M. Tech Dissertation Report

Submitted in partial fulfillment for the award of the Degree of

Master of Technology in WRD

on

DAM SAFETY ANALYSIS OF MATATILA DAM FROM HYDROLOGIC AND HYDRAULIC CONSIDERATIONS

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December, 2018

CERTIFICATE

This is to certify that the dissertation report entitled "**DAM SAFETY ANALYSIS OF MATATILA DAM FROM HYDROLOGIC AND HYDRAULIC CONSIDERATIONS**" is a bonafide record prepared by **Upananda Rath**, Enrollment No. **16548026** under our supervision and guidance, in partial fulfillment of the requirements for the award of Degree of Master of Technology in Water Resources Development & Management (WRD&M) from Indian Institute of Technology Roorkee.

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ACKNOWLEDGEMENT

I sincerely acknowledge my supervisor Dr. S.K. Mishra, Professor & Head, Department of Water Resources Development & Management, IIT Roorkee and co-supervisor Shri Nitya Nanda Rai, Director, Hydrology (S), Central Water Commission, New Delhi for providing me all help & support for preparation of the M.Tech Dissertation Report. I am also thankful to all faculty and staff members of Department of Water Resources Development & Management who have directly or indirectly supported me in completion of the M.Tech Dissertation Report.

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ABBREVIATIONS USED

BAF	Barrier Adjustment Factor
BIS	Bureau of Indian Standards
Cumec	Cubic Meter per Second
Cusec	Cubic Feet per Second
CWC	Central Water Commission
DEM	Digital Elevation Model
Ft	Feet
GIS	Geographical Information System
HMS	Hydrologic Modelling System
hPa	Hexa Pascal
IMD	India Meteorological Department
Km	Killometer
LAF	Location Adjustment Factor
m	Meter
MAF	Moisture Adjustment Factor
MCM	Million Cubic Meter
mm	Millimeter
Mm3	Million Cubic Meter
MMF	Moisture Maximization Factor
PMF	Probable Maximum Flood
PMP	Probable Maximum Precipitation
SPS	Standard Project Flood
WRIS	Water Resource Information System
WMO	World Meteorological Organization

ABSTRACT

Matatila Dam which is located on Betwa river of Ganga basin has faced the problem of flood passing through the spillway safely in the recent past. The hydrology of the Matatila dam was studied during the 1950's. It has been observed that the design flood calculations were first made based on empirical formula and later on frequency analysis. The decision on spillway capacity of a dam, including the decision of its surcharge storage, freeboard etc., constitutes an important hydrologic and engineering decision affecting safety of the dams. BIS published the guidelines for fixing of spillway capacity, BIS 11223-1985, which defines the Design Flood, by classifying the safety of dams as small, intermediate and large. As per the guidelines the Design flood for Matatila dams falls in the category of PMF, which necessitates the review of hydrology of the dam.

The hydrology of a particular basin or project undergoes tremendous changes due to certain factors such as climate change, urbanization, deforestation, soil erosion, heavy spell of short duration rainfall etc. Nearly after 20 years of Matatila dam, Rajghat dam was planned and constructed which is 50 km upstream of the Matatila dam. Thus, it may be necessary to assess the revised design flood and to analyse various scenarios for regulation of the reservoir for the safety of dam. Considering the state-of-the-art technology or methods of analysis and the data availability for rainfall and discharge in recent years, hydrologic modelling in large scale is possible. Modern methods of flood estimation like meteorological-physical methods and availability of mathematical modelling software like ARC-GIS, HEC-HMS may be used for quite accurate estimation of the design flood.

In this study the design floods for both dams have been assessed using the latest techniques available viz. quasi distributed modelling using hydro-meteorological approach. The flood estimation reports and PMP atlas for Ganga basin published by CWC, which provide the Synthetic Unit Hydrographs of the subzone, are based on the record of the past storms and isohyets occurred over the catchment, Moisture Adjustment Factors etc. These have been used in this study. ARC-GIS have been used for catchment delineation using the Digital Elevation Model. HEC-HMS model has been used for convolution of lumped response at the outlet of

each sub-basin and the reservoir routing and channel routing the lumped responses through the reservoirs and a network of channels/rivers.

The results of the model have shown that, compared to the original design flood, the revised design flood at Matatila dam has been exceeded by 46 percent. In this context, the upstream Rajghat reservoir, which is of higher capacity, may be regulated to improve the safety of downstream Matatila dam and it has been studied by generating various scenarios. Five scenarios have been generated by utilising the available freeboard and reducing the spillway capacity. A final design flood has been arrived at 29068 cumec in this study which exceeds the original flood by 24 percent only. It has been recommended that along with the specified rule curve for reservoir operation at Rajghat dam, further suitable structural and/or non-structural measures be adopted to enhance the safety of Matatila Dam.

CHAPTER 1: INTRODUCTION

Matatila dam is one of the national importance dams of India, which was completed in the year 1964. It is located on Betwa river of Ganga basin (Lower Yamuna Sub-basin) near Lalitpur city in Uttar Pradesh State of India. River Betwa which rises from Bhopal in the Vindhyan Plateau at an elevation of 1550ft, passes through granite rocks at a sleep gradient, the river flows on outskirts of distt. Jhansi forming boundary line of U.P. and M.P. and thereafter it mostly flows in U.P., meeting the river Yamuna in Hamirpur Distt in U.P. It is entirely a rain fed river with very high discharge of water during rainy season and extremely low discharge during summers. The construction of dam was started in year 1952 for irrigation purpose. Matatila is a composite masonry and earth dam of height 45.72m and total length as 6300m. The gross storage capacity of the reservoir is 1130 MCM.

The design flood is 23390 cumec and spillway capacity of 15857 cumec. Matatila Dam has faced the problem of flood passing through the spillway safely in the recent past. The hydrology of the dam was studied during the 1950's, and therefore, necessitates its review. The hydrology of a particular basin or project undergoes tremendous changes due to certain factors such as climate change, urbanization, deforestation, soil erosion, heavy spell of short duration rainfall etc. Considering the latest techniques and the data availability for rainfall and discharge in recent years, the hydrologic modelling in large scale is possible. Modern methods of flood estimation like meteorological-physical methods and availability of mathematical modelling software like ARC-GIS, HEC-HMS etc may be used for accurate design flood estimation.

Nearly after 20 years of Matatila dam, Rajghat dam was planned and constructed 50 km upstream of the Matatila dam. Rajghat dam is also a major project having gross storage capacity of 2172 MCM. The dam was designed for PMF as per BIS criteria. The Matatila dam being in the downstream of Rajghat was designed for SPF (i.e. 1000 year flood), but as per the current provision of BIS, it qualifies for the design flood as PMF. In this context, it may be necessary to assess the revised design flood and to analyse various scenarios for reservoir regulation for its safety.

1.1 Research Gap and Rationale

The drainage area of Betwa River at Matatila is 8000 square miles (20,720 $km^{2)}$ and it increases to 8240 square miles at the existing Dhukwan Dam which is 10 miles downstream.

The original estimate of flood is based on the following:

(a) The maximum discharge by Inglis formulae is given by :-

$$Q = \frac{7000 \times A}{\sqrt{A+4}}$$

$$\frac{7000 \times 8000}{\sqrt{8000+4}} = 628000 \text{ Cusecs}$$

Where A is the catchment are in square miles

(b) By Dickens Formulae :-

$Q = C \times A^{\frac{3}{4}}$, where C=775 (for Betwa basin)

$$Q = 775 \times 8000^{\frac{3}{4}} = 660000\ cusecs$$

(c) Maximum Flood discharge recorded at Dhukwan site is

$Q = 575000\ cusecs$, which occurred in the year 1947.

Calculating backwards, by Inglis formulae, discharge for 240 square miles,

$$\frac{7000 \times 240}{\sqrt{240+4}} = 108000 \text{ Cusecs}$$

Since the catchment at Matatila is 240 square miles less than that at Dhukwan the flood at Matatila would be $Q = 575000 - 108000 = 467000\ cusecs$.

Taking the importance of the dam the maximum anticipated flood discharge was taken as 600000 cusecs. Here, it is noable that the absorption is 40000 cusecs and discharge capacity of the spillway is 560000 cusecs with 23 feet depth water on spillway.

However, the design flood adopted for the Matatila dam was 23386 cumec (= 8.26 lakh cusec). This had been computed in the year 1966 from the frequency studies and the return period adopted was 1000 years.

The maximum estimated flood discharge over the spillway is 560000 cusec. 23 flood gates of 60ft length and 23ft height were proposed on the spillway crest. The length of the spillway including piers is 1600 ft. The crest level is R.L. 989 giving a depth of 23 ft over the crest for maximum flood. The spillway section is the standard Creager type high coefficient weir. The spillway piers are spaced 70ft centres with 60ft clear opening.

Francis equation for necessary correction due to end contraction between piers with circular ends in this case is as follows:

$Ln = Lt - 0.045 \times n \times (Hc - Hv)$, where Ln = effective length, Lt = Total length of crest, n= no of end contraction, Hc= depth of water above crest, and Hv = velocity head

$$\mathrm{Ln} = 1380 - 0.045 \times 46 \times 23 = 1332 ft$$

Maximum discharge per ft $q = 560000000/1332 = 420\ \mathrm{cusec/ft}$

$$\mathrm{q} = \mathrm{C} \times \mathrm{H}^{\left(\frac{3}{2}\right)}$$

$\mathrm{C} = 420 \div 23^{\left(\frac{3}{2}\right)} = 3.81$, Creagers profile gives a discharge coefficient up to 4 which will be confirmed by model studies. A bucket with 45ft radius is provided at the downstream toe of the spillway to deflect the flow. Hydraulic jump pool is proposed to dissipate the energy in falling water over the spillway.

From the above hydrology estimation for design flood, it was observed that the flood calculations were first made based on empirical formula and later on frequency analysis. Thus, there is a scope for re-estimating the design flood using the latest methods of detailed hydro-meteorological studies. This physical/hydro-meteorological method is used to estimate Probable Maximum Flood (PMF) for a large meteorologically homogeneous region. The procedure involves compilation and use of observed storm rainfalls for various major events over the catchment or region. It also includes adjustments for moisture availability and topographic effects and also requires a large amount of long-term data obtained by raingauge stations in the study area. This is a time-consuming and expensive process. However, the method can lead to high accuracy and easy application of results.

1.2 Objectives

The objective of this study is as under:

Objective-1: To provide a comprehensive review of dam safety in India with hydrological safety perspective

Objective-2: To evaluate the inflow design flood (design flood) of Matatila Dam and Rajghat dam using the latest techniques i.e. quasi distributed modelling (unit hydrograph approach) using hydro-meteorological method.

Objective-3: To prepare the rule curve for the Rajghat Reservoir operation to improve the safety aspects of Matatila dam and to recommend additional measures for dam safety.

CHAPTER 2: REVIEW OF LITERARURE

2.1 BIS 11223-1985: Design flood for storage dams

The Design Flood, also known as Inflow Design Flood (IDF) is the flood that is selected for design or safety evaluation of the structure. The value of the design flood should increase with increasing consequences of the failure of the structure. Design flood may be defined as the flood adopted for design purposes. It may be Probable Maximum Flood (PMF) or the Standard Project Flood (SPF) or some return period flood depending on the standard security that should be provided against possible failure of the structure.

Table-2.1: Criteria for classification of Dams for Design Flood

Classification	Gross Storage (Mm3)	Hydraulic Head (M)	Design Flood
Small	0.5-10	7.5-12	100 Year Flood
Intermediate	10-60	12-30	SPF
Large	More than 60	More Than 30	PMF

As per the above criteria shown in Table-2.1, Matatila Dam having a hydraulic head more than 30m qualifies for design flood for Probable Maximum Flood (PMF).

Definition of PMF as per BIS 11223-1985*: It is the flood that may be expected from the most severe combination of critical meteorological and hydrological condition that are reasonably possible in the region and is computed by using the Probable Maximum Storm which is an estimate of physical upper limit to maximum precipitation in the basin. This is obtained from the transposition studies of the storms that have occurred over the region and maximizing them for the most critical atmospheric conditions.*

2.2 CWC Manual for Estimation of Design Flood

The following approaches may be adopted for estimation of design flood

(i) Regional Flood Formulae Approach - Only used for preliminary estimates and where no other data is available

(ii) Statistical Approach or Flood Frequency – The limitation it yields only the peak, not shape of hydrograph, difficulty in getting homogeneous data due to developments like dams etc.

(iii) Hydro-meteorological Approach - It is a physical method, causative factors for flood are analyzed, very convenient and sufficiently accurate

In hydro-meteorological approach, the design flood estimation mainly involves estimation of a design storm hyetograph and derivation of catchment response function. The catchment response function can be either a quasi-system model (a unit hydrograph) or a distributed quasi model. In a distributed quasi-model, a catchment is divided into a number of sub-regions and the unit hydrograph of each sub-regions is applied with channel/reservoir routing will define the catchment response.

The main advantage of this method is that it gives a complete flood hydrograph and this allows a realistic determination of its moderating effect while passing through a reservoir or river reach. In this study, the Hydro-meteorological Approach would be followed for estimation the design flood.

2.3 PMP Atlases for Ganga Basin (Vol-I & Vol-II)

Probable Maximum Precipitation (PMP) is an estimate of the physical upper limit to the storm over the catchment, and is obtained by studying all the storms that have occurred over the region and maximizing them for most critical atmospheric condition. This atlas provides the historical storm events, its synoptic movement, isohyets, Depth-Area-Duration analysis etc. Using the above details, the SPS depths may be obtained for the study area. The SPS values are multiplied by the MAF (Moisture Adjustment Factor) of storm for the respective area and duration to get the PMP estimate. **PMP = SPS* MAF**

2.4 HEC-HMS Technical Manual

The methods of hydrological modelling involve estimation of lumped response at the outlet of each sub-basin and then reservoir routing and channel routing the lumped responses through the reservoirs and a network of channels/rivers. The various methods available for routing i.e. Muskingum Method, Kinematic Wave method, Muskingum-Cunge Method, Lag method, Modified Pulse method and Dynamic routing etc. will be looked into according to the applicability in this study.

2.5 Dam Safety

2.5.1 History of Dam Safety

The growth of civilization is inextricably woven around the availability of water world over. Dams are human devices for exploitation of water for irrigation, flood control, hydropower developments and other uses etc., and thus, play a pivotal role in development activities. Dams, however, are not unmixed blessings. They do pose a major hazard in the unlikely event of a failure.

There have been about 200 reservoir failures in 20^{th} century in the world so far, taking more than 8000 lives. International Commission on Large Dams (ICOLD) indicates the number of major dam failures as given in Table-2.2

Table-2.2: Dam failures decade wise (ICOLD)

Year	Approx. No of Failure
Period to 1900	38
1900 to 1909	15
1910 to 1919	25
1920 to 1929	33
1930 to 1939	15
1940 to 1949	11
1950 to 1959	30
1960 to 1965	10
Date unknown	25
Total	202

The dams built in India by and large have performed well, but there have been a few failures. The reported failures of dams in India, as per Central Water Commission (CWC) are given in the Table-2.3

Table-2.3: Dam failures in India (CWC)

SL. No	State	Name of Project	Type	Height (M)	Year of Completion	Year of Failure	Cause of failure
Up to 1950							
1	Madhya Pradesh	Tigra	Masonry	24.03	1914-17	1917	Overtopping followed by slide.
2	Maharashtra	Ashti	Earth	17.70	1883	1933	Slope failure.
3	Madhya Pradesh	Pagara	Composite	27.03	1911-27	1943	Overtopping followed by breach.
1951-1960							
4	Madhya	Palakmati	Earth	14.60	1942	1953	Sliding failure.

	Pradesh						
5	Rajasthan	Dakhya	Earth	NA	1953	1953	Breaching.
6	Uttar Pradesh	Ahrura	Earth	22.80	1953	1953	Breaching.
7	Rajasthan	Girinanda	Earth	12.20	1954	1955	Overtopping followed by breaching.
8	Rajasthan	Anwar	Earth	12.50	1956	1957	Breaching.
9	Rajasthan	Gudah	Earth	28.30	1956	1957	Breached due to bad workmanship.
10	Rajasthan	Sukri	Earth	NA	NA	1958	Breached by leakage through foundation.
11	Madhya Pradesh	Nawagaon	Earth	16.00	1958	1959	Overtopping leading to breach.
12	Rajasthan	Dervakheda	Earth	NA	NA	1959	Breaching.
13	Gujarat	Kaila	Earth	23.08	1955	1959	Embankment collapsed due to weak foundation.
				1961-1970			
14	Maharashtra	Panshet	Earth	53.80	1961	1961	Piping failure leading to breach.
15	Maharashtra	Khadakwasla	Masonry	60.00	1875	1961	Overtopping.
16	Rajasthan	Galwania	Earth	NA	1960	1961	Breaching.
17	Rajasthan	Nawagaza	Earth	NA	1955	1961	Breaching.
18	Madhya Pradesh	Sampna	Earth	21.30	1956	1964	Slope failure on account of inappropriate materials.
19	Madhya Pradesh	Kedarnala	Earth	20.00	1964	1964	Breaching.
20	UttaraKhand	Nanaksagar	Earth	16.00	1962	1967	Breached due to foundation piping.
				1971-1980			
21	Gujarat	Dantiwada	Earth	60.96	1965	1973	Breach on account of floods.
22	Tamil Nadu	Kodaganar	Earth	12.75	1977	1977	Breached on account of floods.
23	Gujarat	Machhu-II	Composite	20.00	1972	1979	Overtopping due to floods.
				1981-1990			
24	Gujarat	Mitti	Earth	16.02	1982	1988	Overtopping leading to breach.
	1991- 2000						
25	Madhya Pradesh	Chandora	Earth	27.30	1986	1991	Breach.
26	Andhra Pradesh	Kadam	Composite	22.50	1958	1995	Over topping leading to breach.
27	Rajasthan	Bhimlot	Masonry	17.00	1958	-	Breached due to inadequate spillway capacity.
				2001-2010			
28	Gujarat	Pratappur	Earth	10.67	1891	2001	Breached on account of floods.
29	Madhya Pradesh	Jamunia	Earth	15.40	1921	2002	Piping leading to breaching.
30	Orissa	Gurilijore	Earth	12.19	1954-55	2004	The abutment structure along with wing and return walls got undermined with foundation scouring.
31	Maharashtra	Nandgavan	Earth	22.51	1998	2005	Excessive rain causing water flow over the waste weir to a

							depth beyond the design flood lift.
32	Madhya Pradesh	Piplai	Earth	16.73	1998	2005	Breach
33	Rajasthan	Jaswant Sagar	Earth	43.38	1889	2007	Piping leading to breaching.
34	Andhra Pradesh	Palemvagu dam	Earth	13.00	U/C	2008	Flash flood resulting in overtopping of the earth dam
35	Madhya Pradesh	Chandiya	Earth	22.50	1926	2008	Breach.
36	Rajasthan	Gararda	Earth	31.76	2010	2010	Examination for cause of failure by state authorities in progress.

The Machhu –II dam disaster took a toll of more than 2000 lives.

The dam safety analysis mainly consists of the flowing aspects:

1. Structural Safety
2. Hydrologic and Hydraulic Safety
3. Seismic safety

In this study, hydrologic and hydraulic safety has been dealt.

2.5.2 Hydrologic Dam Safety

The hydrologic safety covers the study of (i) Inflow Design Flood (PMF, SPF or Return Period Flood), (ii) Outflow Hydrograph from the Spillway and its moderation (iii) Reservoir Routing and Maximum water levels in the reservoir (iv) Gate Operation at different reservoir levels (v) and Free board. It also covers the dam break flood that may occur for different failure modes and preparation of Inundation Maps for Emergency Action Plan (EAP). The main part of the hydrologic safety is thus the design flood review.

As per the findings of the ICLOD, one-third of the failures of the dams are direct result of flood exceeding the capacity of the dam spillway. As per the data of large embankment dams, the most common causes of failure are overtopping accounting for 32% of failures followed by internal erosion accounting to 27% failures. In India, internal erosion (breaching) accounts 44% of dam failures followed by overtopping that accounts for 25% of failures.

After United states and China, India now ranks third in the world in terms of number of Large Dams. There are currently 5264 completed large dams in India with 437 under construction (as per National Register of Large dams, NRLD, CWC). More than three-fourth of the dams, carrying substantial storages are at least two decades old and, for these dams, the original flood peak discharges/volumes were estimated mostly designed with empirical formulas with applied discretions by experienced designers. For such dams there is an urgent need for original design flood estimates to be either supported or reviewed based on scientific data collected in-situ, and on the basis of computational procedures that have been improved since then. Even in many cases of recently constructed dams, the original estimates of the design flood were based on the scarce observed flood records or on record of extreme rainfall events at that time and both of these datasets get strengthened with more values are available, and hence it would be prudent to review those studies.

2.5.3 Revised Design Flood Estimates of Existing Dams in India

Dams for which design flood estimates were revised under the Dam safety program in India have been summarized in the Table-2.4

Table-2.4: Revised Design Flood of Existing Dams in India

Sl. No	Name of Dam	State	Original Design Flood (m3/s)	Revised Design Flood (m3/s)	Design Flood Category	% Increase in Design Flood	Age (Yr)
1	Sanjay Sarovar	MP	16652	15428	PMF	-7.35	28
2	Ari Project	MP	240	1241	SPF	417.08	60
3	Tawa	MP	30800	29619	PMF	-3.83	38
4	Jirbhar	MP	373.5	1074	SPF	187.55	32
5	Thanwar	MP	3993.2	7137	PMF	78.73	32
6	ChandaPatha	MP	424	1226	SPF	189.15	94
7	Barna	MP	13557	13235	PMF	-2.38	37
8	Kankarkhera	MP	144	625	100 Year Flood	334.03	32
9	GopiKrishanSagar	MP	3605	4209	PMF	16.75	17
10	Kharadi	MP	100	1029.8	SPF	929.80	52
11	Nahlesara	MP	271.68	1543.6	SPF	468.17	44
12	Chandra Keshar	MP	870.84	1644	SPF	88.78	36
13	Sagarnadi	MP	186	758	SPF	307.53	46
14	Kolar	MP	8605	8605	PMF	0.00	23
15	Sarathi	MP	289	1651	SPF	471.28	89
16	Sampana	MP	492	788	SPF	60.16	56

17	MooramNallah	MP	185	852	SPF	360.54	62
18	Chawarpani	MP	202.53	453.8	SPF	124.07	52
19	Bundala	MP	838	1512	SPF	80.43	26
20	Marhi	MP	296.7	952	SPF	220.86	33
21	Kunwar ChainSagar	MP	1310	1733	SPF	32.29	11
22	Makroda	MP	598.41	2554	SPF	326.80	32
23	Sanjay Sagar	MP	1565	2039	SPF	30.29	13
24	Sher	MP	120	724	SPF	503.33	33
25	Sundrel	MP	60.81	66.05	100 Year Flood	8.62	25
26	Gangulpara	MP	191.73	607	100 Year Flood	216.59	53
27	GuradiaSurdas	MP	110	215.57	SPF	95.97	16
28	Manjhikhedi	MP	88.52	123.23	100 Year Flood	39.21	22
29	Lasudiakanger	MP	68.74	179.98	100 Year Flood	161.83	31
30	Dhablamata	MP	72.44	133.08	100 Year Flood	83.71	33
31	Deogaon	MP	182.2	476.16	100 Year Flood	161.34	22
32	Birpur	MP	423.99	737	100 Year Flood	73.82	5
33	Birnai	MP	81.13	268	100 Year Flood	230.33	23
34	Umrar	MP	479.78	1449	SPF	202.01	23
35	Kamera	MP	279.99	825	SPF	194.65	103
36	Banksal	Odisha	420	868	SPF	106.67	31
37	Kalo	Odisha	965	1997	SPF	106.94	31
38	Nesa	Odisha	351	364	SPF	3.70	31
39	Sanamachhakandana	Odisha	226	374	SPF	65.49	34
40	Padampurnalla	Odisha	303	443	SPF	46.20	34
41	Budhabudhiani	Odisha	401	903	SPF	125.19	46
42	Balaskumpa	Odisha	132.48	302	SPF	127.96	36
43	Ashokanalla	Odisha	69.34	221	SPF	218.72	26
44	Daha	Odisha	1380	1828	SPF	32.46	25
45	Derjang	Odisha	3952	3590	SPF	-9.16	35
46	Dhanel	Odisha	733	1230	SPF	67.80	48
47	Pillasalki	Odisha	793	1054	SPF	32.91	25
48	Salia	Odisha	1019.42	2464	SPF	141.71	43
49	Sarafgarh	Odisha	695	819	SPF	17.84	28
50	Satiguda(Malkangiri)	Odisha	1060	1883	SPF	77.64	33
51	Talsara	Odisha	820	913	SPF	11.34	29

52	Hirakud	Odisha	42450	69632	PMF	64.03	56
53	Nambiyar	TN	1053.9	1053.9	100 Year Flood	0.00	9
54	Mordhana	TN	10541.1	9820	SPF	-6.84	12
55	Poigaiyar	TN	164.45	208	SPF	26.48	13
56	Adavainainarkoil	TN	356	826	PMF	132.02	11
57	Vadakkupachayar	TN	715.77	1338	SPF	86.93	10
58	Kudumudiyar	TN	573.95	947.92	SPF	65.16	10
59	Rajathopekanar	TN	81.89	172	100 Year Flood	110.04	15
60	Gomukhinadhi	TN	2834	2834	SPF	0.00	48
61	Siddamalli	TN	1920	1162	100 Year Flood	-39.48	25
62	Vidur	TN	6167	7228	SPF	17.20	54
63	Kodaganar	TN	8500	11147	SPF	31.14	20
64	Manimuthar	TN	4522	4969	PMF	9.89	55
65	Manimukhanadhi	TN	926.06	4484	SPF	384.20	43
66	Thirumurthy	TN	447.65	1672	SPF	273.51	55
67	Amaravathy	TN	4062	6544	PMF	61.10	63
68	KullarSandhai	TN	635	673	100 Year Flood	5.98	29
69	NoyyalAthupalayam	TN	92.72	169	100 Year Flood	82.27	21
70	Shoolagirichinnar	TN	547.1	689	100 Year Flood	25.94	28
71	PilavukkalPeriyar	TN	286.57	474	100 Year Flood	65.40	37
72	PilavukkalKovilar	TN	223	333	100 Year Flood	49.33	37
73	Anaikuttam	TN	1708	2096	100 Year Flood	22.72	27
74	Golwarpatti	TN	3207.5	3207.5	100 Year Flood	0.00	20
75	Gundar	TN	264.68	243	SPF	-8.19	30
76	Mukurthy	TN	425	567	PMF	33.41	75
77	Servalar	TN	1982	2454	PMF	23.81	27
78	Porthimund	TN	241	297	PMF	23.24	47
79	Glenmorgan	TN	46	108	SPF	134.78	83
80	Avalanche	TN	705	1765	PMF	150.35	52
81	Kadamparai	TN	517.8	632	PMF	22.05	29
82	Emerald	TN	705	1765	PMF	150.35	52
83	Western Catchment Weir No 1	TN	106	243	SPF	129.25	47
84	Malampuza	Kerala	849.506	4007	PMF	371.69	57
85	Peechi	Kerala	368.119	1799	PMF	388.70	54

86	Neyyar	Kerala	809.4	2643	PMF	226.54	60
87	Chulliar	Kerala	223.7	624	SPF	178.95	42
88	Meenakara	Kerala	472.6	1209	SPF	155.82	48
89	Pothudy	Kerala	682.44	875	SPF	28.22	45
90	Kallada	Kerala	2830	5380	PMF	90.11	26
91	Mangalam	Kerala	245	1533	SPF	525.71	46
92	Kanjirapuzha	Kerala	512.5	1427	PMF	178.44	29
93	Kakki-Anathodu Dam	Kerala	1784	2283	PMF	27.97	46
94	Pamba	Kerala	911.8	1614	PMF	77.01	45

Analysing the above data of 94 dams, it is observed that the revised design flood values have generally exceeded substantially compared to their original values. In 40 out of the 94 dams the revised design flood values exceeded the original adopted values by more than 100%. The change in percentage of the design flood values in some dams is very high e.g. Kharadi dam: 929%, Sher dam: 503% and Mangalam dam: 525%. It is also observed that change in percentage is negative (-ve) for 7 dams and the lowest being -39%.

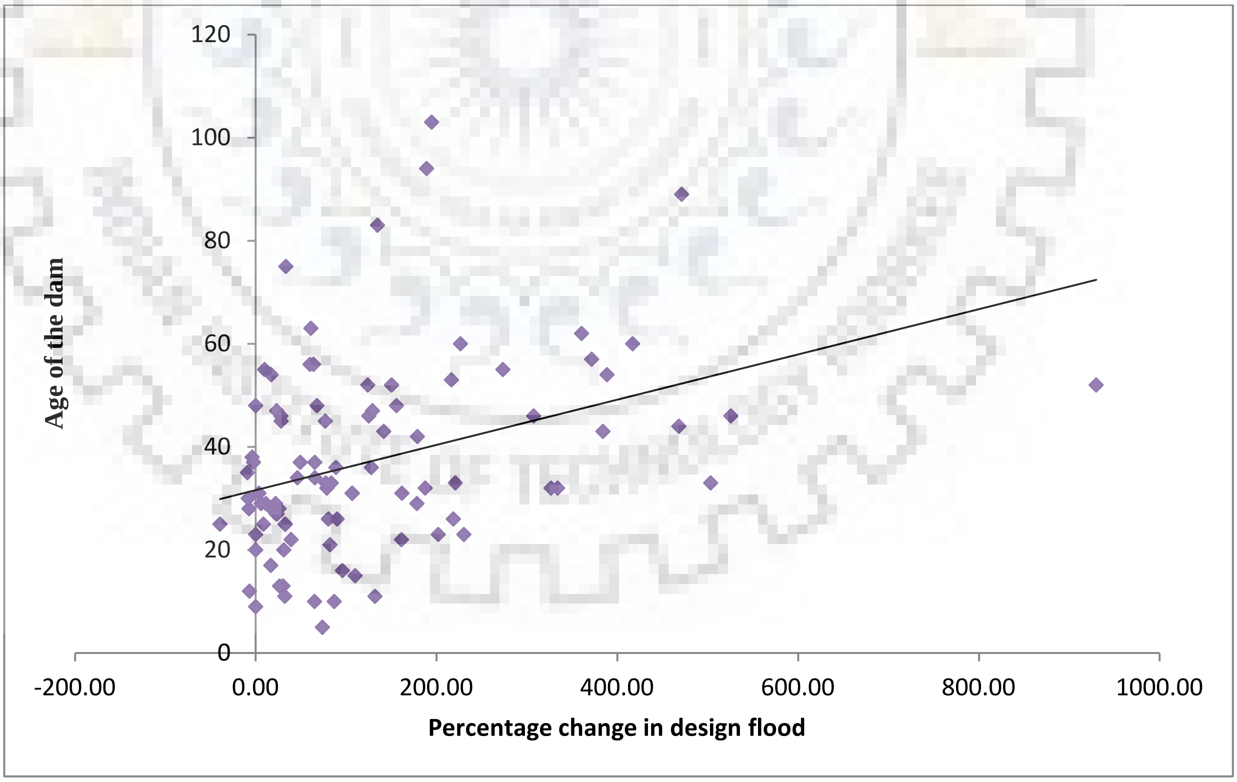

Fig-2.1: Percentage change in design flood vs dam age

To understand if there exists a trend in revised design flood and dam age, a scattered plot was obtained as shown in Fig-2.1. Although the correlation between the two is not strong, the trend line shows an upward trend, which means that as the dams are ageing there is, in general, a need to revise the design flood to ensure hydrologic safety of the dam.

3.1 Study Area

River Betwa which originates near Bhopal in the Vindhya Range flows north east through the State of Madhya Pradesh and Uttar Pradesh in India. Nearly half of its course runs over the Malwa Plateau. The total length of the river from origin to its confluence with Yamuna is 590 Km. Matatila dam and Rajghat Dam are the two important projects constructed in this river. The study area comprises of Betwa River up to Matatila Dam site. The index map of the study area derived from India-WRIS is shown below.

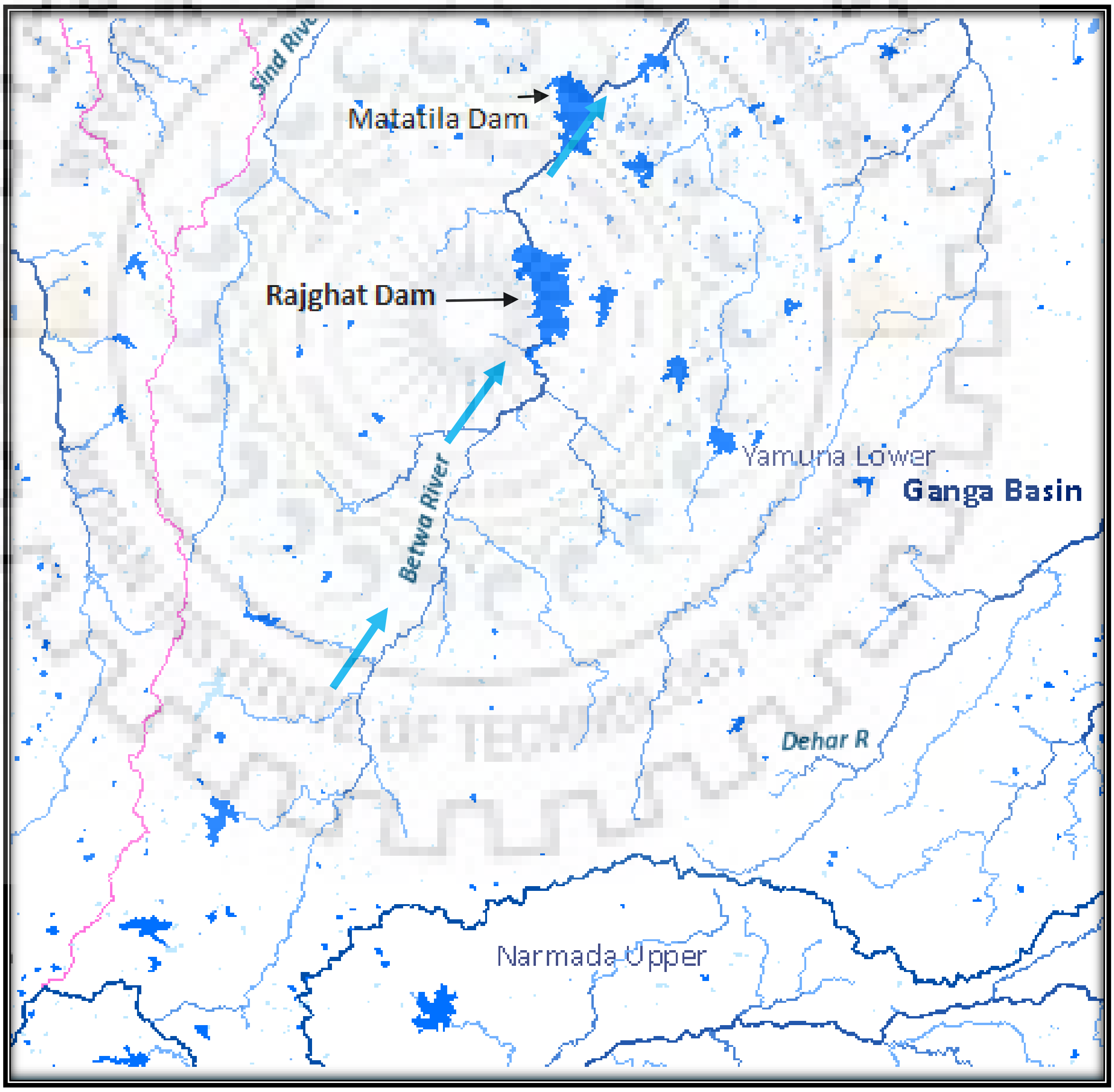

Fig-3.1: Index Map of Study Area

The salient features of both Matatila and Rajghat dam have been shown in Table-3.1.

Table-3.1: Salient features of Matatila and Rajghat Dam

Particulars	Matatila Dam	Rajghat Dam
Year of start	1952	1977
Year of Completion	1964	
Height of Dam (m)	45.72	43.5
Gross Storage Capacity (MCM)	1130	2172
Inflow Design Flood (PMF)	Not available	44555
Inflow Design Flood (SPF)	23390	39014
Spillway Capacity	15857	33893
F.R.L	1012 ft = 308.46m	371.00
HFL/ MWL	1015 ft = 309.37m	373.07
Freeboard		377-373 = 4m
Spillway crest level	989 ft = 301.45 m	357
Spillway Type & Gates	Ogee, 23 gates (18.29x7.11)	18 gates, 15x14.75

3.2 Methodology for design flood and dam safety analysis

Step-1: Generation of Unit Hydrographs

The study area up to the outlet of Matatila dam will be delineated through ARC-GIS using the Digital Elevation Model (DEM). Following steps in Arc-GIS viz. Fill, Flow direction, Flow Accumulation, Pour points, Flow Augmentation, Watershed, Streamline generation, catchment area delineation etc. would be performed to obtain the desired catchment area at the outlet of Matatila dam.

HEC-HMS extension in ARC-GIS would be used to further divide the catchment into a number of sub-basins. While generating the sub basins, limitation for unit hydrograph theory (i.e. area of sub-basins not exceeding 5000 sq. km) would be followed. The synthetic unit hydrographs for each sub-basin would be generated by using physiographic parameters of respective sub-basins, derived from ARC-GIS.

Step-2: Rain Storm Analysis (Historical record of Rainfall)

For estimation of PMF, the first requirement is to find out the Probable Maximum Precipitation (PMP). For this estimation of PMP, past storm details available in the PMP atlases for Ganga Basin (Volume-I and II) will be used. The rainstorms affecting the Betwa River Catchment for 1-day, 2-day and 3-day storm will be studied from the PMP atlases.

Step-3: Storm Transposition and Standard Project Storm (SPS).

Some candidate storms with maximum 1-day, 2-day, 3-day rainfall will be selected for computation. The isohyets of the selected storms will be plotted in Arc-GIS. The storm centre of a candidate storm will be transposed to the catchment area (preferably to c.g. of catchment) so as to obtain the maximum depth of rainfall. Several such trails will be required so as to obtain the maximum storm depth i.e. SPS for 1-day, 2-day, 3-day.

Step-4: Storm Maximization & PMP for 1-day, 2-day, 3-day

The Moisture Maximization Factor (MMF) for a rainstorm in a place is the ratio of precipitable water corresponding to the maximum persisting dew point temperature on record at the original location of the rainstorm in the same fortnight of the month in which the rainstorm occurred to the precipitable water corresponding to the maximum persisting dew point temperature of rainstorm.

The transposition of the rainstorm necessitates application of two adjustments for location and barrier. The combined effect of MMF, Location Adjustment Factor (LAF) and Barrier Adjustment Factor (BAF) is expressed by a single term known as Moisture Adjustment Factor (MAF) and is expressed by the following relation.

MAF = MMF*BAF*LAF

Moisture Adjustment Factor (MAF) will be estimated based on the graphs given in the PMP Atlas of Ganga Basin Vol-1, by utilizing the altitude of the locations above MSL.

Then Probable Maximum Precipitation is **(PMP) = SPS* MAF**

Step-5: Design Rainfall Hyetograph (Time distribution and Critical Sequencing)

After obtaining the PMP, time distribution analysis (TD) for rainfall intensity and critical sequencing of rainfall to maximize the design flood will be performed. The most important factors affecting the peak flood discharge is the variability of rainfall in time. Therefore, to compute flood runoff, it becomes necessary to know what proportion of 24-hr rain usually fell in the heaviest 2-hr or 3-hr and so forth. The short duration storm rainfall values can be determined by applying the Time Distributions (TD) from hourly rainfall data of SRRGs of catchments under study. TD Coefficients available in the PMP atlas for Betwa Sub-basin will be used in this analysis. Critical sequencing of rainfall with two 12 hour bell shaped spells shall be followed. The flowchart for estimation of design flood hydrograph is shown in Fig. 3.2:

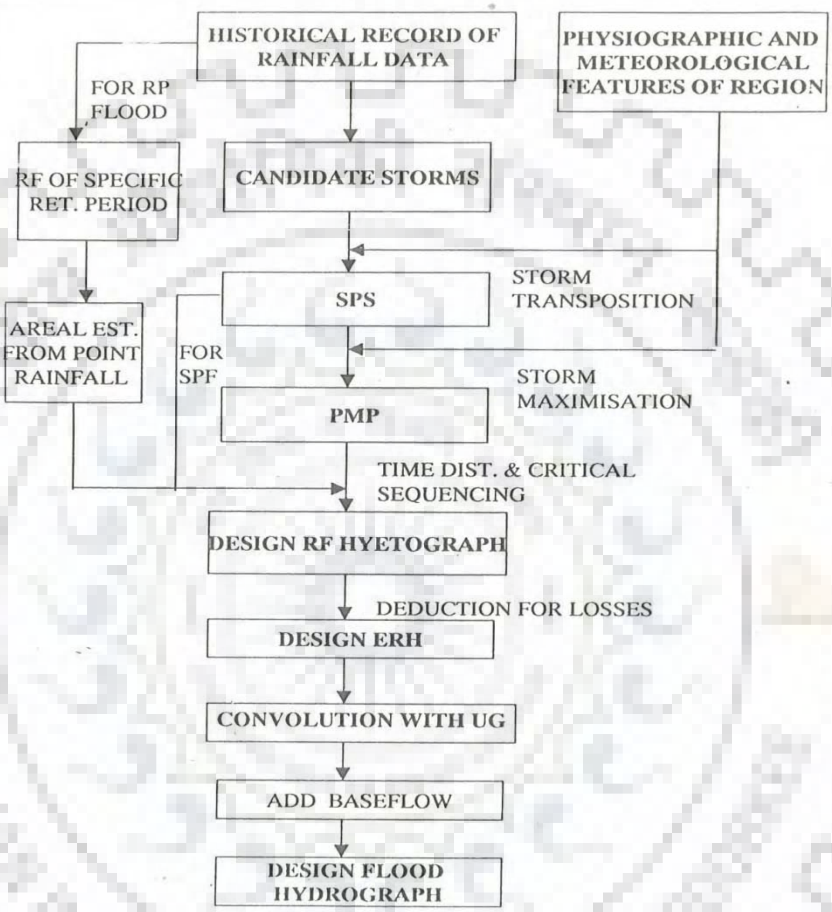

Fig-3.2: Flowchart for Estimation of PMF (Probable Maximum Flood)

Step-6 Hydrologic Modelling using HEC-HMS (Convolution and Routing)

Design flood synthesis using mathematical modelling using HEC-HMS will be performed for convoluting the design storm hyetograph and unit hydrograph. The design flood will be routed through the reaches using a suitable channel routing method. As the Rajghat dam exists just upstream of Matatila Dam, the design flood will be routed through the Rajghat dam reservoir to account for the flood moderation provided by the storage of the dam.

Description of HEC-HMS model:

The HEC HMS model for design flood computation consists of Basin model, Meteorologic model and control specification. The Basin model consists of network of sub basins, junctions, reservoirs, river reaches and outlet. The inputs required for the sub basin are unit hydrograph and base flow. The sub basins are connected to the junctions and reaches are connected from one junction to another junction to carry forward the flood finally to outlet. The inputs required for the reaches are routing method and its parameters. The input required for reservoir is elevation-storage-discharge curve.

The Meteorologic model takes the input of hyetographs sub basin wise, which are linked to respective sub basin in the model. Control specification requires input for time interval and run time for the model to be specified. Time series data manager is used for input of precipitation gages and paired data manager is used for input of Unit hydrographs, storage-discharge and elevation-storage tables. After providing all the necessary inputs we have to run the model for final calculations and the results at each element of the model can be viewed.

Step-7 Comparison of the original design floods with the revised design floods of and analysis of the dam safety scenario.

The estimated design Flood for Matatila Dam would be compared with the original design flood. If it is found unsafe, the scenario of reservoir regulation at Rajghat reservoir would be considered.

(i) To reduce the impingement level for flood at Rajghat reservoir and rout the PMF hydrograph at Rajghat Dam.

(ii) To restrict the spillway capacity of Rajghat to a lower capacity so as to obtain a less peak outflow hydrograph from Rajghat Spillway.

(iii) To encroach some part of the freeboard available at Rajghat dam temporarily to increase the surcharge storage.

CHAPTER 4: DESIGN FLOOD ESTIMATION BY HYDRO METEROLOGICAL METHOD

4.1 Catchment Area Delineation in Arc-GIS

Matatila dam has a height of 45m and Gross Storage Capacity of 1130 MCM. As per the BIS criteria (shown in Table-2.1), the dam is classified as a large dam, and therefore, it qualifies for Probable Maximum Flood (PMF) for its design flood. The total catchment area at Matatila dam (Location :78^0-22'-23''E and 25^0-05'-48''N) site is around 20000 sq. km. For sub-basin delineation from main catchment in Arc-GIS, location of Rajghat Dam (Location: 78^0-13'-05''E and 24^0-56'-00''N) and Gauge-Discharge site of CWC- Basoda (Location: 78^0-56'-04''E and 23^0-54'-06''N) have been used for analysis. The drainage area of Matatila dam located on Betwa River have been divided into number of sub-basins of size less than 5000 sq.km, so that unit hydrograph method is applicable reliably.

The catchment area delineation has been performed using the Arc-GIS and SRTM DEM of 90m resolution is shown at Fig-4.1. The total catchment area is 20342 km^2 at Matatila Dam.

4.1.1 Description of Sub Basins

Sub Basin-1: Catchment of Betwa River between Rajghat Dam and Matatila Dam
Sub Basin-2: Catchment of Narayani River (joining from east side to Betwa River) and Betwa River up to Rajghat dam
Sub Basin-3: Catchment of Betwa from west side of the river joining Sub Basin-1 & 2 in downstream and Subbasin-4 in upstream
Sub Basin-4: Catchment of Betwa River from Basoda G&D site up to Confluence of Betwa & Bina River.
Sub Basin-5: Upstream Catchment of Bina River draining to Sub Basin-4
Sub Basin-6: Catchment between Confluence of Halali River and Betwa River upto Basoda G&D Site
Sub Basin-7: Catchment of Betwa River from origin up to confluence of river Halali and Betwa

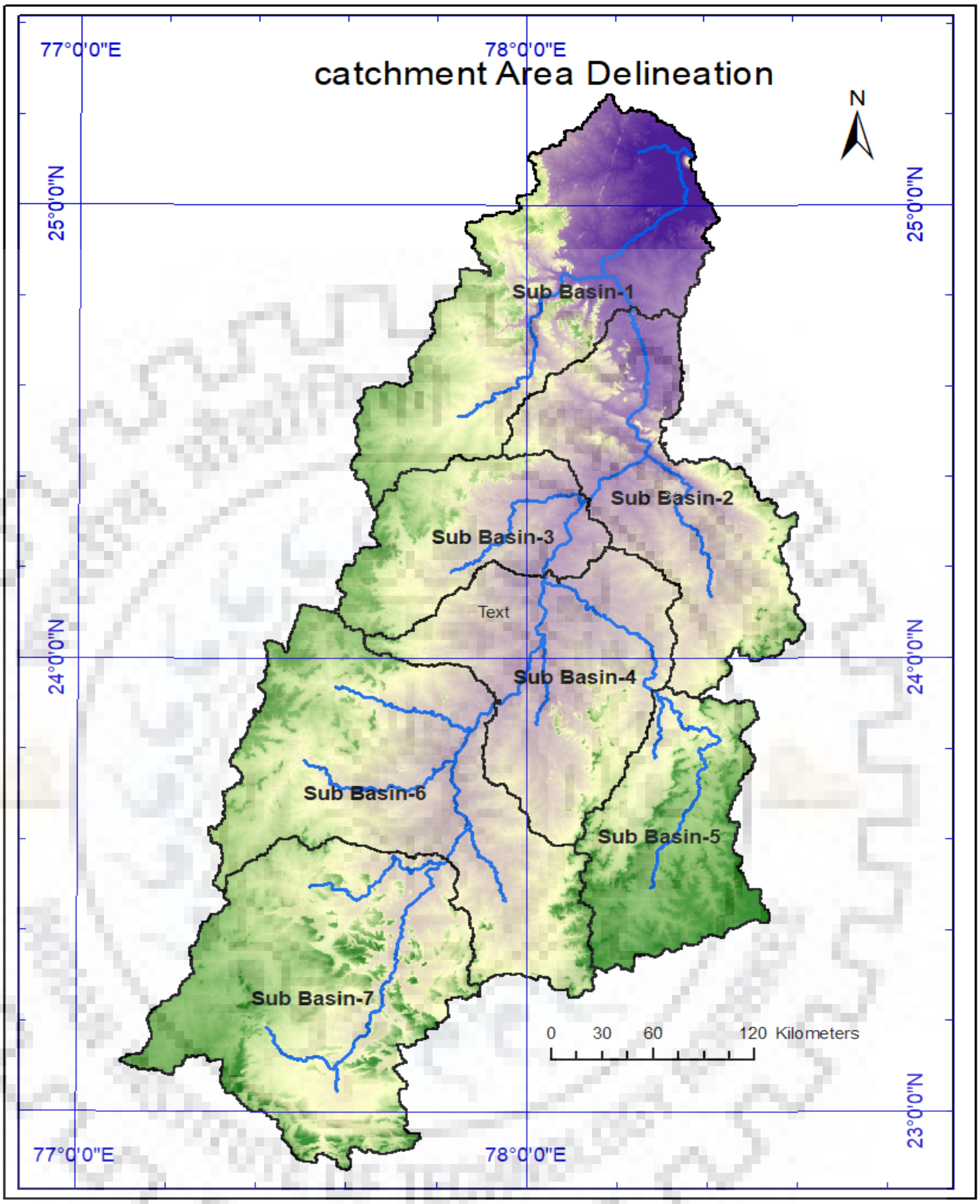

Fig-4.1: Catchment area delineation at Matatila Dam using Arc-GIS

4.2 Estimation of Physiographic parameters

Physiographic parameters of all the seven sub-basins of Matatila Catchment have been estimated from Arc-GIS. These are area in km^2 (A), longest flow path in km (L), flow path from centroid of sub-basin in km (Lc) and equivalent slope (S) etc.

4.2.1 Equivalent Slope Calculation (S)

The Equivalent slope (S) for each sub-basin is calculated along its longest flow path (L). The longest flow path is divided into equal segments of 1km to few kms depending upon length along longest flow path. The reduced level at the end of each segment is obtained from ARC-GIS. The equivalent slope is calculated using the following formulae:

$S = \sum L_i*(D_{i-1} + D_i)/L^2$

A sample calculation of equivalent slope for Sub basin-1 has been shown in Table-4.1

Table-4.1: Equivalent slope of Sub Basin-1

S.No.	Reduced Distance (Km)	Reduced Level (m)	Length of each segment L_i (km)	Height above Datum D_i (m)	$D_{i-1} + D_i$	$L_i*(D_{i-1} + D_i)$
1	0.0	273	0.0	0	0	0
2	4.0	299	4	26	26	104.00
3	14.0	299.5	10.0	26.5	52.5	525.00
4	24.0	304	10	31	57.5	575.00
5	34.0	310	10	37	68	680.00
6	44.0	319	10	46	83	830.00
7	54.0	333	10	60	106	1060.00
8	64.0	353	10	80	140	1400.00
9	74.0	371	10	98	178	1780.00
10	84.0	388	10	115	213	2130.00
11	94.0	395	10	122	237	2370.00
12	104.0	427	10	154	276	2760.00
13	114.0	433	10	160	314	3140.00
14	124.0	445	10	172	332	3320.00
15	134.0	460.5	10	187.5	359.5	3595.00
16	144.0	475	10	202	389.5	3895.00
17	154.0	501	10	228	430	4300.00
18	164.0	539	10	266	494	4940.00
					∑Li*(Di-1 + Di)	37404.00

Equivalent Slope $S = \sum L_i*(D_{i-1} + D_i)/L^2 = 37404/ (164^{2)} = 1.39$

In similar manner the equivalent slopes of all the seven sub basins have been computed. Physiographic properties of each sub basin viz. Area (A), Longest Flow path (L), Flow path from centorid of basin (Lc) have been derived using Arc-GIS. Table-4.2a shows the parameters of all sub basins in the Matatila Catchment.

Table-4.2a: Physiographic properties of each sub basin derived using Arc-GIS

No of Sub-basin	Name of Sub-basin	A Area (Km2)	L (longest Flow Path in km)	Lc (Flowpath from centorid of basin in km)	S (equivalent slope)
1	Subbasin-1	3543	164.026	69.657	1.39
2	Subbasin-2	2735	135.113	69.845	0.84
3	Subbasin-3	1674	94.202	41.291	1.03
4	Subbasin-4	2639	85.697	32.15	0.62
5	Subbasin-5	1969	108.435	60.401	1.21
6	Subbasin-6	3882	117.609	38.834	0.70
7	Subbasin-7	3900	147.113	54.448	0.63
	Total	**20342**			

4.3 Derivation of Synthetic Unit Hydrograph

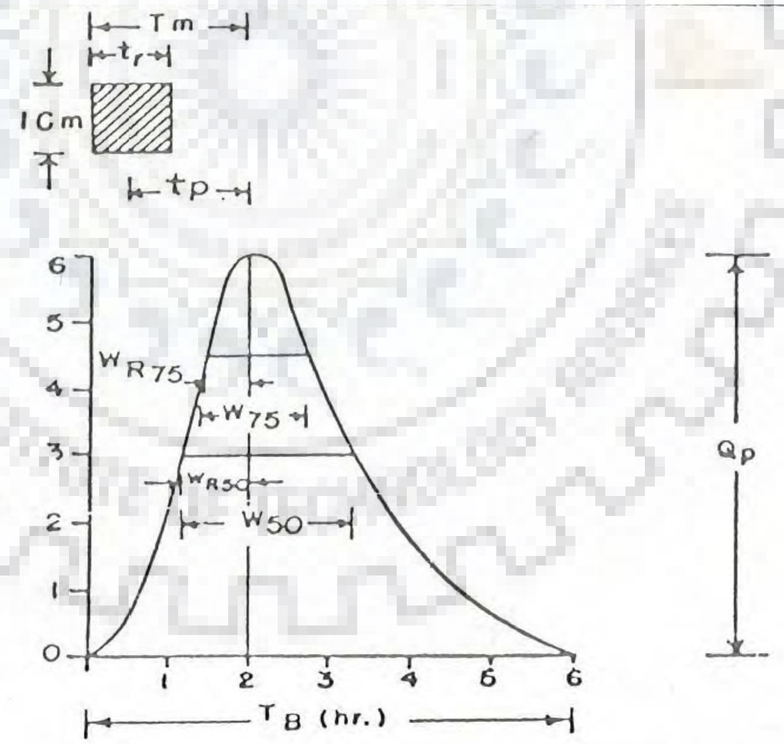

Fig-4.2: Synthetic unit Hydrograph

Unit hydrograph of each sub basin has been derived using the Synthetic Unit Hydrograph (SUH) method by using the catchment properties. CWC has published Flood Estimation Reports (FER) of the country for meteorologically similar regions known as subzones. For this study the FER of Betwa Subzone (1c) was referred.

The parameters of the SUH, as shown in the Fig-4.2 above are defined as:

q_p	Peak Discharge per Area (Cumecs/sq.Km)
t_p	Time from the center of effective rainfall to the peak of the Unit Hydrograph (hour)
W_{50}	Width of the Unit Hydrograph measured at 50% of peak discharge ordinate (hour)
W_{75}	Width of the Unit Hydrograph measured at 75% of peak discharge ordinate(hour)
WR_{50}	Width of the Rising Limb of Unit Hydrograph measured at 50% of peak discharge ordinate(hour)
WR_{75}	Width of the Rising Limb of Unit Hydrograph measured at 75% of peak discharge ordinate(hour)
T_B	Base width of Unit Hydrograph(hour)
T_m	Time from the start of rise to the peak of Unit Hydrograph(hour)
Q_p	Peak Discharge of Unit Hydrograph (Cumecs) = q_pxA
A	Catchment area in Sq.Km

From the FER report of Betwa sub-zone, the Formula used for calculation of the Unit Hydrograph parameters are shown in Table-4.3. Physical parameters as calculated in Table- 5.2 Area (A), Longest Flow Path (L) and Equivalent Slope (S) have been used in the formulae. Calculations are shown in Table-4.2b below.

Table-4.2b: Unit Hydrograph parameters calculation for all Sub-basins

Parameters	**Formulae**	**Sub Basin**						
		SB-1	SB-2	SB-3	SB-4	SB-5	SB-6	SB-7
$\mathbf{q_p}$	$\mathbf{1.331(L/S)^{-0.492}}$	0.127	0.109	0.144	0.117	0.146	0.107	0.091
$\mathbf{t_p}$	$\mathbf{2.195(q_p)^{-0.944}}$	15.50	17.50	13.50	16.50	13.50	18.50	21.50
$\mathbf{W_{50}}$	$\mathbf{2.04(q_p)^{-0.864}}$	16.9	19.8	14.9	18.4	14.7	20.3	23.9
$\mathbf{W_{75}}$	$\mathbf{1.25(q_p)^{-1.026}}$	7.4	8.5	6.7	8.0	6.6	8.6	9.9
$\mathbf{WR_{50}}$	$\mathbf{0.739(q_p)^{-0.968}}$	5.4	6.3	4.8	5.9	4.8	6.5	7.5
$\mathbf{WR_{75}}$	$\mathbf{0.500(q_p)^{-0.813}}$	2.7	3.0	2.4	2.9	2.4	3.1	3.5
$\mathbf{T_B}$	$\mathbf{3.917(t_p)^{0.990}}$	60	68	52	64	52	70	80
$\mathbf{T_m}$	$\mathbf{t_p+t_r/2}$	16	18	14	17	14	19	22
$\mathbf{Q_p}$	$\mathbf{q_p*A}$	451	299	242	310	287	414	354

4.3.1 Plotting and Smoothening of Unit Hydrograph

The unit hydrograph is first plotted in excel based on the W_{50}, W_{75}, WR_{50}, WR_{75}, T_B , T_m and Q_p parameters. The discharge ordinate at each hour is estimated and plotted in excel to resemble true shape of the Unit Hydrograph (smoothing of unit hydrograph). Mandatory check is made so as to obtain 1-unit depth of effective rainfall.

4.3.2 Synthetic Unit Hydrograph of All Sub basins

As explained above the plots of SUH for Sub basin 1 to 7 are shown in Figs. 5.3 to 5.9 respectively. The final value of ordinates of the SUH of all sub basins is shown in Table 5.3.

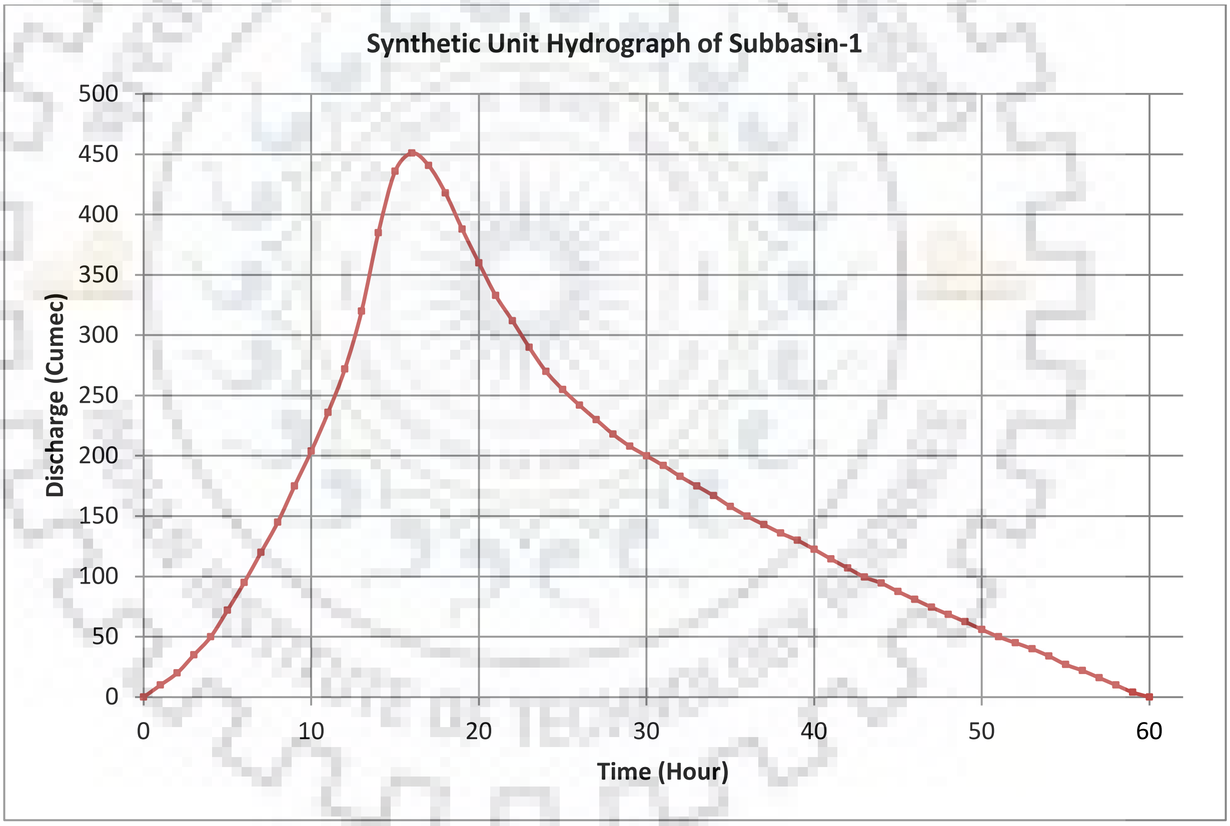

Fig-4.3: Unit Hydrograph of 1-hour duration for SB-1

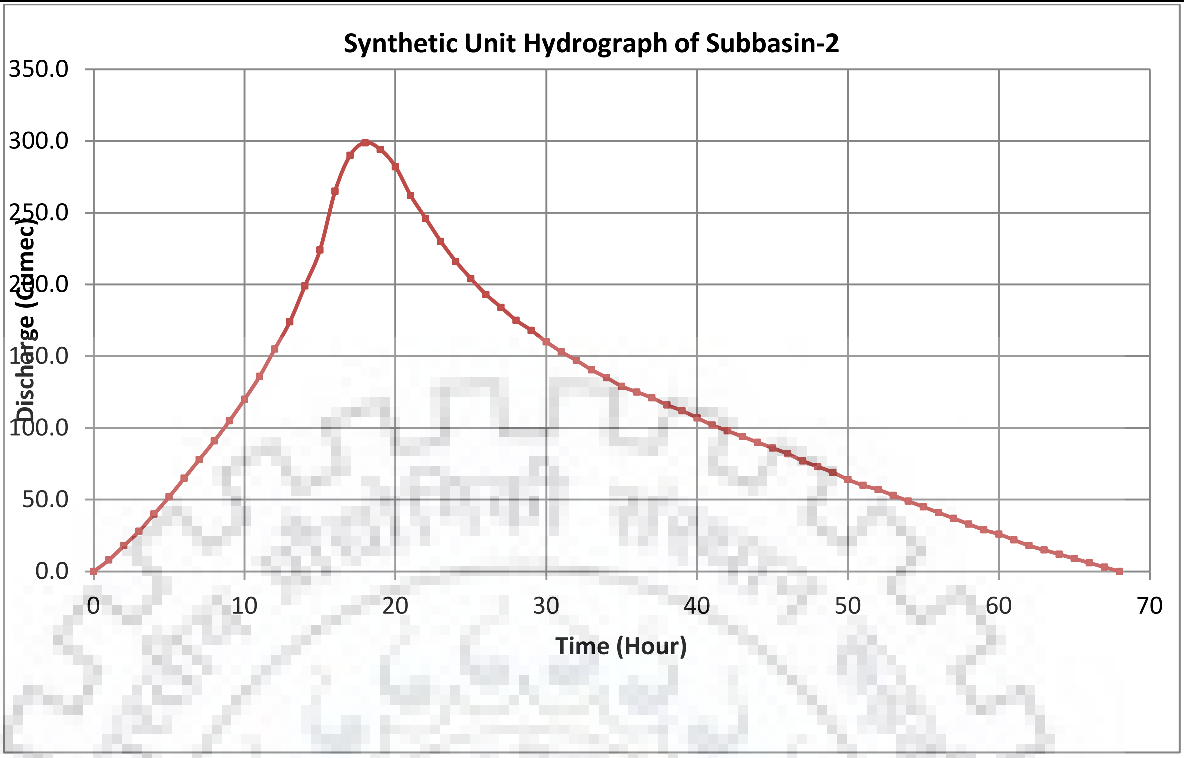

Fig-4.4: Unit Hydrograph of 1-hour duration for SB-2

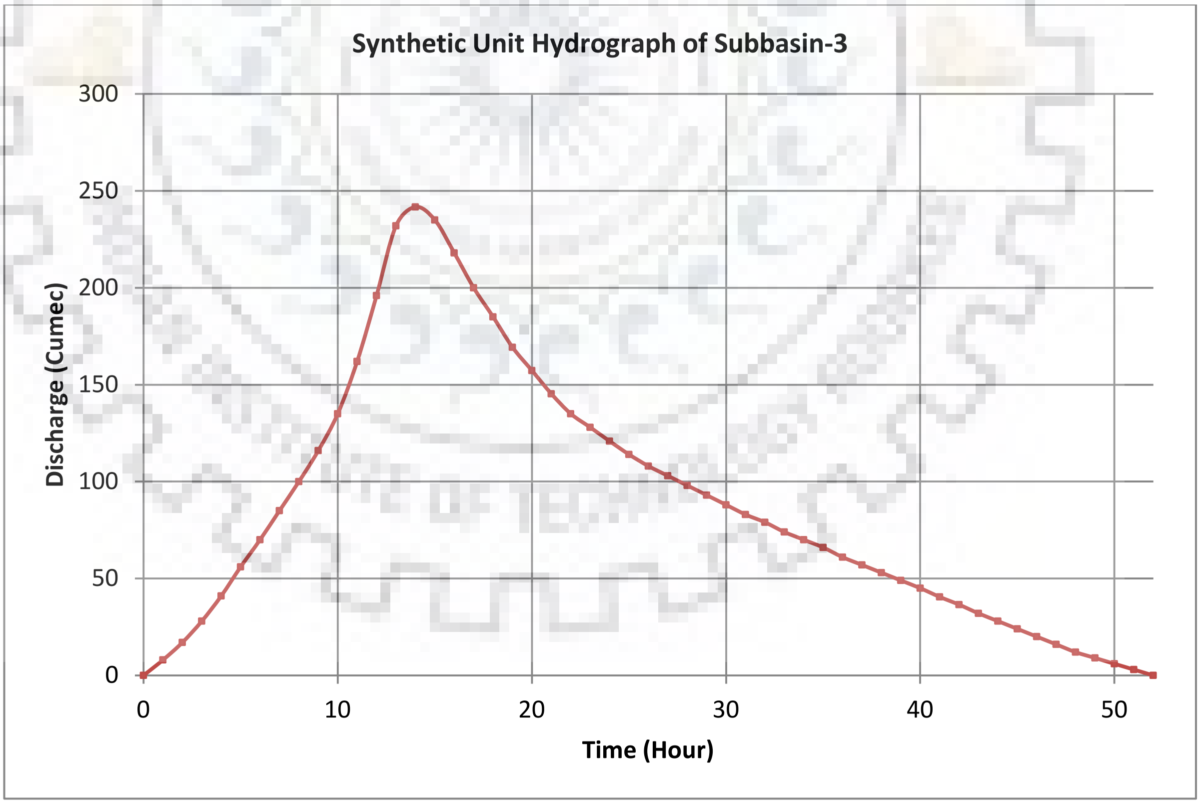

Fig-4.5: Unit Hydrograph of 1-hour duration for SB-3

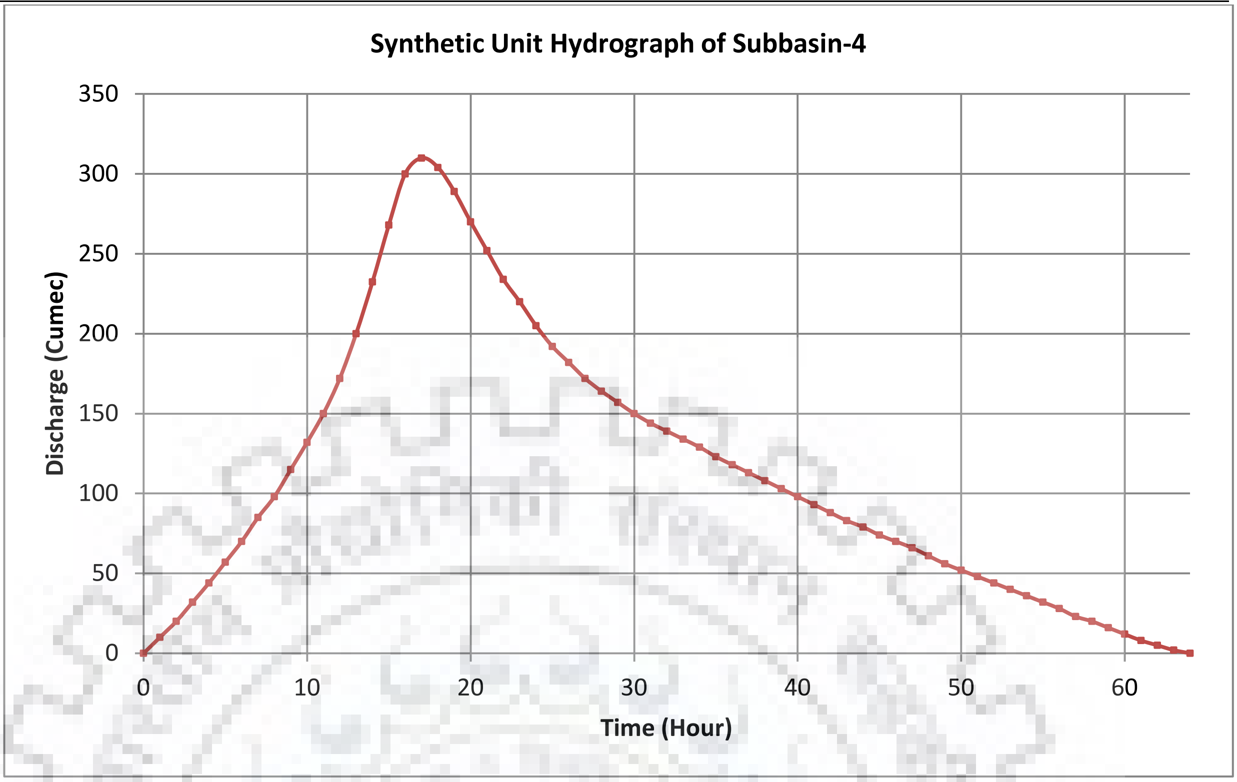

Fig-4.6: Unit Hydrograph of 1-hour duration for SB-4

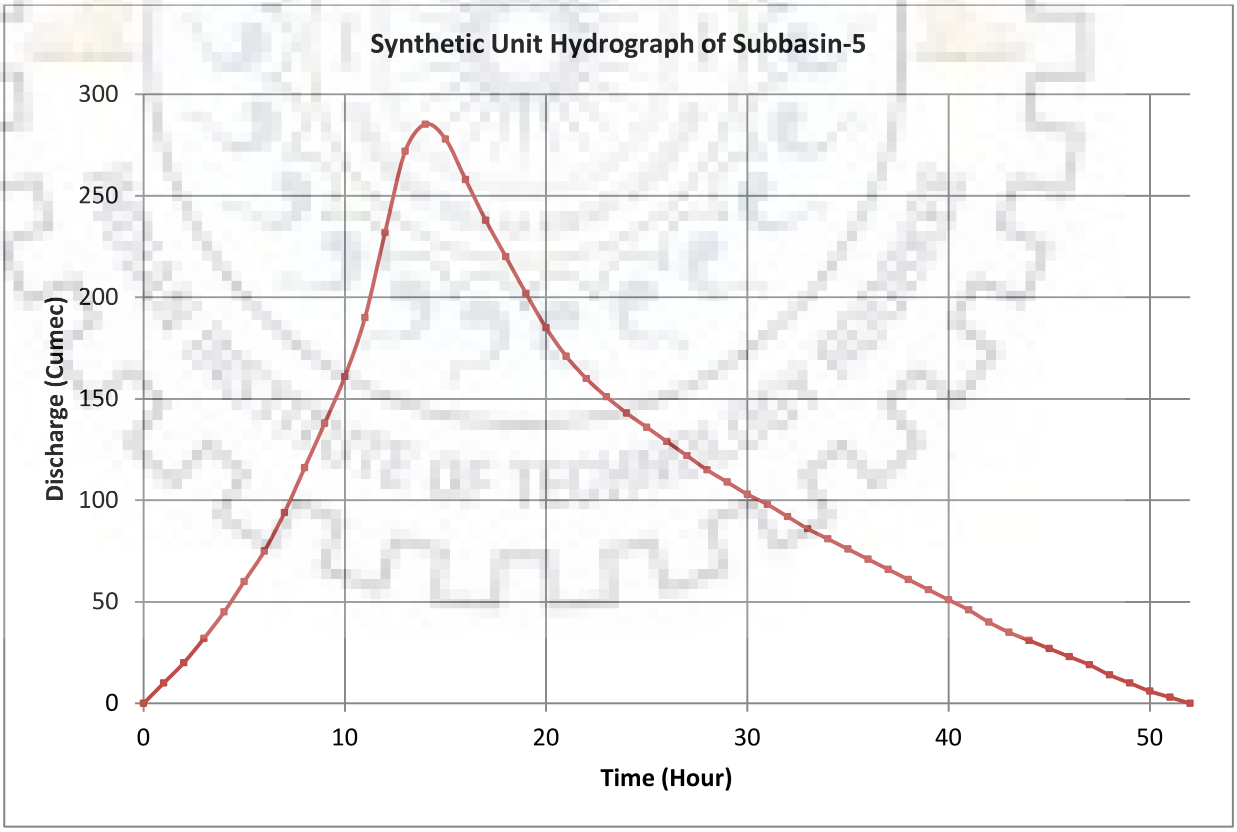

Fig-4.7: Unit Hydrograph of 1-hour duration for SB-5

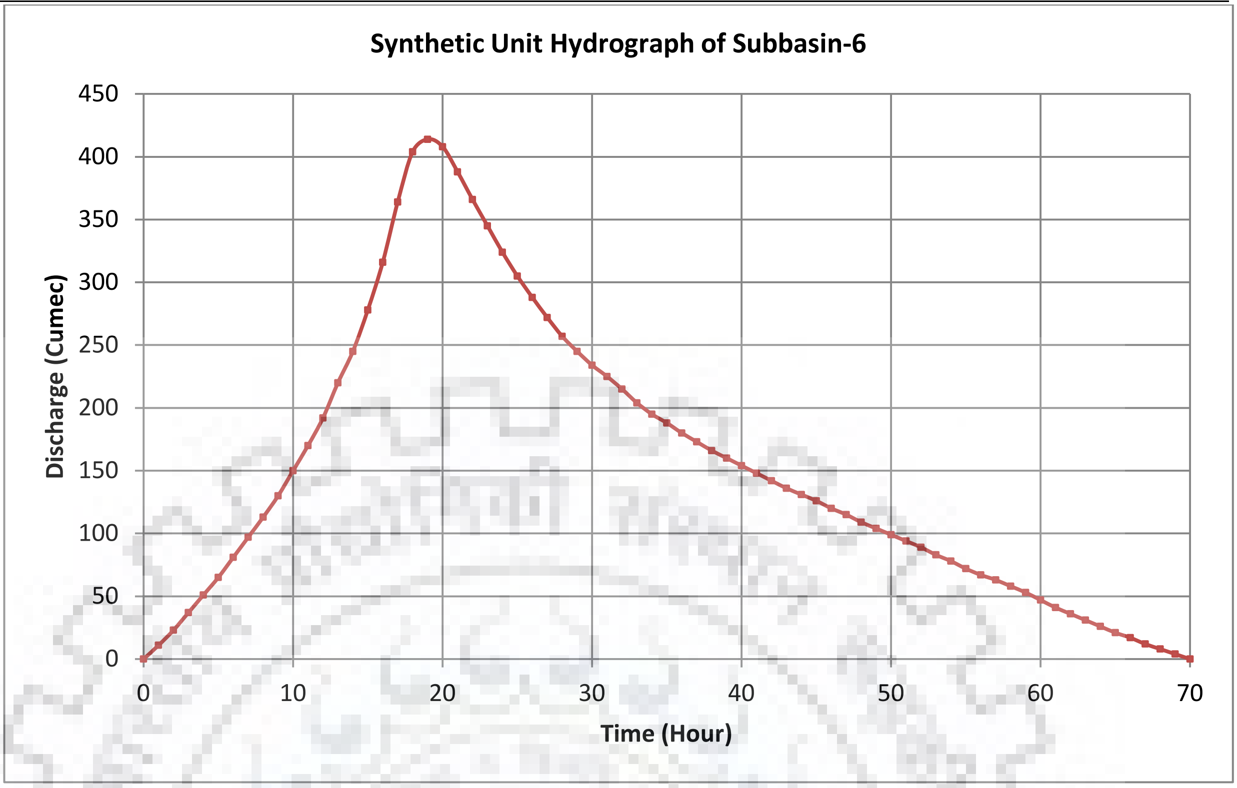

Fig-4.8: Unit Hydrograph of 1-hour duration for SB-6

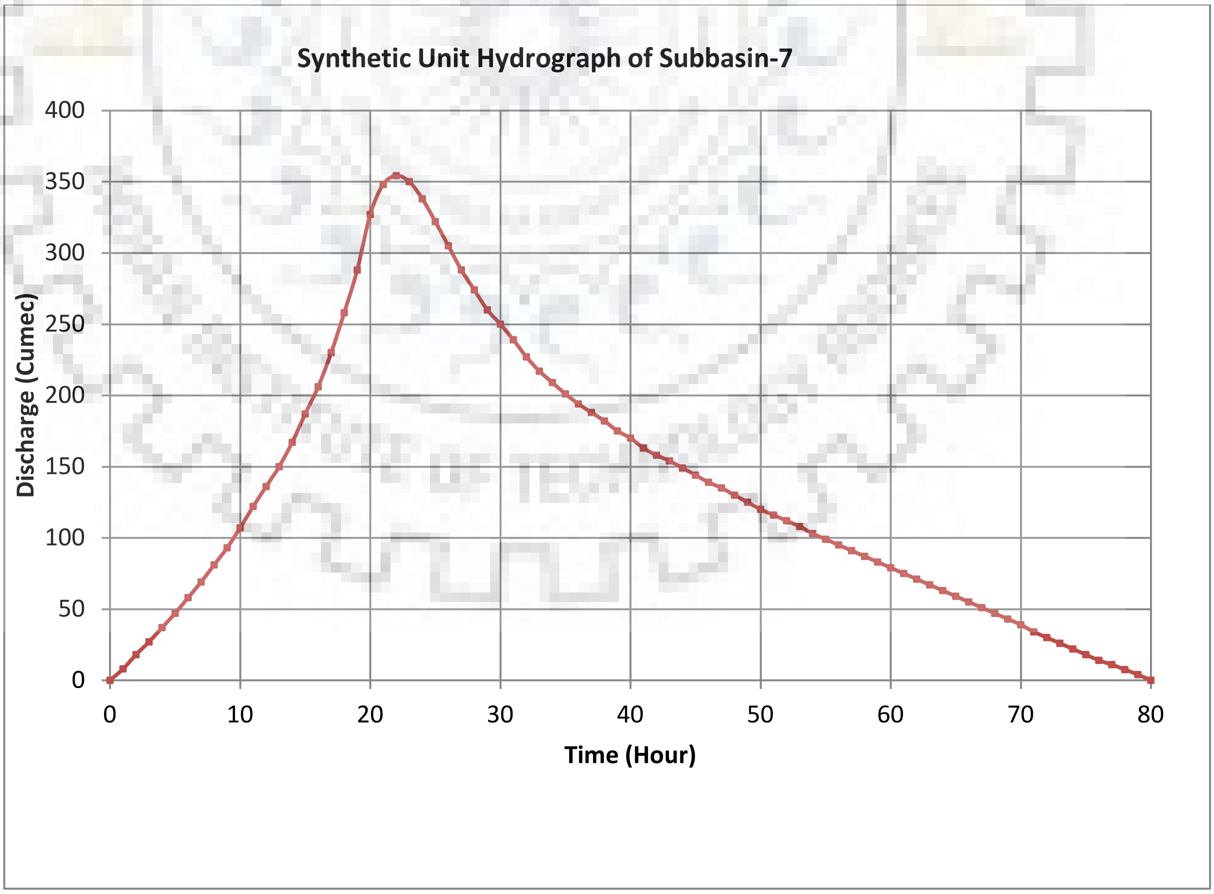

Fig-4.9: Unit Hydrograph of 1-hour duration for SB-7

Table-4.3: Ordinates of Unit Hydrograph for all Sub-basins

Time	Ordinates of 1-Hour Unit Hydrograph of Sub Basins						
	SB-1	SB-2	SB-3	SB-4	SB-5	SB-6	SB-7
(hour)	(cumec)	(cumec)	(cumec)	(cumec)	(cumec)	(cumec)	(cumec)
0	0	0.0	0	0	0	0	0
1	10	8.0	8.0	10	10	11	8
2	20	18.0	17.0	20	20	23	18
3	35	28.0	28.0	32	32	37	27
4	50	40.0	41.0	44	45	51	37
5	72	52.0	56.0	57	60	65	47
6	95	65.0	70.0	70	75	81	58
7	120	78.0	85.0	85	94	97	69
8	145	91.0	100.0	98	116	113	81
9	175	105.0	116.0	115	138.0	130	93
10	204	120.0	135	132	161	150	107
11	236	136.0	162	150.0	190	170	122
12	272	155.0	196.0	172	232.0	192.0	136
13	320	174.0	232	200	272	220	150
14	385	199.0	241.7	232.4	285.3	245	167.0
15	436	224.0	235.0	268	278	278	187
16	451.1	265.0	218.0	300	258	316.0	206
17	440.79	290.0	200.0	309.9	238	364	230
18	418	298.7	185.0	304	220.0	404	258.0
19	388	294	169.3	289	202	413.9	288
20	360	282	157.3	270	185	408	327
21	333	262	145.3	252	171	388	348
22	312	246	135.0	234.0	160	366	354.2
23	290	230.0	128.0	220	151	345	350
24	270	216	120.9	205	143.0	324	338
25	255	204	114.0	192	136	305.0	322
26	242	193	108.0	182	129	288	305
27	230	184	103.0	172	122	272	288
28	218	175	98.0	164	115	257	274.0
29	208	168	93.0	157	109	245	260
30	200	160	88.0	150.0	103	234	250
31	192	153.0	83.0	144	98	225	239
32	183	147.0	79.0	139	92	215	227
33	175	140.5	74.0	134	86	204.0	217
34	167	135.0	70.0	129	81	195	209
35	158	129.0	66.0	123	76	188	201
36	150	125.0	61.0	118	71	180	194
37	143	121.0	57.0	113	66	173	188
38	136	116.0	53.0	108	61	166	182
39	130	112.0	49.0	103	56	160	175.0

40	122.5	107.0	45.0	98	51	154	170
41	114.5	102.0	40.5	93	46	148	163
42	107	98.0	36.5	88	40	142	158
43	99.5	94.0	32.0	83	35	136	154
44	94.5	90.0	28.0	79	31	131	149
45	87.5	86.0	24.0	74	27	126	144
46	81	82.0	20.0	70	23	120	139
47	74.5	77.0	16.0	66	19	115	135
48	68.5	73.0	12.0	61	14	109	130
49	62.5	69.0	9.0	56	10	104	125
50	56	64.0	6.0	52	6	99	120
51	50	60.0	3.0	48	3	94	116
52	45	57.0	0	44	0	89	112
53	40	53.0		40		83	108
54	34	49.0		36		78	103
55	27	45.0		32		72	99
56	22	41.0		28		67	95
57	16	37.0		23		63	91
58	10	33		20		58	87
59	4	29		16		53	83
60	0	26		12		47	79
61		22		8		41	75
62		18		5		36	71
63		15		2		31	67
64		12		0		26	63
65		9				21	59
66		6				17	55
67		3				12	51
68		0				8	47
69						4	43
70						0	39
71							34
72							30
73							26
74							22
75							18
76							14
77							11
78							7.5
79							4
80							0

4.4 Identification of Historical Storm and Standard Project Storm (SPS)

4.4.1 Identification of Historical Storm Events

It is observed from the Unit Hydrograph of all sub basins that base hour of the hydrograph varies from 52 to 80 hours. Thus, storm duration of 3-day (72 hour) has been considered for identifying the peak values of the historical storms for this study. The PMP Atlas for Ganga Basin has been referred, which provides the details of the historical storm events of 1-day, 2-day and 3-day storms affecting the basin. 3-day historical storm locations with peak value which had occurred near the study area have been shown in the Fig-4.10, below.

Fig-4.10 Historical maximum 3-day storm values

Considering the location of catchment, severe rainstorms that can affect the Matatila catchment have been analyzed. Following ten storm events with maximum peaks have been identified for further analysis. The details of storms i.e. storm date, peak value, name and location of storm centre etc. has been shown in Table-4.4.

Table-4.4 List of 3-day storms affecting the Study Area

Sr. No.	Storm Date	Storm Duration	Peak (mm)	Storm Centre	Lat (Deg)	Long (Deg)
1	06-08 Sep 1910	3-Day	505	Atru	24.87	76.67
2	07-09 Aug 1919	3-Day	564	Panna	24.72	80.20
3	29-31 Jul 1923	3-Day	580	Atru	24.87	76.67
4	19-21 Sep 1926	3-Day	830	Bichhia-Dindori	20.95	81.08
5	24-26 Jul 1958	3-Day	536	Chabra/Disp	24.67	76.85
6	27-29 Jul 1965	3-Day	725	Raisen	23.33	77.80
7	22-24 Jul 1971	3-Day	640	Shahabad	25.25	77.13
8	03-05 Sep 1978	3-Day	560	Panna	24.72	80.17
9	15-17 Sep 1990	3-Day	418	Seodha(Seondha)	26.15	78.80
10	03-05 Jul 2005	3-Day	665	Sagar	23.85	78.75

From the above table, it has been identified that following storms i.e. 1. Bichhia-Dindori (peak-830 mm) 2. Raisen (peak-725mm) 3. Sagar (665 mm) and 4. Shahabad (640 mm) would be used for further analysis. The 3-day Isohyets of these storms have been transported and superimposed over catchment area in ARC-GIS. The storms would be superimposed over the catchment in such a manner that the centre of the storm would lie near the centre of catchment, so as to obtain maximum average value of rainfall depth. To arrive at the maximum average value of precipitation for the catchment, a few trails would be performed. The storm, for which maximum average depth of rainfall will be obtained, would be the selected as the Standard Project Storm (SPS) for the Project. The trails of superimposing the storm isohyets over the catchment have been performed and the average rainfall depth obtained from each trail has been shown in the Table-4.5.

Table-4.5: Superimposing Storm Isohyets over the Catchment

Sl No.	Name of Storm	Storm Date	3-day average rainfall depth (mm)			
			Trail-1	Trail-2	Trail-3	Trail-4
1	Bichhia-Dindori	19-21 Sept 1926	561	564	550	558
2	Raisen	27-29 July 1965	320.6	315.5	316.8	-
3	Sagar	03-05 July 2005	406.6	408	417	423
4	Shahabad	22-24 July 1971	353	344	336	-

From the above trails, the maximum average value for the catchment has been obtained from Bichhia-Dindori Storm 564 mm in trail-2, which has been adopted as Standard Project Storm (SPS) for the study.

4.4.2 Synoptic Storm Situation

A severe rainstorm lasting for 3 days occurred south of the Ganga River Basin during 19-21 September 1926. The rainstorm was caused by a cyclonic storm which formed in the Bay of Bengal in the morning of 14 September and moved inland over the Indian region. It crossed the Odisha coast on 16 September and after moving over to Odisha and Madhya Pradesh, it lay centered to the north of the basin on the morning of 18 September. From here, it turned and moved slowly in north direction during 19-23 September before dissipating. Under the influence of the depression, strong to vigorous monsoon conditions prevailed over areas north of the basin as shown in Fig-4.10a. The strong monsoon conditions combined with the slow movement of depression caused heavy and continuous rainfall over the areas south of basin during 19-21 September resulting in a severe rainstorm.

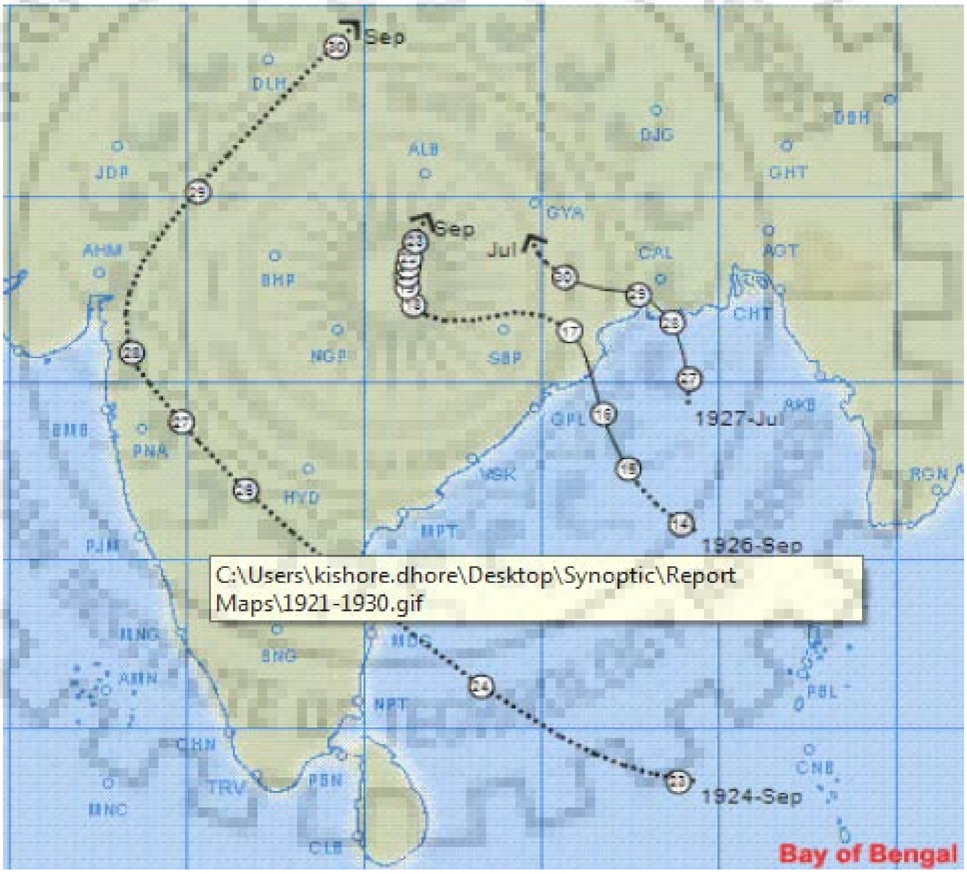

Fig-4.10a Synoptic storm situation

4.4.3 Storm Isohyet Transposition

The storm isohyets of 3-day, 2-day, and 1-day of the Bichhia-Dindori Storm (SPS) have been superimposed over the catchment in similar manner as described above using a number of trials to obtain the maximum average value of rainfall depth for the entire catchment and these are shown in Fig-4.11, 4.12 and 4.13 respectively. Average rainfall depths of 3-day, 2-day, and 1-

day for each sub basins (i.e SB-1 to SB-7) have been computed and shown in Table-4.6, 4.7 and 4.8 respectively.

3-day Storm Isohyet Transposition

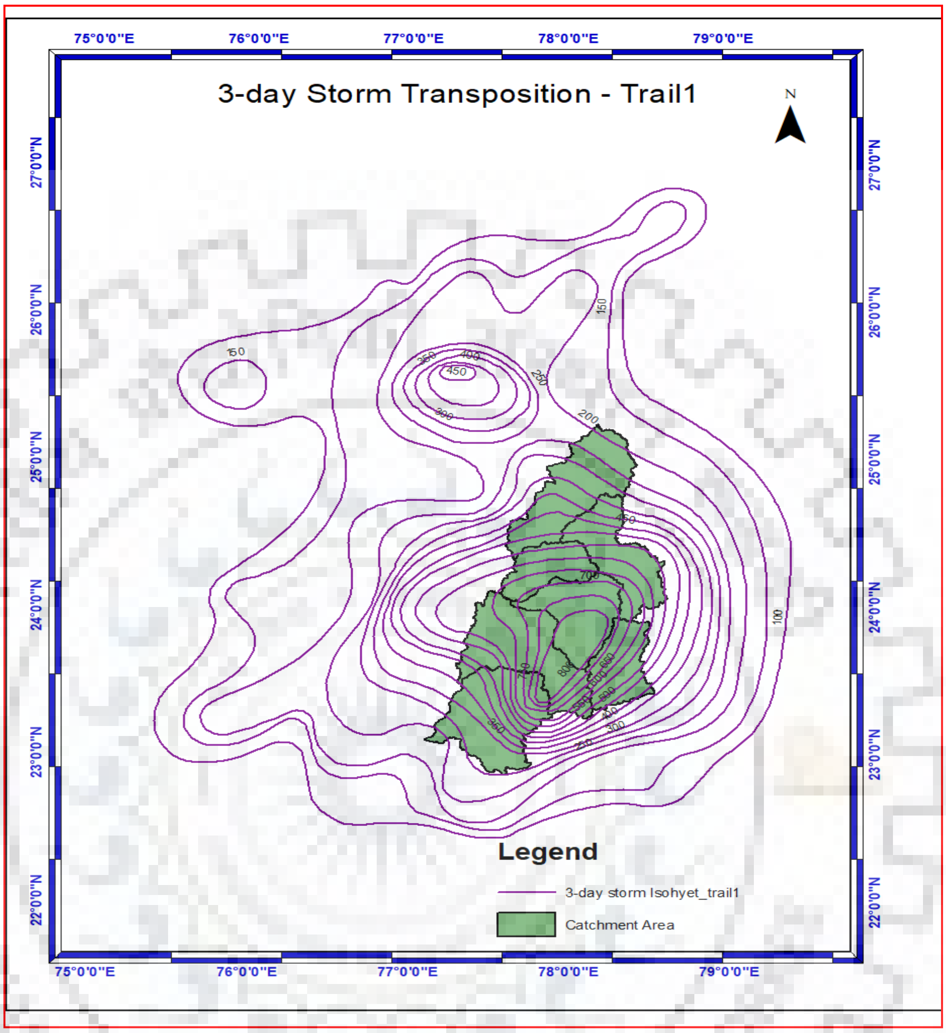

Fig-4.11: 3-day Storm Isohyet Transposition

Table-4.6 Average Rainfall depth of 3-day storm transposition Sub-basin wise

S.No	Sub-Basin	Area in Km2	Mean Rainfall (mm)	Product
1	Subbasin-1	3543	350.25	1240936
2	Subbasin-6	3882	677.15	2628696
3	Subbasin-2	2735	550.11	1504551
4	Subbasin-4	2639	767.45	2025301
5	Subbasin-7	3900	463.43	1807377
6	Subbasin-3	1674	626.61	1048945
7	Subbasin-5	1959	616.15	1207038
	Total	20332		11462843
	Catchment Average Rainfall Depth (mm)			564

2-day Storm Isohyet Transposition

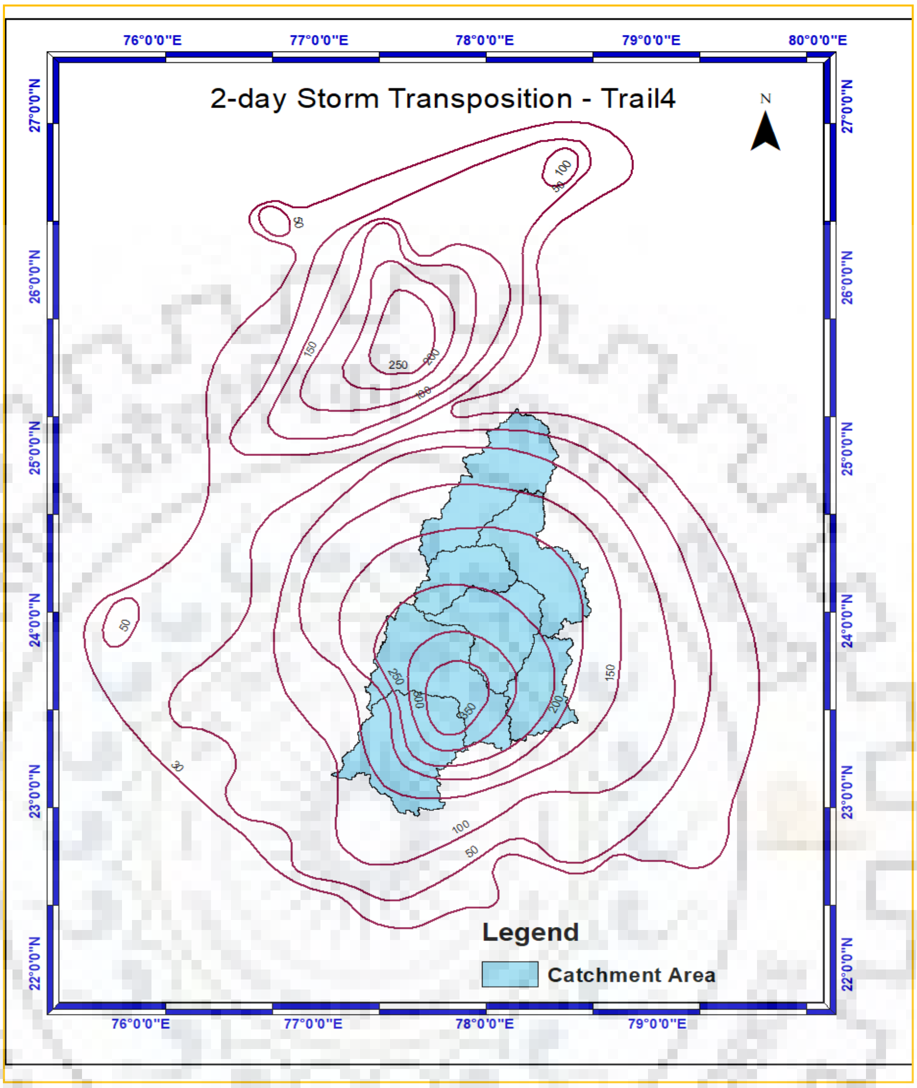

Fig-4.12: 2-day Storm Isohyet Transposition

Table-4.7 Average Rainfall depth of 2-day storm transposition Sub-basin wise

S.No	Sub-Basin	Area in Km2	Mean Rainfall (mm)	Product
1	Subbasin-1	3543	291.37	1032324
2	Subbasin-6	3882	517	2006994
3	Subbasin-2	2735	465.88	1274182
4	Subbasin-4	2639	539.3	1423213
5	Subbasin-7	3900	346.9	1352910
6	Subbasin-3	1674	486.2	813899
7	Subbasin-5	1959	366.6	718169
	Total	20332		8621691
	Catchment Average Rainfall Depth (mm)			424

1-day Storm Isohyet Transposition

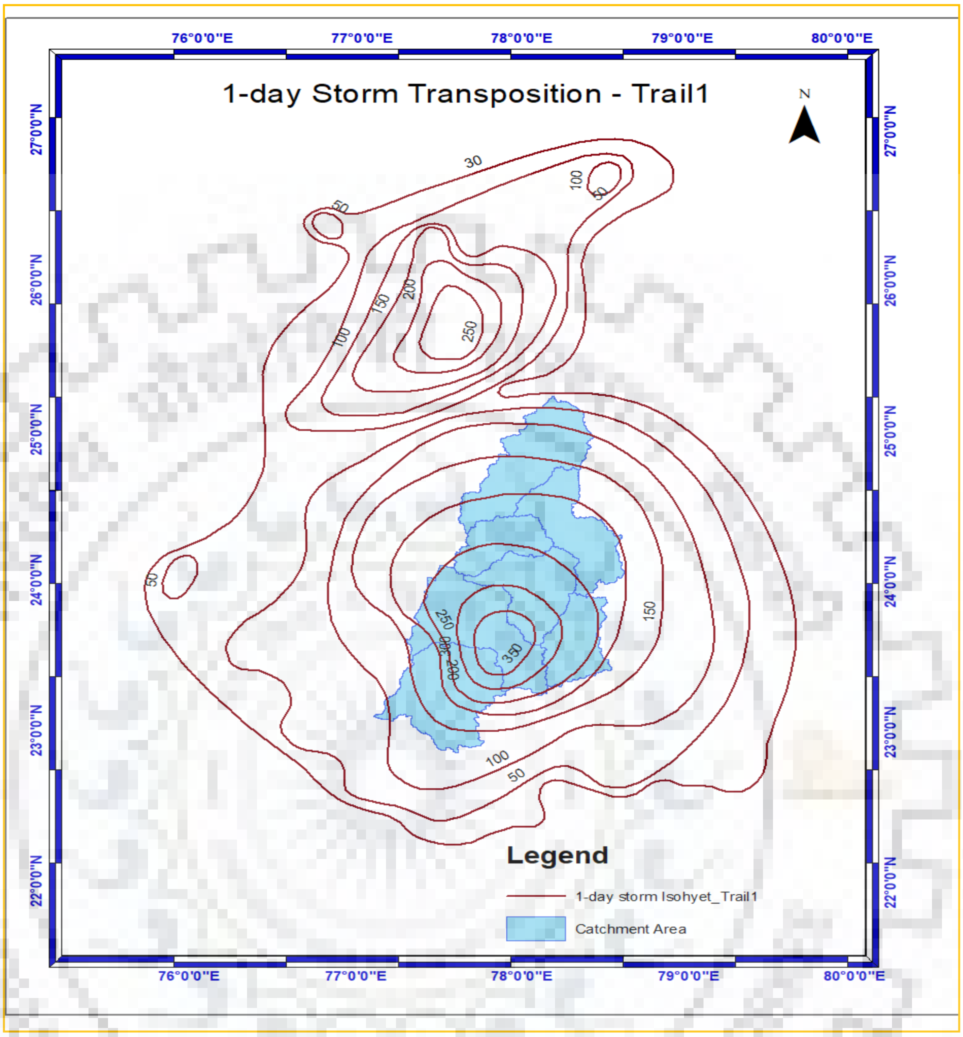

Fig-4.13: 1-day Storm Isohyet Transposition

Table-4.8 Average Rainfall depth of 1-day storm transposition Sub-basin wise

S.No	Sub-Basin	Area in Km2	Mean Rainfall (mm)	Product
1	Subbasin-1	3543	147.88	523939
2	Subbasin-6	3882	301.88	1171898
3	Subbasin-2	2735	215.06	588189
4	Subbasin-4	2639	292.95	773095
5	Subbasin-7	3900	204.62	798018
6	Subbasin-3	1674	245.7	411302
7	Subbasin-5	1959	269.35	527657
	Total	20332		4794098
	Catchment Average Rainfall Depth (mm)			236

4.4.4 Summary of 1-day, 2-day & 3-day SPS depth

Table-4.9: SPS depths of different durations

Sub-Basin	1-day SPS depth (mm)	2-day SPS depth(mm)	3-day SPS depth(mm)
SB-1	148	291	350
SB-2	215	466	550
SB-3	246	486	627
SB-4	293	539	767
SB-5	269	367	616
SB-6	302	517	677
SB-7	205	347	463
Catchment Avg. SPS	236	424	564

4.5 Calculation for Probable Maximum Precipitation (PMP)

4.5.1 Analysis for Moisture Adjustment Factor (MAF)

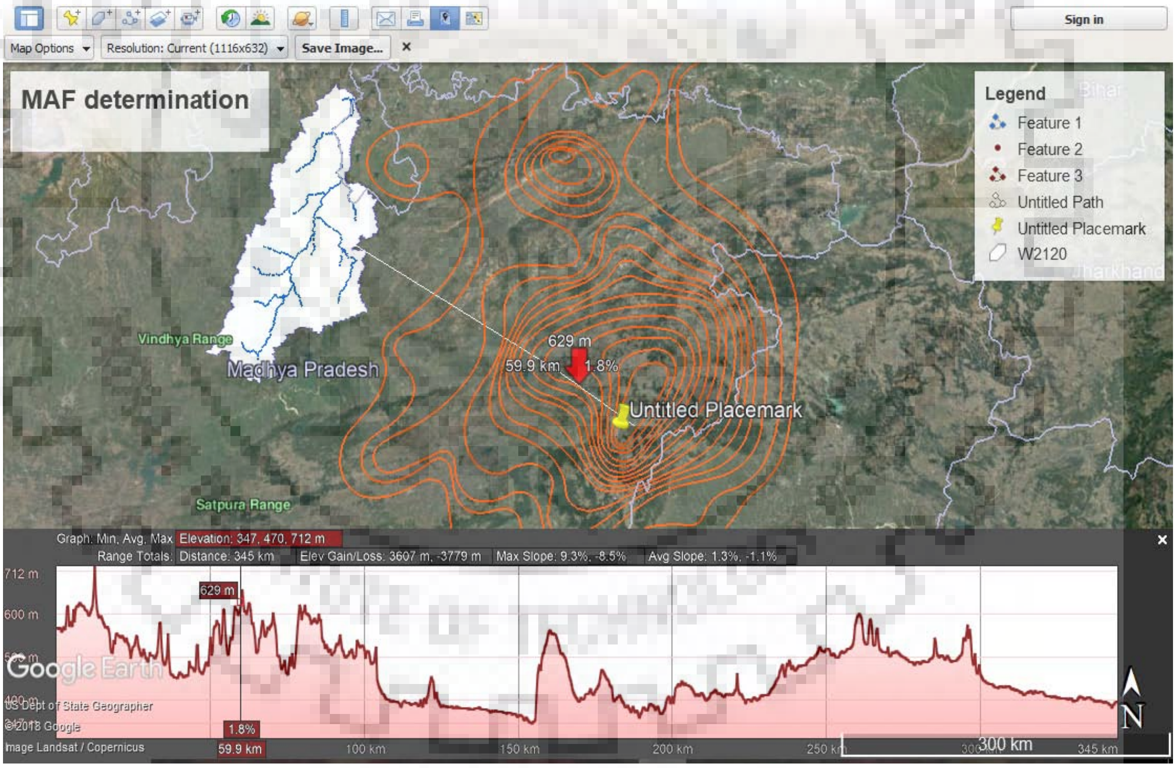

Fig-4.14: Determination of h2 using google earth pro

The parameter for determination of the MAF viz. h1, h2, d1, d2, d3, W1(h1), (W2)h1, (W3)h1 and (W3)h2 has been defined in Table 4.10. Parameter h1 is available for the Bichhia-Dindori storm; however, the parameter h2 has been estimated using the Google Earth Pro as shown in Fig-4.14. Persisting dew point temperatures viz. d1, d2, d3 have been obtained from the PMP

Atlas. The precipitable water column between 1,000hPa and 300 hPa levels viz. (W1)h1, (W2)h1, (W3)h1 and (W3)h2 have been estimated by using the tables available in PMP atlas. MAF calculations have been performed as shown in Table-4.10.

Table-4.10: MAF Computation for the Storm over the Catchment Area

Storm Date : 19-21 Sept 1926		
Storm Name : Bichhia-Dindori		
Definition	Parameter	Value
h1 is mean crest elevation of the barrier between the rainstorm centre and source of moisture with mean crest elevation higher than that of the rainstorm centre	h1	600
h2 is mean crest elevation of the barrier between the original location of rainstorm and the transposed location with mean crest elevation higher than mean elevation of original and transposed locations of rainstorm	h2	600
representative persisting storm dew point temperature (d1)	d1	24.7
maximum persisting dew point temperature (d2) on record at the location of the rainstorm in the same fortnight of the month in which the rainstorm occurred	d2	26
maximum persisting dew point temperature (d3) on record at the transposed location of the rainstorm in the same fortnight of the month in which the rainstorm occurred	d3	26
precipitable water in an atmospheric column between 1,000 and 300 hPa levels	(W1)h1	66.02
	(W2)h1	74.6
	(W3)h1	74.6
	(W3)h2	74.6
Moisture Maximization Factor (MMF)	(W2)h1/(W1)h1	1.13
Location Adjustment Factor (LAF)	(W3)h1/(W2)h1	1.00
Barrier Adjustment Factor (BAF)	(W3)h2/(W3)h1	1
Topography Adjustment Factor (TAF)	LAF x BAF	1.00
Moisture Adjustment Factor (MAF)	**MMFxTAF**	**1.13**

The MAF for the study has been calculated as **1.13.**

4.5.2 PMP values of each Sub Basin

PMP = SPS x MAF. The values of PMP are calculated by multiplying the SPS values with MAF and given in Table-4.11.

Table-4.11: PMP depths of different durations

Sub-Basin	1-day PMP depth (mm)	2-day PMP depth(mm)	3-day PMP depth(mm)
SB-1	167	329	396
SB-2	243	526	622
SB-3	278	549	708
SB-4	331	609	867
SB-5	304	414	696
SB-6	341	584	765
SB-7	231	392	524
PMP of the Catchment	266	479	637

4.6 Effective Rainfall Hyetograph

4.6.1 Incremental daily PMP depth for 1st day, 2nd day and 3rd day

As 3-day PMP has been used for estimating the design flood, and the incremental daily PMP for each sub basin has been worked as follows:

1st day PMP depth = 1day PMP depth

2nd day PMP depth = (2day PMP depth) – (1day PMP depth)

3rd day PMP depth = (3day PMP depth) – (2day PMP depth)

Table-4.12: PMP depths of 1st day, 2nd day and 3rd day

Sub-Basin	1st day PMP depth (mm)	2nd day PMP depth(mm)	3rd day PMP depth(mm)
SB-1	167	162	67
SB-2	243	283	95
SB-3	278	272	159
SB-4	331	278	258
SB-5	304	110	282
SB-6	341	243	181
SB-7	231	161	132

4.6.2 Critical arrangement of PMP depth in 12-hour Bells

To get the critical arrangement of rainfall ordinates, rainfall of each day arranged in such a way that the largest depth is preceded by the 2nd largest and succeeded by the 3rd largest (as per the Manual of Design Flood, CWC). Further, after critical arrangement of the rainfall ordinates, rainfall depths of each day have been arranged in 2 bells of 12 hour each. The rainfall depths for 1st and 2nd 12 hour bells have been taken as 73% and 27% of 24 hour rainfall respectively (from the PMP Atlas Vol-I of Ganga Basin). The critical arrangement of PMP depths and bell arrangement is presented in Table-4.13 and 4.14 respectively.

Table-4.13: Critical arrangement of daily PMP depths

Sub-Basin	1st day PMP depth (mm)	2nd day PMP depth(mm)	3rd day PMP depth(mm)
SB-1	162	167	67
SB-2	243	283	95
SB-3	272	278	159
SB-4	278	331	258
SB-5	282	304	110
SB-6	243	341	181
SB-7	161	231	132

Table-4.14: 12 hour Bell arrangement of daily PMP depths

Sub-Basin	PMP depths(mm) in 12 hour Bells					
	Bell-B1	Bell-B2	Bell-B3	Bell-B4	Bell-B5	Bell-B6
SB-1	118.3	43.7	121.9	45.1	48.9	18.1
SB-2	177.4	65.6	206.9	76.5	69.5	25.7
SB-3	198.6	73.4	202.9	75.1	115.8	42.8
SB-4	202.9	75.1	241.6	89.4	188.2	69.6
SB-5	205.9	76.1	221.9	82.1	80.3	29.7
SB-6	177.4	65.6	248.9	92.1	132.1	48.9
SB-7	117.5	43.5	168.6	62.4	96.1	35.6

The 12 hour bells of each day has been interchanged wherever it has been found feasible to maximize the runoff, in such a way that the maximum depth in any 24 hour sequence shall not exceed the 1-day storm depth x 1.15 (Clock-hour correction factor) or 1day storm depth + 50 mm whichever is less. In this case B1-B2 is interchanged as B2-B1 and B3-B4 as B4-B3 to obtain the sequence. The calculations performed are shown in Table-4.15

Table-4.15: PMP depth of 12 hour bell after bell rearrangement

Sub-Basin	Rearrangement of PMP depths(mm) in 12 hour Bells					
	B2	B1	B4	B3	B5	B6
SB-1	43.7	118.3	45.1	121.9	48.9	18.1
SB-2	65.6	177.4	76.5	206.9	69.5	25.7
SB-3	73.4	198.6	75.1	202.9	115.8	42.8
SB-4	75.1	202.9	89.4	241.6	188.2	69.6
SB-5	76.1	205.9	82.1	221.9	80.3	29.7
SB-6	65.6	177.4	92.1	248.9	132.1	48.9
SB-7	43.5	117.5	62.4	168.6	96.1	35.6

4.6.3 Time Distribution (TD)

The short duration storm rainfall values can be determined by applying the Time Distributions (TD) of rainfall obtained from the analysis of hourly rainfall data of some known major storms in the catchments under study. These values have been taken from the PMP Atlas of Ganga Basin – Betwa Catchment, which analyzed the storm data of 12-hour rain spells from self-recording rain gauges (SRRG) available in the basin to arrive at the distribution coefficients. The time distribution coefficients for the 12 hour bells are shown in the Table-4.16.

Table-4.16: TD coefficients (12-hour)

Time(hr)	Time Distribution (TD) Coefficients for Betwa Catchment for 12-Hour Rain Spells
1	20.50%
2	35.00%
3	44.70%
4	53.50%
5	61.70%
6	68.00%
7	73.80%
8	79.50%
9	85.00%
10	90.30%
11	95.20%
12	100.00%

4.6.4 Hourly distribution of rainfall

The hourly distribution of rainfall of each bell for different sub-basins has been carried out by using PMP depth of rainfall as per Table-4.15 and time distribution coefficient (TD) of 12 hour bell as per Table-4.16. The calculations for cumulative hourly rainfall and incremental hourly rainfall have been performed for each sub-basin i.e. from sub basin-1 to sub basin-7 and given in Table-4.17, 4.18, 4.19, 4.20, 4.21, 4.22 and 4.23 respectively.

Table-4.17: Hourly distribution of rainfall of SB1

Sub Basin	Rainfall Depth of 12 hour Bell(in mm)					
SB1	43.7	118.3	45.1	121.9	48.9	18.1

		Cumulative Hourly Rainfall in mm (12 hour Bell)						Incremental Hourly Rainfall in mm (12 hour Bell)					
Hours	Time distribution	B2	B1	B4	B3	B5	B6	B2	B1	B4	B3	B5	B6
1	20.50%	8.97	24.24	9.24	24.99	10.03	3.71	8.97	24.24	9.24	24.99	10.03	3.71
2	35.00%	15.31	41.39	15.78	42.67	17.12	6.33	6.34	17.15	6.54	17.68	7.09	2.62
3	44.70%	19.55	52.86	20.16	54.49	21.86	8.09	4.24	11.47	4.37	11.83	4.74	1.75
4	53.50%	23.40	63.27	24.12	65.22	26.17	9.68	3.85	10.41	3.97	10.73	4.30	1.59
5	61.70%	26.99	72.97	27.82	75.22	30.18	11.16	3.59	9.70	3.70	10.00	4.01	1.48
6	68.00%	29.74	80.42	30.66	82.90	33.26	12.30	2.76	7.45	2.84	7.68	3.08	1.14
7	73.80%	32.28	87.28	33.28	89.97	36.10	13.35	2.54	6.86	2.62	7.07	2.84	1.05
8	79.50%	34.77	94.02	35.85	96.92	38.88	14.38	2.49	6.74	2.57	6.95	2.79	1.03
9	85.00%	37.18	100.52	38.33	103.62	41.57	15.38	2.41	6.50	2.48	6.71	2.69	0.99
10	90.30%	39.50	106.79	40.72	110.08	44.17	16.34	2.32	6.27	2.39	6.46	2.59	0.96
11	95.20%	41.64	112.58	42.93	116.06	46.56	17.22	2.14	5.79	2.21	5.97	2.40	0.89
12	100.00%	43.74	118.26	45.09	121.91	48.91	18.09	2.10	5.68	2.16	5.85	2.35	0.87

Table-4.18: Hourly distribution of rainfall of SB2

Sub Basin	Rainfall Depth of 12 hour Bell(in mm)					
SB2	65.6	177.4	76.5	206.9	69.5	25.7

		Cumulative Hourly Rainfall in mm (12 hour Bell)						Incremental Hourly Rainfall in mm (12 hour Bell)					
Hours	Time distribution (12 hours) in Percentage	B2	B1	B4	B3	B5	B6	B2	B1	B4	B3	B5	B6
1	20.50%	13.45	36.37	15.69	42.41	14.24	5.27	13.45	36.37	15.69	42.41	14.24	5.27
2	35.00%	22.97	62.09	26.78	72.42	24.32	8.99	9.51	25.72	11.10	30.00	10.07	3.73
3	44.70%	29.33	79.30	34.21	92.48	31.06	11.49	6.36	17.21	7.42	20.07	6.74	2.49
4	53.50%	35.10	94.91	40.94	110.69	37.17	13.75	5.77	15.61	6.73	18.21	6.11	2.26
5	61.70%	40.48	109.46	47.22	127.66	42.87	15.86	5.38	14.55	6.28	16.97	5.70	2.11
6	68.00%	44.62	120.63	52.04	140.69	47.25	17.48	4.13	11.18	4.82	13.03	4.38	1.62
7	73.80%	48.42	130.92	56.48	152.69	51.28	18.97	3.81	10.29	4.44	12.00	4.03	1.49
8	79.50%	52.16	141.04	60.84	164.49	55.24	20.43	3.74	10.11	4.36	11.79	3.96	1.46
9	85.00%	55.77	150.79	65.05	175.87	59.06	21.84	3.61	9.76	4.21	11.38	3.82	1.41
10	90.30%	59.25	160.19	69.10	186.83	62.74	23.21	3.48	9.40	4.06	10.97	3.68	1.36
11	95.20%	62.47	168.89	72.85	196.97	66.15	24.47	3.22	8.69	3.75	10.14	3.40	1.26
12	100.00%	65.61	177.40	76.53	206.90	69.48	25.70	3.15	8.52	3.67	9.93	3.34	1.23

Table-4.19: Hourly distribution of rainfall of SB3

Sub Basin	Rainfall Depth of 12 hour Bell(in mm)					
SB3	73.4	198.6	75.1	202.9	115.8	42.8

		Cumulative Hourly Rainfall in mm (12 hour Bell)						Incremental Hourly Rainfall in mm (12 hour Bell)					
Hours	Time distribution (12 hours) in Percentage	B2	B1	B4	B3	B5	B6	B2	B1	B4	B3	B5	B6
1	20.50%	15.06	40.70	15.39	41.60	23.74	8.78	15.06	40.70	15.39	41.60	23.74	8.78
2	35.00%	25.70	69.50	26.27	71.03	40.54	14.99	10.65	28.79	10.88	29.43	16.79	6.21
3	44.70%	32.83	88.76	33.55	90.71	51.77	19.15	7.12	19.26	7.28	19.69	11.23	4.16
4	53.50%	39.29	106.23	40.16	108.57	61.97	22.92	6.46	17.47	6.61	17.86	10.19	3.77
5	61.70%	45.31	122.51	46.31	125.21	71.46	26.43	6.02	16.28	6.15	16.64	9.50	3.51
6	68.00%	49.94	135.02	51.04	138.00	78.76	29.13	4.63	12.51	4.73	12.79	7.30	2.70
7	73.80%	54.20	146.54	55.39	149.77	85.48	31.62	4.26	11.52	4.35	11.77	6.72	2.48
8	79.50%	58.38	157.86	59.67	161.34	92.08	34.06	4.19	11.32	4.28	11.57	6.60	2.44
9	85.00%	62.42	168.78	63.80	172.50	98.45	36.41	4.04	10.92	4.13	11.16	6.37	2.36
10	90.30%	66.32	179.30	67.78	183.25	104.59	38.68	3.89	10.52	3.98	10.76	6.14	2.27
11	95.20%	69.91	189.03	71.46	193.20	110.26	40.78	3.60	9.73	3.68	9.94	5.68	2.10
12	100.00%	73.44	198.56	75.06	202.94	115.82	42.84	3.53	9.53	3.60	9.74	5.56	2.06

Table-4.20: Hourly distribution of rainfall of SB4

Sub Basin	Rainfall Depth of 12 hour Bell(in mm)					
SB4	75.1	202.9	89.4	241.6	188.2	69.6

		Cumulative Hourly Rainfall in mm (12 hour Bell)						Incremental Hourly Rainfall in mm (12 hour Bell)					
Hours	Time distribution (12 hours) in Percentage	B2	B1	B4	B3	B5	B6	B2	B1	B4	B3	B5	B6
1	20.50%	15.39	41.60	18.32	49.53	38.58	14.27	15.39	41.60	18.32	49.53	38.58	14.27
2	35.00%	26.27	71.03	31.28	84.57	65.87	24.36	10.88	29.43	12.96	35.04	27.29	10.09
3	44.70%	33.55	90.71	39.95	108.01	84.13	31.12	7.28	19.69	8.67	23.44	18.26	6.75
4	53.50%	40.16	108.57	47.81	129.27	100.69	37.24	6.61	17.86	7.86	21.26	16.56	6.13
5	61.70%	46.31	125.21	55.14	149.09	116.12	42.95	6.15	16.64	7.33	19.81	15.43	5.71
6	68.00%	51.04	138.00	60.77	164.31	127.98	47.33	4.73	12.79	5.63	15.22	11.86	4.39
7	73.80%	55.39	149.77	65.96	178.32	138.89	51.37	4.35	11.77	5.18	14.01	10.92	4.04
8	79.50%	59.67	161.34	71.05	192.10	149.62	55.34	4.28	11.57	5.09	13.77	10.73	3.97
9	85.00%	63.80	172.50	75.96	205.39	159.97	59.17	4.13	11.16	4.92	13.29	10.35	3.83
10	90.30%	67.78	183.25	80.70	218.19	169.95	62.86	3.98	10.76	4.74	12.81	9.97	3.69
11	95.20%	71.46	193.20	85.08	230.03	179.17	66.27	3.68	9.94	4.38	11.84	9.22	3.41
12	100.00%	75.06	202.94	89.37	241.63	188.20	69.61	3.60	9.74	4.29	11.60	9.03	3.34

Table-4.21: Hourly distribution of rainfall of SB5

Sub Basin	Rainfall Depth of 12 hour Bell(in mm)					
SB5	76.1	205.9	82.1	221.9	80.3	29.7

		Cumulative Hourly Rainfall in mm (12 hour Bell)						Incremental Hourly Rainfall in mm (12 hour Bell)					
Hours	Time distribution (12 hours) in Percentage	B2	B1	B4	B3	B5	B6	B2	B1	B4	B3	B5	B6
1	20.50%	15.61	42.20	16.83	45.49	16.46	6.09	15.61	42.20	16.83	45.49	16.46	6.09
2	35.00%	26.65	72.05	28.73	77.67	28.11	10.40	11.04	29.85	11.90	32.18	11.64	4.31
3	44.70%	34.03	92.02	36.69	99.20	35.89	13.28	7.39	19.97	7.96	21.53	7.79	2.88
4	53.50%	40.73	110.14	43.91	118.73	42.96	15.89	6.70	18.12	7.22	19.53	7.07	2.61
5	61.70%	46.98	127.02	50.64	136.92	49.55	18.32	6.24	16.88	6.73	18.20	6.58	2.44
6	68.00%	51.78	139.98	55.81	150.91	54.60	20.20	4.80	12.97	5.17	13.98	5.06	1.87
7	73.80%	56.19	151.92	60.58	163.78	59.26	21.92	4.42	11.94	4.76	12.87	4.66	1.72
8	79.50%	60.53	163.66	65.25	176.43	63.84	23.61	4.34	11.73	4.68	12.65	4.58	1.69
9	85.00%	64.72	174.98	69.77	188.63	68.26	25.25	4.19	11.32	4.51	12.21	4.42	1.63
10	90.30%	68.75	185.89	74.12	200.39	72.51	26.82	4.04	10.91	4.35	11.76	4.26	1.57
11	95.20%	72.49	195.98	78.14	211.27	76.45	28.27	3.73	10.09	4.02	10.87	3.93	1.46
12	100.00%	76.14	205.86	82.08	221.92	80.30	29.70	3.65	9.88	3.94	10.65	3.85	1.43

Table-4.22: Hourly distribution of rainfall of SB6

Sub Basin	Rainfall Depth of 12 hour Bell(in mm)					
SB6	65.6	177.4	92.1	248.9	132.1	48.9

		Cumulative Hourly Rainfall in mm (12 hour Bell)						Incremental Hourly Rainfall in mm (12 hour Bell)					
Hours	Time distribution (12 hours) in Percentage	B2	B1	B4	B3	B5	B6	B2	B1	B4	B3	B5	B6
1	20.50%	13.45	36.36	18.87	51.03	27.08	10.02	13.45	36.36	18.87	51.03	27.08	10.02
2	35.00%	22.96	62.09	32.22	87.13	46.24	17.10	9.51	25.72	13.35	36.09	19.16	7.08
3	44.70%	29.33	79.29	41.16	111.27	59.05	21.84	6.36	17.21	8.93	24.15	12.81	4.74
4	53.50%	35.10	94.90	49.26	133.18	70.68	26.14	5.77	15.61	8.10	21.91	11.63	4.30
5	61.70%	40.48	109.45	56.81	153.59	81.51	30.15	5.38	14.55	7.55	20.41	10.83	4.01
6	68.00%	44.61	120.63	62.61	169.27	89.83	33.23	4.13	11.18	5.80	15.68	8.32	3.08
7	73.80%	48.42	130.91	67.95	183.71	97.50	36.06	3.81	10.29	5.34	14.44	7.66	2.83
8	79.50%	52.16	141.03	73.20	197.90	105.03	38.85	3.74	10.11	5.25	14.19	7.53	2.79
9	85.00%	55.77	150.78	78.26	211.59	112.29	41.53	3.61	9.76	5.06	13.69	7.27	2.69
10	90.30%	59.25	160.18	83.14	224.78	119.29	44.12	3.48	9.40	4.88	13.19	7.00	2.59
11	95.20%	62.46	168.88	87.65	236.98	125.77	46.52	3.21	8.69	4.51	12.20	6.47	2.39
12	100.00%	65.61	177.39	92.07	248.93	132.11	48.86	3.15	8.51	4.42	11.95	6.34	2.35

Table-4.23: Hourly distribution of rainfall of SB7

Sub Basin	Rainfall Depth of 12 hour Bell(in mm)					
SB7	43.5	117.5	62.4	168.6	96.1	35.6

		Cumulative Hourly Rainfall in mm (12 hour Bell)						Incremental Hourly Rainfall in mm (12 hour Bell)					
Hours	Time distribution (12 hours) in Percentage	B2	B1	B4	B3	B5	B6	B2	B1	B4	B3	B5	B6
1	20.50%	8.91	24.09	12.79	34.57	19.71	7.29	8.91	24.09	12.79	34.57	19.71	7.29
2	35.00%	15.21	41.14	21.83	59.02	33.64	12.44	6.30	17.04	9.04	24.45	13.94	5.16
3	44.70%	19.43	52.54	27.88	75.38	42.97	15.89	4.22	11.40	6.05	16.36	9.32	3.45
4	53.50%	23.26	62.88	33.37	90.22	51.43	19.02	3.83	10.34	5.49	14.84	8.46	3.13
5	61.70%	26.82	72.52	38.48	104.04	59.31	21.94	3.56	9.64	5.11	13.83	7.88	2.92
6	68.00%	29.56	79.92	42.41	114.67	65.37	24.18	2.74	7.40	3.93	10.62	6.06	2.24
7	73.80%	32.08	86.74	46.03	124.45	70.94	26.24	2.52	6.82	3.62	9.78	5.58	2.06
8	79.50%	34.56	93.44	49.58	134.06	76.42	28.26	2.48	6.70	3.56	9.61	5.48	2.03
9	85.00%	36.95	99.90	53.01	143.34	81.71	30.22	2.39	6.46	3.43	9.27	5.29	1.96
10	90.30%	39.25	106.13	56.32	152.27	86.80	32.10	2.30	6.23	3.31	8.94	5.09	1.88
11	95.20%	41.38	111.89	59.38	160.54	91.51	33.85	2.13	5.76	3.06	8.26	4.71	1.74
12	100.00%	43.47	117.53	62.37	168.63	96.13	35.55	2.09	5.64	2.99	8.09	4.61	1.71

4.6.5 Critical sequence of rainfall

In order to maintain the isohyetal pattern of storm in different sub basins, critical sequence of the hourly rainfall excess of the bells need to be carried out with respect to SUH of either central sub basin or one of the sub basins having the highest SUH peak. For the present case the critical sequence of hourly rainfall of the bells of the all sub basins have been generated with respect to Unit Hydrograph of sub-basin SB-6. For critical sequence the incremental hourly rainfall depths have been arranged in critical order for each bell separately. For this the largest of hourly rainfall depth has been placed against the peak of Unit Hydrograph, then the next largest against the next largest of the Unit Hydrograph ordinate and so on until all hourly rainfall depths get arranged.

The sequence thus obtained for each bell has been reversed to get the critical sequence. The critical sequence of hourly rainfall of different sub basins have been given in Table-4.24, 4.26, 4.28, 4.30, 4.32, 4.34, and 4.36 respectively.

4.6.6 Design base flow

Flood Estimation Report, Betwa Subzone -1(c) recommends a base flow of 0.018 cumec/sq.km. Accordingly, base flow for the present study has been adopted as 0.018 cumec/sq.km for all the sub-basins.

4.6.7 Design loss rate & Effective Rainfall Hyetograph

As per Flood Estimation Report of Betwa Subzone -1(c) a modal value of 2.3 mm/hr can be adopted as design loss rate for Betwa subzone. For the present case a loss rate of 2.3 mm/hour has been adopted. The effective hourly rainfall (design hyetograph) of different sub basins are calculated and shown in Table-4.25, 4.27, 4.29, 4.31, 4.33, 4.35, and 4.37 respectively. The hyetographs of all sub basins are presented in Fig-4.15, 4.16, 4.17, 4.18, 4.19, 4.20, and 4.21 respectively.

Table-4.24: Critical sequencing for hourly rainfall of SB1

Time	UH Ordinates of SB-6	Critical Sequencing of hourly Rainfall in mm (12 hour Bell)						Reverse Critical Sequencing of hourly Rainfall in mm (12 hour Bell)					
(hr)	(cumec)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
12	192												
13	220												
14	245												
15	278	2.10	5.68	2.16	5.85	2.35	0.87	2.14	5.79	2.21	5.97	2.40	0.89
16	316.0	2.41	6.50	2.48	6.71	2.69	0.99	2.32	6.27	2.39	6.46	2.59	0.96
17	364	2.76	7.45	2.84	7.68	3.08	1.14	2.49	6.74	2.57	6.95	2.79	1.03
18	404	4.24	11.47	4.37	11.83	4.74	1.75	2.54	6.86	2.62	7.07	2.84	1.05
19	413.9	8.97	24.24	9.24	24.99	10.03	3.71	3.59	9.70	3.70	10.00	4.01	1.48
20	408	6.34	17.15	6.54	17.68	7.09	2.62	3.85	10.41	3.97	10.73	4.30	1.59
21	388	3.85	10.41	3.97	10.73	4.30	1.59	6.34	17.15	6.54	17.68	7.09	2.62
22	366	3.59	9.70	3.70	10.00	4.01	1.48	8.97	24.24	9.24	24.99	10.03	3.71
23	345	2.54	6.86	2.62	7.07	2.84	1.05	4.24	11.47	4.37	11.83	4.74	1.75
24	324	2.49	6.74	2.57	6.95	2.79	1.03	2.76	7.45	2.84	7.68	3.08	1.14
25	305.0	2.32	6.27	2.39	6.46	2.59	0.96	2.41	6.50	2.48	6.71	2.69	0.99
26	288	2.14	5.79	2.21	5.97	2.40	0.89	2.10	5.68	2.16	5.85	2.35	0.87
27	272												
28	257												
29	245												

Table-4.25: Effective Rainfall for SB-1

Time(hour)	Effective Rainfall in mm after deduction of Losses(2.3cm/hour)					
1	0.00	3.49	0.00	3.67	0.10	0.00
2	0.02	3.97	0.09	4.16	0.29	0.00
3	0.19	4.44	0.27	4.65	0.49	0.00
4	0.24	4.56	0.32	4.77	0.54	0.00
5	1.29	7.40	1.40	7.70	1.71	0.00
6	1.55	8.11	1.67	8.43	2.00	0.00
7	4.04	14.85	4.24	15.38	4.79	0.32
8	6.67	21.94	6.94	22.69	7.73	1.41
9	1.94	9.17	2.07	9.53	2.44	0.00
10	0.46	5.15	0.54	5.38	0.78	0.00
11	0.11	4.20	0.18	4.41	0.39	0.00
12	0.00	3.38	0.00	3.55	0.05	0.00

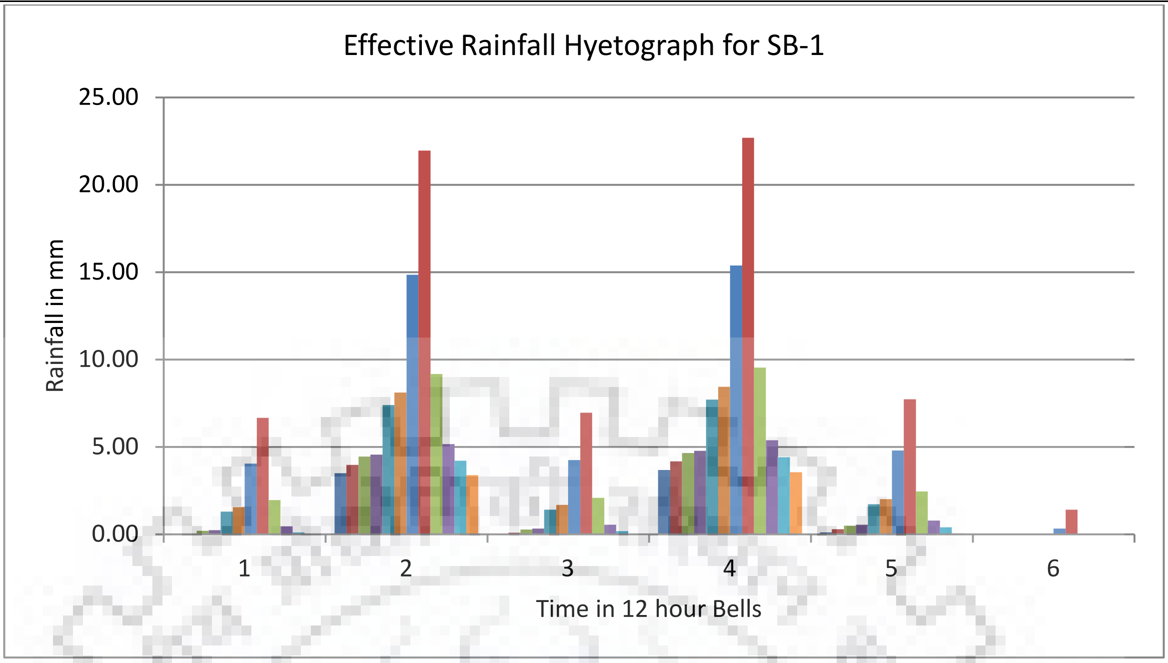

Fig-4.15: Effective Rainfall Hyetograph for SB-1

Table-4.26: Critical sequencing for hourly rainfall of SB2

Time	UH Ordinates of SB-6	Critical Sequencing of hourly Rainfall in mm (12 hour Bell)						Reverse Critical Sequencing of hourly Rainfall in mm (12 hour Bell)					
(hr)	(cumec)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
12	192												
13	220												
14	245												
15	278	3.15	8.52	3.67	9.93	3.34	1.23	3.22	8.69	3.75	10.14	3.40	1.26
16	316.0	3.61	9.76	4.21	11.38	3.82	1.41	3.48	9.40	4.06	10.97	3.68	1.36
17	364	4.13	11.18	4.82	13.03	4.38	1.62	3.74	10.11	4.36	11.79	3.96	1.46
18	404	6.36	17.21	7.42	20.07	6.74	2.49	3.81	10.29	4.44	12.00	4.03	1.49
19	413.9	13.45	36.37	15.69	42.41	14.24	5.27	5.38	14.55	6.28	16.97	5.70	2.11
20	408	9.51	25.72	11.10	30.00	10.07	3.73	5.77	15.61	6.73	18.21	6.11	2.26
21	388	5.77	15.61	6.73	18.21	6.11	2.26	9.51	25.72	11.10	30.00	10.07	3.73
22	366	5.38	14.55	6.28	16.97	5.70	2.11	13.45	36.37	15.69	42.41	14.24	5.27
23	345	3.81	10.29	4.44	12.00	4.03	1.49	6.36	17.21	7.42	20.07	6.74	2.49
24	324	3.74	10.11	4.36	11.79	3.96	1.46	4.13	11.18	4.82	13.03	4.38	1.62
25	305.0	3.48	9.40	4.06	10.97	3.68	1.36	3.61	9.76	4.21	11.38	3.82	1.41
26	288	3.22	8.69	3.75	10.14	3.40	1.26	3.15	8.52	3.67	9.93	3.34	1.23
27	272												
28	257												
29	245												

Table-4.27: Effective Rainfall for SB-2

Time(hour)	Effective Rainfall in mm (12 hour Bell) after deduction of Losses(2.3cm/hour)					
1	0.92	6.39	1.45	7.84	1.10	0.00
2	1.18	7.10	1.76	8.67	1.38	0.00
3	1.44	7.81	2.06	9.49	1.66	0.00
4	1.51	7.99	2.14	9.70	1.73	0.00
5	3.08	12.25	3.98	14.67	3.40	0.00
6	3.47	13.31	4.43	15.91	3.81	0.00
7	7.21	23.42	8.80	27.70	7.77	1.43
8	11.15	34.07	13.39	40.11	11.94	2.97
9	4.06	14.91	5.12	17.77	4.44	0.19
10	1.83	8.88	2.52	10.73	2.08	0.00
11	1.31	7.46	1.91	9.08	1.52	0.00
12	0.85	6.22	1.37	7.63	1.04	0.00

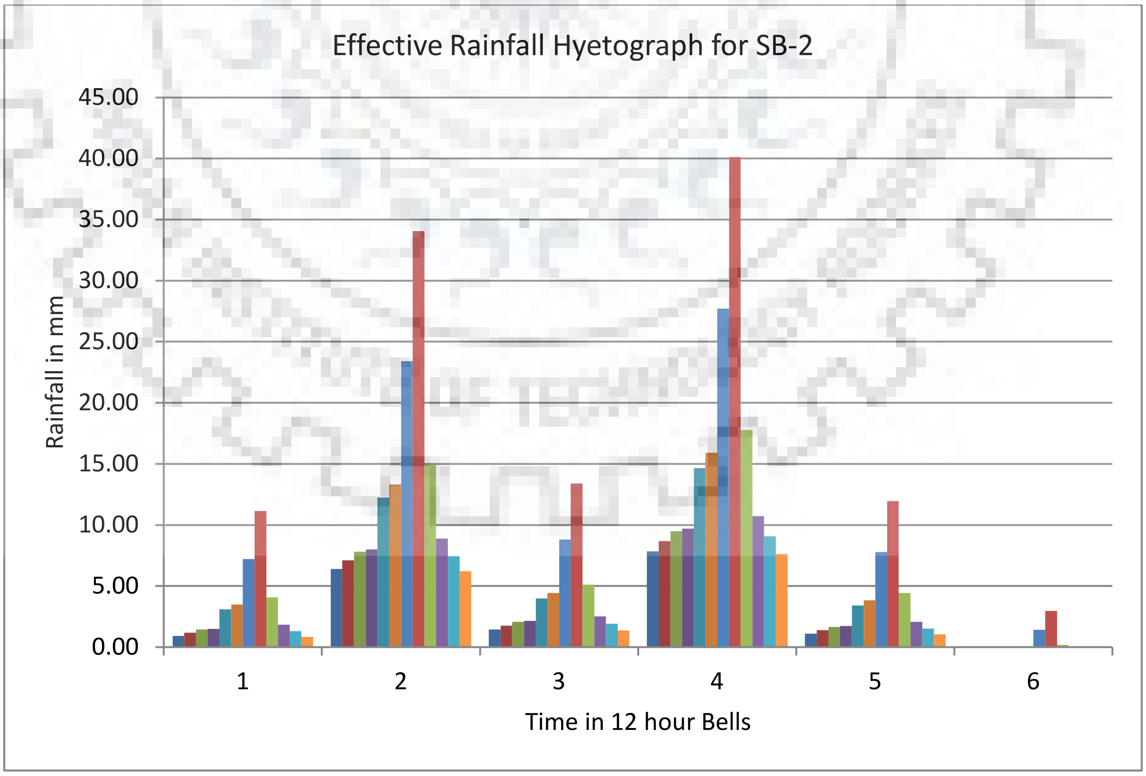

Fig-4.16: Effective Rainfall Hyetograph for SB-2

Table-4.28: Critical sequencing for hourly rainfall of SB3

Time	UH Ordinates of SB-6	Critical Sequencing of hourly Rainfall in mm (12 hour Bell)						Reverse Critical Sequencing of hourly Rainfall in mm (12 hour Bell)					
(hr)	(cumec)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
12	192												
13	220												
14	245												
15	278	3.53	9.53	3.60	9.74	5.56	2.06	3.60	9.73	3.68	9.94	5.68	2.10
16	316.0	4.04	10.92	4.13	11.16	6.37	2.36	3.89	10.52	3.98	10.76	6.14	2.27
17	364	4.63	12.51	4.73	12.79	7.30	2.70	4.19	11.32	4.28	11.57	6.60	2.44
18	404	7.12	19.26	7.28	19.69	11.23	4.16	4.26	11.52	4.35	11.77	6.72	2.48
19	413.9	15.06	40.70	15.39	41.60	23.74	8.78	6.02	16.28	6.15	16.64	9.50	3.51
20	408	10.65	28.79	10.88	29.43	16.79	6.21	6.46	17.47	6.61	17.86	10.19	3.77
21	388	6.46	17.47	6.61	17.86	10.19	3.77	10.65	28.79	10.88	29.43	16.79	6.21
22	366	6.02	16.28	6.15	16.64	9.50	3.51	15.06	40.70	15.39	41.60	23.74	8.78
23	345	4.26	11.52	4.35	11.77	6.72	2.48	7.12	19.26	7.28	19.69	11.23	4.16
24	324	4.19	11.32	4.28	11.57	6.60	2.44	4.63	12.51	4.73	12.79	7.30	2.70
25	305.0	3.89	10.52	3.98	10.76	6.14	2.27	4.04	10.92	4.13	11.16	6.37	2.36
26	288	3.60	9.73	3.68	9.94	5.68	2.10	3.53	9.53	3.60	9.74	5.56	2.06
27	272												
28	257												
29	245												

Table-4.29: Effective Rainfall for SB-3

Time(hour)	Effective Rainfall in mm (12 hour Bell) after deduction of Losses(2.3cm/hour)					
1	1.30	7.43	1.38	7.64	3.38	0.00
2	1.59	8.22	1.68	8.46	3.84	0.00
3	1.89	9.02	1.98	9.27	4.30	0.14
4	1.96	9.22	2.05	9.47	4.42	0.18
5	3.72	13.98	3.85	14.34	7.20	1.21
6	4.16	15.17	4.31	15.56	7.89	1.47
7	8.35	26.49	8.58	27.13	14.49	3.91
8	12.76	38.40	13.09	39.30	21.44	6.48
9	4.82	16.96	4.98	17.39	8.93	1.86
10	2.33	10.21	2.43	10.49	5.00	0.40
11	1.74	8.62	1.83	8.86	4.07	0.06
12	1.23	7.23	1.30	7.44	3.26	0.00

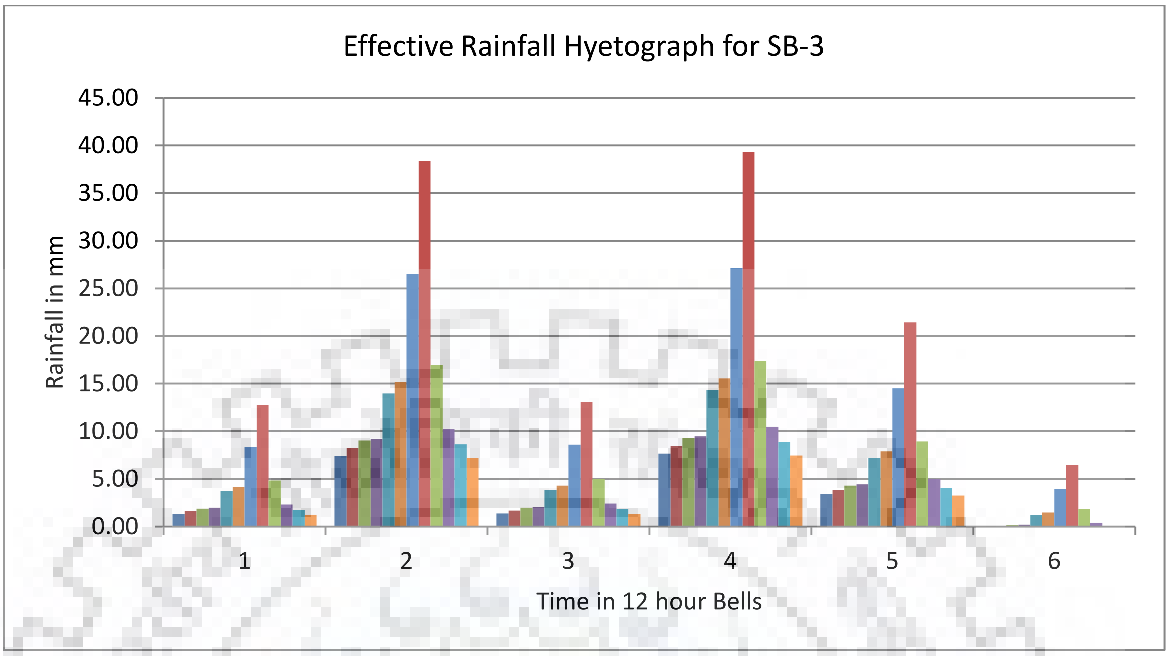

Fig-4.17: Effective Rainfall Hyetograph for SB-3

Table-4.30: Critical sequencing for hourly rainfall of SB4

Time	UH Ordinates of SB-6	Critical Sequencing of hourly Rainfall in mm (12 hour Bell)						Reverse Critical Sequencing of hourly Rainfall in mm (12 hour Bell)					
(hr)	(cumec)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
12	192												
13	220												
14	245												
15	278	3.60	9.74	4.29	11.60	9.03	3.34	3.68	9.94	4.38	11.84	9.22	3.41
16	316.0	4.13	11.16	4.92	13.29	10.35	3.83	3.98	10.76	4.74	12.81	9.97	3.69
17	364	4.73	12.79	5.63	15.22	11.86	4.39	4.28	11.57	5.09	13.77	10.73	3.97
18	404	7.28	19.69	8.67	23.44	18.26	6.75	4.35	11.77	5.18	14.01	10.92	4.04
19	413.9	15.39	41.60	18.32	49.53	38.58	14.27	6.15	16.64	7.33	19.81	15.43	5.71
20	408	10.88	29.43	12.96	35.04	27.29	10.09	6.61	17.86	7.86	21.26	16.56	6.13
21	388	6.61	17.86	7.86	21.26	16.56	6.13	10.88	29.43	12.96	35.04	27.29	10.09
22	366	6.15	16.64	7.33	19.81	15.43	5.71	15.39	41.60	18.32	49.53	38.58	14.27
23	345	4.35	11.77	5.18	14.01	10.92	4.04	7.28	19.69	8.67	23.44	18.26	6.75
24	324	4.28	11.57	5.09	13.77	10.73	3.97	4.73	12.79	5.63	15.22	11.86	4.39
25	305.0	3.98	10.76	4.74	12.81	9.97	3.69	4.13	11.16	4.92	13.29	10.35	3.83
26	288	3.68	9.94	4.38	11.84	9.22	3.41	3.60	9.74	4.29	11.60	9.03	3.34
27	272												
28	257												
29	245												

Table-4.31: Effective Rainfall for SB-4

Time(hour)	Effective Rainfall in mm (12 hour Bell) after deduction of Losses(2.3cm/hour)					
1	1.38	7.64	2.08	9.54	6.92	1.11
2	1.68	8.46	2.44	10.51	7.67	1.39
3	1.98	9.27	2.79	11.47	8.43	1.67
4	2.05	9.47	2.88	11.71	8.62	1.74
5	3.85	14.34	5.03	17.51	13.13	3.41
6	4.31	15.56	5.56	18.96	14.26	3.83
7	8.58	27.13	10.66	32.74	24.99	7.79
8	13.09	39.30	16.02	47.23	36.28	11.97
9	4.98	17.39	6.37	21.14	15.96	4.45
10	2.43	10.49	3.33	12.92	9.56	2.09
11	1.83	8.86	2.62	10.99	8.05	1.53
12	1.30	7.44	1.99	9.30	6.73	1.04

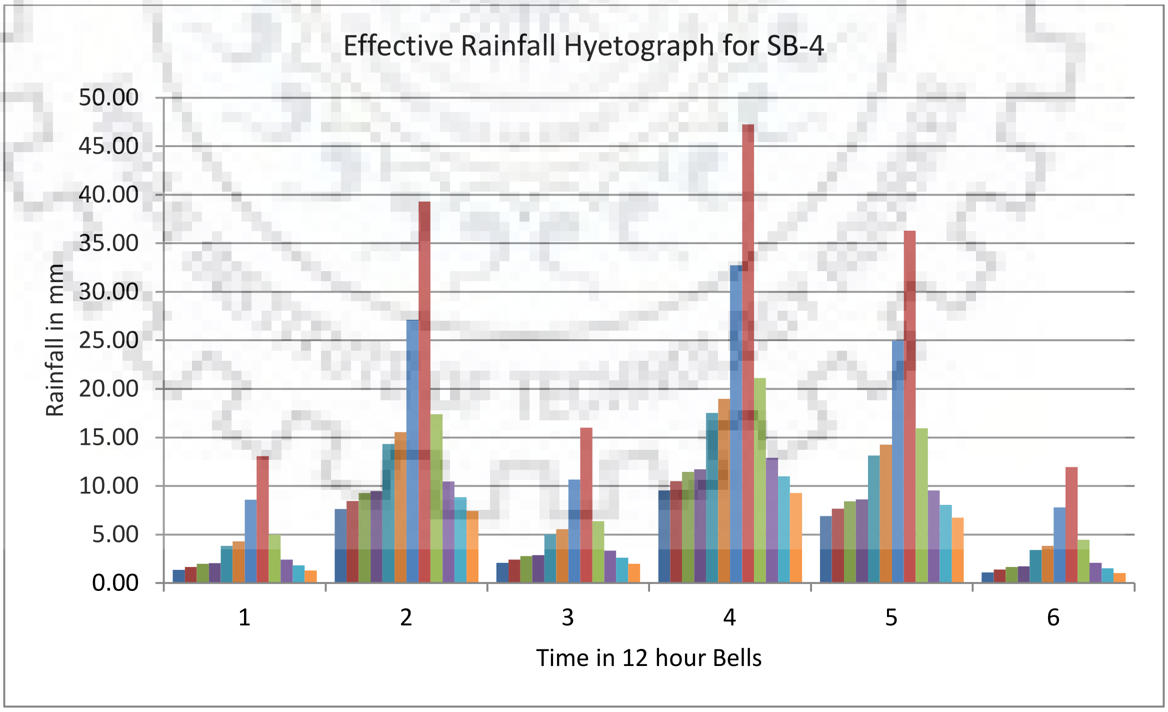

Fig-4.18: Effective Rainfall Hyetograph for SB-4

Table-4.32: Critical sequencing for hourly rainfall of SB5

Time	UH Ordinates of SB-6	Critical Sequencing of hourly Rainfall in mm (12 hour Bell)						Reverse Critical Sequencing of hourly Rainfall in mm (12 hour Bell)					
(hr)	(cumec)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
12	192												
13	220												
14	245												
15	278	3.65	9.88	3.94	10.65	3.85	1.43	3.73	10.09	4.02	10.87	3.93	1.46
16	316.0	4.19	11.32	4.51	12.21	4.42	1.63	4.04	10.91	4.35	11.76	4.26	1.57
17	364	4.80	12.97	5.17	13.98	5.06	1.87	4.34	11.73	4.68	12.65	4.58	1.69
18	404	7.39	19.97	7.96	21.53	7.79	2.88	4.42	11.94	4.76	12.87	4.66	1.72
19	413.9	15.61	42.20	16.83	45.49	16.46	6.09	6.24	16.88	6.73	18.20	6.58	2.44
20	408	11.04	29.85	11.90	32.18	11.64	4.31	6.70	18.12	7.22	19.53	7.07	2.61
21	388	6.70	18.12	7.22	19.53	7.07	2.61	11.04	29.85	11.90	32.18	11.64	4.31
22	366	6.24	16.88	6.73	18.20	6.58	2.44	15.61	42.20	16.83	45.49	16.46	6.09
23	345	4.42	11.94	4.76	12.87	4.66	1.72	7.39	19.97	7.96	21.53	7.79	2.88
24	324	4.34	11.73	4.68	12.65	4.58	1.69	4.80	12.97	5.17	13.98	5.06	1.87
25	305.0	4.04	10.91	4.35	11.76	4.26	1.57	4.19	11.32	4.51	12.21	4.42	1.63
26	288	3.73	10.09	4.02	10.87	3.93	1.46	3.65	9.88	3.94	10.65	3.85	1.43
27	272												
28	257												
29	245												

Table-4.33: Effective Rainfall for SB-5

Time(hour)	Effective Rainfall in mm (12 hour Bell) after deduction of Losses(2.3cm/hour)					
1	1.43	7.79	1.72	8.57	1.63	0.00
2	1.74	8.61	2.05	9.46	1.96	0.00
3	2.04	9.43	2.38	10.35	2.28	0.00
4	2.12	9.64	2.46	10.57	2.36	0.00
5	3.94	14.58	4.43	15.90	4.28	0.14
6	4.40	15.82	4.92	17.23	4.77	0.31
7	8.74	27.55	9.60	29.88	9.34	2.01
8	13.31	39.90	14.53	43.19	14.16	3.79
9	5.09	17.67	5.66	19.23	5.49	0.58
10	2.50	10.67	2.87	11.68	2.76	0.00
11	1.89	9.02	2.21	9.91	2.12	0.00
12	1.35	7.58	1.64	8.35	1.55	0.00

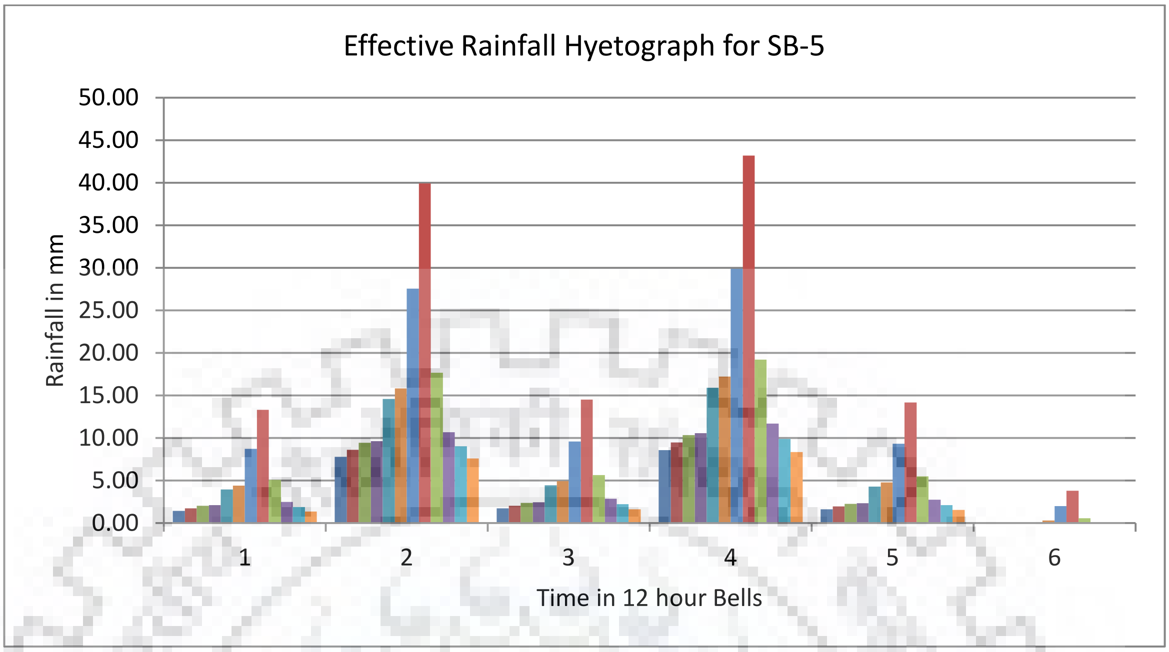

Fig-4.19: Effective Rainfall Hyetograph for SB-5

Table-4.34: Critical sequencing for hourly rainfall of SB6

Time	UH Ordinates of SB-6	Critical Sequencing of hourly Rainfall in mm (12 hour Bell)						Reverse Critical Sequencing of hourly Rainfall in mm (12 hour Bell)					
(hr)	(cumec)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
12	192												
13	220												
14	245												
15	278	3.15	8.51	4.42	11.95	6.34	2.35	3.21	8.69	4.51	12.20	6.47	2.39
16	316.0	3.61	9.76	5.06	13.69	7.27	2.69	3.48	9.40	4.88	13.19	7.00	2.59
17	364	4.13	11.18	5.80	15.68	8.32	3.08	3.74	10.11	5.25	14.19	7.53	2.79
18	404	6.36	17.21	8.93	24.15	12.81	4.74	3.81	10.29	5.34	14.44	7.66	2.83
19	413.9	13.45	36.36	18.87	51.03	27.08	10.02	5.38	14.55	7.55	20.41	10.83	4.01
20	408	9.51	25.72	13.35	36.09	19.16	7.08	5.77	15.61	8.10	21.91	11.63	4.30
21	388	5.77	15.61	8.10	21.91	11.63	4.30	9.51	25.72	13.35	36.09	19.16	7.08
22	366	5.38	14.55	7.55	20.41	10.83	4.01	13.45	36.36	18.87	51.03	27.08	10.02
23	345	3.81	10.29	5.34	14.44	7.66	2.83	6.36	17.21	8.93	24.15	12.81	4.74
24	324	3.74	10.11	5.25	14.19	7.53	2.79	4.13	11.18	5.80	15.68	8.32	3.08
25	305.0	3.48	9.40	4.88	13.19	7.00	2.59	3.61	9.76	5.06	13.69	7.27	2.69
26	288	3.21	8.69	4.51	12.20	6.47	2.39	3.15	8.51	4.42	11.95	6.34	2.35
27	272												
28	257												
29	245												

Table-4.35: Effective Rainfall for SB-6

Time(hour)	Effective Rainfall in mm (12 hour Bell) after deduction of Losses(2.3cm/hour)					
1	0.91	6.39	2.21	9.90	4.17	0.09
2	1.18	7.10	2.58	10.89	4.70	0.29
3	1.44	7.81	2.95	11.89	5.23	0.49
4	1.51	7.99	3.04	12.14	5.36	0.53
5	3.08	12.25	5.25	18.11	8.53	1.71
6	3.47	13.31	5.80	19.61	9.33	2.00
7	7.21	23.42	11.05	33.79	16.86	4.78
8	11.15	34.06	16.57	48.73	24.78	7.72
9	4.06	14.91	6.63	21.85	10.51	2.44
10	1.83	8.88	3.50	13.38	6.02	0.78
11	1.31	7.46	2.76	11.39	4.97	0.39
12	0.85	6.21	2.12	9.65	4.04	0.05

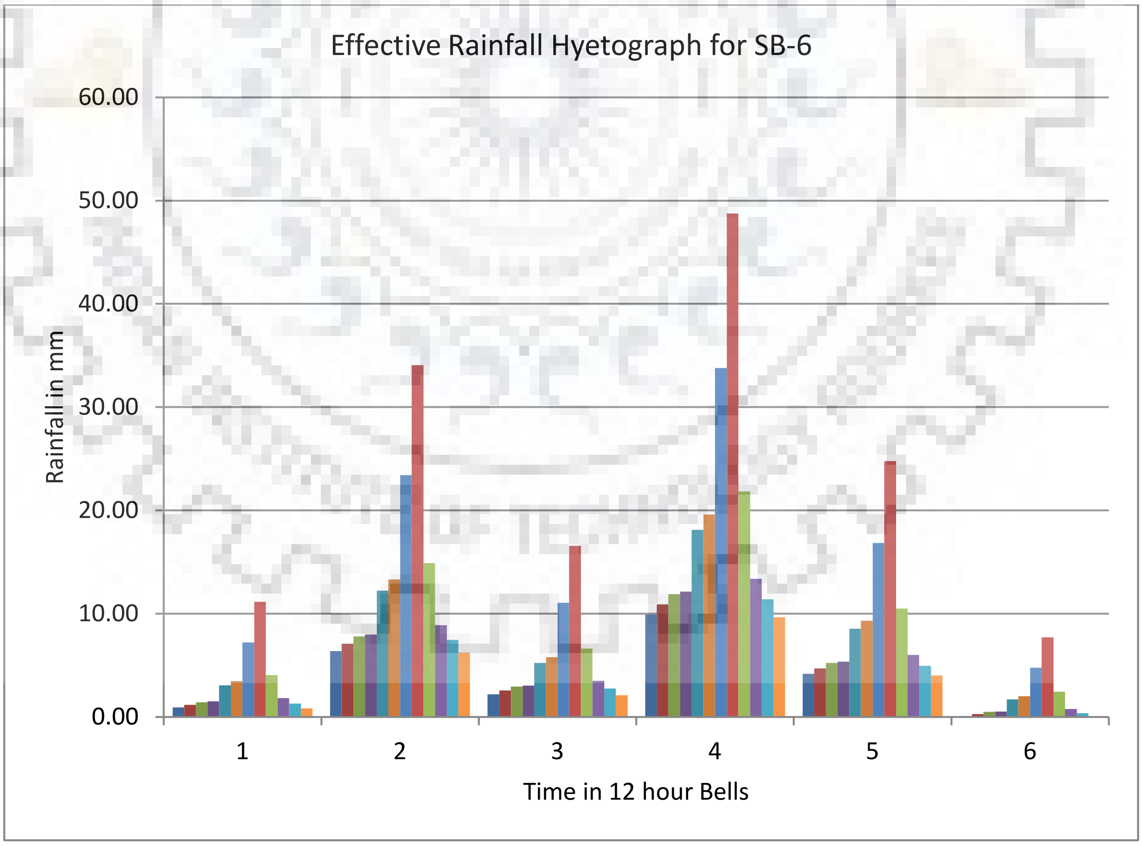

Fig-4.20: Effective Rainfall Hyetograph for SB-6

Table-4.36: Critical sequencing for hourly rainfall of SB7

Time	UH Ordinates of SB-6	Critical Sequencing of hourly Rainfall in mm (12 hour Bell)						Reverse Critical Sequencing of hourly Rainfall in mm (12 hour Bell)					
(hr)	(cumec)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)	(mm)
12	192												
13	220												
14	245												
15	278	2.09	5.64	2.99	8.09	4.61	1.71	2.13	5.76	3.06	8.26	4.71	1.74
16	316.0	2.39	6.46	3.43	9.27	5.29	1.96	2.30	6.23	3.31	8.94	5.09	1.88
17	364	2.74	7.40	3.93	10.62	6.06	2.24	2.48	6.70	3.56	9.61	5.48	2.03
18	404	4.22	11.40	6.05	16.36	9.32	3.45	2.52	6.82	3.62	9.78	5.58	2.06
19	413.9	8.91	24.09	12.79	34.57	19.71	7.29	3.56	9.64	5.11	13.83	7.88	2.92
20	408	6.30	17.04	9.04	24.45	13.94	5.16	3.83	10.34	5.49	14.84	8.46	3.13
21	388	3.83	10.34	5.49	14.84	8.46	3.13	6.30	17.04	9.04	24.45	13.94	5.16
22	366	3.56	9.64	5.11	13.83	7.88	2.92	8.91	24.09	12.79	34.57	19.71	7.29
23	345	2.52	6.82	3.62	9.78	5.58	2.06	4.22	11.40	6.05	16.36	9.32	3.45
24	324	2.48	6.70	3.56	9.61	5.48	2.03	2.74	7.40	3.93	10.62	6.06	2.24
25	305.0	2.30	6.23	3.31	8.94	5.09	1.88	2.39	6.46	3.43	9.27	5.29	1.96
26	288	2.13	5.76	3.06	8.26	4.71	1.74	2.09	5.64	2.99	8.09	4.61	1.71
27	272												
28	257												
29	245												

Table-4.37: Effective Rainfall for SB-7

Time(hour)	Effective Rainfall in mm (12 hour Bell) after deduction of Losses(2.3cm/hour)					
1	0.00	3.46	0.76	5.96	2.41	0.00
2	0.00	3.93	1.01	6.64	2.79	0.00
3	0.18	4.40	1.26	7.31	3.18	0.00
4	0.22	4.52	1.32	7.48	3.28	0.00
5	1.26	7.34	2.81	11.53	5.58	0.62
6	1.53	8.04	3.19	12.54	6.16	0.83
7	4.00	14.74	6.74	22.15	11.64	2.86
8	6.61	21.79	10.49	32.27	17.41	4.99
9	1.92	9.10	3.75	14.06	7.02	1.15
10	0.44	5.10	1.63	8.32	3.76	0.00
11	0.09	4.16	1.13	6.97	2.99	0.00
12	0.00	3.34	0.69	5.79	2.31	0.00

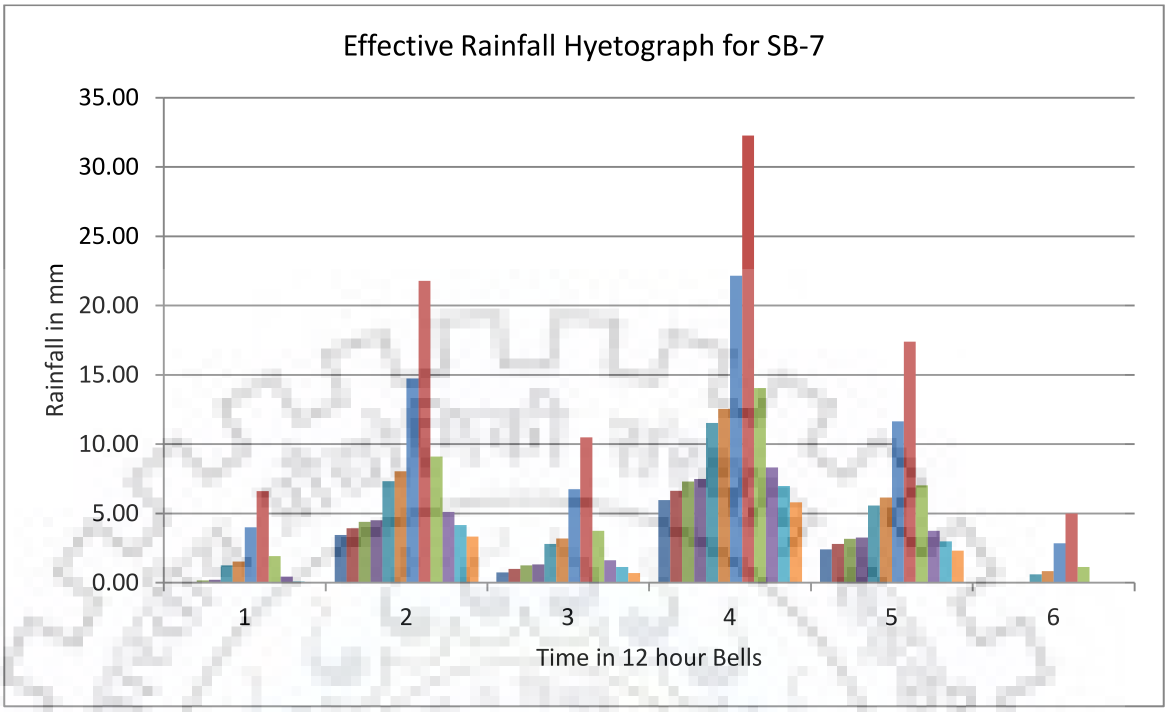

Fig-4.21: Effective Rainfall Hyetograph for SB-7

CHAPTER 5: HEC HMS MODELLING FOR DESIGN FLOOD

5.1 HEC-HMS model set up for Design Flood study of Matatila dam

The design flood computations for Matatila dam have been carried out using HEC-HMS where convolution, reservoir routing and channel routing performed in an integrated manner. The hydrological schematic set up used for design flood computations is presented in Fig-5.1.

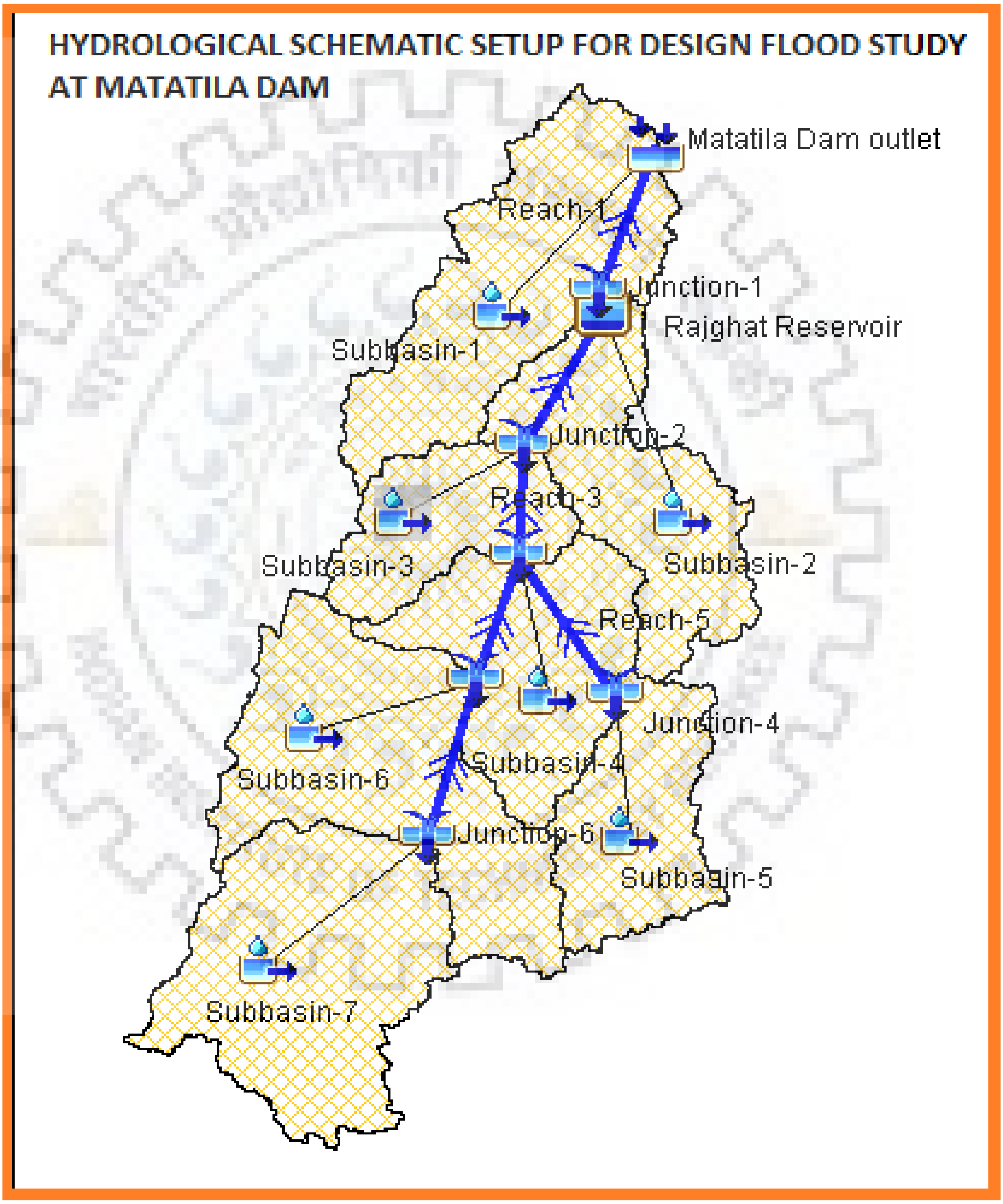

Fig-5.1: Hydrological Schematic set up in HEC-HMS

5.2 Definition of elements in the HEC-HMS model setup

Under the basin (Betwa Basin) there are 7 sub basins, Sub Basin-1 to Sub Basin-7. Connecting the sub basins there are 6 junctions J1 to J6 and 6 reaches R1 to R6, which are defined below. Two other elements one outlet representing the spillway of Matatila dam and a reservoir representing Rajghat dam are also included in the model.

J1 – Junction of sub basin SB-1 and SB-2

J2– Junction of sub basin SB-2 and SB-3

J3– Junction of sub basin SB-3 and SB-4

J4– Junction of sub basin SB-4 and SB-5

J5– Junction of sub basin SB-4 and SB-6

J6 – Junction of sub basin SB-6 and SB-7

R1 – Reach for channel routing connecting from J1 to Matatila dam outlet

R2 – Reach for channel routing connecting from J2 to J1

R3 – Reach for channel routing connecting from J3 to J2

R4– Reach for channel routing connecting from J5 to J3

R5– Reach for channel routing connecting from J4 to J3

R6– Reach for channel routing connecting from J6 to J5

5.3 Model input and parameters

The hourly effective rainfall of all the seven sub basins viz SB1, SB2, SB3, SB-4, SB-5, SB-6, & SB-7 as given in Table-4.25, 4.27, 4.29, 4.31, 4.33, 4.35, and 4.37 have been convoluted with Unit Hydrographs of sub basins from SB-1 to SB-7 as given in Table-4.3 respectively. Using the sub basin components, the transformation method and base flow etc are included. The other required input parameters are also incorporated in the model

The flood hydrograph of sub basin SB-7 has been generated and collected at Junction-6 which is junction point of SB-7 and SB-6. The generated flood has been channel routed from J-6 through reach R-6 to get its response at J-5. The floods generated from SB-6 also gets collected at J-5. The flood from SB-5 has been collected at J-4. Floods from Junction J-5 has been channel routed through Reach R-4 and floods from J-4 has been channel routed through R-5 joined at junction J-4, which also receives the flood from SB4

The Rajghat reservoir has been represented by its elevation-area-capacity and capacity-discharge table. Junction J-1 has been added to get the reservoir routed flood hydrograph just downstream of Rajghat dam.

5.4 Initial condition at Rajghat Reservoir

The initial condition for reservoir routing at Rajghat reservoir has been assumed as FRL (EL 371 m).This is by considering the worst scenario which will give the maximum Design Flood. It is also assumed that all gates of the Rajghat reservoir would be operated as the design flood passes the Rajghat reservoir. The reservoir routed flood hydrograph of Rajghat dam received at Junction J1 has been channel routed through routing reach R1 representing the Betwa river between Rajghat Dam and Matatila dam.

The flood hydrograph obtained thus at the outlet of Matatila Dam is the Inflow Flood Hydrograph (PMF) for Matatila Dam, which is the revised designed flood hydrograph. The revised design flood hydrograph, so obtained may be compared vis a vis with the current design flood of Matatila dam. The adequacy of the Matatila spillway to handle the revised flood may be judged to assess the safety of the Matatila Dam.

5.5 Channel Routing and Reservoir Routing

It may be noted that the reservoir routing has been carried out in HEC-HMS using Modified Puls method. For Channel routing Muskingum method has been adopted. Muskingum channel reach parameter K represent the travel time of flood wave through the routing reach. Hence for the river reaches where higher discharge will pass, average velocity will be higher in comparison to river reaches with lower discharge. Considering the same, the value of Muskingum K has been assigned for different river reaches. Further, for stability of Muskingum routing algorithm, it is necessary that the time interval of inflow hydrograph Δt should be greater than or equal to 2KX, hence routing reaches have been divided into sub reaches to ensure the stability of the routing algorithm. The Muskingum routing parameters viz. K, X and number of sub reaches are given in Table-5.1. The wave velocity of flood is assumed as 8 km/hour in all reaches.

Table-5.1: Muskingum channel routing parameters

Routing Reach	Length (km)	Muskingum K (hour)	Muskingum X	Number of sub reaches
R1	61.7	7.8	0.2	4
R2	67.4	8.5	0.2	4
R3	27.4	3.5	0.2	2
R4	39.8	5	0.2	2
R5	51.2	6.5	0.2	3
R6	45.0	5.7	0.2	3

The ELEVATION-AREA-CAPACITY curve of Rajghat dam used in HEC-HMS model setup to represent the Rajghat reservoir is given in Table-5.2. The elevation-storage and discharge capacity of spillway of Rajghat dam with all 18 gates is given in Table-5.3.

Table-5.2: Elevation – Area – Capacity of Rajghat reservoir

Elevation	Area	Capacity/Storage
(m)	(ha)	(MCM)
357.0 (Spillway crest)	5700	250
358.0	6850	300
359.0	7900	400
360.0	9250	460
361.0	10600	550
362.0	11950	690
363.0	13100	800
364.0	14350	950
365.0	15500	1100
366.0	16600	1250
367.0	17750	1450
368.0	18800	1600
369.0	19950	1800
370.0	21000	2000
370.2	21190	2030
370.4	21380	2060

370.6	21570	2100
370.8	21760	2150
371.0 (FRL)	21950	2200
371.2	22160	2240
371.4	22370	2280
371.6	22580	2324
371.8	22790	2372
372.0	23000	2420
372.2	23210	2488
372.4	23420	2556
372.6	23630	2612
372.8	23840	2656
373.0 (HFL)	24050	2700
374.0	25000	3000

Table-5.3: Elevation – Storage – Discharge for Rajghat Dam Spillway

Elevation	Storage	Discharge
(m)	(MCM)	(cumec)
357	250	0
365	1100	11368
365.5	1175	12468
366	1250	13600
366.5	1330	14702
367	1450	15959
367.5	1525	17189
368	1600	18440
368.5	1700	19860
369	1800	21035
369.5	1875	22373
370	2000	23742
370.5	2075	25135
371 (FRL)	2200	26555
371.5	2300	27999
372	2420	29470
372.5	2590	30602
373 (HFL)	2700	31727
374	3000	**33893 (spillway capacity)**

5.6 Results of the HEC-HMS Model Run

5.6.1 Results of the Model

The global summary of results of the model run has been shown in Fig-5.2

Global Summary Results for Run "FINAL 05092018"

Project: 21-05-2018 Simulation Run: FINAL 05092018

Start of Run: 01Jan2000, 00:00 Basin Model: Betwa Basin
End of Run: 08Jan2000, 00:00 Meteorologic Model: Met 2
Compute Time:05Sep2018, 14:41:39 Control Specifications:Control 1

Show Elements: All Elements Volume Units: MM 1000 M3 Sorting: Hydrologic

Hydrologic Element	Drainage Area (KM2)	Peak Discharge (M3/S)	Time of Peak	Volume (MM)
Subbasin-7	3900	7577.6	03Jan2000, 17:00	371.85
Junction-6	3900	7577.6	03Jan2000, 17:00	371.85
Reach-6	3900	7390.0	03Jan2000, 23:00	371.85
Subbasin-6	3882	13156.9	03Jan2000, 14:00	610.21
Junction-5	7782	19139.1	03Jan2000, 17:00	490.75
Reach-4	7782	18981.4	04Jan2000, 00:00	490.75
Subbasin-5	1959	6563.8	03Jan2000, 08:00	546.35
Junction-4	1959	6563.8	03Jan2000, 08:00	546.35
Reach-5	1959	6199.2	03Jan2000, 14:00	546.35
Subbasin-4	2639	9810.4	03Jan2000, 13:00	712.18
Junction-3	12380	33178.4	03Jan2000, 18:00	546.75
Reach-3	12380	33007.0	03Jan2000, 22:00	546.75
Subbasin-3	1674	5318.3	03Jan2000, 09:00	554.37
Junction-2	14054	37021.3	03Jan2000, 21:00	547.66
Reach-2	14054	36390.5	04Jan2000, 05:00	547.65
Subbasin-2	2735	7522.7	03Jan2000, 12:00	473.34
Rajghat Reservoir	16789	33575.2	04Jan2000, 17:00	535.54
Junction-1	16789	33575.2	04Jan2000, 17:00	535.54
Reach-1	16789	33472.3	05Jan2000, 01:00	535.45
Subbasin-1	3543	5474.8	03Jan2000, 10:00	253.12
Matatila Dam outlet	20332	34191.5	04Jan2000, 21:00	486.25

Fig-5.2: Global Summary of HEC-HMS model run

Simulated hydrograph of Rajghat Reservoir with the Elevation and Storage Curve has been shown in Fig-5.3. The flood hydrographs of Sub Basins 1 to 7 are shown in the Fig-5.4 to Fig-5.10 respectively. Similarly results of Reach 1 to 6 are shown in Fig-5.11 to Fig-5.16 respectively. The flood hydrograph of Matatila dam and Rajghat dam has been shown in Fig-5.17 and Fig-5.18 respectively and the hourly ordinates of Flood hydrograph have been show in Table-5.4 and Table-5.5 respectively.

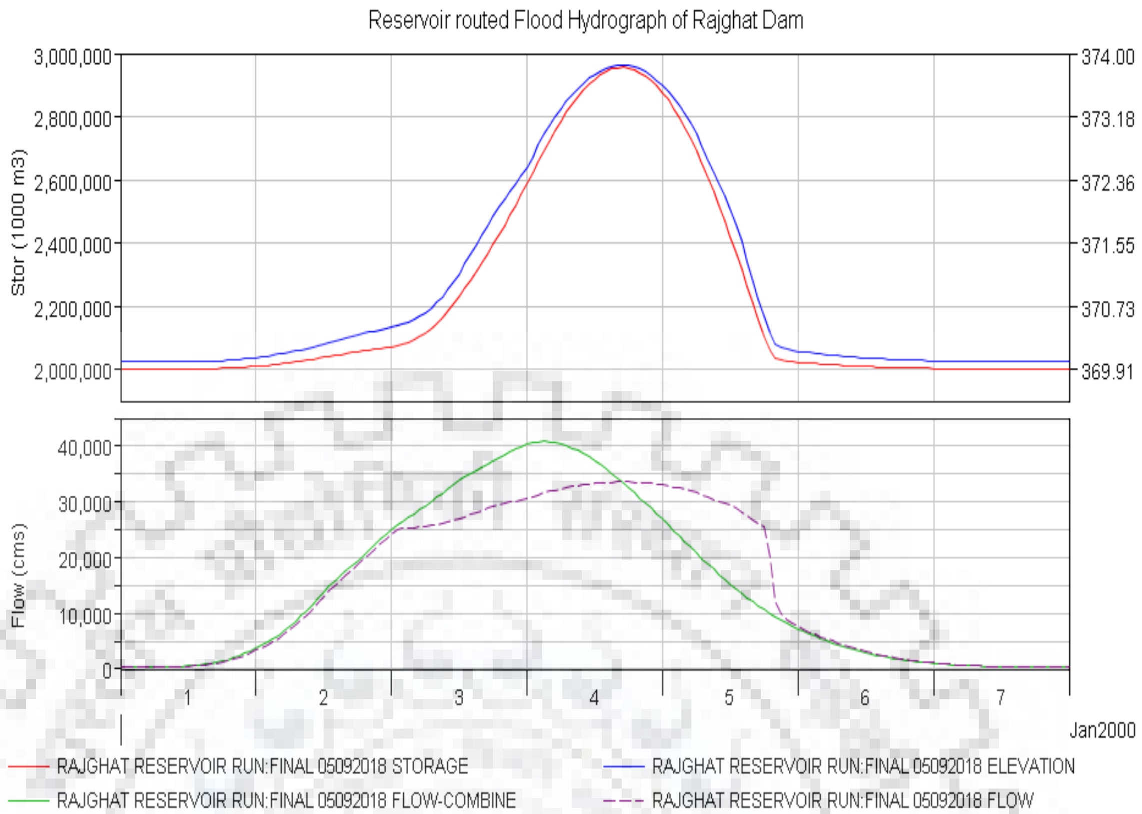

Fig-5.3: Simulated Flood hydrograph at Rajghat reservoir

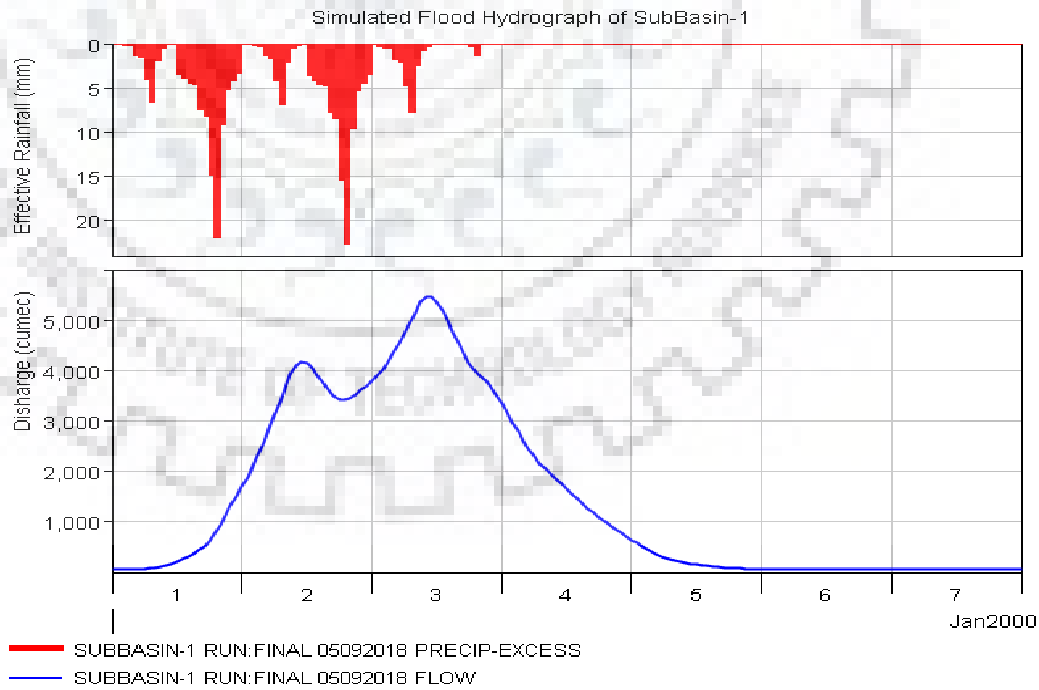

Fig-5.4: Simulated Flood hydrograph of SB-1

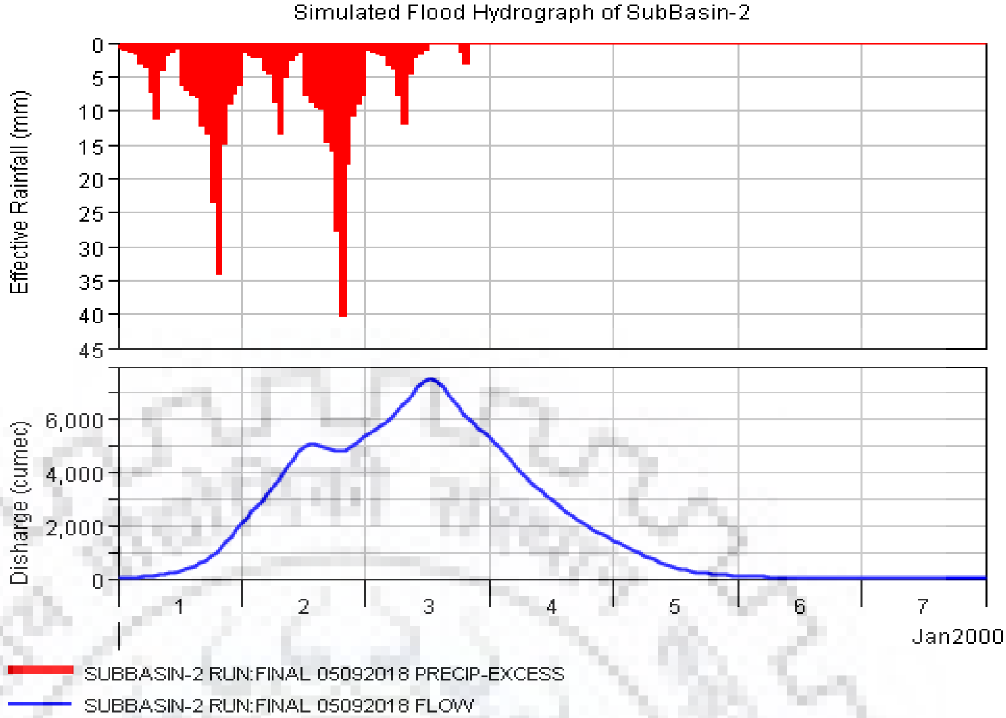

Fig-5.5: Simulated Flood hydrograph of SB-2

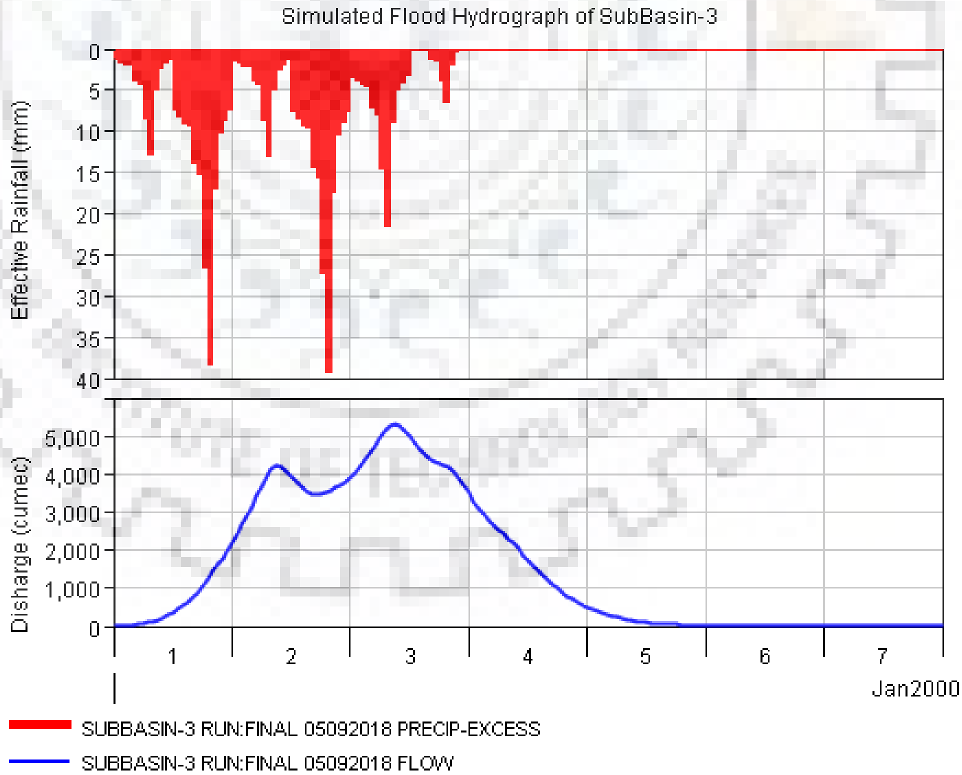

Fig-5.6: Simulated Flood hydrograph of SB-3

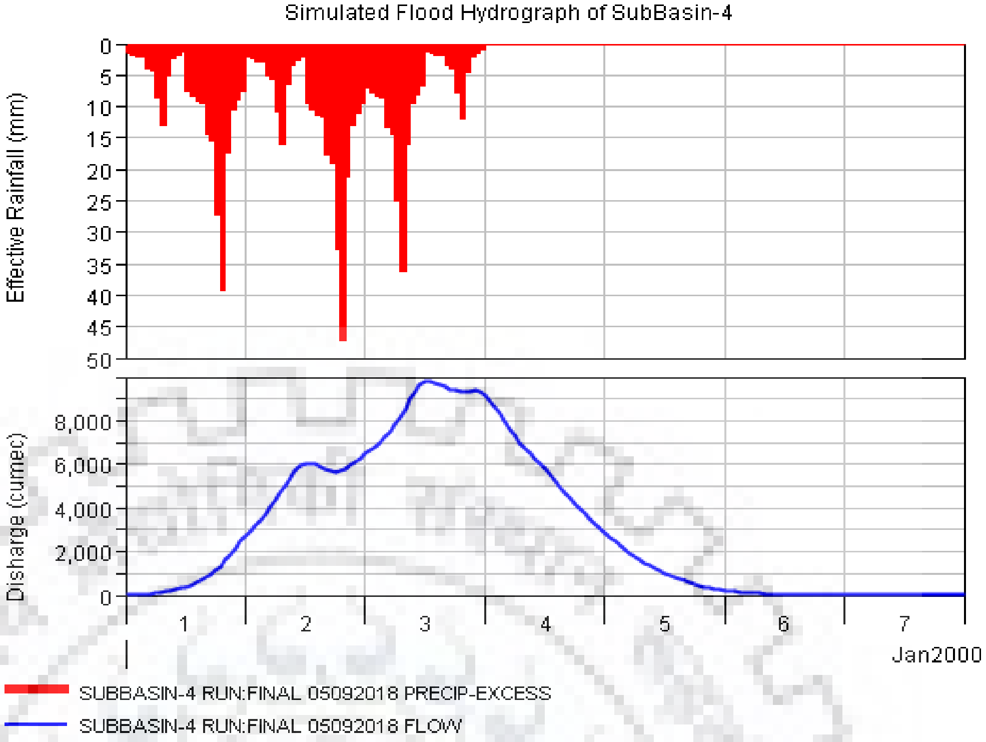

Fig-5.7: Simulated Flood hydrograph of SB-4

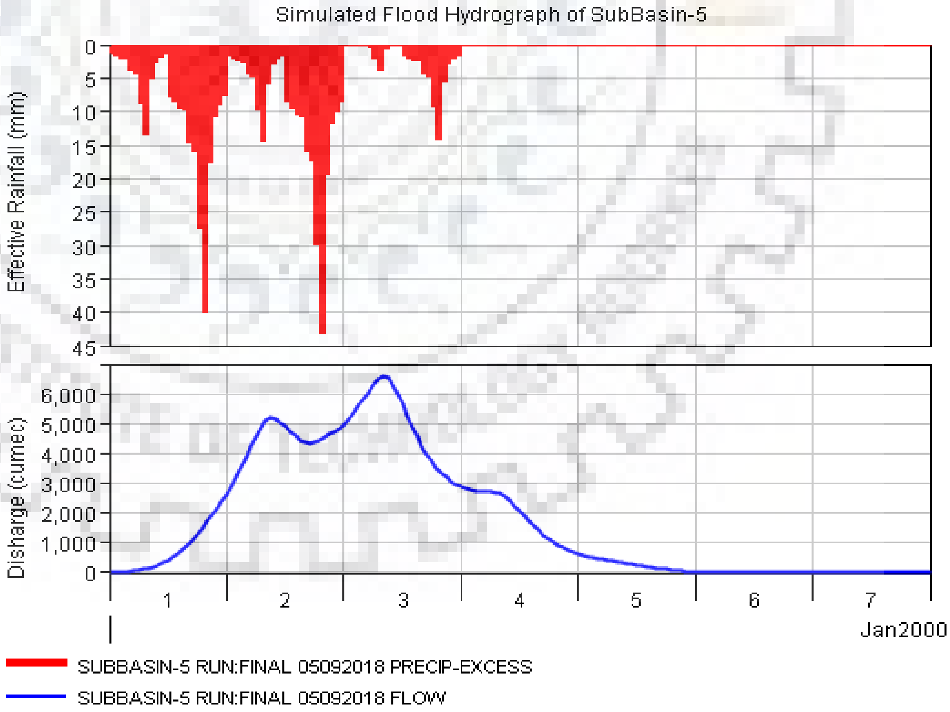

Fig-5.8: Simulated Flood hydrograph of SB-5

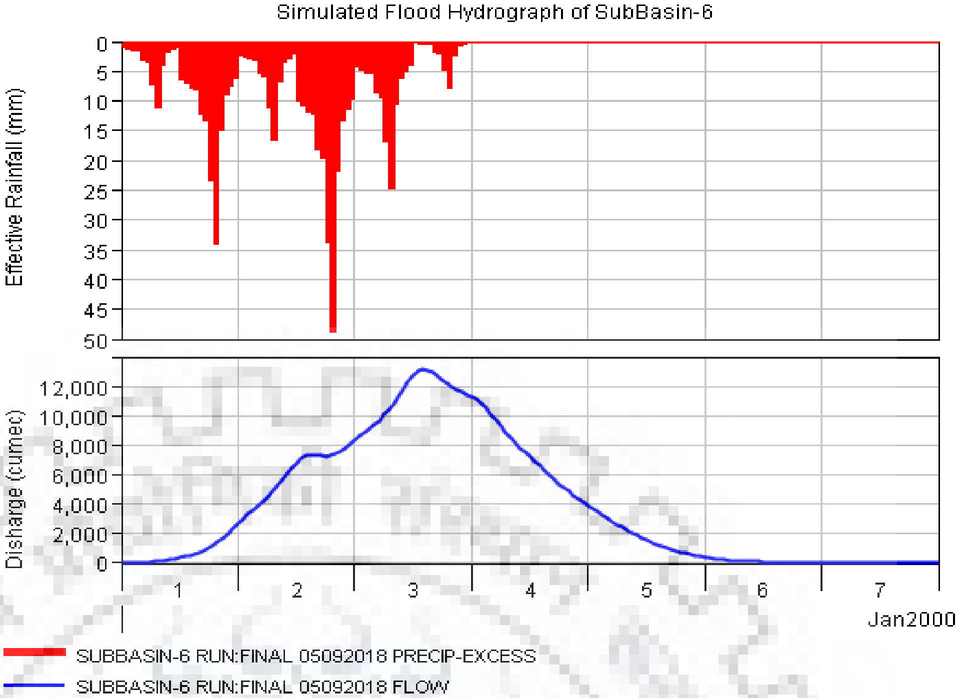

Fig-5.9: Simulated Flood hydrograph of SB-6

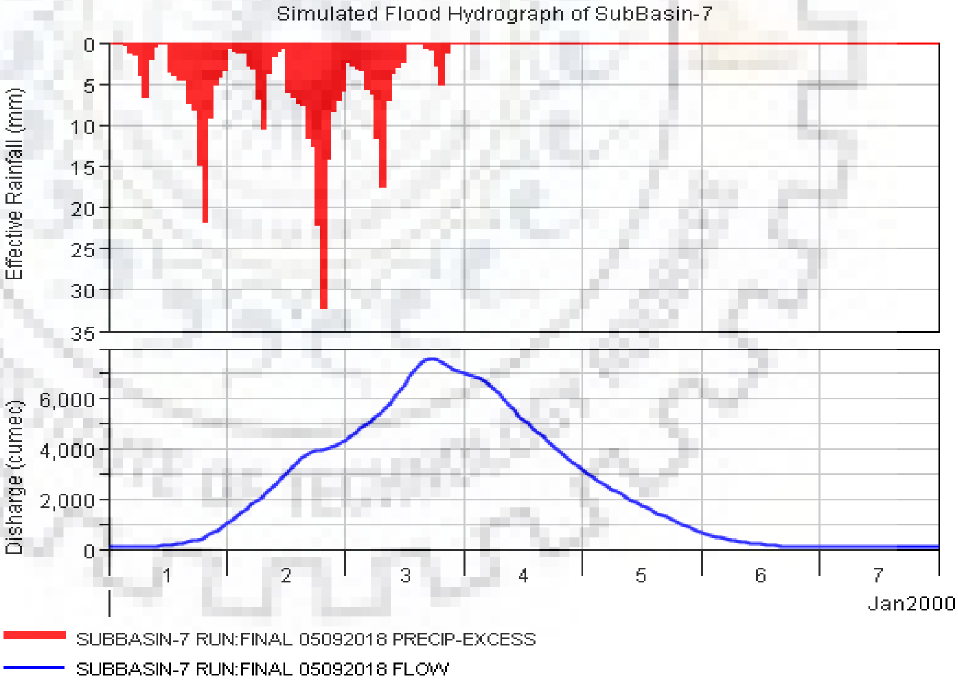

Fig-5.10: Simulated Flood hydrograph of SB-7

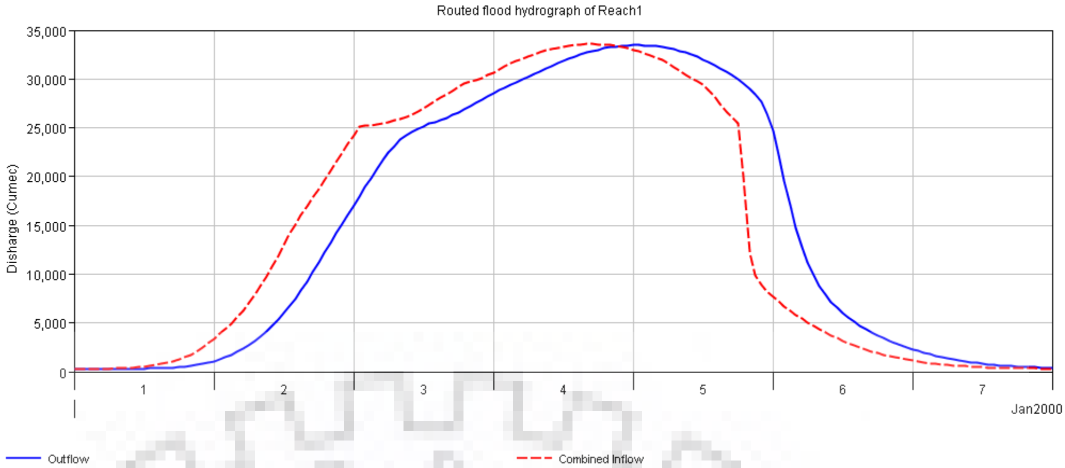

Fig-5.11: Simulated Flood hydrograph of Reach-1

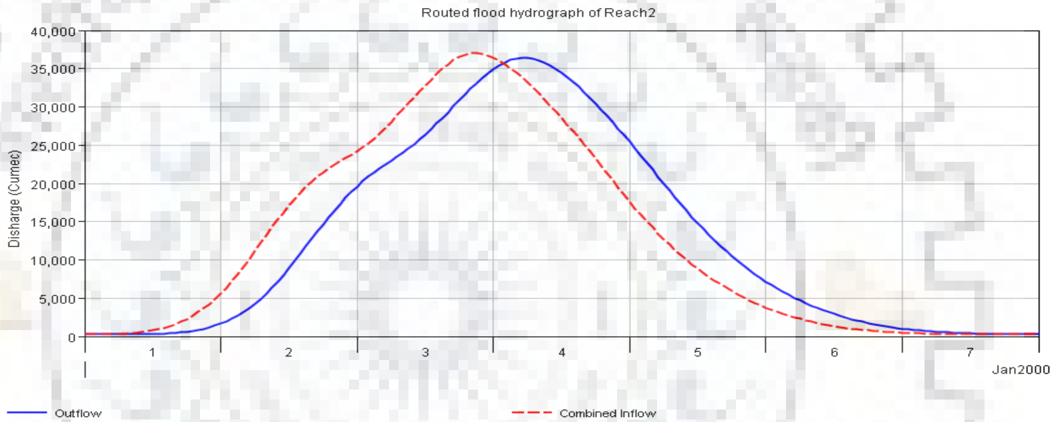

Fig-5.12: Simulated Flood hydrograph of Reach-2

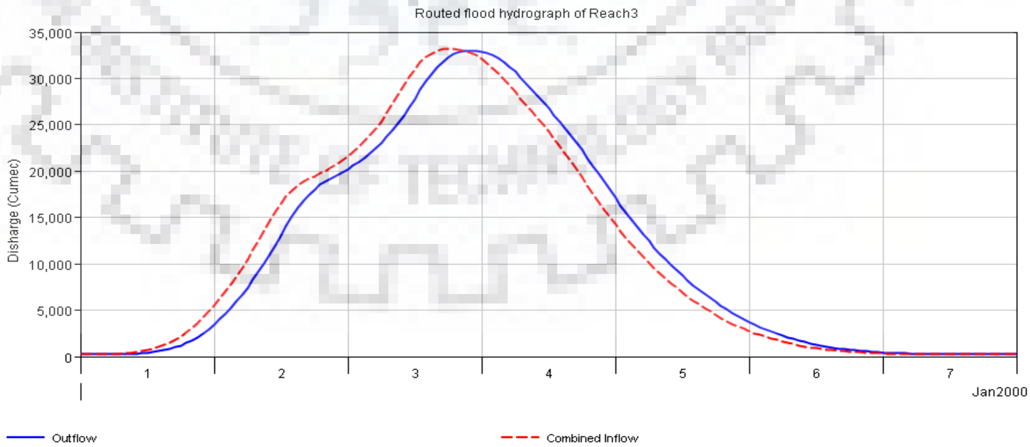

Fig-5.13: Simulated Flood hydrograph of Reach-3

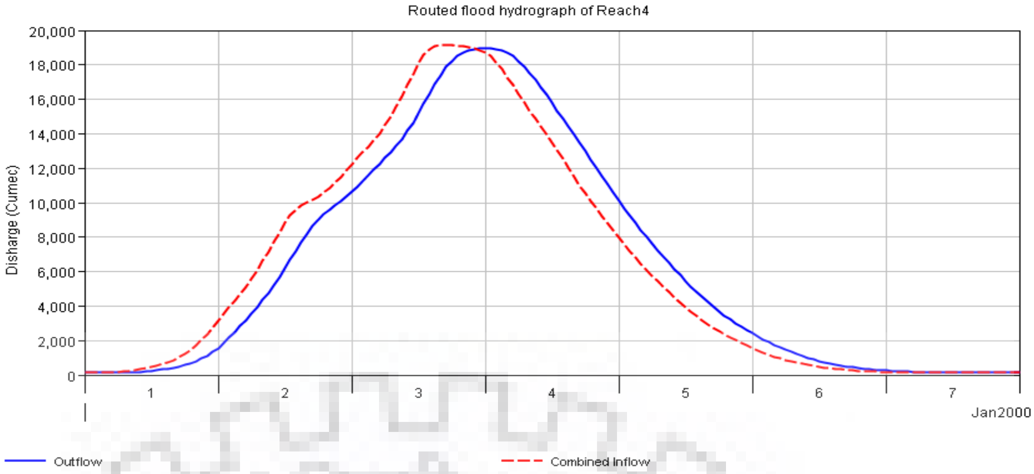

Fig-5.14: Simulated Flood hydrograph of Reach-4

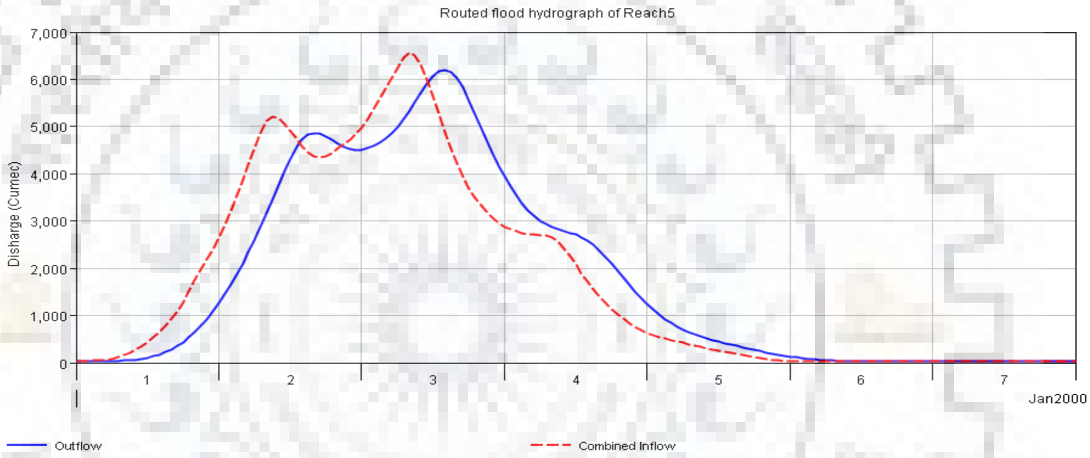

Fig-5.15: Simulated Flood hydrograph of Reach-5

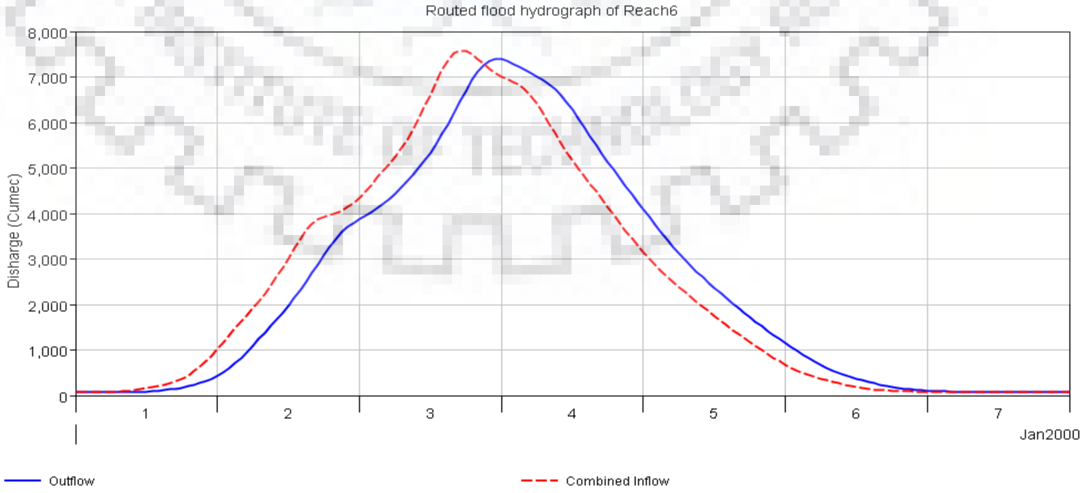

Fig-5.16: Simulated Flood hydrograph of Reach-6

5.6.2 PMF Hydrograph of Matatila dam

Table-5.4: PMF hydrograph of Matatila Dam

Time (Hour)	Discharge (Cumec)	Time (Hour)	Discharge (Cumec)	Time (Hour)	Discharge (Cumec)	Time (Hour)	Discharge (Cumec)
1	366	44	15587	87	33758	130	7959
2	366	45	16625	88	33867	131	7235
3	366	46	17692	89	33958	132	6621
4	366	47	18753	90	34035	133	6088
5	367	48	19789	91	34096	134	5617
6	369	49	20821	92	34144	135	5193
7	372	50	21859	93	34177	136	4807
8	381	51	22919	**94(peak)**	**34192**	137	4451
9	398	52	24004	95	34187	138	4122
10	421	53	25103	96	34162	139	3816
11	450	54	26183	97	34116	140	3531
12	485	55	27195	98	34049	141	3265
13	528	56	28109	99	33962	142	3017
14	583	57	28917	100	33859	143	2785
15	649	58	29593	101	33736	144	2569
16	730	59	30090	102	33594	145	2368
17	828	60	30391	103	33438	146	2181
18	950	61	30545	104	33271	147	2008
19	1100	62	30603	105	33093	148	1847
20	1291	63	30603	106	32898	149	1699
21	1532	64	30590	107	32681	150	1564
22	1813	65	30595	108	32437	151	1439
23	2111	66	30642	109	32160	152	1325
24	2418	67	30736	110	31853	153	1221
25	2751	68	30884	111	31527	154	1127
26	3122	69	31085	112	31190	155	1041
27	3552	70	31313	113	30839	156	964
28	4046	71	31533	114	30462	157	894
29	4601	72	31719	115	30044	158	830
30	5202	73	31872	116	29578	159	773
31	5849	74	31996	117	29058	160	722
32	6544	75	32099	118	28475	161	676
33	7293	76	32191	119	27736	162	636
34	8066	77	32281	120	26559	163	599
35	8804	78	32380	121	24714	164	567
36	9474	79	32497	122	22324	165	539
37	10105	80	32644	123	19697	166	514
38	10733	81	32818	124	17132	167	492
39	11383	82	33003	125	14820	168	473
40	12083	83	33184	126	12848	169	457
41	12853	84	33349	127	11220		
42	13700	85	33501	128	9900		
43	14610	86	33637	129	8831		

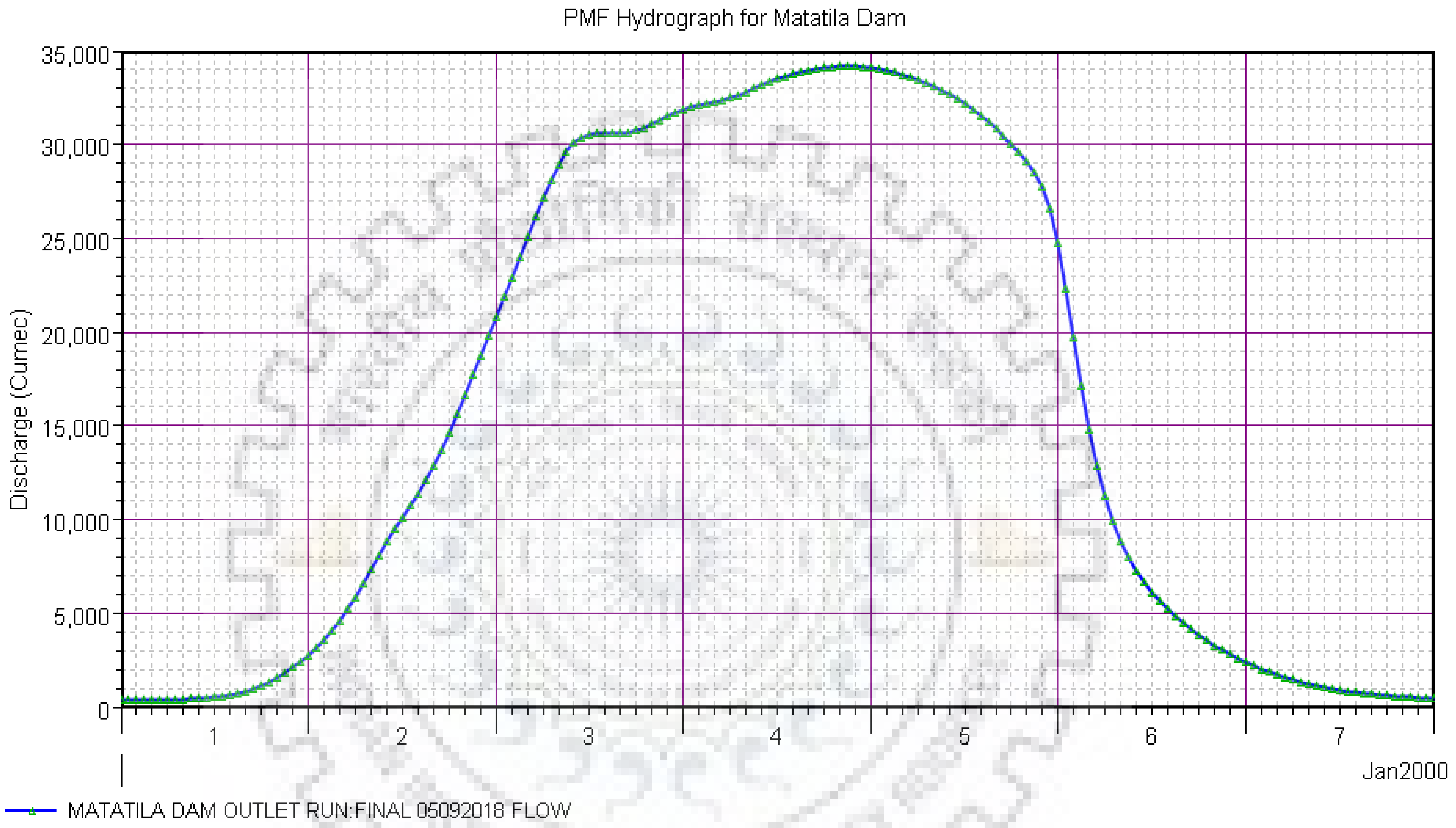

Fig-5.17: PMF Hydrograph for Matatila Dam

5.6.3 PMF Hydrograph of Rajghat Dam

Table-5.5: PMF hydrograph of Rajghat Dam

Time (Hour)	Discharge (Cumec)	Time (Hour)	Discharge (Cumec)	Time (Hour)	Discharge (Cumec)	Time (Hour)	Discharge (Cumec)
1	302	44	20379	87	35945	130	3756
2	303	45	21318	88	35137	131	3472
3	305	46	22267	89	34298	132	3206
4	308	47	23210	90	33432	133	2958
5	313	48	24134	91	32542	134	2725
6	321	49	25004	92	31636	135	2507
7	333	50	25804	93	30713	136	2304
8	352	51	26544	94	29773	137	2116
9	384	52	27237	95	28822	138	1940
10	423	53	27915	96	27857	139	1777
11	469	54	28594	97	26876	140	1627
12	522	55	29285	98	25880	141	1489
13	584	56	30000	99	24874	142	1363
14	661	57	30747	100	23860	143	1248
15	756	58	31522	101	22845	144	1144
16	873	59	32327	102	21831	145	1050
17	1018	60	33153	103	20824	146	964
18	1196	61	33919	104	19831	147	887
19	1412	62	34603	105	18851	148	818
20	1676	63	35216	106	17893	149	755
21	2001	64	35771	107	16963	150	699
22	2378	65	36283	108	16061	151	648
23	2803	66	36790	109	15186	152	603
24	3278	67	37300	110	14343	153	562
25	3779	68	37827	111	13531	154	526
26	4301	69	38363	112	12756	155	494
27	4854	70	38897	113	12021	156	466
28	5451	71	39415	114	11320	157	442
29	6115	72	39895	115	10652	158	421
30	6857	73	40292	116	10016	159	402
31	7674	74	40582	117	9410	160	386
32	8567	75	40760	118	8832	161	373
33	9538	**76(peak)**	**40824**	119	8282	162	361
34	10572	77	40777	120	7757	163	351
35	11661	78	40631	121	7256	164	343
36	12789	79	40394	122	6779	165	336
37	13878	80	40078	123	6323	166	329
38	14908	81	39680	124	5890	167	324
39	15883	82	39209	125	5482	168	320
40	16810	83	38677	126	5094	169	316
41	17699	84	38084	127	4729		
42	18583	85	37426	128	4384		
43	19471	86	36711	129	4060		

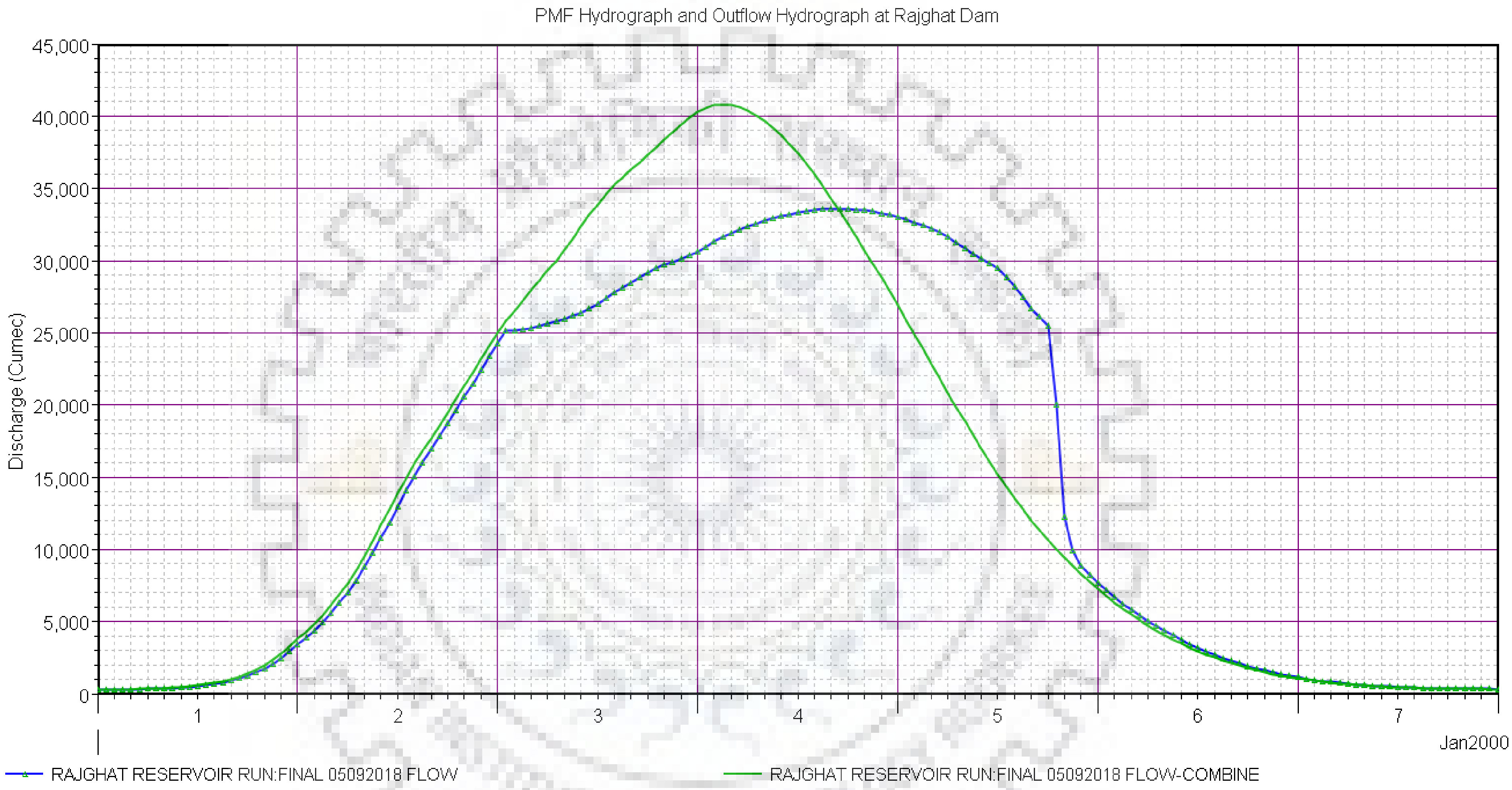

Fig-5.18: PMF Hydrograph for Ragjhat Dam

CHAPTER 6: RESULTS AND DISCUSSIONS

6.1 Comparison of Original vs Revised Design Flood

From the HEC-HMS model the revised design flood (PMF) for Matatila dam and Rajghat dam has been estimated as 34192 cumec and 40824 cumec respectively.

Table-6.1: Original Design flood vs Revised Design Flood

Dam	Original Design Flood (cumec)	Revised Design Flood (cumec)	Percentage Change	Spillway Capacity
Matattila Dam	23390	**34192**	46	15857
Rajghat Dam	44555	**40824**	-8	33893

The revised design flood of Matatila dam has been exceeded the original design flood significantly i.e. increased by 46 percent. Thus, Matatila dam was found unsafe from hydrological considerations. In this scenario, to improve the dam safety aspects of Matatila dam further options would be considered.

6.1.1 Sensitivity Analysis varying the Muskingum routing parameters

A sensitivity analysis has been performed by varying the Muskingum routing parameter “x “. The value of x has been changed to 0.15 and 0.25 for the river reaches in the model. Accordingly, the number of sub-reaches has been adjusted in the model. The values of design flood at Matatila dam have been compared as shown in Table-6.2 below.

Table-6.2: Sensitivity Analysis

Name of Dam	Design Flood value at x=0.2	Design Flood value at x=0.15	Change in %	Design Flood value at x=0.25	Change in %
Matatila dam	34192	34040.8	-0.4	34284.8	0.3
Rajghat Dam	40824	40455.2	-0.9	41111	0.7

It has been observed that the change in design flood by varying routing parameter is less than 1% in both cases, which is not very significant and it can be concluded that the variation in the Muskingum routing parameter “x” has less significance in the end result for design flood estimation.

6.2 Dam Safety Scenarios

The inflow design Flood for Rajghat reservoir was found to be 40824 cumec and after routing the flood through the Rajghat spillway, the outflow hydrograph has reached the peak value of spillway capacity. It may be noted that the impingement level was considered at FRL. Therefore, to obtain a reduced Inflow design flood value for Matatila dam following scenarios may be considered.

(iv) To reduce the impingement level for flood at Rajghat reservoir and rout the PMF hydrograph at Rajghat Dam.

(v) To restrict the spillway capacity of Rajghat to a lower capacity so as to obtain a less peak outflow hydrograph from Rajghat Spillway.

(vi) To encroach some part of the freeboard available at Rajghat dam temporarily to increase the surcharge storage.

Table-6.3 Spillway Release as per Dam safety Scenario

Level	**Storage**	**Discharge**	**Scenario-1**	**Scenario-2**	**Scenario-3**	**Scenario-4**	**Scenario-5**
(m)	1000 m3	m3/s	m3/s	m3/s	m3/s	m3/s	m3/s
357	220000	0					
365	1100000	11368					
365.5	1175000	12468					
366	1250000	13600					
366.5	1330000	14702					
367	1450000	15959					
367.5	1525000	17189					
368	1600000	18440			18440	18440	18440
368.5	1700000	20370			20370	20370	20370
369	1800000	21035			21035	21035	20705
369.5	1875000	22373	22373	22373	22373	22373	21619
370	2000000	23742	23742	23742	23742	23742	22478
370.5	2075000	25135	25135	25135	25135	25135	23298
371	2200000	26555	26555	26555	26555	26555	24083
371.5	2300000	27999	27999	26555	27999	26555	24838
372	2420000	29470	29470	26555	27999	26555	25647
372.5	2590000	30602	29470	26555	27999	26555	26475
373	2700000	31727	29470	26555	27999	26555	27253
374	3000000	33893	29470	26555	27999	27999	27999
375	3300000	36400	29470	26555	27999	27999	*27999*

Five dam safety scenarios have been generated using the above conditions, which are described in detail as under:

6.2.1 Scenario-1: The spillway capacity of Rajghat Reservoir has a capacity of 33893 cumec at 374m level. To reduce the outflow from spillway to a lower value than its capacity, the spillway could be operated at a lower discharge; in this trail scenario following conditions have been set in the model:

(a) Reducing Reservoir level to 1.0m from FRL, i.e. 370.0M and

(b) Reducing Spillway capacity to 29470 cumec.

(c) Increasing maximum water level by 2.0m. i.e. 375m

Results of Scenario-1

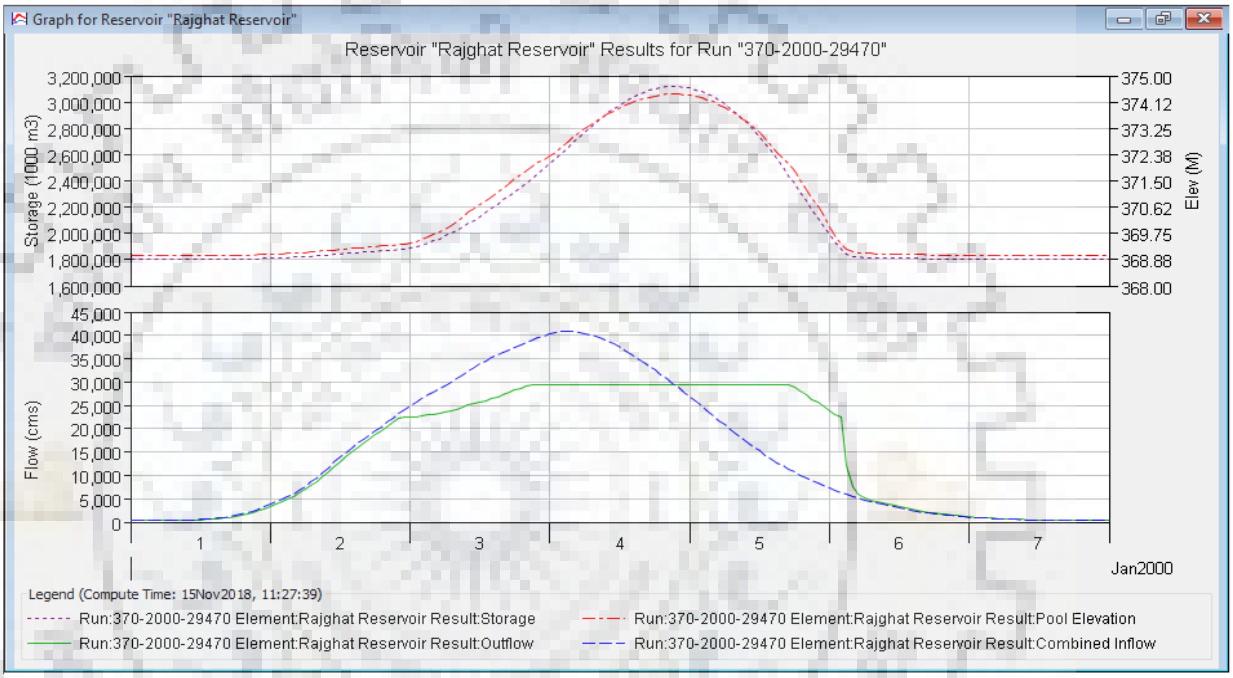

Fig-6.1: Results of Rajghat Reservoir as per Scenario-1

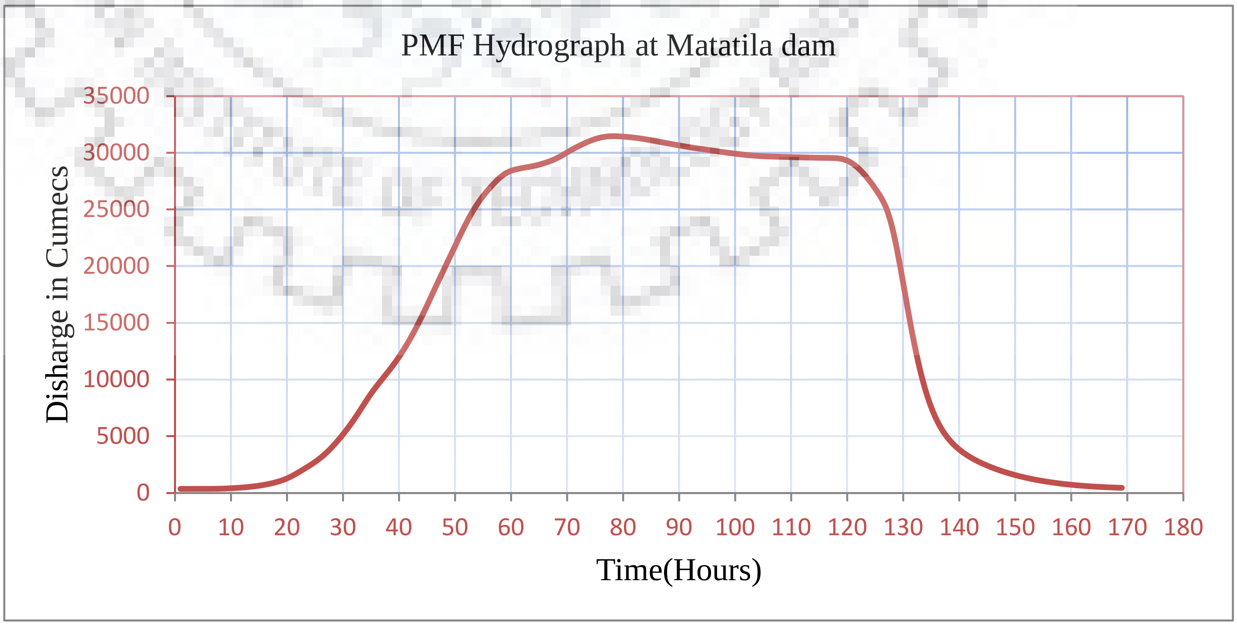

Fig-6.2: PMF hydrograph at Matatila dam as per Scenario-1

Inflow at Matatila dam = 31460 cumec

MWL at Rajghat Dam after routing = 374.4m

6.2.2 Scenario-2: In this scenario the requirement is to further reduce the design flood value. In Scenario-1, the model was run successfully at a spillway capacity 29740 cumec. In this scenario the value could further lower, following conditions have been set in the model;

(a) All conditions as per scenario -1, expect reducing Spillway capacity to 26555 cumec.

Results of Scenario-2: The run failed as the maximum water level was exceeded during Rajghat reservoir routing.

6.2.3 Scenario-3: As the model run failed in the scenario-2, it may be required to reduce the flood impingement level so as to use the additional storage. Further, as the freeboard available at Rajghat dam is 4m and considering relaxation of freeboard to improve safety aspects, 2m of the freeboard may be encroached for flood storage temporarily. So, maximum water level for the Rajghat reservoir has been increased from 373m to 375m in this scenario.

In this trial scenario, the following conditions have been set in the model:

(a) Reducing Reservoir level by 3.0m from FRL, i.e. 368.0M and

(b) Reducing Spillway capacity to 27999 cumec.

(c) Increasing Maximum Water Level by 2.0m to 375m

Results of Scenario-3

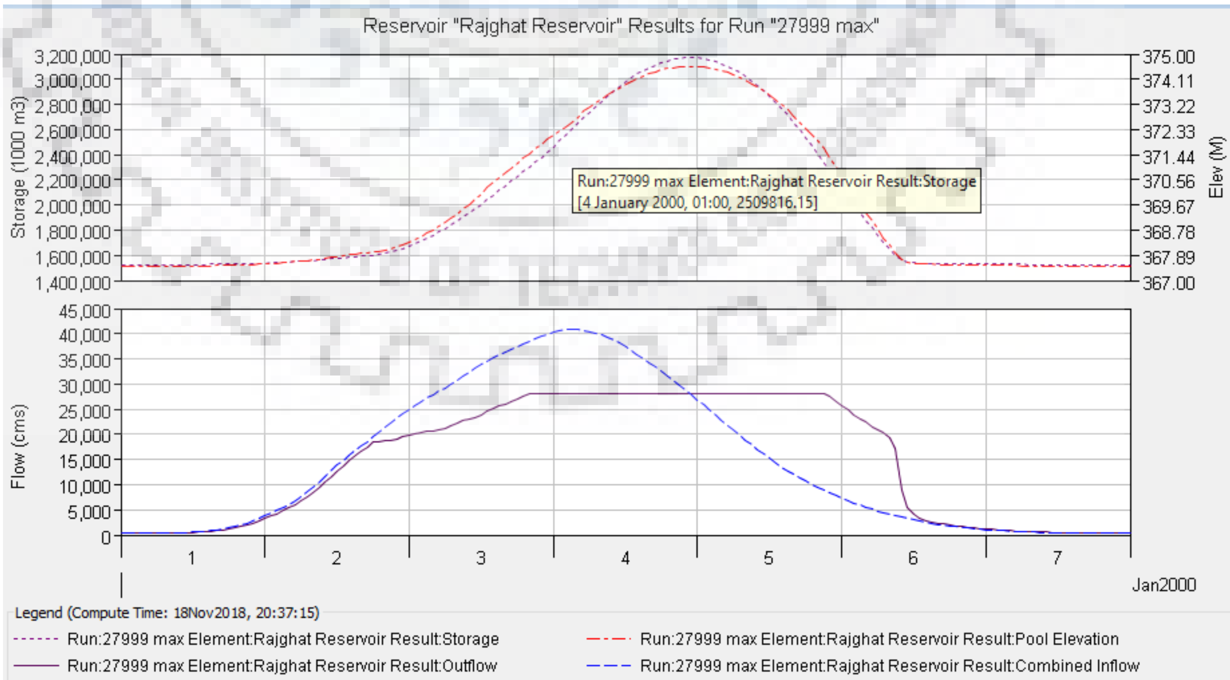

Fig-6.3: Results of Rajghat Reservoir as per Scenario-3

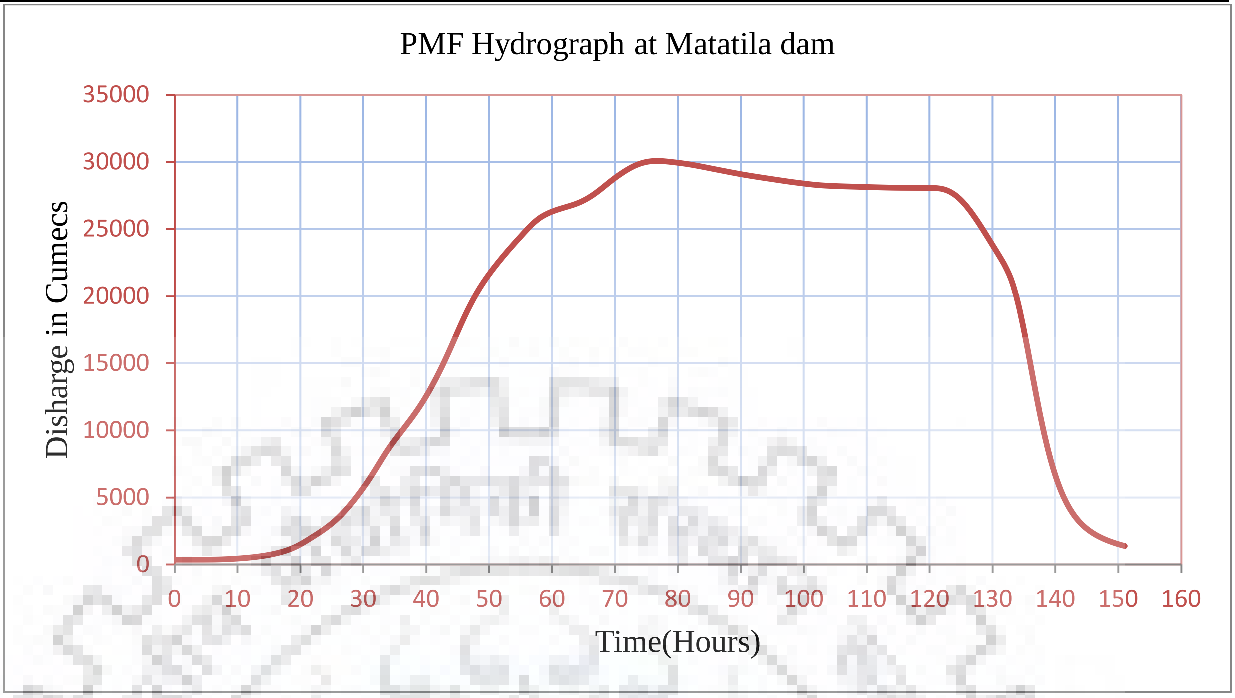

Fig-6.4: PMF hydrograph at Matatila dam as per Scenario-3

Inflow at Matatila dam = 30076 cumec

MWL at Rajghat Dam after routing = 374.6m

6.2.4 Scenario-4: In scenario-3 a flat value of discharge 27999 cumec has been adopted from 371.5 to 375 m reservoir level. In this scenario, from 371.5m to 373m the value of 26555 cumec has been replaced instead of 27999 cumec, to obtain a smooth release from the spillway. The rule curve of gate operation has been shown in Table under scenario-4.

Results of Scenario-4

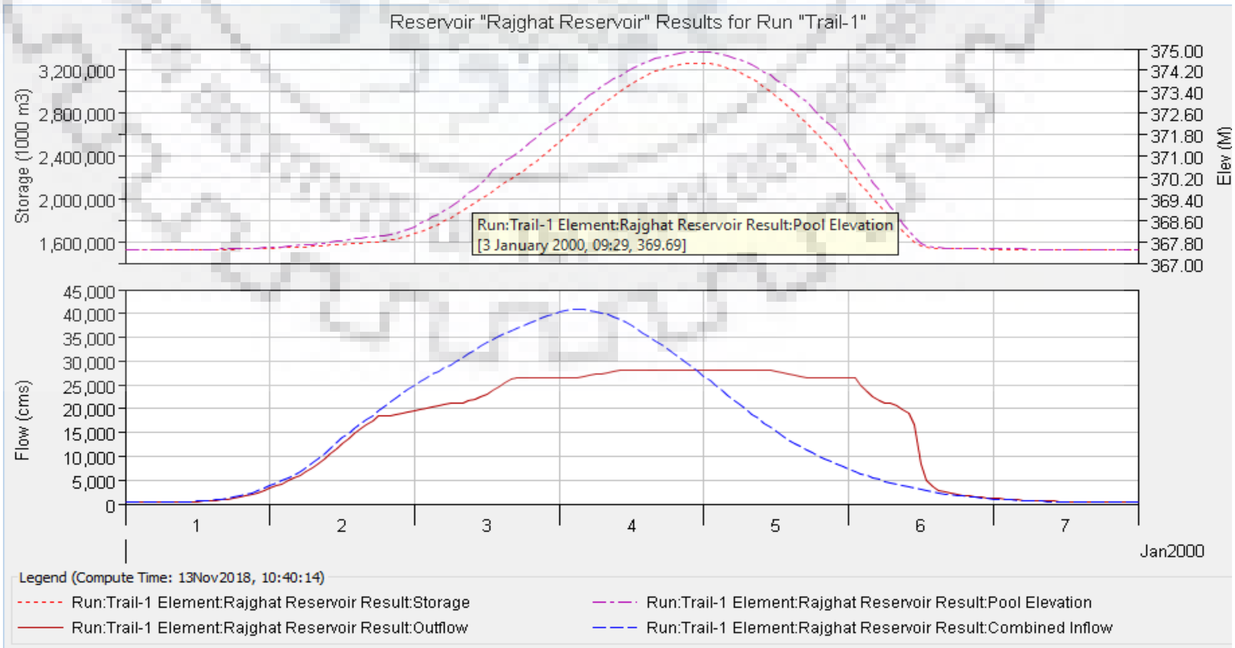

Fig-6.5: Results of Rajghat Reservoir as per Scenario-4

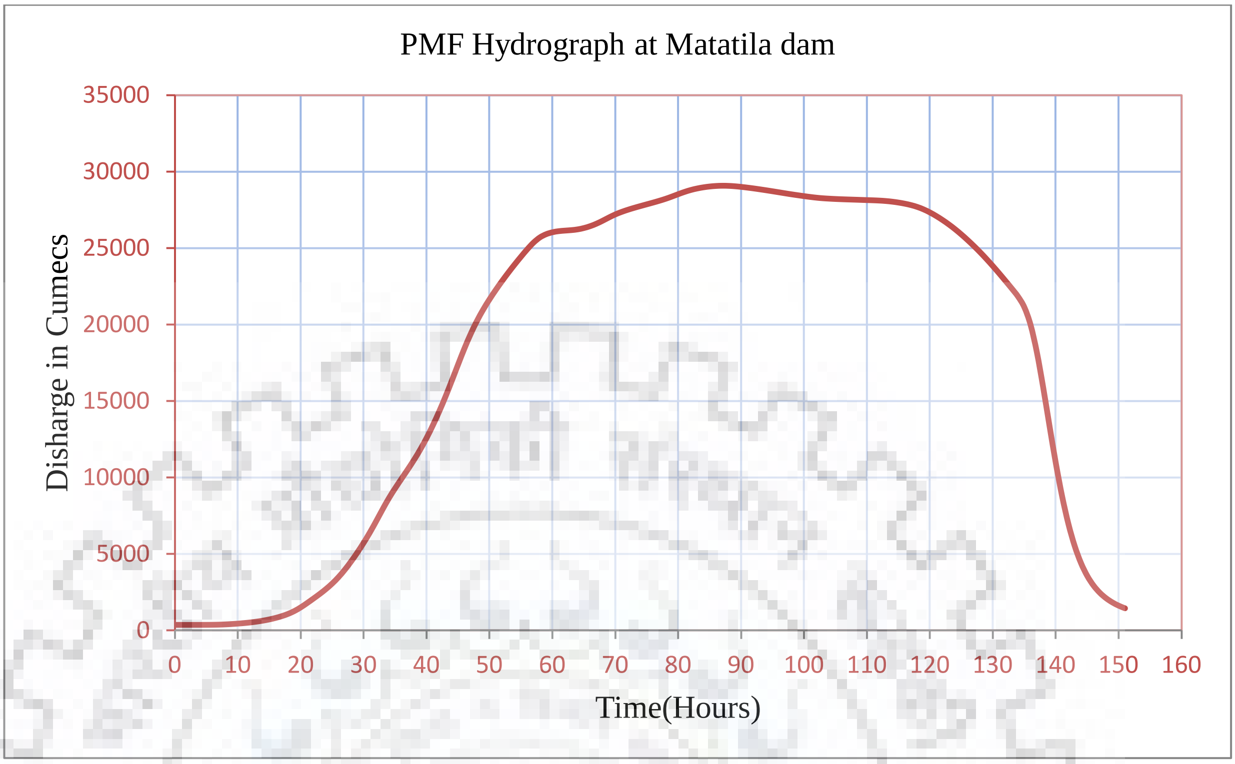

Fig-6.6: PMF hydrograph at Matatila dam as per Scenario-4

Inflow at Matatila dam = 29080 cumec

MWL at Rajghat Dam after routing = 374.9m

6.2.5 Scenario-5: With a closer look at the inflow hydrograph at Matatila dam for scenario-4 (Fig-6.6), it is observed that at the peak of the hydrograph, there is a bump and undulating peak, which may be smoothened by trying a rule curve of gate operation which is realistic and possible in practical operation of gates. The rule curve adopted for this scenario is as per the release of spillway gates at 11m of gate opening. This is highlighted in the Rajghat gate operation schedule, Table-6.4 below.

Table-6.4: Rajghat - Gate Operation (18 gates)

Gate Opening (m)	**R.L.** 368.0	**R.L.** 368.5	**R.L.** 369.0	**R.L.** 369.5	**R.L.** 370.0	**R.L.** 370.5	**R.L.** 371.0	**R.L.** 371.5	**R.L.** 372.0	**R.L.** 372.5	**R.L.** 373.0
0.50	1432	1464	1496	1528	1559	1589	1620	1647	1677	1702	1736
1.00	2771	2838	2908	2979	2021	3105	3165	3236	3302	3357	3424
1.50	4061	4159	4266	4359	4470	4567	4657	4750	4837	4917	4996
2.00	5293	5432	5576	5694	5841	5960	6080	6213	6346	6447	6568
2.50	6485	6669	6830	6997	7173	7322	7482	7638	7793	7931	8084
3.00	7624	7845	8062	8254	8451	8646	8838	9025	9212	9379	9562
3.50	8734	8957	9223	9461	9679	9916	10147	10366	10583	10780	11009

4.00	9773	10039	10347	10603	10855	11149	11423	11660	11909	12151	12703
4.50	10718	11581	11403	11712	12014	12328	12620	12924	13207	13481	13757
5.00	11640	12054	12406	12770	13123	12745	13805	14125	14439	14747	15075
5.50	12545	12965	13376	13778	14172	14541	14924	15299	15673	15987	16328
6.00	13358	13842	14293	14758	15170	15573	16018	16380	16812	17185	17557
6.50	14113	12090	15135	15619	16116	16581	17014	17437	17878	18311	18745
7.00	14809	15385	15921	16472	17011	17361	17985	18471	18951	19390	19858
7.50	15446	16068	16649	17040	17827	18370	18906	19458	19946	20451	20954
8.00	16068	16692	17400	17987	18643	19200	19775	20369	20923	21466	21974
8.50	16603	17296	18013	18675	19347	19976	20620	21260	21855	22367	22978
9.00	17176	18192	18563	19200	20025	20730	21423	22035	22668	23287	23937
9.50	17682	18460	19198	20334	23703	21398	22136	22825	23499	24195	22301
10.00	18110	18966	19772	25125	21270	22007	22809	23890	24318	24943	25635
10.50	**18440**	19801	20268	21111	21904	22663	23394	24209	25010	25714	26489
11.00		**20370**	**20705**	**21619**	**22478**	**23298**	**24083**	**24838**	**25647**	**26475**	**27253**
11.50			21035	22076	22985	23871	24718	25527	26308	27144	27966
12.00				22373	23412	24378	25291	26060	26998	27800	28581
12.50					23742	24805	25570	26583	27633	28490	29323
13.00						25135	26225	27243	28206	29125	29997
13.50							26555	27668	28713	29698	30647
14.00								27999	29140	30205	31220
14.50									29470	30602	31727

Results of Scenario-5

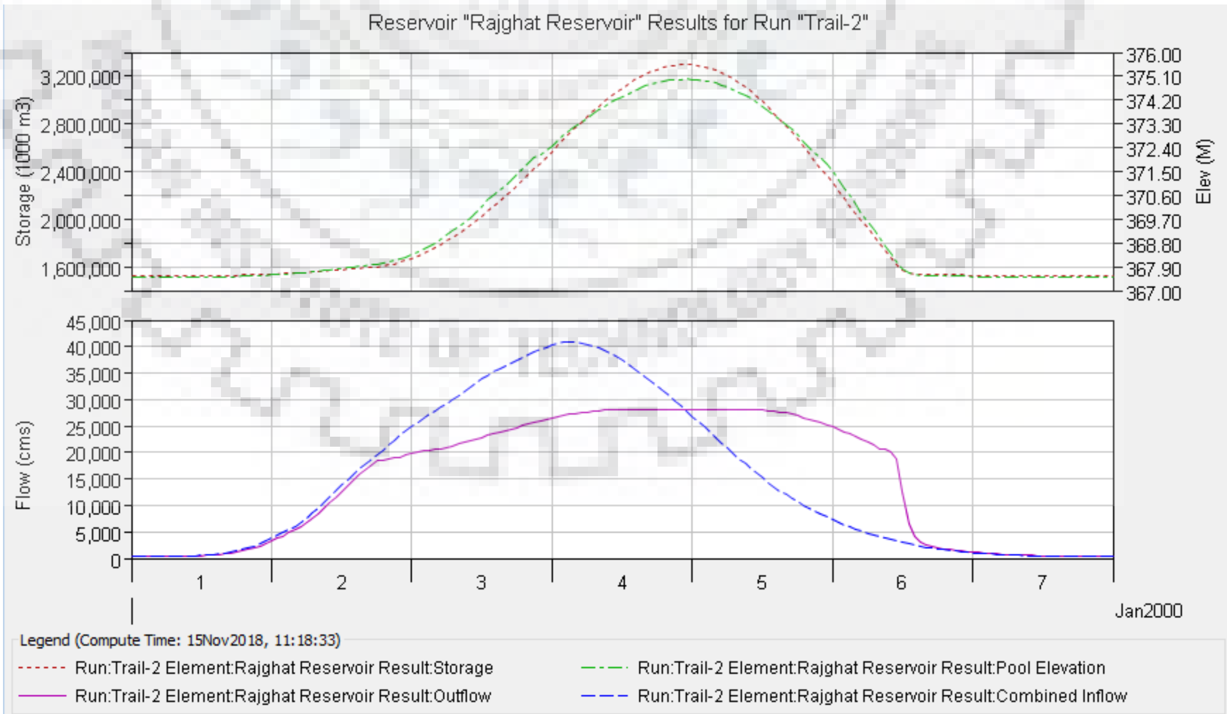

Fig-6.7: Results of Rajghat Reservoir as per Scenario-5

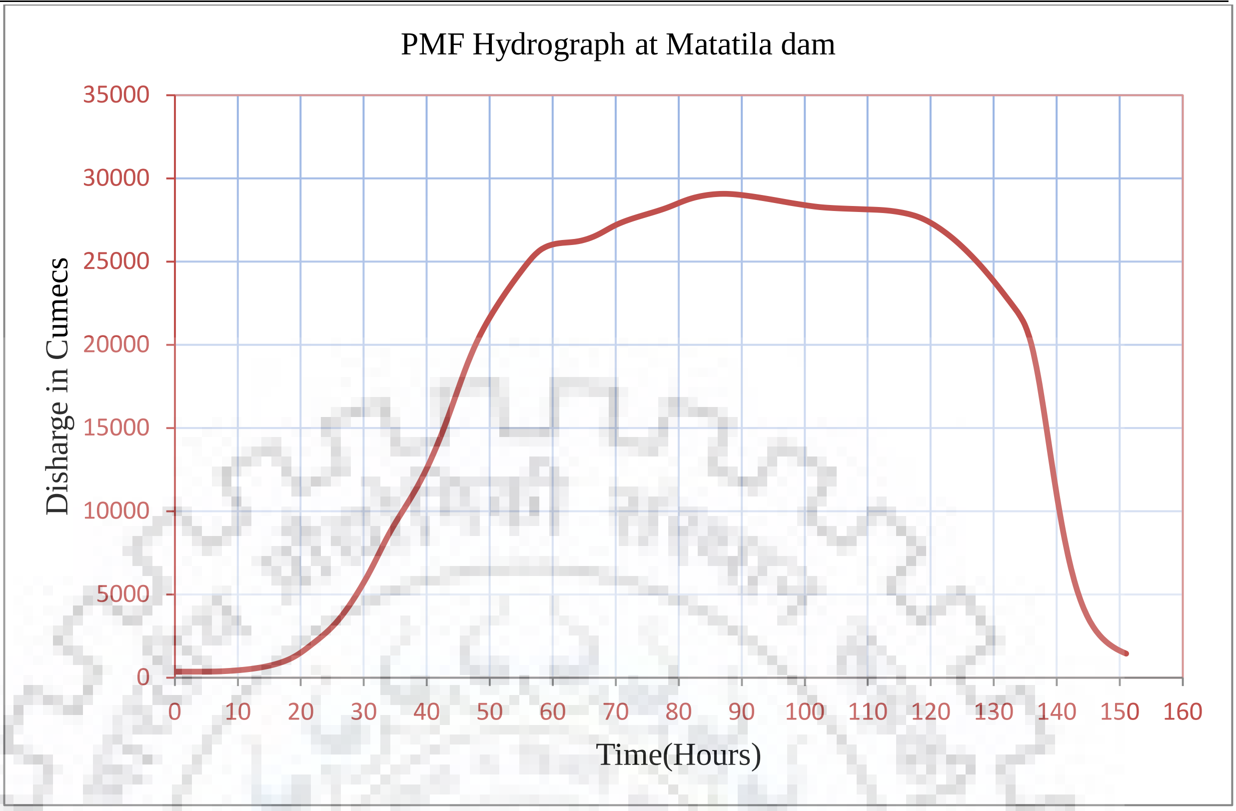

Fig-6.6: PMF hydrograph at Matatila dam as per Scenario-5

Inflow at Matatila dam = 29068 cumec

MWL at Rajghat Dam after routing = 375.0 m

The summary of all the five scenarios have been summarized in Table-6.5.

Table-6.5: Summary results of all Scenarios

Features of Rajghat Dam ; FRL = 371.0m; MWL= 373m; Dam Top = 377m; Freeboard = 4m						
Scenario No	Impingement Level	Gate Operation/ Opening	Outflow from Rajghat (cumec)	Inflow at Matatila (cumec)	Maximum Water Level reached at Rajghat(m)	Remaining freeboard (m)
Existing	FRL - 371.0 M	Maximum Spillway Capacity - 33893 cumec (at 374.0m)	33575.2	34191.5	373.9	3.1
1	Elevation - 370.0 M	Maximum Spillway Capacity -29470 cumec, with specified gate operation as per scenario-1	29470	31460	374.4	2.6

2	Elevation - 370.0 M	Maximum Spillway Capacity - 26555 cumec, with specified gate operation as per scenario-2	-	-	-	-
3	Elevation - 368.0 M	Maximum Spillway Capacity – 27999 cumec, with specified gate operation as per scenario-3	27999	30076	374.6	2.4
4	Elevation - 368.0 M	Maximum Spillway Capacity – 27999 cumec, with specified gate operation as per scenario-4	27999	29080	374.9	2.1
5	Elevation - 368.0 M	Maximum Spillway Capacity – 27999 cumec, with specified gate operation as per scenario-5	27999	29068	375	2

6.3 Discussion on the results of Scenarios

The summary results of the five scenarios are shown in Table-7.5. The resulting peak outflow from Rajghat dam, Inflow at Matatila dam, Maximum water level reached at and the freeboard available at Rajghat reservoir are summarized, which may be compared scenario-wise. In the existing scenario 34191.5 cumec design flood was obtained with available freeboard of 3.1m. The aim of optimising the design flood is achieved at Matatila dam and as per the last scenario it is 29068 cumec. This has been exceeded the original design flood by 24 percent. The dam has found to be unsafe from design flood as per BIS standards. However, the risk has been reduced from 48 percent to 24 percent by operating the Rajghat reservoir as per the prescribed rule curve.

CHAPTER 7: CONCLUSION AND RECCOMENDATIONS

7.1 General

Any development measure such as a dam, building or a bridge, presents a certain degree of risk to life or damage to property, in case of its failure. Dams due to the changing conditions of the population along the downstream river banks also constitute certain degree of hazard in the case of a failure. Dam safety is now considered an inherent feature in the planning, design, construction, mantatinance and operation of dams. The statistical analysis of record of dam failures suggests that one third of the failures of dams are because of inadequate spillway capacity. The hydrologic safety covers the study of Inflow Design Flood (PMF, SPF or Return Period Flood), (ii) Outflow Hydrograph from the Spillway and its moderation (iii) Reservoir Routing and Maximum water levels in the reservoir (iv) Gate Operation at different reservoir levels (v) and Free board etc.

In this study above safety aspects of the Matatila dam have been analysed. The Probable Maximum Flood (PMF), which is the design flood for the Matatila Dam, has been estimated using the latest techniques available viz. quasi distributed modelling using hydro-meteorological approach. This involves estimation of a design storm hyetograph and derivation of catchment response function. The catchment response function in this study is the synthetic unit hydrograph (SUH). In a quasi distributed model, the large catchment under study is divided into a number of sub basins and SUH for each sub basin have been derived from the physiographic properties of respective sub-basins, which are obtained using the ARC-GIS. The Probable Maximum Precipitation (PMP) for the catchment is estimated by using the storm transportation and multiplying the Moisture Adjustment Factor. Critical arrangements of rainfall depths and time distribution analysis have been performed to obtain the Effective Rainfall Hyetograph (ERH). The convolution of ERH and catchment response function is performed in a HEC-HMS model. The lumped responses generated at the outlet of each sub basin have been channel routed and reservoir routed through the reservoir and a network of channels/rivers in the model to obtain the PMF hydrograph. The Rajghat reservoir which is of higher storage capacity and situated a little upstream of Matatila dam has been regulated to improve the safety of downstream Matatila dam. For greater safety of existing dams under floods, relaxation of the ambient conditions at Rajghat dam has been analysed. Using the combinations of relaxations of ambient conditions, five dam safety scenarios are generated and modelled in HEC-HMS. From these scenarios a reduction in the value of inflow design flood has been arrived at Matatila dam as envisaged which exceeds the original flood by twenty four percent only. As per the results obtained from the last scenario the dam has been found to be unsafe from hydrological point of view in PMF scenario.

7.2 Conclusion

The following conclusion emerges from the study:

a) The hydro meteorological approach of design flood estimation is very convenient and accurate and it has many advantages over the conventional method of flood estimation. Causative factors for flood are analyzed in this approach. It gives a complete flood hydrograph and volume of runoff which allows a realistic determination of its moderating effect while passing through a reservoir or river reach.

b) The results of the HEC-HMS model have shown that, compared to the original design flood, the revised design flood at Matatila dam has been exceeded significantly. For the upstream Rajghat reservoir the design flood (PMF) was found to be in order.

c) The design flood of existing dams for which design flood was estimated way back by using flood frequency analysis and regional formulae are need to be updated as per current standards and practice.

d) For greater safety of existing dams under floods, relaxation of the ambient conditions at dam may be analysed. These are (i) reduce the impingement level for flood at reservoir (ii) to restrict the spillway capacity of upstream reservoir to obtain a less peak outflow hydrograph (iii) to encroach some part of the freeboard available to increase the surcharge storage.

e) The potential of flood control of existing upstream reservoirs should be analysed and be used advantageously with specified rule curve for spillway gate operation.

f) If dam is found to be unsafe from hydrological and hydraulic considerations under relaxed ambient conditions, it would be appropriate to adopt structural / non-structural measures to tackle the potential hazard conditions. Inflow flood forecast is one of the effective non-structural measures.

7.3 Recommendations

If the spillway of the dam, has insufficient hydraulic capacity to discharge the adopted design flood, the dam is considered hydrologically unsafe. This may please be noted that a hydrologically unsafe dam should not be considered as structurally unsafe, if it is able to sustain the design forces. For hydrologically unsafe dams, the mitigating measures should be planned in advance to tackle the potential hazardous situations. The feasibility of such type of measures would depend on the resources available to the dam owner. Following mitigating measures viz. structural and non-structural are recommended to meet the inadequate hydraulic capacity.

1. Increasing Discharge Capacity

Increasing the discharge capacity of a spillway is direct measure for the problem of inadequate spillway capacity. Some approaches to providing more spillway discharge capacity are lengthening and adding gates to gated spillways, lowering crest of existing spillway etc. Constructing a new spillway, in which a section of dam may be redesigned as emergency spillway or locating a new spillway in an abutment area or a low saddle at the reservoir rim.

2. Increasing height of the dam and reservoir storage capacity.

Increasing the height of a dam can effectively increase not only the spillway capacity but also the reservoir storage. Before increasing the height of dam, necessary check of stability of structures and geology of dam and energy dissipation arrangements should be made. The height of dams may be increased by increasing height of parapet walls on the upstream side of the dam. For larger height increases, concrete mass may be required to be placed on top and on the downstream face of a masonry dam.

3. Modification of dams to permit overflow

Dams can be constructed to withstand overflow. A masonry dam generally can withstand overflow if the added hydraulic loading does not endanger the stability of the structure and if the overflow will not erode the foundation at the toe of the dam or damage other downstream facilities, such as outlet valves and controls.

4. Emergency Action Plan (EAP) and Dam Break Analysis(DBA)

Where an existing dam is found to have inadequate hydrologic safety in the review, immediate action should be initiated to frame an Emergency Action Plan (EAP) as a disaster prevention measure. It would be obligatory on the part of dam owner to plan disaster preparedness, including Dam Break Analysis (DBA) to meet the unlikely event of a failure indicating the possible inundation downstream of the reservoir consequent to dam break. Such an EAP should be kept active until the permanent corrective measures are evolved and implemented.

5. Reservoir Operation

If one or more dams are located downstream, the flood wave that could result from failure of upstream dam should be routed to evaluate if any of the downstream dams would potentially breach in domino-like action. To ensure the hydrologic safety of a dam, several reservoir regulation requirements need to be followed; maximum and minimum regulated releases from a dam should be specified. The maximum regulated release rate should be specified to prevent flooding or erosion of downstream areas and control the rate of reservoir drawdown. A minimum regulated release capacity facilitates the recovery of flood control storage for use in regulating subsequent flood events.

6. Flood forecasting system

An efficient and reliable flood forecasting system should be established to formulate accurate forecasts of inflow and volume of floods and regulation of gates for efficient flood disposal.

7.4 Limitations and Further scope of study

Limitations of the study:

a. Concurrent rainfall and discharge data were not available for study, to generate the unit hydrographs.
b. The base flow and design loss rate are taken from Flood Estimation reports, as the actual values are not available.

Future scope of study:

a. Structural and operational safety of dam may be analysed.
b. Dam Break studies of both Rajghat and Matatila dam.

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