CS/AR-36/98-99

SIMULATION OF FLOOD HYDROGRAPH USING AN EVENT BASED RAINFALL-RUNOFF MODEL

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ABSTRACT

Present study deals with flood estimation of Narmada catchment upto Jamtara considering the catchment as ungauged and using the physical and geo-morphological characteristics of the catchment. Total area up to Jamtara being about 17100 sq. km., has been sub-divided into five sub-catchments for the purpose of flood estimation, and flood hydrograph for each sub-catchment is computed and routed through river reach to get the final flood hydrograph at Jamtara.

Flood hydrographs for individual sub-catchments are computed using geo-morphological approach while routing of flood hydrographs is accomplished by using the Muskingum-Cunge routing method. Final flood hydrograph so computed is compared with the observed flood hydrograph at Jamtara. In total, five flood events are used in the analysis. The study demonstrates the use of methodology to compute the flood, where the catchment size is large and very little or no information except rainfall is available.

INTRODUCTION

The problem of transformation of rainfall into runoff has been a very active area of research throughout the evolution of the subject of hydrology. In the past, many investigators have tried to relate runoff with the different physiographic and climatic characteristics. The simplest theory proposes to multiply the rainfall with some factor (called the runoff coefficient) to get the runoff. A better way to transform rainfall into runoff is to apply conceptual models in which the various interrelated hydrological processes are conceptualized. More sophisticated procedures are also evolved which are based on the physical concept of the process and try to model this hydrological phenomenon on the basis of physical laws governing them. Actually, many more factors, besides the accuracy, e.g., the availability of data, computing facility, time, resources etc. govern the applicability of a model.

Correct estimation of the flood is one of the most important aspect of the water resources development planning. Today, with availability of more data and the growing awareness for the accuracy in flood estimation, the unit hydrograph, flood routing and flood frequency analysis are commonly used to predict flood flows. However, still for most of the sites, information about runoff is either not available or is insufficient for the complete hydrological analysis. For such cases the available information of the nearby catchment or the information of the region can be used to carry out the further analysis. This approach attempts to establish relationships between model parameters and physically measurable watershed characteristics for gauged catchments. These relationships are then assumed to hold for ungauged watersheds having similar hydrologic characteristics.

Rainfall-runoff relationships for ungauged watersheds have been developed along two complimentary lines: (1) Empirical equations have been developed to relate some individual runoff hydrograph characteristics to watershed characteristics (2) Procedures have been developed to synthesize the entire runoff hydrograph from watershed characteristics. Bernard (1935) model is perhaps the first attempt to synthesize the unit hydrograph (UH) from watershed characteristics. It assumes that the peak of the UH is inversely proportional to the time of concentration, which in turn is assumed to be proportional to a watershed factor. A distribution graph establishes relation between the effective percentage area contributing and overland slopes expressed as the average slope of the hypsometric curve and stream pattern. Taylor and Schwarz (1952), in addition to the watershed characteristics employed by Snyder (1938), introduced the average slope of the main channel. The method of hydrograph synthesis employed by the Soil Conservation Service (SCS) (1971), U.S. Deptt. of Agriculture, uses an average dimensionless hydrograph derived from an analysis of a large number of natural UHs for watersheds varying widely in size and geographical locations. Among different approaches used to estimate discharges of extreme floods are the index flood and regional regression methods (National Research Council (NRC), 1988).

Clark (1945) developed a technique to compute the unit hydrograph of any desired unit period using the concept of instantaneous unit hydrograph. This method utilises the two parameters only i.e. the time of concentration, T_c and storage coefficient R. This storage coefficient has been related with the catchment characteristics. The time of concentration was considered to equal the time interval between the end of rain and the point of contraflexure of the hydrograph recession limb. This time base was measured from the recorded floods and not related to watershed characteristics. Nash (1960) model has two parameters n and K. Nash showed that these parameters were related to the first and second moments of the IUH about the origin. These moments were then correlated empirically with watershed characteristics.

Boyd (1978, 1982) developed the linear watershed bounded network (LWBN) model for synthesis of the IUH employing geomorphologic and hydrologic properties of the watershed. The model divides a watershed into sub-areas bounded by watershed lines using large-scale topographic maps. The model has a large number of lumped storage parameters. Most of these parameters are deduced from geomorphologic properties.

Rodriguez-Iturbe and Valdes (1979) developed an approach for derivation of the IUH by explicitly incorporating the characteristics of drainage basin composition (Horton, 1945; Strahler, 1964; Smart, 1972). The approach coupled the empirical laws of geomorphology with the principles of linear hydrologic systems. Rodriguez-Iturbe and his associates have since extended this approach by explicitly incorporating climatic characteristics and have studied several aspects including hydrologic similarity. Gupta, Waymire and C.T.Wang (1980) examined this approach, and reformulated, simplified and made it more general.

The effect of climatic variation is incorporated by having a dynamic parameter velocity in the formulation of Geomorphological IUH (GIUH). This is a parameter that must be subjectively evaluated. It is shown (Rodriguez-Iturbe, et.al., 1979) that this dynamic parameter "velocity" of the GIUH can be taken as the velocity at the peak discharge time for a given rainfall-runoff event in a basin. This transforms the time invariant IUH throughout the event into a time invariant IUH in each storm occurrence.

In the derivation of GIUH one of the greatest difficulties involved is the estimation of peak velocity. This is a parameter that must be evaluated for each flood event. Rodriguez et.al. (1982) rationalised that velocity must be a function of the effective rainfall intensity and duration and proceeded to eliminate velocity from the results. It leads to the development of geomorphoclimatic instantaneous unit hydrograph. The governing equations consists of the terms such as the mean effective rainfall intensity, Manning's roughness coefficient, average width, and slope of the highest order stream. Janusz Zelazinski (1986) gave a procedure for estimating the flow velocity. It involves the development of the relationship between the velocity and corresponding peak discharge. Panigrahi (1991) estimated the velocity using the Manning's equation and equilibrium discharges. It requires the intensity of each rainfall block for the event for the computation of equilibrium discharge. The channel cross-section at the gauging site, longitudinal slope and Manning's roughness are also required during the computation of the velocity. The methodology has been applied at National Institute of Hydrology to estimate the velocity to derive the Clark model parameters using GIUH approach for the small sub-catchments of upper Narmada and Morel catchment (NIH, 1995;1997).

The Muskingum method of flood routing was first introduced by McCarthy and others (U.S. Army Corps of Engineers, 1960) in connection with the flood control studies of the Muskingum river basin in Ohio, U.S.A. Since its development, this method has been widely used in river engineering practice. Due to its popularity among field engineers and hydrologists, this method has been extensively researched (Singh, 1988). This method was introduced as a linear storage routing method and it is termed herein as the classical or conventional Muskingum method to differentiate it with other variations which were introduced in later years. Later on, in the year 1969, Cunge proposed that the Muskingum method is essentially a linear kinematic wave solution and came out with a physically based alternative to the Muskingum method. The alternative method is popularly known as Muskingum Cunge method. The routing parameter of Muskingum Cunge method can be calculated as a function of the following numerical and physical properties: (i) Reach length; (ii) Reference discharge per unit width; (iii) Kinematic wave celerity; and (iv) Bottom slope. An improved version of the Muskingum-Cunge method is due to Ponce and Yevjevich (1978).

Present report deals with computation of flood hydrograph of Narmada catchment upto Jamtara using the numerical and physical properties of the catchment making use of GUIH based approach for flood estimation and Muskingum Cunge approach for flood routing.

THE CATCHMENT

The Narmada river rises in the Amarkantak Plateau of Maikala range in the Shahdol district of Madhya Pradesh at an elevation of 1057 meters above mean sea level. The river travels a distance of 1312 km and drains an area of about 17100 sq. km. upto Jamtara. The index map of Narmada catchment up to Jamtara is shown in Figure 1. To estimate flood hydrograph at Jamtara, the catchment up to Jamtara has been divided into five sub-catchments namely: Narmada up to Manot (A), Burhner upto Mohegaon (B), Banjar up to Hridaynagar (C), intervening area upto Mandla (D) and area in between Mandla and Jamtara site (E). Table 1 provides the details of these sub-catchments.

Table 1: Catchment area and other physical characteristics of sub-catchments of Narmada up to Jamtara

Sub-catchment	Area (Sq.km.)	Average Width of river bed (mtr)	Average Slope of main river	Manning's n	Length in kms.
Manot A	4980	160	0.00069	0.0353	239
Burhner B	4103	130	0.0021	0.0355	138
Banjar C	3472	200	0.0013	0.0355	185
Local D	375	260	0.00049	0.0355	35
Local E	5812	270	0.00049	0.0355	110

The river has a number of falls in its head reaches. Flowing in a generally south-westerly direction in a narrow and deep valley, the river takes pin head turns at places. Close to Jabalpur, 404 km from the source, the river drops nearly 15 m at the Dhaundhara falls, after which it flows through a narrow channel carved through the famous marble rocks. Figure 2 provides the L-section of the Narmada river and its tributaries up to Jamtara.

Narmada upto Manot lies between east longitude 80°24' to 81°47' and north latitude 22°26' to 23°18' in Mandla and Shadol districts of Madhya Pradesh. The river rises in Maikala range near Amarkantak in Shadol district at an elevation of 1057 meters. It flows for a total length of 239 kilometers upto Manot and drains a total area of about 4980 sq. km.

The Burhner rises in the Maikala range, south-east of Gwara Village in Mandla district of Madhya Pradesh at an elevation of about 900 m, at north latitude 22°32' and east longitude 81°22' and flows in a generally westerly direction for a total length of 177 km to join the Narmada at Basania near Manot. The Burhner drains a total area of 4,118 sq. km.

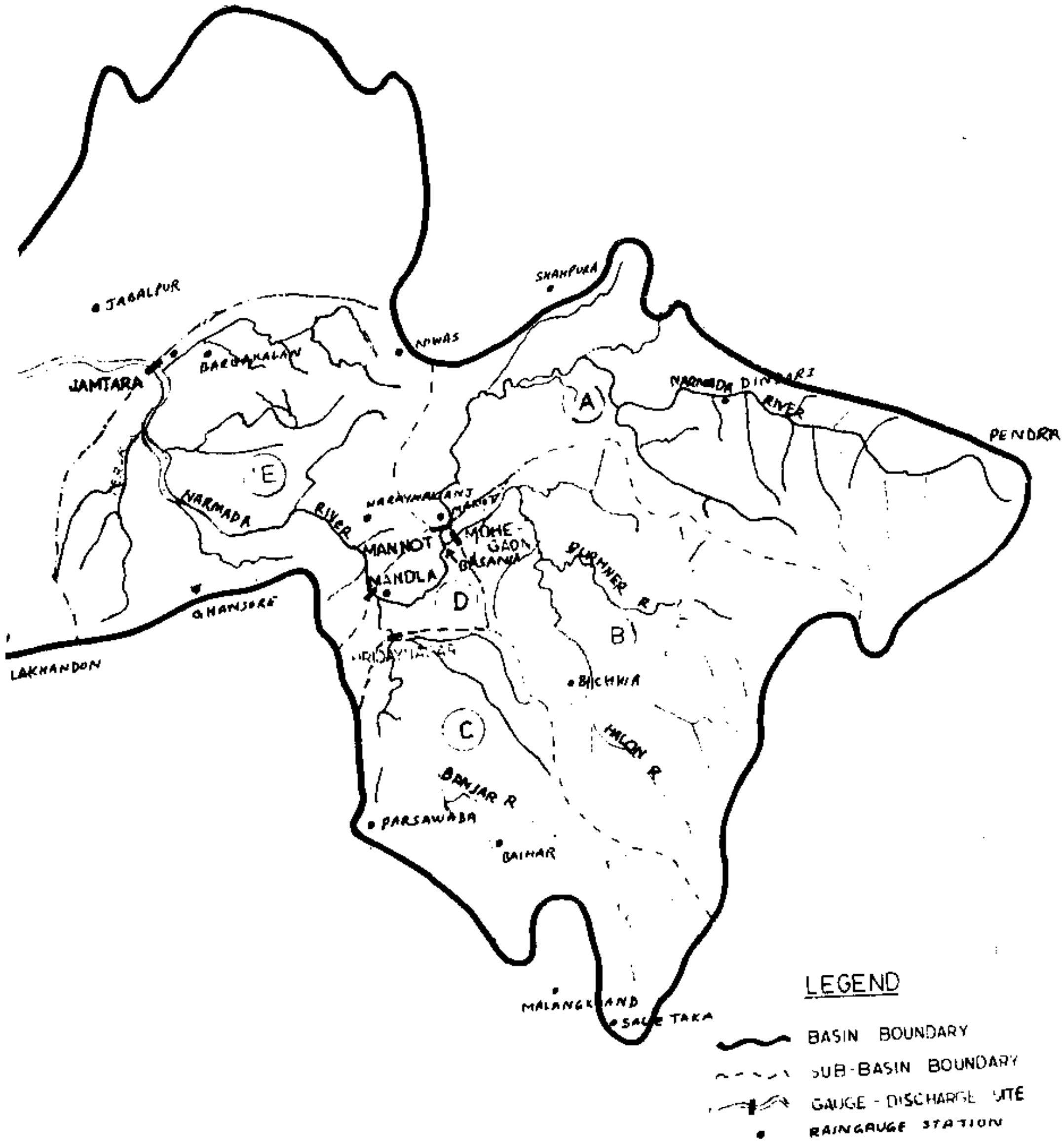
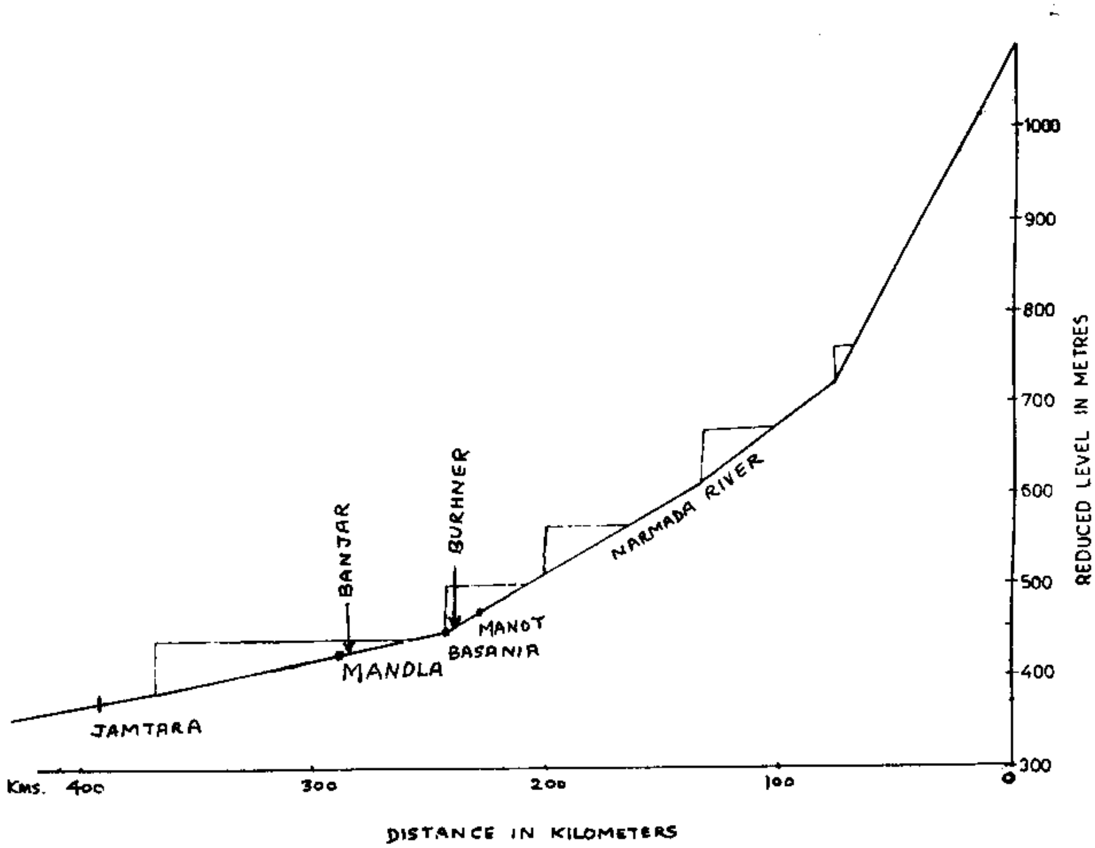
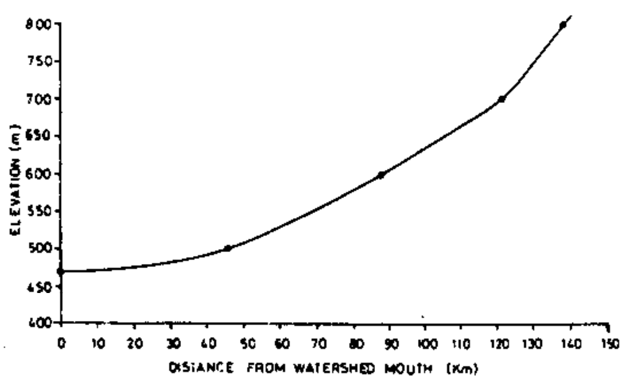
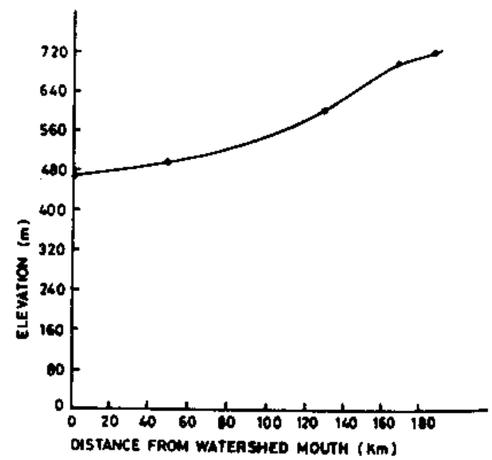


Figure 1: Index map of Narmada catchment up to Jamtara





MAIN CHANNEL PROFILE OF BURHNER RIVER UP TO MOHEGAON



MAIN CHANNEL PROFILE OF BANJAR RIVER UPTO HIRDENAGAR

FIGURE 2: L-SECTION OF NARMADA RIVER AND ITS TRIBUTARIES (UP TO JAMTARA)

The Banjar rises in the Satpura range in the Durg district of Madhya Pradesh near Rampur village at an elevation of 600 m at north latitude 21°42' and east longitude 80°50' and flows in a generally north-westerly direction for a total length of 184 km to join the Narmada from the left near Mandla at the 287th km of its run. The Banjar drains a total area of 3,626 sq.km.

The climate of the area is humid tropical ranging from sub-humid in the east to semi-arid in the west. South west monsoon is the principal rainy season accounting for nearly 90% of the annual rainfall. About 60% of the total annual; rainfall is received during July and August months. The area consists mainly of black soils.

DATA USED

In all five events depending upon the availability of records, are used in the analysis. Table 2 provides the details of these events.

Table 2: Details of storm events used in the analysis

1.	27th August to 29th August 1973
2.	10th August 1975
3.	6th August to 8th August 1977
4.	27th August to 30th August 1978
5.	8th August to 11th August 1979

The analysis makes use of hourly rainfall data apart from physical characteristics of the sub-catchments and channel reaches. Some physical characteristics which were available and used in the analysis, are already given in Table 1. Observed flood hydrograph at Jamtara is used to compare the computed flood hydrograph at Jamtara. Comparison of flood hydrograph at intermediate sites could not be done because of unavailability of recorded information for the period of concern. Raingauge stations used in the analysis differ from sub-catchment to sub-catchment and from event to event. For most of the events only at selected stations hourly rainfall data was available and therefore daily available rainfall at nearby stations is disaggregated and then used to get the average hourly rainfall for the sub-catchment. Table 3 provides the details of raingauge stations used in the analysis where daily rainfall was available while Table 4 gives the details of raingauge stations where hourly rainfall for different events was available. Cumulative values of average hourly rainfall for the whole catchment upto Jamtara and corresponding values of runoff converted in mm are plotted to check the consistency of the rainfall and runoff record. The records were seem to be in order.

Average hourly rainfall for each event for each sub-catchment used in the analysis, is given in Tables 5A to 5E.

Table 3 : Availability of daily rainfall data in and around Narmada catchment up to Jamtara

Sr. No.	Name of raingauge station	Period
1.	Pendra Road	7/8/79 to 10/8/79, 24/8/73 to 30/8/73, 5/8/77 to 9/8/77, 8/8/79 to 12/8/79
2.	Dindori	7/8/79 to 10/8/79, 24/8/73 to 30/8/73, 9/8/75 to 12/8/75 8/8/79 to 12/8/79
3.	Mandla	7/8/79 to 10/8/79, 24/8/73 to 30/8/73, 9/8/75 to 12/8/75 5/8/77 to 9/8/77
4.	Narayanganj	7/8/79 to 10/8/79, 24/8/73 to 30/8/73, 9/8/75 to 12/8/75 5/8/77 to 9/8/77, 8/8/79 to 12/8/79
5.	Niwas	7/8/79 to 10/8/79, 24/8/73 to 30/8/73, 9/8/75 to 12/8/75 5/8/77 to 9/8/77, 8/8/79 to 12/8/79
6.	Shahpur	24/8/73 to 30/8/73, 9/8/75 to 12/8/75
7.	Saletaka	24/8/73 to 30/8/73, 9/8/75 to 12/8/75
8.	Baihar	24/8/73 to 30/8/73, 9/8/75 to 12/8/75, 5/8/77 to 9/8/77, 8/8/79 to 12/8/79
9.	Barerkalan	24/8/73 to 30/8/73, 9/8/75 to 12/8/75, 5/8/77 to 9/8/77 8/8/79 to 12/8/79
10.	Ghansore	24/8/73 to 30/8/73, 9/8/75 to 12/8/75, 5/8/77 to 9/8/77
11.	Lakhandon	24/8/73 to 30/8/73, 9/8/75 to 12/8/75, 5/8/77 to 9/8/77
12.	Bichhia	9/8/75 to 12/8/75, 8/8/79 to 12/8/79
13.	Jamtara	9/8/75 to 12/8/75, 5/8/77 to 9/8/77, 8/8/79 to 12/8/79
14.	Manot	5/8/77 to 9/8/77
15	Malanjkhand	8/8/79 to 12/8/79

Table 4 : **Availability of hourly rainfall data in and around Narmada catchment up to Jamtara**

Sr. No.	Name of raingauge station	Period
1.	Pendra Road	10/8/75
2.	Mandla	6/8/79 to 8/8/79, 27/8/73 to 29/8/73, 10/8/75, 7/8/79 to 10/8/79
3.	Jabalpur	27/8/73 to 29/8/73
4.	Average rainfall for Narmada up to Jamtara	28/8/73 to 31/8/73
5.	Average rainfall for Manot	24/8/78 to 31/8/78
6.	Average rainfall for Banjar	24/8/78 to 31/8/78
7.	Average rainfall for Burhner	24/8/78 to 31/8/78
8.	Average rainfall for Local D	24/8/78 to 31/8/78
9.	Average rainfall for Local E	24/8/78 to 31/8/78
10.	Jamtara	10/8/75, 7/8/79 to 10/8/79
11	Malanjkhand	7/8/79 to 10/8/79

Table 5A : **Average rainfall in mm over the sub-catchments for August 73 storm**

Time (hrs.)	Average rainfall in mm over the sub-catchments				
	Manot A	Burhner B	Banjar C	Local D	Local E
27/8/73 1.00	0.00	0.00	0.00	0.00	0.00
2.00	0.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00	0.00
4.00	0.00	0.00	0.00	0.00	0.00
5.00	0.00	0.00	0.00	0.00	0.00
6.00	0.00	0.00	0.00	0.00	0.00
7.00	0.00	0.00	0.00	0.00	0.00
8.00	0.00	0.00	0.00	0.00	0.00
9.00	0.00	0.00	0.00	0.00	0.00
10.00	0.00	0.00	0.00	0.00	0.00
11.00	0.00	0.00	0.00	0.00	0.00

12.00	0.00	0.00	0.00	0.00	0.00
13.00	0.00	0.00	0.00	0.00	0.00
14.00	0.00	0.00	0.00	0.00	0.00
15.00	0.00	0.00	0.00	0.00	0.00
16.00	0.00	0.00	0.00	0.00	0.00
17.00	3.15	4.61	4.72	6.00	3.15
18.00	5.51	8.08	8.26	10.50	5.50
19.00	1.84	2.69	2.75	3.50	14.73
20.00	0.79	1.15	1.18	1.50	0.79
21.00	0.26	0.38	0.39	0.50	0.31
22.00	0.26	0.38	0.39	0.50	0.76
23.00	12.33	18.07	18.48	23.50	15.33
28/8/73 0.00	0.00	0.00	0.00	0.00	0.65
1.00	0.00	0.00	0.00	0.00	0.00
2.00	0.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00	0.06
4.00	0.00	0.00	0.00	0.00	0.09
5.00	0.00	0.00	0.00	0.00	0.02
6.00	0.00	0.00	0.00	0.00	0.19
7.00	1.85	2.00	1.94	2.55	6.52
8.00	9.79	10.60	10.28	13.51	9.99
9.00	2.50	2.71	2.63	3.45	2.54
10.00	0.00	0.00	0.00	0.00	0.00
11.00	1.63	1.77	1.71	2.25	6.03
12.00	5.98	6.48	6.28	8.26	6.33
13.00	0.00	0.00	0.00	0.00	0.04
14.00	0.00	0.00	0.00	0.00	0.00
15.00	0.00	0.00	0.00	0.00	0.02
16.00	2.72	2.95	2.85	3.75	2.88
17.00	1.63	1.77	1.71	2.25	7.17
18.00	0.87	0.94	0.91	1.20	2.97

19.00	3.70	4.01	3.88	5.11	5.11
20.00	0.65	0.71	0.69	0.90	0.74
21.00	2.72	2.95	2.85	3.75	3.28
22.00	0.00	0.00	0.00	0.00	1.29
23.00	0.00	0.00	0.00	0.00	3.91
29/8/73 0.00	0.00	0.00	0.00	0.00	0.00
1.00	0.00	0.00	0.00	0.00	7.61
2.00	0.00	0.00	0.00	0.00	9.88
3.00	0.00	0.00	0.00	0.00	3.40
4.00	0.00	0.00	0.00	0.00	9.97
5.00	0.00	0.00	0.00	0.00	5.47
6.00	0.00	0.00	0.00	0.00	0.62
7.00	0.00	0.00	0.00	0.00	0.19
8.00	0.00	0.00	0.00	0.00	1.00
9.00	22.87	15.67	12.06	18.30	17.02
10.00	18.29	12.53	9.65	14.64	13.47
11.00	4.57	3.13	2.41	3.66	3.32
12.00	0.00	0.00	0.00	0.00	0.49
13.00	0.00	0.00	0.00	0.00	0.00
14.00	0.00	0.00	0.00	0.00	0.10
15.00	0.00	0.00	0.00	0.00	0.29
16.00	13.72	9.40	7.23	10.98	9.67
17.00	18.29	12.53	9.65	14.64	12.37
18.00	13.72	9.40	7.23	10.98	9.34
19.00	0.00	0.00	0.00	0.00	0.03
20.00	0.00	0.00	0.00	0.00	0.16
21.00	0.00	0.00	0.00	0.00	0.23
22.00	0.00	0.00	0.00	0.00	0.00
23.00	0.00	0.00	0.00	0.00	0.00
30/8/73 0.00	0.00	0.00	0.00	0.00	0.00

Table 5B : Average rainfall in mm over the sub-catchments for August 75 storm

Time (hrs.)	Average rainfall in mm over the sub-catchments				
	Manot A	Burhner B	Banjar C	Local D	Local E
10/8/75 1.00	3.60	5.60	8.00	8.00	3.44
2.00	3.60	5.60	8.00	8.00	4.28
3.00	4.05	6.30	9.00	9.00	5.88
4.00	5.54	7.11	9.00	9.00	5.76
5.00	8.34	10.91	14.00	14.00	6.56
6.00	4.38	6.48	9.00	9.00	6.96
7.00	3.60	5.37	7.50	7.50	6.60
8.00	0.39	0.44	0.50	0.50	2.48
9.00	0.72	0.39	0.00	0.00	0.84
10.00	0.11	0.06	0.00	0.00	0.72
11.00	3.58	1.95	0.00	0.00	0.00
12.00	0.55	0.30	0.00	0.00	0.00
13.00	0.00	0.00	0.00	0.00	0.00
14.00	0.00	0.00	0.00	0.00	0.00
15.00	0.00	0.00	0.00	0.00	0.00
16.00	0.00	0.00	0.00	0.00	0.00
17.00	0.00	0.00	0.00	0.00	0.00
18.00	9.24	5.04	0.00	0.00	0.00
19.00	0.17	0.09	0.00	0.00	0.00
20.00	0.00	0.00	0.00	0.00	0.00
21.00	0.00	0.00	0.00	0.00	0.00
22.00	0.00	0.00	0.00	0.00	0.00
23.00	0.00	0.00	0.00	0.00	0.00
11/8/75 0.00	0.00	0.00	0.00	0.00	0.00

Table 5C : Average rainfall in mm over the sub-catchments for August 77 storm

Time (hrs.)	Average rainfall in mm over the sub-catchments				
	Manot A	Burhner B	Banjar C	Local D	Local E
6/8/77 1.00	0.00	0.00	0.00	0.00	0.08
2.00	0.45	0.43	0.54	0.41	0.33
3.00	0.20	0.19	0.24	0.18	0.15
4.00	0.00	0.00	0.00	0.00	0.60
5.00	0.00	0.00	0.00	0.00	0.44
6.00	0.00	0.00	0.00	0.00	0.40
7.00	0.05	0.05	0.06	0.05	0.04
8.00	0.10	0.10	0.12	0.09	0.07
9.00	0.00	0.00	0.00	0.00	0.00
10.00	0.00	0.00	0.00	0.00	0.20
11.00	2.61	2.48	3.11	2.35	2.14
12.00	0.05	0.05	0.06	0.05	0.16
13.00	0.15	0.14	0.18	0.14	2.12
14.00	0.00	0.00	0.00	0.00	0.00
15.00	0.00	0.00	0.00	0.00	0.00
16.00	0.00	0.00	0.00	0.00	0.00
17.00	0.00	0.00	0.00	0.00	0.00
18.00	0.45	0.43	0.54	0.41	0.73
0.00	1.01	0.95	1.20	0.90	1.82
20.00	1.11	1.05	1.32	1.00	2.86
21.00	2.16	2.05	2.58	1.95	2.82
22.00	9.80	9.30	11.68	8.82	8.52
23.00	4.02	3.81	4.79	3.62	4.93
7/8/77 0.00	8.80	8.34	10.48	7.92	10.01
1.00	18.63	23.44	16.22	27.02	14.71
2.00	6.89	8.67	6.00	10.00	4.82
3.00	11.65	14.67	10.14	16.91	12.44
4.00	8.93	11.24	7.78	12.96	15.40

5.00	4.68	5.89	4.07	6.79	10.99
6.00	1.70	2.14	1.48	2.47	1.80
7.00	9.44	11.88	8.22	13.70	10.19
8.00	0.00	0.00	0.00	0.00	1.99
9.00	0.00	0.00	0.00	0.00	0.00
10.00	0.00	0.00	0.00	0.00	0.00
11.00	0.00	0.00	0.00	0.00	0.00
12.00	0.00	0.00	0.00	0.00	0.00
13.00	0.00	0.00	0.00	0.00	0.16
14.00	0.00	0.00	0.00	0.00	0.22
15.00	1.96	2.46	1.70	2.84	1.65
16.00	0.00	0.00	0.00	0.00	0.00
17.00	0.00	0.00	0.00	0.00	0.00
18.00	0.00	0.00	0.00	0.00	0.00
19.00	0.00	0.00	0.00	0.00	0.00
20.00	0.00	0.00	0.00	0.00	0.00
21.00	0.00	0.00	0.00	0.00	0.00
22.00	0.00	0.00	0.00	0.00	0.00
23.00	0.00	0.00	0.00	0.00	0.00
8/8/77 0.00	0.00	0.00	0.00	0.00	0.00
1.00	0.00	0.00	0.00	0.00	0.00
2.00	0.00	0.00	0.00	0.00	0.00
3.00	1.96	1.96	1.96	1.96	1.96
4.00	0.00	0.00	0.00	0.00	0.00
5.00	0.00	0.00	0.00	0.00	0.00
6.00	0.00	0.00	0.00	0.00	0.00
7.00	0.00	0.00	0.00	0.00	0.00
8.00	0.00	0.00	0.00	0.00	0.00
9.00	0.00	0.00	0.00	0.00	0.00
10.00	0.00	0.00	0.00	0.00	0.00
11.00	0.00	0.00	0.00	0.00	0.00

12.00	0.00	0.00	0.00	0.00	0.00
13.00	0.00	0.00	0.00	0.00	0.00
14.00	0.00	0.00	0.00	0.00	0.00
15.00	1.96	1.96	1.96	1.96	1.96
16.00	0.00	0.00	0.00	0.00	0.00
17.00	0.00	0.00	0.00	0.00	0.00
18.00	0.00	0.00	0.00	0.00	0.00
19.00	0.00	0.00	0.00	0.00	0.00
20.00	0.00	0.00	0.00	0.00	0.00
21.00	0.00	0.00	0.00	0.00	0.00
22.00	0.00	0.00	0.00	0.00	0.00
23.00	0.00	0.00	0.00	0.00	0.00
9/8/77 0.00	0.00	0.00	0.00	0.00	0.00

Table 5D : Average rainfall in mm over the sub-catchments for August 78 storm

Time (hrs.)	Average rainfall in mm over the sub-catchments			
	Manot A	Burhner B	Banjar C	Local D&E
27/8/78 1.00	0	0	0	0
2.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00
4.00	0.00	0.00	0.00	0.00
5.00	0.00	0.00	0.00	0.00
6.00	0.00	0.00	0.00	0.00
7.00	0.00	0.00	0.00	0.00
8.00	0.00	0.00	0.00	0.00
9.00	0.00	0.00	1.12	0.25
10.00	0.00	0.00	5.03	1.27
11.00	0.00	0.00	0.00	0.00
12.00	0.00	0.00	0.11	6.91
13.00	0.00	0.00	0.00	6.48

14.00	0.00	0.00	0.00	0.17
15.00	0.00	0.00	0.00	0.17
16.00	0.00	0.00	5.47	1.40
17.00	0.00	0.00	0.11	0.12
18.00	0.00	0.00	0.00	0.23
19.00	0.00	0.00	0.00	0.26
20.00	0.00	0.00	0.00	0.02
21.00	0.00	0.00	0.00	0.00
22.00	0.00	0.00	0.00	0.00
23.00	0.00	0.00	0.00	0.00
28/8/78 0.00	0.00	0.00	0.00	0.00
1.00	0.00	0.00	0.00	0.00
2.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00
4.00	0.00	0.00	0.00	0.00
5.00	0.00	0.98	1.48	0.84
6.00	0.00	0.00	0.09	0.00
7.00	0.00	0.00	0.28	0.00
8.00	0.00	0.89	1.35	0.77
9.00	0.00	0.00	2.89	1.35
10.00	0.00	0.00	0.00	0.13
11.00	0.00	5.20	7.90	4.48
12.00	0.00	2.64	4.00	2.25
13.00	0.00	0.00	0.00	1.35
14.00	0.00	0.00	0.00	0.10
15.00	0.00	0.00	0.00	0.30
16.00	0.00	0.00	0.00	0.04
17.00	0.00	0.10	0.19	0.09
18.00	0.00	0.04	1.73	0.20
19.00	0.74	0.10	0.00	0.00
20.00	0.30	0.04	0.00	0.14

21.00	0.74	0.10	0.00	0.22
22.00	3.57	0.52	0.03	0.02
23.00	0.45	0.08	0.44	0.02
29/8/78 0.00	0.00	0.29	0.60	0.25
1.00	4.16	2.39	3.27	1.58
2.00	1.87	1.13	4.36	0.30
3.00	2.69	1.17	2.48	0.16
4.00	1.17	1.48	3.94	0.68
5.00	0.00	0.85	2.60	1.64
6.00	0.00	0.00	0.06	2.70
7.00	0.82	0.27	0.03	0.91
8.00	0.35	0.12	0.40	0.02
9.00	0.00	0.04	0.03	1.11
10.00	0.00	0.00	0.14	1.07
11.00	0.58	0.19	0.11	0.38
12.00	4.91	1.63	0.29	0.53
13.00	0.94	0.31	0.00	0.61
14.00	0.00	0.00	0.00	1.36
15.00	2.34	0.78	0.14	0.00
16.00	0.23	0.08	0.31	0.00
17.00	0.00	0.00	0.00	0.00
18.00	0.00	0.00	0.00	0.00
19.00	0.00	0.00	0.06	0.00
20.00	0.23	0.08	0.09	0.03
21.00	0.12	0.04	0.20	0.12
22.00	0.23	0.08	0.60	0.06
23.00	0.00	0.00	0.29	0.00
30/8/78 0.00	2.34	1.05	0.37	0.16
1.00	6.67	2.21	0.09	0.00
2.00	0.17	0.16	0.21	0.11
3.00	0.15	0.17	0.21	0.27

4.00	0.07	0.06	0.11	0.00
5.00	0.07	0.06	0.43	0.00
6.00	0.44	0.41	0.00	0.05
7.00	0.43	0.41	0.96	0.08
8.00	0.02	0.02	1.70	0.00
9.00	0.02	0.03	0.11	0.08
10.00	0.05	0.06	1.60	0.08
11.00	0.03	0.08	1.60	0.52
12.00	0.02	0.02	2.13	0.00
13.00	0.14	0.13	0.32	0.00
14.00	1.52	1.39	0.21	0.00
15.00	0.72	0.73	0.53	0.82
16.00	0.22	0.39	1.28	1.22
17.00	0.90	0.56	0.21	0.27
18.00	0.00	0.00	0.11	0.35
19.00	0.00	0.00	0.00	0.05
20.00	0.00	0.00	0.11	0.00
21.00	0.00	0.00	0.43	0.00
22.00	0.02	0.02	0.11	0.00
23.00	0.00	0.00	0.21	0.00
31/8/78 0.00	0.00	0.00	0.00	0.05
1.00	0.02	0.02	0.11	0.00
2.00	0.00	0.00	0.00	0.00
3.00	0.00	0.00	0.00	0.00
4.00	0.00	0.00	0.00	0.00
5.00	0.00	0.00	0.00	0.00
6.00	0.00	0.00	0.00	0.00
7.00	0.00	0.00	0.44	0.00
8.00	0.00	0.00	0.44	0.00
9.00	0.00	0.00	0.00	0.00
10.00	0.00	0.00	1.11	0.00

11.00	0.00	0.00	0.00	0.00	0.00
12.00	0.00	0.00	0.00	0.00	0.00
13.00	0.00	0.00	0.00	0.00	0.00
14.00	0.00	0.00	0.00	0.00	0.00
15.00	0.00	0.00	0.00	0.00	0.00
16.00	0.00	0.00	0.00	0.00	0.00
17.00	0.00	0.00	0.00	0.00	0.00
18.00	0.00	0.00	0.00	0.00	0.00
19.00	0.00	0.00	0.00	0.00	0.00
20.00	0.00	0.00	0.00	0.00	0.00
21.00	0.00	0.00	0.00	0.00	0.00
22.00	0.00	0.00	0.00	0.00	0.00
23.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00

Table 5E : **Average rainfall in mm over the sub-catchments for August 79 storm**

Time (Hrs.)	Average rainfall in mm over the sub-catchments				
	Manot A	Burhner B	Banjar C	Local D	Local E
8/8/79 0.00	0.11	0.07	0.03	0.12	0.05
1.00	2.25	1.48	0.71	2.42	1.07
2.00	0.59	0.46	0.39	0.63	0.28
3.00	1.02	0.65	0.27	1.09	0.99
4.00	1.29	0.82	0.33	1.38	5.70
5.00	1.98	1.37	0.81	2.13	2.28
6.00	2.41	1.57	0.71	2.59	7.46
7.00	1.34	1.09	0.99	1.44	1.64
8.00	2.41	1.57	0.71	2.59	1.15
9.00	4.82	3.18	1.54	5.18	2.30
10.00	0.70	0.59	0.57	0.75	0.33
11.00	0.11	0.13	0.21	0.12	0.05

12.00	0.05	0.31	0.78	0.06	0.03
13.00	1.72	1.23	0.82	1.84	0.82
14.00	2.31	1.92	1.83	2.48	1.10
15.00	2.95	2.32	1.99	3.17	1.40
16.00	2.68	2.53	2.97	2.88	1.28
17.00	4.29	3.17	2.32	4.61	2.04
18.00	6.38	4.63	3.21	6.85	3.04
19.00	8.63	6.40	4.73	9.27	15.56
20.00	7.83	6.48	6.15	8.41	26.63
21.00	0.05	1.27	3.45	0.06	3.10
22.00	0.00	1.11	3.09	0.00	0.00
23.00	0.00	0.31	0.87	0.00	0.00
9/8/79 0.00	1.03	4.52	11.54	1.46	0.34
1.00	0.51	4.91	15.85	0.73	0.17
2.00	0.51	3.66	11.08	0.73	0.17
3.00	0.51	2.83	7.93	0.73	0.64
4.00	5.66	11.47	12.54	8.03	1.85
5.00	2.06	6.05	11.71	2.92	0.67
6.00	0.00	1.35	5.13	0.00	0.47
7.00	0.00	0.42	1.61	0.00	0.94
8.00	0.00	0.06	0.22	0.00	0.00
9.00	0.00	0.04	0.15	0.00	0.00
10.00	6.69	11.20	5.89	9.49	2.19
11.00	2.06	3.45	1.81	2.92	0.67
12.00	4.12	6.89	3.63	5.84	1.35
13.00	0.00	0.12	0.44	0.00	0.00
14.00	0.00	0.08	0.29	0.00	0.00
15.00	0.00	0.40	1.54	0.00	16.97
16.00	0.00	0.02	0.07	0.00	6.60
17.00	2.57	4.35	2.41	3.65	1.31
18.00	1.54	2.62	1.51	2.19	2.39

19.00	3.60	6.05	3.25	5.11	1.18
20.00	3.60	6.17	3.69	5.11	1.18
21.00	1.54	2.76	2.02	2.19	0.51
22.00	5.66	9.58	5.35	8.03	1.85
23.00	2.06	3.50	2.03	2.92	0.67
10/8/79 0.00	1.95	2.96	3.03	2.09	0.36
1.00	1.30	1.49	0.61	1.39	0.24
2.00	1.95	2.24	0.91	2.09	0.36
3.00	0.00	0.00	0.00	0.00	0.00
4.00	0.00	0.00	0.00	0.00	0.00
5.00	0.00	1.44	4.23	0.00	0.39
6.00	0.00	0.00	0.00	0.00	1.56
7.00	0.00	0.00	0.00	0.00	0.39
8.00	0.00	0.00	0.00	0.00	0.20
9.00	0.00	0.00	0.00	0.00	0.00
10.00	0.00	0.00	0.00	0.00	0.00
11.00	1.95	2.24	0.91	2.09	0.36
12.00	0.00	0.00	0.00	0.00	0.00
13.00	0.00	0.00	0.00	0.00	0.00
14.00	0.00	0.00	0.00	0.00	0.00
15.00	0.00	0.00	0.00	0.00	0.00
16.00	0.00	0.00	0.00	0.00	0.00
17.00	0.00	0.00	0.00	0.00	0.00
18.00	0.00	0.00	0.00	0.00	0.00
19.00	0.00	0.00	0.00	0.00	0.00
20.00	0.00	0.00	0.00	0.00	0.00
21.00	0.00	0.00	0.00	0.00	0.00
22.00	0.00	0.00	0.00	0.00	0.00
23.00	0.00	0.00	0.00	0.00	0.00

METHODOLOGY

In this study the Narmada catchment up to Jamtara has been assumed as ungauged and using the rainfall record, physical properties of the catchment and channel characteristics, flood hydrograph at Jamtara is computed. The whole area up to Jamtara is sub-divided into five sub-catchments A to E as shown in Figure 1. To start with, flood hydrograph for each sub-catchment for each event is computed using the GIUH approach which makes use of physical characteristics of the sub-catchment. These computed flood hydrographs are routed through the respective river reaches to compute the final flood hydrograph at Jamtara.

The network adopted in the study

The network adopted for the whole area is shown in Figure 3. As shown in this Figure, first flood hydrographs for sub-catchments A (Manot) and B (Burhner) are computed. Flood hydrograph of sub-catchment A is routed through a river reach of length 15 km. up to Basania. At Basania, routed flood hydrograph of A is combined with flood hydrograph of B and combined flood hydrograph is routed through a river reach of length 35 km. from Basania to Mandla. Flood hydrograph of sub-catchment C (Banjar) and D (Local) are computed and flood hydrograph of C is routed through a river reach of length 10 km. up to Mandla. At Mandla this routed hydrograph, flood hydrograph of D and routed flood hydrograph of A and B are combined and routed through a river reach of length 110 km. from Mandla to Jamtara. At Jamtara contribution of sub-catchment E (Local) is computed and is added to the routed hydrograph from Mandla to Jamtara to get the final flood hydrograph at Jamtara. All these steps are clearly marked in Figure 3.

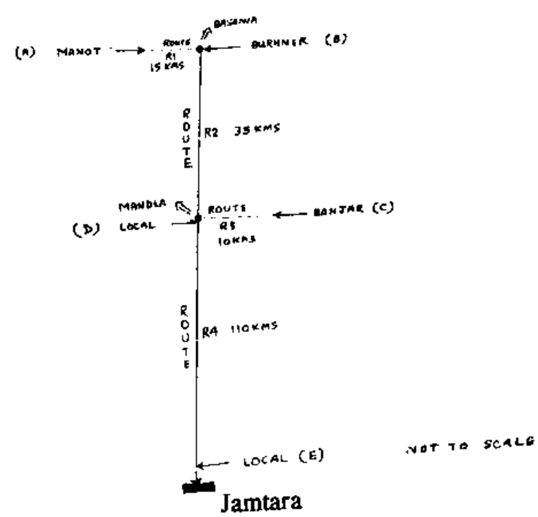


Figure 3 : Network for Narmada catchment up to Jamtara as adopted in the study

Development of unit hydrograph using GIUH approach

Rodriquez-Iturbe and Valdes (1979) first introduced the concept of geomorphologic instantaneous unit hydrograph, which led to the renewal of research in hydrogeomorphology. The expression derived by them yields full analytical, but complicated, expressions for the instantaneous unit hydrograph. They suggested that it is adequate to assume a triangular instantaneous unit hydrograph and only specify the expressions for the time to peak and peak value of the IUH. These expressions are obtained by regression of the peak as well as time to peak of IUH, derived from the analytic solutions for a wide range of parameters with that of the geomorphologic characteristics and flow velocities.

The expressions are given as:

$$q_p = 1.31\ R_L^{0.43}\, V / L_\Omega \qquad \ldots(1)$$

$$t_p = 0.44 (L_\Omega / V)(R_B / R_A)^{0.55} (R_L)^{-0.38} \qquad \ldots(2)$$

where;

L_Ω = the length in kilometers of the main stream

V = the expected peak velocity, in m/sec.

q_p = the peak flow, in units of inverse hours

t_p = the time to peak, in hours

R_B, R_L, R_A = the bifurcation, length and area ratios given by the Horton's laws of stream numbers, lengths and areas respectively.

Empirical results indicate that for natural basins the values for R_B normally ranges from 3 to 5, for R_L from 1.5 to 3.5 and for R_A from 3 to 6 [Smart (1972)].

On multiplying eq. (1) and (2) we get a non-dimensional term q_p x t_p as under.

$$q_{p} \times t_{p} = 0.5764 (R_B / R_A)^{0.55} (R_L)^{0.05} \qquad \ldots(3)$$

This term is not dependent upon the velocity and thereby on the storm characteristics and hence is a function of only the catchment characteristics. This is also apparent from the expression given above.

For the dynamic parameter velocity (V) , Rodriquez et. al. (1979) in their studies assumed that the flow velocity at any given moment during the storm can be taken as constant throughout the basin. The characteristic velocity for the basin as a whole changes throughout as the storm progresses. For the derivation of GIUH, this can be taken as the velocity at the peak discharge time for a given rainfall-runoff event in a basin.

Application of GIUH to sub-catchments of Narmada upto Jamtara

Computation of geo-morphological parameters

For application of GIUH approach, catchment area, time area diagram, R_B,R_L,R_A ratios and velocity V are need to be known. These geomorphological characteristics for a basin, other than the velocity may easily be derived using a Geographical Information System (GIS). GIS is a computer based system for storage, retrieval, manipulation, analysis and display of spatial and associated attributes of a catchment. The input to a GIS may be remotely sensed data, digital models of the terrain, or point or aerial data compiled in the forms of maps, tables or reports. GIS provide a digital representation of watershed characterisation used in hydrologic modelling. Hydrological modelling is one of the most important application of a GIS system.

Computation of the parameters required for geomorphologic study using manual methods like area measurement using dot grid method or using planimeter and length measurement using curvimeter are very tedious, time consuming and also subjected to manual error. On the other hand, by using a GIS, one has the detailed measurements available on the computer within no time and the scope of the manual error is thus brought to a minimum level. In the present work the stream ordering, calculation of various geomorphological characteristics like numbers, lengths, areas of each order are found using GIS technique. Use of GIS has not only made this task relatively easy but accurate as well. ILWIS package is used in this study because of its versatility, efficiency in digitizing and attribute entry, editing capabilities etc.

Various geomorphologic parameters calculated for five sub-catchments using ILWIS package are shown in Table 6. To calculate the average stream area corresponding to a stream order, graphs between stream order and log of average stream length, stream order and log of stream numbers are plotted and slope of the best fit lines passing through these points is computed. For stream number this slope is nothing but R_B, while for stream length it is R_L. To compute average area for each stream order, the relationship between area and length in terms of Horton's laws of drainage-network composition, as proposed by Hack (1957) has been used. The relationship relates the area A_u of a catchment of order u with the bifurcation ratio R_B and $R_{LB,}$ the ratio of length ratio to bifurcation ratio as follows.

$$A_u = \bar{A}_1 R_B^{u-1} \frac{R_{LB}^u - 1}{R_{LB} - 1} \qquad \ldots(4)$$

Finally a graph between stream order and average area is plotted and the slope of the best fit line passing through these points is computed (R_A).

Table 6: Parameters R_B, R_L and R_A of sub-catchments of Narmada up to Jamtara

Sub-catchment	Area (sq.km.)	Length in km.	R_B	R_L	R_A
Manot A	4980	239	3.98	2.15	4.2
Burhner B	4103	138	3.52	1.79	3.94
Banjar C	3472	185	4.45	2.39	4.8
Local D	375	35	3.49	1.78	4.01
Local E	5812	110	3.5	1.69	3.95

Estimation of velocity

For ungauged catchments like the present case, the peak discharge is not known and so the criteria for estimation of velocity based on peak discharge cannot be applied. For the present case, for each sub-basin and for each event peak discharge (Q) in cumec is estimated based on rainfall excess intensity (i) in mm/hr and catchment area (CA) in sq. km. using the following relationship:

$$Q = 0.2778 \times i \times CA \qquad \ldots(5)$$

As in the present study, loss rate of 1 mm/hr has been assumed, in the above equation i, the rainfall excess intensity is equal to actual rainfall intensity minus 1 mm/hr.

Now, based on the cross sectional details of Narmada river and its tributaries, at the outlet of each sub-catchment, using the values of peak discharge as computed above, bed width, value of Manning's roughness coefficient as given in Table 1, and considering the slope of water profile as equivalent to bed slope, velocity for each sub-catchment for each storm is computed. These values are given in Table 7.

Table 7: Values of velocity, maximum intensity and product of tp & qp for storms considered in the analysis

Sub catchment	August 1978 storm			August 1973 storm			August 1975 storm			August 1977 storm			August 1979 storm		
	Velocity in m/sec	Intensity in mm/hr	Product of tp x qp	Velocity in m/sec	Intensity in mm/hr	Product of tp x qp	Velocity in m/sec	Intensity in mm/hr	Product of tp x qp	Velocity in m/sec	Intensity in mm/hr	Product of tp x qp	Velocity in m/sec	Intensity in mm/hr	Product of tp x qp
Manot A	3.97	5.67	0.58	6.82	21.87	0.58	4.61	8.24	0.58	6.26	17.63	0.58	4.47	7.63	0.58
Burhner B	4.93	4.20	0.56	8.64	17.07	6.26	6.95	9.91	0.56	9.64	22.44	0.56	7.02	10.47	0.56
Banjar C	4.10	6.90	0.58	5.95	17.48	0.58	5.28	13.00	0.58	5.63	15.22	0.58	5.57	14.85	0.58
Local D	3.04	5.91	0.57	4.77	22.50	0.57	3.05	13.00	0.55	4.06	26.02	0.57	4.20	8.49	0.57
Local E	3.06	5.91	0.58	4.55	16.02	0.58	3.07	5.96	0.55	4.36	14.40	0.58	4.50	25.63	0.58

Application of Clark model to get shape of UH

Using the values of various geomorphological parameters and velocity, time to peak and peak ordinates of instantaneous unit hydrograph are computed. To get the shape of the IUH, a Clark model is fitted in such a way so that IUH of Clark model and IUH of GIUH give similar peak discharges, time to peak and product of time to peak and peak discharge. Values of parameters of Clark model T_c and R so obtained are tabulated in Table 8.

Table 8: **Values of Clark model parameters time of concentration, T_c and storage coefficient, R for each sub-catchment and for each storm considered in the analysis**

Sub-catchment	August 1978 storm		August 1973 storm		August 1975 storm		August 1977 storm		August 1979 storm	
	Time of concentration (T_c) Hrs.	Storage coefficient (R) Hrs.	Time of concentration (T_c) Hrs.	Storage coefficient (R) Hrs.	Time of concentration (T_c) Hrs.	Storage coefficient (R) Hrs.	Time of concentration (T_c) Hrs.	Storage coefficient (R) Hrs.	Time of concentration (T_c) Hrs.	Storage coefficient (R) Hrs.
Manot A	19.12	25.55	11.23	15.15	16.40	22.20	12.21	16.35	16.95	22.59
Burhner B	9.18	12.93	5.34	7.52	6.52	8.92	4.78	6.61	5.87	7.92
Banjar C	13.53	18.82	9.44	13.02	10.63	14.82	9.83	13.75	9.93	13.84
Local D	3.68	5.37	2.54	3.48	3.57	5.28	2.53	3.48	2.46	3.44
Local E	12.09	16.78	8.02	11.17	11.95	16.71	8.51	12.06	8.29	11.35

Computation of excess rainfall

Average excess rainfall of each sub-catchment is applied on the computed unit hydrograph to get the flood hydrograph. To determine the excess rainfall, the initial loss and infiltration and other losses are subtracted from the storm rainfall. Since all the storms considered are late monsoon storms, the initial losses are assumed to be zero considering saturated soil conditions to prevail at the time of occurrence of storm. It has been found that for these sub-catchments uniform loss rates generally varies between 0.5 to 1.0 mm per hours. Keeping in view the recommendations of the CWC in its Manual, in the analysis, uniform losses are taken at a constant rate of 1.0 mm/hour.

Consideration of base flow

Since the base flow is determined by the antecedent rainfall conditions, after examining the past historical floods, literature and results of other studies, it is decided to add base flow at Jamtara only. Thus, a value of 1500 cumec has been assumed as base flow at Jamtara.

Routing of flood hydrograph

The routing equation for conventional Muskingum method is given as:

$$O_{n+1} = C_o I_{n+1} + C_1 I_n + C_2 O_n \qquad \text{....(6)}$$

which, C_o , C_1 and C_2 are routing coefficients defined in terms Δt, K, and X as follows:

$$C_o = \frac{(\Delta t / K) - 2X}{2(1-X) + (\Delta t / K)} \qquad \text{....(7)}$$

$$C_1 = \frac{(\Delta t / K) + 2X}{2(1-X) + (\Delta t / K)} \qquad \text{....(8)}$$

$$C_2 = \frac{2(1-X) - (\Delta t / K)}{2(1-X) + (\Delta t / K)} \qquad \text{....(9)}$$

where, I = Inflow, O = Outflow, K = a time constant or storage coefficient and X = a dimensionless weighting factor.

The Muskingum method can calculate runoff diffusion, ostensibly, by varying the parameter X. A numerical solution of linear kinematic wave equation using a third order - accurate scheme (Courant number C = 1) leads to pure flood hydrograph translation. Other numerical solutions to the linear kinematic wave equation invariably produce a certain amount of numerical diffusion and/or dispersion. The Muskingum and linear kinematic wave routing equation are strikingly similar. Further, unlike the kinematic wave equation, the diffusion wave equation does have the capability to describe the physical diffusion.

From these propositions, Cunge (1969) concluded that the Muskingum method is essentially a linear kinematic wave solution and that the flood wave attenuation shown by the calculation is due to the numerical diffusion of the scheme itself. He discretized the kinematic wave equation on the x-t plane (Figure.4) in such a way that parallels the Muskingum method to prove this assertion and came out with a physically based alternative to the Muskingum method. The alternative method is popularly known as Muskingum Cunge method.

Muskingum Cunge method

The kinematic wave equation is given as:

$$\partial Q/\partial t + c\ \partial Q/\partial x = 0.0 \qquad \text{...(10)}$$

in which, c is the kinematic wave celerity and Q is the discharge.

Eq.(10) was discretized by Cunge (1969) on the x-t plane (Figure. 4) in a way that parallels the Muskingum method, wherein the spatial derivative was centred and the temporal derivative was off-centered by means of a weighting factor X. The resulting equation is given as:

$$\frac{X\,(Q_j^{n+1} - Q_j^{n}) + (1-X)\,(Q_{j+1}^{n+1} - Q_{j+1}^{n})}{\Delta t} + c\,\frac{(Q_{j+1}^{n} - Q_j^{n}) + (Q_{j+1}^{n+1} - Q_j^{n+1})}{2\Delta x} = 0 \qquad \text{....(11)}$$

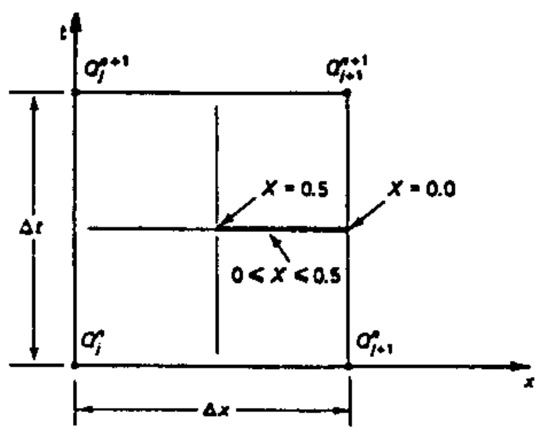


Figure 4: Space-time discretization of Kinematic wave equation paralleling Muskingum Method

Solving Eq.(11) for the unknown discharge leads to the following equation:

$$Q_{j+1}^{n+1} = C_0\,Q_j^{n+1} + C_1\,Q_j^{n} + C_2\,Q_{j+1}^{n} \qquad \text{....(12)}$$

The routing coefficients are:

$$C_0 = \frac{c\,(\Delta t/\Delta x) - 2X}{2(1-X) + c\,(\Delta t/\Delta x)} \qquad \text{....(13)}$$

$$C_1 = \frac{c\ (\Delta t/\Delta x) + 2X}{2(1-X) + c\ (\Delta t/\Delta x)} \qquad ....(14)$$

$$C_2 = \frac{2(1-X) - c\ (\Delta t/\Delta x)}{2(1-X) + c\ (\Delta t/\Delta x)} \qquad ....(15)$$

By defining:

$$K = \Delta x/c \qquad ....(16)$$

it is seen that the two sets of Eqs.(14) to (16) and (7) to (9) are the same.

Eq.(16) confirms that K is in fact the flood wave travel time, i.e. the time taken for a given discharge to travel the reach length ΔX with the kinematic celerity c. In a linear mode, c is constant and equal to a reference value, and in non-linear mode, it varies with discharge.

The hydraulic diffusivity (υ_h) which is a characteristic of flow and channel is defined as:

$$\upsilon_h = \frac{Q_o}{2TS_o} = \frac{q_o}{2S_o} \qquad ....(17)$$

in which, $q_o = Q_o/T$ is the reference flow per unit channel width.

A unique feature of the Muskingum method is the grid independence of the calculated outflow hydrograph. If numerical dispersion minimized (keeping Courant number C close to one), the calculated outflow at the downstream end of a channel reach will be essentially the same regardless of how many sub-reaches are used in the computation. This is because X is a function of Δx and the routing co-efficients C_o , C_1 and C_2 vary with reach length.

An improved version of the Muskingum-Cunge method is due to Ponce and Yevjevich (1978). The Courant number, C, is defined as the ratio of wave celerity (c) to grid celerity $\Delta X/\Delta t$ i.e.

$$C = c\ \Delta t/\Delta x \qquad ....(18)$$

The grid diffusivity is defined as the numerical diffusivity as:

$$u_g = c\Delta x/2 \qquad ....(19)$$

The Cell Reynolds number (Roache, P., 1972) is defined as the ratio of hydraulic diffusivity (Eq.17) to grid diffusivity (Eq.19). This leads to

$$D = q_o/S_o\, c\Delta x \qquad ....(20)$$

in which, D = Cell Reynolds number.

Re-writing eqn. (13), (14) and (15) to express routing co-efficients in terms of Courant and Cell Reynolds numbers:

$$C_o = \frac{-1 + C + D}{1 + C + D} \qquad ....(21)$$

$$C_1 = \frac{1 + C - D}{1 + C + D} \qquad ....(22)$$

$$C_2 = \frac{1 - C + D}{1 + C + D} \qquad ....(23)$$

Thus C and D are the two routing parameters required to be estimated for Muskingum-Cunge method.

Estimation of Routing Parameters

(a) Estimation of parameter C (Courant number)

The parameter C can be estimated using Eq.(18). It requires an estimate for wave celerity (c) in addition to grid size (Δx, Δt). The wave celerity can be calculated with either

$$c = \beta v \qquad ....(24)$$

or

$$c = 1/T\, dQ/dy \qquad ....(25)$$

Where, v is the average flow velocity; T is the top width; and β is an exponent in the discharge (Q) area (A) rating equation given as

$$Q = \alpha (A)^{\beta} \quad \text{....(26)}$$

The calculation of β is a function of frictional type and cross sectional shape.

Theoretically, Eq. (24) and (25) are the same. For practical applications, if a stage-discharge rating and cross sectional geometry are available (i.e. stage - discharge - top width tables), Eq.(25) is preferred over Eq.(24) because it accounts directly for cross sectional shape. In the absence of a stage discharge rating and cross sectional data, Eq.(24) can be used to estimate flood wave celerity. The velocity υ in Eq.(24) can be taken as the velocity at reference flow. The choice of reference flow has bearing on the calculated results although the overall effect is likely to be small. The peak flow value has the advantage that it can be readily ascertained, although a better approximation may be obtained by using an average value.

(b) Estimation of parameter D (Cell Reynold Number)

Cell Reynold numbers (D) can be calculated using the reach length (Δx), reference discharge per unit width q_o, kinematic wave celerity (c), and bottom slope (S_o) in Eq.(20).

Application of Muskingum-Cunge method

The steps involved in flood routing through a channel reach using the Muskingum- Cunge Method are given as follows:

(i) Estimate the parameter C (Courant number) using the following equation:

$$C = c\, \Delta t / \Delta x \quad \text{....(27)}$$

The wave celerity c is computed using the procedure described earlier. The temporal and spatial resolutions (Δt and Δx) should be such that the routing co-efficient C_o should not be negative as well as the value of Courant number (C) should be close to one in order to minimise the numerical dispersion.

(ii) Estimate the parameter D (Cell Reynold number) using the following equation

$$D = q_o / S_o\, c.\Delta x \quad \text{....(28)}$$

The wave celerity (c) and reference discharge, q_o ($=Q_o/T$) per unit width are used together with channel slope S_o and reach length Δx in the above equation to provide the parameter D (Cell Reynold number). Ensure whether $-1 + C + D > 0$ which is the practical criterion to avoid the negative values of C_o in Muskingum Cunge routing.

(iii) Estimate the routing co-efficients C_o , C_1 and C_2 using eq. (21) to (23).

(iv) Route the inflow hydrograph (Q) using the following equation in order to have the outflow hydrograph (Q_{j+1}):

$$Q_{j+1}^{n+1} = C_o\ Q_j^{n+1} + C_1\ Q_j^n + C_2\ Q_{j+1}^n \quad ....(29)$$

(v) If the channel is divided into sub-reaches, the steps (i) to (iv) should be repeated for all the sub-reaches considering the outflow from the first sub-reach as inflow to second sub-reach and so on.

In the present study a computer programme is formulated and available information about the reach i.e. reach length, cross-section details, Manning's N values, bed slope etc. are supplied. As cross sectional details are available, Eq. (25) has been used to compute c. Computational interval is kept fixed equal to one hour and reach length is varied to have positive value of C_o. Computer programme automatically takes care of this requirement as well as other requirements.

RESULT DISCUSSIONS

Time to peak and peak discharge of observed and computed hydrographs at Jamtara for all events are given in Table 6. These hydrographs are shown in Figure 5. From the Table and Figure it is clear that computed flood hydrographs do not differ much from the observed one. Time to peak is almost same in all the cases. Also Peak ordinate of computed hydrographs is 93 %, 94%, 130%, 104% and 92% of peak ordinate of observed hydrograph for 1978, 1973, 1977, 1979 and 1975 storms respectively. Similarly when these hydrographs are compared on volume basis, it is observed that volume of computed hydrographs is 121%, 94%, 119%, 117% and 80% of volume of observed hydrograph for 1978, 1973, 1977, 1979 and 1975 storms respectively.

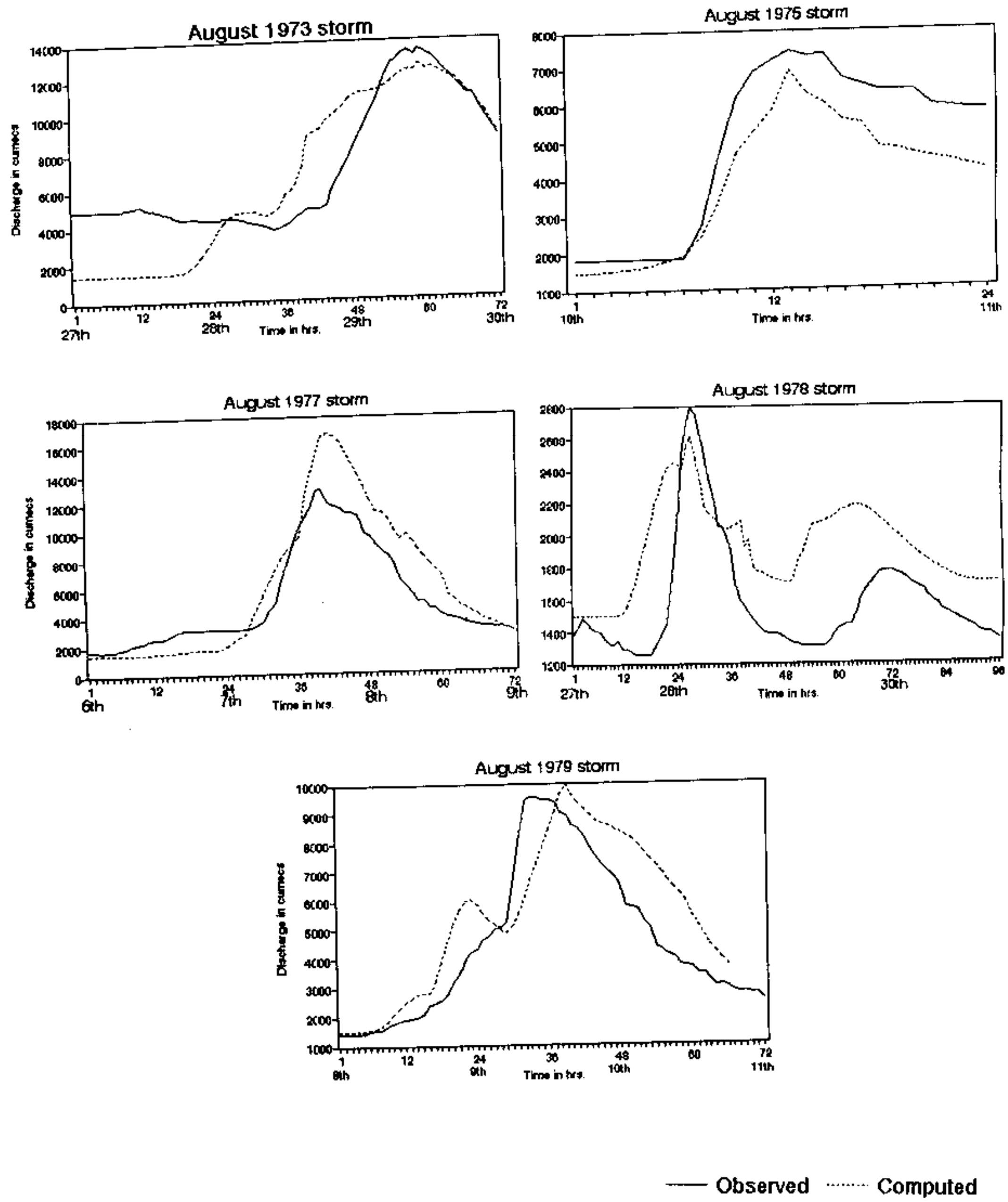


Figure 5: Observed and computed hydrographs at Jamtara for all events

Table 9: Time to peak and peak ordinates of observed and computed flood hydrographs at Jamtara for different storm events

Storm event	Observed Hydrograph		Computed Hydrograph	
	Time to peak in hrs.	Peak ordinate in cumec	Time to peak in hrs.	Peak ordinate in cumec
August 78 storm	27.00	2792.00	26.00	2610.00
August 73 storm	59.00	13480.00	58.00	12628.00
August 77 storm	40.00	12900.00	41.00	16819.00
August 79 storm	33.00	9550.00	39.00	9924.00
August 75 storm	13.00	7430.00	13.00	6871.00

The miss-match in the shape, time to peak and peak ordinate of the observed and computed hydrograph can be attributed to the following:

1. Hourly rainfall data was available at some stations only and therefore average rainfall used for the sub-catchment may not be truly representative of the conditions.
2. Cross sectional details were available at selected locations only. For other locations suitable values have been assumed. Moreover, in the analysis, cross-section of regular shape has been considered. Similarly, values of bed slope and Manning's N if not estimated correctly, may also influence the results.
3. Antecedent conditions prior to the on set of storm also affect the hydrograph's shape. Observed flood hydrograph used for comparison may be affected by the rainfall occurred earlier. These affects are not accounted far in the study.
4. Movement of storm may also affect shape of the hydrograph.
5. Velocity is computed based on the assumption that whole area is contributing. For large catchments this assumption may not be true.

CONCLUSIONS

In the absence of sufficient information, a method based on physical characteristics of catchment and channel is tried to get representative flood hydrograph for a large catchment.

Final flood hydrographs as computed using GIUH based approach and Muskingum-Cunge routing technique, seem to be satisfactory when compared with corresponding observed hydrographs.

Inspite of some limitations and assumptions which are discussed in the previous section, as in the present analysis, only physical characteristics of a catchment have been used, it can be very well applied to ungauged catchments for computation of flood hydrographs.

REFERENCES

Bernard, M. (1935). "An approach to determinate stream flow". Transactions of the American Society of Civil Engineers 100:347-62.

Clark, C.O.. (1945). "Storage and the unit hydrograph". Transactions of the American Society of Civil Engineers 110:1419-46.

Cunge, J.A. (1969). " On the subject of a flood propagation computation method (Muskingum Method)", J. Hydraulic Res., IAHR, 7(2), pp.205-230.

Gupta, V.K., C.T. Wang and E. Waymire. (1980). "A representation of an instantaneous unit hydrograph from geomorphology", Water Resource Research,, 16(5), 855-862, 1980.

Hack, J.T. (1957). "Studies of longitudional stream profiles in Virginia and Maryland", Geol. Surv. Profess. Paper 294-B.

Horton, R.E. (1945). "Erosional development of stream and their drainage basin : hydrophysical approach to quantitative morphology", Geol. Soc. Am. Bull., 56, 275-370, 1945.

Janusz Zelazinski. (1986)."Application of the geomorphological instantaneous unit hydrograph theory to development of forecasting models in Poland", Hydrological Sciences Journal,31,2,6/1986.

McCarthy, G.T. (1938). "The unit hydrograph and flood routing". Unpublished manuscript presented at a conference of the North Atlantic Division, June 24, U.S. Army Corps of Engineers.

Nash, J.E. (1960). "A unit hydrograph study with particular reference to British Catchments", Proc. Inst. Civil Engg., 17, 249-282, 1960.

Naional Institute of Hydrology. (1995). "Derivation of GIUH for small catchments of Upper Narmada and Tapi Subzone (Subzone 3C) - Part I", TR(BR)-132 NIH 1994-95, *National Institute of Hydrology, India.* .

National Institute of Hydrology. (1997). "Rainfall Runoff Modelling of Morel Catchment for Design Flood Estimation", *CS(AR)-2/97-98, National Institute of Hydrology, India.*

National Research Council (NRC) (1988). "Estimating probabilities of extreme floods, methods and recommended research", National Academy Press, washington, DC, 141 pp.

Panigrahi, R.K. (1991). "Derivation of Nash model parameters from geomorphological instantenouh unit hydrograph", M.E. Dissertation, DOH, UOR, Roorkee, India

Ponce, V.M. and V. Yevjevich (1978). "Muskingum-Cunge Method with variable Parameters", Journal of the Hydraulics Division, ASCE, Vol.104, No.HY12, December, pp.1663-1667.

Roache, P. (1972). "Computational Fluid Dynamics" Hermosa Publishers, New Mexico, Albuquerque.

Rodriguez-Iturbe, I., and J.B. Valdes. (1979). "The geomorphologic structure of hydrologic response", Water Resource Research, 15(6), 1409-1420, 1979.

Rodriguez-Iturbe, I., M. Gonzalas Sonabria, and R.C. Bras. (1982). "The Geomorpho-climatic theory of the instantaneous unit hydrograph", Water Resources Reserve., 18(4), 877-886, 1982.

Singh, V.P. (1988). " Hydrologic Systems - Vol.I Rainfall-runoff modelling", Perntice Hall, Englewood Cliffs, New Jersey, U.S.A.

Smart, J.S. (1972). "Statistical Properties of Stream Length", Water Resources Research 4(5). 1001-1014.

Snyder, F.F. (1938). "Synthetic unit graphs". Transactions of the American Geophysical Union 19:447-54.

Soil Conservation Service. (1971). Hydrology. SCS National Engineering Handbook, sec. 4. U.S. Department of Agriculture, Washington, D.C.

Strahler, A.N. (1964). "Quantitative geomorphology of drainage basins and channel networks", Section 4-II, In handbook of applied hydrology, edited by V.T.Cow, 4-39:4-76, New York: Mcgraw-Hill.

Taylor, A.B. and H.E. Schwarz. (1952). "United hydrograph lag and peak flow related to basin characteristics". Transactions of the American Geophysical Union 33:235-46.

U.S. Army Corps of Engineers (1960). "Routing of Floods through river channels", EM1110-2-1408.