a unit hydrograph at a point the data required will consist of:

1) Gauge data at specified duration.
2) Sufficient number of discharge observations for development of stage discharge curve.
3) Rainfall data from sufficiently good number of rain gauge stations of specified duration.

On the other hand when a catchment model is to be developed, a large number of hydrological and hydrometeorological parameters are to be defined which need many data. For example the data required for development of SSARR model are:

1) Several years of precipitation, temperature and discharge data.
2) Basin area, elevation, location and distribution of hydrometeorological stations.
3) Information regarding soil water infiltration curve, impervious areas, percent slopes, type and extent of vegetation cover etc.

3. Warning Time Required

Technique like routing/ rainfall-runoff model for upper catchments should be adopted to increase the warning time. For a very small and flashy river, even the rainfall-runoff method does not provide sufficient warning time. In such cases meteorological methods can be used for flash flood guidance even with a lack of accuracy.

4. Purpose of the Forecast

For flood purposes, the main requirement is water stage. But for reservoir regulation purposes, the total volume of the incoming flood as well as its time distribution is required and hence for inflow forecasting we have to adopt such a method which will produce the above two information.

3.2. PRECISION IN OBSERVATIONS-CWC PRACTICE

It is well known that the hydrological observation stations require considerable initial investment as well as substantial running and maintenance cost and therefore it is essential to have a very critical evaluation of the requirement and plan the network of data observation stations in such a way that all the essential information are available at the minimum cost.

1) With respect to actual observations the WMO Technical Regulations use the term "precision of observation or of reading" which is defined as the smallest

unit of division on a scale of measurement from which a reading, either directly or by estimation, is possible.

Table 3.1: Desirable Precision of Observation and Frequency of Measurement for Hydrological Forecasting

Element	Precision	Reporting interval	Measure by automatic land station
Precipitation - Total	± 2 mm below 40 mm	6 hours[3]	Yes
Amount and form[2]	± 5% above 40 mm		Yes
River stage	± 0.01 mm	6 hours[5]	Yes
Lake level	± 0.01 m	Daily	Yes
Soil moisture	± 10% field capacity	Weekly	Yes
Frost depth	± 2 cm below 10 cm ± 20% above 10 cm	Daily	Yes
Water equivalent of Snow on ground	± 2 mm below 20 mm ± 20% above 10 cm	Daily	Yes
Depth of snow cover	± 2 cm below 20 cm ± 10% above 20 cm	Daily	Yes
Density of snow cover	± 10%	Daily	-
Water temperature[6]	± 0.1°C in 0-4°C — range	Daily	Yes
(rivers and lakes)	Otherwise ± 1°C	Daily	Yes
Surface temperature snow	± 1°C	Daily	Yes
Temperature profiles (Snow and lakes)	± 1°C	Daily	Yes
River and lake ice	± 0.02 m below 0.2 m ± 10% above 0.2 m	Daily	-
Water level (in wells)	± 0.02 m	Weekly	Yes
Net radiation	± 0.4 MJm^{-2}/day below 8 MJm^{-2}/day ± 5% above 3MJm^{-2}/day	Daily	Yes
Air temperature	± 0.1°C	6 hours	Yes
Wet. bulb temperature	± 0.1°C	6 hours	Yes
Wind-movement	± 10%	6 hours	Yes
Pan evaporation	± 0.5 mm	Daily	Yes

Source : CWC (1989) : Manual on Flood Forecasting, Central Water Commission, Govt. of India New Delhi March 1989.

2) It may be necessary to distinguish solid and liquid forms of precipitation.
3) Varies from one hour to one day, depending on river response. Event reporting for example, after 2 mm of rain required -for flash flood forecasts.
4) Depends on sensitivity of stage discharge relationship to stage change and can be ± 1 mm accuracy. If possible an accuracy characterized by a relative standard deviation of ± 5 per cent should be arrived at.

5) See note 3. Event reporting may be appropriate for flash flood forecasts.

6) Hourly reporting with ± 0.3°C for ice forecasting.

3.3. PRECISION IN STREAM FLOW MEASUREMENT

There are two types of errors namely random and systematic. Random errors are caused purely on chance fluctuation and are errors involved in the measurement of depth and width. The systematic errors are associated with a particular instrument or on the methodology of stream flow measurement.

It is necessary to have an idea of the errors involved in stream flow measurements by Velocity-Area method so as to guard against them and for taking corrective steps in the observation. CWC (1989) has provided following guidelines on acceptable errors.

Errors in Measuring Width (X_b)

This type of error is usually not much and a value of ± 0.5% may be taken.

1. Errors due to the Choice of Number of Verticals (X_m)

m (Number of Verticals)	X M
8	± 5 percent
15	± 3 percent
25	± 2 percent
50	± 1 percent

2. Errors in Measuring Depth (X_d)

This type of error can be reduced by repeated measurement. However, a value of ± 2.5% may be taken for ibis type of errors.

3. Errors on account of duration of exposure of Current Meter (X_f)

For an exposure time of 40 secs. The recommended value is ± 6 percent.

4. Errors due to the choice of number of point in a vertical (X_o)

Method	X_o
Velocity distribution	± 0.5 percent
2 point	± 3.0 percent
1 point (.6d)	± 3.5 percent

5. The Overall Random Errors (X_q)

$$X_q = \pm X_m^2 + \frac{1}{m}\left(X_b^2 + X_d^2 + X_v^2\right)$$

Where, $X_v = \pm\left(\frac{X_f}{P} + X_o^2\right)$

And P = number of point in the vertical where velocities have been observed.

3.4. ERROS IN FLOOD FORESCAST

The three main sources of error in forecast are:-

1) error at source i.e error in observed data (random error)
2) error during transmission , and
3) the computational error.

1) Errors at Source

The errors at source may be either the instrumental error (systematic error) or the observational errors (random error) or the recording error. The observational and the recording errors can be avoided to a great extent by proper supervision and checks at different levels at frequent intervals. The instrumental errors may be classified in two groups, viz; (i) errors of sudden or emergent nature, and (ii) errors which creep slowly over long time. The errors of the first type are because of sudden problems with the equipments e.g., washing away of gauge post and error during fixation of new gauge posts by staff members not fully trained for the job. A careful processing of data might reveal such errors. Besides, arrangements are generally made for such situations. For example, gauge marks are painted on nearby permanent structures (such as bridge pier, steps of the ghat etc.) near the proper gauge sites.

The error of second type is rather difficult to be noticed during the routine data processing. Such errors in the gauge data, for example, may be because of slow settlement of gauge posts in sandy beds or in discharge data, due to deterioration in rating of the current meter, these errors can be detected by frequent checking.

2) Errors during Transmission

The errors during the transmission can be minimized by adhering to the procedure laid down for transmission of data. Further, any error noticed during the processing and the analysis should be immediately checked and rectified. Proper training of the personnels engaged in transmission of data is a must.

3) Computational Errors

The computational errors are of two types: the errors associated with the computational instruments such as calculators etc., and the human error. Such errors are quite possible and to avoid them, it is necessary, to check the formulated forecasts at two or more levels preferably by, using different approaches. For example a forecast formulated with the help of a co-axial diagram can be rechecked with the help of a mathematical equation representing the co-axial diagram. Further it will be desirable to have another check by some other technique. A water profile diagram - a very basic and rather crude tool for forecasting, may also be used before the issue of the formulated forecast. Such checks will considerably reduce the possibility of computational errors.

4) Unexpected Situations

Formulation of forecast may be handicapped because of:

1) Non-availability of all the desired data/ information. and
2) Deviation from the defined boundary conditions.

1) Non-availability of All Required Data in Time

The non-availability of the desired information at the time of forecast formulation is a common problem. This generally results in delay in formulation of forecast which will cause loss of precious time which is not desirable.

Therefore, it is necessary that the alternative methods/ techniques are available so that the forecast could be formulated using the available limited data with known degree of accuracy.

2) Deviation from Defined Boundary Conditions:

A not so common but very important phenomenon is the situation when there is deviation from the defined boundary conditions of the model. Some of the examples of such situations are:

(a) Breach in the flood embankments of river.

Under such situations, the commonly used model for forecast do not work any more and the necessary information about the condition of the breaches etc. are to be collected round-the clock and duly considered while formulating the forecast.

(b) Rain of very high intensity at locations in between the base and the forecasting i stations.

This becomes very important when the intermediate catchment is considerable and the same is not incorporated with due to weightage in the model.

(c) Unexpected regulation of the control structures.

A sudden closure/opening of gates without advance information to the forecasting centre may adversely affect the forecast performance.

3.5. FORECAST EVALUATION

A forecast is considered to be accurate if the difference between the forecast level and the corresponding observed level is within a permissible extent of deviation.

3.5.1. Central Water Commission (India) – Criteria for Forecast Performance

A simple and common criteria is being adopted presently in CWC to evaluate the forecast performance. In the case of river stage, forecast of ± 15 cm. variation between forecast level and actual level is allowed and similarly 20% of inflow is allowed in case of inflow forecasts. In real Life, pattern of peak flood could differ from river to river. In river which get flash floods, the travel time, is generally very short viz., a few hours only. In some of the forecast sites like Banda on Ken (India – Ganga Basin), the actual rate of rise in flood level has been found to be very rapid (3.5 m. in 24 hours) (CWC 1989). Hence in such cases the margin of 15 till may be too low and not be justified. In some other sites like the Ganga main stem downstream of Allahabad the rate of rise is 5-10 cm. in 24 hours and as such 15 cm. may be a liberal and high figure to be allowed for variation. Similar consideration regarding volume of inflow can lead to different yard sticks for different inflow forecasting sites.

3.5.2. Evaluation of forecasts by simple methods

1) **Criteria for verification :**The commonly used criteria for forecast verification are as follows:-

   i) Relative Error

   $R = (Y_0 - Y_f)$

   ii) Absolute Error

   $A = I\ (Y_0 - Y_f)\ I$

   iii) Efficiency

$$E = 1 - \frac{(Y_o - Y_f)^2}{(Y_o - Y_n)^2}$$

Where Y_0 = Actual level at $(N+T)^{th}$ time

Yf =Forecast level at $(N+T)^{th}$ hour

Y =Nth hour level on the basis of which the forecast has been formulated

E = A measure of efficiency.

Examples:

Example-I

i)	Water level at'N'th hour at forecasting station	44.38 m
ii)	Predicted water level for $(N+T)^{th}$ hour	
	(T = Travel time in hours)	45.50 m

iii) Water level actually observed 44.78 m

iv) Difference between actual and forecast level (iii-ii) (-) 0.72 m

As per present practice the evaluation is considered with respect to the level, which is minus 72 cm in the present case. The difference is beyond the limit of ±15 cm. The deviation is, therefore, unacceptable.

Example- II

iv) Water level at'N'th hour at forecasting station 46.78 m

v) Predicted water level for (N +T)th hour 46.76 m

vi) Water level actually observed 46.77 m

vii) Difference between actual and forecast level (iii-ii) (+) 001 m

Example-III

i) Level/capacity at Nth hour at forecasting site 99.35m 5360.70 MCM

ii) Predicted inflow upto $(N+T)^{th}$ hour at forecasting site 200 MCM

iii) Level/capacity at $(N+T)^{th}$ hour at forecasting site 99.93m 5592.88 MCM

iv) Actual storage received in the Reservoir during 'T' hours. 232.18 MCM

v) Actual outflow from the Reservoir during 'T' hours. 4.16 MCM

vi) Net inflow received in the Reservoir during 'T' hours 236.34 MCM

vii) Difference between actual inflow & Forecasted inflow (vi-ii) (+) 36.34 MCM

viii) Error in inflow forecast $\frac{36.34}{236.34} \times 100 = 15.38\%$

(Actual inflow is more than forecast inflow by 15.38% but is within ± 20% limit of accuracy).

CHAPTER IV

ANALYSIS OF FACTORS AFFECTING ACCURACY OF HYDROMETEOROLOGICAL APPROACH

4.1. HYDROMETEORLOGICAL APPROACH

The various steps involved in the method are indicated by flow chart given in Figure 4.1. The catchments response function used can be either a lumped system model or a distributed lumped system model. In the former, a unit hydrograph is assumed to represent the entire catchments area. In the distributed model, the catchment is divided into smaller sub-regions, and the unit hydrographs of each sub-region applied with channel and/or reservoir routing will define the catchments response. The main advantage of the hydro meteorological approach is that it gives a complete flood hydrograph and this allows making a realistic determination of its moderating effect while passing through a reservoir or a river reach.

This approach however is subjected to certain limitations such as:

1) Requirement of long term hydro meteorological data for estimation of design storm parameters.
2) The knowledge of rainfall process as available today has severe limitations and therefore, physical modeling of rainfall to compute PMP is still not attempted.
3) Maximization of historical storms for possible maximum favorable conditions is presently done on the basis of surface dew point, data. Surface dew point data may not strictly represent moisture availability in the upper atmosphere.
4) Availability of SRRG data for historical storms is too poor.
5) Many of the assumptions in the UG theory are not satisfied in practice.
6) Many times, data of good quality and adequate quantity is not available for derivation of UG.

Nevertheless, the hydrometeorological approach has been found to be a useful tool in design flood studies. Hydrometeorological approach preferably based on site specific information is suggested for the estimation of design flood of intermediate and large dams, especially when the storage has a significant effect on modifying the design flood hydrograph as it flows through the reservoir.

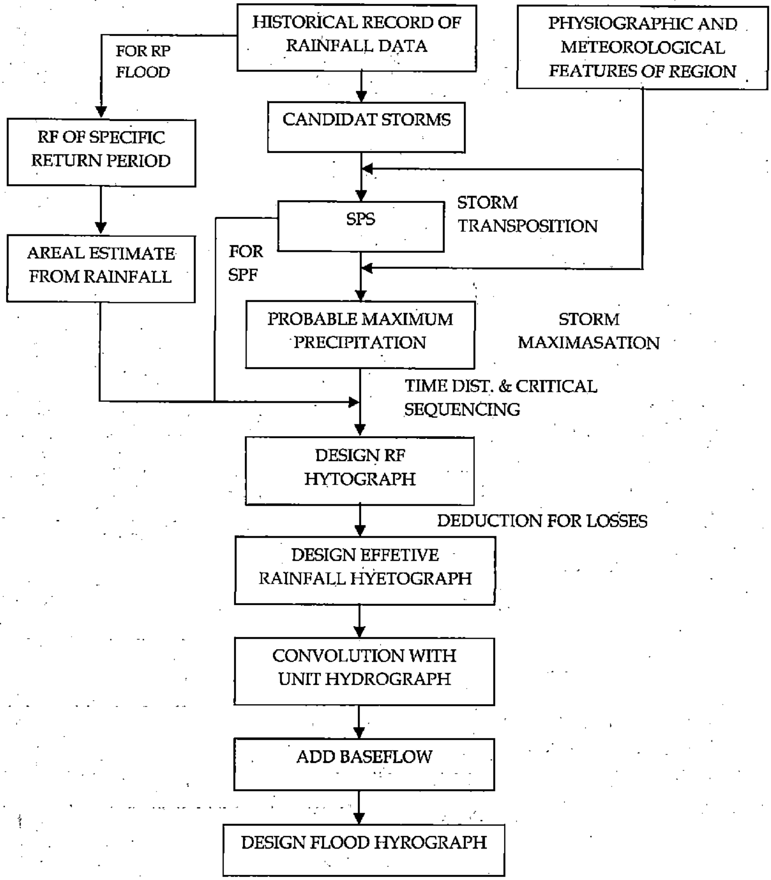


Figure 4.1: Hydrometeorological Approach

4.2. FACTORS

Unlike flood frequency approach which is adopted for estimating flood magnitude of a desired return period, the hydrometeorological approach can be used to find flood hydrograph for a specified storm (standard project storm, probable maximum precipitation, storm rainfall of a given return period, or real time occurrence of storm in catchment). The hydrometeorological approach is depicted in Figure 4.1. There are several factors which affect the accuracy of flood (design flood or flood forecast) estimated using hydrometeorological approach.

1) Design storm: Assumption regarding uniform distribution over catchments and constant rate of rainfall over the storm duration.
2) Unit Hydrograph: Theory and practical limitations, assumption regarding linear behavior of catchments.
3) Infiltration loss and baseflow: i) arbitrariness in procedure to estimate loss rate to get excess rainfall and, ii) Base flow separation to get direct runoff hydrograph
4) Stationary catchments: validity of considering the catchments characteristics to be stationary (time invariant).

4.3. THE BAITARNI RIVER BASIN

4.3.1. River System

The Baitarani basin (Orissa State of India) covering 12.789 km^2 is roughly circular in shape in upper portion and elongated in lower portion. The basin as a whole has maximum length of 423 km in the North-west to South-east direction and a maximum width of 193 km in North-east to Southwest direction. The Baitarani river rises in the hill ranges of Keonjhar district at an elevation of about 900 m. The river flows initially in a generally Northerly and North-easterly direction for a total length of 80 km up to Jainthgarh. Thereafter, it takes an almost right angle turn and flows in a generally South-easterly direction up to Jajpur for a length of 194 km changing direction again towards the East, the river continues to flow for another 81 km and joins the Bay of Bengal near Palmyras Point. The Salandi joins the Baitarani from the left at the 314 km of its run, North-west of Rajkalika. The Matai, another left bank tributary, joins the Baitarani at the 343 km of the latter's run near Dhamra. The Baitarani in its lower reaches is known by the name of Dhamra. The total length of the river from the head to its outfall into the sea is 355 km and lies entirely in the state of Orissa.

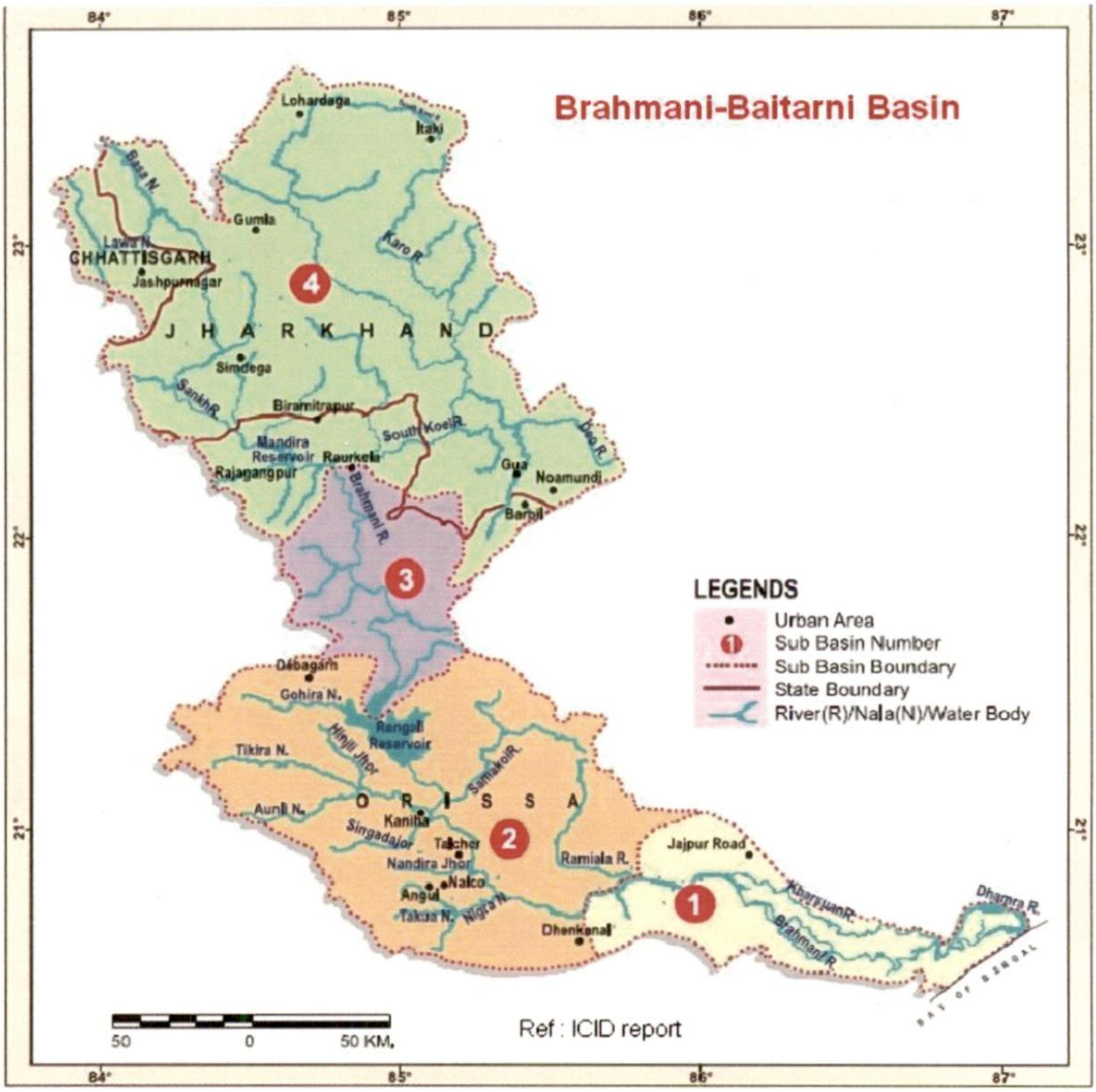


Figure 4.2: Catchment's Area of Baitarani River Basin

4.4. UNIT HYDROGRAPH FOR BATARNI RIVER BASIN (CA 8370 sq.Km) UP TO ANANDPUR

Following methods have been used to derive UH. These are available in CWC report (CWC 1989).

1) UH from isolated flood: Isolated flood hydrograph observed during 13 August 1976 to 15 August 1976.
2) Nash model for IUH: Storm rainfall and flood hydrograph observed during 26 September 1975 to 28 September 1975.
3) UH by Collin's method.
4) Average UH based on the three UH derived using three different methods.
5) 1 hour synthetic UH for the Mahanadi subzone (CWC 1982)

- Length of main stream from farthest point up to outlet at Anandpur, L= 144 km.
- Length of main stream from a point on main stream nearest to centroid and up to outlet at Anandpur, Lc= 70 km.
- Slope, S= 4.27 m/km.
- tp (hrs) = $1.97\ (L \bullet L_c/\sqrt{S})^{0.24}$ = 15.12 hrs, say 15.5 hrs.
- top (hrs) = tp + 0.5 hr = 16 hrs
- peak discharge per unit area, $q_p = 1.12\ tp^{-0.66} = 0.18348$ cumec/sq.km.
- $W_{50} = 2.195\ (q_p)^{-1.008} = 12.126253$ hrs
- $W_{75} = 1.221\ (q_p)^{-0.95} = 6.113593$ hrs
- $WR_{50} = 0.995\ (q_p)^{-0.94} = 4.898$ hrs
- $WR_{75} = 0.532\ (q_p)^{-0.93} = 2.5749$ hrs
- $T_b = 5.72\ (tp)^{0.77} = 47.2$ hrs
- $Qp = q_p \times A = 157.246$, say 157 cumec.

Table 5.1 shows the different UHs derived for Baitarani River basin. The Unit hydrographs are for 1 mm excess rainfall in three hours.

Table 4.1: Flood Hydrograph by Applying Different 3 Hrs Unit Hydrograph (Rainfall Non Uniform)

Time	3 hrs U.H Isolated Event (cumecs)	Flood Hydrograph	3 hrs U.H from IUH (cumecs)	Flood Hydrograph	3 hrs U.H from Collin's Method (cumec)	Flood Hydrograph	3 hrs U.H Average (cumecs)	Flood Hydrograph	3 hrs U.H (Synthetic)	Flood Hydrograph
1	2	3	4	5	6	7	8	9	10	11
0	0	300	0	300	0	300	0	300	0	300
3	8	329.28	8	328.56	15	353.55	8	328.56	7	324.99
6	33	419.36	47	469.99	65	536.17	34	423.58	21	376.89
9	120	781.33	101	715.33	118	817.58	102	715.33	42	491.14
12	145	1033.59	128	1042	134	1172.25	130	981.39	77	704.99
15	155	1600.59	132	1431.43	125	1548.59	143	1457.96	134	1072.08
18	110	1790.72	113	1730.1	107	1831.48	119	1718.3	148	1406.96
21	70	**2056.12**	88	**1908.91**	80	**1920.08**	82	**1948.65**	112	1709.83
24	52	1885.36	65	1846.23	56	1799.22	59	1877.66	79	**1878.33**
27	32	1580.22	45	1636.42	37	1531.65	40	1635.5	54	1833.06
30	28	1250.28	26	1338.99	24	1246.67	28	1330.97	38	1622.45
33	18	914.89	17	1065.13	16	974.42	20	1021.86	28	1276.04
36	12	761.14	10	815.31	9	753.3	14	809.6	22	999.7
39	6	596.57	7	645.69	5	597.19	8	648.14	16	796.91
42	3	516.8	4	506.69	2	481.87	4	537.64	10	658.74
45	0	426.7	2	432.48	0	408.27	2	455.08	5	555.59
48	0	375.79	0	375.21		357.23	0	392.54	0	469
51		331.23		346.08		324.98		351.28		408.19
54		313.56		320.41		309.43		320.87		350.4
57		301.23		309.01		300.85		309.04		322.48
60		300.09		300.82		300.06		300.82		302.05
63		300.01		300.06		300		300.06		300.14
66		300		300				300		300.01
										300.00

Table 4.2: Flood Hydrograph by Applying Different 3 Hrs Unit Hydrograph (Rainfall Uniform)

Time	3 hrs U.H Isolated Event (cumecs)	Flood Hydrograph	3 hrs U.H from IUH (cumecs)	Flood Hydrograph	3 hrs U.H from Collin's Method (cumecs)	Flood Hydrograph	3 hrs U.H Average (cumecs)	Flood Hydrograph	3 hrs U.H (Synthetic) (cumecs)	Flood Hydrograph
1	2	3	*4*	5	*6*	7	*8*	9	*10*	*11*
0	0	300	0	300	0	300	0	300	0	300
3	8	329.28	8	328.56	15	353.55	8	328.56	7	324.99
6	33	471.49	47	520.83	65	631.5	34	474.42	21	421.38
9	120	983	101	1007.38	118	1218.21	102	924.76	42	618.78
12	145	1759.08	128	1633.39	134	1846.72	130	1589.96	77	942.35
15	155	2344.19	132	2064.28	125	**2153.87**	143	2121.21	134	1468.82
18	110	**2361.27**	113	**2141.77**	107	2100.34	119	**2244.92**	148	2053.09
21	70	1965.29	88	1944.15	80	1845.01	82	2010.91	112	**2270.23**
24	52	1437.32	65	1612.69	56	1501.12	59	1577.89	79	1975.05
27	32	1069.13	45	1278.8	37	1155.37	40	1194.77	54	1507.88
30	28	840.87	26	*977.11*	24	877.39	28	924.76	38	1142.47
33	18	688.9	17	731.07	16	679.04	20	733.04	28	892.9
36	12	585.03	10	562.81	9	543.81	14	605.78	22	735.06
39	6	480.18	7	466.09	5	447.92	8	509.38	16	627.65
42	3	403.29	4	404.69	2	379.89	4	428.92	10	538.58
45	0	346.29	2	364.46	0	335.26	2	368.86	5	454.55
48	0	313.2	0	330.86		308.8	0	330.86	0	377.15
51		300		308.8		300		308.8		322
54		300		300		300		300		300

4.5. FLOOD ESTIMATION USING DIFFERENT UH

1. Storm rainfall: two cases are considered.
   - Distribution of excess rainfall is non uniform

Time (hrs)	Station Rainfall (mm)		
	A	B	C
0 – 3	15	0	0
3 – 6	0	14	2.7
6 - 9	0	0	12.3

   - Excess rainfall is assumed to be uniform over entire catchments

Time (hrs)	Station Rainfall (mm)
0 – 3	3.57
3 – 6	6.63
6 - 9	4.4

Thiessen weight of A, B, C are 0.238, 0.405, and 0.357. For example, during 3 to 6 hr the rainfall is 0.238 x 0 + 0.405 x 14 +0.357 x 2.7 = 6.63 mm.

2. Flood Hydrograph.

Base flow has been taken as 300 m3/s constant through out duration of flood hydrograph. Table 5.1 shows the UH and flood hydrographs by applying different 3 hr UH when rainfall is non uniform. Table 5.2 shows the UH and flood hydrographs by applying different 3 hr UH when rainfall is assumed to be uniform.

4.6. ERROR IN FLOOD FORECASTING DUE TO USE OF DIFFERENT UH

Analyses in Table 5.1 and Table 5.2 have been used to compare:

- Estimated peak flood with observed peak
- Estimated time to peak with observed time to peak.

Comparison has been made both for non uniform rainfall (actual) and assumption of uniform rainfall has also been work out.

Table 4.3: Error in Forecasting of Peak Flood and Time to Peak due to use of Different UH when Rainfall is Non Uniform

UH used	Q peak	Time to peak	Observed Q peak	Observed time to peak	Error in Peak Discharge Estimation	Error in Time to peak Estimation
ISOLATED	2056.12	21	2180.00	18	5.68	-16.67
IUH	1908.91	21	2180.00	18	12.44	-16.67
COLLIN'S	1920.08	21	2180.00	18	11.92	-16.67
AVERAGE	1948.65	21	2180.00	18	10.61	-16.67
SYNTHETIC	1878.33	24	2180.00	18	13.84	-33.33

Table 4.4: Error in Forecasting of Peak Flood and Time to Peak due to use of Different UH when Rainfall is Uniform

UH used	Q peak	Time to peak	Observed Q peak	Observed time to peak	Error in Peak Discharge Estimation	Error in Time to peak Estimation
ISOLATED	2361.27	18	2180.00	18	-8.32	0.00
IUH	2141.77	18	2180.00	18	1.75	0.00
COLLIN'S	2153.87	15	2180.00	18	1.20	16.67
AVERAGE	2244.92	18	2180.00	18	-2.98	0.00
SYNTHETIC	2270.23	21	2180.00	18	-4.14	-16.67

Table 4.5: Error in Forecasting of Peak Flood and Time to Peak due to Assumption of Uniform Rainfall over the Catchment

When non uniform rainfall distribution is considered		When rainfall is assumed to be uniform	
Qp (cumecs)	Tp (hrs)	Qp (cumecs)	Tp (hrs)
1948.65	21	2244.92	18
Error in Qp Estimation (%)	Error in Tp Estimation (%)	Error in Qp Estimation (%)	Error in Tp Estimation (%)
10.61	-16.67	-2.98	0

Observed flood was 2180 cumec and observed time to peak was 18 hours.

4.7. MEASURES OF FORECAST ERROR

Forecast accuracy is best assessed by retrospective comparison of forecast actually made or that might have been made, and the values observed during the forecast period. Maidment (1993) has suggested the following parameters to estimate forecast accuracy. Let Q_f (i) be the forecasted stream flow and $Q_o(i)$ be the observed stream flow during the same period and define M_f and M_0, the means of the forecast and observations for the same period, as follows:

$$M_f = \frac{1}{n}\sum_{i=1}^{n} Q_f(i)$$

$$M_r = \frac{1}{n}\sum_{i=1}^{n} Q_o(i)$$

where, n is the total number of values,

The following are widely used measures of forecast errors:

Bias: $B = M_f - M_o$ (4.1)

Mean squared error: $MSE = \frac{1}{n}\sum_{i=1}^{n}\left[Q_f(i) - Q_o(i)\right]^2$ (4.2)

Root mean squared error: $\mathrm{RMSE} = (\mathrm{MSE})^{0.5}$ (4.3)

Variance: $\mathrm{V} = \mathrm{MSE} - \mathrm{B}^2$ (4.4)

Relative Bias: $\mathrm{RB} = \frac{\mathrm{B}}{\mathrm{M_o}}$ (4.5)

Mean absolute error: $\mathrm{MAE} = \frac{1}{n}\sum_{i=1}^{n}\left[Q_f(i) - Q_o(i)\right]$ (4.6)

Relative mean absolute error: $\mathrm{RMAE} = \frac{\mathrm{MAE}}{\mathrm{M_o}}$ (4.7)

Forecast efficiency: $\mathrm{E} = 1 - \frac{\mathrm{MAE}}{\mathrm{V}}$ (4.8)

R squared: $\mathrm{R}^2 = \left[\frac{\frac{1}{\mathrm{n}}\sum_{i=1}^{n}\mathrm{Q_o}\,(\mathrm{i})\,\mathrm{Q_f}(\mathrm{i}) - \mathrm{M_o M_f}}{\left(\frac{1}{\mathrm{n}}\sum_{i=1}^{n}\mathrm{Q_o^2} - \mathrm{M_o^2}\right)\left(\frac{1}{\mathrm{n}}\sum_{i=1}^{n}\mathrm{Q_f^2} - \mathrm{M_f^2}\right)}\right]$ (4.9)

Bias and relative bias are measures of systematic error in the forecast, that is over a number of events, they measure the degree to which the forecast is consistently above or below the actual value. Variance is a measure of the variability, or scatter, of a number of forecasts about the true value, and is therefore a measure of the random error. Mean square error, root mean square error, mean absolute error, relative mean absolute error, and forecast efficiency are all measures that incorporate both systematic and random errors. A perfect forecast exists only if both the bias and the variance are zero, which occurs only when all estimated values are identical to the observations R2 is the square of the correlation coefficient between the reference value and estimated value. Although R2 is a widely used measure of forecast accuracy, care must be taken if appreciable bias is present, since W evaluates the accuracy of a forecast with respect to random error only. The highest value of R2, 1.0, can be achieved for cases where there is a constant bias in forecast; that is, the estimated value is equal to the reference value plus or minus a constant. For this reason, instead of using R2, forecast accuracy is better assessed by using the bias and the variance, or the bias and the mean absolute error MAE or RMAE is preferred to MSE because, when compared to squared error measures, absolute error measures are less dominated by a small number of large errors, and are thus a more reliable indicator of typical error magnitudes.

In the following sections, reliability in terms of above mentioned parameters has been computed for following:

i) Collin's UH (complete shape, rising portion only) with reference to average UH (from Isolated flood, IUH and Synthetic UH)

ii) Synthetic UH (complete shape, rising portion only) with reference to average UH (from Isolated flood, IUH and Collin's UH)

4.8. RELIABILITY ANALYSIS OF UH AND FORECASTED FLOOD

Three hour Unit Hydrograph using following methods have been derived for Baitarani Basin in earlier part in this Chapter.

1. Unit Hydrograph based on observed flood with isolated peak.
2. Unit Hydrograph based on Nash model for IUH
3. Synthetic UH based on Snyder's model.
4. UH based on Collin's method.

For reliability analysis of the Collin's Unit Hydrograph, Average Hydrograph based on single isolated flood, Nash model and Snyder's model is taken as reference Unit Hydrograph.

For reliability analysis of the Synthetic Unit Hydrograph, Average Unit Hydrograph based on isolated flood, Nash Model and Collin's model is considered as reference Unit Hydrograph.

Reliability analysis has been carried out separately for:

1. Complete Unit Hydrograph shape and
2. Rising portion only.

Reliability of rising portion only is important in forecasting of rise in water level only at the forecasting site such as for flood embankment, town situated on river bank. Where as for reservoir complete hydrograph has to be forecasted.

Table 4.6: Reliability Analysis of Collin's Unit Hydrograph (for complete UH)

No.	Time (hr)	U.H from Isolated (Cumecs)	U.H from IUH (cumecs)	U.H from Synthetic	U.H from Average (Observed) (Cumecs)	U.H from Collin's Method (Cumecs)	(U_f-U_o)	$(U_f-U_o)^2$
1	0	0	0	0	0	0	0	0
2	3	8	8	7	14	15	1	1
3	6	33	47	21	64	65	1	1
4	9	120	101	42	117	118	1	1
5	12	145	128	77	133	134	1	1
6	15	155	132	134	145	125	-20	400
7	18	110	113	148	104	107	3	9
8	21	70	88	112	77	80	3	9
9	24	52	65	79	53	56	3	9
10	27	32	45	54	35	37	2	4
11	30	28	26	38	22	24	2	4
12	33	18	17	28	14	16	2	4
13	36	12	10	22	7	9	2	4
14	39	6	7	16	4	5	1	1
15	42	3	4	10	3	2	-1	1
16	45	0	2	5	1	0	-1	1
17	48	0	0	0	0	0	0	0
Σ					793	793		450

1. Mean of forecasts: $M_f = \frac{1}{n}\sum_{i=1}^{n} U_f(i) = \frac{793}{17}$ =61.00

2. Mean of observations: $M_o = \frac{1}{n}\sum_{i=1}^{n} U_o(i) = \frac{793}{17}$ =61.00

3. Bias: $B = M_f - M_o = 61.00 - 61.00 = 0.00$

4. Mean squared error: $MSE = \frac{1}{n}\sum_{i=1}^{n}\left[U_f(i) - U_o(i)\right]^2$ = 26.471

5. Variance: $V = MSE - B^2 = 26.471 - (0.000)^2$ =26.471

6. R squared: $$R^2 = \left[\frac{\frac{1}{n}\sum_{i=1}^{n} U_o(i)\, U_f(i) - M_o M_f}{\left(\frac{1}{n}\sum_{i=1}^{n} U_o^2 - M_o^2\right)\left(\frac{1}{n}\sum_{i=1}^{n} U_f^2 - M_f^2\right)}\right] = 0.990$$

Bias is zero, but variance is high. Coefficient of determination is good.

Table 4.7: Reliability Analysis of Collin's Unit Hydrograph (for rising portion only)

No.	Time (hr)	U.H from Isolated (Cumecs)	U.H from IUH (cumecs)	U.H from Synthetic	U.H from Average (Observed) (Cumecs)	U.H from Collin's Method (Cumecs)	(Uf-Uo)	(Uf-Uo)2
1	0	0	0	0	0	0	0	0
2	3	8	8	7	14	15	1	1
3	6	33	47	21	64	65	1	1
4	9	120	101	42	117	118	1	1
5	12	145	128	77	133	134	1	1
Σ					328	332		4

1. Mean of forecasts: $M_f = \frac{1}{n}\sum_{i=1}^{n} U_f(i) = \frac{332.00}{17}$ =55.333

2. Mean of observations: $M_o = \frac{1}{n}\sum_{i=1}^{n} U_o(i) = \frac{328.00}{17}$ =54.667

3. Bias: $B = M_f - M_o = 55.333 - 54.667$ = 0.667

4. Mean squared error: $MSE = \frac{1}{n}\sum_{i=1}^{n}\left[U_f(i) - U_o(i)\right]^2$ = 0.667

5. Variance: $V = MSE - B^2 = 0.667 - (0.667)^2$ =0.222

6. R squared: $R^2 = \left[\frac{\frac{1}{n}\sum_{i=1}^{n} U_o(i)\,U_f(i) - M_o M_f}{\left(\frac{1}{n}\sum_{i=1}^{n} U_o^2 - M_o^2\right)\left(\frac{1}{n}\sum_{i=1}^{n} U_f^2 - M_f^2\right)}\right]$ = 1.000

Compared to zero Bias (B) with regard to complete Unit Hydrograph, the Bias in rising portion is 0.66 however, Variance is significantly less (0.222) and Coefficient of Determination is 1.0. From the analysis it is concluded that Collin's Unit Hydrograph can be used for flood forecasting at a town on river bank where only size of water level need to be forecast.

Table 4.8: Reliability Analysis of Synthetic Unit Hydrograph (for complete UH)

No.	Time (hr)	U.H from Isolated (Cumecs)	U.H from IUH (cumecs)	U.H from Collin's Method (Cumecs)	U.H from Average (Observed) (Cumecs)	U.H from Synthetic (Estimated) (Cumecs)	$(U_f\text{-}U_o)$	$(U_f\text{-}U_o)^2$
1	0	0	0	0	0	0	0	0
2	3	8	8	15	6	7	1	1
3	6	33	47	65	20	21	1	1
4	9	120	101	118	41	42	1	1
5	12	145	128	134	76	77	1	1
6	15	155	132	125	140	134	-6	36
7	18	110	113	107	139	148	9	81
8	21	70	88	80	113	112	-1	1
9	24	52	65	56	80	79	-1	1
10	27	32	45	37	55	54	-1	1
11	30	28	26	24	39	38	-1	1
12	33	18	17	16	29	28	-1	1
13	36	12	10	9	23	22	-1	1
14	39	6	7	5	17	16	-1	1
15	42	3	4	2	10	10	0	0
16	45	0	2	0	5	5	0	0
17	48	0	0	0	0	0	0	0
Σ					793	793		128

1. Mean of forecasts: $$M_f = \frac{1}{n}\sum_{i=1}^{n} U_f(i) = \frac{793}{17} \qquad =61.000$$

2. Mean of observations: $$M_o = \frac{1}{n}\sum_{i=1}^{n} U_o(i) = \frac{793}{17} \qquad =61.000$$

3. Bias: $$\mathrm{B} = \mathrm{M_f} - \mathrm{M_o} = 61.00 - 61.00 = 0.00$$

4. Mean squared error: $$\mathrm{MSE} = \frac{1}{n}\sum_{i=1}^{n}\left[U_f(i) - U_o(i)\right]^2 \qquad = 7.529$$

5. Variance: $$\mathrm{V} = \mathrm{MSE} - \mathrm{B}^2 = 7.529\text{- }(0.000)^2 \qquad =7.529$$

6. R squared: $$\mathrm{R}^2 = \left[\frac{\frac{1}{\mathrm{n}}\sum_{i=1}^{n}\mathrm{U_o}\,(\mathrm{i})\,\mathrm{U_f}\,(\mathrm{i}) - \mathrm{M_o M_f}}{\left(\frac{1}{\mathrm{n}}\sum_{i=1}^{n}\mathrm{U_o^2} - \mathrm{M_o^2}\right)\left(\frac{1}{\mathrm{n}}\sum_{i=1}^{n}\mathrm{U_f^2} - \mathrm{M_f^2}\right)}\right] \qquad = 0.997$$

Table 4.9: Reliability Analysis of Synthetic Unit Hydrograph (for rising portion only)

No.	Time (hr)	U.H from Isolated (Cumecs)	U.H from IUH (cumecs)	U.H from Collin's Method (Cumecs)	U.H from Average (Observed) (Cumecs)	U.H from Synthetic (Estimated) (Cumecs)	(U_f-U_o)	$(U_f-U_o)^2$
1	0	0	0	0	0	0	0	0
2	3	8	8	15	6	7	1	1
3	6	33	47	65	20	21	1	1
4	9	120	101	118	41	42	1	1
5	12	145	128	134	76	77	1	1
6	15	155	132	125	140	134	-6	36
7	18	110	113	107	139	148	9	81
Σ					**422**	**429**		**121**

1. Mean of forecasts: $M_f = \frac{1}{n}\sum_{i=1}^{n} U_f(i) = \frac{429}{6}$ =22.333

2. Mean of observations: $M_o = \frac{1}{n}\sum_{i=1}^{n} U_o(i) = \frac{422}{6}$ =23.333

3. Bias: $B = M_f - M_o = 22.333 - 23.333$ = -1.000

4. Mean squared error: $MSE = \frac{1}{n}\sum_{i=1}^{n}\left[U_f(i) - U_o(i)\right]^2$ = 6.000

5. Variance: $V = MSE - B^2 = 7.529 - (0.000)^2$ =5.000

6. R squared: $R^2 = \left[\frac{\frac{1}{n}\sum_{i=1}^{n} U_o(i)\,U_f(i) - M_o M_f}{\left(\frac{1}{n}\sum_{i=1}^{n} U_o^2 - M_o^2\right)\left(\frac{1}{n}\sum_{i=1}^{n} U_f^2 - M_f^2\right)}\right]$ = 0.995

Average Unit Hydrograph is based on analysis of observed storm events and related flood hydrographs. Synthetic Unit Hydrograph is based on catchments characteristics. Higher Coefficient of Determination and lower value of Bias are obtained for complete Unit Hydrograph in comparison to rising portion only.

4.9. CONCLUSIONS

Based on case study of Baitarni river basin, the following conclusions are drawn:

1) When spatial variation in rainfall is significant, it is not proper to take the area average rainfall for estimation of flood hydrograph. In such case catchments area

should be divided into sub basins so that rainfall distribution over each sub basin is rather uniform. Unit Hydrograph is applied to find flood hydrograph at outlet of each sub basins and then routed through channel up to catchment outlet to arrive at flood hydrograph at catchment outlet.

2) When various storms are considered for development of Unit Hydrograph for the same catchment a marked variation is observed in the peak as well as the time of occurrence of the peak. Therefore average Unit Hydrograph needs to be derived giving higher weightage for the Unit Hydrograph derived from severe storms. Different unit hydrographs should be identified for the various conditions which have major influence on formation and time distribution of the runoff. These unit hydrograph may then be judiciously applied under different conditions.

3) Methods used in separation of losses from storm rainfall are empirical and arbitrary. Instead of assuming an average infiltration of loss rate (0-index) for entire storm, different loss rates in different portions of the storm, can be assumed. (Φ-index underestimates losses in beginning portion of storm and over estimates loss rate in later portion of storm. Further (φ-index may significantly vary spatially due to different land use soil cover and soil characteristics and antecedent moisture condition. Therefore different 0-index may be used for different areas.

4) Methods used for base flow separation (while deriving Unit Hydrograph) or addition (while estimating flood hydrograph) are rather arbitrary. Same method should be consistently used in derivation of Unit Hydrograph and application of Unit Hydrograph.

5) Recent flood hydrographs should be used for derivation of UH. Changes in land use, land cover over the catchment area should be evaluated using remote sensing data and should be duly considered in the analysis.

Errors in forecasting floods in term of estimation of peak discharge and time to peak due to use of different UH when rainfall is not uniform and when rainfall is assumed to be uniform over the catchment are significant as shown below. In Baitarani basin the observed flood was 2180 cumec and observed time to peak was 18 hours.

When non uniform rainfall distribution is considered		When rainfall is assumed to be uniform	
Qp (cumecs)	Tp (hrs)	Qp (cumecs)	Tp (hrs)
1948.65	21	2244.92	18
Error in Qp Estimation (%)	Error in Tp Estimation (%)	Error in Qp Estimation (%)	Error in Tp Estimation (%)
10.61	-16.67	-2.98	0

A perfect forecast exists only if both Bias (B) and Variance (V) are zero. Bias and Variance are important parameters in addition to coefficient of determination (R^2) (Maidment, 1993).

Reliability of rising portion only is important in forecasting of rise in water level at the forecasting site such as for flood embankment, town situated on river bank. For reservoir complete hydrograph has to be forecasted.

From the reliability analysis, it is concluded that Collin's Unit Hydrograph can be used for flood forecasting at a town on river bank where only size of water level need to be forecast.

Higher Coefficient of Determination and lower value of Bias are obtained for complete Unit Hydrograph in comparison to rising portion only.

CHAPTER V

ANALYSIS OF FACTORS IN FLOOD FREQUENCY APPROACH

5.1. THE FLOOD FREQUENCY APPROACH

Various steps involved in flood frequency approach are explained in the flow chart given in Figure 5.1. The method would be common whether annual flood peaks or annual storm values are dealt with. The advantages of this approach are:

1) Catchment area characteristics and hydrometeorological data are not required,
2) the method can be computerized to a great extent,
3) Associated probability estimates are available.

However, the method has certain limitations as:

1) Several years observations on flood peaks are required. The analysis yields only the flood magnitudes and not volume or shape of the hydrograph.
2) Correct inference about the distribution which fits the sample data for a site is crucial as different distributions fitted to same data result in different estimated values especially in the extrapolated range.
3) Difficulties in having homogenous data due to developments like construction of new storage structures, u/s, etc.
4) Sufficiently long data length to allow reliable estimation of population parameters from the sample data is necessary.
5) Elements of risk and uncertainty are inherent in any flood frequency analysis.

5.1.1. Selection of Return Period for a Given Level of Risk of Exceedance

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

$$P(X > X_T) = 1 - (1 - \frac{1}{T})^r$$

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 in 100-year. 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.

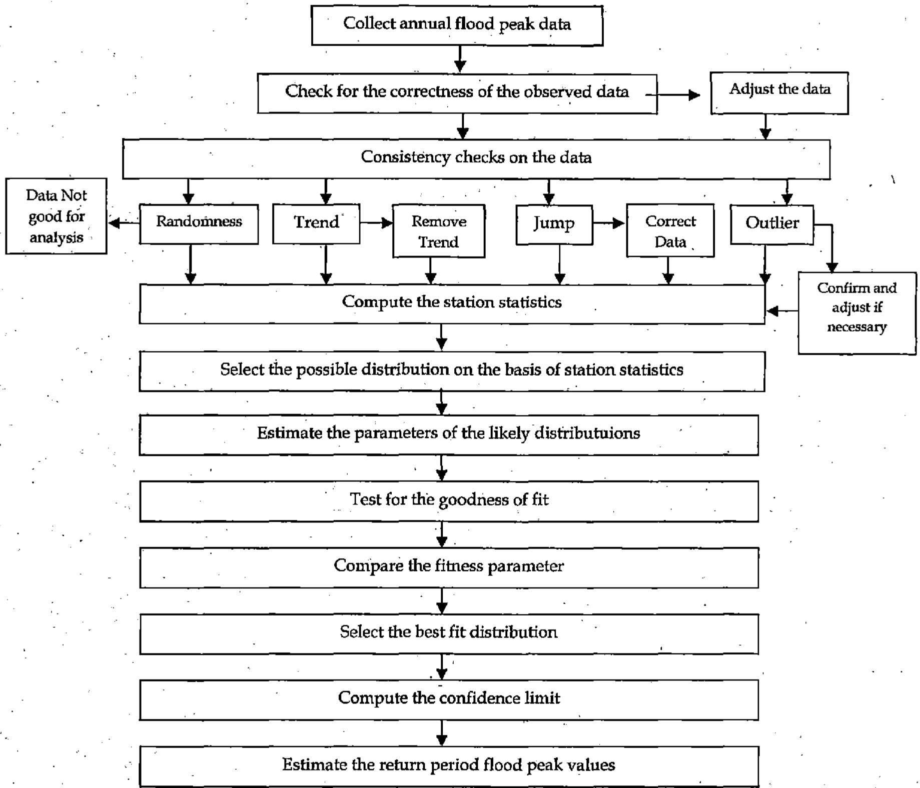


Figure 5.1: Flood Frequency Approach

$$T = \frac{1}{1-(1-P)^{1/r}}$$

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. In the above derivations, sampling errors are ignored. Since, this is not normally the case, there are further risks 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.

5.2. FACTORS AFFECTING ACCURACY OF FLOOD ESTIMATION

Estimation of design flood is an important component of dam safety analysis. Absolute safety of dam from flood is unrealistic. A rational hydrologic design must therefore take into consideration the risk of flooding and consequent damages. The risk of damage is equivalent to the probability of occurrence of flood larger than the design flood (WMO, 1994). Design flood criteria are often specified in terms of flood corresponding to a return period T or exceedance probability P (P=1/T). Several factors influence the reliability of estimate of T year flood. These are:

1) Length of data i.e. no. of years record of annual maximum floods.
2) Data series should be random, consistent and free from, jump, trend and outlier.
3) Choice of plotting position formulae out of several formulae given in literature (Hazen, Weibull, Gringorten, etc.)
4) Choice of theoretical probability distribution considered for application.
5) Occurrence of an extreme rare event in the data series. Whether it is rejected as an outlier or included as a very important observed value.

Effect of various factors on design flood estimation using flood frequency approach is analyzed through case study of annual maximum floods at Bhakra dam site on river Sutlej in India.

5.3. ANNUAL FLOOD SERIES AT THE BHAKRA DAM SITE

Bhakra Nangal Dam is across the Sutlej River, near the border between Punjab and Himachal Pradesh in northern India. The dam, located in the village of Bhakra in the Bilaspur region of Himachal Pradesh, is Asia's largest at 225.55 m (740 ft) high. It is the highest gravity dam in the world. The length of the dam (measured from the road above it) is 518.25 m; it is 304.84 m broad. Its reservoir, known as the "Gobind Sagar", stores up to 9340 million m3 of water, enough to drain the whole of Chandigarh, parts of Haryana, Punjab and Delhi. The 90 km long reservoir created by the Bhakra Nangal Dam is spread over an area of 168.35 km^2. In terms of storage of water, it is the second largest dam in India, the first being Indira Sagar dam in Madhya Pradesh with capacity of 12.22 billion m3. The dam was part of the larger multipurpose Bhakra Nangal Project whose aims were to prevent floods in the Sutlej-Beas river valley, to provide irrigation to adjoining states and to provide hydro-electricity and was constructed with an aim to provide irrigation to the Punjab and Himachal Pradesh. The dam provides irrigation to 10 million acres (40,000 km^2) of fields in Punjab, Himachal Pradesh, Haryana, Delhi and Rajasthan. It also became a tourist spot for the tourists during later years because of it huge size and uniqueness. Another reason behind the construction of the dam was to prevent damage due to monsoon floods.

Figure 5.2: Bhakra Dam Site at Sutlej River

Observed annual flood peak data of 84 years (from 1909 to 1992) are given in table 5.1 which covers 51 years pre construction period (1909 to 1959) and 33 years post construction period (1960 to 1992).

In the following paragraphs, following analysis have been carried out using the observed data:

1) Randomness of data series is checked using peak and trough analysis.
2) Effect of length of data is analyzed by considering following three different series:
   i) Pre construction flood series (1909 to 1959)
   ii) post construction flood series (1960 to 1992)
   iii) Entire flood series (1909 to 1992)
3) Choice of plotting position formulae. Probability of exceedance of observed flood peaks have been computed using following empirical formulae:
   i) Hazen formulae
   ii) Weibull formulae
   iii) Gringorten formulae

4) Presence of Jump and Trends has been check by applying moving average method applied to mean and standard deviation a) including highest observed peak and b) excluding the highest observed peak in the series.

5) Outlier test for the highest and lowest observed series.

6) Changes in statistical properties due to:
   i) Different length of data
   ii) Inclusion/exclusion of highest observed value as outlier.

7) Choice of probabilities distribution:
   i) Log normal
   ii) Extreme value type I
   iii) Pearson type III

Table 5.1: Annual Maximum Floods (1909 to 1992) and Estimation of Peaks and Troughs

Year	Peak flow (cumecs)	Score	Year	Peak flow (cumecs)	Score	Year	Peak flow (cumecs)	Score
1909	3,653	-	1937	3,138	1	1965	3,189	1
1910	5,635	1	1938	5,805	1	1966	3,990	0
1911	3,653	1	1939	3,093	0	1967	5,701	1
1912	6,683	1	1940	1,723	1	1968	4,803	1
1913	5,635	1	1941	3,656	0	1969	6,308	1
1914	7,079	1	1942	6,598	1	1970	3,643	1
1915	4,332	0	1943	5,274	0	1971	17,227	1
1916	3,766	1	1944	2,294	1	1972	2,125	1
1917	5,125	1	1945	2,384	0	1973	7,697	1
1918	1,982	1	1946	3,819	0	1974	2,567	1
1919	5,182	1	1947	7,808	1	1975	6,516	1
1920	4,248	1	1948	4,531	0	1976	5,432	1
1921	4,587	1	1949	3,256	1	1977	4,244	1
1922	4,446	0	1950	4,984	0	1978	10,726	1
1923	3,398	1	1951	9,203	1	1979	2,842	1
1924	6,711	1	1952	5,239	0	1980	4,246	0
1925	5,412	0	1953	4,814	1	1981	7,593	1
1926	3,455	1	1954	5,635	1	1982	4,056	1
1927	4,000	1	1955	5,352	0	1983	4,172	1
1928	2,398	1	1956	2,704	1	1984	2,474	1
1929	4,588	0	1957	3,285	0	1985	6,960	1
1930	6,938	1	1958	3,931	1	1986	5,182	0
1931	2,033	1	1959	3,191	1	1987	2,461	1
1932	5,040	1	1960	5,221	1	1988	9,010	1
1933	3,299	1	1961	4,698	1	1989	4,192	1
1934	4,332	0	1962	5,407	1	1990	7,166	1
1935	5,182	1	1963	5,224	1	1991	2,649	0
1936	3,993	0	1964	6,716	1	1992	6,629	-

Highest observed flood is 17,227 m^3/s in the year 1971 which pertains to post construction period. The flood frequency analysis in post construction period is influenced by choice for inclusion/exclusion of this rare event in the data series.

5.4. CHECK FOR RANDOMNESS OF THE DATA SERIES

The commonly used method to check the randomness of the given series of the peak annual flow data is based on the observation of the number of peaks and troughs. Defining a peak as the occurrence of a value Yt such that,

$$Yt\text{-}1 < Yt > Yt\text{+}i$$

and a trough as a value yt such that

$$Yt\text{-}1 > Yt < Yt\text{+}l$$

the test of randomness is conducted as follows.

The total number of data is 84 and the total "score" i.e., the total number of peaks and troughs in the data series works out to be 61 (Table 5.1). Therefore the mean and the variance are calculated as follows.

$$\text{Mean} = \frac{2}{3}(N - 2)$$

Where N is the total number of the data. For N=84,

$$\text{Mean.value} = \frac{2}{3}(84 - 2) = 54.67$$

$$\text{Variance} = \frac{(16N - 29)}{90} = \frac{16x84 - 29}{90} = 14.611$$

$$\text{Normal variate} = \frac{61 - 54.67}{14.61} = 0.433$$

The normal deviate is less than 1.96 i.e., the value corresponding to 5% probability. Therefore, there is no real reason to suggest that the 84 annual peak flow values form other than a random series.

5.5. CHECK FOR CONSISTENCY, JUMP, TREND AND OUTLIERS

5.5.1. Consistency

The plot of the annual peak flood data given in table 5.1 is shown in Fig. 5.3. The visual inspection of the data indicates a certain degree of variation in the average value of the peak flood in the later years of observation. The plot exhibits, in general, a slightly rising trend. However, with a view to examine the possibility of variations in the characteristics of the data in the pre-construction and the post-construction periods, the data series has been bifurcated in two parts – one for the period from 1909 to 1959 and the other for the period from 1960 to 1992. The salient features of the two data series are illustrated in Figure. 5.3.

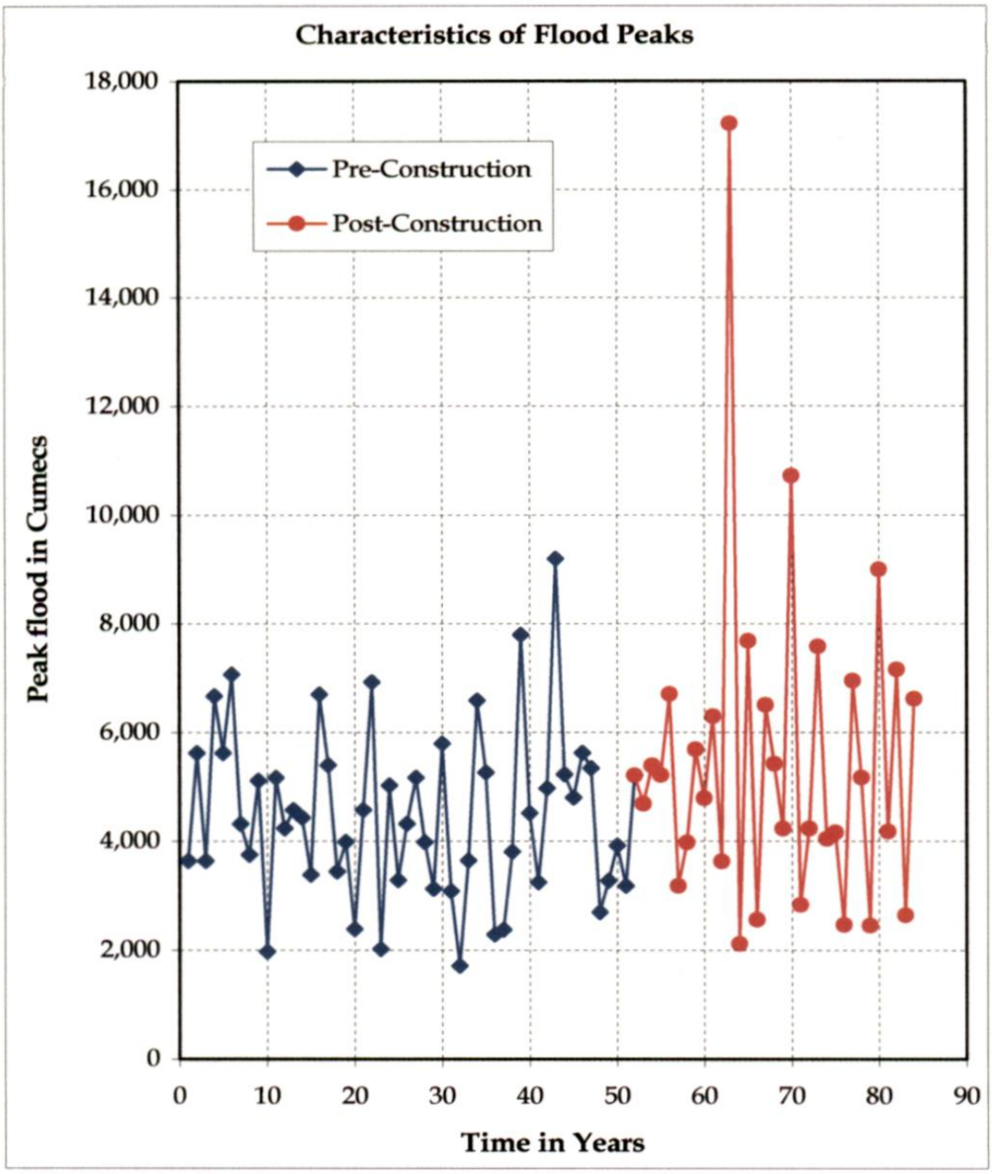


Figure 5.3: Characteristics of Flood Peaks

5.5.2. Presence of Jump and Trends

With a view to evaluate the presence of the features like the trend or the jump in the series, analysis has been carried out by using the moving average method. The results of three analyses indicate clearly the presence of jump in the data series. Similarly a slight trend is also exhibited. The results of analysis do not suggest any specific change in the data characteristics in the post-construction stage. However the changes in the characteristics of the data over are clearly depicted.

The test for the presence of jump indicates that the jump is present in the data series and that the same has to be accounted for in further analysis for the flood frequency analysis. However, a close scrutiny of the data indicate that the jump as illustrated in Figure. 5.4 and Figure. 5.5, are mainly because of specific peak flood value of the order of 17,227 m^3/s. Once the specific annual peak flood data is removed from the series, the presence of the jump is not exhibited as may be seen in Figures. 5.4 and 5.5.

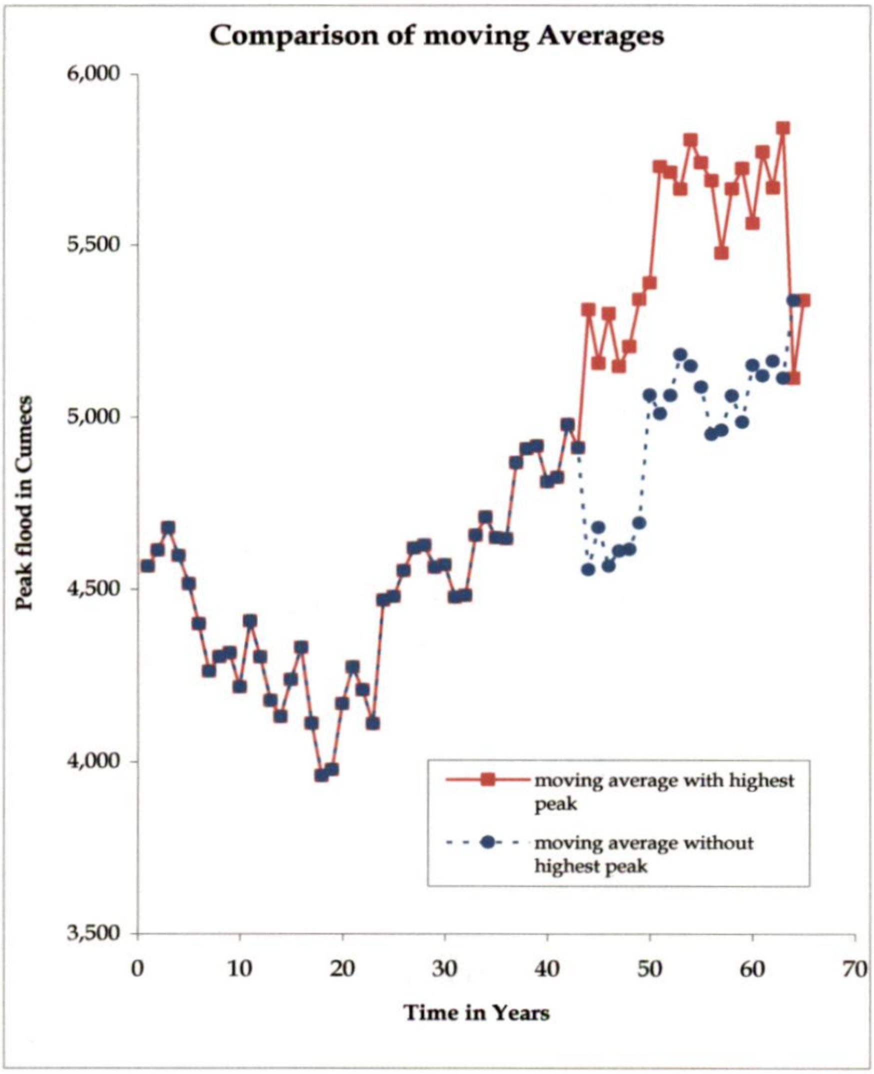


Figure 5.4: Comparison of moving Averages

In order to examine the possibility of the specific annual flood peak data of 17,227 m^3/s being erroneous, the corresponding values of the rainfall record were scanned. The peak value has been reported to be observed on August 6, 1971. The flood hydrograph for the period from 0300 Hrs of 6.8.71 to 2400 Hrs `of 7.8.71 indicates a total volume of 99.93 mm of runoff against the average value of 250.00 mm of the rainfall recorded at the various stations in the basin. Thus the runoff during the specific extraordinary flood event is only about 40% of the rainfall falling over the basin suggesting that reported value of the annual peak flood of 17,227 m^3/s can not be considered to be erroneous.

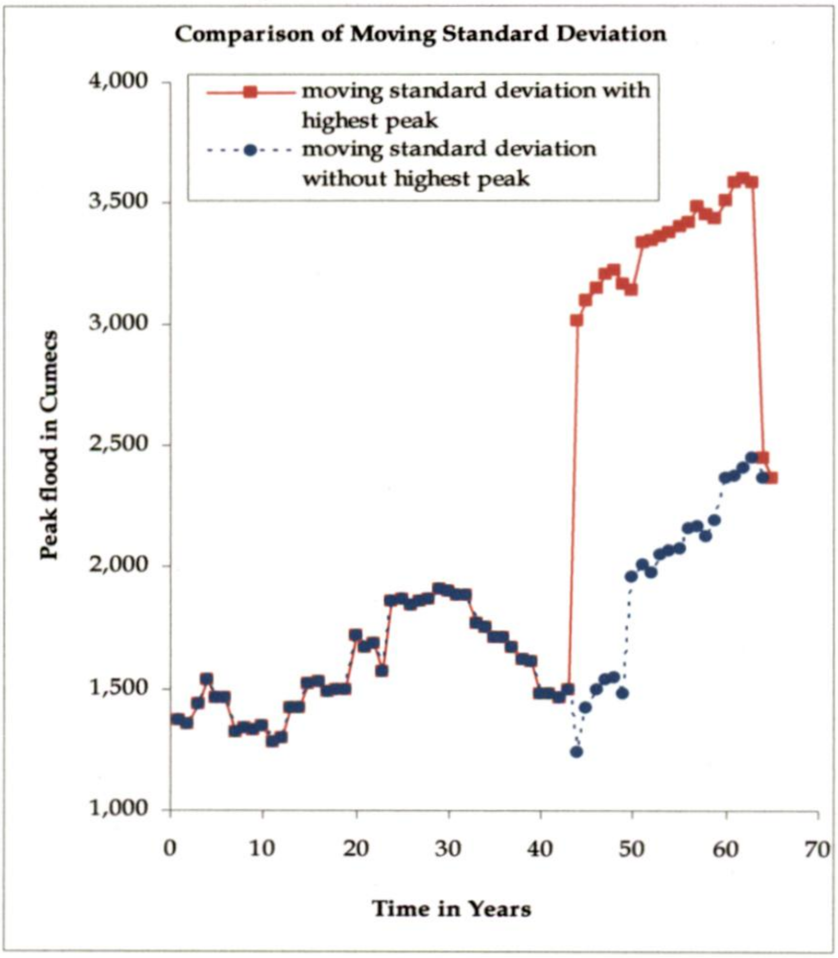


Figure 5.5: Comparison of moving Standard Deviation

5.5.3. Test for Outliers

In order to check whether some of the data of flood events in the given series are, outliers, the following analysis is carried out. The statistics of the log transformed annual peak discharge series are computed.

Mean Logarithm	8.407
Standard Deviation of Logs	0.410
Skewness Co-efficient of Logs	0.144
Number of years	84

Since the value of the skewness coefficient lies between -0.40 and +0.40, the test for both the low outliers as well as high outliers are to be carried out.

Check for Low Outliers: The low outlier threshold (X_L) is computed by

$$X_L = \overline{X} - K_N S$$

where, X_L = Low outlier threshold

X =Mean of the log transformed series

S = Standard deviation of the log transformed series

K_N = Outliers test K values for 10 percent significance level for a Normal distribution

X_L = 8.407+2.957 x 0.410= 7.195

Q_L= anti log (7.195) = 1,332 cumec

There is no value below this threshold value. Therefore, the low outliers are not present.

Check for High Outliers: The high outlier threshold (X_H) is computed by

$$X_H = \overline{X} - K_N S$$

where, X_H = High outlier threshold

X = Mean of the log transformed series

S = Standard deviation of the log transformed series

K_N = Outliers test K values for 10 percent significance level for a normal distribution

X_H = 8.407+2.957 x 0.410 9.619

Q_H = anti log (9.619) = 15,051 cumec

The 1971 flood peak value of 17,227 m^3/s exceeds this value. Therefore the meteorological conditions at the time of occurrence of the 1971 peak flood value need to be investigated to establish whether such flood could actually occurs or there is error in this data.

5.6. STATISTICAL CHARACTERISTIC OF THE FLOW SERIES

Table 5.2 shows statistical parameter (mean, std. deviation, skewness, kurtosis) for different data series. Skewness of the preconstruction series (1909-1959) is much less compared to skewness of post construction series (1960-1992).

Table 5.2: Statistical Parameters for diferent Data series

Parameter	All the Annual Maximum Data Series (1909-1992)		Pre-Construction Series (1909-1959)		Post-Construction Series (1960-1992)	
	Normal value	Logs value	Normal value	Logs value	Normal value	Logs value
No.of year	84.00	84.00	51.00	51.00	33.00	33.00
Mean (cumecs)	4,875.81	8.41	4,480.43	8.34	5,486.85	8.50
Maximum (cumecs)	17,227.00	9.75	9,203.00	9.13	17,227.00	9.75
Minimum (cumecs)	1,723.00	7.45	1,723.00	7.45	2,125.00	7.66
Variance	4,991,138.25	0.17	2,469,491.09	0.13	8,453,007.45	0.21
Standard Deviation	2,234.09	0.41	1,571.46	0.37	2,907.41	0.46
Skewness Coeff. of Variance	2.34	0.14	0.61	-0.37	2.23	0.32
Kurtosis	10.45	0.60	0.53	-0.03	7.52	0.48

As the flow data series has a relatively high value of skewness (+2.234), the normal distribution which is symmetrical in nature will not be applicable.

5.7. CHOICE OF PLOTTING POSITION FORMULA

Three commonly used formulas for estimating the probability of the different values of the annual flood peaks are:

1) Hazen formula $P (X \geq x) = (m - 0.5)$
2) Weibull formula $P (X \geq x) = m/(N+1)$
3) Gringorten formula $P (X \geq x) = (m-0.44)/(N+0.12)$

All the above three formulae have been used to compute the plotting positions or probability of exceedence of the various observed flood peaks as shown in Table 5.3. It is seen that lowest values are not affected by the choice of plotting formula. Highest values are significantly affected (Table 5.3). Plotting position computed by Weibull formula is considerably different from that computed using Hazen or Gringorton formula.This may cause significant error in extrapolation

Attempt has been made to plot the values of the different annual peak flood and the respective probability of exceedence on the probability paper.

The following probability papers are used.

1) Normal probability paper
2) Log normal probability paper, and
3) Extreme value Type-I Distribution (with coefficient of skewness = 1.139) probability paper.

Table 5.3: Ranking of the Data and Plotting Position

SI	Value of X	Plotting position (%) according to			SI	Value of X	Plotting position (%) according to		
		Hazen	Weibull	Gringorton			Hazen	Weibull	Gringorton
		p(X≥x)=(m-0.5)/N	p(X≥x)=m/(N+1)	p(X≥x)=(m-0.44)/(N+0.12)			p(X≥x)=(m-0.5)/N	p(X≥x)=m/(N+1)	p(X≥x)=(m-0.44)/(N+0.12)
(1)	(2)	(3)	(4)	(5)	(1)	(2)	(3)	(4)	(5)
1	17,227	0.595	1.176	0.666	43	4,531	50.595	50.588	50.594
2	10,726	1.786	2.353	1.854	44	4,446	51.786	51.765	51.783
3	9,203	2.976	3.529	3.043	45	4,332	52.976	52.941	52.972
4	9,010	4.167	4.706	4.232	46	4,332	54.167	54.118	54.161
5	7,808	5.357	5.882	5.421	47	4,248	55.357	55.294	55.350
6	7,697	6.548	7.059	6.610	48	4,246	56.548	56.471	56.538
7	7,593	7.738	8.235	7.798	49	4,244	57.738	57.647	57.727
8	7,166	8.929	9.412	8.987	50	4,192	58.929	58.824	58.916
9	7,079	10.119	10.588	10.176	51	4,172	60.119	60.000	60.105
10	6,960	11.310	11.765	11.365	52	4,056	61.310	61.176	61.293
11	6,938	12.500	12.941	12.553	53	4,000	62.500	62.353	62.482
12	6,716	13.690	14.118	13.742	54	3,993	63.690	63.529	63.671
13	6,711	14.881	15.294	14.931	55	3,990	64.881	64.706	64.860
14	6,683	16.071	16.471	16.120	56	3,931	66.071	65.882	66.049
15	6,629	17.262	17.647	17.309	57	3,819	67.262	67.059	67.237
16	6,598	18.452	18.824	18.497	58	3,766	68.452	68.235	68.426
17	6,516	19.643	20.000	19.686	59	3,656	69.643	69.412	69.615
18	6,308	20.833	21.176	20.875	60	3,653	70.833	70.588	70.804
19	5,805	22.024	22.353	22.064	61	3,653	72.024	71.765	71.992
20	5,701	23.214	23.529	23.252	62	3,643	73.214	72.941	73.181
21	5,635	24.405	24.706	24.441	63	3,455	74.405	74.118	74.370
22	5,635	25.595	25.882	25.630	64	3,398	75.595	75.294	75.559
23	5,635	26.786	27.059	26.819	65	3,299	76.786	76.471	76.748
24	5,432	27.976	28.235	28.008	66	3,285	77.976	77.647	77.936
25	5,412	29.167	29.412	29.196	67	3,256	79.167	78.824	79.125
26	5,407	30.357	30.588	30.385	68	3,191	80.357	80.000	80.314
27	5,352	31.548	31.765	31.574	69	3,189	81.548	81.176	81.503
28	5,274	32.738	32.941	32.763	70	3,138	82.738	82.353	82.691
29	5,239	33.929	34.118	33.951	71	3,093	83.929	83.529	83.880
30	5,224	35.119	35.294	35.140	72	2,842	85.119	84.706	85.069
31	5,221	36.310	36.471	36.329	73	2,704	86.310	85.882	86.258
32	5,182	37.500	37.647	37.518	74	2,649	87.500	87.059	87.447
33	5,182	38.690	38.824	38.707	75	2,567	88.690	88.235	88.635
34	5,182	39.881	40.000	39.895	76	2,474	89.881	89.412	89.824
35	5,125	41.071	41.176	41.084	77	2,461	91.071	90.588	91.013
36	5,040	42.262	42.353	42.273	78	2,398	92.262	91.765	92.202
37	4,984	43.452	43.529	43.462	79	2,384	93.452	92.941	93.390
38	4,814	44.643	44.706	44.650	80	2,294	94.643	94.118	94.579
39	4,803	45.833	45.882	45.839	81	2,125	95.833	95.294	95.768
40	4,698	47.024	47.059	47.028	82	2,033	97.024	96.471	96.957
41	4,588	48.214	48.235	48.217	83	1,982	98.214	97.647	98.146
42	4,587	49.405	49.412	49.406	84	1,723	99.405	98.824	99.334

The plot on the above probability papers are shown in Figures 5.6 to 5.8. In all the three cases the probability of exceedence as computed by the Hazen formula has been used. Figures 5.6 and 5.7 suggest that the data do not provide good fit to the normal and the Gumbel's Extremal Value Type-I (with coefficient of skewness=1.139) distribution. On the other hand, the plot on the Log normal probability paper suggests a reasonably good fit. However, attempt has been made to apply some of the commonly used probability distributions.

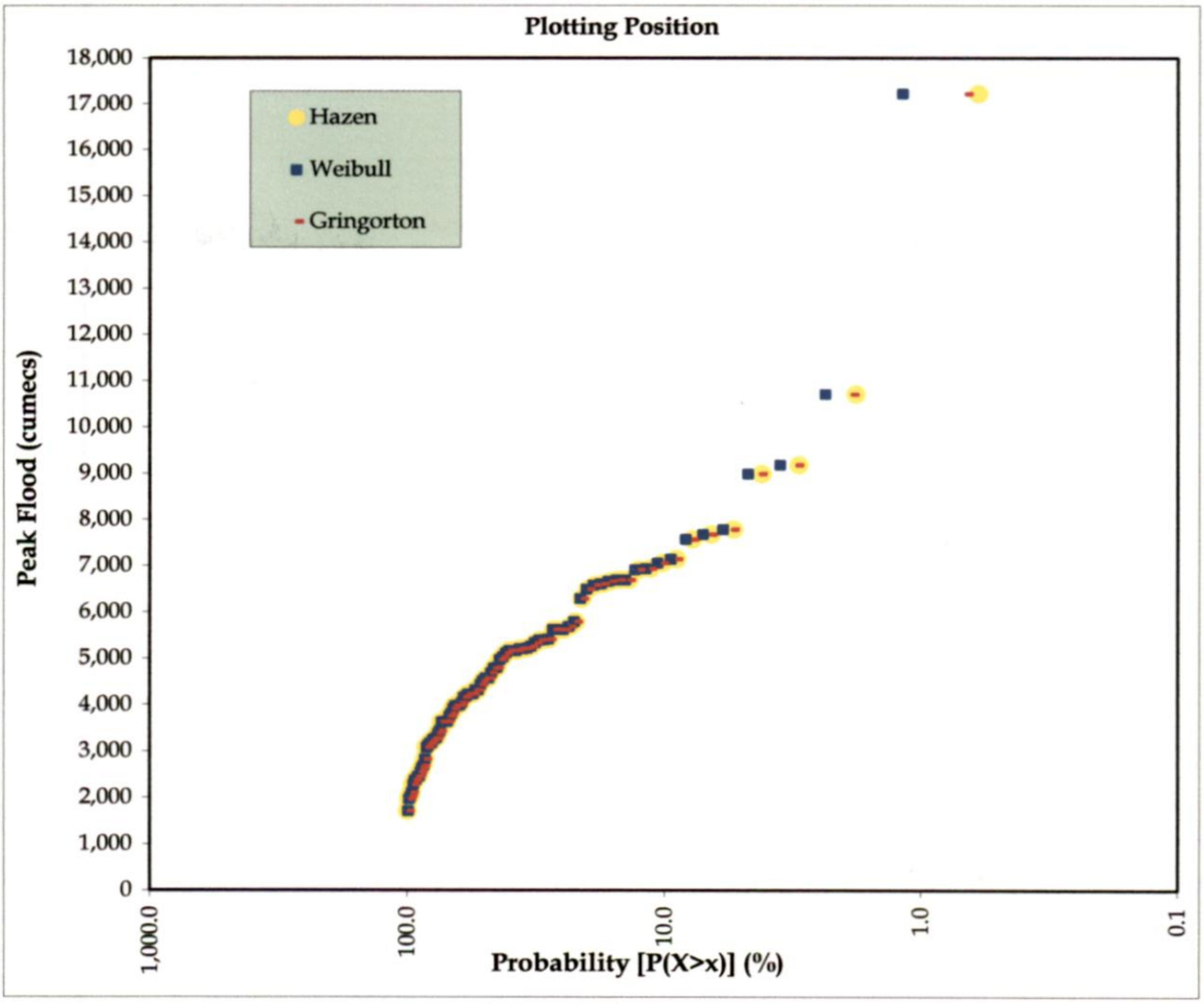


Figure 5.6: Plotting Position

5.8. CHOICE OF THEORITICAL PROBABILITY DISTRIBUTION

An attempt has been made to examine suitability of the various probability distributions to the data series corresponding to the pre-construction and post-construction stage as the data series for both the pre-construction and post-construction periods are sufficiently long. The following distributions which are generally used for the flood frequency analysis have been considered.

1) Normal Distribution,
2) Log Normal Distribution with 2-parameters

3) Extreme Value Type-I (EV1) Distribution (with coefficient of skewness= 1. 139), and
4) Pearson Type III Distribution (PIII).

The results of the analysis are summarized in Table 6.4 and table 6.5.

Table 5.4: The Statistical Parameters of Probability Distributions

No.	Distribution	Mean	SD	Skewness	Kurtosis
I	**1909-1992 (84 years)**				
1	Normal	4875.810	2234.086	2.339	10.449
2	Log Normal	8.407	0.410	0.142	0.605
3	Extreme Value Type I	4875.810	2234.086	2.339	10.449
4	Pearson Type III	4875.810	2234.086	2.339	10.449
II	**Pre Construction 1909-1959 (51 years)**				
1	Normal	4480.431	1571.461	0.608	0.531
2	Log Normal	8.345	0.365	-0.374	-0.028
3	Extreme Value Type I	4480.431	1571.461	0.608	0.531
4	Pearson Type III	4480.431	1571.461	0.608	0.531
III	**Post Construction 1960-1992 (33 years)**				
1	Normal	5,486.848	2,907.406	2.228	7.517
2	Log Normal	8.502	0.470	0.321	0.477
3	Extreme Value Type I	5,486.848	2,907.406	2.228	7.517
4	Pearson Type III	5,486.848	2,907.406	0.321	7.517

Statistical parameters of the original series are used in Normal, EV Type I and Pearson Type III distributions where as for log Normal distribution the parameter of log transformed are used. Post construction series is highly skewed. Log transformation helps in significant reduction in series.

5.9. FLOODS FOR DIFFERENT RETURN PERIOD

Floods corresponding to the return period of 1,000 year and 10,000 year have been computed by using all the distribution and the same are given in the Table 5.5.

If peak flood of about 17,227 m^3/s observed in the year 1971 is taken as real event and taken as a part of the sample of 84 annual maximum, the estimated value of 1,000 year return period and of 10,000 year return period appears to be on lower side on its face value. However, the analysis for the outliers suggests the 1971 flood peak value to be an higher outlier which in general means that it may be equivalent to a flood of a return period of more than 100 years.

Table 5.5: Estimated Design Flood by Various Frequency Distributions

No.	Distribution	25 yr Flood	50 yr Flood	100 yr Flood	500 yr Flood	1000 yr Flood	10000 yr Flood
I	**1909-1992 (84 years)**						
1	Normal	8,787.69	9,464.62	10,072.29	11,309.98	11,756.79	12,672.77
2	Log Normal 2	9,179.92	10,394.69	11,621.49	14,586.27	15,833.23	18,732.75
3	Extreme Value Type I	9,811.06	11,124.46	12,428.17	15,440.83	16,736.01	21,036.26
4	Pearson Type III	9,829.58	11,541.61	13,306.06	17,586.57	19,503.04	26,154.53
II	**Pre Construction 1909-1959 (51 years)**						
1	Normal	7,232.06	7,708.21	8,135.65	9,006.24	9,320.53	9,964.83
2	Log Normal 2	7,980.01	8,914.20	9,845.60	12,054.50	12,968.34	15,064.06
3	Extreme Value Type I	8,063.84	9,015.03	9,959.20	12,141.03	13,079.03	16,193.36
4	Pearson Type III	7,530.70	8,196.97	8,826.11	10,188.95	10,745.44	12,506.29
III	**Post Construction 1960-1992 (33 years)**						
1	Normal	10,577.72	11,458.66	12,249.47	13,860.18	14,441.66	15,633.69
2	Log Normal 2	11,033.96	12,687.24	14,381.34	18,563.72	20,355.83	24,588.81
3	Extreme Value Type I	12,377.22	14,199.29	16,007.90	20,187.32	21,984.12	27,949.79
4	Pearson Type III	11,918.18	14,087.28	16,312.60	21,680.76	24,073.76	32,344.27

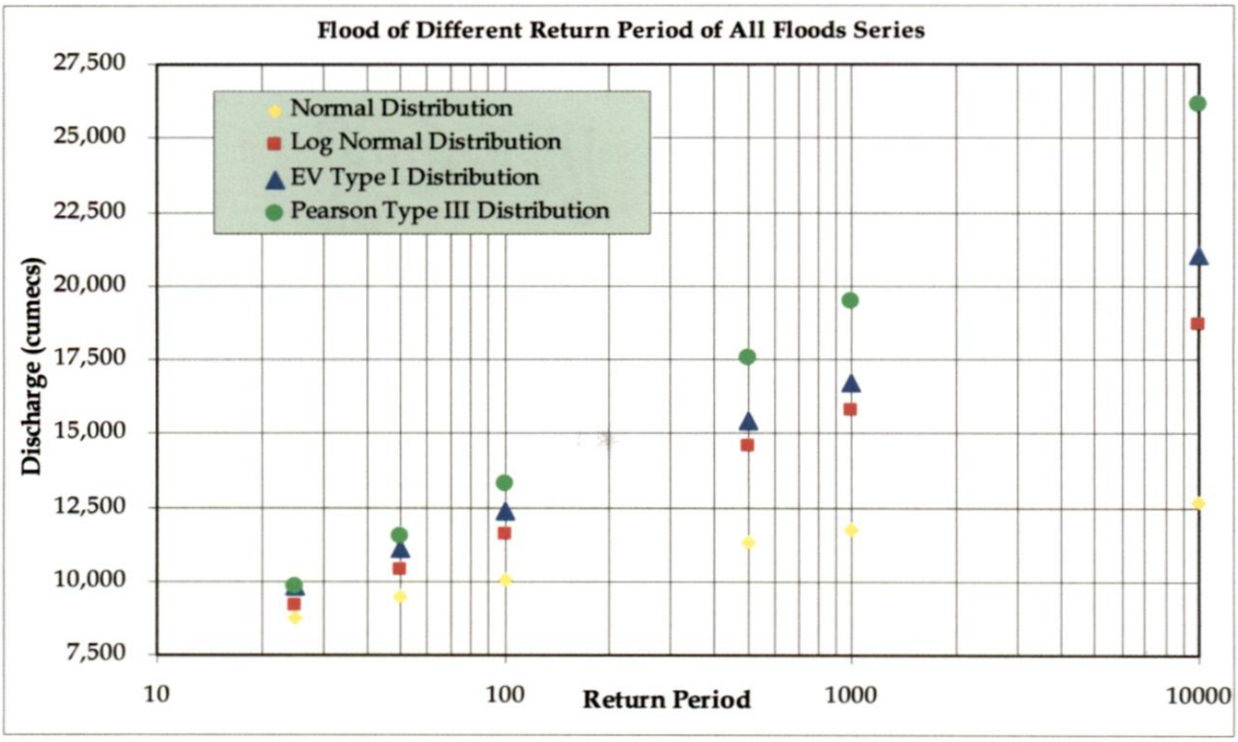


Figure 5.7: Flood of Different Return Period.

Flood estimates by different methods are compared in Figure 5.7. As shown in Figure 5.7 Pearson Type III provides higher estimates of return year floods compared to other probability distributions. Pre construction period estimate of 1000 year floods

10,745.44 cumec where as post construction period estimate is 24,073.76 cumec which is 224.04% of the preconstruction estimate.

Choice of probability distribution has significant affect on flood estimate. 1000 year flood by EV Type I probability distribution is 16,736.01 cumec where as Pearson Type III provides estimate of 19,503.04cumec which is 1.17 time more.

5.10. CONCLUSIONS

1 Estimation of design flood is an important component of dam safety analysis. Design flood criteria are often specified in terms of flood corresponding to a return period T or exceedance probability P (P=1/T). Several factors influence the reliability of estimate of T year flood. Elements of risk and uncertainty are inherent in any flood frequency analysis as subjectivity is involved in making choice about length of data, method of probability distribution, plotting position etc. The judgment of a professional experienced in hydrologic analysis becomes necessary to enhance the usefulness of flood frequency approach.

2 Effect of various factors on design flood estimation is analysed through case study of floods at Bhakra dam site on river Sutlej in India. Following analysis have been carried out using the observed data:

3 Peak and trough analysis shows that the data series is random. Effect of length of data is analysed by considering following three different series:

   i) Pre construction flood series (1909 to 1959):

   ii) post construction flood series (1960 to 1992):

   iii) Entire flood series (1909 to 1992):

4 Choice of plotting position formulae. Probability of exceedance of observed flood peaks have been computed using i) Hazen formula, ii) Weibull formula, iii) Gringorten formula. It is seen that lowest values are not affected by the choice of plotting formula. Highest values are significantly affected (table 5.3). This may cause significant error in extrapolation

5 Presence of Jump and Trends has been check by applying moving average method (figure 5.2) to mean value of twenty year data a) including highest observed peak and b) excluding the highest observed peak in the series. A rising trend in the mean is observed. Trend is significantly influenced by a single value 17227 cumec observed in the year 1971.

6 Outlier test for the highest and lowest observed series shows that the 17227 cumec flood observed in the year 1971 is a high outlier. An element of subjectivity is introduced in the analysis by inclusion/exclusion of this variate in the sample. Changes in statistical properties occur due to this variety.

7 Inclusion/exclusion of highest observed value as outlier: Highest observed flood is 17227 m^3/s in the year 1971 which pertains to post construction period. The flood frequency analysis in post construction period is influenced by choice for inclusion/exclusion of this rare event in the data series.

8 Choice of probabilities distribution: Following three methods have been compared: i) Normal, ii) Log normal, iii) Extreme value type I, iv) Pearson type III. For the same data series, different probability distributions provide significantly different estimates e.g. using 1909-1992 data series, 10000 year estimate by EVI is 21,036.26 m^3/s and by PIII is 26,154.53 m^3/s (24.33 % higher).

9 Using same probability distribution but different samples from same population also result in significantly different estmates. 10000 year flood estimate using LN II probability distribution are 18,732.75 cumec (1909-92 data series), 15,064.06 cumec (1909-59 data series) and 24,588.81 m^3/s (1960-92 data series).

10 For the safety of dam and other hydraulic structures, it is very important to understand the limitations of the flood frequency approach due to various factors influencing the flood estimate.

CHAPTER VI

PMP ESTIMATION AND PMF EVALUATION OF SPILLWAY ADEQUACY FOR WONOGIRI DAM RESERVOIR

6.1. UNCERTAINTY IN PMP ESTIMATE

The estimation of the PMP involves some uncertainty. The most accurate maximization processes rely upon upper air data, which may not be available at desired location. The maximization is applied to large historic storms, leading to questions of how transposable these storms may be. Also, the storms are expressed in terms of depth-area curves, with the actual shape (geographical distribution) of the original storm being lost.

6.1.1. Complexity of Physical Processes and Change with Time

For large basins, factors such as interception, surface storage, and infiltration capacity can vary greatly across the basin. These parameters can also change with time due to factors such as land use, development, and post-flood channel changes. This complexity is highlighted by the variance in the rainfall-runoff response noted in the existing data-sets.

6.1.2. Data Limitations

Even if the physical processes were fully understood, a great deal of data is required to precisely model the physics of the rainfall-runoff response. Available DTM and GIS vector data-sets can assist in quantifying certain geometric aspects of basins, such as sub-basin delineation and slopes. However, these data-sets offer limited ability to assess surface storage, overland flow characteristics, and network capacity. Subsurface parameters affecting infiltration capacity may also vary significantly over a basin. The timing of the storm can also vary across the basin. The lack or limited operation of a runoff gauge at the design site will also affect the ability to calibrate the model.

6.1.3. Extrapolation Errors

Estimates of PMP may be typically in the range of double the typical large storm. With the wet initial moisture conditions and typical hydrologic models which apply most of the losses to the first part of the rainfall, the majority of the increased rainfall input is assumed to become runoff. This results in peak flow and runoff volume estimates for PMF that are in the range of 4 to 5 times largest historic events for high runoff potential basins, and even higher ratios for less productive basins (Alberta 2009). This significant extrapolation beyond observed events introduces significant uncertainty into PMF modeling. There may also be limitations on the timing of the PMP in order to reach these large rainfall values. The large number of parameters combined with the range of uncertainty results in a great range of possible results.

Application of PMF modeling to certain dam sites in Alberta (Alberta, 2009) has yielded a significant range of estimates from various sources. Attempts to improve consistency in these estimates have identified many of the potential problems. Some of the key issues identified include the magnitude of inputs (PMP, snowmelt), combination of inputs (e.g. rain plus snow, rain before PMP), limited availability of hydrologic data, complexity of the models, and changes in physical processes between the calibration data and the extrapolated PMF scenario.

6.2. RATIONALIZATION OF DESIGN STORM PARAMETERS-INDIAN PRACTICE

Recommended procedure in India (CWC 1993) is as given below:

6.2.1. The Design storm and the Critical Design Rainfall Duration

In general, Design Storm of duration equivalent to base period of the UH (in respect of fan shaped catchments of 5000 sq.kms and below) rounded to the next nearest value which is in multiples of 24 hrs and less than and equal to 72 hrs is considered adequate.

For assembling design storm hyetograph elements during the storm, duration equivalent to the base period (as referred in Para above) adjusted to next nearest value in multiples of 12 hrs or 72 hrs whichever is less shall be adopted. This procedure eliminates additional volume accruing in the design flood.

In respect of large catchments (where distributed models are used for designating the response of the catchments) the storm duration for causing the PMF, is to be equivalent to 2.5 times the travel time from the farthest point (time of concentration) to the site of structure.

6.2.2. Clock Hour Correction

Correction for point rainfall conversion from observational day, to 24 hrs for PMP rainfall value shall be 50 mm. No clock hour correction is required for catchments above 5000 sq.km.

6.2.3. Storm Transposition

The practices that may be followed for D-A-D analysis and Area Correction Factor are given below:

1) Point rainfall values need no reduction up to about 50 sq.kms. or catchments whose basin lags are less than two hours on the presumption that the areal average of rainfall in about two hrs. is almost the same as the point rainfall. The length and breadth ratio of fan shaped or more or less a circular catchment is almost the same as the ratio of major to minor axis of ellipse and generally matches with the shape of the eye of the storm isohyetal map. Therefore, application of point rainfall as Standard Project Storm (SPS) is recommended even for catchments up to 100 sq.kms. if the elongation ratio is less than 1.5.
2) It is recommended to apply DAD curves for catchments in the range of 50 - 500 sq.km. with elongation ratio (i.e. length/ breadth) not more than 1.5, since average rainfall in sub-catchments within that size and elongation ratio of the catchments is unlikely to differ significantly.
3) Where the shape of the project catchment matches with the shape of the isohyetal pattern of the storm under consideration, DAD values would suffice for catchments areas up to about 1000 sq.km.
4) For situations other than those specified, like for elongated catchments, storm transposition is recommended.
5) Where the project catchment is intercepted by an existing dam, storm transposition is preferred. If this is not possible necessary adjustment shall be made to DAD values obtained for parts and full catchment with the assumption that the storm is centered in intercepted or free catchments at different times and the severity of the flooding from those shall be examined.
6) In India and Indonesia record of SRRG data being limited it is not possible to derive storm centered relationships for within storm durations of 24 hrs. Except for 1 day, 2 day and 3 day within 3-day storm. Therefore, DAD curves of severe storms in the region may be used for obtaining 1-day areal rainfall from 1-day point SPS/PMP.

6.2.4. Moisture Maximization

Where dew point data, along the moisture path, is not available to base estimation of moisture maximization, a value of 25% for inland areas and relatively lesser values of 10% for Coastal areas may be adopted. These factors may be uniformly applied for the total period of the storm.

6.2.5. Loss Rate

It is recommended that loss rate of 1-2 mm/hr depending upon catchment characteristics and nature of vegetation may be applied. While continuing to apply 1.0 to 2.0 mm/hr., it is essential to check the resulting design storm rainfall and runoff ratio in order to readjust the loss rate such that the losses and consequent runoff volume corresponding to the given rain depths may not become unrealistic for the nature and size of the catchment and none of rainfall increments become less than the loss rate resulting in breaks/lull within the total design rainfall duration.

6.2.6. Temporal Distribution

The temporal distribution of design storm depths may be based on the average distribution of maximum consecutive hour rainfalls worked out from the SRRG data in the region where the data of more than 6 spells of 250 mm is available.

For catchments less than 50 sq.km. the following time distribution pattern is recommended;

R_t = R12 x (t/12)0.3 where,

t = any short time interval in hours within 24 hrs.

R_t = rainfall depth at t hours

R12 = 12 hour area design storm depth of the catchments for any day within the design storm duration

The design hyetograph maybe represented in two bells per day. The combination of the bell arrangement and the arrangement of rainfall increments within each of the bell shaped spells may be representing the maximum flood producing characteristic.

The critical arrangement of increments in each bell is to be such that the time lag between peak intensities of two spells may be minimum. The cumulative pattern of all the increments in the order of their positioning should resemble the natural mass curve pattern as observed by an SRRG of the project region.

While arranging the increments within each spell as mentioned above care may be taken to see that the sum of the consecutive increments in any t-hour within storm duration shall not exceed the t-hour area PMP.

6.3. THE WONOGIRI DAM RESERVOIR (INDONESIA)

6.3.1. Salient Features

The Bengawan Solo River originates on southwest slope of Lawu Mountain in Tertiary Volcanic mountainous area and flows westward along the series of mountains. The Solo River generally takes a northward direction, receiving the Alang River, Temon River, Tirtomoyo River and Keduang River immediately upstream of the Wonogiri Dam. The salient features of the Wonogiri Dam and Reservoir are summarized in Table 6.1 below.

Table 6.1: The Salient Features of the Wonogiri Dam and Reservoir

Dam type	Rockfilll	Normal High Water Level	EL. 136.0 m
Dam height	40 m	Design Flood Water Level	EL. 138.3 m
Crest length	830m	Extra Flood Water Level	EL. 139.1 m
Embankment volume	1,223,300 m^3	Crest Height of Dam	EL. 142.0 m
Catchment area	1,350 km^2	Spillway (Radial gate)	7.5m x 7.8m x 4nos.
Reservoir area	90 km^2	Standard Highest Flood Discharge (60-year flood)	4,000 m^3/s
Gross storage capacity	735 x 106 m^3	Flood outflow discharge	400 m^3/s
Active storage capacity	615 x 106 m^3	Design flood discharge (100-year flood x 1.2)	5,100 m^3/s
Flood control storage capacity	220 x 106 m^3	PMF	9,600 m^3/s
Irrigation & hydro power storage capacity	440 x 106 m^3	Installed capacity	12.4 MW
Sediment storage capacity	120 x 106 m^3	Design head	20.4 m
Sediment deposit level	EL. 127.0 m	Max. discharge	75 m^3/s
Control water level during flood season	EL. 135.3 m	Annual energy output	50,000 MWh

Source: JICA (1978) "Wonogiri Multipurpose Dam Project, Part I Summary Report on Detail Engineering Services, January 1978, Nippon Koei Co., Ltd."

The Wonogiri multipurpose dam (Figure. 6.1 & Figure. 6.6), is the only large dam on the mainstream of the Bengawan Solo River, which is the largest river in the Java with a catchment area of around 16,100 km^2 and a length of about 600 km.

6.3.2. Design Flood of Wonogiri Dam Reservoir

The inflow discharge exceeding 4000 m^3/s is called as a flood in the Wonogiri Dam operation rule. Three design floods had been defined to determine the operating water level and design of spillway and dam main body as shown in the Table 6.2.below.

Table 6.2: Design Floods of Wonogiri Dam Reservoir

Design Flood		Peak Inflow Discharge	Remark
Standard Highest Flood Discharge	(SFHD)	4,000 m^3/s	Project design flood for flood control corresponding to the Recorded maximum flood in 1966 which recurrence interval of 60 years
Spillway Design Flood	(Design Flood)	5,100 m^3/s	1.2 times of 100-year probable flood
Probable Maximum Flood	(PMF)	9,600 m^3/s	(Extraordinary flood)

Source: JICA (1978) "Wonogiri Multipurpose Dam Project, Part I Summary Report on Detail Engineering Services, January 1978, Nippon Koei Co., Ltd."

6.3.3. Present Condition of Hydrological Data Collection

Rainfall Gauging Stations

(1) Daily rainfall data of thirty six (36) rainfall gauging station are available in and around the Wonogiri dam catchments. However, daily rainfall data from 2001 comprises numerous lack of record. Especially from 2004 to 2006, some rainfall stations are not operated. According to the local government staffs recently rainfall gauging station at the Sidoharjo is without gauge keeper because the responsible staff had already retired and no person is available for handing over the duty.

(2) In 1980's the Wonogiri Dam Construction Project had installed 4 rainfall telemetry stations in the dam catchment area. However, there are two problems as follows;

i) Telemetry systems are completely broken so the real time rainfall information does not reach the Wonogiri dam management office.

ii) Record paper for the hourly rainfall recorder is not available. Hence, they reuse a same paper over and over again. It makes too difficult to read the record because of crossing the record lines.

iii) Some rainfall gauging stations are located under the trees. It would cause smaller rainfall evaluation than that of actual volume.

Figure 6.1: Location Map of the Wonogiri Dam Reservoir.

Discharge Measurement

Periodical discharge measurements with river cross section survey are not carried out. The observation works in the Wonogiri dam catchment area are not carried out since 1998. Hence, the water level record can not be translated to the discharge.

During the wet season, a lot of record papers have been damaged by rainfall water. So many charts are not readable because of ink spread due to rainfall water and thin ink condition also.

6.4. PMP ANALYSIS OF WONOGIRI DAM WATERSHED

6.4.1. Thiessen Polygon Map

Thiessen polygon map over the Wonogiri watershed is prepared to estimate mean hourly rainfall over each tributary's basin. Totally 15 rainfall stations are selected to prepare the Thiessen polygon map. Selection of rainfall station is made from the view point of availability of hourly rainfall and distribution of each location. Thiessen polygon map is given below and its weight on each tributary basin is presented in Table 1.3.

Table 6.3: Weightage of Thiessen Polygon of Tributary of Wonogiri Watershed

No.	Rainfall Station	Subbasin					Remnant Area
		Keduang	Tirtomoyo	Temon	Bengawan Solo	Alang River	
3a	Nawangan	0.000	0.000	0.000	0.304	0.223	0.000
1	Pracimantoro PP 115a	0.000	0.000	0.000	0.000	0.554	0.000
5	Plumbon Skt 28	0.000	0.000	0.000	0.000	0.222	0.427
8	Wuryantoro 114b	0.000	0.000	0.000	0.000	0.000	0.390
12	Beji Skat 15	0.000	0.000	0.344	0.035	0.000	0.020
13	Ngancar	0.000	0.068	0.656	0.611	0.000	0.000
16	Tirtomoyo 131a	0.026	0.436	0.000	0.027	0.000	0.000
18	Watugede	0.000	0.409	0.000	0.024	0.000	0.000
19	Nguntoronadi Skt 25	0.014	0.000	0.000	0.000	0.000	0.000
20	Manyaran 114a	0.000	0.000	0.000	0.000	0.000	0.163
25	Jatiroto 130a	0.297	0.087	0.000	0.000	0.000	0.000
27	Jatisrono 131	0.343	0.000	0.000	0.000	0.000	0.000
53	Girimantoro PP 125b	0.197	0.000	0.000	0.000	0.000	0.000
86	Tawangmangu No 130	0.049	0.000	0.000	0.000	0.000	0.000
MD-6	Purwantoro 132	0.072	0.000	0.000	0.000	0.000	0.000
	Total	1.000	1.000	1.000	1.000	1.000	1.000

6.4.2. Basin Mean Hourly Rainfall on each Tributary basin

After the supplementation of hourly rainfall data, basin mean rainfall was estimated by the Thiessen polygon method for each tributary basin.

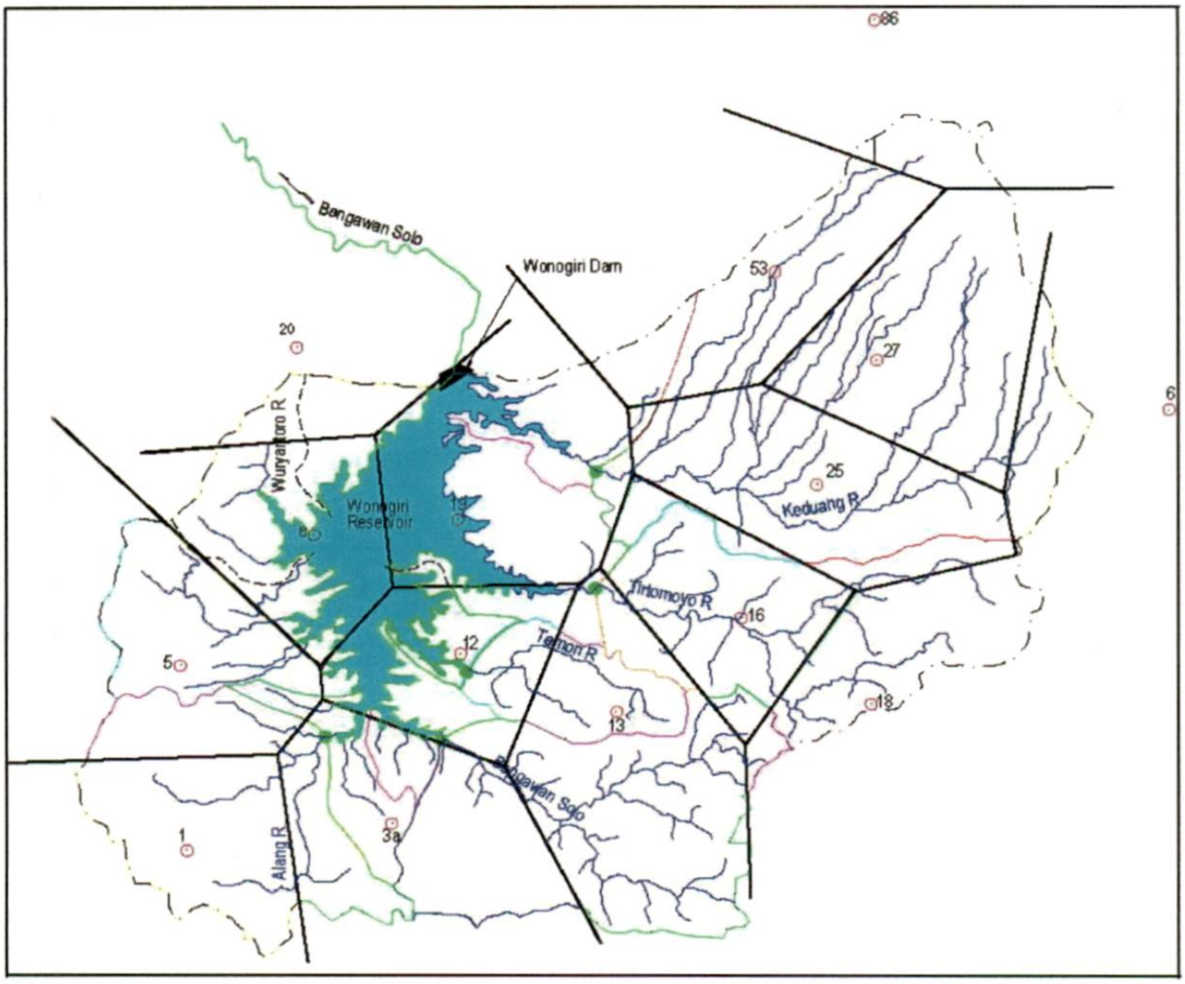


Figure 6.2: Thiessen polygon Map of the Wonogiri Dam Reservoir.

6.4.3. Estimation of Probable Maximum Precipitation

Storm rainfalls stations are appropriate with the Wonogiri Dam Reservoir watershed (Showing in Figure 6.3) have been selected.

Estimation of the PMP is based on Hershfield statistical Method using a series of the annual maximum daily rainfall records. The procedure described below is based on 'Manual for Estimation of Probable Maximum Precipitation' (hereafter referred to as "the Manual") published by World Meteorological Organization (WMO) in 1986.

a) Estimation Method :

The Hershfield's equation is expressed as follows:

$$X_{PMP} = \overline{X}_n + K_m \bullet \sigma_n$$

Where, X_{PMP} : Point value of PMP (mm)

$\overline{X}_n$: Mean annual maximum rainfall (mm)

K_m : Frequency factor-Function of rainfall duration and mean annual maximum rainfall

σ_n : Standard deviation of a series of n annual maximum rainfall.

- Adjustment of X_n and σ_n for Maximum Observed Event

  Such a rare event, called an outlier, may have an appreciable effect on the mean (X_n) and standard deviation (σ_n) of the annual series. The magnitude of the effect is less for long records than for short, and it varies with the rarity of the event, or outlier. This has been studied by Hershfield [1961] using hypothetical series of varying length.

- Adjustment of K_m

  According to the manual, records of 24 hours rainfall for some stations in the climatologically observation programme were used in the determination of an enveloping value of K_m. In the PMP estimation, K_m is largest of all calculated K values for all stations in a given area. The value of K is calculated using the following Eq.

  $$K = (X_m - \overline{X}_{n-m}) / \sigma_{n-m}$$

  where $X_m, \overline{X}_{n-m}, \sigma_{n-m}$, are the highest, mean and standard deviation respectively excluding the X_m value from the series.

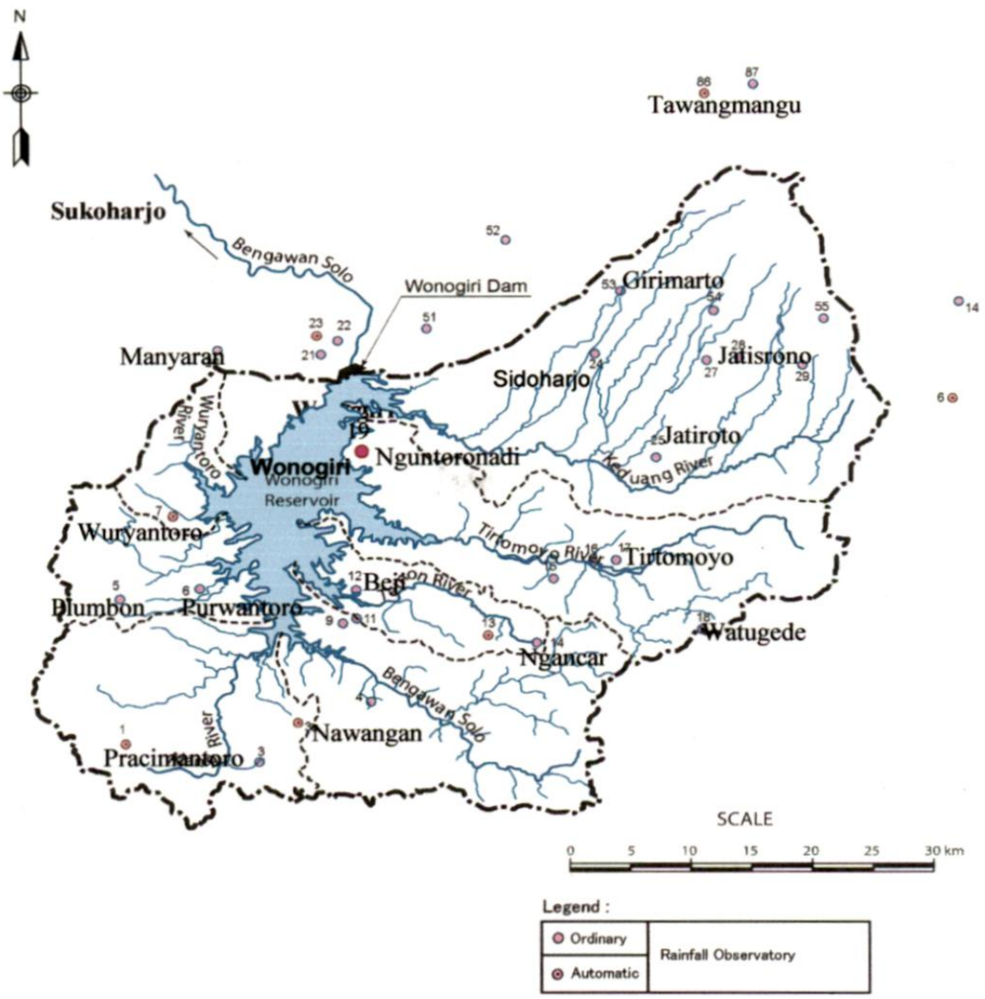
b) Selection of Rainfall Station

Fifteen (15) rainfall stations, (Table 6.4), are selected to estimate point value of PMP because of following reasons (See Figure 6.3 for location of the stations).

- The rainfall stations are located in Wonogiri reservoir watershed which has highest annual rainfall in the reservoir watershed. Point value of PMP may mostly occur in such area from view point of rainfall record.
- Data availability is sufficient.

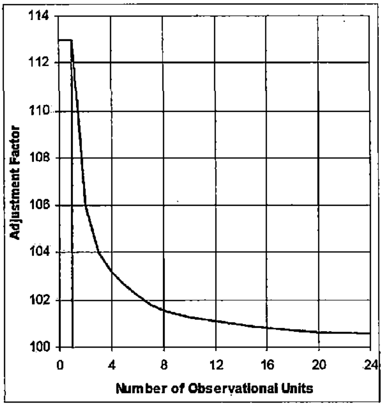
Table 6.4: Rainfall Data on Selected Stations

No.	Rainfall Station	Observation Period	The number of Annual Maximum Rainfall Data, n
1	Nawangan	1975 - 2005	30
2	Ngancar	1975 - 2005	30
3	Tawangmangu	1975 - 2005	30
4	Purwantoro	1975 - 2005	30
5	Jatiroto	1976 - 2004	28
6	Jatisrono	1978- 2005	27
7	Nguntoronadi	1977- 2002	25
8	Tirtomoyo	1976- 2003	26
9	Beji	1976- 2003	27
10	Plumbon	1976- 2004	28
11	Pracimantoro	1976- 2004	28
12	Wuryantoro	1976- 2004	28
13	Girimarto	1976- 2004	28
14	Manyaran	1976- 2004	28
15	Watugede	1976- 2004	28



Source : (1) Isohyetal Map: Survey and Study For The Development of Sala River Basin, Jan. 1974, OTCA Japan, (Supporting Report Part-I: Hydrology),
(2) Monthly Mean Rainfall Data : PBS

Figure 6.3: Location of the Rainfall Station



Source: Manual for Estimation of Probable Maximum Precipitation, Second Edition, 1986, WMO

X_m : Point value of PMP (mm)

c) Clock Time Adjustment Factor (fo)

Since the recorded daily rainfall is computed based on the single fixed observation time interval (8 a.m to 8 p.m the next day), the PMP value yielded by the statistical procedure should be increased multiplying by the adjustment factor (fo). The adjustment factor curve is presented by Dr. Hershfield. Applying that the number of observation units is equal to 1, the fo value is obtained to be 113%. Finally, the point PMP is adjusted using the adjustment factor fo as follows:

PMP = fo x X_m

where, fo : Adjustment factor (= 1.13)

d) Area Reduction Factor (ARF)

Area reduction of rain storms within the Wonogiri dam catchment was analyzed based on the relationship between the basin mean daily rainfall (more than 40 mm) and its maximum point rainfall. The area reduction factor means the ratio of basin mean rainfall to point rainfall.

ARF = R_b / Rmax

where, ARF : Area reduction factor

R_b : Major basins mean daily rainfall (mm)

R_{max} : Maximum point rainfall (mm).

Table 6.5: Relationships between Maximum Daily Rainfalls with Basin Mean Rainfall

Year	Date	Basin Average Rainfall (mm)	Max. Point Rainfall (mm)	Reduction Factor
1975	21-Mar	56.5	128.0	0.441
	27-Mar	55.9	133.0	0.420
	31-Mar	56.6	135.0	0.419
	12-Sep	69.0	161.4	0.427
1977	19-Jan	70.9	137.0	0.518
	24-Jan	51.5	124.0	0.415
1985	14-Feb	58.6	166.0	0.353
	5-Mar	52.1	103.0	0.505
	7-Mar	70.1	125.0	0.561
	8-Mar	85.6	214.0	0.400
1988	5-Feb	105.2	257.0	0.409
1999	11-Dec	52.6	123.0	0.428
2003	4-Jan	56.1	125.0	0.449
2004	3-Dec	71.9	102.0	0.705
2005	14-Mar	51.7	125.0	0.414

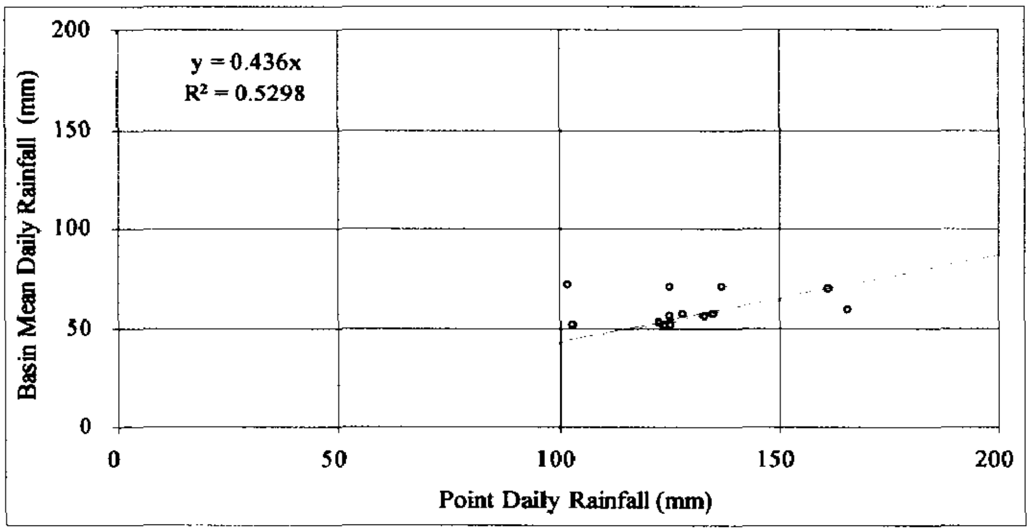


Figure 6.4: Point Maximum Daily Rainfalls and Basin Mean Rainfall Relationships

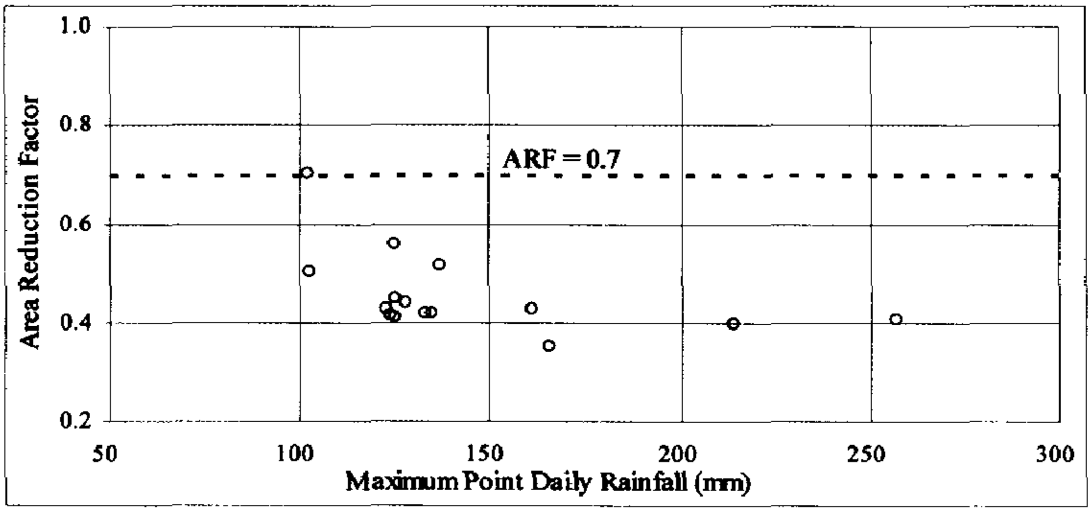


Figure 6.5: Area Reduction Factor

Table 6.6: Adjustment of (X_n) Mean Annual Maximum Rainfall

Rainfall Station	Nguntoro nadi	Tirtomoyo	Beji	Plumbon	Pracimant oro	Wuryanto ro	Girimarto	Manyaran	Watugede
X_n	81.9	83.7	75.6	88.0	82.0	88.8	97.8	97.3	88.8
n	26	27	28	29	29	29	28	29	29
X_{n-m}	80.4	80.9	69.7	86.5	80.1	87.1	95.0	94.2	86.9
X_{n-m} / X_n	0.98	0.97	0.92	0.98	0.98	0.98	0.97	0.97	0.98
X_n Adjusment Factor	0.99	0.99	1.00	1.00	1.00	1.00	1.01	0.99	1.00
Adjusment Factor	1.01	1.01	1.01	1.01	1.01	1.01	1.02	1.02	1.02
Adjusted X_n	**81.9**	**83.7**	**76.4**	**88.9**	**82.8**	**89.7**	**100.7**	**97.7**	**90.1**

Table 6.7: Adjustment of (S_n) Standard Deviation

Rainfall Station	Nguntoro nadi	Tirtomoyo	Beji	Plumbon	Pracimant oro	Wuryanto ro	Girimarto	Manyaran	Watugede
S_n	20.1	27.9	41.3	18.4	18.6	18.3	34.2	25.5	25.5
n	26	27	28	29	29	29	28	29	29
S_{n-m}	18.9	24.4	27.9	16.9	16.1	16.2	30.7	20.0	23.1
S_{n-m} / S_n	0.94	0.87	0.68	0.92	0.86	0.89	0.90	0.79	0.90
S_n Adjusment Factor	0.87	0.75	0.86	0.87	0.75	0.86	0.87	0.75	0.86
Adjusment Factor	1.05	1.04	1.04	1.04	1.04	1.04	1.05	1.04	1.04
Adjusted S_n	**18.3**	**21.8**	**36.9**	**16.7**	**14.5**	**16.3**	**31.2**	**19.9**	**22.8**

Table 6.8: Adjustment of (K_m) Function of Rainfall Duration and Mean Annual Max. Rainfall

Rainfall Station	Nguntoro nadi	Tirtomoyo	Beji	Plumbon	Pracimant oro	Wuryanto ro	Girimarto	Manyaran	Watugede
X_n	81.9	83.7	75.6	88.0	82.0	88.8	97.8	97.3	88.8
n	26	27	28	29	29	29	28	29	29
K_m	2.09	3.12	5.95	2.57	3.35	3.01	2.50	4.44	2.43
Adjusted K_m	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**

As a result, the largest 24 hour duration PMP of 234.3 mm at Beji is selected as a 24 hour PMP on the Wonogiri dam watershed.

Table 6.9: Calculation of 24 hour Probable Maximum Precipitation

Annual Maximum Daily Rainfall X (mm)									
YEAR	Nguntoronadi	Tirtomoyo	Beji	Plumbon	Pracimantoro	Wuryantoro	Girimarto	Manyaran	Watugede
1976		50.0	82.0	77.0	**134.0**	97.0	**172.0**	100.0	104.0
1977	114.0	105.0	80.0	100.0	83.0	95.0	78.0	50.0	72.0
1978	90.0	61.0	75.0	81.0	108.0	29.0	140.0	70.0	73.0
1979	70.0	113.0	85.0	85.0	44.0	88.0	88.0	87.0	66.0
1980	72.0	68.0	65.0	51.0	50.0	75.0	85.0	101.0	67.0
1981	75.0	69.0	175.0	57.0	74.0	82.0	86.0	130.0	75.0
1982	71.0	67.0	65.0	64.0	76.0	88.0	83.0	70.0	75.0
1983	91.0	68.0	**236.0**	97.0	91.0	**136.0**	69.0	113.0	72.0
1984	71.0	75.0	47.0	89.0	68.0	96.0	107.0	**183.0**	75.0
1985	103.0	67.0	92.0	123.0	69.0	115.0	120.0	81.0	84.0
1986	85.0	68.0	60.0	85.0	103.0	104.0	79.0	84.0	75.0
1987	57.0	61.0	67.0	78.0	97.0	85.0	83.0	95.0	92.0
1988	104.0	90.0	130.0	98.0	105.0	109.0	155.0	107.0	139.0
1989	70.0	76.0	56.0	116.0	85.0	100.0	92.0	98.0	68.0
1990	70.0	69.0	45.0	85.0	105.0	66.0	106.0	73.0	135.0
1991	86.0	85.0	67.0	**130.0**	77.0	100.0	70.0	100.0	91.0
1992	84.0	53.0	57.0	99.0	91.0	80.0		113.0	105.0
1993	109.0		38.0	102.0	102.0	90.0	95.0	99.0	70.0
1994	94.0	87.0	40.0	86.0	61.0	82.0	126.0	57.0	89.0
1995	**120.0**	115.0	75.0	86.0	70.0	82.0	95.0	94.0	95.0
1996	98.0	84.0	65.0	91.0	81.0	96.0	95.0	75.0	94.0
1997	105.0	53.0	40.0	67.0	59.0	73.0	90.0	122.0	97.0
1998	83.0	80.0	80.0	85.0	74.0	93.0	114.0	102.0	94.0
1999	60.0	**157.0**	55.0	89.0	78.0	83.0	108.0	107.0	**143.0**
2000	45.0	101.0	70.0	87.0	85.0	77.0	102.0	110.0	143.0
2001	45.0	153.0	60.0	68.0	85.0	73.0	85.0	95.0	109.0
2002	58.0	60.0	60.0	64.0	71.0	79.0	93.0	68.0	67.0
2003		125.0	50.0	114.0	75.0	95.0	91.0	110.0	50.0
2004				99.0	77.0	107.0	84.0	127.0	55.0
2005									
n	26	27	28	29	29	29	28	29	29
X_n (Mean)	81.9	83.7	75.6	88.0	82.0	88.8	97.8	97.3	88.8
X_m (Max.)	120.0	157.0	236.0	130.0	134.0	136.0	172.0	183.0	143.0
X_{n-m}	80.4	80.9	69.7	86.5	80.1	87.1	95.0	94.2	86.9
X_{n-m} / X_n	0.98	0.97	0.92	0.98	0.98	0.98	0.97	0.97	0.98
X_n Adjustment Factor :	0.99	0.99	1.00	1.00	1.00	1.00	1.01	0.99	1.00
Adjustment Factor :	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
Adjusted X_n :	81.9	83.7	76.4	88.9	82.8	89.7	99.7	96.8	89.7

Table 6.9: Calculation of 24 hours Probable Maximum Precipitation (Contd)

	x^2								
YEAR	Nguntoronadi	Tirtomoyo	Beji	Plumbon	Pracimantoro	Wuryantoro	Girimarto	Manyaran	Watugede
1976		2,500	6,724	5,929	**17,956**	9,409	**29,584**	10,000	
1977	12,996	11,025	6,400	10,000	6,889	9,025	6,084	2,500	
1978	8,100	3,721	5,625	6,561	11,664	841	19,600	4,900	5,329
1979	4,900	12,769	7,225	7,225	1,936	7,744	7,744	7,569	4,356
1980	5,184	4,624	4,225	2,601	2,500	5,625	7,225	10,201	4,489
1981	5,625	4,761	30,625	3,249	5,476	6,724	7,396	16,900	5,625
1982	5,041	4,489	4,225	4,096	5,776	7,744	6,889	4,900	5,625
1983	8,281	4,624	**55,696**	9,409	8,281	**18,496**	4,761	12,769	5,184
1984	5,041	5,625	2,209	7,921	4,624	9,216	11,449	**33,489**	5,625
1985	10,609	4,489	8,464	15,129	4,761	13,225	14,400	6,561	7,056
1986	7,225	4,624	3,600	7,225	10,609	10,816	6,241	7,056	5,625
1987	3,249	3,721	4,489	6,084	9,409	7,225	6,889	9,025	8,464
1988	10,816	8,100	16,900	9,604	11,025	11,881	24,025	11,449	19,321
1989	4,900	5,776	3,136	13,456	7,225	10,000	8,464	9,604	4,624
1990	4,900	4,761	2,025	7,225	11,025	4,356	11,236	5,329	18,225
1991	7,396	7,225	4,489	**16,900**	5,929	10,000	4,900	10,000	8,281
1992	7,056	2,809	3,249	9,801	8,281	6,400		12,769	11,025
1993	11,881		1,444	10,404	10,404	8,100	9,025	9,801	4,900
1994	8,836	7,569	1,600	7,396	3,721	6,724	15,876	3,249	7,921
1995	**14,400**	13,225	5,625	7,396	4,900	6,724	9,025	8,836	9,025
1996	9,604	7,056	4,225	8,281	6,561	9,216	9,025	5,625	8,836
1997	11,025	2,809	1,600	4,489	3,481	5,329	8,100	14,884	9,409
1998	6,889	6,400	6,400	7,225	5,476	8,649	12,996	10,404	8,836
1999	3,600	**24,649**	3,025	7,921	6,084	6,889	11,664	11,449	**20,449**
2000	2,025	10,201	4,900	7,569	7,225	5,929	10,404	12,100	20,449
2001	2,025	23,409	3,600	4,624	7,225	5,329	7,225	9,025	11,881
2002	3,364	3,600	3,600	4,096	5,041	6,241	8,649	4,624	4,489
2003		15,625	2,500	12,996	5,625	9,025		12,100	2,500
2004				9,801	5,929	11,449		16,129	3,025
2005									
	7,114.2	7,784.7	7,422.3	8,090.1	7,070.3	8,218.3	10,726.0	10,112.0	8,539.8
S_n (Mean)	20.1	27.9	41.3	18.4	18.6	18.3	34.2	25.5	25.5
S_{n-m}	18.9	24.4	27.9	16.9	16.1	16.2	30.7	20.0	23.1
S_{n-m} / S_n	0.94	0.87	0.68	0.92	0.86	0.89	0.90	0.79	0.90
S_n Adjustment Factor :	0.87	0.75	0.86	0.87	0.75	0.86	0.87	0.75	0.86
Adjustment Factor :	1.05	1.04	1.04	1.04	1.04	1.04	1.05	1.04	1.04
Adjusted S_n :	18.3	21.8	36.9	16.7	14.5	16.3	31.2	19.9	22.8
K_m :	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**	**5.95**	**5.95 #**
	2.09	3.12	5.95	2.57	3.35	3.01	2.50	4.44	2.43
Unadjusted PMP :	191.0	213.2	296.2	188.2	169.2	187.0	285.6	215.1	225.6
Adjustment factor (f_0)	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13
Adjustment of PMP :	215.9	240.9	334.7	212.7	191.2	211.3	322.8	243.1	254.9
Adjustment of Point PMP to Wonogiri Watershed :	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
Adjusted 48 hours PMP for Wonogiri Watershed :	**151.1**	**168.7**	**234.3**	**148.9**	**133.8**	**147.9**	**225.9**	**170.1**	**178.5**

By applying the Hershfield statistical method the depth of the PMP in any duration for each rainfall station can be obtained. Following are the result of estimation of depth of PMP in 48-hour, 72-hour, 96-hour duration, and 168-hour duration in mm.

Table 6.10: Calculation of 48-hour Probable Maximum Precipitation

Annual Maximum Daily Rainfall X (mm)									
YEAR	Nawangan	Ngancar	Wonogiri	Wonogiri Dam	Tawangmangu	Purwantoro	Sidoharjo 125	Jatiroto 130a	Jatisrono 131
1975	165.3	**206.0**	**169.0**	147.4	174.0	177.0			
1976	136.0	128.0	117.0	102.0	168.0	103.0		134.0	
1977	109.9	140.0	152.0	117.7	74.0	107.0	115.0	107.0	
1978	99.0	91.0	94.7	90.9	100.0	90.0	104.0	91.0	95.0
1979	74.0	136.0	100.0	80.2	129.0	104.0	89.0	86.0	70.0
1980	96.0	117.0	165.0	143.9	115.0	91.0	76.0	78.0	75.0
1981	117.0	110.0	80.0	60.2	124.0	105.0	186.0	100.0	103.0
1982	97.0	97.0	77.0	79.4	151.0	65.0	104.0	87.0	74.0
1983	55.0	93.0	71.0	101.9	161.0	96.0	91.0	80.0	109.0
1984	99.0	104.0	128.0	128.5	109.0	212.0	102.0	185.0	128.0
1985	144.0	187.0	129.2	111.5	187.0	142.0	**188.0**	**186.0**	**170.0**
1986	70.0	142.0	124.0	**248.2**	120.0	95.0	81.0	167.0	89.0
1987	118.0	204.0	122.5	115.1	178.6	124.0	130.0	110.0	87.0
1988	114.0	139.0	123.1	121.5	**234.0**	105.0	163.0	112.0	125.0
1989	76.0	115.0	74.4	73.4	155.0	120.0	151.0	84.0	103.0
1990	76.0	79.0	85.8	77.2	126.0	87.0	128.0	100.0	93.0
1991	90.0	138.0	161.0	96.8	132.0	101.0	106.0	99.0	94.0
1992	108.0	104.0	109.0	133.8	154.0	133.0	109.0	110.0	119.0
1993	**183.4**	95.0	113.0	198.0	198.0	134.0	113.0	86.0	148.0
1994	102.0	146.0	98.0	96.5	124.0	89.0	135.0	92.0	106.0
1995	156.0	69.0	156.0	124.0	183.0	120.0	118.0	137.0	103.0
1996	97.0	97.0	74.0	117.4	149.0	169.0	70.0	147.0	118.0
1997	70.0	89.1	123.0	162.5	139.0	75.0	102.0	78.0	81.0
1998	104.0	101.0	116.0	88.5	124.0	135.0	79.0	87.0	112.0
1999	107.0	94.0	162.0	94.2	133.0	82.0	109.0	110.0	118.0
2000	100.0	85.0	109.0	86.0	163.0	**229.0**	92.0	116.0	93.0
2001	87.0	102.0	82.0	120.0	144.0	87.0	88.0	102.0	82.0
2002	71.0	103.0	74.0	152.5	145.0	132.0	82.0	90.0	69.0
2003	80.0	77.0	105.0	110.5	114.0	78.0		98.0	86.0
2004	45.0	88.0	92.0	116.0	127.0	0.0		147.0	96.0
2005	93.0		82.0	92.5	182.0	98.0			
n	31	30	31	31	31	31	26	29	27
X_n (Mean)	101.3	115.9	111.9	115.8	145.7	112.4	112.0	110.6	101.7
X_m (Max.)	183.4	206.0	169.0	248.2	234.0	229.0	188.0	186.0	170.0
X_{n-m}	98.5	112.8	110.0	111.3	142.8	108.5	108.9	107.9	99.1
X_{n-m} / X_n	0.97	0.97	0.98	0.96	0.98	0.97	0.97	0.98	0.97
X_n Adjustment Factor :	0.99	0.99	1.00	1.00	1.00	1.00	1.01	0.99	1.00
Adjustment Factor :	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
Adjusted X_n :	101.3	115.9	113.0	116.9	147.2	113.5	114.2	110.0	102.7

Table 6.10: Calculation of 48-hour Probable Maximum Precipitation (contd)

	x^2								
YEAR	Nawangan	Ngancar	Wonogiri	Wonogiri Dam	Tawangmangu	Puwantoro	Siddharjo 125	Jatiroto 130a	Jatisrono 131
1975	27,316	**42,436**	**28,561**	21,727	30,276	31,329			
1976	18,496	16,384	13,689	10,414	28,224	10,609		17,956	
1977	12,081	19,600	23,104	13,864	5,476	11,449	13,225	11,449	
1978	9,801	8,281	8,960	8,255	10,000	8,100	10,816	8,281	9,025
1979	5,476	18,496	10,000	6,439	16,641	10,816	7,921	7,396	4,900
1980	9,216	13,689	27,225	20,711	13,225	8,281	5,776	6,084	5,625
1981	13,689	12,100	6,400	3,622	15,376	11,025	34,596	10,000	10,609
1982	9,409	9,409	5,929	6,300	22,801	4,225	10,816	7,569	5,476
1983	3,025	8,649	5,041	10,384	25,921	9,216	8,281	6,400	11,881
1984	9,801	10,816	16,384	16,512	11,881	44,944	10,404	34,225	16,384
1985	20,736	34,969	16,683	12,432	34,969	20,164	**35,344**	**34,596**	**28,900**
1986	4,900	20,164	15,376	**61,603**	14,400	9,025	6,561	27,889	7,921
1987	13,924	41,616	15,000	13,248	31,895	15,376	16,900	12,100	7,569
1988	12,996	19,321	15,149	14,762	**54,756**	11,025	26,569	12,544	15,625
1989	5,776	13,225	5,529	5,388	24,025	14,400	22,801	7,056	10,609
1990	5,776	6,241	7,362	5,960	15,876	7,569	16,384	10,000	8,649
1991	8,100	19,044	25,921	9,370	17,424	10,201	11,236	9,801	8,836
1992	11,664	10,816	11,881	17,902	23,716	17,689	11,881	12,100	14,161
1993	**33,632**	9,025	12,769	39,204	39,204	17,956	12,769	7,396	21,904
1994	10,404	21,316	9,604	9,312	15,376	7,921	18,225	8,464	11,236
1995	24,336	4,761	24,336	15,376	33,489	14,400	13,924	18,769	10,609
1996	9,409	9,409	5,476	13,783	22,201	28,561	4,900	21,609	13,924
1997	4,900	7,935	15,129	26,406	19,321	5,625	10,404	6,084	6,561
1998	10,816	10,201	13,456	7,832	15,376	18,225	6,241	7,569	12,544
1999	11,449	8,836	26,244	8,874	17,689	6,724	11,881	12,100	13,924
2000	10,000	7,225	11,881	7,396	26,569	**52,441**	8,464	13,456	8,649
2001	7,569	10,404	6,724	14,400	20,736	7,569	7,744	10,404	6,724
2002	5,041	10,609	5,476	23,256	21,025	17,424	6,724	8,100	4,761
2003	6,400	5,929	11,025	12,210	12,996	6,084		9,604	7,396
2004	2,025	7,744	8,464	13,456	16,129	0		21,609	9,216
2005	8,649		6,724	8,556	33,124	9,604			
	11,187.5	14,621.7	13,403.3	14,805.0	22,261.8	14,932.6	13,491.8	13,124.5	10,874.7
S_n (Mean)	30.5	34.6	29.7	37.5	32.2	47.9	30.9	30.0	23.0
S_{n-m}	27.0	30.8	28.3	29.1	28.3	43.1	27.5	26.9	19.1
S_{n-m} / S_n	0.89	0.89	0.95	0.78	0.88	0.90	0.89	0.90	0.83
S_n Adjustment Factor :	0.87	0.75	0.86	0.87	0.75	0.86	0.87	0.75	0.86
Adjustment Factor :	1.05	1.04	1.04	1.04	1.04	1.04	1.05	1.04	1.04
Adjusted S_n :	27.9	27.0	26.6	33.9	25.1	42.8	28.3	23.4	20.6
K_m :	**4.70**	**4.70**	**4.70**	**4.70**	**4.70**	**4.70**	**4.70**	**4.70**	**4.70 #**
	3.14	3.03	2.08	4.70	3.23	2.79	2.88	2.90	3.71
Unadjusted PMP :	232.1	242.5	237.9	276.3	265.0	314.7	246.9	220.0	199.5
Adjustment factor (f_0)	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13
Adjustment of PMP :	262.3	274.1	268.8	312.2	299.4	355.6	279.0	248.7	225.4
Adjustment of Point PMP to Wonogiri Watershed :	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
Adjusted 48 hours PMP for Wonogiri Watershed :	**183.6**	**191.8**	**188.2**	**218.5**	**209.6**	**248.9**	**195.3**	**174.1**	**157.8**

Table 6.11: Calculation of 72-hour Probable Maximum Precipitation

Annual Maximum Daily Rainfall X (mm)									
YEAR	Nguntoronadi	Tirtomoyo	Beji	Plumbon	Pracimantoro	Wuryantoro	Girimarto	Manyaran	Watugede
1976		76.0	123.0	120.0	142.0	118.0	203.0	220.0	122.0
1977	142.0	172.0	215.0	**212.0**	158.0	153.0	89.0	64.0	123.0
1978	129.0	115.0	87.0	81.0	**189.0**	29.0	160.0	70.0	130.0
1979	97.0	123.0	116.0	120.0	99.0	152.0	103.0	100.0	118.0
1980	139.0	106.0	95.0	99.0	75.0	128.0	180.0	123.0	103.0
1981	128.0	164.0	197.0	95.0	115.0	131.0	113.0	173.0	169.0
1982	121.0	158.0	109.0	87.0	91.0	115.0	125.0	174.0	192.0
1983	109.0	97.0	**381.0**	102.0	134.0	136.0	100.0	165.0	127.0
1984	76.0	106.0	98.0	107.0	105.0	120.0	145.0	**271.0**	119.0
1985	**184.0**	198.0	172.0	191.0	100.0	187.0	180.0	184.0	214.0
1986	121.0	95.0	109.0	104.0	181.0	129.0	119.0	104.0	129.0
1987	80.0	112.0	116.0	132.0	127.0	132.0	125.0	121.0	145.0
1988	137.0	200.0	234.0	172.0	128.0	**194.0**	172.0	237.0	212.0
1989	134.0	128.0	100.0	160.0	114.0	121.0	92.0	151.0	153.0
1990	84.0	131.0	80.0	129.0	130.0	99.0	122.0	131.0	**236.0**
1991	105.0	150.0	116.0	182.0	88.0	135.0	85.0	190.0	111.0
1992	118.0	93.0	86.0	129.0	125.0	114.0		148.0	130.0
1993	134.0		68.0	208.0	118.0	130.0	149.0	183.0	135.0
1994	178.0	166.0	72.0	128.0	112.0	129.0	160.0	99.0	165.0
1995	140.0	149.0	89.0	112.0	152.0	130.0	169.0	117.0	144.0
1996	113.0	135.0	71.0	116.0	126.0	110.0	148.0	98.0	136.0
1997	161.0	100.0	50.0	70.0	87.0	99.0	110.0	122.0	148.0
1998	121.0	118.0	115.0	144.0	88.0	191.0	164.0	119.0	160.0
1999	89.0	**223.0**	115.0	89.0	148.0	116.0	157.0	120.0	209.0
2000	87.0	160.0	140.0	134.0	130.0	108.0	**223.0**	195.0	152.0
2001	100.0	207.0	85.0	117.0	112.0	101.0	146.0	130.0	161.0
2002	85.0	70.0	115.0	99.0	129.0	122.0	178.0	135.0	85.0
2003		140.0	110.0	147.0	141.0	142.0	136.0	110.0	65.0
2004				142.0	142.0	107.0	124.0	189.0	56.0
n	26	27	28	29	29	29	28	29	29
X_n (Mean)	119.7	136.7	123.7	128.6	123.7	126.8	140.6	146.3	144.6
X_m (Max.)	184.0	223.0	381.0	212.0	189.0	194.0	223.0	271.0	236.0
X_{n-m}	117.1	133.4	114.2	125.6	121.3	124.4	137.5	141.9	141.3
X_{n-m} / X_n	0.98	0.98	0.92	0.98	0.98	0.98	0.98	0.97	0.98
X_n Adjustment Factor :	0.99	0.99	1.00	1.00	1.00	1.00	1.01	0.99	1.00
Adjustment Factor :	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
Adjusted X_n :	119.7	136.7	125.0	129.8	124.9	128.1	143.4	145.6	146.0

Table 6.11: Calculation of 72-hour Probable Maximum Precipitation (contd)

	x^2								
YEAR	Nguntoronadi	Tirtomoyo	Beji	Plumbon	Pracimantoro	Wuryantoro	Girimarto	Manyaran	Watugede
1976		5,776	15,129	14,400	20,164	13,924	41,209	48,400	
1977	20,164	29,584	46,225	**44,944**	24,964	23,409	7,921	4,096	
1978	16,641	13,225	7,569	6,561	**35,721**	841	25,600	4,900	16,900
1979	9,409	15,129	13,456	14,400	9,801	23,104	10,609	10,000	13,924
1980	19,321	11,236	9,025	9,801	5,625	16,384	32,400	15,129	10,609
1981	16,384	26,896	38,809	9,025	13,225	17,161	12,769	29,929	28,561
1982	14,641	24,964	11,881	7,569	8,281	13,225	15,625	30,276	36,864
1983	11,881	9,409	**145,161**	10,404	17,956	18,496	10,000	27,225	16,129
1984	5,776	11,236	9,604	11,449	11,025	14,400	21,025	**73,441**	14,161
1985	**33,856**	39,204	29,584	36,481	10,000	34,969	32,400	33,856	45,796
1986	14,641	9,025	11,881	10,816	32,761	16,641	14,161	10,816	16,641
1987	6,400	12,544	13,456	17,424	16,129	17,424	15,625	14,641	21,025
1988	18,769	40,000	54,756	29,584	16,384	**37,636**	29,584	56,169	44,944
1989	17,956	16,384	10,000	25,600	12,996	14,641	8,464	22,801	23,409
1990	7,056	17,161	6,400	16,641	16,900	9,801	14,884	17,161	**55,696**
1991	11,025	22,500	13,456	33,124	7,744	18,225	7,225	36,100	12,321
1992	13,924	8,649	7,396	16,641	15,625	12,996		21,904	16,900
1993	17,956		4,624	43,264	13,924	16,900	22,201	33,489	18,225
1994	31,684	27,556	5,184	16,384	12,544	16,641	25,600	9,801	27,225
1995	19,600	22,201	7,921	12,544	23,104	16,900	28,561	13,689	20,736
1996	12,769	18,225	5,041	13,456	15,876	12,100	21,904	9,604	18,496
1997	25,921	10,000	2,500	4,900	7,569	9,801	12,100	14,884	21,904
1998	14,641	13,924	13,225	20,736	7,744	36,481	26,896	14,161	25,600
1999	7,921	**49,729**	13,225	7,921	21,904	13,456	24,649	14,400	43,681
2000	7,569	25,600	19,600	17,956	16,900	11,664	**49,729**	38,025	23,104
2001	10,000	42,849	7,225	13,689	12,544	10,201	21,316	16,900	25,921
2002	7,225	4,900	13,225	9,801	16,641	14,884	31,684	18,225	7,225
2003		19,600	12,100	21,609	19,881	20,164		12,100	4,225
2004				20,164	20,164	11,449		35,721	3,136
	15,120.4	20,278.0	19,559.2	17,837.5	16,003.3	17,031.7	21,697.7	23,718.7	22,717.0
S_n (Mean)	28.2	39.7	65.2	36.2	26.7	30.8	44.1	48.1	42.5
S_{n-m}	25.6	36.7	43.2	33.2	24.1	38.7	40.8	42.7	38.4
S_{n-m} / S_n	0.91	0.92	0.66	0.92	0.90	1.26	0.93	0.89	0.90
S_n Adjustment Factor :	0.87	0.75	0.86	0.87	0.75	0.86	0.87	0.75	0.86
Adjustment Factor :	1.05	1.04	1.04	1.04	1.04	1.04	1.05	1.04	1.04
Adjusted S_n :	25.7	31.0	58.3	32.8	20.8	27.5	40.2	37.5	38.1
K_m :	**6.17**	**6.17**	**6.17**	**6.17**	**6.17**	**6.17**	**6.17**	**6.17**	**6.17** #
	2.62	2.44	6.17	2.60	2.81	1.80	2.09	3.03	2.47
Unadjusted PMP :	278.6	328.1	485.0	332.1	253.4	297.9	391.7	377.0	380.9
Adjustment factor (f_0)	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13
Adjustment of PMP :	314.8	370.7	548.0	375.3	286.3	336.6	442.7	426.1	430.4
Adjustment of Point PMP to Wonogiri Watershed :	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
Adjusted 48 hours PMP for Wonogiri Watershed :	**220.3**	**259.5**	**383.6**	**262.7**	**200.4**	**235.6**	**309.9**	**298.2**	**301.3**

Table 6.12: Calculation of 96-hour Probable Maximum Precipitation

YEAR	Annual Maximum Daily Rainfall X (mm)								
	Nguntoronadi	Tirtomoyo	Beji	Plumbon	Pracimantoro	Wuryantoro	Girimarto	Manyaran	Watugede
1976		99.0	182.0	134.0	162.0	164.0	203.0	244.0	158.0
1977	148.0	230.0	231.0	186.0	155.0	152.0	108.0	83.0	97.0
1978	159.0	105.0	105.0	104.0	189.0	29.0	169.0	78.0	142.0
1979	119.0	123.0	152.0	110.0	121.0	152.0	106.0	110.0	142.0
1980	103.0	145.0	105.0	79.0	102.0	135.0	150.0	172.0	151.0
1981	165.0	231.0	211.0	126.0	113.0	182.0	158.0	184.0	185.0
1982	121.0	158.0	102.0	93.0	111.0	100.0	136.0	174.0	208.0
1983	109.0	130.0	**421.0**	131.0	164.0	160.0	128.0	178.0	183.0
1984	84.0	133.0	120.0	157.0	105.0	168.0	147.0	271.0	176.0
1985	**290.0**	244.0	256.0	**246.0**	111.0	182.0	277.0	284.0	**255.0**
1986	151.0	132.0	139.0	116.0	**237.0**	129.0	141.0	115.0	193.0
1987	126.0	168.0	151.0	111.0	136.0	134.0	161.0	146.0	162.0
1988	258.0	200.0	258.0	172.0	128.0	194.0	199.0	**320.0**	238.0
1989	103.0	142.0	106.0	120.0	114.0	160.0	92.0	104.0	113.0
1990	101.0	117.0	80.0	129.0	170.0	100.0	153.0	150.0	242.0
1991	170.0	150.0	158.0	182.0	99.0	155.0	85.0	190.0	109.0
1992	123.0	82.0	106.0	140.0	122.0	125.0		171.0	176.0
1993	130.0		68.0	140.0	118.0	130.0	165.0	183.0	161.0
1994	161.0	128.0	105.0	163.0	153.0	154.0	**227.0**	105.0	179.0
1995	171.0	162.0	109.0	132.0	131.0	**200.0**	181.0	137.0	174.0
1996	131.0	162.0	83.0	161.0	147.0	111.0	146.0	124.0	150.0
1997	161.0	102.0	60.0	75.0	87.0	107.0	121.0	150.0	148.0
1998	126.0	137.0	115.0	114.0	133.0	126.0	185.0	102.0	135.0
1999	116.0	179.0	135.0	123.0	113.0	140.0	187.0	120.0	210.0
2000	117.0	140.0	170.0	142.0	167.0	111.0	160.0	140.0	190.0
2001	140.0	**267.0**	105.0	117.0	128.0	140.0	158.0	130.0	220.0
2002	115.0	70.0	135.0	110.0	154.0	125.0	130.0	122.0	75.0
2003		223.0	140.0	139.0	185.0	177.0	189.0	110.0	70.0
2004				171.0	144.0	164.0	124.0	227.0	107.0
n	26	27	28	29	29	29	28	29	29
X_n (Mean)	142.2	154.0	146.7	135.3	137.9	141.6	154.8	159.4	166.4
X_m (Max.)	290.0	267.0	421.0	246.0	237.0	200.0	277.0	320.0	255.0
X_{n-m}	136.3	149.7	136.6	131.3	134.4	139.5	150.3	153.7	163.3
X_{n-m} / X_n	0.96	0.97	0.93	0.97	0.97	0.99	0.97	0.96	0.98
X_n Adjustment Factor :	0.99	0.99	1.00	1.00	1.00	1.00	1.01	0.99	1.00
Adjustment Factor :	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
Adjusted X_n :	142.2	154.0	148.2	136.6	139.3	143.0	157.9	158.6	168.1

Table 6.12: Calculation of 96-hour Probable Maximum Precipitation (contd)

	x^2								
YEAR	Nguntoronadi	Tirtomoyo	Beji	Plumbon	Pracimantoro	Wuryantoro	Girimarto	Manyaran	Watugede
1976		9,801	33,124	17,956	26,244	26,896	41,209	59,536	
1977	21,904	52,900	53,361	34,596	24,025	23,104	11,664	6,889	
1978	25,281	11,025	11,025	10,816	35,721	841	28,561	6,084	20,164
1979	14,161	15,129	23,104	12,100	14,641	23,104	11,236	12,100	20,164
1980	10,609	21,025	11,025	6,241	10,404	18,225	22,500	29,584	22,801
1981	27,225	53,361	44,521	15,876	12,769	33,124	24,964	33,856	34,225
1982	14,641	24,964	10,404	8,649	12,321	10,000	18,496	30,276	43,264
1983	11,881	16,900	**177,241**	17,161	26,896	25,600	16,384	31,684	33,489
1984	7,056	17,689	14,400	24,649	11,025	28,224	21,609	73,441	30,976
1985	**84,100**	59,536	65,536	**60,516**	12,321	33,124	76,729	80,656	**65,025**
1986	22,801	17,424	19,321	13,456	**56,169**	16,641	19,881	13,225	37,249
1987	15,876	28,224	22,801	12,321	18,496	17,956	25,921	21,316	26,244
1988	66,564	40,000	66,564	29,584	16,384	37,636	39,601	**102,400**	56,644
1989	10,609	20,164	11,236	14,400	12,996	25,600	8,464	10,816	12,769
1990	10,201	13,689	6,400	16,641	28,900	10,000	23,409	22,500	58,564
1991	28,900	22,500	24,964	33,124	9,801	24,025	7,225	36,100	11,881
1992	15,129	6,724	11,236	19,600	14,884	15,625		29,241	30,976
1993	16,900		4,624	19,600	13,924	16,900	27,225	33,489	25,921
1994	25,921	16,384	11,025	26,569	23,409	23,716	**51,529**	11,025	32,041
1995	29,241	26,244	11,881	17,424	17,161	**40,000**	32,761	18,769	30,276
1996	17,161	26,244	6,889	25,921	21,609	12,321	21,316	15,376	22,500
1997	25,921	10,404	3,600	5,625	7,569	11,449	14,641	22,500	21,904
1998	15,876	18,769	13,225	12,996	17,689	15,876	34,225	10,404	18,225
1999	13,456	32,041	18,225	15,129	12,769	19,600	34,969	14,400	44,100
2000	13,689	19,600	28,900	20,164	27,889	12,321	25,600	19,600	36,100
2001	19,600	**71,289**	11,025	13,689	16,384	19,600	24,964	16,900	48,400
2002	13,225	4,900	18,225	12,100	23,716	15,625	16,900	14,884	5,625
2003		49,729	19,600	19,321	34,225	31,329		12,100	4,900
2004				29,241	20,736	26,896		51,529	11,449
	22,228.0	26,172.6	26,910.1	19,498.8	20,037.1	21,219.2	26,230.1	28,989.0	29,847.3
S_n (Mean)	44.7	49.4	73.4	34.6	32.0	34.2	47.6	59.7	46.3
S_{n-m}	34.2	45.1	51.9	28.1	26.4	33.0	51.3	52.3	42.8
S_{n-m} / S_n	0.77	0.91	0.71	0.81	0.82	0.96	1.08	0.88	0.92
S_n Adjustment Factor :	0.87	0.75	0.86	0.87	0.75	0.86	0.87	0.75	0.86
Adjustment Factor :	1.05	1.04	1.04	1.04	1.04	1.04	1.05	1.04	1.04
Adjusted S_n :	40.8	38.6	65.6	31.3	24.9	30.6	43.5	46.6	41.4
K_m :	**5.48**	**5.48**	**5.48**	**5.48**	**5.48**	**5.48**	**5.48**	**5.48**	**5.48 #**
	4.49	2.60	5.48	4.08	3.89	1.83	2.47	3.18	2.14
Unadjusted PMP :	366.0	365.4	507.8	308.3	275.9	310.8	396.2	413.8	395.0
Adjustment factor (f_0)	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13
Adjustment of PMP :	413.6	412.9	573.8	348.4	311.8	351.2	447.7	467.6	446.4
Adjustment of Point PMP to Wonogiri Watershed :	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
Adjusted 48 hours PMP for Wonogiri Watershed :	**289.5**	**289.0**	**401.7**	**243.9**	**218.2**	**245.9**	**313.4**	**327.3**	**312.4**

Table 6.13: Calculation of 168-hour Probable Maximum Precipitation

Annual Maximum Daily Rainfall X (mm)									
YEAR	Nguntoronadi	Tirtomoyo	Beji	Plumbon	Pracimantoro	Wuryantoro	Girimarto	Manyaran	Watugede
1976		120.0	241.0	225.0	196.0	170.0	241.0	259.0	164.0
1977	183.0	185.0	247.0	230.0	208.0	165.0	158.0	121.0	132.0
1978	171.0	140.0	166.0	125.0	189.0	29.0	230.0	119.0	200.0
1979	132.0	153.0	192.0	150.0	142.0	158.0	206.0	140.0	152.0
1980	164.0	177.0	140.0	97.0	118.0	178.0	194.0	198.0	167.0
1981	187.0	263.0	233.0	160.0	146.0	193.0	191.0	269.0	250.0
1982	164.0	259.0	154.0	141.0	112.0	192.0	175.0	207.0	366.0
1983	189.0	149.0	**657.0**	143.0	181.0	206.0	184.0	227.0	249.0
1984	136.0	198.0	196.0	207.0	145.0	171.0	278.0	284.0	207.0
1985	323.0	**363.0**	306.0	267.0	170.0	221.0	**321.0**	340.0	**377.0**
1986	214.0	207.0	210.0	239.0	**312.0**	178.0	196.0	203.0	281.0
1987	146.0	208.0	248.0	208.0	210.0	198.0	219.0	208.0	271.0
1988	263.0	283.0	399.0	199.0	259.0	**249.0**	232.0	**351.0**	308.0
1989	122.0	176.0	203.0	178.0	193.0	170.0	105.0	151.0	154.0
1990	149.0	189.0	137.0	165.0	170.0	141.0	195.0	217.0	291.0
1991	173.0	196.0	228.0	**306.0**	135.0	164.0	85.0	323.0	152.0
1992	125.0	134.0	105.0	158.0	178.0	154.0		176.0	240.0
1993	244.0		126.0	172.0	156.0	172.0	262.0	204.0	217.0
1994	**326.0**	224.0	145.0	239.0	218.0	225.0	277.0	184.0	247.0
1995	198.0	218.0	131.0	194.0	206.0	238.0	308.0	201.0	316.0
1996	171.0	174.0	90.0	175.0	158.0	158.0	193.0	134.0	172.0
1997	152.0	110.0	82.0	86.0	114.0	172.0	195.0	175.0	154.0
1998	166.0	173.0	125.0	168.0	133.0	208.0	260.0	155.0	228.0
1999	172.0	198.0	200.0	185.0	148.0	204.0	205.0	173.0	242.0
2000	133.0	226.0	218.0	198.0	180.0	117.0	295.0	153.0	189.0
2001	160.0	337.0	150.0	117.0	172.0	143.0	209.0	190.0	273.0
2002	167.0	95.0	175.0	155.0	194.0	179.0	182.0	219.0	101.0
2003		238.0	220.0	173.0	209.0	204.0	222.0	175.0	114.0
2004				205.0	142.0	178.0	162.0	235.0	107.0
n	26	27	28	29	29	29	28	29	29
X_n (Mean)	181.9	199.7	204.4	181.6	175.7	177.1	214.2	206.6	223.1
X_m (Max.)	326.0	363.0	657.0	306.0	312.0	249.0	321.0	351.0	377.0
X_{n-m}	176.2	193.5	187.7	177.1	170.8	174.5	210.2	201.4	217.7
X_{n-m} / X_n	0.97	0.97	0.92	0.98	0.97	0.99	0.98	0.98	0.98
X_n Adjustment Factor :	0.99	0.99	1.00	1.00	1.00	1.00	1.01	0.99	1.00
Adjustment Factor :	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01	1.01
Adjusted X_n :	181.9	199.7	206.5	183.4	177.4	178.8	218.5	205.5	225.4

Table 6.13: Calculation of 168-hour Probable Maximum Precipitation (contd)

	x^2								
YEAR	Nguntoronadi	Tirtomoyo	Beji	Plumbon	Pracimantoro	Wuryantoro	Girimarto	Manyaran	Watugede
1976		14,400	58,081	50,625	38,416	28,900	58,081	67,081	
1977	33,489	34,225	61,009	52,900	43,264	27,225	24,964	14,641	
1978	29,241	19,600	27,556	15,625	35,721	841	52,900	14,161	40,000
1979	17,424	23,409	36,864	22,500	20,164	24,964	42,436	19,600	23,104
1980	26,896	31,329	19,600	9,409	13,924	31,684	37,636	39,204	27,889
1981	34,969	69,169	54,289	25,600	21,316	37,249	36,481	72,361	62,500
1982	26,896	67,081	23,716	19,881	12,544	36,864	30,625	42,849	133,956
1983	35,721	22,201	**431,649**	20,449	32,761	42,436	33,856	51,529	62,001
1984	18,496	39,204	38,416	42,849	21,025	29,241	77,284	80,656	42,849
1985	104,329	**131,769**	93,636	71,289	28,900	48,841	**103,041**	115,600	**142,129**
1986	45,796	42,849	44,100	57,121	**97,344**	31,684	38,416	41,209	78,961
1987	21,316	43,264	61,504	43,264	44,100	39,204	47,961	43,264	73,441
1988	69,169	80,089	159,201	39,601	67,081	**62,001**	53,824	**123,201**	94,864
1989	14,884	30,976	41,209	31,684	37,249	28,900	11,025	22,801	23,716
1990	22,201	35,721	18,769	27,225	28,900	19,881	38,025	47,089	84,681
1991	29,929	38,416	51,984	**93,636**	18,225	26,896	7,225	104,329	23,104
1992	15,625	17,956	11,025	24,964	31,684	23,716		30,976	57,600
1993	59,536		15,876	29,584	24,336	29,584	68,644	41,616	47,089
1994	**106,276**	50,176	21,025	57,121	47,524	50,625	76,729	33,856	61,009
1995	39,204	47,524	17,161	37,636	42,436	56,644	94,864	40,401	99,856
1996	29,241	30,276	8,100	30,625	24,964	24,964	37,249	17,956	29,584
1997	23,104	12,100	6,724	7,396	12,996	29,584	38,025	30,625	23,716
1998	27,556	29,929	15,625	28,224	17,689	43,264	67,600	24,025	51,984
1999	29,584	39,204	40,000	34,225	21,904	41,616	42,025	29,929	58,564
2000	17,689	51,076	47,524	39,204	32,400	13,689	87,025	23,409	35,721
2001	25,600	113,569	22,500	13,689	29,584	20,449	43,681	36,100	74,529
2002	27,889	9,025	30,625	24,025	37,636	32,041	33,124	47,961	10,201
2003		56,644	48,400	29,929	43,681	41,616		30,625	12,996
2004				42,025	20,164	31,684		55,225	11,449
	35,848.5	43,747.4	53,791.7	35,251.9	32,687.3	32,975.4	49,336.4	46,285.5	55,092.3
S_n (Mean)	52.5	62.1	109.5	47.9	42.8	40.3	58.8	60.1	72.8
S_{n-m}	44.7	54.2	67.7	42.4	34.8	38.6	54.6	54.5	66.1
S_{n-m} / S_n	0.85	0.87	0.62	0.89	0.81	0.96	0.93	0.91	0.91
S_n Adjustment Factor :	0.87	0.75	0.86	0.87	0.75	0.86	0.87	0.75	0.86
Adjustment Factor :	1.05	1.04	1.04	1.04	1.04	1.04	1.05	1.04	1.04
Adjusted S_n :	47.9	48.4	98.0	43.3	33.4	36.0	53.7	46.8	65.1
K_m :	**6.94**	**6.94**	**6.94**	**6.94**	**6.94**	**6.94**	**6.94**	**6.94**	**6.94 #**
	3.35	3.13	6.94	3.04	4.06	1.93	2.03	2.75	2.41
Unadjusted PMP :	514.3	535.5	886.1	483.8	409.0	428.7	590.9	530.5	676.9
Adjustment factor (f_0)	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13	1.13
Adjustment of PMP :	581.2	605.1	1001.3	546.6	462.2	484.4	667.8	599.5	764.9
Adjustment of Point PMP to Wonogiri Watershed :	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70	0.70
Adjusted 48 hours PMP for Wonogiri Watershed :	**406.8**	**423.6**	**700.9**	**382.7**	**323.5**	**339.1**	**467.4**	**419.6**	**535.4**

6.5. PMF AND SPILLWAY ADEQUACY OF WONOGIRI DAM RESERVOIR

6.5.1. Objectives and Background

The economic efficiency objective requires that the location and capacity of a reservoir be selected so that the net benefit is maximized. However, the capacity thus found may well be exceeded by rare meteorological events with inflow volumes or inflow rates greater than the reservoir's design capacity.

The capacity exceedance presents a significant risk to the public downstream of the reservoir. Unless the reservoir has been designed to release the excessive water in a controlled manner, the reservoir may fill and overtop. This may lead to catastrophic dam failure. Accordingly, some guidelines and policy are to design a dam, and particularly the dam's spillway, to pass safely a flood event caused by an occurrence of a rare event one much larger than the design capacity of the reservoir. A spillway capacity evaluation provides the information necessary for this design.

Spillway capacity studies are required for both proposed and existing spillways. For proposed spillways, the studies provide flow rates required for sizing and configuring the spillway. For existing spillways, the studies ensure that the existing configuration meets current safety requirements. These requirements may change as additional information about local meteorology becomes available, thus changing the properties of the likely extreme events. Further, as the watershed changes due to development or natural shifts, the volume of runoff into the reservoir due to an extreme event may change, thus rendering a historically safe reservoir unsafe. In that case, the spillway will need to be modified or an auxiliary spillway may be constructed.

6.5.2. Extreme events

Performance of a water-control measure can be evaluated with three broad categories of hydrometeorologic events: (1) historical events; (2) frequency-based events; and (3) an estimated limiting value event. Evaluation with historical events is useful for providing information that is easily understood by and relevant to the public. For example, a useful index of performance of a reservoir could be a report of the damage reduction attributable to that reservoir during the flood of record. The utility of frequency-based events has been discussed in earlier chapter. The final category of event, the estimated limiting value, is described by Chow, et al. (1988) as follows:

The practical upper limit on the hydrologic design scale is not infinite...Some hydrologists recognize no upper limit, but such a view is physically unrealistic. The *lower limit of the design scale is zero in most cases...Although the true upper limit is usually unknown, for practical purposes an estimated upper limit may be determined. This estimated limiting value (ELV) is defined as the largest magnitude possible for a hydrologic event at a given location, based upon the best available hydrologic information.*

Thus the utility of the ELV event is to demonstrate how a damage reduction measure would perform in the worst reasonable case a case that is very unlikely, but still possible. This is the approach used for spillway studies.

6.5.3. Analysis Procedures

To meet the objective of a reservoir spillway capacity study, the following steps are typically taken:

1. Develop a model of the contributing watershed and channels.
2. Define the extreme-event rainfall: the PMP.
3. Compute the inflow hydrograph to the reservoir: the PMF.
4. Develop a model of the performance of the reservoir and spillway.
5. Use the model to simulate reservoir performance with the hydrograph from step 3, routing the PMF through the reservoir, over the spillway, and through downstream channels.
6. Compare the performance of the spillway to the established criteria to determine if the spillway adequately meets the criteria.

HEC-HMS is a convenient tool to use for this analysis. Its application within this procedure is illustrated with the case study below.

6.5.4. Watershed and Reservoir Description

Wonogiri Dam and Reservoir are located on Upper Solo river in the Bengawan Solo river basin of central Java. The reservoir was completed in 1980 with the construction of Wonogiri Dam, an earth and rockfill structure. The reservoir and dam are shown in Figure 6.6.

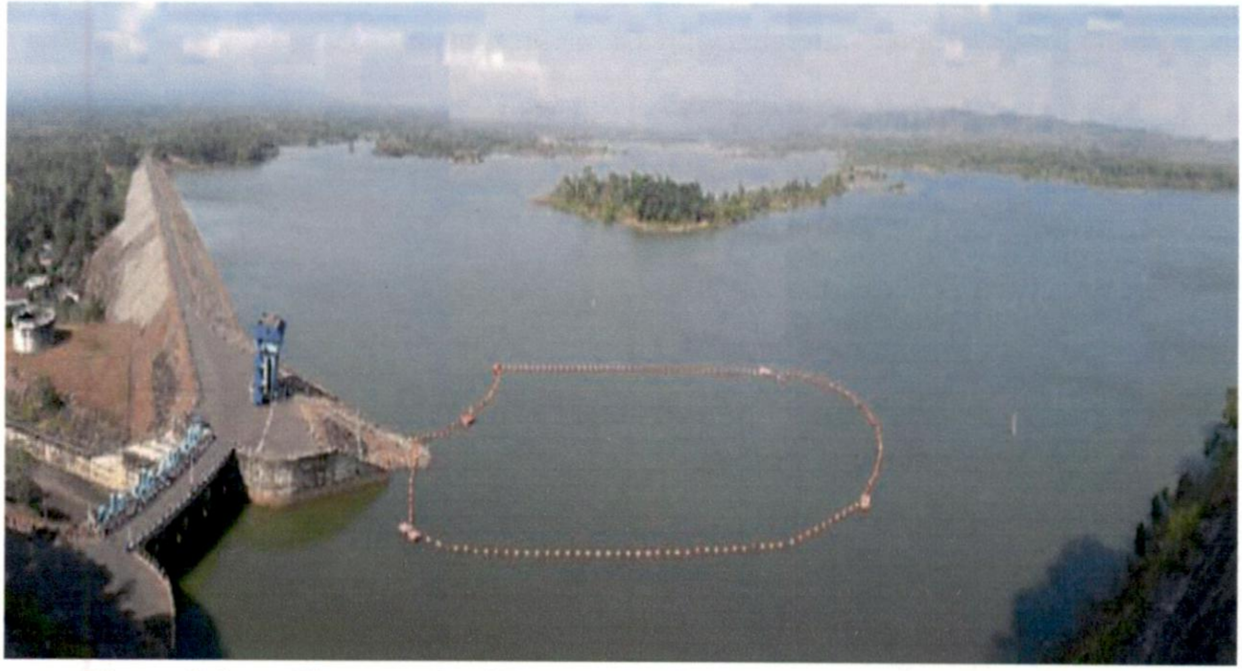


Figure 6.6: Photograph of Dam and Spillway of Wonogiri Dam Reservoir

The reservoir was constructed to store water for power generation, and it provides incidental flood control, irrigation and and water supply. Releases are made also for fish and wildlife needs downstream. The top of the dam is at elevation 142.0 meter. The spillway crest elevation 131.0 m. The contributing watershed area to the reservoir is 1240 square km.

6.5.5. Decisions and Information required

The Wonogiri Dam spillway initially was designed to carry safely a large event thought to be approximately the 1,000-year flood event. However, the risk of failure is of concern, so the spillway capacity is to be the PMP and PMF. The following questions are relevant.

- Will the existing spillway pass the PMF? That is, will the dam be overtopped if the PMF flows enter the reservoir?
- If not, how can the dam and spillway be modified to pass safely the PMF?

To answer the questions, the PMF must be computed and routed. The spatial extent of the analysis was limited to the portion of the watershed that contributes flow to the reservoir, to the reservoir itself, and to the area immediately downstream. This contributing area had been defined in the design studies; otherwise the analyst could have used topographic data to delineate the watershed. In this case, the model extended downstream of the reservoir only a short distance. However, if development in the downstream floodplain is such that dam failure poses a significant risk, the model should be extended further. Only by doing so will information be available for assessing the risk and for developing emergency plans.

6.5.6. Model Selection and Parameter Estimation

A variety of options are available in HEC-HMS software.

- **Runoff volume method.** The initial and constant-rate runoff volume method has been chosen. This method was used to represent the watershed characteristics during dam design. During PMF analysis, a common assumption is that the antecedent moisture saturates the soil before the PMP occurs. When this happens, the rate of infiltration approaches a constant value. The advantage of the initial and constant-rate method is that this physical condition can be represented well with the model. Another advantage is the simplicity of the method, which has only two parameters.
- **Transform method.** The Clark's unit hydrograph has been selected. This is the method that used to represent the watershed characteristics. This method requires two parameters: time of concentration, Tc, and storage coefficient, R. Studies by the California Department of Water Resources yielded predictors for these parameters. The analyst did use the rather limited rainfall data for 3 historical events and computed reservoir inflow hydrographs using the Clark unit hydrograph method. When compared with inflow hydrographs inferred from reservoir records, the fit adequate have been judged.
- Baseflow method. The analysis did not include baseflow in the model.

The PMF represents runoff from the most severe combination of critical meteorologic and hydrologic conditions for the watershed. During such events, travel times tend to be significantly shorter. Consequently, it is common to adjust unit hydrograph parameters to "peak" the unit hydrograph (USACE, 1991), increasing the maximum runoff and shortening the runoff time. As a general rule of thumb, reservoir inflow unit hydrographs for PMF determinations have been peaked 25 to 50%. By reviewing observed runoff hydrographs from other severe storms in the region, ultimately, shortening Tc and reducing R to achieve a unit hydrograph peak approximately 50% greater than that found with the original best-estimates of the parameters. The values selected for PMF analysis were Tc = 1.0 hours and R = 2.3 hours. The analysis selected a 15-minute simulation time interval, consistent with this estimated time of concentration.

6.5.7. Boundary Condition: PMP Development

Previous section in this chapter has developed PMP calculation procedures for all sub basin of the Wonogiri dam reservoir watershed. And then to perform the storm analysis; the resulting hyetograph is stored in DSS for input to HEC-HMS.

As the availability of data increases, the PMP estimates from the data project report may require adjustment in order to better define the conceptual PMP for a specific sub basin. Therefore, it is appropriate to refine PMP estimates with site specific or regional studies performed by Hersfiled statistical method in determining PMP, with this. PMP data are given in Table 6.14.

Table 6.14: Summary of PMP Depth Duration Data of Wonogiri Reservoir Watershed

Day	Duration (hr)	Depth (mm)
-	1	22.4
1	24	234.3
2	48	248.9
3	72	383.6
4	96	401.7
7	168	700.9

The PMP estimates were provided as a 168-hour storm, divided into 6-hour increments. These 6-hour values can be arranged into a storm temporal distribution that is front-, middle-, or end-loaded. The five 6-hour intervals with greatest depth were grouped into a 24-hour sequence, and the remaining intervals were arranged as described below to complete definition of the rainfall event. Within the peak 24-hour sequence, the five 6-hour values are distributed in an alternating block sequence, with largest values in the center. For this watershed, the computation time interval selected was 15 minutes, so depths for durations shorter than 6 hours and for intervals less than 6 hours are needed. To develop these, the analysis plotted the logarithms of depths and durations, as shown in Figure 6.7, and interpolated for intermediate durations. Some smoothing of the plotted function was required. Interpolated depths are shown in Table 6.15.

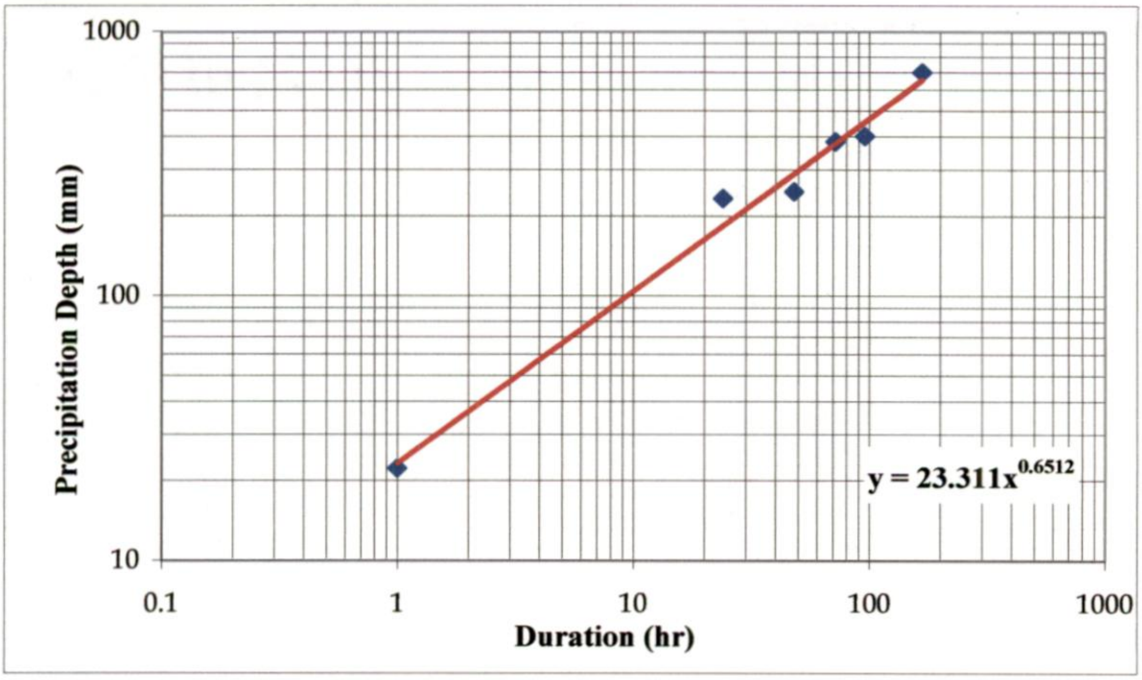


Figure 6.7: PMP depth-duration curve for Wonogiri Watershed

Table 6.15: Extended PMP depth-duration data for HEC-HMS input

Duration (hr)	Depth (mm)
0.25	9.45
0.5	14.84
1	22.35
2	36.61
3	47.67
4	57.49
5	66.49
6	74.87
12	117.58
24	234.32
48	248.94
72	383.62
96	401.68
168	700.90
240	827.09

To specify the PMP depths, the analyst used the **Frequency Storm** precipitation method. The **Component Editor,** which is shown in Figure 6.8, does not permit entry of a 72-hour rainfall depth, so depth for a duration of 96 hours (4 days) was estimated and entered. The peak volume stored in the reservoir is a function of the PMF peak discharge. A 2-day event could have been selected rather than the 4-day event. The 2-day event would yield the same peak discharge, stage, and volume of water in the reservoir.

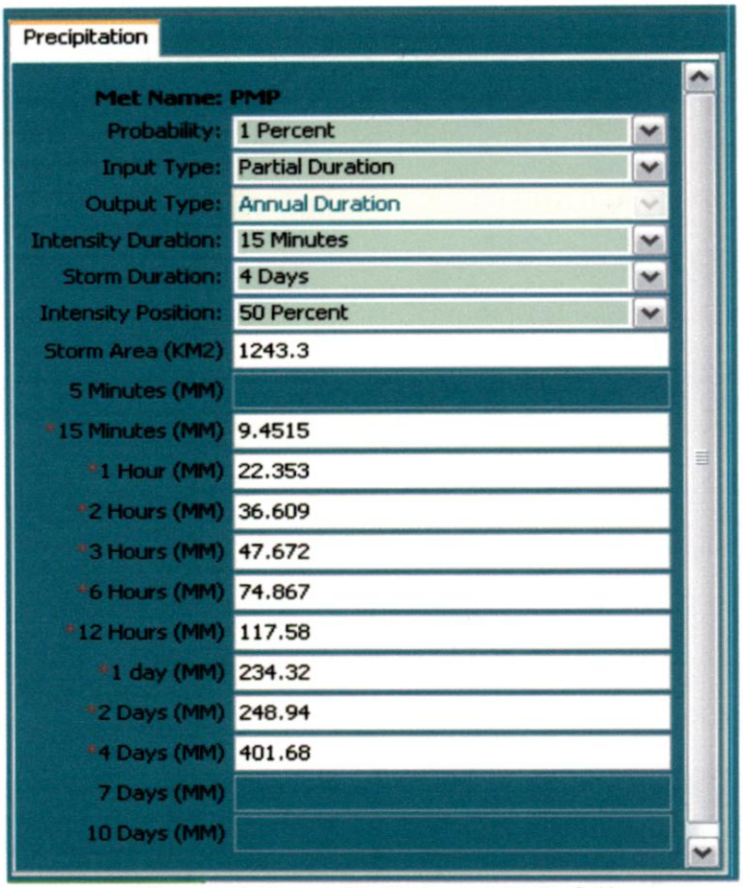


Figure 6.8: PMP Input Rainfall

6.5.8. Reservoir model

In addition to the model of runoff, a model of the reservoir and dam in HEC-HMS also developed. The resulting basin model is shown in Figure 6.9.

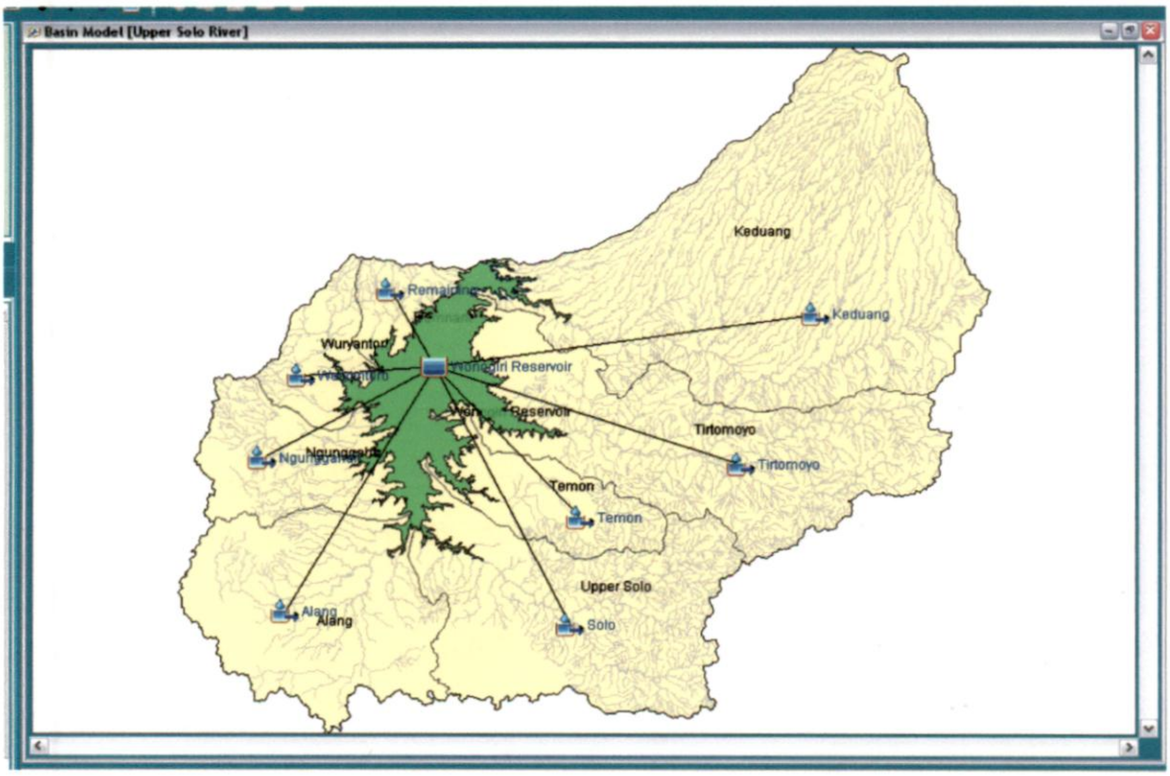


Figure 6.9: Basin Model for PMP Evaluation

Table 6.16 shows the elevation-storage curve for Wonogiri Reservoir. The existing spillway crest is at elevation 131.0 meter and the crest length is 30 m. This information was found in the original design documents. However, if the data had not been available, the elevation-volume relationship would be developed from topographic and bathymetric surveys.

This analysis, based on dam-safety regulations. Per these regulations, any low level outlets through the dam are assumed not operable, and all outflows from the reservoir must pass over the spillway. The analysis also considered the possibility of tailwater control. However, because all flow would pass over the elevated spillway, tailwater was not a factor.

The reservoir was modeled using the **Outflow Structures** routing method. The elevation-storage curve shown in Table 6.16 was used along with a spillway outlet. The spillway outlet was modeled using a **Specified Spillway** with a spillway crest elevation at 131.0 m, a spillway length of 30.0 m, and a discharge coefficient of 1.97.

Table 6.16: Elevation-storage Data for Wonogiri Reservoir

Elevation (m)	Reservoir Storage (MCM)	Elevation (m)	Reservoir Storage (MCM)
119.0	0.01	131.0	166.84
120.0	0.35	132.0	206.92
121.0	2.39	133.0	254.09
122.0	5.44	134.0	307.35
123.0	9.70	135.0	366.63
124.0	16.68	135.3	385.76
125.0	26.98	136.0	432.76
126.0	40.64	137.0	507.15
127.0	57.86	138.0	589.22
128.0	78.46	138.3	615.54
129.0	103.04	139.0	679.06
130.0	132.03	139.1	688.35

6.5.9. Initial Conditions

This analysis had to select two initial conditions for the analysis: (1) the initial state of the watershed, and (2) the initial state of the reservoir. For the first condition, the analyst reasoned that the watershed was likely to be saturated when an extreme event occurred, and thus set the initial loss equal to 0.00 mm. For the second condition, the analysis referred dam safety regulations and found that these specified that the initial reservoir water surface elevation should equal the spillway crest elevation. Thus spillway flow is initiated with inflow. This conservative initial condition was accepted and implemented by specifying **Initial Elevation** of 131.0 m in the reservoir **Component Editor.**

6.5.10. Application

The HEC-HMS model was completed, and the event simulated (Figure 6.10). A peak spillway discharge of 5678.5 m^3/s was computed and the maximum water surface elevation in the reservoir was 136.6 m ((Figure 6.11). As the top of the dam is at 142.0 m, this means that the dam would not be overtopped by the event. In addition, from the analysis recognized that if precipitation depths were underestimated, if the unit hydrograph was not peaked adequately, or if the reservoir performance was modeled a bit optimistically, the pool elevation, in fact, would be smaller. Further, it knows that other factors, such as wind-driven waves, could well increase the pool elevation even more.

Research revealed that local dam safety regulations require a minimum difference of 1.5 meter to account for uncertainty in estimates. Thus the dam was considered able to pass reliably the spillway design event.

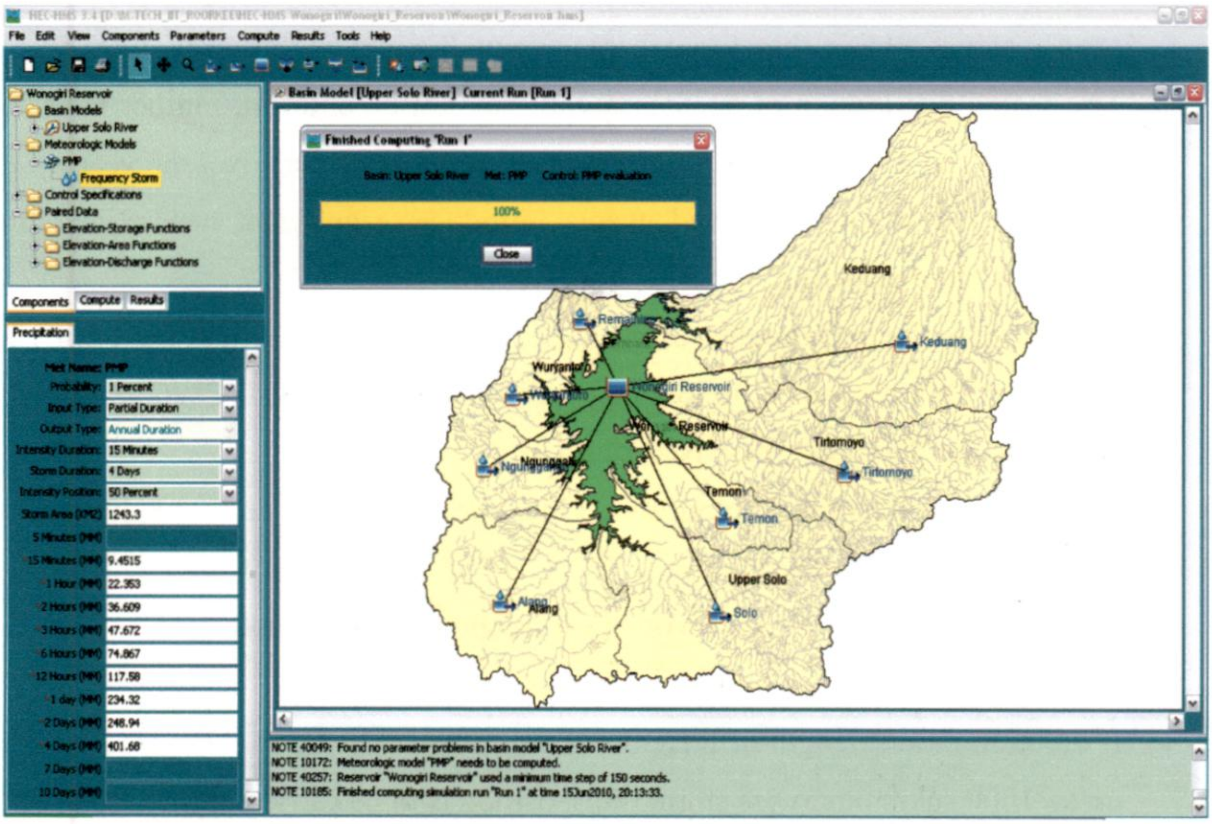


Figure 6.10: Complete Running Model of Spillway Adequacy for Wonogiri Reservoir

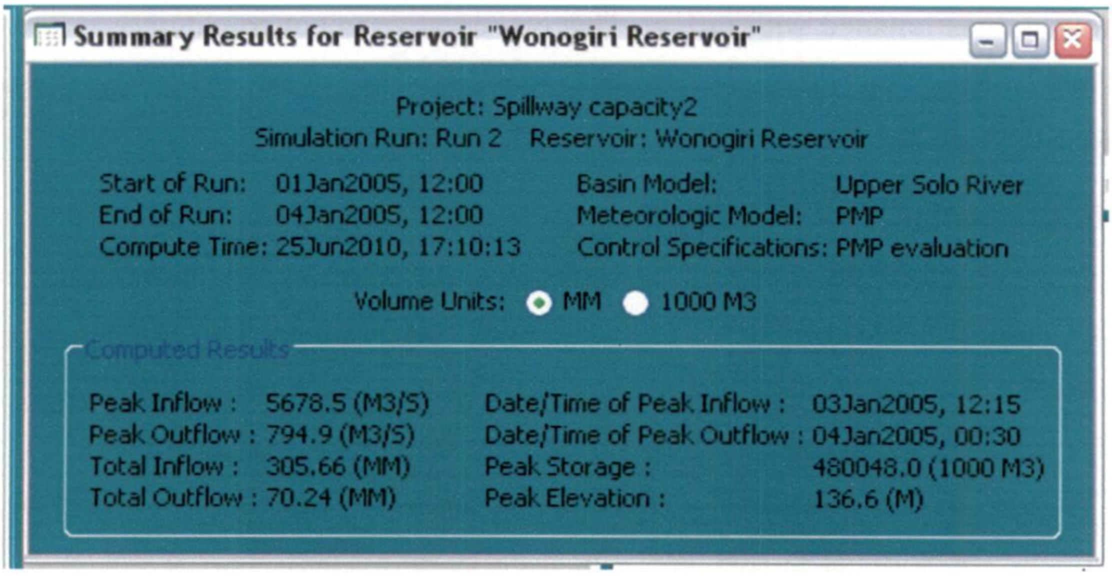


Figure 6.11: Summary Result for Wonogiri Reservoir

6.6. CONCLUSION

1. Uncertainty in PMP estimation arises due to:
   i) non availability of upper air data for moisture correction at desired location
   ii) actual shape (areal distribution) of original storms lost as storm are expressed in term of depth are curves.
   iii) Factors such as interception, surface storage and infiltration capacity can vary greatly across a basin therefore where as rainfall could be uniformly distributed. The spatial distribution of excess rainfall may not be uniform. Further land use changes occur with time causing changes in these parameters over the year.
   iv) Extrapolation beyond observed events introduces significant uncertainty into PMP and PMF modeling.
2. Keeping in view the uncertainties as discussed above and to maintain uniformity in procedure for PMP procedure, following practice is recommended (CWC 1993):
   i) Design storm duration = base period of UH.
   ii) If CA is more than 5000 sqkm divide into sub catchments and take duration = 2.5 time the time of concentration.
   iii) 24 hr PMP = 50 mm + 1 day PMP. No correction of CA > 5000 sqkm.
   iv) No area reduction factor for CA $\leq$ 50 sqkm.
   v) If duration ration (length of breadth ration) is less than 1.5, DAD value are use up to 1000 sqkm otherwise storm transposition is to be carried out.
   vi) Design hyetograph consist of two bells (each 12 of 12 hr) per day, arrangement of increments is such that maximum flood is product.

3. Case study of Wonogiri Reservoir
   i) Improvements are necessary in observation and recording of rainfall and discharge data. This is very important for safety of dam.
   ii) Clock hour correction is not constant (50 mm as followed in India) but is based on an adjustment factor which is 1.13.
   iii) Area reduction factor is based on basin mean daily rainfall and maximum point rainfall.
   iv) Largest depth of PMP (mm) for the catchment estimated by Hershfield equation for any duration are shown in Table 6.17.

Table 6.17. Summary of PMP Depth-Duration (mm-hr) for Wonogiri Watershed

Day	Duration (hr)	Depth (mm)	Station
-	1	22.4	Jatisrono 131
1	24	234.3	Beji
2	48	248.9	Puwantoro
3	72	383.6	Puwantoro
4	96	401.7	Beji
7	168	700.9	Beji

4. HEC-HMS can be used to compute the PMF, using PMP depths as input. Likewise, dam and spillway performance can be simulated with the reservoir model included in HEC-HMS. For that reason, the analysis must derive and specify functions that describe how the reservoir will perform.
5. The peak spillway discharge is 5678.5 m^3/sec as computed using HEC-HMS software whereas the spillway capacity is 5100 m^3/sec. The spillway capacity of 5100 m^3/s corresponds to 1.2 times 100 years probable flood (Table 6.2) where as the probable maximum flood as given in project report (Table 6.2) is 9600 m^3/sec. Therefore it is concluded that there is adequate spillway capacity in the Wonogiri dam.

CHAPTER VII

DAM BREAK ANALYSIS FOR DAM SAFETY

7.1. INTRODUCTION

Dam failures are often caused by overtopping of the dam due to inadequate spillway capacity during large inflow to the reservoir from heavy precipitation-generated runoff. Dam failures may also be caused by seepage or piping through the dam or along internal conduits, slope embankment slides, earthquake damage and liquefaction of earthen dams from earthquakes and land slide generated waves in the reservoir. Usually the response time available for warning is much shorter than for precipitation-runoff floods. The protection and evacuation of the public from the consequences of dam failures has taken an increasing importance as population has concentrated in areas vulnerable to dam break disasters.

Occurrence of a series of dam failures has increasingly focused attention of scientific workers on the need for developing generally applicable models and methods to evaluate flash floods due to dam failure and for routing them through downstream areas, susceptible to heavy losses, so that potential hazards might be evaluated. Using these methods, inundated areas, flow depths and flow velocities can be estimated for different hypothetical dam failure situations. With the help of such studies, it could be possible to issue warnings to the downstream public and prepare strategies for disaster management when there is a failure of dam. The main difficulty in using such mathematical models is the failure description adopted in the model. Under these circumstances, a suitable assumption with regard to the adjustment of actual failure mode to suit the model failure mode is necessary.

The DAMBRK model developed by U.S. National Weather Services (NWS) attempts to represent the current state-of-art in understanding of dam failures and the utilization of hydrodynamic theory to predict the dam break wave formation and its downstream progression. The model has wide applicability; it can function with various levels of input data ranging from rough estimates to complete data specification, the required data is readily accessible and it is economically feasible to use, i.e. it requires a minimal computation effort on large computing facilities. The model consists of three functional parts, viz. (i) description of the dam failure mode, (ii) computation of the time history (hydrograph) of the outflow through the breach, and (iii) routing of the outflow hydrograph through the downstream valley. This determines the changes in the hydrograph due to valley storage, frictional resistance, downstream bridges or dams, and to determine the resulting water surface elevations (stages) and flood wave travel time.

This chapter presents the literature reviewed on dam break analysis and application study of Wonogiri dam in Indonesia.

7.2. CONCEPT OF THE SAFEST DAM

Taking into account increased public preoccupation with dam safety issues and the uncertainty in determination of extreme loading conditions (particularly in developing countries), a modified dam type, designated as the Safest Dam, is postulated by Stevens M.A. & Linard J. (2002). The Safest Dam is a low-strength, symmetrical section RCC embankment constructed on any rock foundation that is considered acceptable for a conventional gravity dam of the same height (Figure 7.1). The ungated spillway, occupying the entire dam crest length, discharges along the downstream face of the dam into a preformed concrete-lined energy dissipater. The dominant features of the Safest Dam are:

i) Satisfies conventional stability requirements, without reliance on elements of uncertain long-term reliability-specifically, foundation and internal drainage and waterproofing facilities.
ii) Safely passes the upper-limit inflow design flood without the outflow peak ever exceeding the inflow peak under any circumstances.
iii) Loads its foundation in compression over the entire contact area and the maximum principal stress within the dam is compressive under all normal and unusual load combinations.

7.3. REVIEW OF LITERATURE

The literature on dam break studies is vast. There are various aspects to these studies. However, only routing aspect is presented here and other aspects are beyond the scope of this dissertation. For the sake of simplicity, previous studies on dam break are presented here under three categories, viz, (1) Analytical models, (2) experimental models and (3) Numerical models. Excellent review articles have been presented by Basco (1989), Almeida et al. (1994) and Singh (1996).

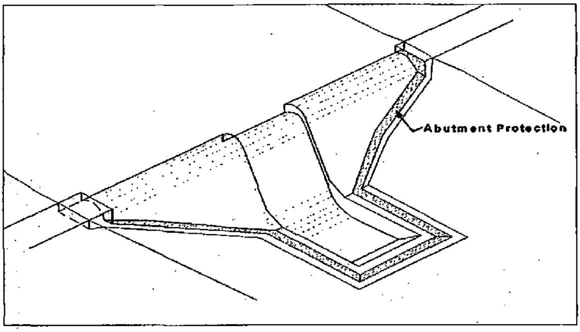


Figure 7.1: The Safest Dam

7.3.1. Analytical Models

Studies to understand the basic mechanics of dam break flows (DBF) are very old and date back to the earliest attempt by Ritter in 1892. Ritter derived an analytical solution for the hydrodynamic problem of instantaneous dam-break in a frictionless and horizontal channel of rectangular shape. In Ritter's solution, both the reservoir and the channel were assumed to be infinite and the channel downstream was assumed to be dry. The flow depth (h) and velocity (u), at any place downstream of the dam are functions of distance (x), time (t) and reservoir water level (h_0). The analytical solutions given by Ritter (1892) are ;

$$h = \frac{1}{9g}\left(2C_0 - \frac{x}{t}\right) \tag{7.1}$$

$$u = \frac{2}{3}\left(C_0 + \frac{x}{t}\right) \tag{7.2}$$

Where, wave celerity, $C_0 = \sqrt{gh_0}$. According to these equations, the flow depth and the discharge attained at the dam-site are constant in time and represent critical flow condition there. The shape of the free surface is a parabola and the tip speed is twice that of the disturbance propagated upstream. Later, Dressier (1952) and Whitham (1955) included the effect of the bed resistance in the analysis of DBF and derived analytical expressions for the velocity and height of the wave-front. Pohle (1952) considered two dimensional flow in x and z direction Using Lagrange representation, he concluded that in the initial regime, the vertical acceleration is the predominant parameter. When the vertical acceleration is decreasing, the effect of channel cross-sectional geometry, bed friction and bed slope become more important and the wave profile will then converge to one-dimensional analytical solution.

Stoker (1957) extended the Ritter solution to the case of wet-bed condition in the downstream. He derived analytical expressions for the surface profile in terms of the initial depths upstream and downstream of the dam. In Stoker's solution, there are four distinct zones, viz. two undisturbed zones, one each in the upstream and downstream side, one drawdown zone and one zone with a constant bore height (Fig. 1). In Stoker's solution, the velocity of bore propagation and the constant bore height are attained instantaneously. The analytical equations derived by Hunt (1982, 1987) considered finite length reservoirs. However, Hunt's solution was based on the assumption of a kinematic wave.

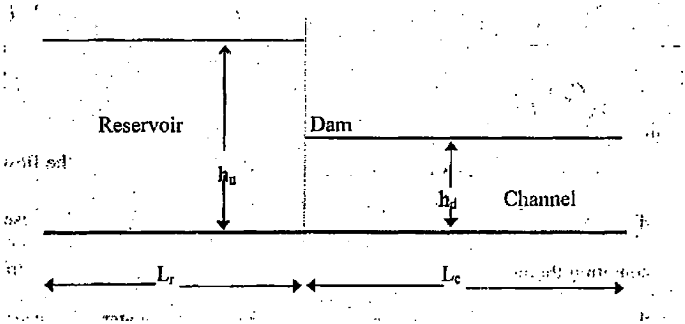


Figure 7.2: Initial depths upstream and downstream of the dam.

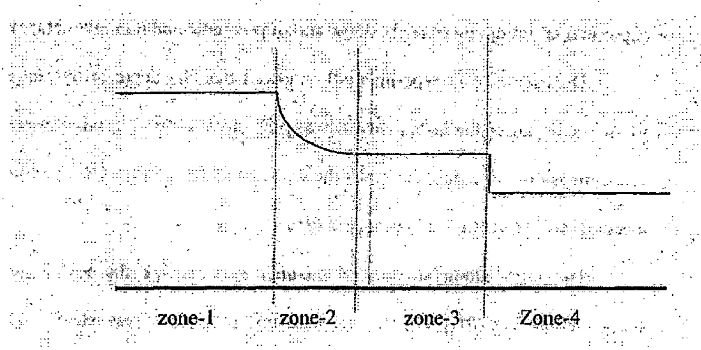


Figure 7.3: Four distinct zone by Stoker's Solution.

7.3.2. Experimental Model

The complexity of the unsteady flow due to a dam failure necessitates for more accurate modeling, than the analytical models. Experimental modeling is one of the methods to analyse the real flow phenomenon. Some important purposes of experimental modeling are verification of computational models, complete analysis of real cases, and more understanding of the DBF problem.

Escande et al. (1961) presented detailed results obtained from experimental studies using a 1.6 km reservoir and 12 km long downstream reach with fixed bed. They presented the front wave profile, due to the sudden failure of a dam for different conditions as well as the variation of front wave velocity with bed roughness,, initial reservoir head and initial channel flow. One of the complete set of laboratory data on dam break flows was collected at the U.S.A. Army Engineers, Waterways Experiment Station (WES, 1960). Rajor (1973) presented results for dam break flows, obtained through experimental modeling of real valleys and of prismatic and non prismatic channel.

Dressier (1954) experimentally showed that the depth at the dam site docs not attain a constant value instantaneously as predicted by Ritter. It takes approximately nine non-dimensional time units to reach the constant Ritter's value. He also found that the tip speed of dam break flow is less than $2\sqrt{gh_o}$.

The DBF along an alluvial channel may change the valley geomorphology. Simons et al. (1980) presented experimental data to assess change in flood stage, resistance to flood, and transport of deposited sediment following failure of a dam. He concluded that in general, when a dam fails the interaction between the water and sediment transport and the river stability is not well understood.

All the above experimental studies are for straight channel reaches, however, the natural channels are seldom straight and meander in the channel alignment produce lateral gradients in the flow surface. Miller and Chaudhry (1989) presented the experimental results for dam break flows in meandering channels. Memos (1983) presented on experimental results that at a partial dam failure (breach width less than valley width) three-dimensional effects are dominant during the first instance of the break. Martin (1983) presented the results of a total dam break in a rectangular and in a channel with divergent side walls (dry bed). Similar observations were also presented for convergent and divergent channels by Townson and Al-Salihi (1989) and Bellos et al. (1992).

7.3.3. Numerical Models

A numerical model is the most convenient tool for a fast and systematic analysis of dam break flow. Generally in a numerical model, the dam break flow is simulated by three consequential steps, i.e. (i) routing of the inflow hydrograph from the reservoir inlet to the dam site, (ii) dam break mechanism and (iii) routing of the dam break flow in the downstream channel. All the numerical models may be categorized, depending on the equations used to model the phenomenon, numerical scheme used to solve the equations, and, implementation of different boundary physical conditions.

Governing equations used:

In most of the numerical models, available in literature, one dimensional St.Venant equations are used as the governing equations (Fennema & Chaudhry) 1987, Molls and Molls (1998), Fread 1988). One dimensional St.Venant equations assume a hydrostatic pressure distribution along vertical plane.; Basco (1989) pointedi out limitations to the St.Venant equations in dam break flows analysis. In some studies, one-dimensional Boussinensq equations are used to simulate the dam break flow (Carmo et al. 1993, Gharangik and Chaudhry 1991, Mohapatra and Singh 2000). Two-dimensional St.Venant equations in x,y plane are used for dam break flow analysis by some researchers (Fennema and Chaudhary 1990, Alcrudo and Gracia-Novarro 1992, Mohapatra and Bhallamudi 1996). Two dimensional Navier -Stokes equations in x and z plane to study dam break flows are also presented in literature (Hirt and Nichols 1981, Tome and McKee 1994 and Mohapatra et al. 1999).

Numerical methods:

Different numerical methods available in the literature are (1) finite element method, (ii) finite difference method and (iii) method of characteristics. Detailed descriptions of the above methods are available in Chaudhry (1993).

DAMBRK model is the National weather service (NWS) dam break flood forecasting model developed by Fread (1979, 1988). In this model, the expanded form of St.Venant equations are used for routing of dam break floods in channels. This model allows the failure timing interval and terminal size and shape of breach as input. It gives the extent of and the time of occurrence of flooding in the downstream valley by routing the outflow hydrograph through the valley.

SMPDBK is a simplified version of the dam break flood forecasting model presented by Wetmore and Fread (1984) for quick prediction of downstream flooding caused by a dam failure. This model is an useful forecasting tool in a dam failure emergency when warning response time is short, little data are available and large computer facilities beyond one's reach. It is also very useful for preparing disaster contingency plans. This model consists of three main components: (i) calculation of the peak discharge at the dam, (ii) approximation of the downstream channel as a prismatic channel and (iii) determination of the peak flood at specified cross section of the downstream channel.

MIKE 11 is a software package developed at the Danish Hydraulic Institute (DHI) for the simulation of flow, sediment transport and water quality of estuaries, rivers, irrigation systems and similar water bodies. It is developed especially for application of micro computers. It offers an unique and user-friendly tool for dam break flow analysis. This model consists of a number of modules which in principle operate independently and give a rational and user friendly execution and enhance the flexibility of the package. Presently, MIKE 11 is also available with MIKE - SHE. It is workable in a window environment, the graphics facilities are excellent, and compatibility to G1S makes it versatile software for dam break flow. MIKE 21 also developed at Danish Hydraulic Institute uses two dimensional St. Venant equations, governing the flow.

TELEMAC system (Hervouet 1996) uses finite element method to solve the governing flow equations for the analysis of the dam break flows. Complicated river geometry can be considered by this computer program. A module solving Boussinesq equations is also available in TELEMAC.

Besides the above computers software for dam break flows, some research papers dealing with advanced techniques are presented below.

Alam et al. (1995) presented the collocation method in conjunction with Quintic Hermite elements to solve the system of flow equation for DBF. Quntic Hermite eliments are used to provide the high resolution required in the solution of discontinuities for producing satisfactory stable solution. This model can simulate both sub-critical and super-critical flows in different parts of the channel or in a sequence in time.

A general mathematical model was developed by Molls and Chaudhry (1995) to solve unsteady, two-dimensional depth averaged equations. This model uses boundary fitted coordinates and includes effective stresses. It may be used to analyze sub and super- critical flows. The time differencing is accomplished using a second order accurate Beam and Warming method, while the spatial derivatives are approximated by second order accurate central finite differencing. The equations are solved on a non-staggered grid using an alternating-direction implicit scheme. The model is used to analyze a wide variety of hydraulics problems including a dam break simulation. A characteristic - based upwind, explicit numerical scheme is developed by Jin and Fread (1997) for one-dimensional unsteady flow modeling of dam break flows into the (NWS) FLDWAV model, in combination with the original four point implicit scheme. The new explicit scheme is extensively tested and compared with the implicit scheme and provides improved versatility and accuracy in some situations, such as waves due to large dam break and other unsteady flow with near critical mixed flow regimes. A technique for implicit - explicit multiple routing is introduced to incorporate the advantages of using both schemes.

A high resolution time marching method was presented by Mingham and Causon (1998) for solving the two dimensional shallow water equations. This method uses a cell centered formulation with collocated data rather than a space-staggered approach. Spurious oscillations are avoided by employing monotonic upstream scheme for conservation laws (MUSCL) reconstruction with an approximate Riemann Solver in a two-step Runge-Kutta time stepping schemes. A finite volume implementation on a boundary conforming mesh is chosen to accurately map the complex geometries. These features enable the model to deal with dam break phenomena involving flow discontinuities, sub-critical and super-critical flows. The method is applied to several bore wave propagation and dam break flow problems.

A list of 23 numerical models for dam break flows is presented in Table 7.1 (Molinaro and Fillippo 1992).

Table 7.1: List of 23 Dam-Break Numerical Models

Sl. No.	Agency	Name of Models
1	USA/National Weather Service	DAMBRK (original)
2.	USA/National Weather Service	SMPDBK (simplified dam-Break)
3.	BOSS	BOSS DAMBRK
4.	HAESTED METHODS	HAESTED DAMBRK
5.	Binnie & Partners	UKDAMBRK
6.	USA/COE-Hydrologic Engineering Centre	HEC-Programs
7.	Tarns	LATIS
8.	Institute of Water Resources and Hydroelectric Power Research (IWAR), PR China	DKB 1
9.	Institute of Water Resources and Hydroelectric Power Research (IWAR), PR China	DKB 2
10.	Royal Institute of Technology, Stockholm	TVDDAM
11.	Cemagrer	RUBBER 3
12.	Delft Hydraulics	WENDY
13.	Delft Hydraulics	DELFLO/DELQUA
14.	Consultin Engineers Reiter Ltd.	DYX.10
15.	ANU-Reiter Ltd.	DYNET-ANUFLOOD
16.	ENEL Centro di Ricerca Hydraulics	RECAS
17.	ENEL Centro di Ricerca Hydraulics	FLOOD2D
18.	ENEL Centro di Ricerca Hydraulics	STREAM
19.	Danish Hydraulic Institute	MIKE 11
20.	ETH Zurich	FLORIS
21.	Danish Hydraulic Institute	MIKE 21
22.	EDF-Loabratoire National Hydraulique	RUPTURE
23.	EDF-Loabratoire National Hydraulique	TELEMAC

Source: Molinaro and Fillippo 1992

7.4. BRIEF DESCRIPTION OF NWS-DAMBRK MODEL

The DAMBRK model attempts to represent the current state-of- the art in understanding of dam failure and the utilization of hydrodynamic theory to predict the dam break wave formation and its downstream progression. The basic code of the computer program was developed over a period of several years by D.L. Fread of the National Weather Services (NWS). The model has wide applicability as it can function with various levels of input data specifications and requires minimal computation effort on large computing facilities.

The model consists of three functional parts:

1. Description of the dam failure mode.
2. Computation of outflow hydrograph through the breach as affected by the breach description, reservoir storage characteristics, spillway outflows and downstream tail

water elevations; and

3. Routing of the outflow hydrograph through the downstream valley in order to determine the change in the hydrograph due to valley storage, frictional resistance, downstream bridges or dams, and to determine the resulting water surface elevations and flood wave travel time.

7.4.1. Assumptions

The following assumptions are used in the model development:

1. Cross sections in the downstream channel are oriented perpendicular to the flow so that the water surface is horizontal across the section.
2. The channel boundaries are rigid, i.e. cross sections do not change their shape due to scour or deposition.
3. The pool elevation at which breaching begins, rate of breach development, and shape and size of the breach must be supplied by the user.

7.4.2. Dam Failure

The failure time and terminal size and shape of the breach are given as input for the model. The shape is specified by a parameter z as shown in Figure 7.4, identifying the side slope of the breach (i.e. 1 vertical : z horizontal slope). Rectangular, triangular, or trapezoidal shapes may be specified through this parameter. The final breach size is controlled by the parameter and the terminal width (b) of the breach bottom. The breach bottom width is assumed to start at a point and then increases at a linear rate over the failure time interval (T_b) until the terminal width is attained and the breach bottom has eroded to the final elevation (h_{bm}) which is usually but not necessarily, the bottom of the reservoir or outlet channel section. If the failure time 'T_b' is less than ten minutes, the width of the breach bottom starts at a finite value of 'b' rather than a point. This corresponds to instantaneous failure. The breach may form due to overtopping ($h > h_0$) or piping ($h < h_0$), where, h_0 is the dam height.

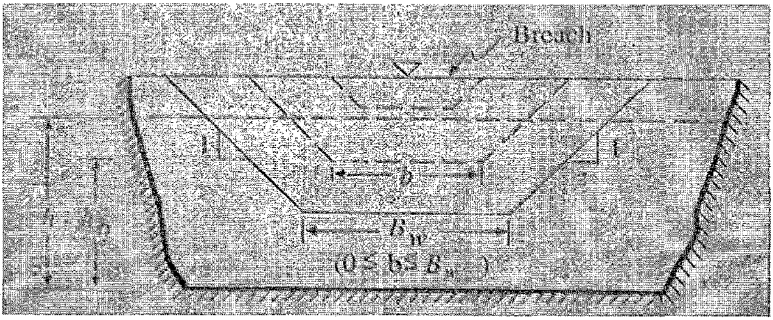


Figure 7.4: Formation of breach of the dam

7.4.3. Data Requirement

The input data requirements for the 'NWS-DAMBRK' program are flexible. When a detailed analysis is not feasible due to lack of data or insufficient data preparation time, the unknown or unavailable data can be ignored (left blank in the input file or omitted altogether). Nonetheless the resulting approximate analysis is more accurate and convenient to obtain than that could be computed by other techniques. The input data can be basically classified into two groups: pertaining to dam & upstream reach and downstream reach.

i) Data group pertaining to dam and upstream reach:

- **Reservoir data-** inflow hydrograph, length of reservoir, initial elevation of water in reservoir, elevation of water in reservoir when breach occurs, elevation of top of dam, elevation of bottom of dam, and reservoir volumes or surface areas and their corresponding elevations. For dynamic routing in the reservoir, cross-section details are required, as follows: mileage of the cross sections from the dam, a table of top widths (active and inactive), and corresponding elevations at each sections, hydraulic resistance coefficients (Manning's roughness coefficients), expansion/contraction coefficients, slope of the downstream channel for the first mile below the dam, and initial conditions in the upstream channel/reservoir.
- **Breach data** - time taken for the full breach formation, final bottom width of breach, side slope of breach, and final elevation of breach bottom.
- **Spillway data-** spillway rating curve, elevation of uncontrolled spillway crest, coefficient of discharge of uncontrolled spillway, elevation of centre of

submerged gate opening, coefficient of discharge of crest of dam, and constant discharge from dam like discharge through turbines.

ii) Data group pertaining to downstream routing reach:

- **Cross section details** - mileage of the cross sections from the dam, a table of top widths (active and inactive), and corresponding elevations at each sections, hydraulic resistance coefficients (Manning's roughness coefficients), expansion/contraction coefficients, slope of the downstream channel for the first mile below the dam, and initial conditions in the downstream channel.

7.4.4. Program Capabilities:

- **Reservoir Routing:** An inflow hydrograph can be routed through a reservoir using either storage or dynamic routing. Outflow at the dam at any instant is computed by summing the discharge over the spillway, over the top of the dam, through the breach, through a gated outlet and through turbines,
- **Breach Simulation: Two types of breaching may be simulated:**
    - An overtopping failure in which the breach shape can be triangular, rectangular or trapezoidal which grows progressively downward from the dam crest with time.
    - A piping failure in which the breach can be simulated as a rectangular orifice that grows with time and is centered at any specified elevation within the dam. If the elevation of water surface in the reservoir, when breach occurs, is below the top of the dam, the model will automatically take the failure as a piping failure.
- **River Routing**: The breach outflow hydrograph is routed through the downstream river valley using the one-dimensional St. Venant's equations.

7.4.5. Other Capabilities

i) **Lateral Inflow and Outflow:** The program treats the flow as being uniformly distributed in a reach between two adjacent downstream cross-sections. The user must specify the sequence number of the cross-section immediately upstream of where the lateral flow occurs.

ii) **Super-Critical Flow:** The 'DAMBRK' program can simulate flow that is either sub-critical or super-critical. However, only one type of flow can be accommodated in a given routing reach throughout the duration of the flow. Super-critical flow usually occurs when the slope of the downstream valley exceeds about 9.5 m/km (= 50 ft/mile). In that case two upstream boundary conditions, i.e. reservoir outflow hydrograph and a looped rating curve based on the Manning's equation in which the slope is defined as the water surface slope at the end of the previous time period, are required.

iii) **Multiple-Dam Modeling:** DAMBRK has the capability to model a situation in which two or more dams exist in series. There exists a choice of two methods for simulating dam break flows in a valley having multiple dams.

iv) **Flood Plain Modeling:** For situations in which the main channel and overbanks each carry substantial portions of the flow, and the mean velocity in the main channel differs largely from that in the overbanks, the flood plain modelling capability of 'DAMBRK' can be used. It enables representation of a cross-section with three separate components: left overbank, main channel and right overbank. The program determines conveyance for each cross sectional components separately and sums it to obtain the total conveyance of the cross section. Separate tables of elevation versus width and sets of 'n' values and reach lengths should be specified for each component.

v) **Landslide Modeling**: DAMBRK program is capable of simulating the generation of a wave due to landslide into a reservoir.

vi) **Routing Losses:** 'DAMBRK' is also able to simulate losses of water that vary with time in accordance with flow magnitude. The user is required to specify the maximum rate of lateral outflow.

The 'DAMBRK' program has the capability of simulating 12 different cases corresponding to different combinations of various reservoir and channel routing techniques with the above special options.

7.5. METHODOLOGY

A brief description of the methodology used for the basic program capabilities is given in this chapter.

7.5.1. Reservoir Routing

In this model, the reservoir routing may be performed either using storage routing or dynamic routing.

a. **Storage Routing:** Under the assumption that the reservoir surface is horizontal at all times, the hydrologic storage routing technique based on the law of conservation of mass:

$$I - Q = \frac{ds}{dt} \tag{7.3}$$

where I=reservoir inflow, Q= reservoir outflow, ds/dt= rate of change of storage volume.

Equation (1) can be expressed in finite difference form as:

$$(I + I')/2 - (Q + Q')/2 = \delta s / \delta t \tag{7.4}$$

in which I' and Q' denotes values at time t and $(t + \delta t)$ and the notation approximates the differential. The term (δs) may be expressed as,

$$\delta s = (As + A's)(h - h')/2 \tag{7.5}$$

in which, As is the reservoir surface area corresponding to the elevation h and it is a function of time t. The discharge Q which is to be evaluated from equation (7.4) is a function of h and this unknown h is evaluated using Newton-Raphson iteration technique and thus the discharge corresponding to h is estimated.

b. **Dynamic Routing:** When the breach is specified to form almost instantaneously so as to produce a negative wave within the reservoir, and/or the reservoir inflow hydrograph is significant enough to produce a positive wave progressing through the reservoir, a routing option which simulates the negative and/or positive wave occurring within the reservoir may be used in 'DAMBRK' model. Such a technique is referred to as dynamic routing. The routing principle is the same as dynamic routing in river reaches and it is performed using St. Venant's equation. The St. Venant's equations are based on conservation of mass:

$$\frac{\partial Q}{\partial x} + \frac{\partial (A + A_0)}{\partial t} = q \tag{7.6}$$

and conservation of momentum:

$$\frac{\partial Q}{\partial t} + \frac{\partial (Q^2 / A)}{\partial x} + gA\left[\frac{\partial h}{\partial x} + S_f + S_e\right] + L = 0 \tag{7.7}$$

where, A and A_0 are active and inactive flow areas, x= distance along the channel, t= time, q = lateral inflow or outflow /unit distance along the channel, g = gravitational acceleration, Q= discharge, h = water surface elevation, S_f = friction slope and S_c= expansion-contraction loss slope.

The friction slope and expansion-contraction loss slope are evaluated by the following equations

$$S_f = \frac{n^2/Q/Q}{2.21A^2R^{4/3}} \tag{7.8}$$

and

$$S_e = \frac{K\Delta(Q/A)^2}{2g\Delta x} \tag{7.9}$$

where n= Manning's roughness coefficient, R = A/B where B = top width of active portion of the channel and K = expansion-contraction coefficient.

7.5.2. Reservoir Outflow Computation

The total reservoir outflow Q at any instant is the sum of flow through the breach, flow through dam outlets, spillway and over the dam crest. As already mentioned, two types of breaching may be simulated. Flow through an overtopping breach at any instant is calculated using a broad-crested weir equation. In the case of a piping failure, instantaneous flow through the breach is calculated with either orifice or weir equations depending on the relation between pool elevation and the top of the orifice. The breach begins when the reservoir water surface elevation exceeds a user specified elevation H_f and grows linearly in time until H_b=H_{bm}, where H_b is the elevation of the breach bottom at any time and H_{bm} is the final elevation of the breach bottom. H_{bm} is usually taken to be the channel bottom or the dominant ground elevation of the dam, except when this is not physically justifiable due to backwater effect. Therefore, cross sectional information immediately downstream of the dam in order to calculate tail water elevation for any needed correction for partial submergence is required. An overtopping failure is simulated if $H_f = H_d$ where H_d is the elevation of top of the dam.

The peak shape of the outflow hydrograph due to dam breach is governed largely by the geometry of the breach and its development with time. The tail water is estimated from Manning's equation. The geometric properties for this are obtained from the input cross section immediately downstream of the dam. This estimated tail water depth does not include any dynamic effects or back water effects due to downstream constrictions. When such effects are there, the simultaneous method of computation should be used.

In this study, OPTION 4 and OPTION 7 of DAMBRK model are used for analyzing the wave propagation characteristics. The former includes dynamic routing in both upstream and downstream reaches of the dam, and the latter does dynamic routing only through the river valley considering there is no dam existing.

7.6. INPUT DATA

In the present dissertation work, a study area of length 3.1 km along the Bengawan Solo river, from downstream of Wonogiri dam, has been considered. The width of the river near the dam site is nearly 100 meter. It ranges from 100 to 200 m at downstream locations. Average bed slope of the river in this area is 1 in 3000.Schematic diagram of solo river downstream of dam is given in Figure 7.5.

As per the requirement of this study, data have been collected from project report. However, no data is available for bed roughness of the study area. The breach parameters are also not available as Wonogiri dam has never failed.

In this section the used data, both available and assumed, are presented. A case of hypothetical dam break is considered in the present study and the flood due to this is routed upto 3.1 km downstream of dam. The inflow hydrograph to the reservoir is assumed (Figure. 7.7). In this figure the recession limb of the hydrograph is shown. It is assumed that the rising limb of the hydrograph results in filling the reservoir upto top of dam. It may be noted that, the design discharge for the spillway is 5,100 m^3/s and the peak discharge in the inflow hydrograph is 9,578 m^3/s. The area-elevation relationship for the reservoir is presented in Figure. 7.8. Although the capacity of the reservoir (volume) corresponding to different elevations are available for a more accurate computation, the surface area elevation relationship of the reservoir is provided to the model as input. The discharge through the spillway is shown as a rating curve in Figure. 7.9. In this figure, x-axis represents the discharge in m^3/s and y-axis the head in meter over spillway.

The dam is assumed to break by overtopping failure. The breach parameters are assumed and the used values are: breach width = 150 m, time to breach =1 hr, and side slope of breach = 0 which corresponds to a vertical breach section. The final level of breach corresponds to the channel bed level (119. 0 m) at dam site. The deepest bed levels along the river, downstream of the dam, are presented in Figure. 7.10. The cross sections covering the flood plain area, located at eight different locations are presented in Figure. 7.11. (a-h). In these figures, widths (m) and elevations (m) are shown in the x and y axis respectively. 'O' in the x-axis represents the ground level at the left bank of Bengawan solo river. These cross sections are obtained from different project documents and survey of Bengawan Solo river topo sheets. The bed roughness coefficient is 0.03 for the main river. The computational distance step sizes (Δx) are different for different reaches. An input data file used for the computer programme is given in Appendix -3.

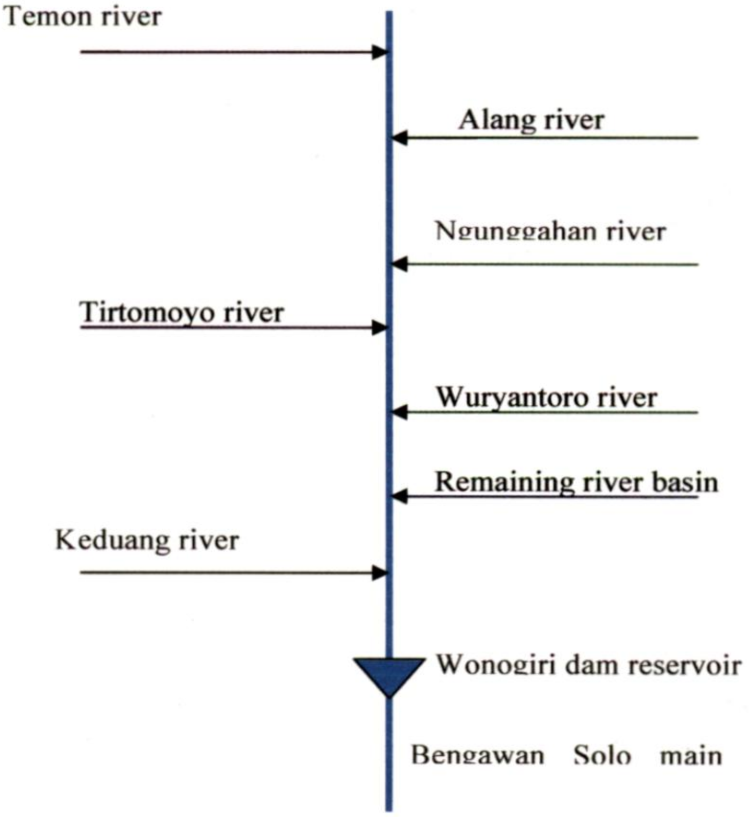


Figure 7.5: Schematic Diagram of Solo River reach from Wonogiri Dam Reservoir

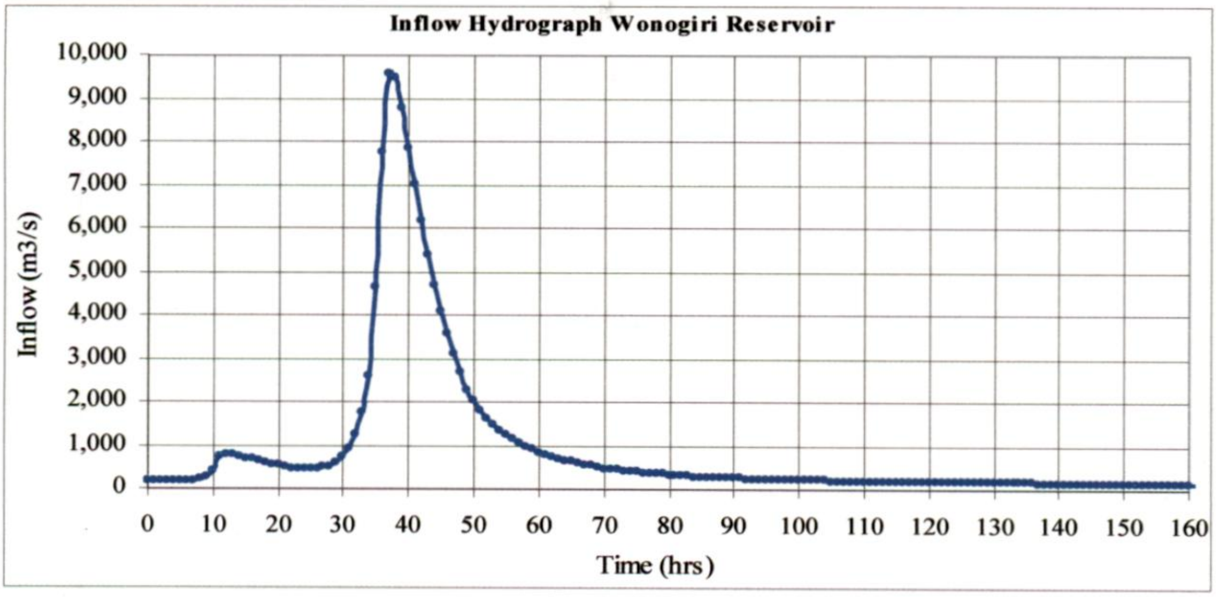


Figure 7.6: Inflow Hydrograph of Wonogiri Dam

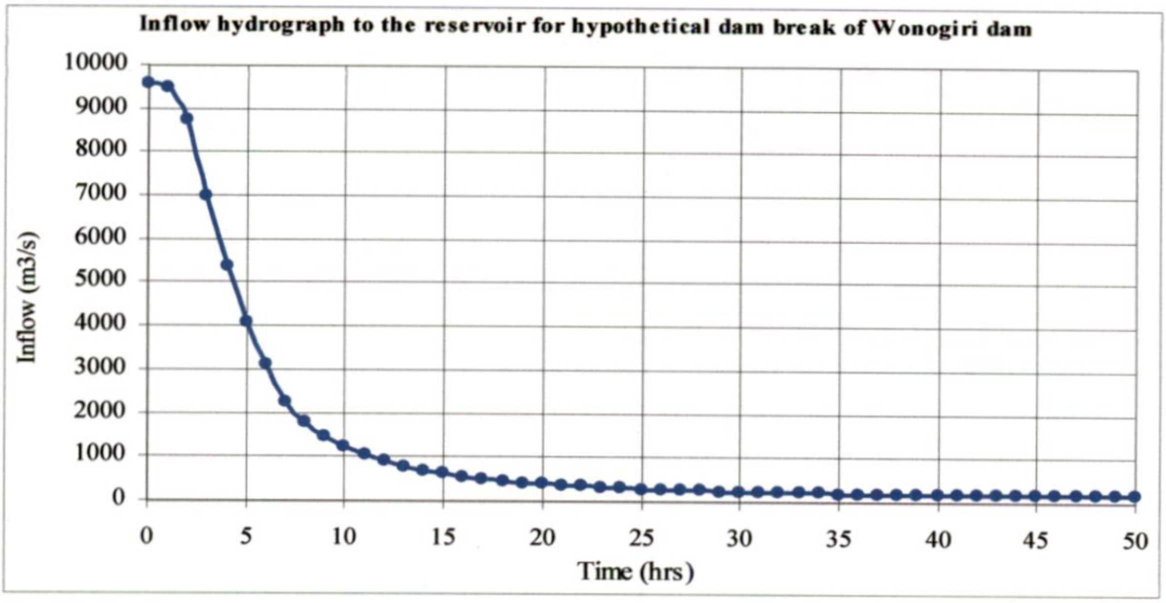


Figure 7.7: Assumed Inflow Hydrograph for Hypothetical Dam Break

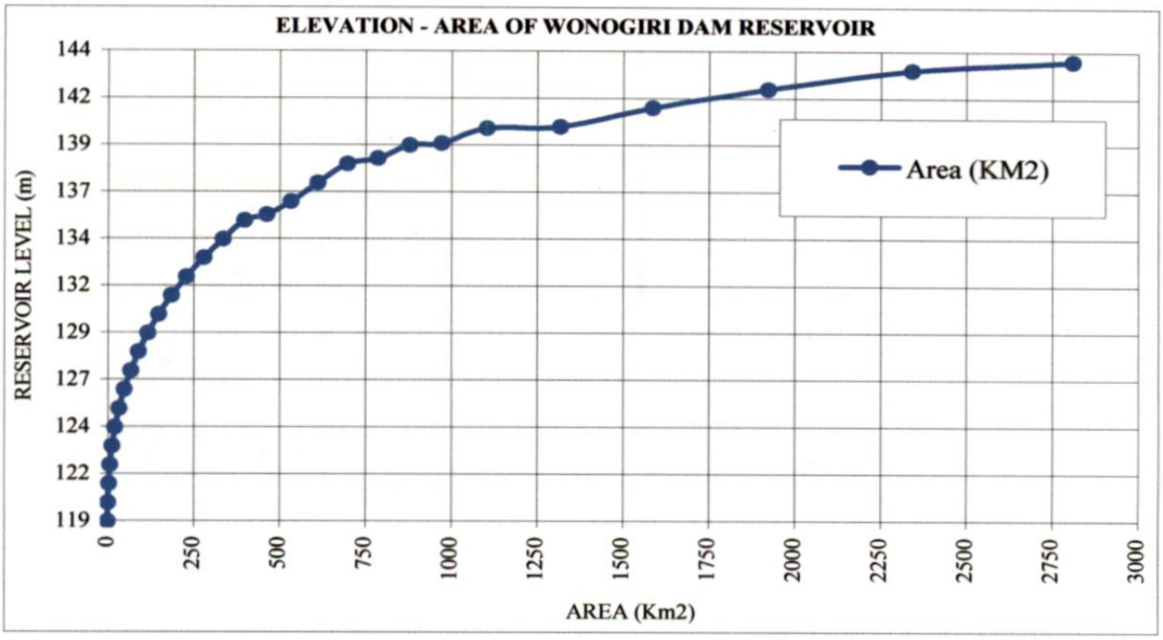


Figure 7.8: Elevation-Area of Wonogiri Dam Reservoir

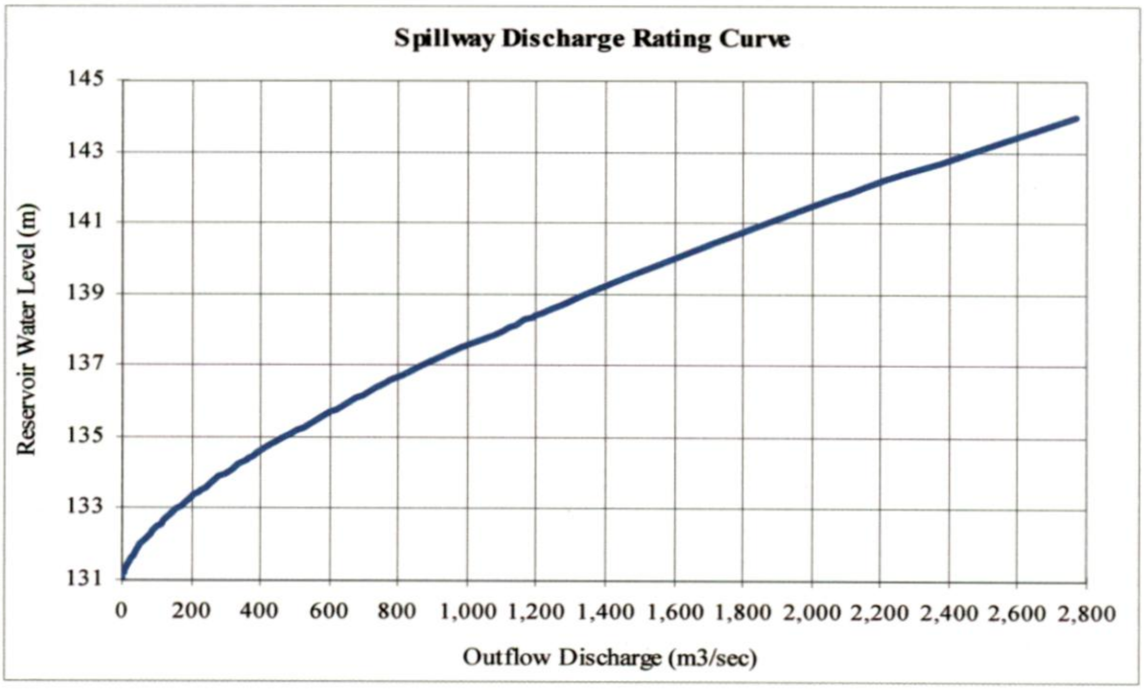


Figure 7.9: Spillway Rating Curve of Wonogiri Dam Reservoir

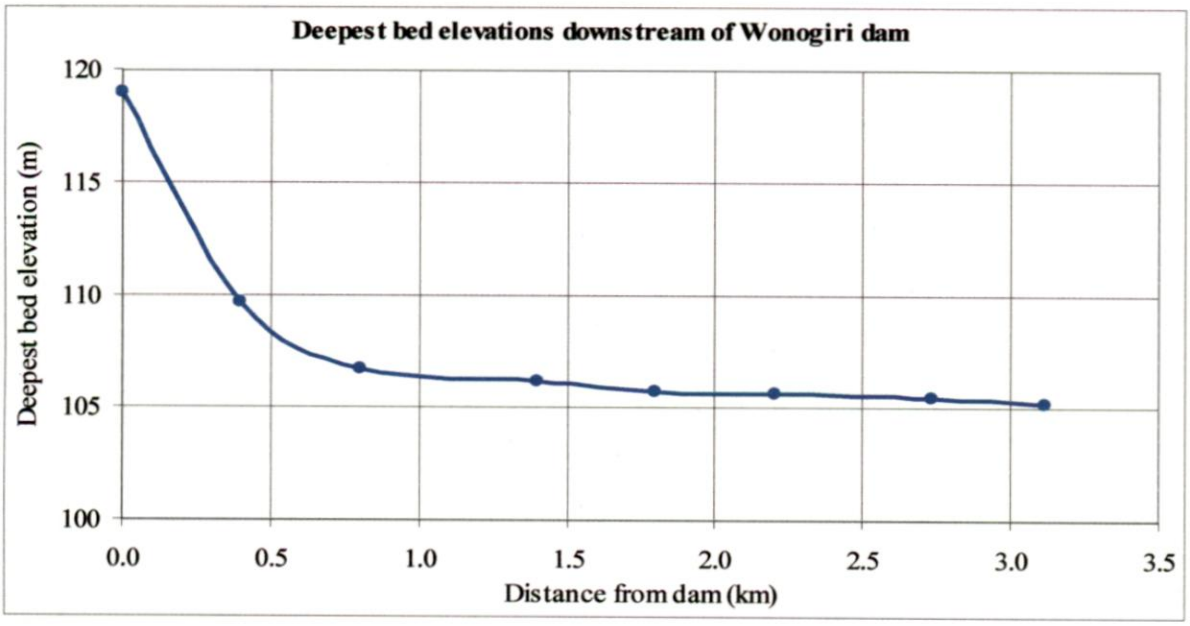
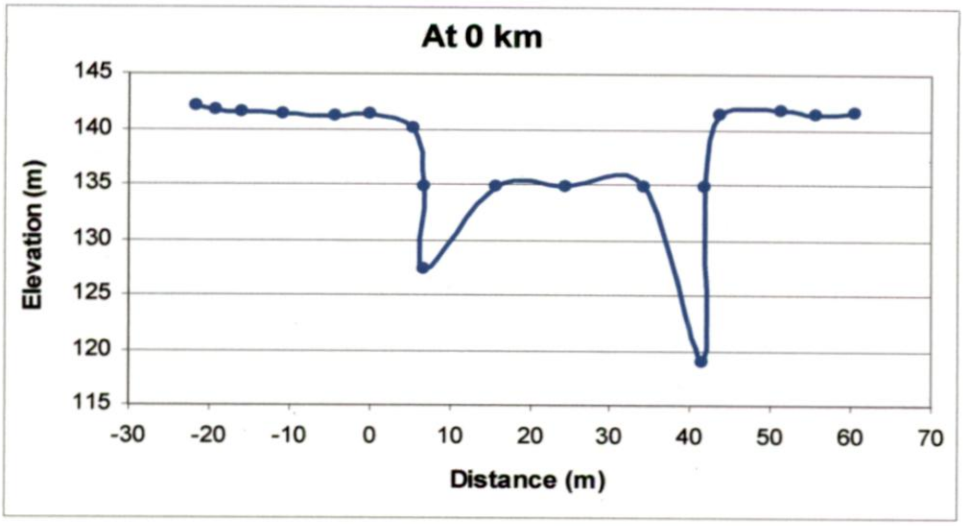
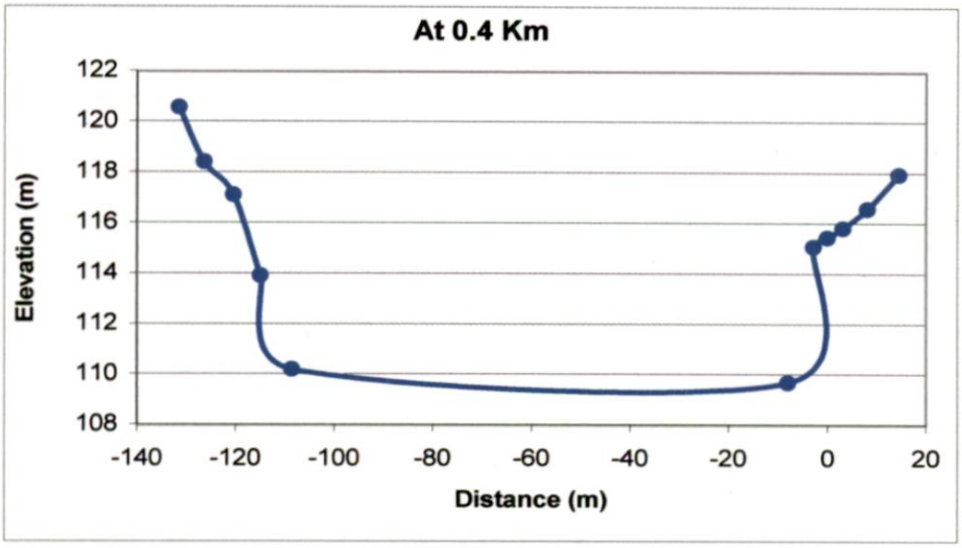


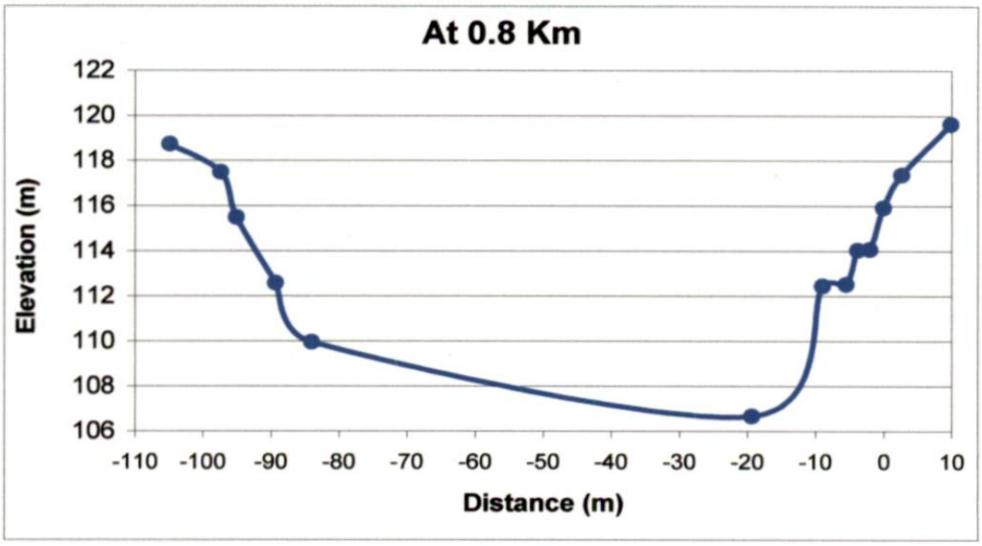
Figure 7.10: Deepest Bed Elevations Downstream of Wonogiri Dam Reservoir



a) At 0 Km

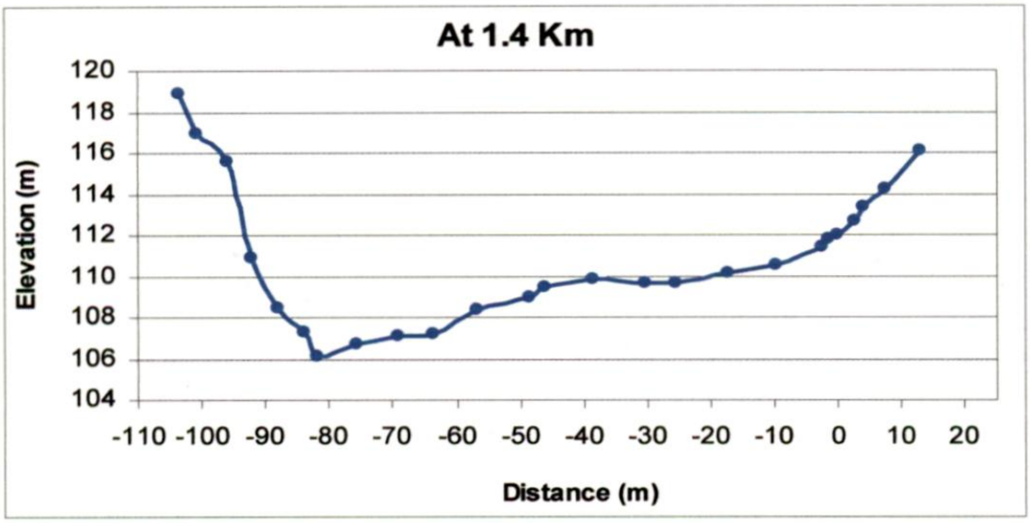


b) At 0.4 Km

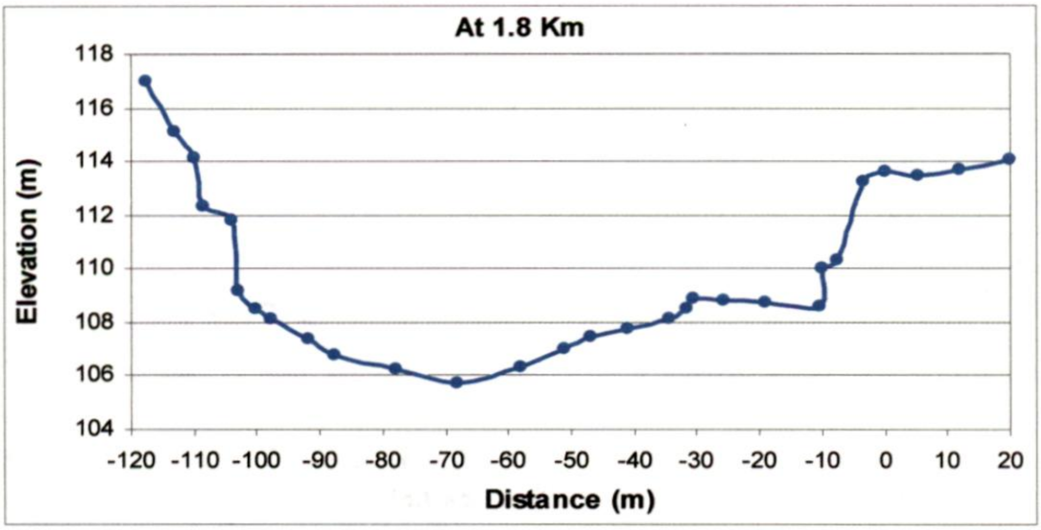


c) At 0.8 Km

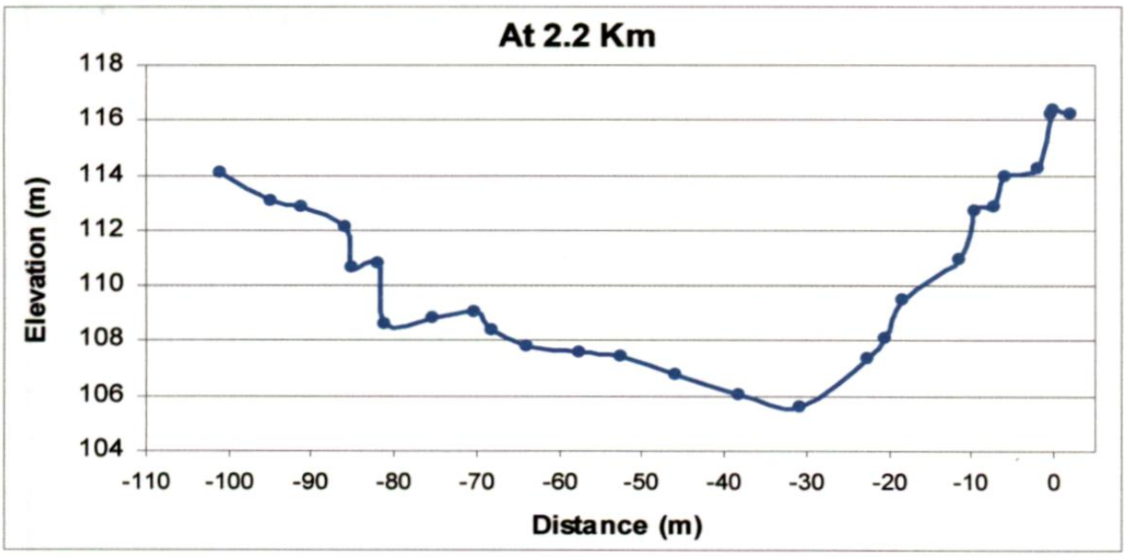
Figure 7.11: Cross Section Downstream of Wonogiri Dam Reservoir



d) At 1.4 Km

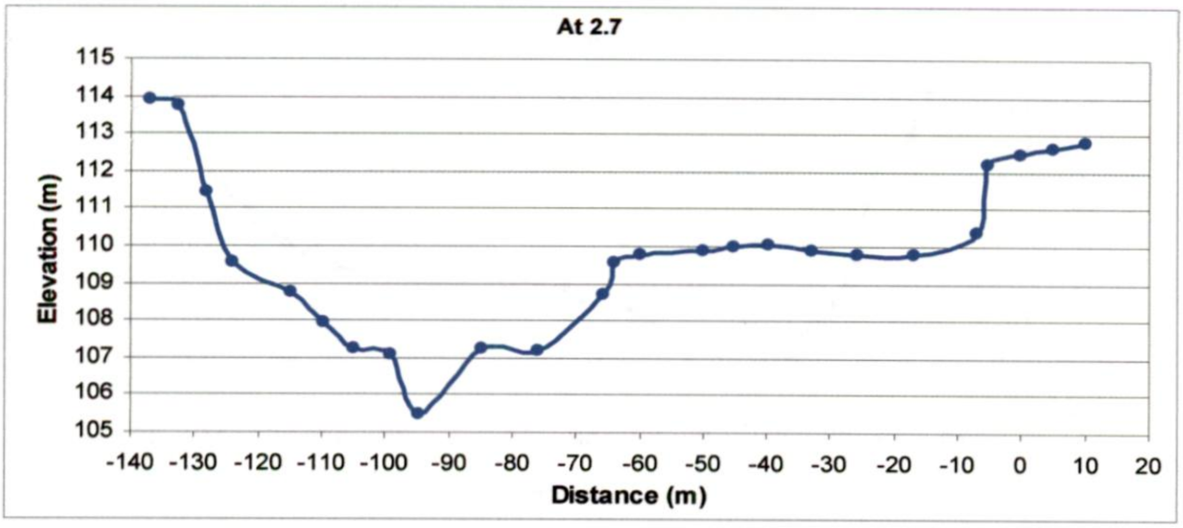


e) At 1.8 Km

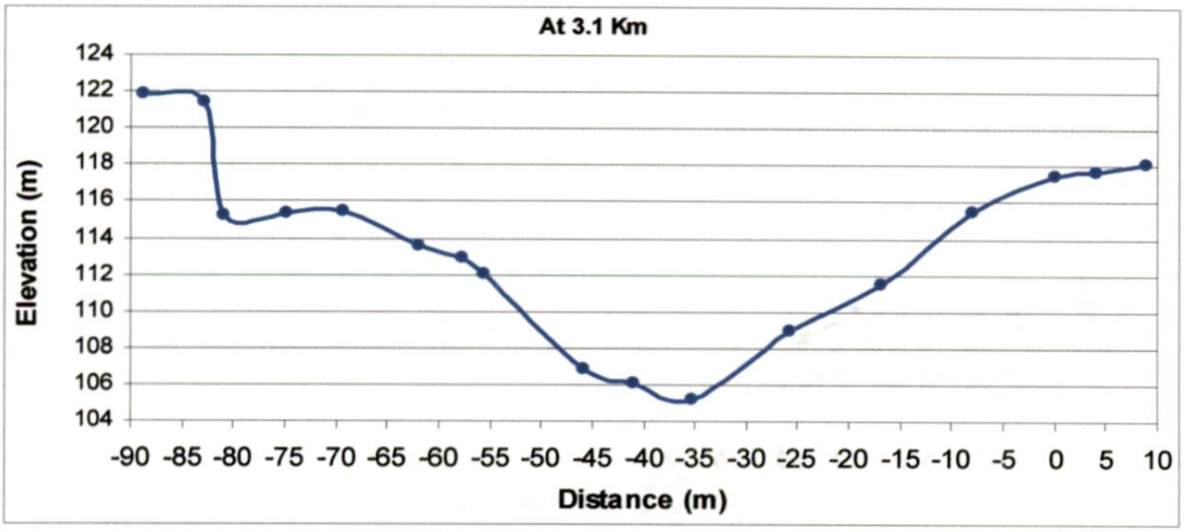


f) At 2.2 Km

Figure 7.11: Cross Section Downstream of Wonogiri Dam Reservoir (Contd)



g) At 2.7 Km



h) At 3.1 Km

Figure 7.11: Cross Section Downstream of Wonogiri Dam Reservoir (Contd)

7.7. RESULT AND DISCUSSION

7.7.1. Routing of Design Discharge

Before considering the dam break flow, a case is considered for the design discharge through the spillway. This flow is routed in the study area i.e. upto 3.1 km downstream of Wonogiri dam. The reservoir water level for this case is assumed to be at F R.L. i.e. 142.0 m. The result of computations using DAMBRK is presented in Figure. 7.12. In this figure, maximum water elevations attained at different locations downstream of dam are shown.

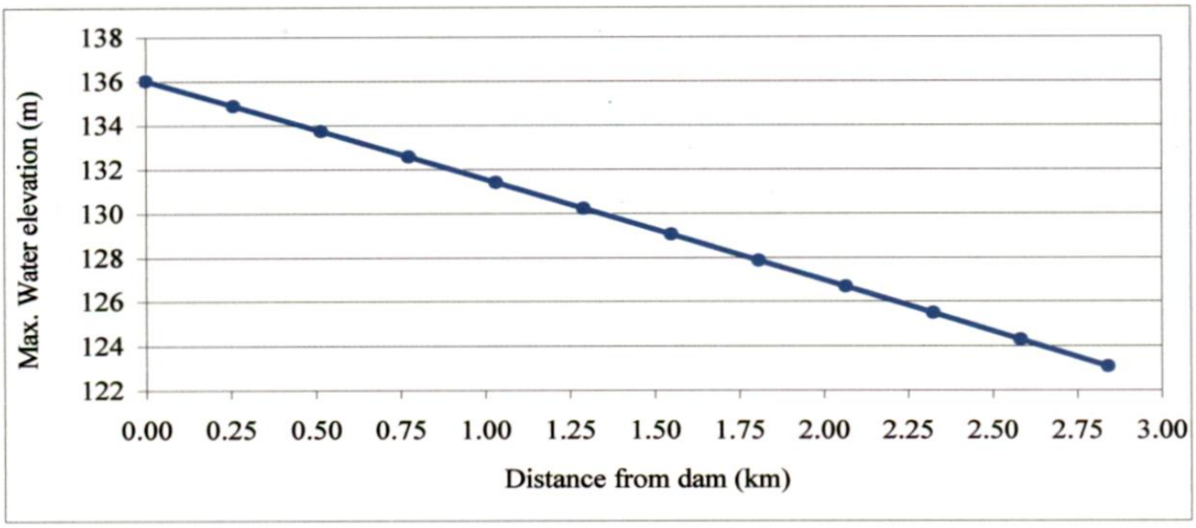


Figure 7.12: Maximum Water Elevation Downstream of Wonogiri Dam Reservoir due to the Design Discharge of the Spillway

7.7.2. Dam Break Flow

The data described in earlier section are used as input to study the dam break flow. Different results obtained from the output of the computer programme, are described below.

The reservoir depletion table due to the dam break flow is given in Table, Appendix-4. The resulting hydrograph due to the hypothetical dam break at the dam site and at 3.1 km are presented in Figure 7.13 and Figure 7.14 respectively. The peak discharge at the dam site is 19,289 m^3/s and it gradually decreases to 19,203 m^3/s at a distance of 3.1 kms. (Figure 7.14). The peak discharge decreases and time to peak discharge increases as the dam break flood moves downstream. This indicates the general characteristics of a flood wave propagation. The stage hydrographs, for this dam break study, at dam site, and at 3.1 km are presented in Figure. 7.15. A summary of results for this case is given in Table 7.2.

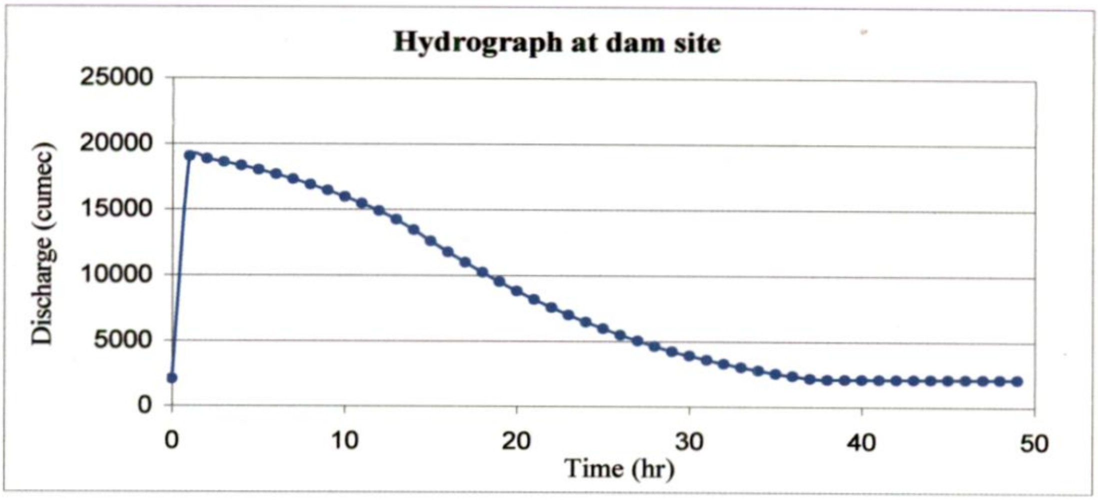


Figure 7.13: Out flow Hydrograph at the Dam Site

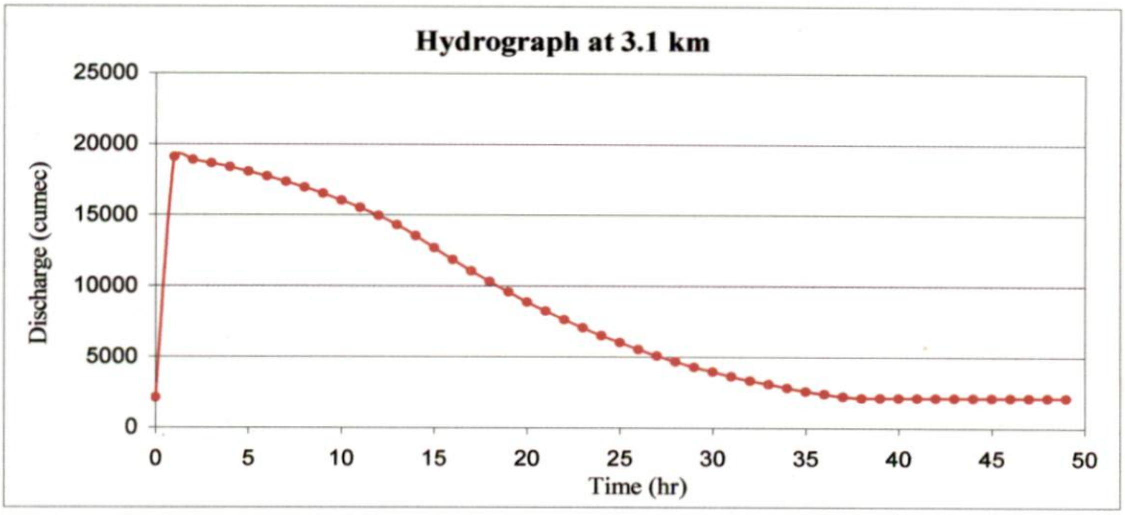


Figure 7.14: Out Flow Hydrograph at 3.1 km

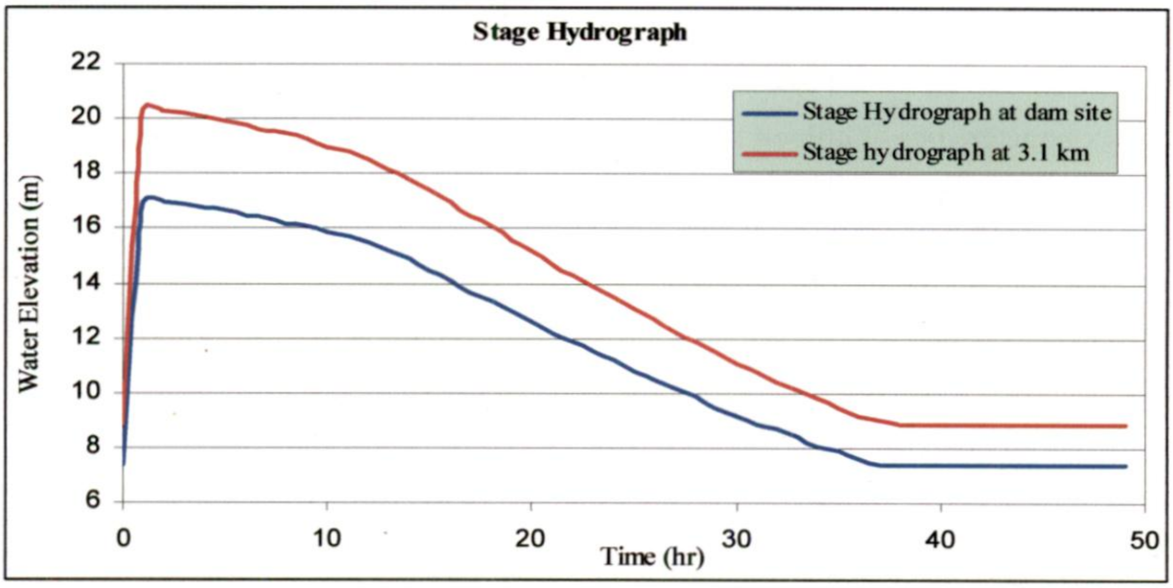


Figure 7.15: Stage Hydrograph at Dam Site and at 3.1 km

Table 7.2: Summary of Results for Dam Break Flow

Distance from dam (km)	Peak discharge (cumecs)	Maximum Elevation (m)	Time to Maximum Elevation (hr)
0.00	19289.00	136.06	0.460
0.26	19253.00	134.92	0.490
0.52	19245.00	133.77	0.510
0.78	19239.00	132.61	0.540
1.03	19235.00	131.44	0.560
1.29	19230.00	130.26	0.590
1.55	19226.00	129.08	0.610
1.81	19222.00	127.89	0.631
2.07	19218.00	126.70	0.652
2.32	19214.00	125.50	0.674
2.58	19210.00	124.30	0.696
2.84	19207.00	123.09	0.718
3.10	19203.00	121.89	0.741

7.8. CONCLUSION

In this dissertation work, dam break flow analysis for Wonogiri dam has been performed assuming a hypothetical dam failure case. A mathematical model 'DAMBRK' has been used for this purpose. This model employs one-dimensional St. Venant equations and four Point Preismann scheme for channel routing. Data required for the above study are obtained from project report. As it was a case of hypothetical dam failure, breach parameters were assumed. Conclusions derived from the present study are given below.

i) The dam break peak discharge at the dam site is 19,289 m^3/sec and it gradually decreases to 19,203 m^3/sec at a distance of 3.1 kms.

ii) The dam break peak discharge at dam site is 19,289 m^3/s is significantly higher than the probable maximum flood (9600 m^3/sec).

iii) The maximum water level at the dam site is 136.06 m and at 3.1 km distance it is 121.89 m.

CHAPTER VIII

CONCLUSION

1. There have been 26 major dam failures in India in post independence period. Further, literature review of flood estimates of 62 large dams shows those reassessed design floods are significantly larger than earlier estimates.
2. Dam failures reported in literature are not a random sample and represent perhaps the worst out of all the existing dams. Hence it may be wrong to assume that all existing dams are unsafe.
3. Hydrology is a data based science. With occurence of more severe events in a large sample, earlier estimate of design flood are bound to be revised up ward as illustrated through flood reviews study of Hirakud dam, Gandhi Sagar dam and other dams in India.
4. Due to intensive flood plain occupancy (economic development and increase in density of population) in downstream of dam, it is of almost importance to eliminate possibility of dam failure either completely or partially as there will be extensive damage to property and heavy loss of life in case of dam failure.
5. A critical flood analysis based on dam break study is justified for the important dams. It should be possible to accept the cost of dam safety analysis if the same results in a) acceptance of reduced design flood and acceptance of higher risk of failure or b) acceptance of increased design flood for safety of property and life in downstream flood plain.
6. Physical process of flood formation is not fully understood therefore a degree of subjectivity is involved in modeling and estimation of design storm and design flood. Subjectivity should be minimised by evolving consensus and codifying the criteria, and procedures for estimation of design flood.
7. When various storms are considered for development of Unit Hydrograph for the same catchment a marked variation is observed in the peak as well as the time of occurrence of the peak. Therefore average Unit Hydrograph needs to be derived giving higher weightage for the Unit Hydrograph derived from severe storms. Different unit hydrographs should be identified for the various conditions which

have major influence on formation and time distribution of the runoff. These unit hydrograph may then be judiciously applied under different conditions.

8. Methods used in separation of losses from storm rainfall are empirical and arbitrary. Instead of assuming an average infiltration of loss rate (0-index) for entire storm, different loss rates in different portions of the storm, can be assumed. (Φ-index underestimates losses in beginning portion of storm and over estimates loss rate in later portion of storm. Further (φ-index may significantly vary spatially due to different land use soil cover and soil characteristics and antecedent moisture condition. Therefore different 0-index may be used for different areas.
9. Methods used for base flow separation (while deriving Unit Hydrograph) or addition (while estimating flood hydrograph) are rather arbitrary. Same method should be consistently used in derivation of Unit Hydrograph and application of Unit Hydrograph.
10. Recent flood hydrographs should be used for derivation of UH. Changes in land use, land cover over the catchment area should be evaluated using remote sensing data and should be duly considered in the analysis.

Errors in forecasting floods in term of estimation of peak discharge and time to peak due to use of different UH when rainfall is not uniform and when rainfall is assumed to be uniform over the catchment are significant as shown below. In Baitarni basin the observed flood was 2180 cumec and observed time to peak was 18 hours.

When non uniform rainfall distribution is considered		When rainfall is assumed to be uniform	
Qp (cumecs)	Tp (hrs)	Qp (cumecs)	Tp (hrs)
1948.65	21	2244.92	18
Error in Qp Estimation (%)	Error in Tp Estimation (%)	Error in Qp Estimation (%)	Error in Tp Estimation (%)
10.61	-16.67	-2.98	0

A perfect forecast exist only if both Bias (B) and Variance (V) are zero. Bias and Variance are important parameters in addition to coefficient of determination (R^2) (Maidment, 1993).

Reliability of rising portion only is important in forecasting of rise in water level at the forecasting site such as for flood embankment, town situated on river bank. For reservoir complete hydrograph has to be forecasted.

From the reliability analysis, it is concluded that Collin's Unit Hydrograph can be used for flood forecasting at a town on river bank where only size of water level need to be forecast. Higher Coefficient of Determination and lower value of Bias are obtained for complete Unit Hydrograph in comparison to rising portion only.

11. Elements of risk and uncertainty are inherent in any flood frequency analysis as subjectivity is involved in making choice about length of data, method of probability distribution, plotting position etc. Effect of various factors on design flood estimation is analysed through case study of floods at Bhakra dam site on river Sutlej in India. Following analysis have been carried out using the observed data:
12. Peak and trough analysis shows that the data series is random. Effect of length of data is analysed by considering following three different series:

    iv) Pre construction flood series (1909 to 1959):

    v) post construction flood series (1960 to 1992):

    vi) Entire flood series (1909 to 1992):
13. Choice of plotting position formulae: Probability of exceedance of observed flood peaks have been computed using i) Hazen formula, ii) Weibull formula, iii) Gringorten formula. It is seen that lowest values are not affected by the choice of plotting formula. Highest values are significantly affected. This may cause significant error in extrapolation
14. Presence of Jump and Trends has been check by applying moving average method to mean value of twenty year data a) including highest observed peak and b) excluding the highest observed peak in the series. A rising trend in the mean is observed. Trend is significantly influenced by a single value 17,227 m^3/s observed in the year 1971.
15. Inclusion/exclusion of highest observed value as outlier: Highest observed flood is 17227 m^3/s in the year 1971 which pertains to post construction period. The flood frequency analysis in post construction period is influenced by choice for inclusion/exclusion of this rare event in the data series.
16. Choice of probabilities distribution: For the same data series, different probability distributions provide significantly different estimates e.g. using 1909-1992 data

series of Bhakra dam, 10000 year estimate by EVI is 21,036.26 m^3/s and by PIII is 26,154.53 m^3/s (24.33 % higher).

17. Effect of different samples: Using same probability distribution but different samples from same population also result in significantly different estimates. 10000 year flood estimate using LN II probability distribution are 18,732.75 cumec (1909-92 data series), 15,064.06 cumec (1909-59 data series) and 24,588.81 m^3/s (1960-92 data series).
18. Keeping in view the uncertainties unestimation of PMP and to maintain uniformity in procedure for PMP procedure, the practice as recommended by Central Water Commission (CWC 1993) should be followed.
19. Case study of Wonogiri Reservoir
    i) Improvements are necessary in observation and recording of rainfall and discharge data. This is very important for safety of dam.
    ii) Clock hour correction is not constant (50 mm as followed in India) but is based on an adjustment factor which is 1.13.
    iii)Area reduction factor is based on basin mean daily rainfall and maximum point rainfall.
    iv)Largest depths of PMP (mm) for Wonogiri watershed estimated by Hershfield equation for any duration are shown in Table below.

Day	Duration (hr)	Depth (mm)	Station
-	1	22.4	Jatisrono 131
1	24	234.3	Beji
2	48	248.9	Puwantoro
3	72	383.6	Puwantoro
4	96	401.7	Beji
7	168	700.9	Beji

20. HEC-HMS can be used to compute the PMF, using PMP depths as input. Likewise, dam and spillway performance can be simulated with the reservoir model included in HEC-HMS. For that reason, the analysis must derive and specify functions that describe how the reservoir will perform.
21. The peak spillway discharge is 5678.5 m^3/sec as computed using HEC-HMS software whereas the spillway capacity is 5100 m^3/sec. The spillway capacity of 5100 m^3/s corresponds to 1.2 times 100 years probable flood (Table 6.2) where as the probable maximum flood as given in project report (Table 6.2) is 9600 m^3/sec.

Therefore it is concluded that there is adequate spillway capacity in the Wonogiri dam.

22. In this dissertation work, dam break flow analysis for Wonogiri dam has been performed assuming a hypothetical dam failure case. A mathematical model 'DAMBRK' has been used for this purpose. This model employs one-dimensional St. Venant equations and four Point Preismann scheme for channel routing. Data required for the above study are obtained from project report. As it was a case of hypothetical dam failure, breach parameters were assumed. Conclusions derived from the present study are given below:

    i) The dam break peak discharge at the dam site is 19,289 m^3/sec and it gradually decreases to 19,203 m^3/sec at a distance of 3.1 kms.

    ii) The dam break peak discharge at dam site is 19,289 m^3/s is significantly higher than the probable maximum flood (9600 m^3/sec).

    iii) The maximum water level at the dam site is 136.06 m and at 3.1 km distance it is 121.89 m.

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APPENDICES

Appendix 1: Input data for PMF estimation and evaluation of spillway adequacy using HEC-HMS software.

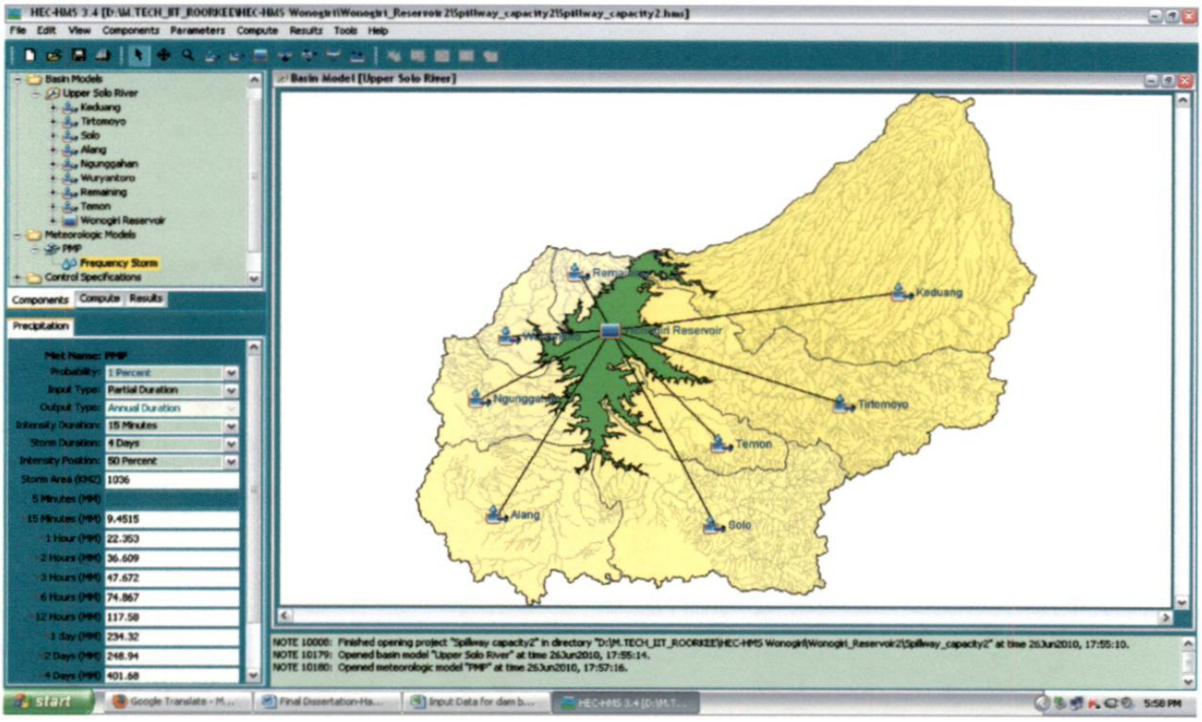
a. Table spillway rating curve (Elevation-Discharge)

Elevation (m)	Discharge (m^3/s)	Elevation (m)	Discharge (m^3/s)	Elevation (m)	Discharge (m^3/s)	Elevation (m)	Discharge (m^3/s)
131.0	0.000	132.8	132.581	134.6	395.487	136.4	737.850
131.1	1.622	132.9	143.781	134.7	412.080	136.5	762.310
131.2	4.588	133.0	155.281	134.8	428.898	137.0	868.589
131.3	8.429	133.1	170.723	134.9	445.939	137.5	984.366
131.4	12.978	133.2	183.061	135.0	463.200	138.0	1100.103
131.5	18.137	133.3	195.684	135.1	488.150	138.3	1165.660
131.6	24.539	133.4	208.583	135.2	506.117	138.5	1213.890
131.7	30.923	133.5	221.755	135.3	524.300	139.0	1337.280
131.8	37.781	133.6	237.708	135.4	542.696	139.5	1464.589
131.9	45.081	133.7	251.553	135.5	561.301	140.0	1595.700
132.0	52.800	133.8	265.656	135.6	580.115	140.5	1730.505
132.1	62.299	133.9	280.014	135.7	599.134	141.0	1868.906
132.2	70.985	134.0	294.622	135.8	618.357	141.5	2010.812
132.3	80.040	134.1	314.387	135.9	637.781	142.0	2156.138
132.4	89.451	134.2	329.722	136.0	657.404	143.0	2456.741
132.5	99.204	134.3	345.297	136.1	677.224	144.0	2770.145
132.6	111.110	134.4	361.111	136.2	697.240		
132.7	121.687	134.5	377.159	136.3	717.449		

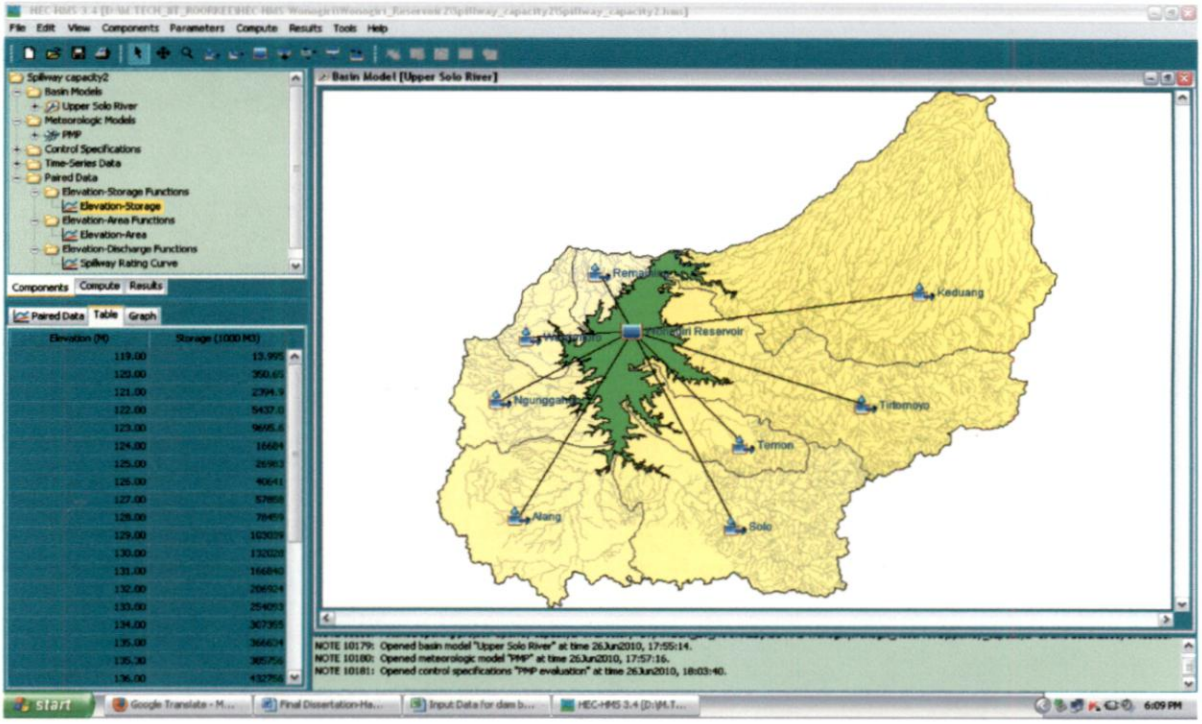
b. Table Elevation-Area relationship of Wonogiri dam reservoir

Elevation (m)	Cum Area (km2)	Elevation (m)	Cum Area (km2)
119.00	34.50	134.00	335713.800
120.00	1077.20	135.00	398206.700
121.00	3785.50	135.30	462996.797
122.00	7183.00	136.00	533125.297
123.00	12625.00	137.00	611349.697
124.00	21259.70	138.00	697632.997
125.00	33151.10	138.30	786392.397
126.00	48670.20	139.00	879015.997
127.00	67494.40	139.10	972178.268
128.00	90017.20	139.90	1079796.104
129.00	116558.60	140.00	1098072.368
130.00	148438.60	141.00	1306239.186
131.00	185689.100	142.00	1567675.346
132.00	228827.100	143.00	1895486.174
133.00	279411.300	143.50	2088921.103

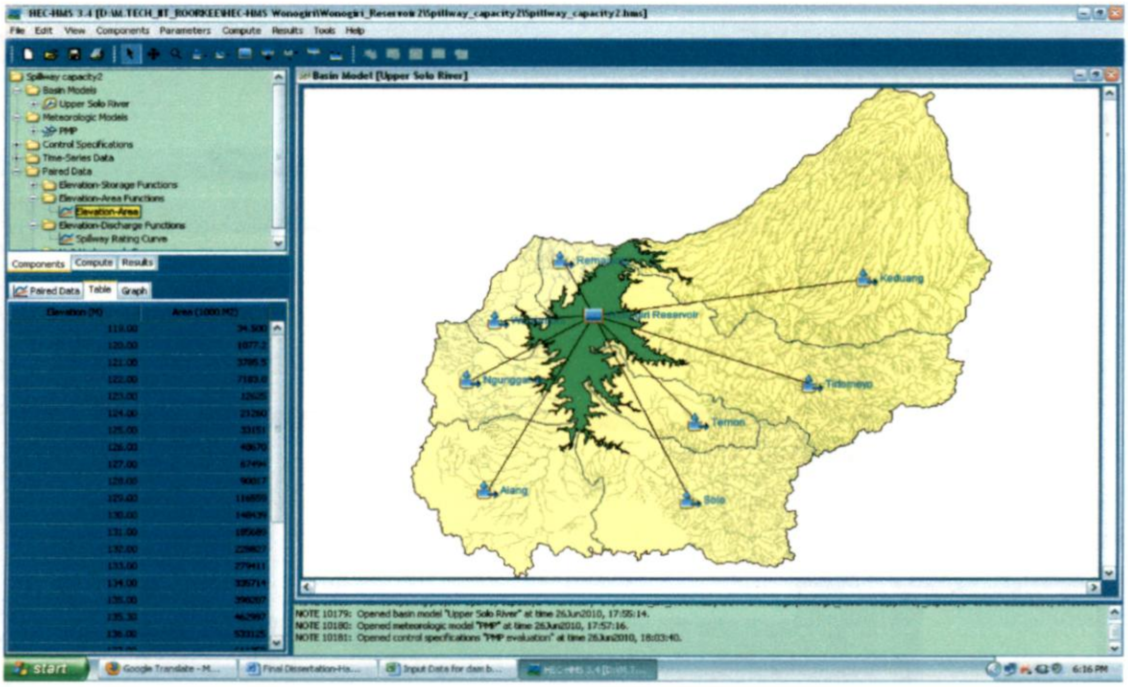
c. Meteorological model of PMF evaluation of spillway capacity of Wonogiri dam reservoir



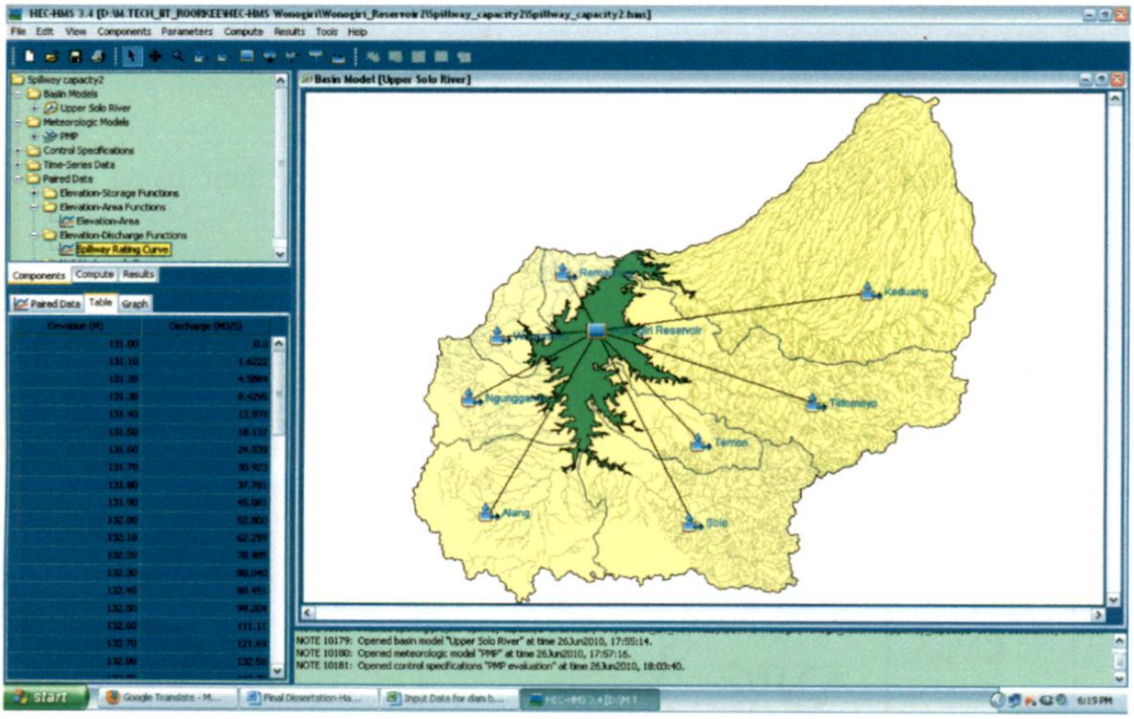
d. Input data of elevation-storage function of Wonogiri dam reservoir



e. Input data of elevation-area function of Wonogiri dam reservoir



f. Input data of elevation-discharge function of spillway of Wonogiri dam reservoir



Appendix 2: Result of PMF estimation and evaluation of spillway adequacy using HEC-HMS software.

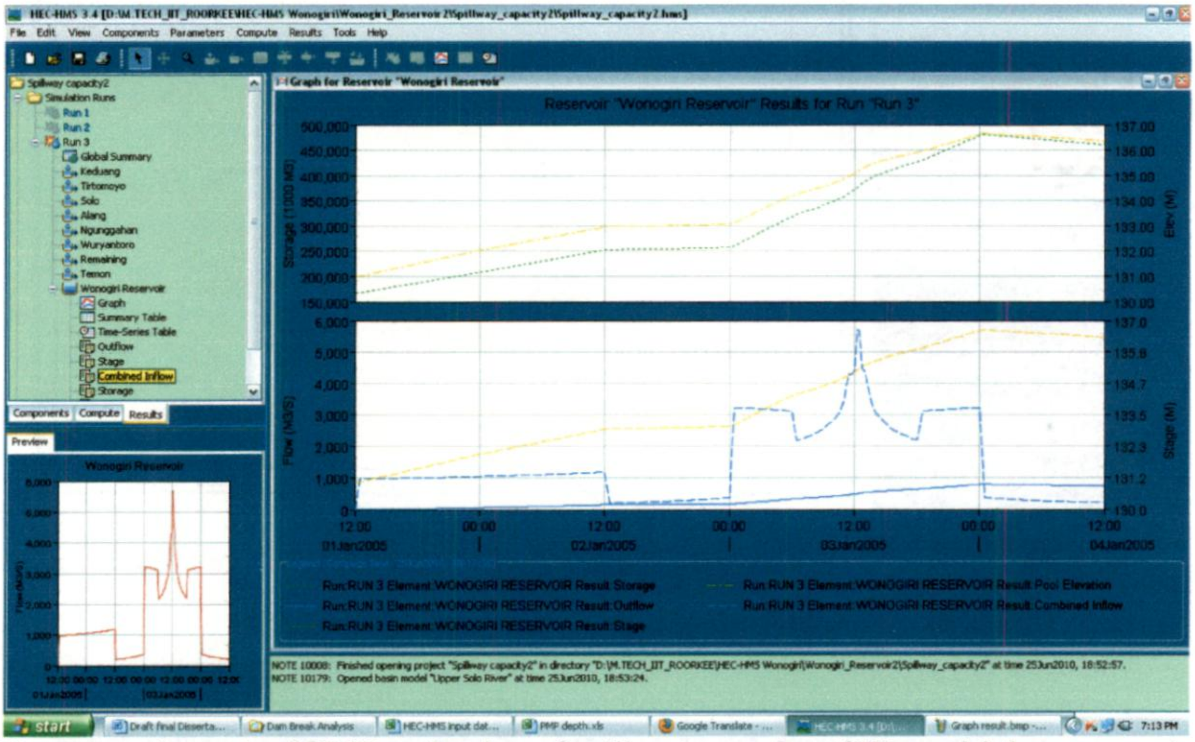


Figure graph result of PMF evaluation of spillway capacity of Wonogiri dam reservoir

Appendix 3: Input data for dam break flow for Wonogiri dam using DAMBRK software.

a. Elevation-Area relationship

Area (sq.km)	Elevation,H (m)
468.1	143.5
131.3	139.9
88.8	138.3
70.1	136.0
50.6	133.0
26.5	129.0
11.9	125.0
3.4	122.0
0.0	119.0

b. Table Stage-discharge of Spillway

Discharge Q (m3/s)	Head above spillway crest (m)
0.0	0.0
52.8	1.0
155.3	2.0
294.6	3.0
463.2	4.0
657.4	5.0
868.6	6.0
1100.1	7.0
1337.3	8.0
1595.7	9.0
2156.1	11
2770.1	12

c. Inflow hydrograph of Wonogiri dam reservoir

Time (hr)	Inflow (m3/s)	Time (hr)	Inflow (m3/s)
0	9578	26	281
2	8775	28	258
4	5383	30	240
6	3117	32	225
8	1820	34	212
10	1243	36	202
12	911	38	194
14	701	40	187
16	566	42	181
18	468	44	175
20	399	46	171
22	349	48	167
24	311	50	164

d. Table cross section at downstream of Bengawan Solo river from dam site

CS-1	XS	0.0				
	HS	119.0	134.8	140.0	141.3	141.5
	BS	0.0	16.6	36.4	43.8	79.5
CS-2	XS	0.4				
	HS	109.7	110.2	113.9	115.1	117.9
	BS	0.0	108.5	115.0	118.0	120.0
CS-3	XS	0.8				
	HS	106.7	110.0	112.5	114.1	119.6
	BS	0.0	74.0	80.3	88.0	114.8
CS-4	XS	1.4				
	HS	106.2	107.3	109.9	112.0	116.1
	BS	0.0	15.0	45.0	90.0	105.8
CS-5	XS	1.8				
	HS	105.7	107.4	108.9	110.0	113.6
	BS	0.0	60.0	72.8	92.8	110.0
CS-6	XS	2.2				
	HS	105.6	107.4	109.1	112.2	114.1
	BS	0.0	35.0	52.0	75.0	94.9
CS-7	XS	2.7				
	HS	105.5	107.1	109.6	110.0	112.2
	BS	0.0	14.4	60.1	85.0	120.0
CS-8	XS	3.1				
	HS	105.2	106.9	109.3	115.5	118.2
	BS	0.0	15.0	49.3	61.2	89.8

Appendix 4: Depletion table of dam break flow analysis for Wonogiri dam using DAMBRK software.

I	K	TTP(I)	Q(I)	H2	YB	D	SUB	VCOR	OUTV	BB	COFR	QI(I)	QBRECH	QSPIL
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	0	0	2156	142	142	126.16	1	1	0	0	3.1	9578	0	2156
2	1	0.004	2157	142	141.7	126.16	1	1.08	0	3	3.1	9576	1	2156
3	1	0.008	2161	142	141.4	126.16	1	1.04	0.1	6	3.1	9575	5	2156
4	1	0.012	2170	142	141.1	126.17	1	1.03	0.1	9	3.1	9573	14	2157
5	1	0.016	2185	142	140.8	126.19	1	1.02	0.1	12	3.1	9572	28	2157
6	1	0.02	2206	142	140.5	126.22	1	1.02	0.2	15	3.1	9570	49	2157
7	1	0.024	2234	142	140.2	126.25	1	1.01	0.2	18	3.1	9568	77	2157
8	1	0.028	2270	142	139.9	126.3	1	1.01	0.2	21	3.1	9567	113	2157
9	1	0.032	2314	142	139.6	126.35	1	1.01	0.3	24	3.1	9565	157	2158
10	1	0.036	2368	142	139.3	126.41	1	1.01	0.3	27	3.1	9564	211	2158
11	1	0.04	2432	142	139	126.49	1	1.01	0.3	30	3.1	9562	274	2158
12	1	0.044	2506	142	138.7	126.57	1	1.01	0.4	33	3.1	9560	348	2158
13	1	0.048	2590	142	138.4	126.67	1	1.01	0.4	36	3.1	9559	432	2158
14	1	0.052	2686	142	138.1	126.77	1	1.01	0.4	39	3.1	9557	528	2159
15	1	0.056	2794	142	137.8	126.89	1	1.01	0.5	42	3.1	9556	636	2159
16	1	0.06	2914	142	137.5	127.01	1	1.01	0.5	45	3.1	9554	755	2159
17	1	0.064	3046	142	137.2	127.15	1	1.01	0.6	48	3.1	9552	888	2159
18	1	0.068	3192	142	136.9	127.29	1	1.01	0.6	51	3.1	9551	1033	2159
19	1	0.072	3351	142.01	136.6	127.44	1	1.01	0.6	54	3.1	9549	1192	2159
20	1	0.076	3524	142.01	136.3	127.6	1	1.01	0.7	57	3.1	9547	1365	2160
21	1	0.08	3712	142.01	136	127.77	1	1.01	0.7	60	3.1	9546	1553	2160
22	1	0.084	3915	142.01	135.7	127.95	1	1.01	0.8	63	3.1	9544	1755	2160
23	1	0.088	4133	142.01	135.4	128.13	1	1.01	0.9	66	3.1	9543	1973	2160
24	1	0.092	4366	142.01	135.1	128.32	1	1.01	0.9	69	3.1	9541	2207	2160
25	1	0.096	4617	142.01	134.8	128.52	1	1.02	1	72	3.1	9539	2457	2160
26	1	0.1	4883	142.01	134.5	128.72	1	1.02	1.1	75	3.1	9538	2723	2160
27	1	0.104	5167	142.01	134.2	128.93	1	1.02	1.1	78	3.1	9536	3007	2161
28	1	0.108	5469	142.01	133.9	129.15	1	1.02	1.2	81	3.1	9535	3309	2161
29	1	0.112	5789	142.01	133.6	129.36	1	1.02	1.3	84	3.1	9533	3629	2161
30	1	0.116	6128	142.01	133.3	129.59	1	1.02	1.4	87	3.1	9531	3968	2161
31	1	0.12	6486	142.01	133	129.82	1	1.02	1.5	90	3.1	9530	4326	2161
32	1	0.124	6864	142.01	132.7	130.05	1	1.03	1.6	93	3.1	9528	4704	2161
33	1	0.128	7263	142.01	132.4	130.28	1	1.03	1.7	96	3.1	9527	5102	2161
34	1	0.132	7683	142.01	132.1	130.52	1	1.03	1.8	99	3.1	9525	5522	2161
35	1	0.136	8125	142.01	131.8	130.77	1	1.03	1.9	102	3.1	9523	5964	2161
36	1	0.14	8590	142.01	131.5	131.02	1	1.04	2	105	3.1	9522	6429	2161
37	1	0.144	9078	142.01	131.2	131.27	1	1.04	2.1	108	3.1	9520	6917	2161
38	1	0.148	9591	142.01	130.9	131.52	1	1.04	2.3	111	3.1	9519	7430	2161
39	1	0.152	10129	142.01	130.6	131.78	1	1.05	2.4	114	3.1	9517	7969	2161
40	1	0.156	10694	142.01	130.3	132.05	1	1.05	2.6	117	3.1	9515	8534	2161
41	1	0.16	11287	142.01	130	132.31	1	1.06	2.7	120	3.1	9514	9126	2161
42	1	0.164	11908	142.01	129.7	132.58	1	1.06	2.9	123	3.1	9512	9748	2161
43	1	0.168	12560	142.01	129.4	132.86	1	1.07	3.1	126	3.1	9511	10400	2161
44	1	0.172	13244	142.01	129.1	133.13	1	1.07	3.2	129	3.1	9509	11083	2161
45	1	0.176	13961	142.01	128.8	133.42	1	1.08	3.4	132	3.1	9507	11801	2161
46	1	0.18	14714	142.01	128.5	133.7	1	1.08	3.6	135	3.1	9506	12554	2161
47	1	0.184	15505	142.01	128.2	133.99	1	1.09	3.9	138	3.1	9504	13345	2160
48	1	0.188	16335	142.01	127.9	134.29	1	1.1	4.1	141	3.1	9503	14176	2160
49	1	0.192	17209	142.01	127.6	134.59	1	1.11	4.3	144	3.1	9501	15049	2160
50	1	0.196	18128	142.01	127.3	134.9	1	1.12	4.6	147	3.1	9499	15969	2160
51	1	0.2	19097	142.01	127	135.21	1	1.13	4.9	150	3.1	9498	16938	2160
52	1	0.204	19252	142.01	127	135.26	1	1.13	5.1	150	3.1	9496	17093	2159
53	1	0.208	19283	142	127	135.27	1	1.13	5.4	150	3.1	9494	17124	2159
54	1	0.212	19288	142	127	135.27	1	1.13	5.7	150	3.1	9493	17130	2159
55	1	0.216	19289	142	127	135.27	1	1.13	6	150	3.1	9491	17131	2159
56	1	0.22	19288	142	127	135.27	1	1.13	6.2	150	3.1	9490	17130	2158
57	1	0.224	19287	142	127	135.27	1	1.13	6.5	150	3.1	9488	17130	2158
58	1	0.228	19286	142	127	135.27	1	1.13	6.8	150	3.1	9486	17129	2158

I	K	TTP(I)	Q(I)	H2	YB	D	SUB	VCOR	OUTV	BB	COFR	QI(I)	QBRECH	QSPIL
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
59	1	0.232	19285	142	127	135.27	1	1.13	7.1	150	3.1	9485	17128	2158
60	1	0.236	19284	142	127	135.27	1	1.13	7.4	150	3.1	9483	17127	2157
61	1	0.24	19283	142	127	135.27	1	1.13	7.6	150	3.1	9482	17127	2157
62	1	0.244	19282	142	127	135.27	1	1.13	7.9	150	3.1	9480	17126	2157
63	1	0.248	19281	142	127	135.27	1	1.13	8.2	150	3.1	9478	17125	2157
64	1	0.252	19280	142	127	135.27	1	1.13	8.5	150	3.1	9477	17125	2156
65	1	0.256	19280	142	127	135.27	1	1.13	8.7	150	3.1	9475	17124	2156
66	1	0.26	19279	142	127	135.27	1	1.13	9	150	3.1	9474	17123	2156
67	1	0.264	19278	142	127	135.27	1	1.13	9.3	150	3.1	9472	17122	2156
68	1	0.268	19277	142	127	135.27	1	1.13	9.6	150	3.1	9470	17122	2156
69	1	0.272	19276	142	127	135.27	1	1.13	9.9	150	3.1	9469	17121	2156
70	1	0.276	19276	142	127	135.27	1	1.13	10.1	150	3.1	9467	17120	2156
71	1	0.28	19275	142	127	135.27	1	1.13	10.4	150	3.1	9466	17120	2155
72	1	0.284	19274	142	127	135.27	1	1.13	10.7	150	3.1	9464	17119	2155
73	1	0.288	19273	142	127	135.27	1	1.13	11	150	3.1	9462	17118	2155
74	1	0.292	19272	142	127	135.27	1	1.13	11.2	150	3.1	9461	17118	2155
75	1	0.296	19272	142	127	135.27	1	1.13	11.5	150	3.1	9459	17117	2155
76	1	0.3	19271	142	127	135.27	1	1.13	11.8	150	3.1	9458	17116	2155
77	1	0.304	19270	141.99	127	135.27	1	1.13	12.1	150	3.1	9456	17116	2155
78	1	0.308	19269	141.99	127	135.27	1	1.13	12.4	150	3.1	9454	17115	2155
79	1	0.312	19268	141.99	127	135.27	1	1.13	12.6	150	3.1	9453	17114	2155
80	1	0.316	19268	141.99	127	135.27	1	1.13	12.9	150	3.1	9451	17114	2154
81	1	0.32	19267	141.99	127	135.27	1	1.13	13.2	150	3.1	9450	17113	2154
82	1	0.324	19266	141.99	127	135.27	1	1.13	13.5	150	3.1	9448	17112	2154
83	1	0.328	19265	141.99	127	135.27	1	1.13	13.7	150	3.1	9446	17112	2154
84	1	0.332	19264	141.99	127	135.27	1	1.13	14	150	3.1	9445	17111	2154
85	1	0.336	19264	141.99	127	135.27	1	1.13	14.3	150	3.1	9443	17110	2154
86	1	0.34	19263	141.99	127	135.27	1	1.13	14.6	150	3.1	9441	17110	2154
87	1	0.344	19262	141.99	127	135.27	1	1.13	14.8	150	3.1	9440	17109	2154
88	1	0.348	19261	141.99	127	135.27	1	1.13	15.1	150	3.1	9438	17108	2154
89	1	0.352	19260	141.99	127	135.27	1	1.13	15.4	150	3.1	9437	17107	2153
90	1	0.356	19260	141.99	127	135.27	1	1.13	15.7	150	3.1	9435	17107	2153
91	1	0.36	19259	141.99	127	135.26	1	1.13	16	150	3.1	9433	17106	2153
92	1	0.364	19258	141.99	127	135.26	1	1.13	16.2	150	3.1	9432	17105	2153
93	1	0.368	19257	141.99	127	135.26	1	1.13	16.5	150	3.1	9430	17105	2153
94	1	0.372	19256	141.99	127	135.26	1	1.13	16.8	150	3.1	9429	17104	2153
95	1	0.376	19256	141.99	127	135.26	1	1.13	17.1	150	3.1	9427	17103	2153
96	1	0.38	19255	141.99	127	135.26	1	1.13	17.3	150	3.1	9425	17103	2153
97	1	0.384	19254	141.99	127	135.26	1	1.13	17.6	150	3.1	9424	17102	2152
98	1	0.388	19253	141.99	127	135.26	1	1.13	17.9	150	3.1	9422	17101	2152
99	1	0.392	19252	141.99	127	135.26	1	1.13	18.2	150	3.1	9421	17101	2152
100	1	0.396	19252	141.99	127	135.26	1	1.13	18.5	150	3.1	9419	17100	2152
101	1	0.4	19251	141.99	127	135.26	1	1.13	18.7	150	3.1	9417	17099	2152
102	1	0.404	19250	141.98	127	135.26	1	1.13	19	150	3.1	9416	17099	2152
103	1	0.409	19249	141.98	127	135.26	1	1.13	19.4	150	3.1	9414	17098	2152
104	1	0.415	19248	141.98	127	135.26	1	1.13	19.7	150	3.1	9412	17097	2152
105	1	0.42	19247	141.98	127	135.26	1	1.13	20.1	150	3.1	9409	17096	2151
106	1	0.427	19245	141.98	127	135.26	1	1.13	20.6	150	3.1	9407	17095	2151
107	1	0.434	19244	141.98	127	135.26	1	1.13	21.1	150	3.1	9404	17093	2151
108	1	0.442	19242	141.98	127	135.26	1	1.13	21.6	150	3.1	9401	17092	2151
109	1	0.45	19241	141.98	127	135.26	1	1.13	22.2	150	3.1	9397	17091	2151
110	1	0.46	19239	141.98	127	135.26	1	1.13	22.9	150	3.1	9393	17089	2150
111	1	0.47	19237	141.98	127	135.26	1	1.13	23.6	150	3.1	9389	17087	2150
112	1	0.482	19234	141.98	127	135.26	1	1.13	24.4	150	3.1	9385	17085	2150
113	1	0.494	19232	141.98	127	135.26	1	1.13	25.2	150	3.1	9380	17083	2149
114	1	0.508	19229	141.97	127	135.26	1	1.13	26.2	150	3.1	9374	17081	2149
115	1	0.523	19226	141.97	127	135.25	1	1.13	27.3	150	3.1	9368	17078	2148
116	1	0.54	19223	141.97	127	135.25	1	1.13	28.4	150	3.1	9361	17075	2148

I	K	TTP(I)	Q(I)	H2	YB	D	SUB	VCOR	OUTV	BB	COFR	QI(I)	QBRECH	QSPIL
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
117	1	0.558	19219	141.97	127	135.25	1	1.13	29.7	150	3.1	9354	17072	2147
118	1	0.578	19215	141.97	127	135.25	1	1.13	31.1	150	3.1	9346	17069	2147
119	1	0.601	19210	141.96	127	135.25	1	1.13	32.6	150	3.1	9337	17065	2146
120	1	0.625	19205	141.96	127	135.25	1	1.13	34.3	150	3.1	9327	17060	2145
121	1	0.652	19200	141.96	127	135.25	1	1.13	36.2	150	3.1	9316	17056	2145
122	1	0.682	19194	141.96	127	135.24	1	1.13	38.2	150	3.1	9304	17051	2144
123	1	0.714	19187	141.95	127	135.24	1	1.13	40.5	150	3.1	9291	17045	2143
124	1	0.75	19180	141.95	127	135.24	1	1.13	42.9	150	3.1	9277	17039	2142
125	1	0.789	19172	141.94	127	135.24	1	1.13	45.7	150	3.1	9261	17032	2141
126	1	0.833	19163	141.94	127	135.23	1	1.13	48.7	150	3.1	9244	17024	2139
127	1	0.88	19153	141.93	127	135.23	1	1.13	51.9	150	3.1	9225	17016	2138
128	1	0.933	19142	141.93	127	135.23	1	1.13	55.6	150	3.1	9203	17007	2136
129	1	0.991	19131	141.92	127	135.22	1	1.13	59.5	150	3.1	9180	16996	2135
130	1	1.054	19117	141.92	127	135.22	1	1.13	63.9	150	3.1	9155	16985	2133
131	1	1.124	19103	141.91	127	135.22	1	1.13	68.7	150	3.1	9127	16973	2131
132	1	1.201	19087	141.9	127	135.21	1	1.13	74	150	3.1	9096	16959	2128
133	1	1.285	19069	141.89	127	135.21	1	1.13	79.8	150	3.1	9062	16944	2126
134	1	1.378	19050	141.88	127	135.2	1	1.13	86.2	150	3.1	9025	16928	2123
135	1	1.48	19029	141.87	127	135.19	1	1.13	93.2	150	3.1	8984	16909	2120
136	1	1.593	19005	141.86	127	135.18	1	1.13	100.9	150	3.1	8939	16889	2116
137	1	1.716	18979	141.84	127	135.18	1	1.13	109.3	150	3.1	8889	16866	2113
138	1	1.852	18950	141.83	127	135.17	1	1.13	118.6	150	3.1	8834	16842	2108
139	1	2.002	18917	141.81	127	135.16	1	1.13	128.8	150	3.1	8772	16814	2104
140	1	2.166	18881	141.79	127	135.15	1	1.13	140	150	3.1	8493	16783	2099
141	1	2.347	18841	141.77	127	135.13	1	1.13	152.3	150	3.1	8186	16748	2093
142	1	2.547	18794	141.75	127	135.12	1	1.13	165.8	150	3.1	7848	16709	2086
143	2	2.766	18741	141.72	127	135.1	1	1.13	180.6	150	3.1	7477	16664	2078
144	2	3.007	18681	141.69	127	135.08	1	1.13	196.8	150	3.1	7068	16612	2069
145	2	3.272	18612	141.65	127	135.06	1	1.13	214.6	150	3.1	6618	16553	2059
146	2	3.563	18533	141.61	127	135.03	1	1.13	234.1	150	3.1	6124	16486	2048
147	2	3.884	18442	141.57	127	135	1	1.13	255.4	150	3.1	5580	16408	2035
148	2	4.237	18338	141.51	127	134.97	1	1.13	278.8	150	3.1	5115	16319	2020
149	2	4.625	18218	141.45	127	134.93	1	1.13	304.3	150	3.1	4675	16216	2002
150	2	5.052	18080	141.38	127	134.88	1	1.13	332.2	150	3.1	4191	16099	1982
151	2	5.521	17922	141.3	127	134.83	1	1.13	362.7	150	3.1	3659	15964	1959
152	2	6.038	17739	141.2	127	134.77	1	1.13	395.8	150	3.1	3093	15807	1932
153	2	6.606	17527	141.09	127	134.7	1	1.13	431.9	150	3.1	2724	15627	1901
154	2	7.231	17284	140.96	127	134.62	1	1.14	471	150	3.1	2319	15419	1865
155	2	7.918	17001	140.81	127	134.52	1	1.14	513.5	150	3.1	1873	15179	1823
156	2	8.675	16674	140.64	127	134.41	1	1.14	559.3	150	3.1	1625	14900	1774
157	2	9.506	16292	140.43	127	134.28	1	1.14	608.7	150	3.1	1385	14575	1717
158	2	10.422	15841	140.19	127	134.12	1	1.14	661.6	150	3.1	1173	14192	1650
159	2	11.428	15302	139.9	127	133.92	1	1.14	718	150	3.1	1006	13732	1570
160	2	12.535	14634	139.54	127	133.67	1	1.14	777.7	150	3.1	855	13159	1476
161	3	13.753	13726	139.03	127	133.33	1	1.14	839.9	150	3.1	727	12381	1345
162	2	15.093	12577	138.37	127	132.86	1	1.15	903.3	150	3.1	597	11389	1188
163	2	16.567	11373	137.66	127	132.35	1	1.15	966.8	150	3.1	488	10349	1025
164	2	18.188	10137	136.91	127	131.79	1	1.15	1029.6	150	3.1	425	9279	858
165	2	19.971	8875	136.12	127	131.16	1	1.15	1090.6	150	3.1	373	8193	683
166	2	21.932	7646	135.3	127	130.5	1	1.15	1149	150	3.1	329	7107	540
167	2	24.09	6452	134.46	127	129.79	1	1.14	1203.7	150	3.1	294	6052	400
168	1	26.463	5308	133.63	127	129.03	1	1.14	1254	150	3.1	264	5047	261
169	2	29.074	4254	132.81	127	128.23	1	1.13	1298.9	150	3.1	239	4114	141
170	2	31.946	3355	132.03	127	127.45	1	1.11	1338.2	150	3.1	218	3276	80
171	2	35.105	2576	131.29	127	126.65	1	1.1	1372	150	3.1	202	2554	23
172	2	38.58	1947	130.61	127	125.89	1	1.08	1400.3	150	3.1	188	1948	0
173	2	42.402	1456	130.01	127	125.18	1	1.07	1423.7	150	3.1	177	1457	0
174	2	46.607	1072	129.47	127	124.51	1	1.06	1442.8	150	3.1	169	1072	0

N.B. I-Time Step from start of analysis, K-Iterations necessary to solve flow equations, TIP(I)-Elapsed time from start of analysis (hr), Q (I)-Total outflow from dam (cms), H2-Elevation of water surface at dam (m), YB-Elevation of bottom of breach (m), D-Estimated Depth of flow immediately downstream (m), SUB-Submergence coefficient, VCOR-Velocity correction, OUTVOL-Total volume discharged from time of breach (M.Cum), BB-Breach width (m), COFR-Rectangular breach discharge coefficient, QI(I)-Inflow to reservoir (cms), QBRECH-Breach outflow(cms), QSPIL-Spillway outflow(cms).