LECTURE -10

FLOOD FORECASTING - DETERMINISTIC METHODS

BY

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OBJECTIVES

The objective of this lecture is to present procedures for real time flood forecasting using unit hydrograph based approaches includingmethods for updating the loss rate and unit hydrograph parameters during the period of forecasts.

1.0 INTRODUCTION

The unit hydrograph method has long been recognised as a useful tool for converting excess rainfall to direct surface runoff by linear transformation. The assumptions underlying this method and their limitations with regard to areal size, linearity and uniform spatial and temporal distribution of rainfall have been discussed in most of the text books and research papers. In brief, the main characteristics of the unit hydrograph are :

i) It gives the time distribution of the discharge hydrograph of a watershed produced by a uniform net rain of given depth precipitated on the area.

ii) It shows how this net rain is transformed into direct surface runoff at the outlet. This transformation is assumed to be a linear process.

iii) It is a characteristics for a given watershed, it shows the integrated effect of the surface features on the routing of the rain through the catchment.

One of the important areas in hydrology pertains to the study of the transformation of the time distribution of rainfall on the catchment to the time distribution of runoff. This transformation is studied by first relating the volume of rainfall to the volume of direct surface runoff, thus determining the time distribution of rainfall excess (the component responsible for direct surface runoff on the catchment) and then transforming it to the time distribution of direct runoff through a discrete or continuous mathematical model .The first step decides the volume of the input to the catchment and therefore any error in its determination is directly transmitted through the second step to the time distribution of direct runoff. A number of watershed conceptual models find this component for each time step through a number of stores representing various processes on the catchment. The parameters of these models including those in the functional relationship are determined from the historical record and their performance is tested by simulating some of the rainfall-runoff events which have not been used in the parameter estimating process. The models need to be run continuously so that the status of various stores is available at all times. One of the operational uses of these models is in the area of real time flood forecasting required for real time operation of the reservoir. In such a situation these models are run by inputting the rainfall and forecasts are issued assuming no rainfall beyond the time of forecast value of the rainfall in the future.

The infiltration part of these models and their context decide the volume of input. At the time of calculation the catchment is also performing the transformation operation to produce the direct runoff at the gauging station. Since the model is simulating the action of the catchment it would be appropriate to make use of this information in finding out the contribution which the rainfall is going to make to the direct runoff on the catchment. However, the complexity of these models does not lend itself to this exercise during the event. Of late methods based on unit hydrograph approach have been formulated for real time forecasting which overcome the difficulties associated with complex hydrological models.

The Hydrologic Engineering Centre (HEC) of the US Army Corps of Engineers, USA has developed a computer model HEC-1F, a modification of model HEC-1, for the purpose of real time forecasting. HEC- 1F model uses the unit hydrograph technique with constant loss rate to forecast the runoff. Forecasting by HEC-1F model is accomplished by re-estimating the unit hydrograph parameters and the loss rate parameters as additional rainfall runoff data are reported and using these updated parameters the future flows are estimated for forecasting. The Snyder's synthetic unit hydrograph described by two parameters is used as unit hydrograph model. For the estimation of the unit hydrograph and constant loss rate parameters, the model uses unvariate search technique. HEC(1984) provides the complete details of the model. In this lecture two different methods for real time forecasting, based on unit hydrograph approach, have been described.

2.0 METHODS BASED ON UNIT HYDROGRAPH APPROACH FOR REAL TIME FLOOD FORECASTING

2.1 Method: 1

Chander et al (1984) developed a Unit Hydrograph Based Forecast Model. This model uses the classical method of unit hydrograph in transforming rainfall excess to runoff. It recognises that rainfall over a catchment does not produce an immediate response by way of measurable increase in runoff at the gauging station and designates this initial delay as initial lag (T). The model uses first appreciable rise in hydrograph and estimates the ø-index value for the flood event. This value is updated as more record becomes available. At each step the estimate of ø-index is used to determine the rainfall excess, which in turn is convolved with the unit hydrograph of the catchment to forecast the resulting flood hydrograph. The rainfall 'T' hours prior to the rise in the hydrograph is considered to contribute to interception, depression storage and infiltration and is therefore not used in evaluating rainfall excess Mathematically the runoff is computed using the following equation:

$$Q_{i+T} = \sum_{j=1}^{i \le m} (P_{i-j+1} - F_i)\, U_j \qquad \ldots(1)$$

where

Q_i = direct runoff at time (i)

P_i = precipitation at time (i)

F_i = ø-index value at time (i)

U_j = Unit hydrograph ordinates. Unit time of UH is considered to be equal to the discrete time D of the eq. (1) (j=1,2 ... m)

m = number of unit hydrograph ordinates

F_i at each step is computed and updated using eqn. (2) as under:

$$\Phi_i = \frac{\sum_{j=1}^{i} \sum_{k=1}^{i} (P_{j-k+1} U_k - Q_{j+T}) \quad (\sum_{k=1}^{i} U_k)}{\sum_{j=1}^{i} (\sum_{k=1}^{j} U_k)^2}$$

2.1.1 Estimation of the model parameters

The parameters of the model are the initial lag 'T', D-hour unit hydrograph ordinates and the ϕ-index value.

(a) Estimation of initial lag

The rainfall over a catchment does not produce an immediate response by way of measurable increase in runoff at the gauging site. In a flood producing rain the point of start of rising limb of the hydrograph indicates that the surface runoff has started contributing to the flow at the gauging site. This sudden rise is attributed to the occurrence of rainfall excess on the catchment because of rainfall intensities which are higher than the infiltration rate. Since the infiltration rate on the catchment is not known the best guess of the starting time of rainfall excess on the catchment is the time of occurrence of the inflection point on the rainfall mass curve immediately preceeding the inflection point of the flow mass curve. The time distance between these inflection points is termed as initial lag 'T' The initial lag varies upon from storm to storm depending on its areal distribution. However, the value of this parameter is known at the time of forecast and therefore this variability does not pose any problem in formulating the forecast.

(b) Estimation of D-hour Unit Hydrograph

Normally, several separate and distinct isolated uniform intensity storms, if available, are used to derive the same duration unit hydrographs. The peak values and times from the beginning to the peak for the separate hydrographs are averaged to sketch a typical unit hydrograph such that the total area under the curve is equal to 1 cm. of runoff. An average D-hour unit hydrograph, thus obtained, is used in the forecasting model.

(c) Estimation of ϕ-index and forecasted runoff

ϕ-index is the other parameters which needs to be determined to enable the computation of forecasts using eq.(1). The parameters is computed using Eq.(2) when a sudden rise is experienced at the gauging site. The baseflow is assumed to be constant and is added to the computed forecast of direct runoff values to obtain the forecast of runoff. As rainfall and runoff progresses in time, eq.2 is used to update the value of ϕ-index and updated forecast is issued. In all forecast computations it is assumed that no rainfall occurs beyond the time of forecast and the latest value of ϕ-index is valid for all times prior to the time, of forecast.

2.1.2 Application of the Model to a Catchment

Real time forecasting is performed using Eq.(1) for the isolated events of the catchment. It is presumed that rainfall and runoff data at discrete time D are available to the forecaster through a telemetric system installed on the catchments. The steps involved in the computation are:-

i) Compute the Theissen Weights to obtain average rainfall on the catchment

ii) Derive an average D-hour unit hydrograph analysing the observed rainfall-runoff data for various isolated events.

iii) The forecast formulation waits the occurrence of the first rise in the hydrograph at the gauging site leading to the determination of Q_{i+T}. (For example if hourly hydrograph values are (30,30,70......), then the base flow is taken as 30 and maintained at this value and

Q_{i+T} is taken as 40). Determine the initial lag by plotting the rainfall and flow mass curves.

iv) Determine initial value of F using eq.(2) with known value of Q_{i+T}.

v) Use the value of ϕ-index obtained from the previous step to compute rainfall excess for (T+1) time steps.

vi) Compute the forecasted values of direct surface runoff at various lead times or the whole resulting direct runoff hydrograph from eq.1 using the known rainfall excess and assuming no rainfall occurs beyond the time of forecast.

vii) Update the value of ϕ-index (F) at each time step k using eq.2 and repeat step (v) and (vi) to formulate the forecast. In step (v) the rainfall excess is computed for (T+k) time steps.

The above procedure was used to formulate forecasts for Krishna Wunna River catchment at Bridge No.807/1. Chander et al(1984) .The catchment, shown in Fig.1, extends from longitude $78^{o}42'$ to $79^{o}1'$ and latitude $20^{o}54'$ to $20^{o}11'$. It is rectangular in shape and covers an area of 823.6 km^2. Rainfall - runoff data are available for the period 1965-73 (eight years). The five storm events that produced largest recorded flows were selected for the study. The date of events have been listed in table -1, alongwith the initial lag time in hours estimated using the procedure described in the earlier part of this lecture

Table 1- The storm events studied and their initial lag time in hours

Sl.No.	Date of events	Initial lag time in Hours
1.	700822*	2
2.	700828	2
3.	700904	1
4.	710622	1
5.	730707	2

Events are numbered in six digits. The first two digits from left to right refer to year, the next two the month , and the last two the day. For example, the storm event dated August 22,1970 is identified as 700822,

In the study conducted by Chander et al (1984) due to the limited number of events, the unit hydrograph derived from a single event (event No.1 dated 700822 in table 1) is adopted as an average unit hydrograph and is therefore designated as the calibration events and used for formulating the forecasts. The

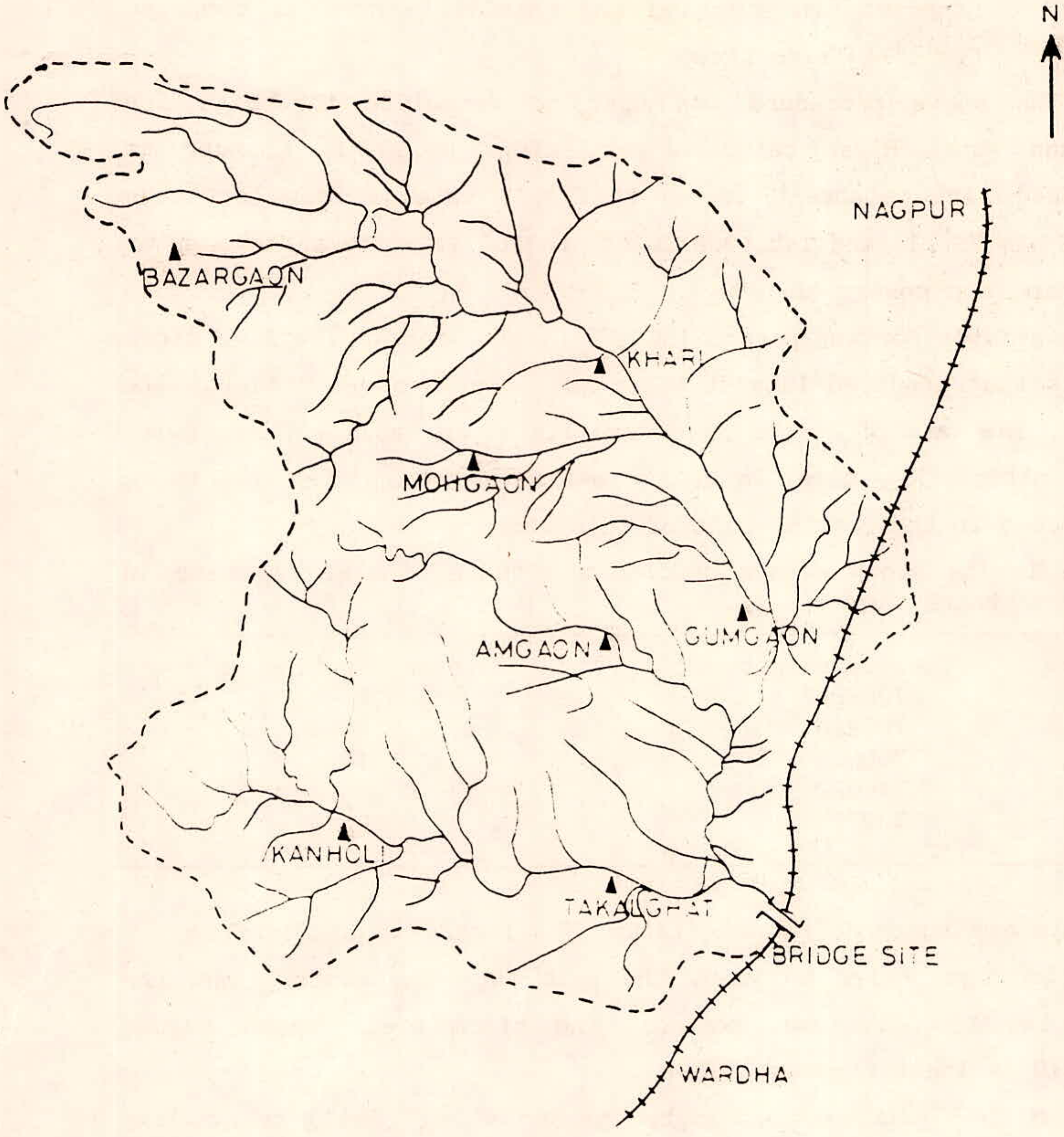

FIG. 1 THE KRISHNA WUNNA RIVER CATCHMENT AT BRIDGE SITE NO. 807

1-hour unit hydrograph ordinates derived from the calibration event by collin's method are used in eq. 1 and eq. 2 for formulating the forecasts. Forecasts for various lead times for all the five events are made from eq. (1) and eq. (2) using the described procedure. The computation is done every hour for the rising portion of the hydrograph. The observed and r^al time forecasts for one calibration event and the four test events are shown in fig. 2 to 6. The forecast evaluation criteria and the observed discharge value have been compared using the following numerical criteria:

i) Co-efficient of variation of the residual error

$$Y = \frac{1}{n} [\Sigma (Q_c - Q_o)^2]^{1/2} / \bar{Q}_o \qquad \ldots(3)$$

ii) Ratio of relative error to the mean

$$R = \frac{1}{n} \Sigma (Q_c - Q_o) / \bar{Q}_o \qquad \ldots(4)$$

iii) Ratio of absolute error to the mean

$$a = \frac{1}{n} \Sigma \mid Q_c - Q_c \mid / \bar{Q}_o \qquad \ldots(5)$$

iv) Co-efficient of correlation between the observed and the forecasted values :

$$C = \frac{n \Sigma Q_c Q_o - \Sigma Q_o \Sigma Q_c}{(DQ_o \quad DQ_c)^{1/2}} \qquad \ldots(6)$$

where,

Q_o denotes the observed discharge

Q_c denotes the forecast value of the discharge

n denotes the total number of observations.

$$\bar{Q}_o = \frac{1}{n} \Sigma Q_o \qquad \ldots(7)$$

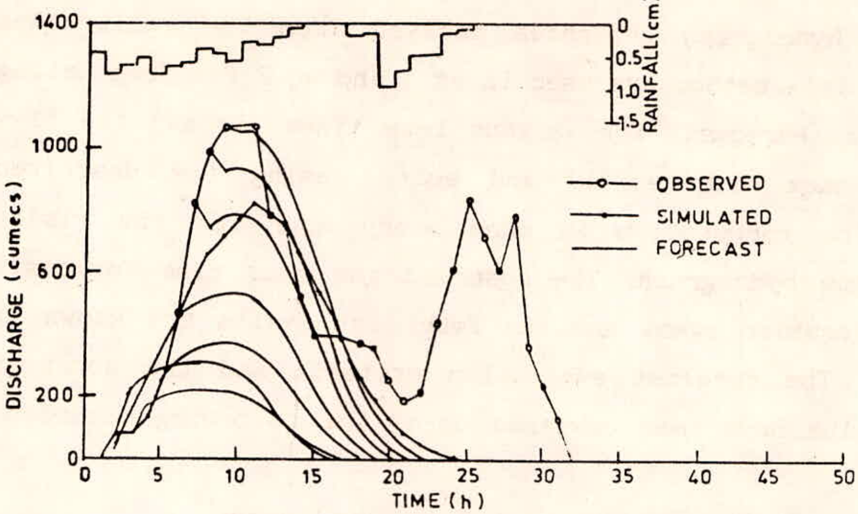

FIG. 2 OBSERVED AND SIMULATED HYDROGRAPHS AND REAL TIME HOURLY FORECASTS FOR THE EVENT 700904

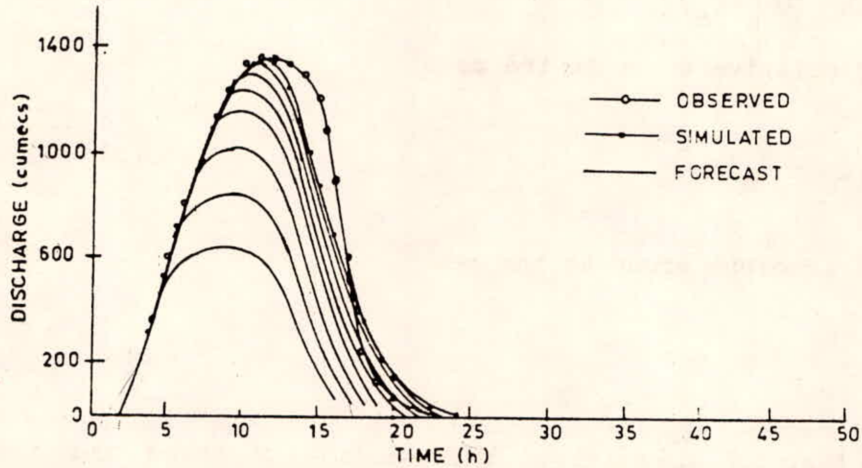

FIG. 3 OBSERVED AND SIMULATED HYDROGRAPHS AND REAL TIME HOURLY FORECASTS FOR THE EVENT DATED 700822

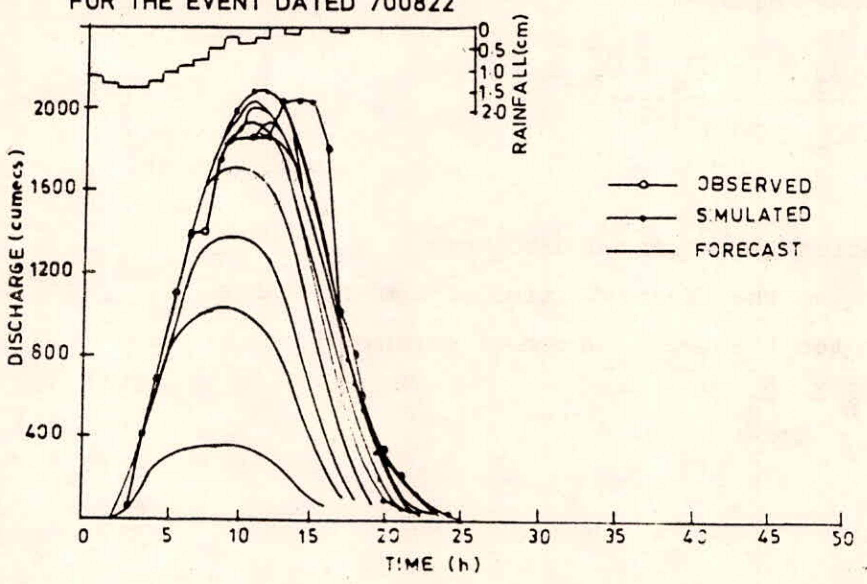

FIG. 4 OBSERVED AND SIMULATED HYDROGRAPHS AND REAL TIME HOURLY FORECASTS FOR THE EVENT DATED 700828

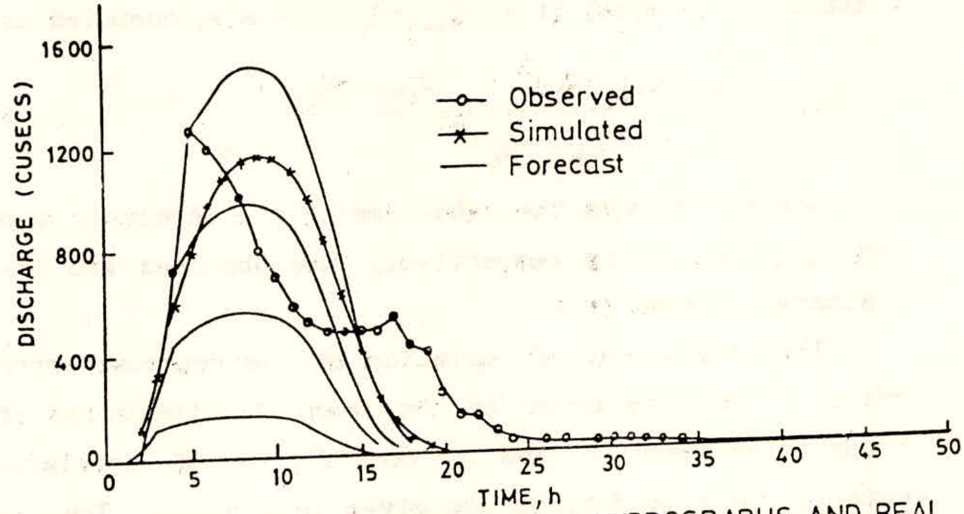

FIG. 5-OBSERVED AND SIMULATED HYDROGRAPHS AND REAL TIME HOURLY FORECASTS FOR THE EVENT 710622

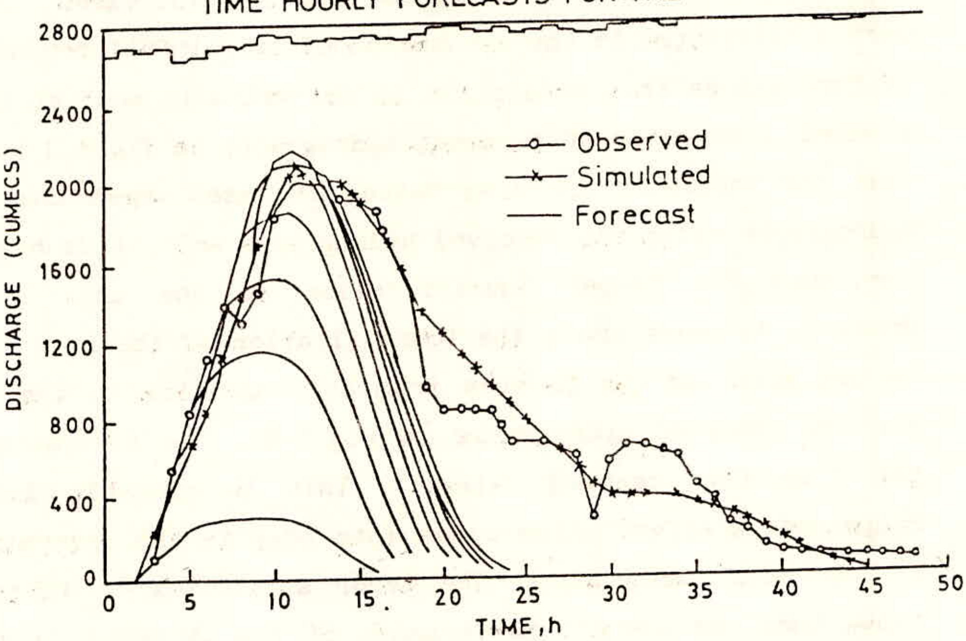

FIG. 6 - OBSERVED AND SIMULATED HYDROGRAPHS AND REAL TIME HOURLY FORECASTS FOR THE EVENT

$$DQ_o = n\,\Sigma Q_o^2 - (\Sigma\, Q_o)^2 \qquad \ldots(8)$$

$$DQ_c = n\Sigma\, Q_c^2 - (\Sigma\, Q_c)^2 \qquad \ldots(9)$$

The proportion of variance accounted for by the model when compared to no model (i.e. $\bar{Q}_{t+1} = \bar{Q}_t$) is also computed as under :

$$R_1 = \frac{\Sigma_t (Q_{t+1} - Q_t)^2 - (Q_{t+1} - Q_{t+1})^2}{\bar{t}\ (Q_{t+1} - Q_t)^2} \qquad \ldots(10)$$

Where 1 denotes the lead time of the forecasts made and Q_{t+1} and Q_{t+1} represents respectively the observed and the forecast discharge at time (t+1).

The coefficient of variation of the residual error (y), the ratio of absolute error to the mean (A), the ratio of relative error to the mean (R) and the co-efficient of correlation (C) for each of these lead times are given in table -2. The results show that the forecasts deteriorate as lead time increases. This can be partly attributed to the assumption of no rainfall beyond the time of forecast as this assumption is not valid in most of the events studied. Comparison of forecast hydrographs in fig.2,3 and 6 shows that the forecasts are also better in cases where the simulated hydrographs match the observed hydrographs well, thus highlighting the need for proper identification of the unit hydrograph. However, in cases where the identification of the unit hydrograph is not good (as can be seen from the variation in the simulated and the observed hydrographs in Fig.1-5). The forecasts are not far from the observed values . This is probably due to the compensating effect which comes into play in the determination of F (ϕ-index) using eq.2. The error statistics in table-2 also shows that the overall performance of the UH model is better to that no model.

TABLE 2 Error statistics of the Forecast values

Error Statistics	1. Hr. Lead Time Forecasts		2. Hr. Lead Time Forecasts		3. Hr. Lead Time Forecasts	
	No. Model	UH Model	No. Model	UH Model	No. Model	UH Model
Coefficient of Residual Errors (Y)	0.24	0.17	0.40	0.26	0.48	0.31
Relative error of variation(R)	-0.19	0.04	-0.34	-0.09	-0.42	-0.16
Absolute Error of variation(A)	0.19	0.13	0.34	0.19	0.42	0.23
Coefficient of correlation(C)	0.9611	0.9577	0.8680	0.8676	0.8678	0.8640
Proportion of Variance with No Model	-	0.47	-	0.57	-	0.58

2.2 Method 2

In method 1, discussed above, only the constant loss rate parameters is continuously updated as the rainfall progresses and rainfall runoff data is reported, while the unit hydrograph derived from storm rainfall runoff data is considered as the representative unit hydrograph of the catchment and is used for real time forecasting of the events. However, because of the inherent non-linearity in the rainfall-runoff process and other factors affecting the shape of the storm hydrograph, a single unit hydrograph would not be able to give a good reproduction of the actual storm hydrographs as it was seen the application of the method 1 for few storms. In order to introduce the variable unit hydrograph ordinates in the mechanism of real flood forecasting, Perumal, Singh and Seth (1984) presented a method which uses the unit hydrograph of Nash's n-linear reservoir cascade model in discrete form with two parameters n_d and k_d, and a constant loss rate (F) for accounting the infiltration and other losses of rainfall. The parameters n_d, k_d and F are estimated in real time using the currently available data. Resenbrock optimisation technique, an improved version of the unvariate search technique, is used to estimate and update the parameters n_d k_d and F at each time step during the forecast period. A computer software developed based on the above concept has been used for forecasting of the direct surface runoff (DSRO) for one, two and three hours lead periods using the currently available data. The methodology has been adopted for real time flood forecasting of the DSRO and no attempt has been made to incorporate the baseflow in the estimation procedure.

2.2.1 Estimation of Unit Hydrograph Using Discrete Linear Equal Reservoir Cascade Model

O'Connor (1976) derived the unit hydrograph using Nash's cascade equal linear reservoir model in discrete form. The governing difference equation is given as:

$$y_m = \left(\frac{1}{1 + K_d \nabla} \right)^{n_d} x_m \qquad \ldots.(11)$$

where, n_d and k_d = parameters for discrete Nash model,

∇ = difference operator

= 1-B

B = Backward operator

y_m = DSRO ordinates

x_m = Excess Rainfall ordinates

= $(P - F)_m$

F = Constant loss rate parameter

The conversion of the excess rainfall into direct surface runoff is performed using the following equation, popularly known as convolution summation equation.

$$y_m = \sum_{j=1}^{m} (P-F)_{m-j+1} U_j \qquad \ldots(12)$$

where, U_j represents the j^{th} ordinate of discrete unit hydrograph having the same discrete interval as that of the rainfall excess and direct surface runoff (DSRO).

For known excess rainfall and the unit hydrograph ordinates, the resulting DSRO may be computed using eqn. (12).

O'Connor (1976) derived the form of discrete unit hydrograph for Discrete Nash Cascade Model. Its form is given as:

$$U_j = \frac{\lfloor j+n_d-1 \rfloor}{\lfloor n_d \rfloor \lfloor j \rfloor} p^{n_d} q^{j-1} \qquad \ldots(13)$$

where, j = 1,2,3,...... (discrete interval)

$p = \frac{1}{1 + k_d}$

$q = 1 - p$

k_d Storage coefficient of discrete linear reservoir.

n_d = Number of linear reservoir, for Nash Cascade Model

Eqn. (13) may be rewritten after replacing j by (j-1) from eqn. (13)

$$U_{j-1} = \frac{\lfloor j+n_d-2 \rfloor}{\lfloor n_d \rfloor \lfloor j-1 \rfloor} p^{n_d} q^{j-2} \qquad \ldots(14)$$

Division eqn.(13) by eqn.(14) the recursive form of equation for unit hydrograph ordinates may be obtained which is expressed as:

$$U_j = \frac{j-n_d-2}{j-1} \, q \, U_{j-1} \qquad ..(15)$$

The first ordinate of unit hydrograph, U_1, which corresponds to the time index j = 1, is obtained from eqn(12) as

$$U_1 = p^n \qquad ..(16)$$

Equation (15) and (16) provide estimates for discrete unit hydrograph using Discrete Nash Cascade Model. The advantage of using Nash Discrete Model avoids the computations of Gamma function as required by its counterpart in the continuous case.

2.2.2 Real time flood forecasting using Discrete Nash Cascade model

The methodology for formulating the forecasts using Discrete Nash Cascade Model is given below:

i) Obtain the initial values of parameters n_d, k_d and F for Rosen brock optimisation technique based on the calibration of past observed events in the catchment.

ii) Wait for, at least, three observations of rainfall and direct surface runoff ordinates.

iii) Based on the available observations of rainfall and DSRO, find out the optimum parameters n_d, K_d and F for fitting the DSRO ordinates available at the time of forecast using the corresponding rainfall data. The optimisation may be carried out by Rosen brock optimisation technique using the following objective function:

$$S = \sum_{j=1}^{m} (y_j - \hat{y}_j)^2 * WT_j \qquad ..(17)$$

where, S = Objective function

yO_j = Observed DSRO ordinates

$\hat{y}_j$ = Computed DSRO ordinates using eqn(12)

WT_j = The weighting factor applied to square of the observed and computed DSRO ordinates

The weighting factor, WT_j, is given as:

$$WT_j = \left(\frac{j}{m+1}\right)^2 \qquad ..(18)$$

in which

m = the time index corresponding to the time forecast.

Further, in the optimisation procedure, the parameters n_d, k_d and F are constrained to zero at the lower level, and the parameter F is constrained to the maximum loss which is computed by using the excess of the current rainfall on the basis of equating it with the corresponding volume of DSRO.

iv) Forecast the floods for the lead periods of one hour, two hour and three hour etc. based on the current rainfall information in the following steps:

- Use optimum parameters n_d & k_d obtained at step (iii) in eqn.(5) and (6) in order to arrive at the estimate of unit hydrograph, U_j.
- Use current rainfall information, P and optimum loss rate parameter, F together with the estimated of unit hydrograph, U_j in eqn.(12) to forecast the DSRO hydrograph ordinates. The ordinates of DSRO at one hour later, two hour later and three hour latter etc.

from the current rainfall information available, provides the forecasts for one hour lead period, two hour lead period and three hour lead period etc.

The following example illustrates the computation of objective function as well as the forecasted direct surface runoff hydrograph based on the available information on rainfall and direct surface runoff for a storm event over a catchment.

Example 1: For a storm event over a catchment of size 824 sq.km. the currently available information on rainfall and direct surface runoff are given below:

Time (hrs.)	:	1	2	3
Rainfall(mm)	:	1.78	3.435	4.325
Direct surface runoff(m^3/s)	:	1.846	3.269	72.692

Discrete Nash cascade model parameters and loss rate parameter, (n_d, k_d and F), obtained from Resenbrock optimisation technique based on the above available information on rainfall and DSRO, are 8.85, 0.41 and 1.76 respectively.

(A) Find out the objective function and forecasted hydrograph of DSRO based on the available information.

(B) If the rainfall and Direct surface runoff become available at 4^{th} hour, which are 5.752 mm and 97.115 m^3/s respectively, find out the updated forecast for DSRO. The updated parameters n_d, k_d and F based on the four values of rainfall and runoff are assumed to be 8.88, 0.52 and 1.95 respectively.

SOLUTION:

(a) The following steps are involved:

<u>Step-I</u> Derive unit hydrograph using eqn.(5) & (6).

Since n_d = 8.85, K_d = 0.41 hr and F = 1.76,

$\therefore \quad p = \left(\frac{1}{1 + K}\right) = 0.710, \; q = 1-p = 0.29$

$$U_1 = (p)^{n_d}$$

$$U_j = \frac{j + n_d - 2}{j - 1} \, q \, U_{j-1}$$

j	1	2	3	4	5	6
U_j	0.048	0.123	0.176	0.185	0.159	0.119

j	7	8	9	10	11	etc.
U_j	0.078	0.0489	0.0281	0.0153	0.0079	

Step-II Compute the forecasted DSRO using the derived UH.

Table 3: Computation of Forecasted Direct Surface Runoff.

No. (j)	Time (Hrs)	Rainfall (mm)	Excess rainfall (mm)	*UH Ordinates (m^3/s)	Contribution of First Block (m^3/s)	Contribution of Second Block (m^3/s)	Contribution of Third Block (m^3/s)	Forecasted DSRO (m /s)
(1)	(2)	(3)	(4)=(3)-F	(5)	(6)	(7)	(8)	(9)=(6)+(7)+(8)
1	1	1.78	0.02	10.99	0.22	-	-	(0.22)
2	2	3.435	1.675	28.15	0.56	18.41		(18.97)
3	3	4.325	2.565	40.28	0.81	47.15	28.19	(76.15)
4	4			42.34	0.85	67.47	72.20	140.52
5	5			36.39	0.73	70.92	103.3	174.95
6.	6			27.24	0.54	60.95	108.6	170.09
7	7			18.08	0.36	45.63	93.34	139.33
8	8			11.19	0.22	30.28	69.87	100.37
9	9			6.43	0.13	18.74	46.38	65.25
10	10			3.50	0.07	10.77	28.70	39.54
11	11			1.81	0.04	5.86	16.49	22.39
etc.	etc.			etc	etc	etc	etc	etc

*UH ordinates (Col.(5)) $= \frac{U_j * CA}{3.6}$ m^3/s, here CA = 824 sq.km.

Thus,

one hour lead forecast =140.52 m^3/s

Two hour lead forecast =174.95 m^3/s

Three hour lead forecast =170.09 m^3/s

etc.

Step-III : Compute the objective function using eqn.(17) and (18). here m = 3

Table :4 Computation of objective function.

No.	Time (hrs)	Observed DSRO, Y_j (m^3/s)	Computed DSRO, $\hat{Y}_j$ (m^3/s)	Weight WT_j	Objective Function value
(1)	(2)	(3)	(4)	(5)	(6) = $(Y_j-\hat{y}_j)^2 * WT_j$
1	1	1.846	0.22	0.0625	0.165
2	2	3.269	18.97	0.250	61.630
3	3	72.692	76.15	0.563	6.73

$$\therefore S = \sum_{j=1}^{m} (Y_j - \hat{Y}_j)^2 * WT_j = 68.525 \ (m^3/s)^2$$

(B) In the solution of the second part of the problem the following steps are involved.

Step-I: Derive unit hydrograph using updated parameters of Discrete Nash Cascade Model obtained from Rosen brock optimisation technique based on four hour information of the rainfall and DSRO. Use eqn.(15) and (16).

Since n_d = 8.88, k_d = 0.52 and F = 1.95

$\therefore \quad p = \left(\frac{1}{1+K_d}\right) = 0.66, \quad q = 1-p = 1 - 0.66 = 0.34.$

$U_1 = (p)^{n_d} = (0.66)^{8.88} = 0.025$

$$U_j = \frac{j + n_d - 2}{j - 1} \, q \, U_{j-1}$$

j	1	2	3	4	5	6	7	8
U_j	0.025	0.075	0.126	0.155	0.157	0.138	0.109	0.0788

j	9	10	11	etc.
U_j	0.053	0.034	0.021	etc.

<u>Step-II</u> Compute the forecasted DSRO using the derived UH.

The computations are shown in Table 5.

Table 5 : Computation of Forecasted Direct Surface Runoff

Time (Hrs)	Rainfall (mm)	Excess rainfall (mm)	*UH Ordinates (m^3/s)	Contribution of First Block (m^3/s)	Second Block (m^3/s)	Third Block (m^3/s)	Fourth Block (m^3/s)	Forecasted DSRO (m^3/s)
(1)	(2)	(3)=(2) -F	(4)	(5)	(6)	(7)	(8)	(9)=(5)+(6) (7)+(8)
1	1.78	0	5.72	0	-	-	-	0
2	3.435	1.485	17.17	0	8.49	-	-	8.49
3	4.325	2.375	28.84	0	25.49	13.585	-	39.08
4	5.752	3.802	35.48	0	42.83	40.779	21.747	105.36
5			35.94	0	52.69	68.495	65.280	186.47
6			31.59	0	53.37	82.265	109.65	245.29
7			24.95	0	46.91	85.358	134.89	267.16
8			18.04	0	37.05	75.026	136.64	248.72
9			12.13	0	26.79	59.256	120.11	206.16
10			7.78	0	18.01	42.845	94.86	155.72
11			4.81	0	11.55	28.809	68.59	108.95
etc.			etc	etc	etc	etc	etc	etc

$$\text{*UH ordinates} = \frac{U_j * CA}{3.6} \quad m^3/s, \; CA = 824 \text{ sq.km.}$$

Thus one hour lead forecast = 186.47 m^3/s

Two hour lead forecast = 245.29 m^3/s

Three hour lead forecast= 267.16 m^3/s

etc.

2.2.3 Application of the Model to a catchment:

Real time forecasting for a catchment is performed using discrete Nash Cascade Model in the following steps :

i) Identify moderate to intense isolated rainfall runoff events from the historical records.

ii) Compute the Thiessen Weights and hence average rainfall on the catchment.

iii) Derive Unit Hydrograph parameters n_d and k_d from rainfall runoff records of isolated events, using Rosenbrock optimisatioin procedure

iv. Use n_d and k_d obtained from step (iii) as initial estimates of parameters in formulating the forecasting procedure. In the beginning, initial estimate for the loss rate parameter F is assumed based on the three ordinates of observed rainfall land direct runoff.

v... Optimise the unit hydrograph parameters n_d, $k_d$and lossparameter F using Rosenbrock optimisation procedure at eachtime step.

vi. Use optimised value of n_d and k_d in equation (15) and (16) to compute unit hydrograph ordinates.

vii. Compute the forecasted values of direct surface runoff at various lead times substituting the unit hydrograph ordinates obtained from step (vi) and optimised values of loss rate parameters F in equation (12) taking known rainfall values upto the time computations are being made.

viii. Consider the optimised parameter value of $n_{d,}$ k_d and F as initial parameter estimates for the next time step and repeat step (v) to (viii) to formulate the forecast considering the rainfall-runoff information arrived at the previous time step through a telemetric system or any other system installed on the catchment.

The above methodology has been used to forecast the direct surface runoff for Krishna Wunna River catchment at Bridge No.807/1 for which method-1 was employed to formulate the forecast. Storm runoff data of the six events available at National Institute of Hydrology are used for testing the methodology. The initial values of n_d and k_d for all the storms have been taken as 9.0 and 0.5 respectively using the general trend of these values obtained by calibrating the model for these events. The initial value of F is assumed on the basis of first three ordinates of observed rainfall and direct runoff for each event.

Forecasting for the lead periods of one hour, two hour and three hour is made using the current available data for the event under consideration. Forecasts for these lead periods, for all the vents are given in table 6 to 11, alongwith the updated parameters $n_{d,}$ k_d and F at each timestep. The forecasted hydrograph based on the current available information of rainfall and runoff upto peak are shown in fig 7 to 12 along with the simulated hydrographs for all the six events. The assumption of no rainfall beyond the time of forecast is obviously reflected in under predicting the rising limb of the hydrograph. However, it can be seen from the table that the one hour lead forecasts are comparatively better than those for two and three hour lead forecasts. The performance of the methodology adopted is also seen by the better forecasts made once the rainfall ceases. It is also seen that after the cessation

REAL TIME FORECASTING DETAILS

Sl. No.	Updated parameters n_δ	k_δ (hrs)	F (mm/hr)	Observed rainfall (mm)	Observed DSRO (m /s)	One hour lead forecast (m s)	Two hour Lead forecast (m /s)	Three hours Lead forecast (m /s)
	TABLE 6: FOR STORM EVENT OF 24.7.1967							
1.	-	-	-	1.557	1.591	-	-	-
2.	-	-	-	1.746	3.182	-	-	-
3.	-	-	-	4.035	4.773	-	-	-
4.	9.70	0.54	1.60	2.994	16.364	32	-	-
5.	9.87	0.71	1.65	6.235	247.955	33	58	-
6.	9.64	0.44	1.45	2.833	359.545	258	58	79
7.	9.52	0.35	1.50	2.563	551.14	414	322	82
8.	9.52	0.35	1.50	0.851	432.73	423	388	331
9.	9.52	0.35	1.50	2.269	334.32	355	355	309
10.	9.52	0.35	1.50	1.715	285.92	286	262	262
11.	9.37	0.33	1.42		177.50	212	209	174
12.	9.05	0.33	1.48		119.09	129	145	139
13.	9.05	0.33	1.48		70.68	81	80	93
14.	8.90	0.33	1.51		42.27	43	46	47
15.	8.93	0.33	1.48		23.86	24	23	25
16.	8.95	0.33	1.48		20.45	12	12	12
17.	8.95	0.33	1.48		17.04	6	6	6
18.	8.95	0.33	1.46		18.63	3	3	3
19.	8.97	0.33	1.49		10.22	1	1	1
20.	9.08	0.33	1.48		6.81	0.5	0.5	0.5
21.	9.07	0.33	1.45		3.41	0.2	0.2	0.2

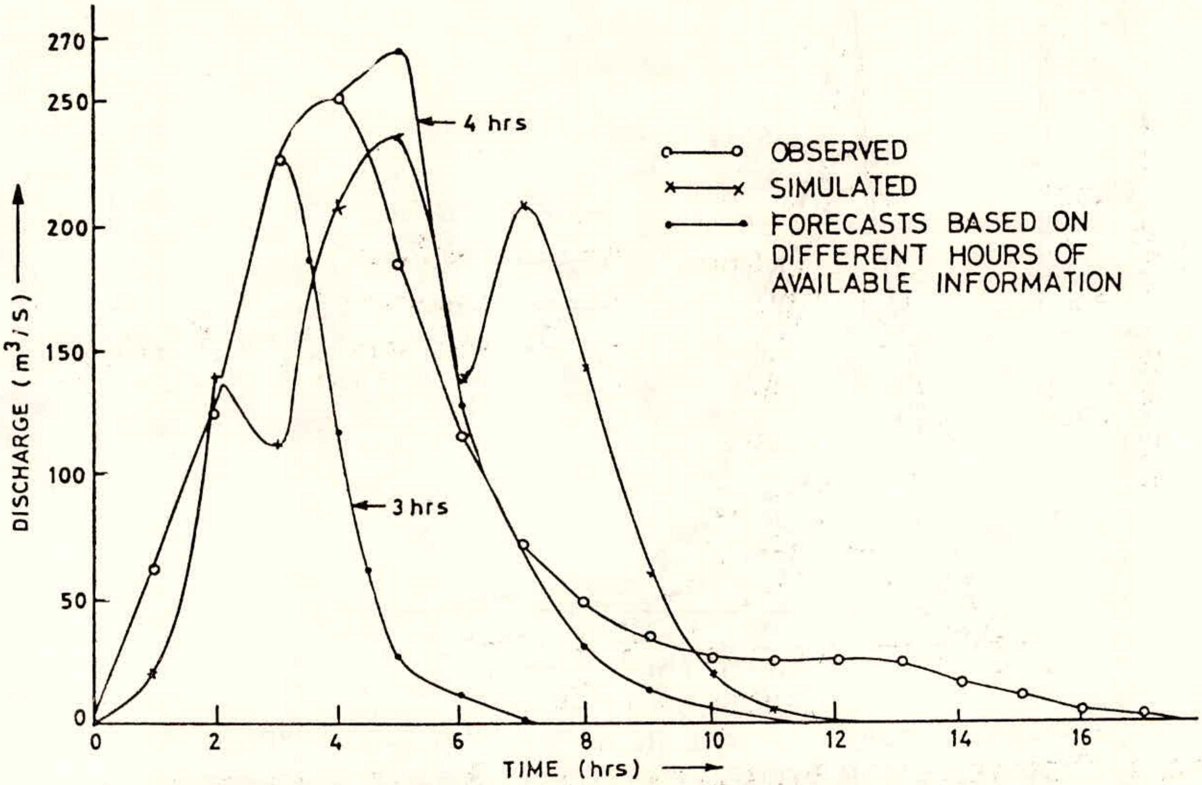

FIG.7—OBSERVED, SIMULATED AND REAL TIME FORCASTS HYDROGRAPHS FOR THE STORM EVENT OF 4-7-1968

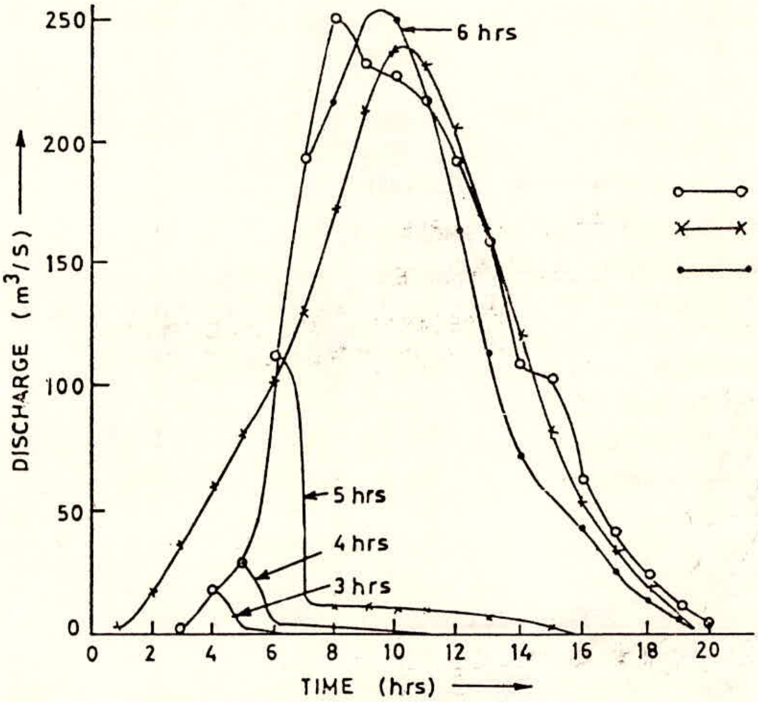

FIG. 8- OBSERVED SIMULATED AND REAL TIME FORECASTS HYDROGRAPHS FOR THE STORM EVENT OF 9-8-1973

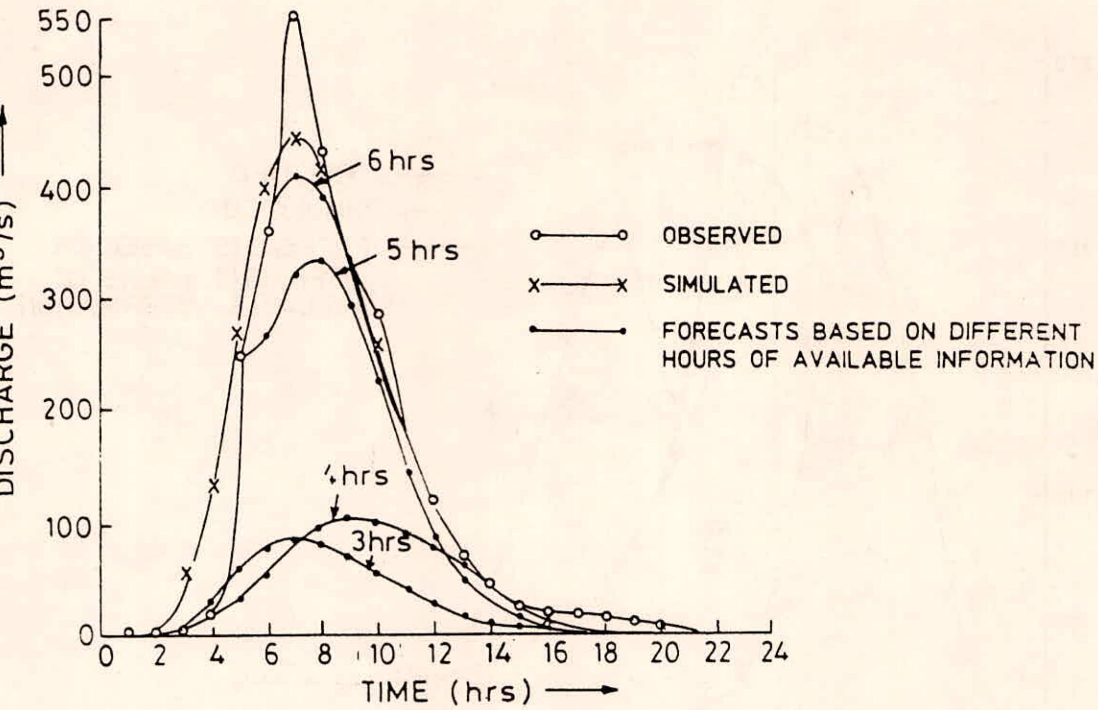

FIG. 9 OBSERVED, SIMULATED AND REAL TIME FORECASTS HYDROGRAPHS FOR THE STORM EVENT OF 24-7-1967

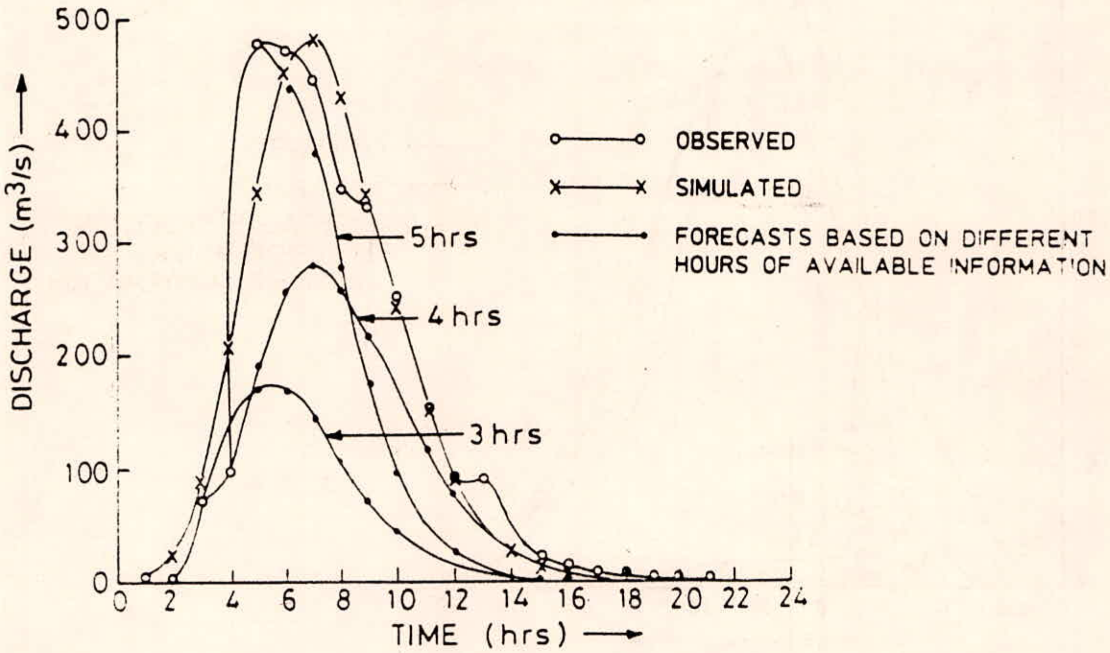

FIG. 10 OBSERVED, SIMULATED AND REAL TIME FORECASTS HYDROGRAPHS FOR THE STORM EVENT OF 6-9-1969

Table 7: FOR STORM EVENT OF 6.9.1969

1.	-	-	-	1.78	1.846	-	-	-
2.	-	-	-	3.435	3.269	-	-	-
3.	-	-	-	4.325	72.692	-	-	-
4.	8.85	C.41	1.76	5.752	97.115	138	-	-
5.	8.88	0.52	1.95	4.235	481.51	182	174	-
6.	8.78	0.33	1.95	3.295	470.96	436	244	170
7.	8.78	0.33	1.95	1.730	445.39	442	386	266
8.	8.78	0.33	1.95	0.295	344.81	352	352	285
9.	8.58	0.32	1.93	1.637	331.23	236	242	242
10.	9.09	0.34	1.79	0.373	250.65	187	142	149
11.	9.69	0.35	1.77	1.046	148.1	133	110	77
12.	9.69	0.35	1.77	0	92.50	76	76	60
13.	9.74	0.36	1.76	0	91.92	45	41	40
14.	9.74	0.36	1.72	0.06	26.35	24	23	20
15.	10.04	0.36	1.71	0.96	17.77	14	11	11
16.	10.04	0.36	1.70	6.47	15.19	7	6	5
17.	10.04	0.36	1.69	0.83	9.62	3	3	3
18.	10.04	0.36	1.69	0	9.04	1	1	1
19.	10.04	0.36	1.68	0.033	7.46	0.6	0.5	0.5
20.	10.04	0.36	1.68	0.261	6.89	0.5	0.3	0.3
21.	10.04	0.36	1.68		6.31	0.1	0.1	0.1
22.	10.04	0.36	1.67		4.73	0	0	0
23.	10.04	C.36	1.67		4.15	0	0	0
24.	10.04	C.36	1.67		3.58	0	0	0

Table 8 : FOR STORM EVENT OF 4.7.1968

1.	-	-	-	3.129	61.75	-	-	-
2.	-	-	-	4.235	126.38	-	-	-
3.	-	-	-	2.157	226.0	-	-	-
4.	9.96	0.17	1.94	4.577	250.63	185	-	-
5.	10.06	0.17	2.03	3.863	185.23	269	116	-
6.	10.13	0.24	2.69	0.592	114.88	212	208	61
7.	9.99	0.09	2.81	4.539	74.50	52	135	129
8.	9.48	0.22	2.83	0	49.13	160	15	68
9.	9.02	0.25	2.83	2.757	33.75	77	76	3
10.	8.81	0.13	2.72	0	28.38	23	28	27
11.	8.80	0.07	2.72	0.441	28.00	6	6	8
12.	9.35	0.08	2.72		27.63	2	2	1
13.	9.45	0.09	2.72		27.25	0	0	0
14.	9.35	0.09	2.72		18.88	0	0	0
15.	9.65	0.09	2.72		13.50	0	0	0
16.	9.75	0.09	2.72		6.13	0	0	0
17.	9.85	0.09	2.72		5.75	0	0	0
18.	9.95	0.09	2.72		5.38	0	0	0

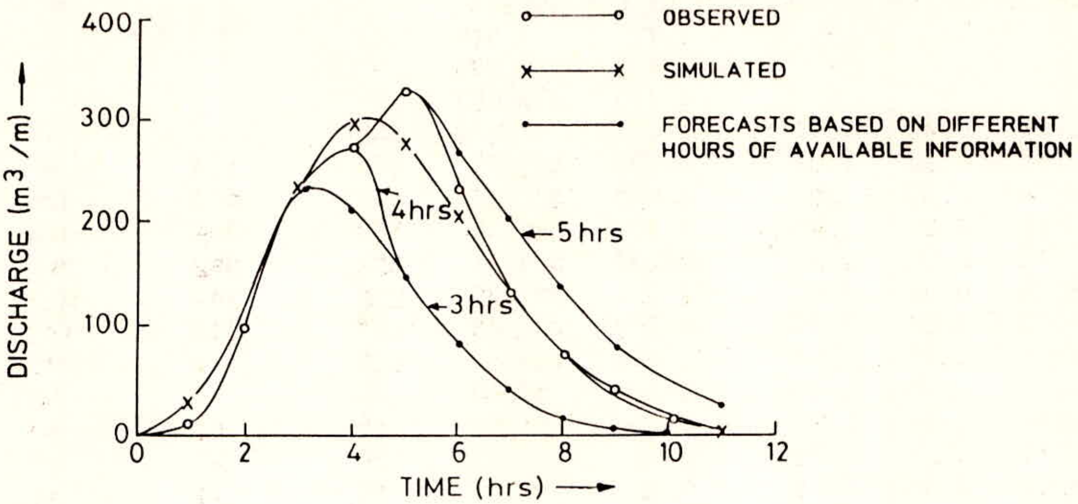

FIG. 11 OBSERVED, SIMULATED AND REAL TIME FORECASTS HYDROGRAPHS FOR THE STORM EVENT OF 10-8-1970

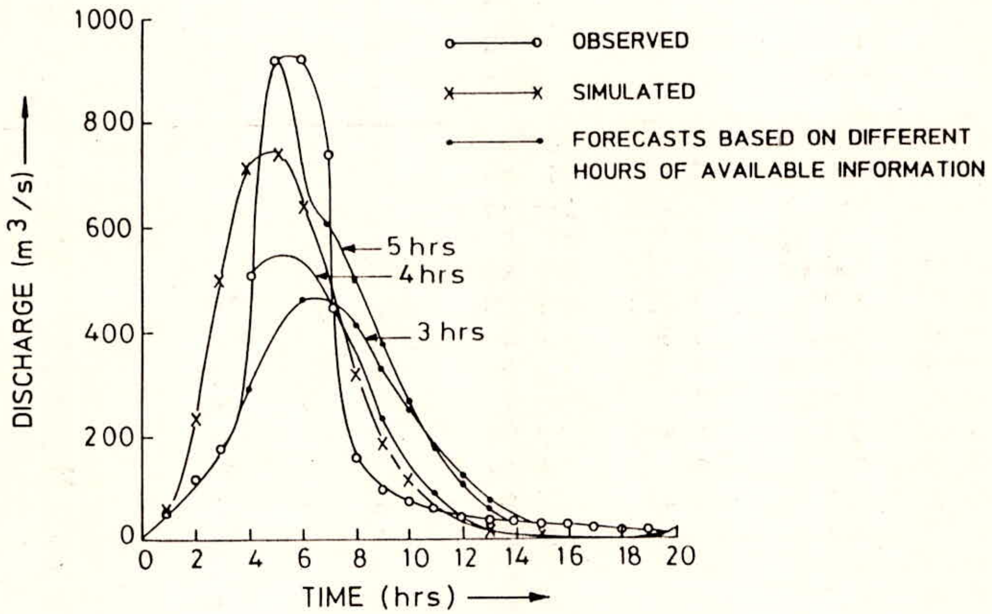

FIG. 12 OBSERVED, SIMULATED AND REAL TIME FORECASTS HYDROGRAPHS FOR THE STORM EVENT OF 22-8-1973

Table 9 : FOR STORM EVENT OF 9.8.1973

1.	-	-	-	2.152	0.5	-	-	-
2.	-	-	-	2.035	0.9	-	-	-
3.	-	-	-	0.554	1.4	-	-	-
4. 9.84	0.68	2.14	1.638	16.8	0	-	-
5. 9.94	0.68	2.07	2.022	27	2	0	-
6. 10.04	0.68	2.00	2.190	113	5	2	0
7. 10.14	0.68	1.93	4.795	193	12	5	2
8. 10.26	0.41	1.07	0.785	249	215	13	5
9. 10.11	0.34	1.10		229	261	249	15
10. 10.11	0.35	1.10		225	246	246	247
11. 10.21	0.35	1.10		215	202	200	199
12. 10.51	0.37	1.07		190	148	146	144
13. 12.01	0.36	1.08		156	131	98	96
14. 12.71	0.37	0.92		106	90	88	60
15. 12.81	0.37	0.94		102	65	57	56
16. 13.11	0.38	0.90		62	57	40	34
17. 13.21	0.38	0.89		40	35	33	23
18. 13.31	0.38	0.86		23	21	20	21
19. 13.41	0.38	0.86		9	11	12	12
20. 13.51	0.38	0.86		4	6	6	6

Table 10: FOR STORM EVENT OF 10.8.1970

1.	-	-	-	1.843	10	-	-	-
2.	-	-	-	3.779	100	-	-	-
3.	-	-	-	2.750	230	-	-	-
4.	9.12	0.18	1.42	0.74	270	212	-	-
5.	10.06	0.19	1.39	0.31	330	149	148	-
6.	10.31	0.34	0.37		230	267	82	83
7.	10.30	0.27	0.45		130	170	206	39
8 .	10.45	0.37	0.51		70	88	105	142
9.	10.31	0.27	0.53		40	47	47	59
10.	10.27	0.27	0.53		10	21	23	23
11.	10.21	0.27	0.53		2	9	9	11

Table 11 : FOR STORM EVENT OF 22.8.1973

1.	-	-	-	5.67	53.81	-	-	-
2.	-	-	-	8.78	122.62	-	-	-
3.	-	-	-	6.75	171.43	-	-	-
4.	10.16	0.37	2.30	2.50	510.24	285	-	-
5.	12.91	0.39	2.23	2.39	919.05	549	399	-
6.	12.07	0.36	1.45	2.19	917.85	649	539	450
7.	11.64	0.38	1.58	1.22	736.67	598	603	456
8.	11.58	0.33	0.63	0.03	150.48	655	503	500
9.	11.56	0.21	0.85		89.29	320	509	306
10.	12.27	0.24	0.95		73.11	113	196	365
11.	12.27	0.24	1.49		56.91	64	55	110
12.	12.27	0.24	1.49		42.11	32	31	25
13.	12.21	0.24	1.59		41.52	13	15	14
14.	12.21	0.24	1.59		38.33	6	6	6
15.	12.21	0.24	1.59		37.14	3	3	2
16.	12.21	0.24	1.59		35.95	1	1	1
17.	12.21	0.24	1.59		24.76	0	0	0
18.	12.21	0.24	1.59		14.57	0	0	0
19.	12.21	0.24	1.59		12.38	0	0	0
20.	12.21	0.24	1.59		11.90	0	0	0

of rainfall the parameters n_d, k_d and F are more or less remain constant in subsequent optimisation runs. The final values of parameters n_d and k_d for all six events are different and as such the optimisation of the parameters for each event is necessary. If constant values of n_d and k_d are assumed for all events in the catchment then it may lead to corresponding erroneous estimation of parameter F.

3.0 REAL TIME FLOOD FORECASTING FOR LARGE CATCHMENTS

If the catchment area is more than 5000 sq km, the catchment is considered to be large For such a catachment the methodology given for real time flood forecasting using unit hydrograph based approach can not be applied as such. In this case the forecasts may be formulated dividing the catachment into sub-catchments and linking the discussed methodology with a suitable flood routing technique whose parameters may also be updated in real time using a suitable optimisation technique.

Other approach for forecasting the flood in real time for large catchments could be based on statistical methods, which can be presented either in the form of graphical relations or mathematical equations. A large number of data, covering a wide range conditions, are analysed to derive the relationships which inter alia include gauge to gauge relationship with or without additional parameter and rainfall peak stage relationship. These methods are most commonly used in India as well as other countries of the world (CWC, 1988).

4.0 REMARKS

i) The method based on the unit hydrograph of Nash's n cascade linear reservoir model in discrete form coupled with the constant loss rate function gives reasonable forecasts.

ii) Updating of only loss rate function may give erroneous estimate of the loss rate function, which may sometime provide inaccurate forecasts.

iii) Updating of unit hydrograph parameter (n_d & k_d) and loss rate function F at each time step help in formulating an accurate forecast using the available rainfall- runoff data at the time of forecast.

iv) As long as the rainfall continues, the forecasts are under estimated. However, after the rainfall cessation considerable improvement in the forecast may be observed.

v) The methods discussed in the lecture note are applicable for the small to moderate size catchments as it may be assumed that unit hydrograph theory and principles are valid for such size of catchments.

vi) The provision of forecasting of rainfall is not taken into account in the methodology and therefore the forecasting for different lead periods is based on the current available information. Thus the communication network should be quite effective in order to supply the station rainfall data at the forecasting station in real time.

vii) For larger size catchment, as such the above methods can not be applied. It involves the division of the catchments into the subcatchments and then the application of the above methods for each sub-catchment coupled with channel flood routing methods in formulating the real time forecasting procedures for larger size catchments.

viii) As more number of parameters have to be estimated and updated at each time step, one forecasting station may be enough for the larger size catchments in formulating the accurate forecasts. However, the unit hydrograph and loss parameters for each sub-catchment may be updated at each time step if the required rainfall -runoff data are available. The updated parameters are used to formulate the forecasts for each sub-catchments. Then the forecasted flood for different sub-catchments are routed through the main channel in the order of sub-catchments locations and the routing parameters are updated based on the observed runoff at the site of interest. Additional sites on the main channel upstream to the site of interest may help in providing better estimates of the routing parameters, for different channel reaches.

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