Flood Forecasting Techniques

PRINT VERSION MODULE

MODULE OBJECTIVES

- To get familiarized with a few Statistical parameters
- To grasp difference between empirical vs. theoretical frequency distribution
- To understand & perform various tests to ensure fitness of data for flood frequency analysis
- To learn how to plot confidence band and its significance
- To grasp the meaning and significance of confidence band; confidence limit; outliers; expected probability etc.

INTRODUCTION

The goal of flood forecast is to issue advance warning about water level or discharge large enough that threatens safety of structures and flood plain activities. As observed in previous module, an advance warning of this nature help authorities adopt a series of measures to contain adverse impacts of flood.

Unlike several other disasters, approaching flood can be forecast ahead of its occurrence with advance collection of hydro-meteorological data, and its transformation into flood water level or flood hydrograph. Succeeding paragraphs of this module unfolds a range of commonly employed models in India.

FLOOD FORECASTING TECHNIQUES

According to the various concepts used in developing models, the models can be classified into five categories.

a)   Based on correlation/coaxial diagrams between two variables or even more;

b)   Mathematical equations developed using regression/multiple linear regression techniques which combines independent variable with one or more than one variable;

c)   Hydrological models
        c.1   Rainfall run-off model
             i) Lumped
             ii) Quasi-distributed
             iii) Distributed
        c.2   Routing techniques
             i) Lumped, & Distributed

d)   Hydraulic models
        i) Dynamic Wave routing

e)   Data driven hydrological models
        i) Artificial Neural Networks
        ii) Fuzzy expert system design for FF
        iii) ANFIS (Adaptive Neuro-Fuzzy Inference System) models

a. Correlation/Co-axial Diagrams

Forecasters in India have developed a large set of correlation, and coaxial diagrams which display the pattern of correlation exhibited by two or more variables. Such charts are relatively less complex, and are quite popular among its users. Nevertheless, they need periodical updating to account for constant alteration in catchment characteristics and river regime. One out of several such diagrams used in India is shown here. When a number of tributaries affect the water level at the forecasting station, the variation in water level at base station (base station is a location upstream of forecast station) on the main river as well as base stations on the tributariesare considered to prepare co-axial diagrams. One such diagram developed for formulation of forecast at Patna (Gandhighat) on river Ganga is shown in Fig.1. In this diagram, water level fluctuation at Patna takes into account the variation in water level at Buxar on river Ganga; Darauli on river Ghaghra; Chopan on river Sone; and Rewaghat on river Gandak.
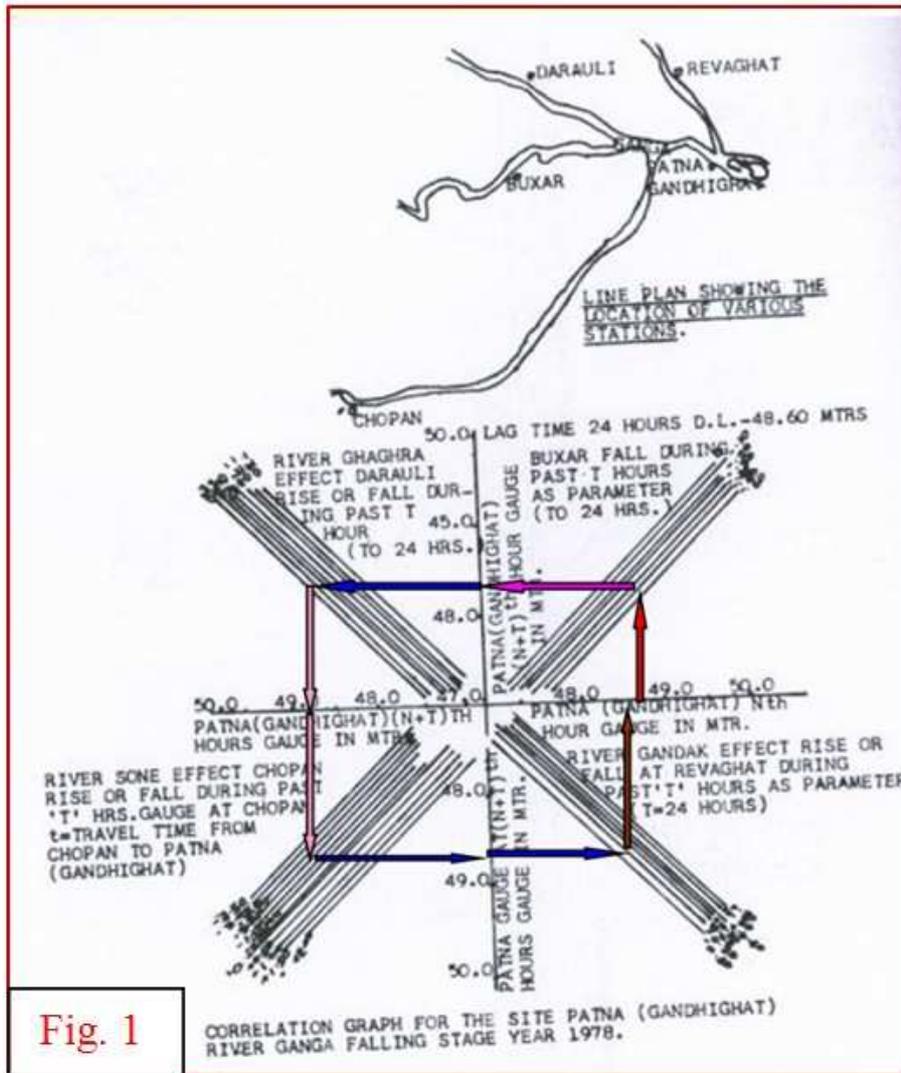
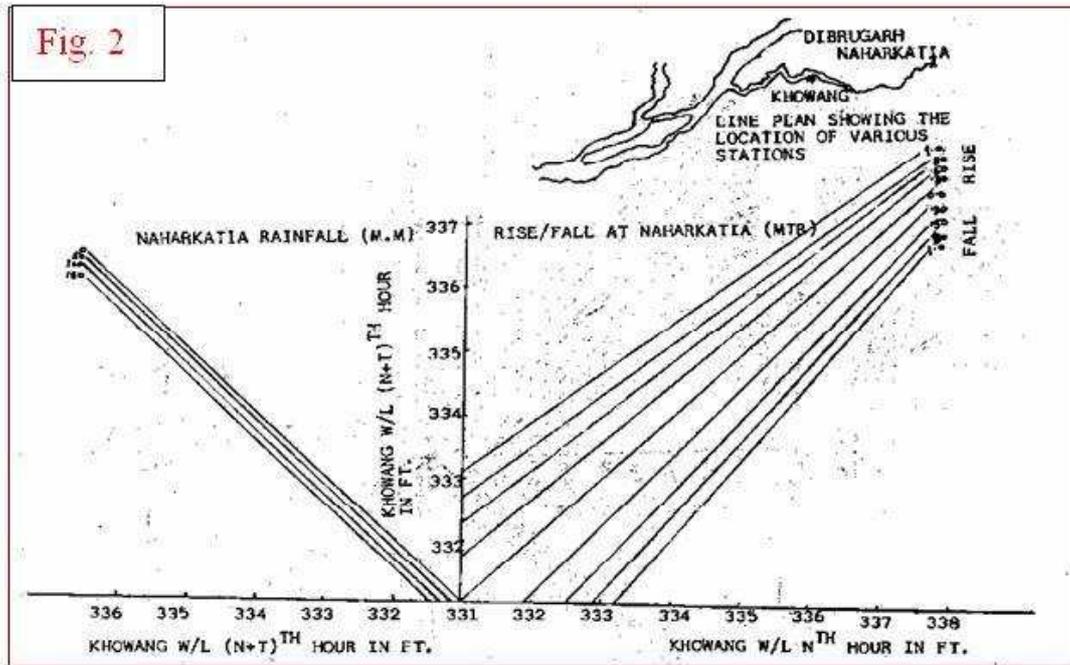Fig. 1 CORRELATION GRAPH FOR THE SITE PATNA (GANDHIGHAT) RIVER GANGA FALLING STAGE YEAR 1978.

This concept can also be extended to account for rainfall in upland area. Fig.2 is for formulating the forecasts at Khowang on river Brahmaputra considering rise and fall in water level at Naharkatia site. Additionally, rainfall observation at Naharkatia is also accounted for to incorporate its likely influence to the water level at Khowang.

Fig. 2

DIBRUGARH
NAHARKATIA
KHOWANG
LINE PLAN SHOWING THE LOCATION OF VARIOUS STATIONS

NAHARKATIA RAINFALL (M.M)   RISE/FALL AT NAHARKATIA (MTR)

RISE

FALL

KHOWANG W/L $(N+T)^{TH}$ HOUR IN FT.

336 335 334 333 332 331 332 333 334 335 336 337 338

KHOWANG W/L $(N+T)^{TH}$ HOUR IN FT.   KHOWANG W/L N^{TH} HOUR IN FT.

However, these charts carry limitations in that they provide only peak flow or water level information, and drop no hint about the shape of likely flood hydrograph at forecast site. This aside, there is absence of statistical test to measure the strength of correlation between dependent and independent variables. Nevertheless, such diagrams are proved quite useful in absence of fully developed network of hydro-meteorological stations; skilled personnel to operate sophisticated models; and seamless flow of data from remote locations to forecast centre.

b. Mathematical Equations

Unlike previous method, this method defines relationship mathematically among variables by 'regression/multiple regression techniques'. The strength of such pattern is easily determined by correlation coefficient, 'r', and thus subjective judgment of a person in drawing a best-fit line is eliminated. Mathematical equations offer much ease in calculation of dependent variable, and in turn speed up forecast process. Chart at Fig. 3 displays an equation that estimates water level at downstream location, Mahemdabad, Gujarat with change in water level at upstream site. A respectable degree of r_2 as 0.9854 is achieved by introducing a time lag/shift of 4 hrs between two sets of data. The arrival of this time lag is based on output obtained through cross-correlogram technique. With no time lag, two sets of data are poorly correlated.

Fig. 3
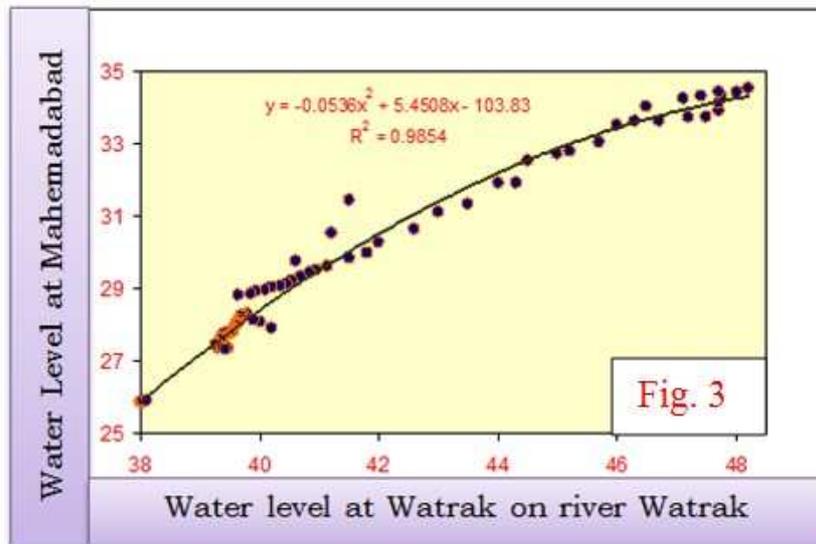
Another approach is to develop a mathematical model relating forecast station water level with water level of a tributary joining in-between base and forecasting station, and of base station. This method is elaborated by an example comprising three stations. Location of stations may be visualized as shown in Fig.4. Table 1 lists water levels observed at these locations.
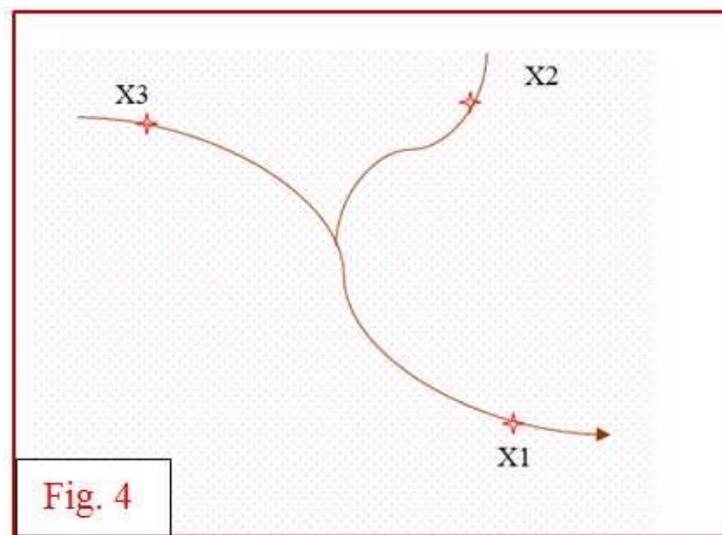

Fig. 4

Table 1

Water level at forecasting station in m (X1 observed)	tributary water level in (X2)	water level at base station in m (X3)	X1 computed or Estimated
80.3	112.2	165.3	80.6
80.5	112.2	165.8	80.6
82.2	112.7	166.6	82.0
84.3	113.3	167.2	83.6
85.6	114.0	168.0	85.5
85.7	114.1	168.4	85.8
86.0	114.3	168.8	86.3
86.3	114.3	169.3	86.3
87.1	114.6	170.2	87.1
$\Sigma X1 = 758.0$	$\Sigma X2 = 1021.7$	$\Sigma X3 = 1509.6$	
Standard Deviation = 2.578			Sest = 0.357

A linear multiple regression equation with X1 as dependent variable and X2, X3 independent variables can be expressed as below.

$$X_1 = a + b\,X_2 + c\,X_3$$

The coefficients a, b and c are estimated by the method of least square. Three equations noted down below help solve three unknowns.

$$\Sigma X_1 = an + b\Sigma X_2 + \Sigma X_3$$
$$\Sigma X_1.X_2 = a\Sigma X_2 + b\Sigma X_2{}^2 + c\Sigma X_2.X_3$$
$$\Sigma X_1.X_3 = a\Sigma X_3 + b\Sigma X_2.X_3 + c\Sigma X_3{}^2$$

This can also be arranged in matrix form. Below is an arrangement of three matrices; where [X1], [R] and [A] are 3*1, 3*3, 3*1 matrix

ΣX_1	=	n	ΣX_2	ΣX_3	a
$\Sigma X_1.X_2$	=	ΣX_2	$\Sigma X_2{}^2$	$\Sigma X_2.X_3$	b
$\Sigma X_1.X_3$	=	ΣX_3	$\Sigma X_2.X_3$	$\Sigma X_3{}^2$	c

Substituting all summation terms, we get

[X1] 3*1		[R] 3*3			[A] 3*1
758.0	=	9	1021.7	1509.6	a
86069.2	=	1021.7	115992.8	171385.2	b
127174.5	=	1509.6	171385.2	253231.7	c

Matrix [A] containing all three unknown coefficients is solved by multiplying [R]-1 matrix with [X1] matrix.

$$[A] = [R]^{-1}\,[X1]$$

Solving the matrix in MS Excel results regression equation as below:

$$X_1 = -223.017 + 2.71\,X_2 - 0.0003\,X_3$$

Computed/estimated X1 with the help of this equation appears in the last column of first table. Correlation coefficient, r for the defined equation is **0.99** suggesting higher degree of correlation among variables and can be adopted as forecast model.

Standard Deviation (SD) and S_{est} are needed to estimate r2. To determine SD, reader may consult flood frequency module. S_{est} is determined by $\underline{}/\Sigma(X_{i,\ obs} - X_{i,comp})^2/(n-2)$. Finally, r^2 is determined by following equation.

$$S_{est} = SD\ (\text{or } \sigma)\ \sqrt{(1 - r^2\ )}$$

Equation of the type $X_1 = a\,.\,X_2{}^{b1}\,.\,X_3{}^{b2}$ .. can also be evaluated by converting them into a linear form by logarithmic transformation. Secondly, in the current example, two independent variables are water level. Reader can substitute it by other variables or add more variables to this equation. Solution of coefficients follows similar steps.

Another relationship derived by multiple regression technique determines the change in water level at forecast site bases on the variations recorded at two upstream sites, commonly known as base stations. While preceding equation relates water levels of two sites, this equation

correlates variation in water levels at different sites.

A Mathematical Model using Muskingum Outflow Equation (after Hydrology by H M Raghunath)

According to Muskingum method, outflow and inflow at two time steps, (t+1)& trelated to by equation (I).

$$O_{t+1} = C_o I_{t+1} + C_1 I_t + C_2 O_t \quad ..... \text{ (I)}$$

For a few initial time steps of observed inflow hydrograph, such as I_1, I_2 ---, I_4, and O_1, O_2 ---, O_4 of outflow hydrograph, a set of equations, with the help of eq. (I) can be written as below
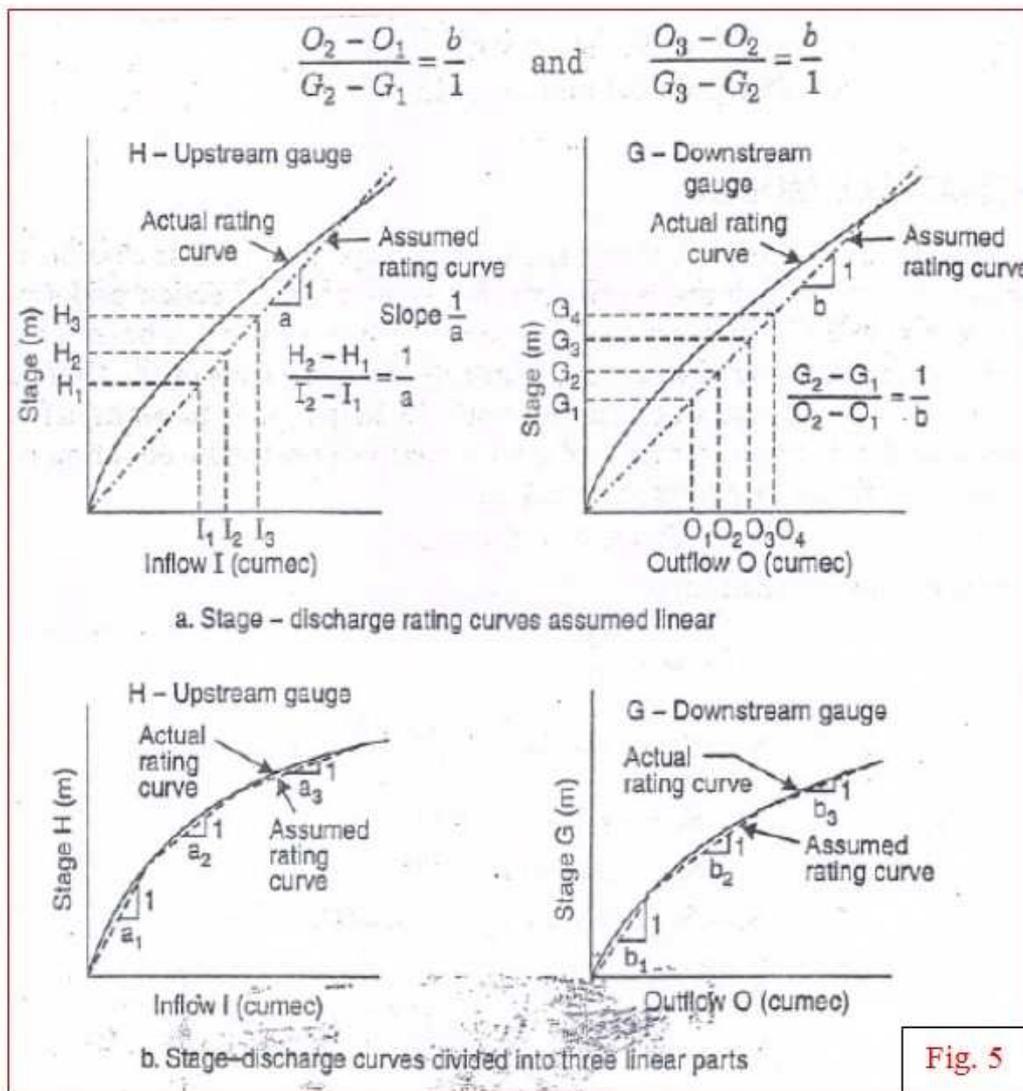
$$(O_3 - O_2) = C_o (I_3 - I_2) + C_1 (I_2 - I_1) + C_2 (O_2 - O_1) \quad ..... \text{(II)}$$

$$(O_4 - O_3) = C_o (I_4 - I_3) + C_1 (I_3 - I_2) + C_2 (O_3 - O_2) \quad ..... \text{(III)}$$

Now, assuming that discharge and water level curves at either location a straight line; and denoting water level at upstream and downstream sites as H & G respectively (Fig. 5), we can say that

$$(I_3 - I_2)/(H_3 - H_2) = (I_2 - I_1)/(H_2 - H_1) = a$$

$$(O_3 - O_2)/(G_3 - G_2) = (O_2 - O_1)/(G_2 - G_1) = b$$

$$\frac{O_2 - O_1}{G_2 - G_1} = \frac{b}{1} \quad \text{and} \quad \frac{O_3 - O_2}{G_3 - G_2} = \frac{b}{1}$$

a. Stage – discharge rating curves assumed linear

b. Stage–discharge curves divided into three linear parts

Fig. 5

Replacing discharge component of equations (II) & (III) with this, we get

$$(G_3 - G_2) = C_2(G_2 - G_1) + C_0 . (a/b) . (H_3 - H_2) + C_1 . (a/b) . (H_2 - H_1) \quad ...(IV)$$

$$(G_4 - G_3) = C_2(G_3 - G_2) + C_0 . (a/b) . (H_4 - H_3) + C_1 . (a/b) . (H_3 - H_2) \quad ....(V)$$

Or, we can simply write these equations as

$$(G_4 - G_3) = x_1(G_3 - G_2) + x_2 . (H_4 - H_3) + x_3 . (H_3 - H_2) \quad ... (VI), \text{ or}$$

$$G_{4,3} = x_1 G_{3,2} + x_2 H_{4,3} + x_3 . H_{3,2}$$

Equation (VI) combines change in water level at downstream site with changes in water level at upstream site. At this stage, *__reader may please note thatsubscripts 4, 3, 2 in above equation denotes difference water level at time (t+1), t & (t-1) at respective stations__*. A set of equations,like this, may be obtained by suitably picking up data from observed hydrographs to estimate coefficients x_1, x_2, & x_3 by matrix method as elaborated earlier. While doing so, it is highly recommended to check 'r2' value to ensure that model is worth for the purpose it is defined. This sort of equation can be developed for rising and falling stages separately. Further refinement is possible by dividing stages into two or three ranges with each range represented by unique equation (please see Fig.5). Additionally, equation (VI) considers only one

station/site in the upstream. In case, water level at downstream site happens to be affected by more than one site, a modified Muskingum equation can be written as

$$G_{4.3} = x_1\,G_{3.2} + x_2\,H_{4.3} + x_3.\,H_{3.2} + x_4\,H'_{4.3} + x_5.\,H'_{3.2} + x_6\,H''_{4.3} + x_6.\,H''_{3.2}$$

Where, H, H', H''represent water level at three upstream sites. Number of equations formed in this manner need to be solved for coefficients by matrix method. The steps involved in the process with same set of data used in previous example are illustrated below:

Step 1

With two independent variables (water level at two upstream sites) and one dependent variable (water level at forecast station), equation (VI) takes following form.

$$G_{4.3} = x_1\,G_{3.2} + x_2\,H_{4.3} + x_3.\,H_{3.2} + x_4\,H'_{4.3} + x_5.\,H'_{3.2}$$

Step 2

With known water levels at respective sites (Fig. 4 & Table 1), a set of values each representing change in water level at various time interval is tabulated next in matrix form

[G] (7 * 1)		[H] (7 *5)					[X] (5 * 1)	
$G_{4.3}$		$G_{3.2}$	$H_{4.3}$	$H_{3.2}$	$H'_{4.3}$	$H'_{3.2}$		
1.7	=	0.2	0.5	0.0	0.8	0.5	$x_1=$	0.87
2.1	=	1.7	0.6	0.5	0.6	0.8	$x_2=$	1.78
1.3	=	2.1	0.7	0.6	0.8	0.6	$x_3=$	-3.88
0.1	=	1.3	0.1	0.7	0.4	0.8	$x_4=$	-1.12
0.3	=	0.1	0.2	0.1	0.4	0.4	$x_5=$	2.55
0.3	=	0.3	0.0	0.2	0.5	0.4		
0.8	=	0.3	0.3	0.0	0.9	0.5		

Please note that (7 * 1), (7*5) & (5 * 1) denote matrix size; G is water level at X1; H at X2; & H' at X2 site.

Values of coefficients are determined by solving the matrix by an equation given below. Here, equations are more than number of unknown. Therefore, H is a rectangular matrix.In MS excel, standard deviation, matrix multiplication, inverse and transpose commands are defined by STDEV, MMULT, MINVERSE, and TRANSPOSE respectively. Cell (s), where user intends to get result needs to be selected before executing the function.

$$[X] = [\,(H^T.H)^{-1}.\,[H^T].[G]$$

With coefficients indicted in above table, a mathematical equation takes following form for use in flood forecast.

$$G_{4.3} = 0.87G_{3.2} + 1.78\,H_{4.3} - 3.38\,H_{3.2} - 1.12\,H'_{4.3} + 2.55\,H'_{3.2}$$

While seeking to define an equation by this approach, caution is needed toward the inherent assumption associated with the method, i.e. a plot between water level and discharge should closely follow a linear trend in that range for which user/forecaster intends to relate parameters. A comparison between observed vs. computed water level at X1 and relevant

statistical parameters which measure the strength of model are presented in Table 2.

	Observed Water level at X1	Computed Water level at X1	(Obs -Com)^2
	82.2	81.9	0.066
	84.3	84.2	0.016
	85.6	85.7	0.006
	85.7	85.8	0.007
	86.0	86.3	0.107
	86.3	85.9	0.127
	87.1	87.4	0.069
Sd =	1.611861 **Sest=**		0.315
r =		0.98	

Table 2

c. Hydrological Models

c.1 Rainfall-Runoff Models

i) Lumped Models

Example below is an excerpt from Manual on Flood Forecasting (p 239-244), published by CWC in 1989, wherein 3-hr duration unit hydrograph (owing to 1mm effective rainfall over the basin/catchment)for a basin area of 8570 sqkm is given along with mean rainfall events over the basin. The base flow at the beginning of storm is 300 cumec. As per the report received at 1900 hrs on 13th September, the average rainfall observed at different hours was as follows:


Outlet/Terminus point
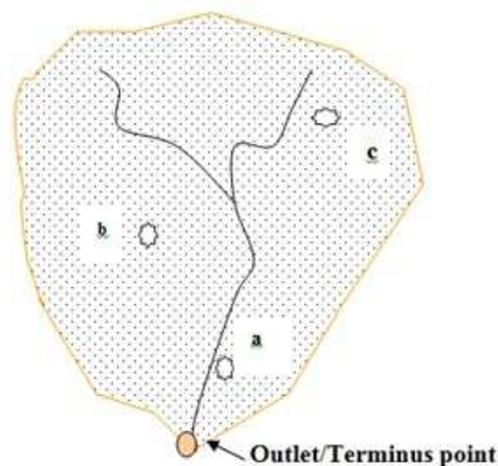
**Schematic Diagram of basin with locations
of three Rain gauges & its Outlet where
advance warning is intended**

Time (hrs.)	Unit Hydrograph (cumec)
0	0
3	8
6	34
9	103
12	130
15	141
18	119
21	83
24	59
27	40
30	28
33	20
36	14
39	8
42	4
45	2
48	0

Duration (Hrs.)	Rainfall Amount
09-12	19.4 mm
12- 15	20.0 mm
15-19	14.9 mm
Total Rainfall	**54.3 mm**

Additional information available is a diagram, Fig. 6(based on historical data) correlating total rainfall and runoff in varying base flow conditions (shown right). Referring to this diagram, runoff against a base flow of 300 cumec and 54.3 mm rainfall is 14.5mm implying a loss of 39.8mm during rainfall period. Form this, it is gathered that there is a loss rate of 4.43 mm per hour.

Fig. 6
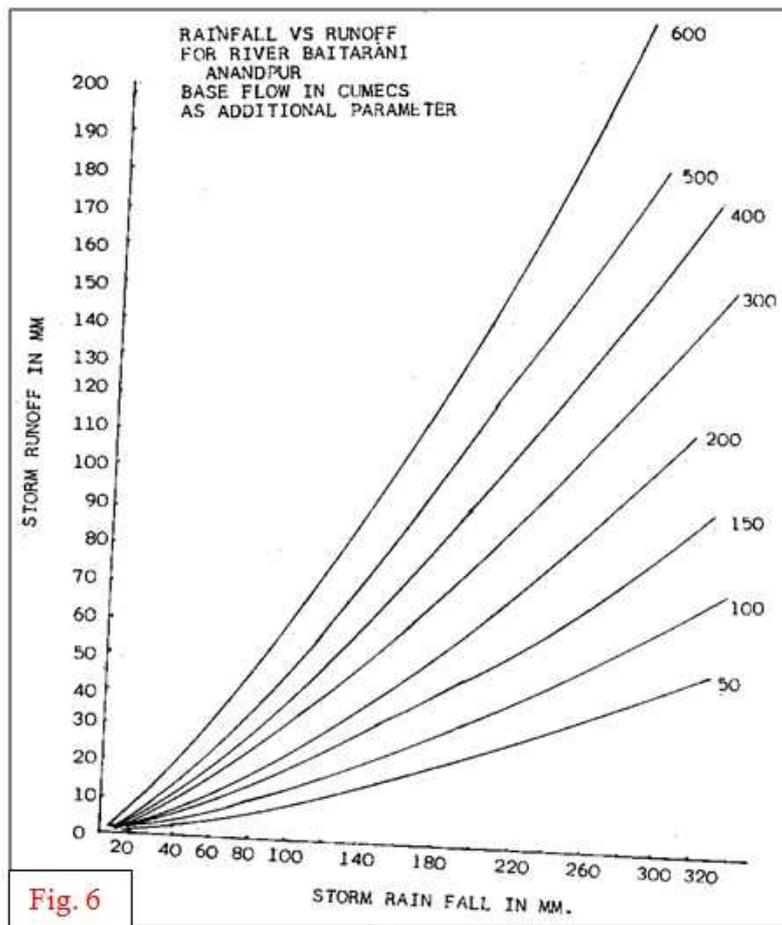
This point beyond, a model is developed in HEC-HMS by keying in information gathered as

above(Fig.7). Basin in the model is represented by an element 'subbasin1'. This element hosts basin information; loss mechanism;transformation process and base flow contribution besides observed hydrograph at terminus point, if available (Fig.8).Convolution of UH can be attempted in MS excel also. HEC-HMS software can be downloaded by visiting site **http://www.hec.usace.army.mil/software/hec-hms/download.html** , and is available for free.
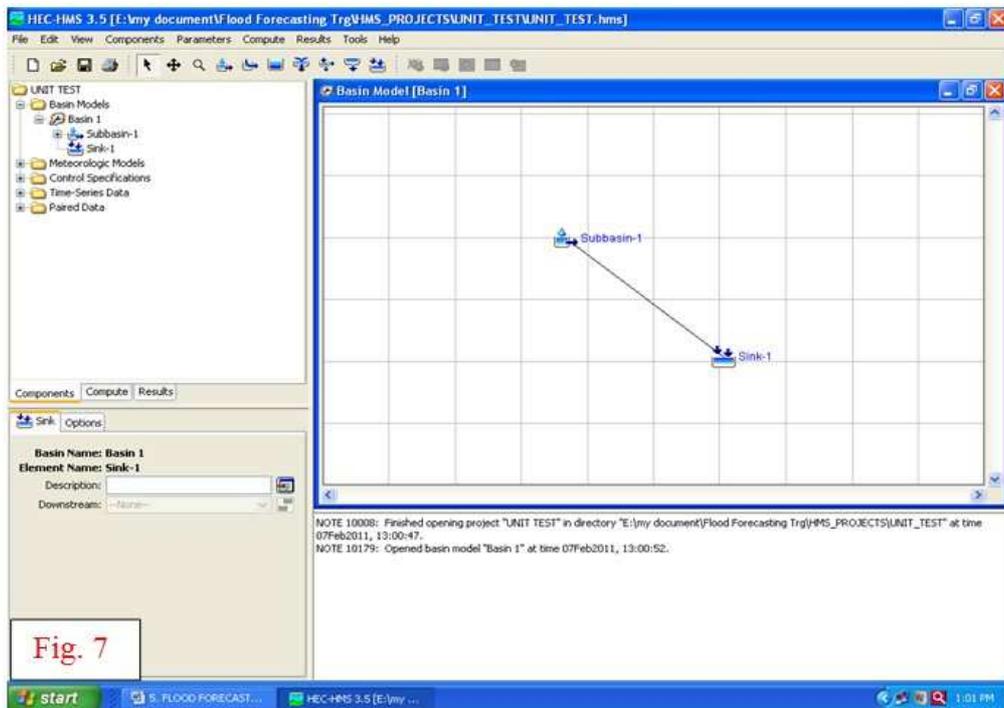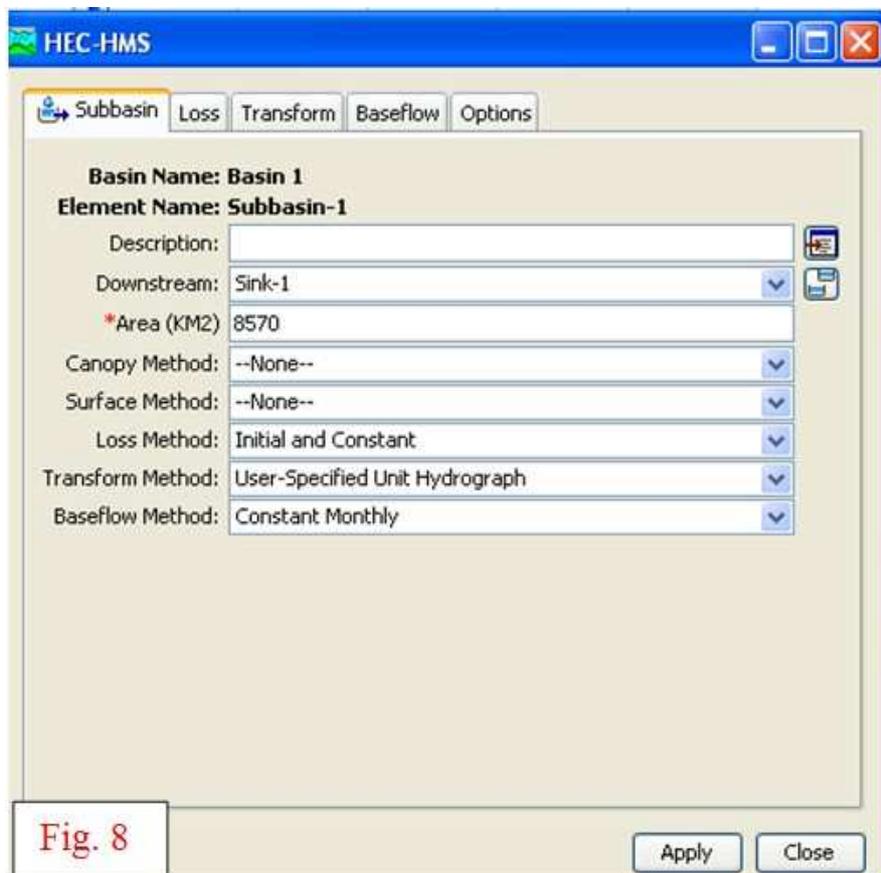

Fig. 7

Fig. 8

Once, model is ready with a network of elements and all inputs; a 'run' is performed to generate resulting flood hydrograph which closely resembles the ordinates listed in the Manual. A

hypothetical elevation vs. discharge rating is also fed in the model to produce water level corresponding to variation in discharge at terminal location (Fig. 9).
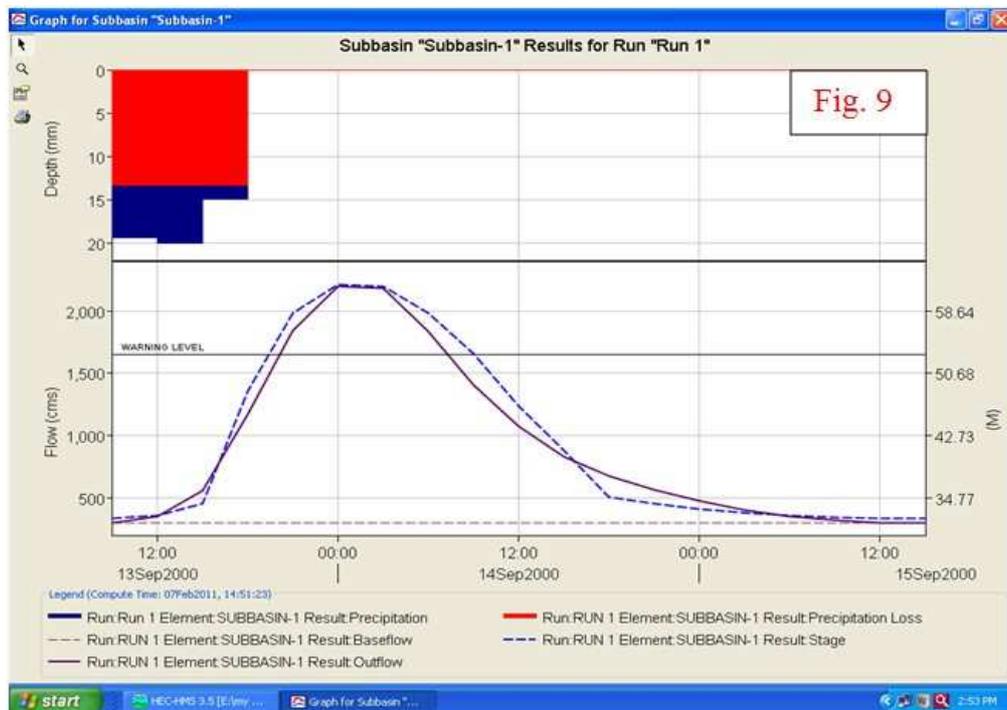

Table 3 lists a set of output information that is quite handy for issuance of forecast ahead of its actual occurrence. Even though the ordinate's interval is every three hours in the current listing, user can elect appropriate interval to extract information of his desire.

Table 3

Time-Series Results for Subbasin "Subbasin-1"

Project: UNIT TEST
Simulation Run: Run 1   Subbasin: Subbasin-1

Start of Run: 13Sep2000, 09:00   Basin Model: Basin 1
End of Run: 15Sep2000, 15:00   Meteorologic Model: Met 1
Compute Time: 07Feb2011, 14:51:23   Control Specifications: Control 1

Date	Time	Precip (MM)	Loss (MM)	Excess (MM)	Direc... (M3/S)	Base... (M3/S)	Total... (M3/S)	Stage (M)
13Sep2000	09:00				0.0	300.0	300.0	32.1
13Sep2000	12:00	19.40	13.29	6.11	48.9	300.0	348.9	32.5
13Sep2000	15:00	20.00	13.29	6.71	261.4	300.0	561.4	34.0
13Sep2000	18:00	14.90	13.29	1.61	870.3	300.0	1170.4	48.4
13Sep2000	21:00	0.00	0.00	0.00	1540.2	300.0	1840.2	58.4
14Sep2000	00:00	0.00	0.00	0.00	1899.6	300.0	2199.6	62.0
14Sep2000	03:00	0.00	0.00	0.00	1882.5	300.0	2182.5	61.8
14Sep2000	06:00	0.00	0.00	0.00	1532.6	300.0	1832.6	58.3
14Sep2000	09:00	0.00	0.00	0.00	1109.0	300.0	1409.0	53.2
14Sep2000	12:00	0.00	0.00	0.00	773.9	300.0	1073.9	46.5
14Sep2000	15:00	0.00	0.00	0.00	534.5	300.0	834.5	40.9
14Sep2000	18:00	0.00	0.00	0.00	374.5	300.0	674.5	34.8
14Sep2000	21:00	0.00	0.00	0.00	264.8	300.0	564.8	34.0
15Sep2000	00:00	0.00	0.00	0.00	175.0	300.0	475.0	33.4
15Sep2000	03:00	0.00	0.00	0.00	100.7	300.0	400.7	32.9
15Sep2000	06:00	0.00	0.00	0.00	51.9	300.0	351.9	32.5
15Sep2000	09:00	0.00	0.00	0.00	19.9	300.0	319.9	32.3
15Sep2000	12:00	0.00	0.00	0.00	3.2	300.0	303.2	32.2
15Sep2000	15:00	0.00	0.00	0.00	0.0	300.0	300.0	32.1

ii) Quasi Distributed Model

Following illustration demonstrates, with the help of Fig. 10, application of UH in conjunction with MUSKNIGUM routing method to estimate magnitude of flood and time of its occurrence. The set of data inputted here remains the same as for previous case. According to procedures illustrated in earlier example, UH considered for analysis represents an area of 8570 sqkm, and its convolution is based on average areal rainfall over the region. This concept of convolution of UH runs a risk of overestimating the flood because of departure from one of its fundamental assumption that rainfall is uniformly distributed over the region for a specified time. This may not be true for an area as large as 8570 sqkm. Additionally, this approach ignores likely impact of channel storage on flood attenuation. In order to adhere to this basic assumption, UH concept is usually applicable for an area less than or upto 5000 sqkm. Example quoted in the manual (p 244-253) overcomes this violation by sub-dividing the entire basin into three sub-basins A, B & C of area 2040 sqkm, 3470 sqkm and 3060 sqkm respectively, and assuming contribution of rain gauges a, b & c to respective sub-basins only. Accordingly, ordinates of UH has also been altered by a ratio between area of the respective part to the total area. Routing of flow along the reach is done by MUSKINGHUM method. Parameters K, X have been taken from the example and its stability is ensured by adhering to constraint, such as 2KX should be less than T. Additionally, equal contribution of 100 cumec as base flow from three parts is assumed. A basin delineated into three parts with two routing reaches is presented below. As discussed earlier, rainfall recorded at rain gauge 'c' contributes to sub-basin 'C', and therefore, its flood appears at basin outlet having propagated through reach -1 and reach -2. Similarly, sub-basin 'B' receives rainfall observed at rain gauge 'b', and resulting runoff travels through reach- 2 only. Sub-basin

'A' responds to rainfall at 'a', and its effect is visible at outlet (no routing is involved in this case).

Fig. 10

Sub-basins
C
B
A

Reach - 1

Reach - 2

Outlet/Terminus point
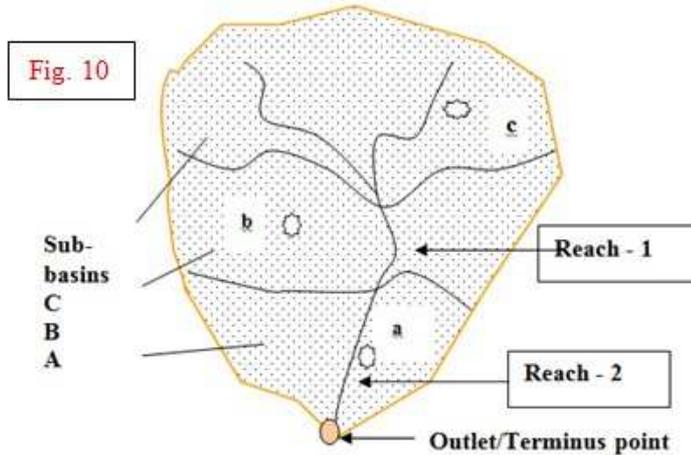
Basin partitioned into three parts. How to delineate it?
Various GIS based software offer excellent tools to perform this task quickly.

In agreement with discussion in preceding paragraph, UH for each basin& Muskingum parameters for two reaches are given in Tables 4 & 5.

Table 4

Time in hrs	UH basin A	UH basin B	UH basin C
24:00:00	0	0	0
3:00	1.904	3.24	2.856
6:00	8.092	13.77	12.138
9:00	24.514	41.715	36.771
12:00	30.94	52.65	46.41
15:00	33.558	57.105	50.337
18:00	28.322	48.195	42.483
21:00	19.754	33.615	29.631
24:00:00	14.042	23.895	21.063
3:00	9.52	16.2	14.28
6:00	6.664	11.34	9.996
9:00	4.76	8.1	7.14
12:00	3.332	5.67	4.998
15:00	1.904	3.24	2.856
18:00	0.952	1.62	1.428
21:00	0.476	0.81	0.714
24:00:00	0	0	0

Table 5

Muskingum Routing [Basin 1]

Show Elements: All Elements   Sorting: Hydrologic

Reach	Muskingum K (HR)	Muskingum X	Number of Subreaches
Reach-1	3	0.45	1
Reach-2	3	0.45	1

Apply   Close

Rainfall excess at each station is at Table 6.

Table 6

Duration (Hrs.)	Effective Rainfall at 'a'	Effective Rainfall at 'b'	Effective Rainfall at 'c'
09-12	15.0	0.0	0.0
12-15	0.0	14.0	2.7
15-19	0.0	0.0	12.3

A model (Fig. 11) duplicating three sub-basins and its reaches is created in HEC-HMS followed by data entry. HEC-HMS generated runoff at Outlet (Junction-2) appears at Fig. 12. Also shown there is change in water level according to fluctuation in discharge at this point of basin. Option is also available to mark warning level to distinguish critical period when there will be heightened risk because of swelling river.
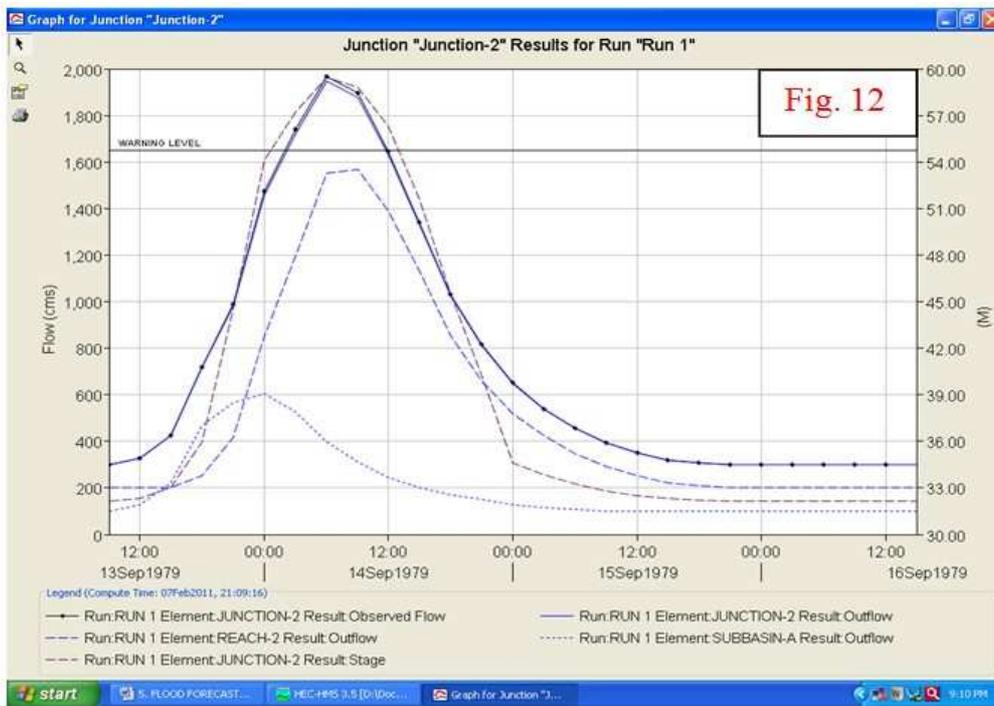

Fig. 11

Fig. 12
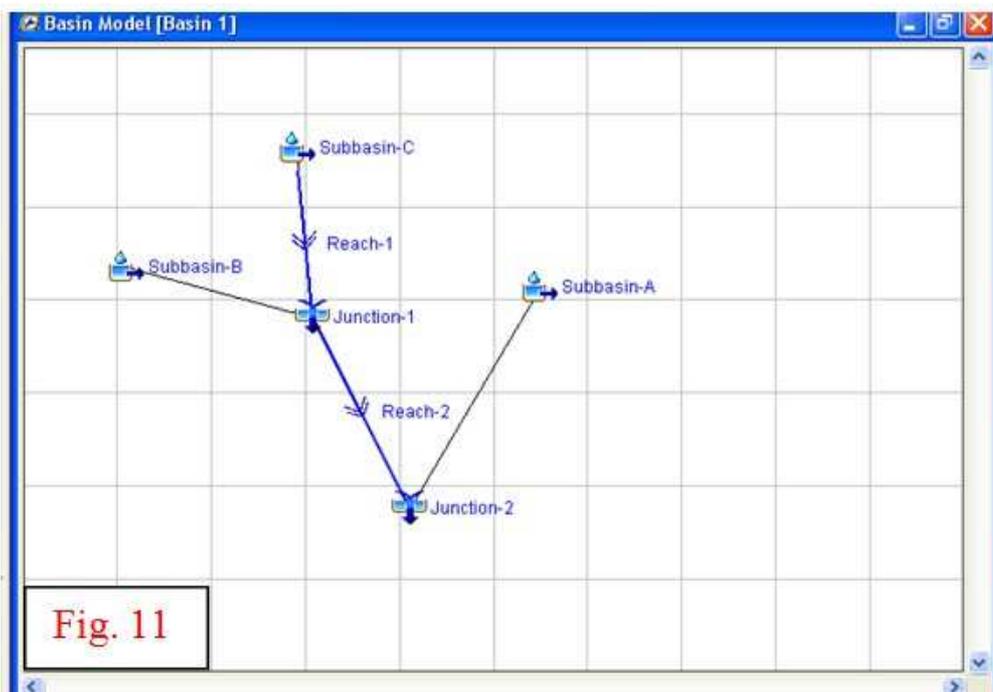
Table7 lists ordinates of flood hydrograph against time and compares with the result as presented in Manual. Options are also available in the HEC-HMS to observe resulting hydrographs for each and every element shown in model.

Table 7

Date	Time	Inflow from... (M3/S)	Inflow from... (M3/S)	Outflow (M3/S)	Obs Flow (M3/S)	Stage (M)
13Sep1979	09:00	200.0	100.0	300.0	300.0	32.1
13Sep1979	12:00	200.0	128.6	328.6	328.6	32.3
13Sep1979	15:00	202.2	221.4	423.6	423.6	33.0
13Sep1979	18:00	251.1	467.7	718.9	720.1	35.9
13Sep1979	21:00	417.4	564.1	981.5	986.8	44.5
14Sep1979	00:00	852.8	603.4	1456.1	1471.7	54.1
14Sep1979	03:00	1194.7	524.8	1719.5	1739.2	57.2
14Sep1979	06:00	1550.5	396.3	1946.8	1967.4	59.5
14Sep1979	09:00	1566.4	310.6	1877.0	1894.8	58.8
14Sep1979	12:00	1392.3	242.8	1635.1	1647.4	56.4
14Sep1979	15:00	1133.1	200.0	1333.1	1341.8	51.7
14Sep1979	18:00	855.0	171.4	1026.4	1032.4	45.5
14Sep1979	21:00	660.6	150.0	810.6	814.8	40.3
15Sep1979	00:00	520.0	128.6	648.6	651.6	34.6
15Sep1979	03:00	423.7	114.3	537.9	540.1	33.8
15Sep1979	06:00	348.2	107.1	455.3	456.5	33.3
15Sep1979	09:00	292.7	100.0	392.7	393.3	32.8
15Sep1979	12:00	251.4	100.0	351.4	351.7	32.5
15Sep1979	15:00	220.9	100.0	320.9	320.9	32.3
15Sep1979	18:00	209.1	100.0	309.1	309.1	32.2
15Sep1979	21:00	200.8	100.0	300.8	300.8	32.1
16Sep1979	00:00	200.1	100.0	300.1	300.1	32.1
16Sep1979	03:00	200.0	100.0	300.0	300.0	32.1
16Sep1979	06:00	200.0	100.0	300.0	300.0	32.1
16Sep1979	09:00	200.0	100.0	300.0	300.0	32.1
16Sep1979	12:00	200.0	100.0	300.0	300.0	32.1
16Sep1979	15:00	200.0	100.0	300.0		32.1

Time-Series Results for Junction "Junction-2"

Project: FLOOD FORECASTING
Simulation Run: Run 1   Junction: Junction-2

Start of Run: 13Sep1979, 09:00   Basin Model: Basin 1
End of Run: 16Sep1979, 15:00   Meteorologic Model: Met 1
Compute Time: 07Feb2011, 21:09:16   Control Specifications: Control 1

Points to note

- Regardless of duration of rainfall and its distribution over time, UH of known duration, say t-hr is to be fed in the HEC-HMS keeping its ordinates spaced at t-hr apart. For example, if UH of 1mm rainfall derived is of 3hr duration, ordinates of UH must be entered at 3hr interval. Software automatically converts this UH to duration according to rainfall distribution over catchment. This process rid us of steps needed for conversion of a UH from one duration to another.
- A relationship between stage/water level and corresponding discharge at forecast station is best represented by a rating curve or a power equation of the type $Q = c*(G-G_0)^{\wedge b}$.

  Caution is required here to feed latest rating curve of the site in the software; which is best estimate of the prevailing river regime. Secondly, fitting a rating curve does need some technical skill. HYMOS software performs this task with ease.

iii) Distributed Model

Forecast estimated by applying hydrological models such as one presented in preceding paragraphs tends to vary widely from real values, where assumptions in unit hydrograph or routing models are violated by prevailing hydro-meteorological conditions over catchment. For example, rainfall is non-uniform over the basin; it is not stationary and moving across the basin;

rainfall is concentrated in one pocket and leaving holes elsewhere. Apart from this, soil type and land-use pattern also vary over the catchment/basin that govern the rising and falling limb of resulting hydrograph. In lumped model, these characteristics are represented by a single SCS CN (curve number) applicable for entire area under study. Scenario,where spatial and temporal variations are dominating factors, demands application of distributed model to accurately capture the basin response.

Presented here is a distributed model developed and analyzed using Water Modeling System (WMS) and HEC-HMS. WMS software developers, on request, provide one time license key for 14 days for software evaluation purpose only. For its continued use, one needs to buy it. In this model, WMS software first delineates a watershed for an outlet point selected by the user, and thereafter creates grid (Fig. 13 & 14).
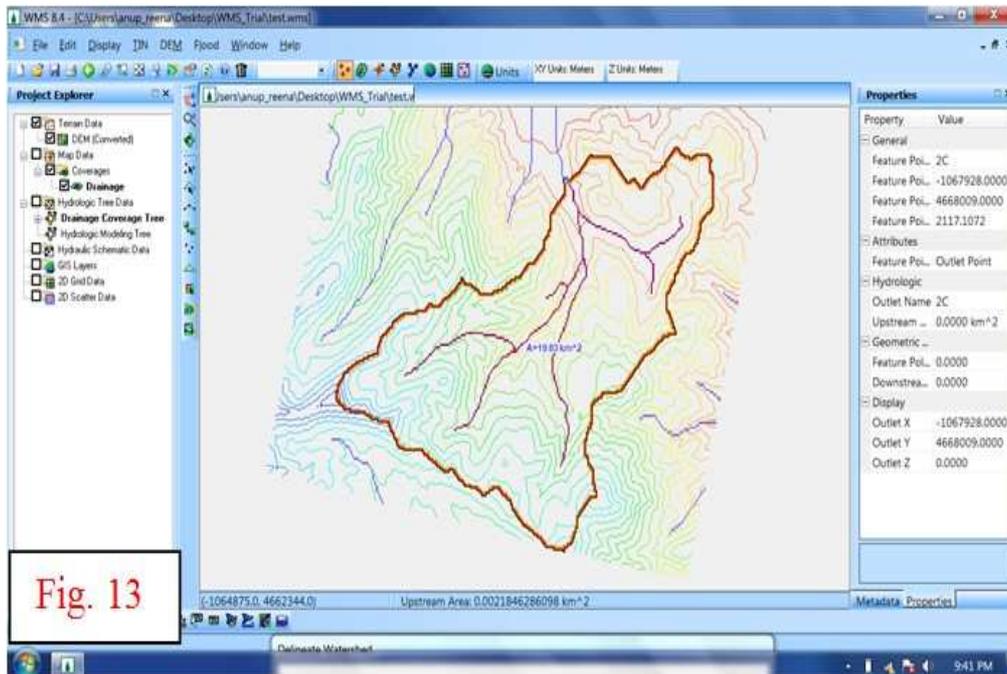
Fig. 13
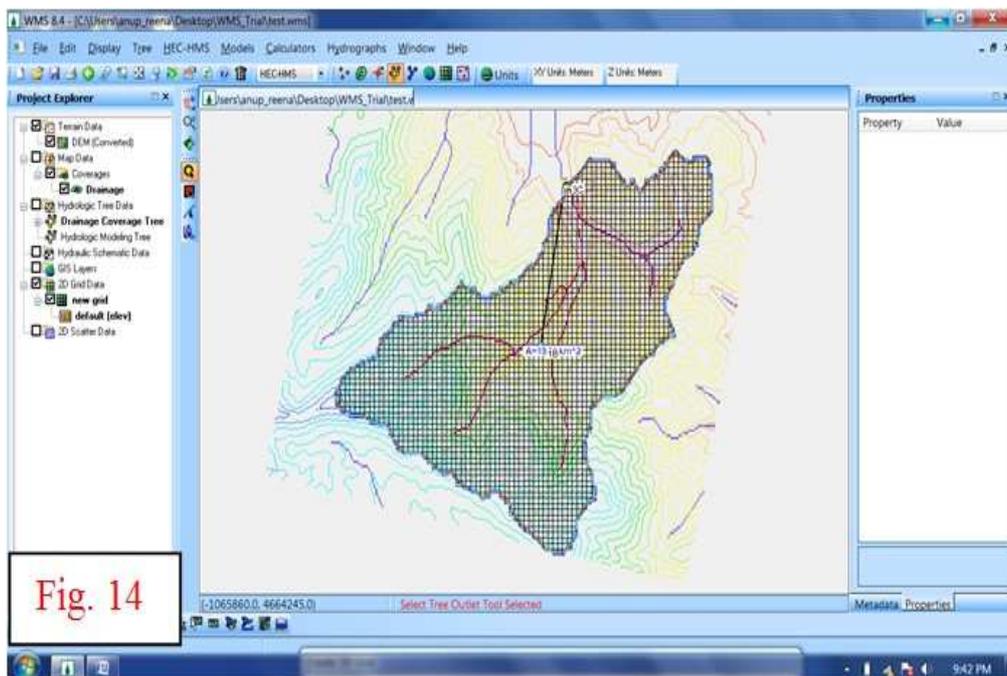
Fig. 14

For each grid, it determines CN values according to its soil type and land-use cover. Once land component is over, software prepares a gridded precipitation database based on rainfall input

provided by the user. A set of these gridded information are subsequently exported to HEC-HMS for simulation run. HEC-HMS calculates the rainfall excess and route it to outlet by MODCLARK method (Fig. 15). A couple of screenshots display model set-up and results obtained at the end. WMS software can be downloaded by visiting site
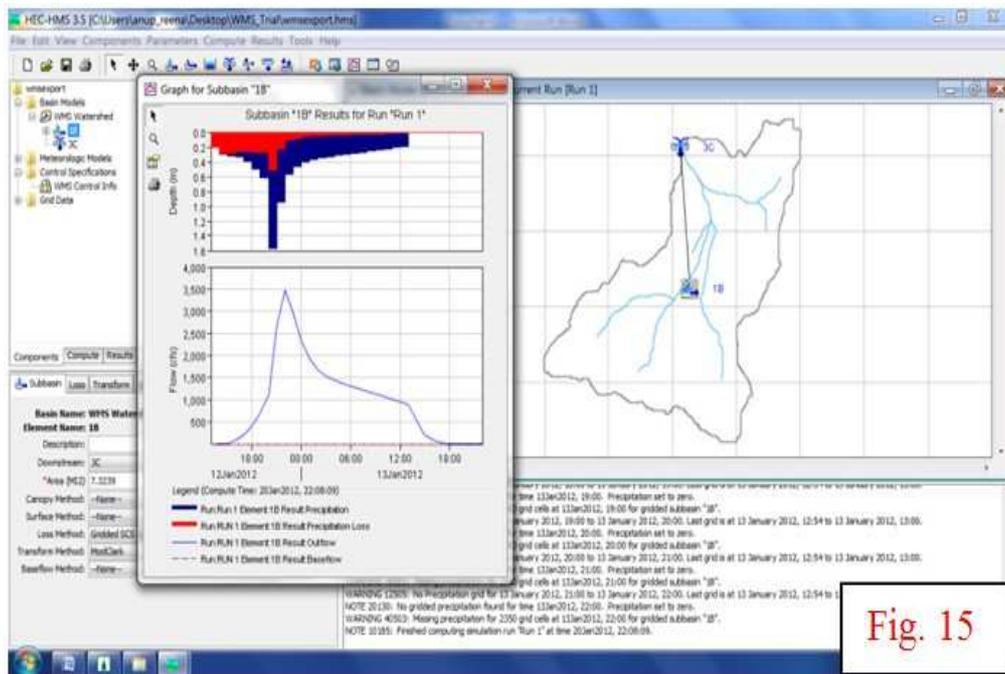**http://www.aquaveo.com/downloads.**

Fig. 15

c.2 Routing Techniques

i) Lumped, & Distributed Routing

(a) Storage equation

The Muskingum method of stream flow routing is most frequently used because of its simplicity, as it works with known inflow hydrograph and some fitted parameters without seeking additional information. However, in order to get high degree of accuracy, this method should be for gradually varied flow and not in cases where reach is often affected by backwater or unsteady flow condition. The two fundamental equations for stream flow routing by Muskingum method are:
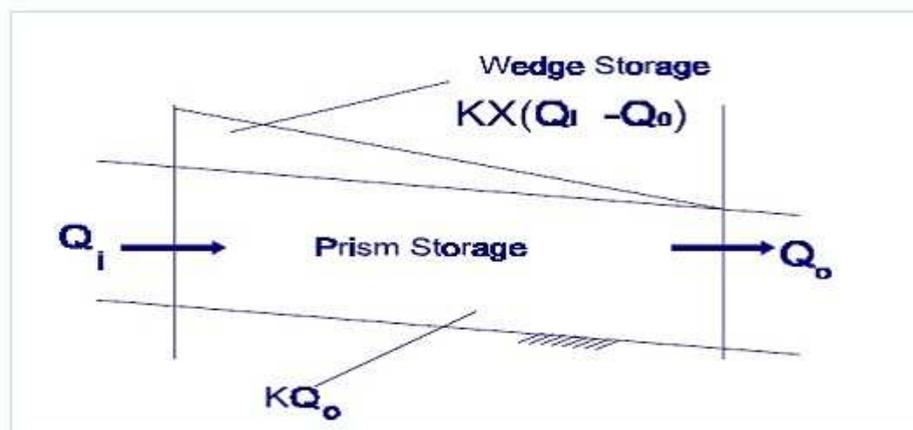
S = Prism storage + Wedge storage

$$= K.Q_0 + K.X.(Q1-Q_0)$$

Where,

S = Total Storage

K= A constant in time unit denotes the time of travel of flood wave through the reach. So, if flood wave velocity or celerity is C, C equals L/K, where L is reach length.

X = A dimensionless factor which defines the relative weights given to inflow and outflow in determining storage. (Mostly varies between 0.1 & 0.3 and ranges between 0 & 0.5)

Q1 or I = Inflow rate.

Q_0 orO = Outflow rate.

(b) Continuity Equation

$$\frac{I_t + I_{t+1}}{2}\Delta t - \frac{O_t + O_{t+1}}{2}\Delta t = S_{t+1} - S_1$$

Where,

I_t, I_{t+1} = Inflow rate at time t & t + Δt

O_t, O_{t+1} = Outflow rate at time t & t + Δt

S_t, S_{t+1} = Storage at time t & t + Δt

Combining these two equations we, get

$$O_{t+1} = C_0 I_{t+1} + C_1 I_t + C_2 O_t \text{------------ (I)}$$

The coefficients C_0, C_1 and C_2 are given respectively by

$$C_0 = \frac{0.5\Delta t - Kx}{K - Kx + 0.5\Delta t} \qquad C_1 = \frac{0.5\Delta t + Kx}{K - Kx + 0.5\Delta t} \qquad C_2 = \frac{K - Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t}$$

The coefficients are connected by the relation

$$C_0 + C_1 + C_2 = 1$$

Equation (I) with known coefficients, C_0, C_1 & C_2 computes outflow with inflow and outflow at time t & t+1. However, accurate estimation/selection of K, X, Δt and subdivision of river reach is central to successfulMuskingum routing. That is why these parameters are also termed as *'tuning knobs' of the model and merit due attention at the time of their estimation.*

Determination of K and X

Even if the feasible range for the parameter X is (0, 0.5), there are other constraints apply to selection of X. With Muskingum routing, the distance step, Δx, is defined indirectly by the number of steps into which a reach is divided for routing.

We will dig into example data set presented below (Table 8) to estimate these values. Later with

HEC-HMS, optimized value for these parameters will be extracted. As with other models, $\Delta x/\Delta t$ is selected in a manner to approximate c, where c = average wave speed (also celerity) over a distance increment Δx.

If total reach length is L, and travel time is K, Wave speed, C is

$$c = L/K = L/n.\Delta t$$

If there are n sub-reaches, and each sub-reach requires Δt time for discharge to flow past, $K = n.\Delta t$

So, the number of steps,

$$n = K/\Delta t.$$

For current example, distance between two stations is around 112 km;an estimated value of K is assessed about 34 hour with table 1(time interval between inflow peak and outflow peak- a rough estimation for K to begin with). Inflow flood hydrograph ordinates are at 2 hours interval, the routing reach should be divided in 17 steps, i.e. 34/2 to get the outflow hydrograph 112 km below. This leads to less attenuation as compared to routing carried out in a single step for the entire reach.

Secondly, the parameters K, X and computational time step ?t must also be selected in a manner so as to ensure that the Muskingum model/its coefficients must be rational. This implies that the parenthetical terms of the coefficients C1, C2 and C3 must be non- negative. To maintain this, values of K and X must be so chosen so that the combination falls within the shaded region shown below. In other words, K, X &Δt must satisfy a condition given by

$$2KX \leq \Delta t \leq 2K(1-X)$$

Parameters chosen in violation of this condition produce an un-stable solution and HEC-HMS will prompt the user in its message box accordingly.Against this backdrop and selected K and Δt, estimated X is 0.03 **($\Delta t/2K$).**
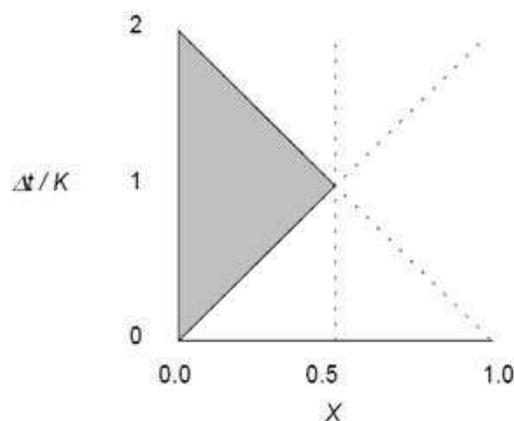


Optimal values of K and X for Muskingum method

Table 8

Date	Time	Inflow M³/S at Japla on river Sone	Outflow M³/S at Koelwar on river Sone
18-Aug-79	12:00	1,025.00	800
18-Aug-79	14:00	1,075.00	800
18-Aug-79	16:00	1,120.00	800
18-Aug-79	18:00	1,225.00	790
18-Aug-79	20:00	1,520.00	780
18-Aug-79	22:00	2,455.00	760
18-Aug-79	24:00:00	3,425.00	750
19-Aug-79	2:00	3,985.00	750
19-Aug-79	4:00	4,380.00	730
19-Aug-79	6:00	**5,035.00**	710
19-Aug-79	8:00	4,880.00	790
19-Aug-79	10:00	4,625.00	660
19-Aug-79	12:00	4,180.00	640
19-Aug-79	14:00	3,615.00	630
19-Aug-79	16:00	3,270.00	620
19-Aug-79	18:00	3,110.00	610
19-Aug-79	20:00	3,005.00	610
19-Aug-79	22:00	2,905.00	620
19-Aug-79	24:00:00	2,805.00	630
20-Aug-79	2:00	2,710.00	650
20-Aug-79	4:00	2,615.00	1,450.00
20-Aug-79	6:00	2,515.00	2,400.00
20-Aug-79	8:00	2,500.00	3,100.00
20-Aug-79	10:00	2,455.00	3,400.00
20-Aug-79	12:00	2,410.00	3,650.00
20-Aug-79	14:00	2,305.00	3,850.00
20-Aug-79	16:00	2,240.00	**3,930.00**
20-Aug-79	18:00	2,140.00	3,920.00
20-Aug-79	20:00	2,020.00	3,870.00
20-Aug-79	22:00	1,910.00	3,790.00
20-Aug-79	24:00:00	1,800.00	3,650.00
21-Aug-79	2:00	1,715.00	3,530.00
21-Aug-79	4:00	1,630.00	3,400.00
21-Aug-79	6:00	1,540.00	3,360.00
21-Aug-79	8:00	1,530.00	3,320.00
21-Aug-79	10:00	1,505.00	3,188.00
21-Aug-79	12:00	1,475.00	3,150.00
21-Aug-79	14:00	1,415.00	2,950.00
21-Aug-79	16:00	1,360.00	2,850.00
21-Aug-79	18:00	1,330.00	2,700.00
21-Aug-79	20:00	1,305.00	2,600.00
21-Aug-79	22:00	1,275.00	2,500.00
21-Aug-79	24:00:00	1,250.00	2,440.00
22-Aug-79	2:00	1,235.00	2,380.00
22-Aug-79	4:00	1,225.00	2,320.00
22-Aug-79	6:00	1,225.00	2,280.00
22-Aug-79	8:00	1,085.00	2,220.00
22-Aug-79	10:00	995.00	2,150.00
22-Aug-79	12:00	950.00	2,050.00
22-Aug-79	14:00	915.00	1,960.00
22-Aug-79	16:00	885.00	1,850.00
22-Aug-79	18:00	865.00	1,750.00
22-Aug-79	20:00	875.00	1,630.00
22-Aug-79	22:00	895.00	1,500.00
22-Aug-79	24:00:00	915.00	1,380.00
23-Aug-79	2:00	950.00	1,280.00
23-Aug-79	4:00	980.00	1,200.00
23-Aug-79	6:00	1,025.00	1,100.00
23-Aug-79	8:00	995.00	1,040.00
23-Aug-79	10:00	950.00	1,000.00
23-Aug-79	12:00	865.00	970
23-Aug-79	14:00	845.00	930
23-Aug-79	16:00	825.00	880
24-Aug-79	18:00	805	850
25-Aug-79	20:00	825	820

(Figure in bold indicates maximum value of that series)

Having defined Muskingum parameters, application of HEC-HMS in routing a reach by Muskingum method is demonstrated. A screenshot (Fig.16) displays the river network and its feeding boundary developed using Geo-HMS extension installed on ArcView software, and a model set up in the HEC-HMS.

Fig. 16

Message box pane of HEC-HMS
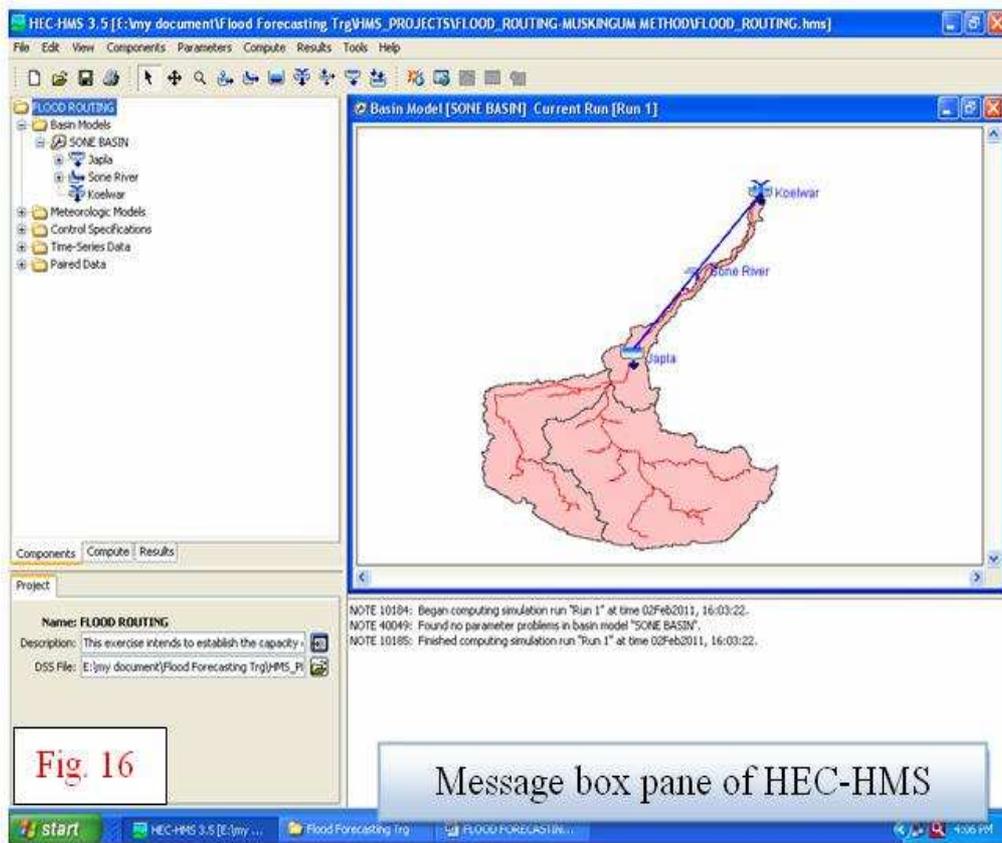
Routing parameters for reach entered in HEC-HMS is exactly the same as deliberated above (Fig.17).

Fig. 17
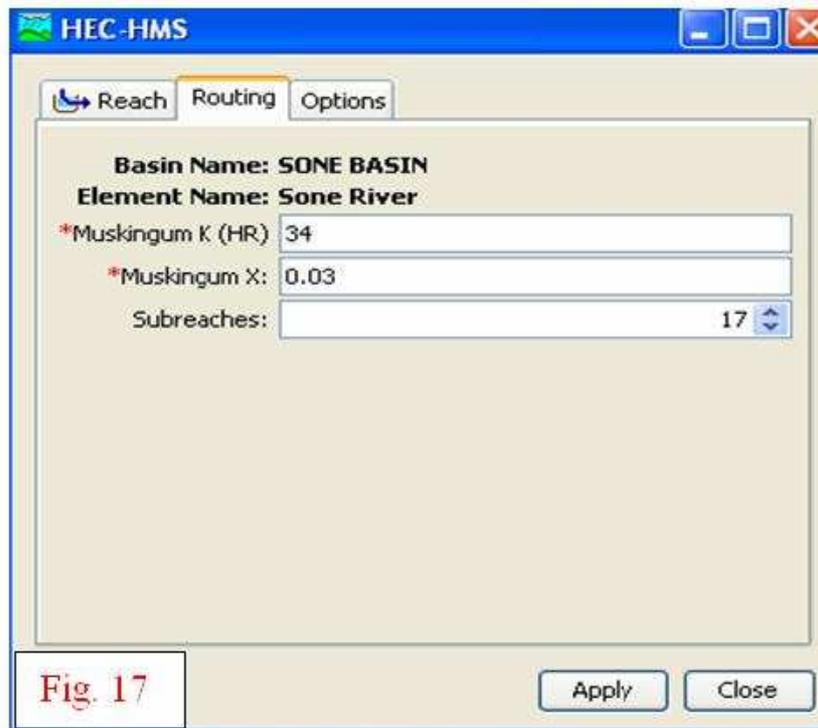
After first 'Run', screenshot (Fig. 18) plots the simulated result against observed discharge at downstream end along with variation in water level (water level vs. outflow relation used here is made-up one, and is used for demonstration purpose only). The water level profile can be picked up for forecast purpose. Also seen is inflow hydrograph at upstream end. Reader may notice the lower right box (also message box) of Fig. 16 reporting no instability with elected Muskingum parameters. Fig. 19 is summary result for this 'Run'.

At this stage, HEC-HMS offers another useful tool to optimize Muskingum parameters having finished first 'Run'. Screenshot at Fig.20 displays initial and optimized values of Muskingum parameters along with simulated and observed outflow, which is a slight improvement upon first 'Run'. Reader may adopt this new set of parameters to finalize their model for forecasting trial. In doing so, it must be noticed that river regime is not a fixed entity over time and tends to

exhibitcontinual changes in its course, geometry. That is why it had better develop a model based on flood events occurred recently, and discard earlier model to eliminate possibility of largevariation in forecast value.
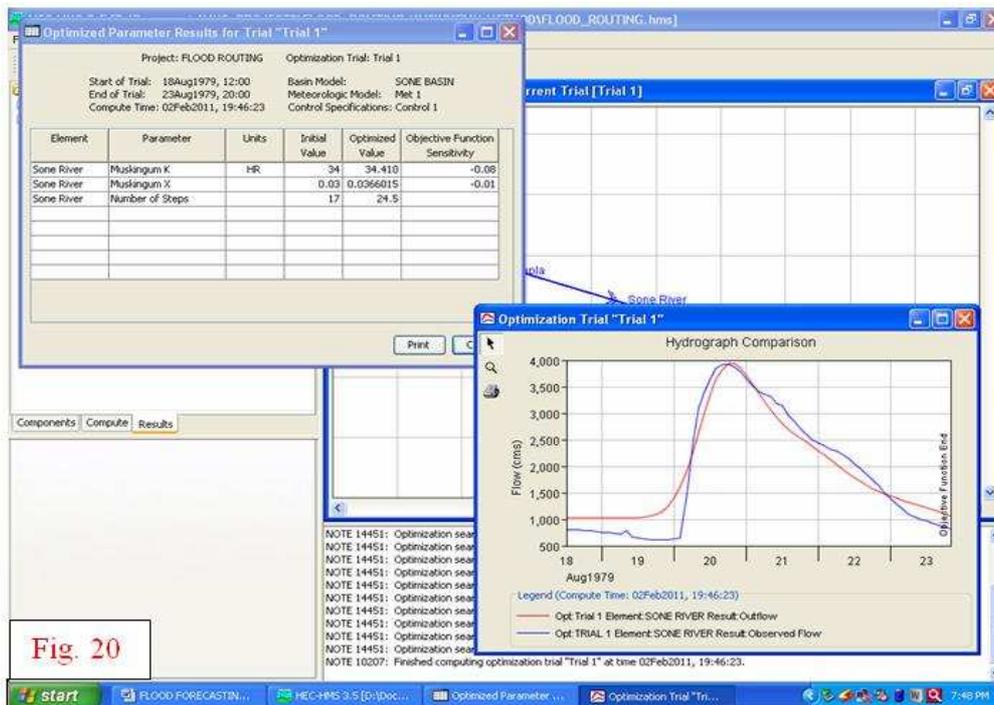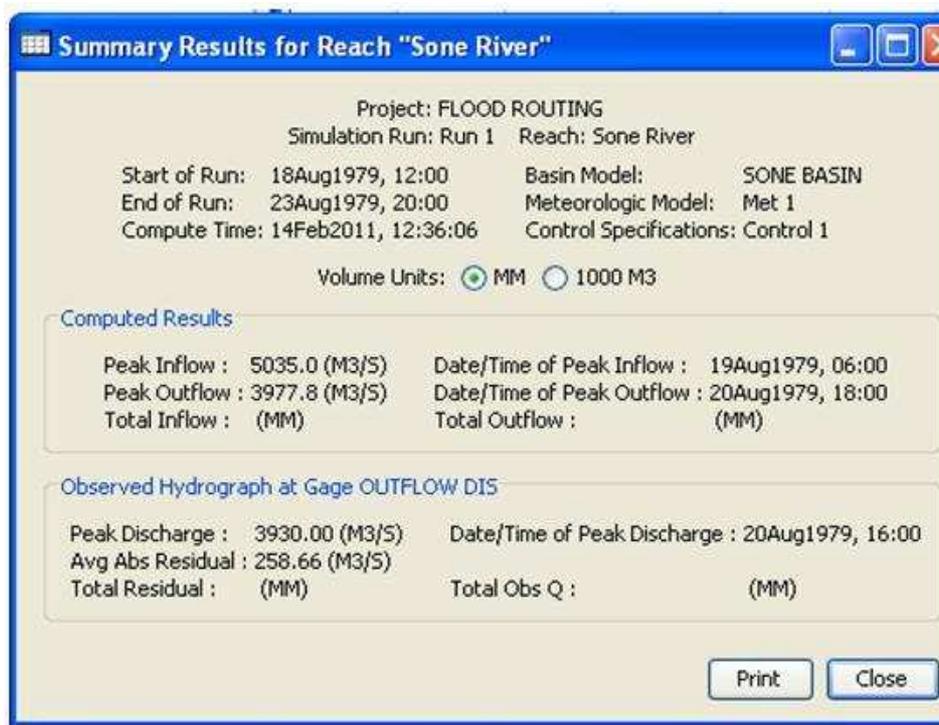

Fig. 20

Muskingum method operates with steady or gradually varied flow, and does not reveal any information in between the base and forecast station. That is why it is also termed as 'Lumped model'. If the forecaster intends to gather water profile and its propagation along river reach, distributed models, such as Muskingum-Cunge, Kinematic Wave could be alternatives. This module skips discussion about these methods for the present. Next version of this module will also discuss these methods at length. Nevertheless, keen readers may refer to 'Technical Reference Manual' of HEC-HMS for more information about it.

d. Hydraulic Models

i) Dynamic Wave Routing Technique

To demonstrate the strength of this approach through an example, the succeeding part of this module illustrates the application of HEC-RAS software in routing an unsteady flow hydraulically. HEC-RAS software is a one-dimensional flow hydraulic model designed to aid hydraulic engineers in channel flow analysis and flood-plain determination. The results of the model can be applied in floodplain management studies including flood forecasting. Like HEC-HMS, this software is also available for free, and can be downloaded from web-page **http://www.hec.usace.army.mil/software/hec-ras/hecras-download.htm**l .

By the end of this part of module, you will be able to:

- Import and edit cross-sectional geometry data
- Perform a unsteadyflow simulation for flood forecasting
- View and analyze HEC-RAS output and use GIS RAS mapper for flood delineation.

In this part, a real case study of unsteady flood modelling through HEC-RAS has been dealt with for a reach (Karad - Kurundwad, chainage 140 km to 260 km) in Krishna river. Lateral inflows to the main river on the corresponding dates have been considered at Sangam, where tributary Panchganga contributes to the Krishna flow. The technique provides a reliableinitialization of stage/discharge profile for the flood forecast. The examinations includingthe initialization of stage profile, conservation of mass, iteration convergence, Manning's N,effectiveness evaluation, and convergence with optimum theta (implicit weighing factor) values are conductedto verify the forecast capability.The forecasting results show that the stage recalculated by updatingthe Manning N, in current time has a good agreementwith the observed stage.

About Hydraulic Routing

Hydraulic routing employs the full dynamic wave (St. Venant) equations. These are the continuity equation and the momentum equation, which take the place of the storage-discharge relationship used in hydrologic routing. The equations describe flood wave propagation with respect to distance and time. Henderson (1966) rewrites the momentum equation as follows:

$$S_f = S_0 - \left(\frac{\partial y}{\partial x}\right) - \left(\frac{V \partial V}{g \partial x}\right) - \frac{1}{g}\frac{\partial V}{\partial t}$$

Where,
S_f = friction slope (frictional forces), in m/m;

S_o = channel bed slope (gravity forces), in m/m;

2^{nd}term = pressure differential;

3^{rd}term = convective acceleration, in m/sec^2;

Last term = local acceleration, in m/sec^2

$$q = A\left(\frac{\partial V}{\partial x}\right) - \left(VB\frac{\partial y}{\partial x}\right) - B\frac{\partial y}{\partial t}$$

The description of each term:

$A\left(\dfrac{\partial V}{\partial x}\right)$ = prism storage

$VB\left(\dfrac{\partial y}{\partial x}\right)$ = wedge storage

$B\left(\dfrac{\partial y}{\partial t}\right)$ = rate of rise

q = lateral inflow

The full dynamic wave equations are considered to be the most accurate solution to *unsteady*, one dimensional flow, but are based on the following assumptions used to derive the equations (Henderson, 1966):

- Velocity is constant and the water surface is horizontal across any channel section.
- Flows are gradually varied with hydrostatic pressure prevailing such that vertical acceleration can be neglected.
- No lateral circulation occurs.
- Channel boundaries are considered fixed and therefore not susceptible to erosion or deposition.
- Water density is uniform and flow resistance can be described by empirical formulae (Manning, Chezy) Solution to the dynamic wave equations can be divided into two categories: approximations of the full dynamic wave equations, and the complete solution.

Fully Dynamic Wave Routing solution

Complete hydraulic models solve the full Saint Venant equations simultaneously for unsteady flow along the length of a channel. They provide the most accurate solutions available for calculating an outflow hydrograph while considering the effects of channel storage and wave shape (Bedient and Huber, 1988). The models are categorized by their numerical solution schemes which include characteristic, finite difference, and finite element methods.
The finite difference method describes each point on a finite grid by the two partial differential equations and solves them using either an explicit or implicit numerical solution technique. Explicit methods solve the equations point by point in space and time along one time line until all the unknowns are evaluated then advance to the next time line (Fread, 1985). Implicit methods simultaneously solve the set of equations for all points along a time line and then proceed to the next time line (Liggett and Cunge, 1975). The implicit method has fewer stability problems and can use larger time steps than the explicit method. Finite element methods can be used to solve the Saint Venant equations (Cooley and Mom, 1976). The method is commonly applied to two-dimensional models.

Assumptions

The assumptions given above for all hydraulic models (one-dimensional flow, fixed channel, constant density, and resistance described by empirical coefficients) apply to dynamic routing. It is also assumed that the cross sections used in the model fully describe the river's geometry, storage, and flow resistance.

Limitations

The major drawback to fully dynamic routing models is that they are time-consuming and data

intensive, and the numerical solutions often fail to converge when rapid changes (in time or space) are being modeled. This can be addressed by adjusting the time and distance steps used in the model; sometimes, however, memory or computational time limits the number of time and distance steps that may be used. Additionally, fully dynamic one-dimensional routing models do not describe situations (such as lakes and major confluences) where lateral velocities and forces are important.

Starting a Project

Start the HEC-RAS 4.1.0 program. The following window should subsequently show up (Fig. 21).

Henceforth, this window will be referred to as the main project window. A **Projec**t in RAS refers to all of the data sets associated with a particular river system. To define a new project, select **File/New Project** to bring up the main project window (Fig. 22).

You first need to select your working directory, and then a title (say Karad_FF), and file name (Karad_FF.prj). All project filenames for HEC-RAS are assigned the extension ".prj". Click on the **OK** button and a window will open confirming the information you just entered. Again click the **OK** button. The project line in your main project window should now be filled in. The **Project Description** line at the bottom of the main project window allows you to type a detailed name for the actual short **Project** name. If desired, you may click on the ellipsis to the right of the **Description** bar, and additional space for you to type a lengthy **Description** will appear. Any time you see an ellipsis in a window in HEC-RAS, it means you may access additional space for writing descriptive text.

For each HEC-RAS project, there are three required components-

- **Geometry data-** The **Geometry** data, for instance, consists of a description of the size, shape, and connectivity of stream cross-sections.
- **Flow** data- **Flow** data contains discharge rates.
- **Plan** data- **Plan** data contains information pertinent to the run specifications of the model,

including a description of the flow regime.

Each of these components is explored below individually. The schematic picture in Fig. 23 depicts the Krishna -Koyna river confluence at Karad and we will be analyzing a reach Karad - Kurundwad.


Fig. 23
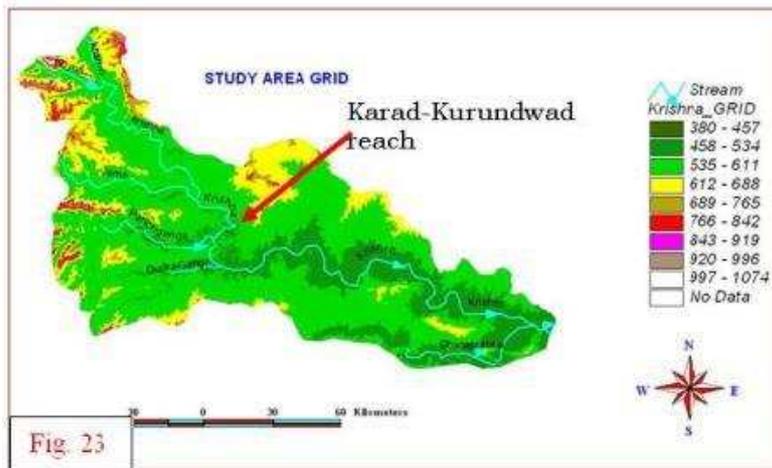
Importing and Editing Geometric Data

The first of the components, we will consider now, is the channel geometry. To analyze stream flow, HEC-RAS represents a stream channel and floodplain as a series of cross-sections along the channel. To create our geometric model, we can do it by three ways.

i) From HMS DSS files
ii) From Geo-RAS (derived from DEM/TIN)
iii) By manual entry of Geometric data

This HEC-RAS geometry file contains physical parameters describing cross-sections. To view the data, select Edit/Geometric Data from the project window. The cross sections of Krishna are obtained from Upper Krishna Division topographic survey record.
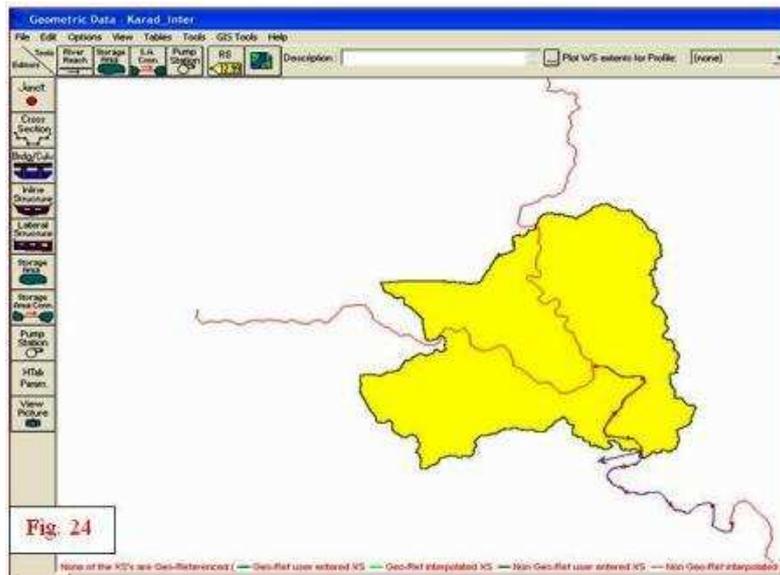
Fig. 24

The resulting view (Fig. 24)shows a schematic of Krishna& its tributary Koynariver with the area of study. This is the main geometric data editing window. The red tick marks denote individual cross-sections. Choices under the **View** menu provide for zoom and pan tools. The

six buttons on the left side of the screen are used to input and edit geometric data. The [River Reach ←] and [Junct. ●] buttons are used to create the reach schematic. A reach is simply a subsection of a river, and a junction occurs at the confluence of two rivers. Since our reach schematic is already defined, we have no need to use these buttons. The [Cross Section] , [Brdg/Culv] , [Inline Weir Spillway] and buttons are used to input and edit geometric descriptions for cross-sections, and hydraulic structures such as bridges, culverts, and weirs. The [View Picture] allows you to associate an image file (photograph) with a particular cross-section. Click on the [Cross Section] button to open the cross-section data window:

Confluence at Karad

Fig. 25
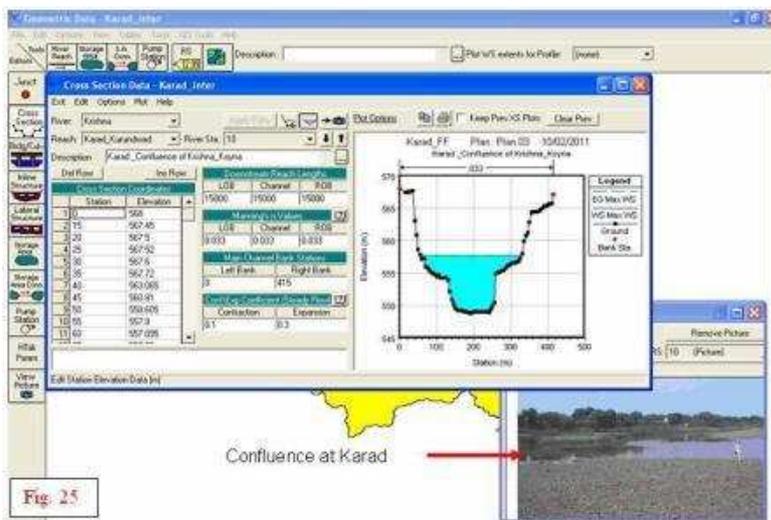
The data used to describe the cross-sections include the river station/XS number, lateral and elevation coordinates for each terrain point (station & elevation columns), Manning's roughness coefficients ✋ (Table 9), reach lengths between adjacent cross-sections, left and right bank station, and channel contraction and expansion coefficients (here 0.1 & 0.3 have been taken for smooth transitions) (refer to page 87 of Technical Reference Manual of HEC-RAS). These data are obtained by field surveys (Fig. 25).

Table 9

X-section No & Name	Manning's N (as computed from discharge)
10-Karad	0.033
7-Narasingpur	0.033
6-Khed	0.032
5-Arjunwad	0.031
1-Kurundwad	0.054

The [↓][↑] buttons can be used to toggle between different cross-sections. To edit data, simply double-click on the field of interest. You may notice that this action caused all of the data fields to turn red and it enabled the "Apply Data" button. Whenever you see input data colored **red** in HEC-RAS, it means that you are in edit mode. There are two ways to leave the edit mode:

1. Click the **"Apply Data"** button. The data fields will turn black, indicating you're out of edit mode, and the data changes are applied.
2. Select *Edit/Undo Editing*. You'll leave the edit mode without changing any of the data.

3. To actually see what the **Kurundwad** X-section (**Fig. 26**)looks like; select the **Plot/Plot Cross-Section** menu item on this window.

Fig. 26
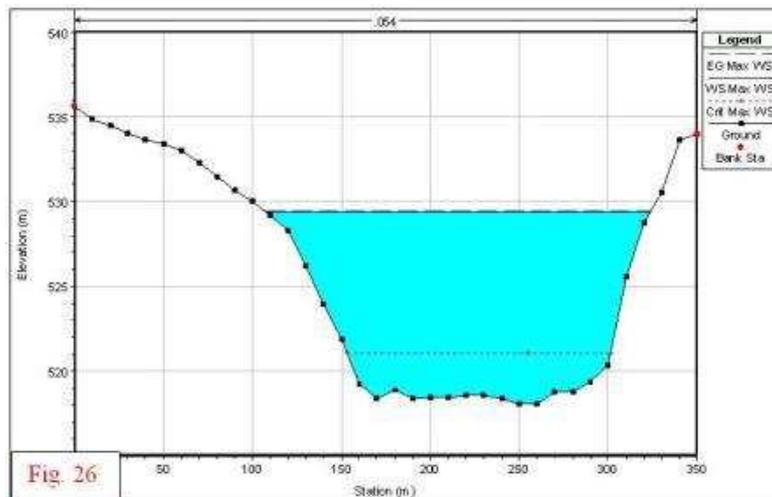
The cross-section points appear black and bank stations are denoted with red. Manning roughness coefficients appear across the top of the plot. Again, the ⬇⬆ buttons can be used to maneuver between different cross-sections. Any solid black areas occurring in a cross-section represent blocked obstructions. These are areas in the cross-section through which no flow can occur. Some cross-sections contain *green arrows and gray areas*. This symbolism is indicative of the presence of a bridge or culvert. Input data and plots specifically associated with bridges and culverts can be accessed from the main geometric data editor window by clicking on the button. Take a little time to familiarize yourself with the geometric data by flipping through some different cross-sections and bridges/culverts. When you are finished, return to the geometric editor window and select **File/Save Geometric Data**. Return to the main project window using **File/Exit** Geometry Data Editor.

Importing and Editing Flow Data

Enter the flow editor using Edit/Unsteady Flow Data from the main project window. Instead of importing an existing HEC-RAS flow file, you can use stream flow output from an HEC-HMS model run.

The coordinates of the cursor (time, flow rate) are displayed at the bottom right corner of the plot. Gridlines can be shown by invoking the Options/Grid menu item.

The direct step method uses a known water surface elevation (and several hydraulic parameters) to calculate the water surface elevation at an adjacent cross-section. For a sub-critical flow regime, computations begin at the d/s end. The present data set corresponds to1st& 2nd July, 2006 flood. The Flood Hydrograph at Karad (July 1 & 2, 2006) and Rating Curve at Kurundwad are entered as per the actual available dataset. Click on the Initial conditions and enter the value of initial flow at Karad. The initial flow of 888.04 m3/s at Karad on the day 1 at 0100 hrs is entered. Click on OK. The flood hydrograph and rating curve plots along with data view can be seen in Fig 28&29.

All of the required flow parameters have now been entered into the model. From the file menu, select Save unsteady Flow Data and save the flow data under the name "Karad flows."
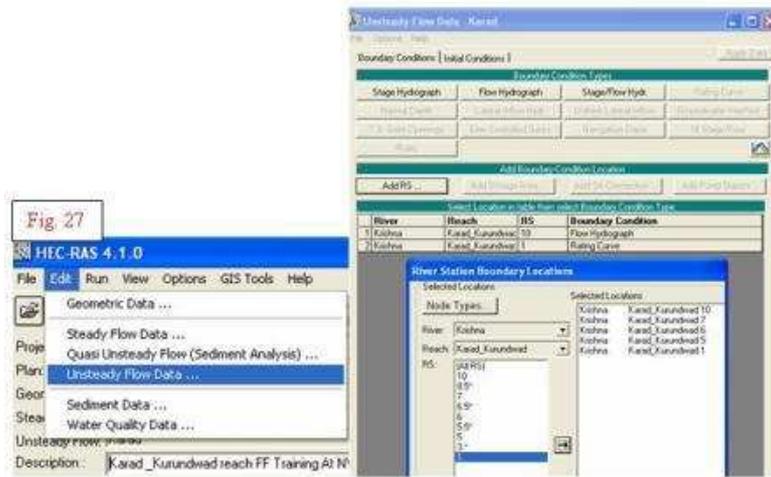
Fig. 27

Click on the button from the unsteady flow data window. HEC-RAS allows the user to set the **boundary conditions** and **Initial conditions** at the points (u/s, d/s or internal locations) or as shown in Fig. 27. The boundary conditions and initial flow conditions are filled in as per the actual data.


Fig. 28
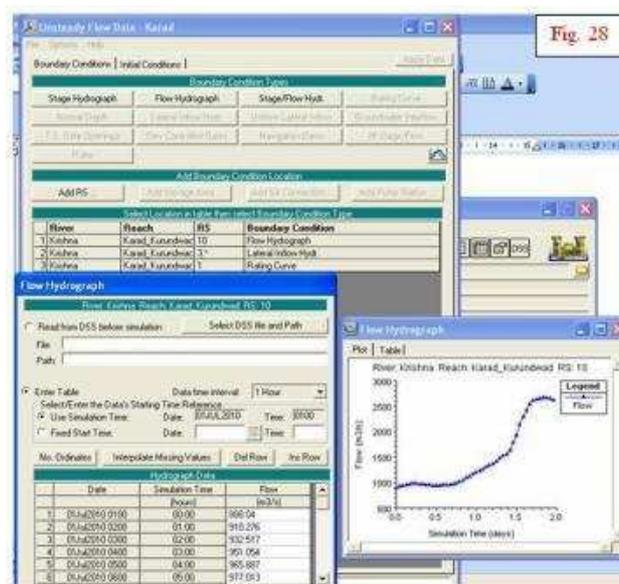
Similarly, enter the values for Kurundwad after clicking on the rating curve button and see the curve by pressing the Plot data option.

Fig. 29
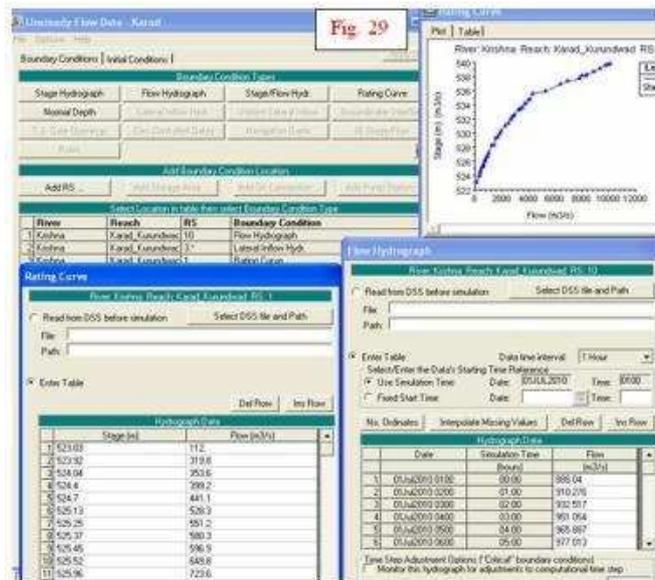
If you want the rating curve at known site, then you can enter it by clicking on **Options>Observed (measured) data> rating curves (gages) (Fig. 30).**

To leave the flow data editor and return to the HEC-RAS project window, choose **File/Exit Flow Data Editor.**

Fig. 30
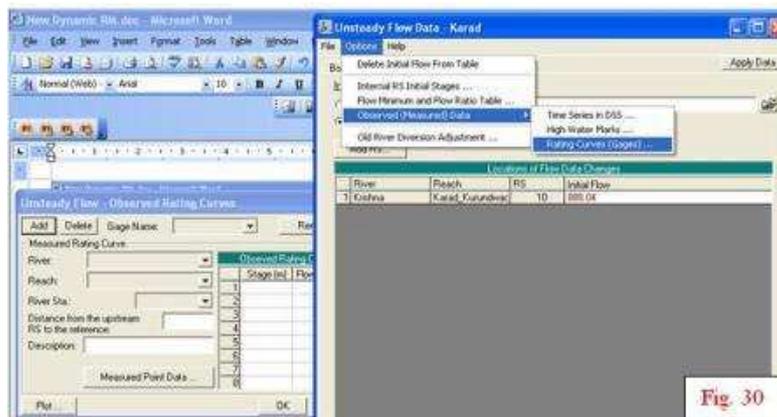
Executing the Model

With the geometry and flow files established, select **Run/ Unsteady Flow Analysis** from the project window. But before running the model, one final step is required: definition of a plan. The plan specifies the geometry and flow files to be used in the simulation. To define a plan, select **File/New Plan** andYou'll be subsequently asked to provide a plan title and a 12 character identifier as depicted in the Fig. 31.
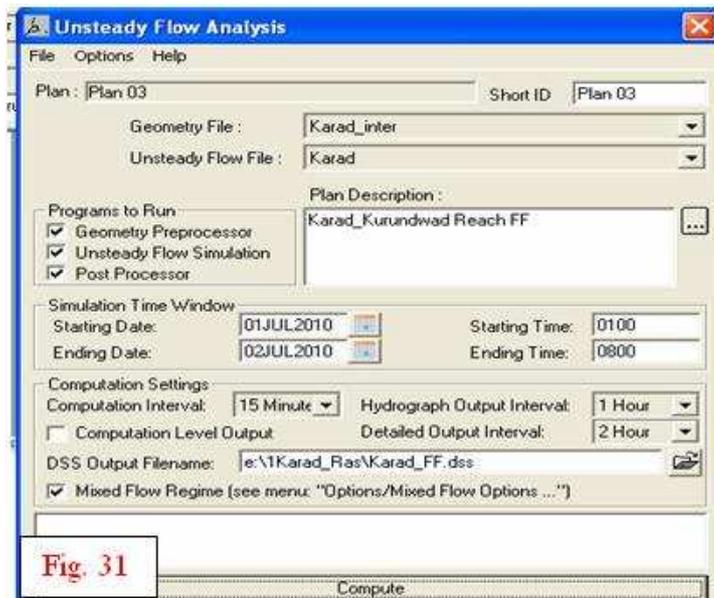
Fig. 31

To execute the model, ensure that the flow model parameters set properly, and click **compute** button
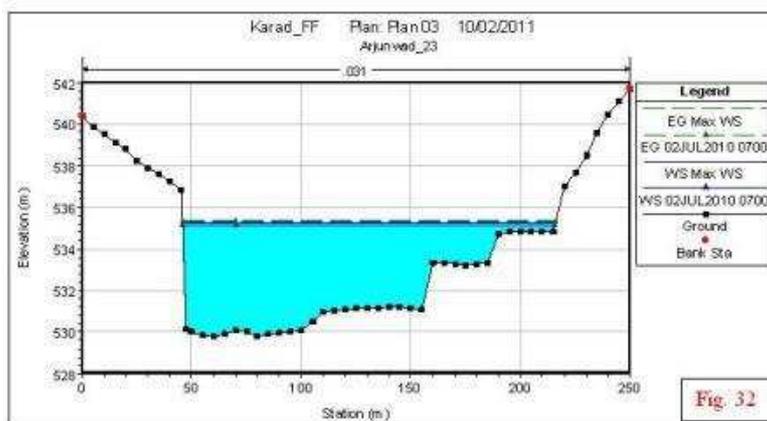

Fig. 32

However, the output view (Fig. 33) also shows the elevation of the total energy head line (shown in the legend as "EG 02Jul2010 0700"), the water surface ("WS 02Jul2010 0700"). As with the cross-section geometry editor, you can use the [↓] [↑] button to scroll to other cross-sections. For a profile of the entire reach, select **View/Water Surface Profiles** from the project window.
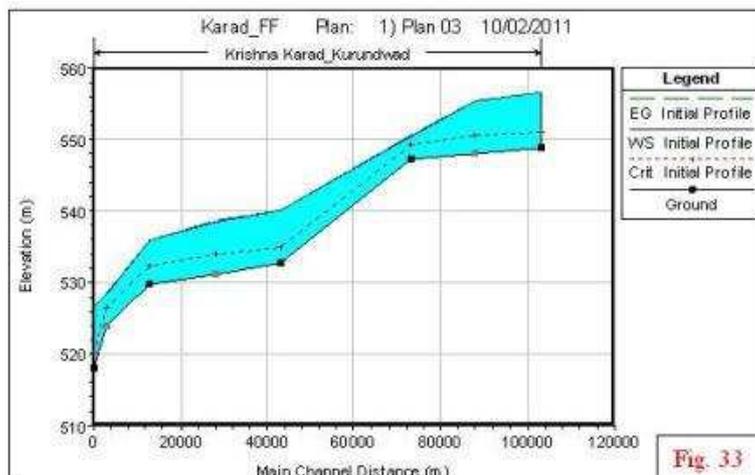

Fig. 33

Using the **Options/Zoom In** menu option, you can focus on a particular stretch of reach to see how the water surface relates to structures in the channel such as bridges. Other available options for graphical display of output data include plots of velocity distribution **(View/Cross-Sections/Options/Velocity Distribution)** and pseudo 3D plots **(View/X-Y-Z Perspective Plots)**. Spend a little time playing around with some of the display options.

For hydraulic design, it is often useful to know the calculated values of various hydraulic parameters. HEC-RAS offers numerous options for tabular output data display. From project window, choose **View/ Detailed Output Table**. It is to note that the simulated value is *1674.38* m^3/s against the actual observed value of *1674.34* m^3/s at Kurundwad (Fig. 34) (X-section_1) (0800 hrs on 2nd July, 2006, i.e, end of simulation period). The simulated water level is *529.37*m against observed value of *529.145* m.

Fig. 34

Additional tabular output data can be accessed from the invoking **View/Profile Output Table** (Fig.35) from the main project window. Numerous formats and data types can be viewed by selecting different tables from the **Std. Tables** menu.

Fig. 35

The resulting table includes a number of hydraulic parameters, including water surface elevation, head losses, and cross-sectional area. At the bottom of the window, error and notes (if any) resulting from the steady flow computations are shown. As you scroll through the cross-sections, take a look at some of the error messages. For this model, some X-sections have been added as the warning it showed to interpolate cross section.

Stability of the model

The vital factors which affect the model stability and numerical accuracy are:

1. Cross Section Spacing
2. Computation time step

3. Theta weighting factor
4. Solution iterations & tolerances

Cross sections should be placed at representative locations to describe the changes in geometry. Additional cross sections should be added at locations where changes occur in discharge, slope, velocity, and roughness. Cross sections must also be added at levees, bridges, culverts, and other structures. Bed slope plays an important role in cross section spacing. Steeper slopes require more cross sections. Streams flowing at high velocities may require cross sections on the order of 30m or less. Larger uniform rivers with flat slopes may only require cross sections on the order of 300m or more.

Theta is a weighting applied to the finite difference approximations when solving the unsteady flow equations. Theoretically Theta can vary from 0.5 to 1.0. However a practical limit is from 0.6 to 1.0. Theta of 1.0 provides the most stability. Theta of 0.6 provides the most accuracy. The default in RAS is 1.0. Once the model is developed, reduce theta towards 0.6, as long as the model stays stable. The stability problems are due to:

1. Too large time step.
2. Not enough X-sections
3. Model goes to critical depth - RAS is limited to subcritical flow for unsteady flow simulations. Bad d/s boundary condition (i.e. rating curve or slope for normal depth). Bad X- section properties, commonly caused by: levee options, ineffective flow areas, Manning's n values, etc.

If this happens, note the simulation time when the program either blew up or first started to oscillate. Turn on the "*Detailed Output for Debugging*" option and re-run the program. View the text file that contains the detailed log output of the computations. Locate the simulation output at the simulation time when the solution first started to go bad. Find the river station locations that did not meet the solution tolerances. Then check the data in this general area.

Calibration of the Model

The model can be calibrated by changing the hydraulic parameters. Open *Unsteady flowanalysis>Options>Computation options and tolerances*. The *theta* (implicit weighing factor) value as shown in figure can be changed from 0.6 to 1 and repeated simulations can be run with changed iterations and Changed Manning's *N* to validate the actual results (Fig. 36). Some Manning's *N* values have been cited from the literature (Fig. 37), but the actual values are to be calibrated to have the model match with the real conditions. (Page 81 of Reference Manual). You can also set the initial conditions during simulation and write detailed log output for debugging by clicking options> Output Options as shown in *Figure 19*. You can check data before execution by clicking **Option**>Check data before execution.

The present simulation yielded very fitting results as regards discharge, but the water level difference remained at 22cms (529.37m simulated against 529.15m obs). The simulated matching results may be because of the single reach simulation, recent observed X-sections and no other streams joining the reach except at Sangam. Further, this might have accrued due to the exact observed data input at the X-section points.
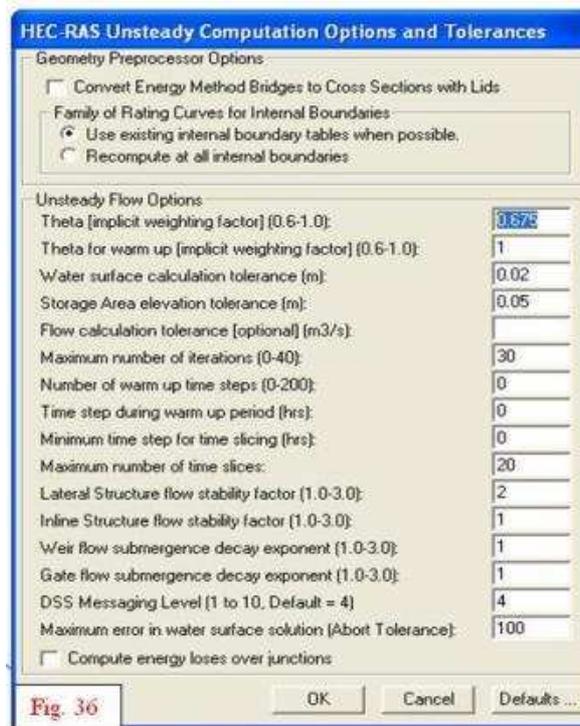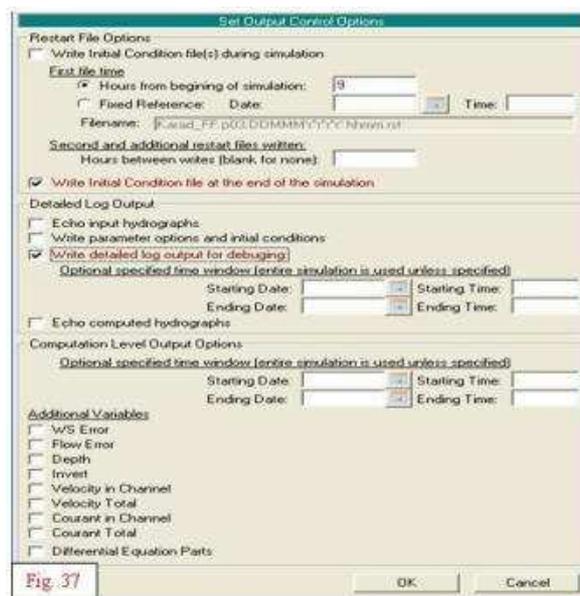
Fig. 36

HEC-RAS Unsteady Computation Options and Tolerances

Geometry Preprocessor Options

☐ Convert Energy Method Bridges to Cross Sections with Lids

Family of Rating Curves for Internal Boundaries
◉ Use existing internal boundary tables when possible.
○ Recompute at all internal boundaries

Unsteady Flow Options

Theta [implicit weighting factor] (0.6-1.0):	0.675
Theta for warm up [implicit weighting factor] (0.6-1.0):	1
Water surface calculation tolerance (m):	0.02
Storage Area elevation tolerance (m):	0.05
Flow calculation tolerance [optional] (m3/s):	
Maximum number of iterations (0-40):	30
Number of warm up time steps (0-200):	0
Time step during warm up period (hrs):	0
Minimum time step for time slicing (hrs):	0
Maximum number of time slices:	20
Lateral Structure flow stability factor (1.0-3.0):	2
Inline Structure flow stability factor (1.0-3.0):	1
Weir flow submergence decay exponent (1.0-3.0):	1
Gate flow submergence decay exponent (1.0-3.0):	1
DSS Messaging Level (1 to 10, Default = 4)	4
Maximum error in water surface solution (Abort Tolerance):	100

☐ Compute energy loses over junctions

[OK] [Cancel] [Defaults ...]

Fig. 37

Set Output Control Options

Restart File Options
☐ Write Initial Condition file(s) during simulation
First file time
◉ Hours from begining of simulation: 9
○ Fixed Reference: Date: [ ] Time: [ ]
Filename: R_and_FF.p01.DDMMMYYYY hhmm.rst
Second and additional restart files written:
Hours between writes (blank for none): [ ]
☑ Write Initial Condition file at the end of the simulation

Detailed Log Output
☐ Echo input hydrographs
☐ Write parameter options and initial conditions
☑ Write detailed log output for debugging
Optional specified time window (entire simulation is used unless specified)
Starting Date: [ ] Starting Time: [ ]
Ending Date: [ ] Ending Time: [ ]
☐ Echo computed hydrographs

Computation Level Output Options
Optional specified time window (entire simulation is used unless specified)
Starting Date: [ ] Starting Time: [ ]
Ending Date: [ ] Ending Time: [ ]

Additional Variables
☐ WS Error
☐ Flow Error
☐ Depth
☐ Invert
☐ Velocity in Channel
☐ Velocity Total
☐ Courant in Channel
☐ Courant Total
☐ Differential Equation Parts

[OK] [Cancel]

e) Data driven hydrologic models

Sometimes,it is argued that deterministic, reductionist models are inappropriate for real-time forecasting because of the inherent uncertainty that characterizes river catchment dynamics and the problems of model overparameterization. The advantages of alternative, efficiently parameterized data-based mechanistic models, identified and estimated using statistical methods, are discussed.

Neuromorphic modelling techniques are now well established methods for describing physical processes occurring in the aquatic environment. The development in information technology over the last decade has presented opportunities of extended computational ability together with improved data manipulation, storage and retrieval. As a result, the Neuromorphic models are now being used more extensively in the management, design and operation of water based assets. The reason behind this is that in many areas of applications pertaining to the complex

flow systems, the demands on computing time are of such a magnitude, which is far from acceptable. An elementary brief part of ANN has been added here in the distance learning course for easy insight, though there are advanced architectures like recurrent neural network (RNN), radial basis function (RBF),self-organizing map (SOM) and othersused in flood forecasting

i) Artificial Neural Networks

An artificial neural network is nothing but a collection of interconnected processing elements (PEs). The connection strengths, also called the network weights, can be adapted such that the network's output matches a desired response.
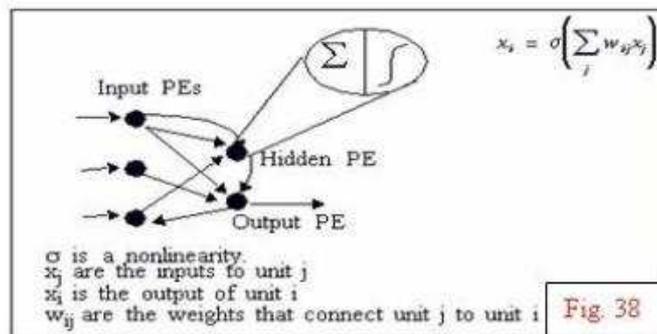
Typical architecture of single hidden layer neural network

Fig. 38depicts a typical multilayer perceptron, which has been used in this research, which resembles a black box model, where a set of a data like x1, x2, x3…xn are fed directly to the network through the input layer, and subsequently produces expected result y in the output layer. The output is determined by the architecture of the network.

In multi-layered perceptron, hidden layer means a third layer of processing elements or units in between the input and output layers that increases computational power. In principle, the hidden layer can be more than one layer. In practice the number of neurons in this layer is evaluated by trial and error. Hornik et al. (1989) proved that a single hidden layer containing a sufficient number of neurons can be used to approximate any measurable functional relationship between the input data and the output variable to any desired accuracy. In addition, De Villars and Barnard (1993) showed that an ANN comprising of two hidden layers tends to be less accurate than its single hidden layer counterpart. In this, a single hidden layer ANN has been used.

Each input xi (i =1,…,n) is attenuated by a factor wij, more commonly called a weight of the network, which is associated with the connection linking input xi to hidden neuron j (j = 1, ….,k), where, k is the number of neurons in the single hidden layer. The weighted sum of the incoming signals entering a neuron is fed via an activation function, which is non-linear, producing a value that in turn, act as an input signal sent to the output layer. This is repeated for the output weights. The following expression gives the output value of the network.

$$y = \psi \left[\sum_{j=1i}^{N} \varphi \left\{ \sum_{i=1}^{N} x_i w_{ij} \right\} a_j \right] \tag{1}$$

Where, the sigmoidal activation function ψ is given by

$$\psi(x) = \frac{1}{1+e^{-\alpha x}} \qquad (2)$$

This function given at eq. 2 is a continuous function that varies gradually between asymptotic values 0 and 1 or -1 and +1. Where, α is the slope parameter, which adjusts the abruptness of the function as it changes between the two asymptotic values. Sigmoid functions are differentiable, which is an important feature of neural network theory.

To obtain the best approximations, it is needed to determine the optimum set of weights w_{ij} and a_j that will yield the least mean square value of the desired response. Thus the following performance criterion needs to be satisfied.

$$\min_{a, w_{ij}} \frac{1}{2} E\left[(y - y_D)^2\right] \qquad (3)$$

The configuration chosen for the ANN models are shown in Fig. 39, where the bias inputs have the effect of lowering or increasing the net result of the activation function. The activation used is the sigmoidal function, which has the purpose of limiting the permissible amplitude range of the output values to some finite value.

Fig 39
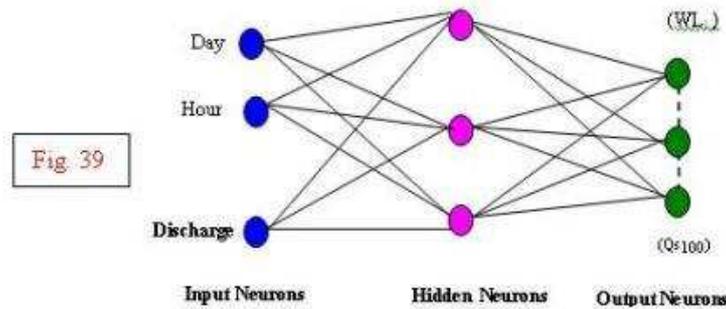
Normalization of the data

It is mentioned that the sigmoidal function can take the values ranging in the (0, 1) domain, a normalisation of the values of the input variables are done. The standard procedure in neural network theory gives the normalisation equation used for this purpose.

$$SF = (SR_{max} - SR_{min}) / (X_{max} - X_{min}) \qquad (4)$$
$$Xp = SR_{min} + (X - X_{min}) * SF \qquad (5)$$

Where: X - actual value of a numeric column, X_{min} - minimum actual value of the column, X_{max} - maximum actual value of the column, SR_{min} - lower scaling range limit , SR_{max} - upper scaling range limit and SF - scaling factor, Xp-pre-processed value.

Interpretation of results

Numerous goodness of fit statistical criteria are proposed in the literature for evaluating hydrological modelling results. Here, only two of these are considered in this study, namely RMSE, and Nash - Sutcliffe coefficient (1970). RMSE can take any positive value but the closer it is to zero, the better the model performs. When the Nash value is between 0 and 1, the forecast

model does better than simply forecasting using y_o. The closer the Nash index is to one, the better. These performance criteria are used as basis of comparison to select the best model.

$$RMSE = \sqrt{\frac{1}{N}\sum_{i}^{N}(Y_o - Y_p)^2} \qquad (6)$$

Nash-Sutcliffe coefficient is defined by

$$Nash = 1 - \frac{\sum_{i}^{N}(Y_o - Y_p)^2}{\sum_{i}^{N}(Y_o - \bar{Y_o})^2} \qquad (7)$$

Where, Y_o = Observed daily gauge of the catchment on day i;
Y_p = Predicted daily gauge of the catchment on day i;

As could be seen from Fig 40, the modelproved their capability in predicting the data, especially the stage data, which shows a high correlation of the observed and predicted data.

Fig. 40
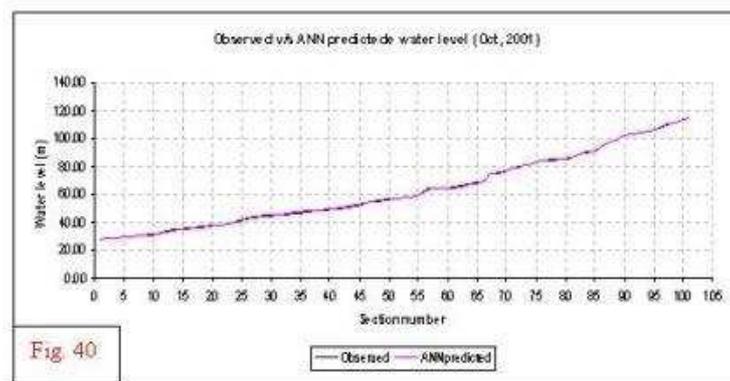
ii) Fuzzy expert system design for flood forecasting

Linguistic terms are chosen to describe the input variable stage and the results. Further refinement of the models could not be achieved by adding extra membership functions. Gaussian membership functions (the function is generally suited for Indian rivers) can be used. Applying a similar method of data classification, membership functions are determined for the output variable discharge.

Rule definition

Some years of average hourlystage data and expert knowledge are used to create a rule base for the fuzzy logic model. Rules are defined for both the high and low extreme conditions, with regard to actual occurrences, because of the physical nature of the relationships. Depending on number of membership functions for each input variable; the minimum rule base is created. For each data point, all rules are evaluated.

Fuzzy model construction

The platform selected for the fuzzy logic expert system is MATLAB and MATLAB'S Fuzzy Logic Toolbox. The variables are combined into rules using the concept of 'AND'. The fuzzy operator 'minimum' is applied as the 'AND' function to combine the variables. No weightings are applied, which means no rule is emphasized as more important in respect to estimating the discharge. Implication is performed with the minimum function, and aggregation is performed with the maximum function. The centre of gravity method is applied as a means of defuzzification of the output membership functions to determine a crisp set. Based on this structure a baseline model fuzzy logic expert system for stage-discharge relationship is constructed for the G&D stations. Alternate functions for the expert system are investigated through sensitivity analysis.

Sensitivity analysis

A sensitivity analysis is performed for the fuzzy logic operator AND, and for methods of implication, aggregation and defuzzification. The results of changing a single operator or method while the rest of the model is held constant are compared with the results from the baseline model. The results are evaluated on the basis of correct linguistic matches. Based on this sensitivity analysis, the AND operator 'minimum' and the implication method 'minimum' are found to perform better than the product method. The fuzzy logic and ANN models are evaluated based on their ability to predict the discharge.

### iii) ANFIS (Adaptive Neuro-Fuzzy Inference System) models

The hybrid system of learning has been attempted at combining ANN and fuzzy logic for developing the stage-discharge relationship to achieve a faster rate of convergence by controlling the learning rate parameter with fuzzy rules. The objective is to get a minimizer, which has a low computing cost and a large convergence domain. This learning ability is achieved by presenting a training set of different examples to the network and using learning algorithm, which changes the weights in such a way that the network reproduces a correct output with the correct input values. The main dissimilarity between fuzzy logic system (FLS) and neural network is that FLS uses heuristic knowledge to form rules and tunes these rules using sample data, whereas NN forms "rules" based entirely on data. Sugeno type ANFIS can be used. Gaussian membership functions can be used with rule bases, because of their low computational requirements and as it has the important properties of smooth mapping, universal approximation, normal distributions can be approximated well by this type of functions.Learning rate control by fuzzy logic has been depicted at fig 41. (FLC - fuzzy logic controller, MLP - multilayer perceptron

Fig. 41
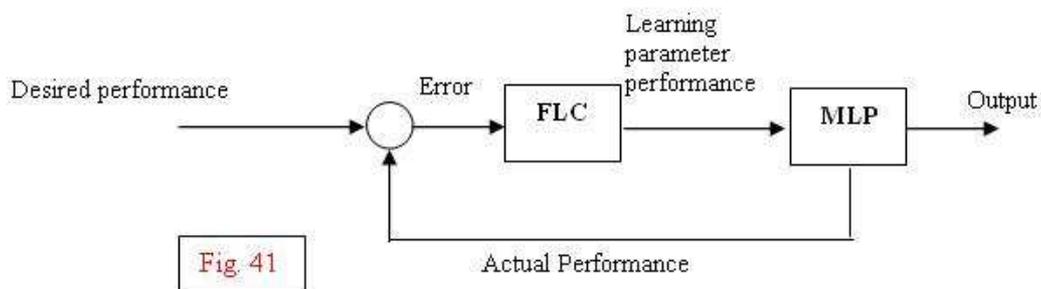
Validation and comparison of results

The ANN, fuzzy and neuro-fuzzy models thus developed is validated and compared with the observed data points and the statistical measures of goodness-of-fit of the neuromorphic models. Numerous goodness of fit statistical criteria are proposed in the literature for

evaluating hydrological modelling results. Goodness of fit can be tested from standard statistics literature as has been shown in the aforesaid ANN paragraph.Fig 42 shows the validation and comparison of models with observed data.

Fig 42
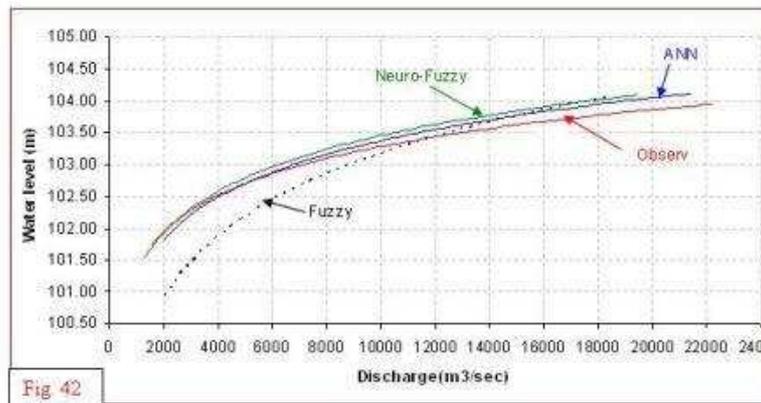
Conclusions

As could be seen in preceding paragraphs, advance warning about the incoming flood peak and its probable time of occurrence can be achieved by several models. However, selection of a particular method or model, and its accuracy for a given site is largely governed by threefactors - data availability; forecaster's knowledge of, and his experience with the basin; and forecaster's familiarity with software to be used in the forecast process.

The illustrated texts mentioned in this module are just the trail of a beginning and more of the subject and in-depth precision knowledge base, the readers are suggested to refer to advanced literature layouts.

REFERENCES

1. Flood forecasting Manual, (1989), Central Water Commission, New Delhi
2. Raghunath, H M., (2006), Book on Hydrology principles analysis and design, New Age International
3. Technical references Manual, (2004), HEC-HMS, USACE
4. Technical references Manual, (2004), HEC-RAS, USACE
5. Wheater, H. S., Jakeman, A. J. &Beven. K. J. (1993) Progress and directions in rainfall-run-of modelling. In Modelling change in environmental systems (ed. A. J. Jakeman, M. B. Beck &M. J. McAleer), pp. 101-132, Wiley.

MORE LITERATURES OM NEURAL NETWORK AND FUZZY LOGIC CAN BE FOUND AT THE FOLLOWOING REFERENCES

1. *Dastorani, M.T. and Wright., N.G., 2002, Artificial neural network based real-time river flow prediction, Hydroinformatics, Proc. of 5th Int. Conf., Cardiff, UK.*
2. *Haykin, Simon, (1999) Neural Networks: A Comprehensive Foundation, Prentice Hall.*
3. *Karunanithi, N., Genny, W.J., and Whitley, D 1994. Neural networks for river flow prediction., journal of Computational Civil eng, 8(2),201-220.*
4. *McCulloch, W.S., Pitts, W. 1943. A logical calculus of the ideas imminent in nervous activity.Bull.Math.Biosphys.5, 115- 133.*
5. *Nash, J.E., and Sutcliffe, J.V., 1970. River flow forecasting through conceptual models, part-1- A discussion of principles. Journal of Hydrology, 10(3), 282-290.*

6. *Rajsekharan, S. and G. A Vijayalakshmi Pai, 1996. Neural networks, fuzzy logic and genetic algorithm, Prentice Hall of India, pp-34-86.*

7. *Rumelhart, D.E, Hinton, G.E, and Williams, R.J, 1986. "Learning internal representation by back propagation errors.", Nature 323, 533-536.*

8. *Sankhua, R. N, 2006, Application of artificial neural network for daily river stage forecast in the Brahmaputra River, Water and Energy International Journal, CBIP, New Delhi, Vol. 63, No 3, pp.55-62, Jul.-Sep.*

9. *Sankhua, R. N, 2006, Monitoring of Morphological Changes of Indian Rivers through ANN Based Spatio-Temporal Model- An approach, National Seminar on silting of rivers in India, February, 12-13, New Delhi.*

10. *Sankhua, R. N, 2006, Spatio-Temporal Modeling of Hydrological Variability for the river Brahmaputra using Artificial Neural Network, proc. International Symposium on Role of Water Sciences in Transboundary River Basin Management, Ubon Ratchathani, Thailand, March 10-12,pp-25-31*

11. *Zhu, M.L, and Fujita, M (1994), "Comparisons between fuzzy reasoning and neural networks methods to forecast runoff discharge." J. of Hydroscience, Hydraulic eng., 12(2), 131-141.*

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ACKNOWLEDGEMENT