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Research paper

Extended Muskingum method for flood routing

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Abstract

Routing of floods is essential to control the flood flow at the flood control station such that it is within the specified safe limit. In this paper, the applicability of the extended Muskingum method is examined for routing of floods for a case study of Hirakud reservoir, Mahanadi river basin, India. The inflows to the flood control station are of two types – one controllable which comprises of reservoir releases for power and spill and the other is uncontrollable which comprises of inflow from lower tributaries and intermediate catchment between the reservoir and the flood control station. Muskingum model is improved to incorporate multiple sources of inflows and single outflow to route the flood in the reach. Instead of time lag and prismoidal flow parameters, suitable coefficients for various types of inflows were derived using Linear Programming. Presently, the decisions about operation of gates of Hirakud dam are being taken once in 12 h during floods. However, four time intervals of 24, 18, 12 and 6 h are examined to test the sensitivity of the routing time interval on the computed flood flow at the flood control station. It is observed that mean relative error decreases with decrease in routing interval both for calibration and testing phase. It is concluded that the extended Muskingum method can be explored for similar reservoir configurations such as Hirakud reservoir with suitable modifications.

Keywords: Extended Muskingum; Linear programming; Mahanadi river basin

1. Introduction

Flood routing is an important aspect in reservoir operation for flood control. This requires suitable flood routing relationship explicitly in the formulation of the policy. The releases from reservoir during floods should be so controlled that the total flow at a downstream station is within the safe limit. The downstream station at which the specified maximum flow is to be restricted is herein after referred as flood control station. The factors causing floods at flood control station are the release for power and spill from reservoir, measured inflow to the river from tributaries between the reservoir and the flood control station and unmeasured lateral flow from the intermediate catchment. The last two factors, viz., measured inflow from intermediate catchment and unmeasured lateral flows are not under human control. Only the release from reservoir can be controlled considering the safety and other criteria of the reservoir. There would be some time lag in terms of hours or even days for the release of water from the reservoir to reach the flood control station. It is necessary to know the effect of the released quantity from the reservoir, at the flood control station at the time of taking decision about the reservoir releases. The flood routing equation is specifically to be developed for this purpose, and is considered as an important element in reservoir operation.

Flood routing is necessary for most of the reservoirs in general and very much essential for Hirakud reservoir, Mahanadi river basin, India in specific. In Mahanadi river basin, three reservoirs were originally proposed for full development of the basin (Patri, 1993). But only one was constructed in the year 1956 at Hirakud mainly to mitigate floods. As no additional reservoirs are constructed till date, the existing reservoir is used both for flood control and conservation purposes. Various conservation purposes of reservoir include irrigation, hydropower,

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drinking water supply, low-flow augmentation etc. Average monsoon inflow into the reservoir, observed in the history, is about six times more than the live storage of the reservoir. So flood control during high flood is extremely difficult and sometimes is not possible. To design a suitable flood control measure, it is first necessary to optimize the resultant flow at Naraz by suitably routing the flood in the reach between the reservoir and Naraz. Efforts are continuing to extract maximum benefits from this single reservoir of the basin. It is observed that the operating policy in the form of rule curves for the reservoir was changed six times from 1958 till to date. These changes are due to the climatic and hydrologic changes in the basin and changes in objectives of the reservoir. In this regard, sustainable and systematic flood control policy for Hirakud reservoir is to be evolved in a scientific way (Baliarsingh, 2000). The objectives of the present study are as follows:

1. Exploring the applicability of extended Muskingum method (with multiple inflows – single outflow) as flood routing model which has the ability to account for the lateral inflows.
2. Exploring the applicability of Linear Programming for determining Muskingum parameters C_0, C_1, and C_2 for various types of inflows.
3. Development of relationship between discharge and other parameters.
4. Validation of above methodologies to the case study of Hirakud reservoir, Mahanadi river basin, India.

The paper is organized as follows: literature review, description of case study, data for flood routing, results and discussion and conclusions.

2. Literature review

The methods of flood routing are broadly classified as empirical, hydraulic, and hydrological (Fread, 1981). A number of soft computing related techniques were used for flood forecasting in addition to Muskingum method. A brief literature review is presented to provide an overview.

Preliminary concepts and numerous applications of Artificial Neural Networks (ANN) to hydrology are available (ASCE, 2000a,b; Fernando and Jayawardena, 1998). Cheng and Chau (2001), Cheng and Chau (2002) proposed fuzzy iteration methodology and three-person multi-objective conflict decision model respectively for reservoir flood control operation for a case study of Fengman Reservoir, China. Chau et al. (2005) employed the Genetic Algorithm based Artificial Neural Network (ANN-GA) and the Adaptive Network based Fuzzy Inference System (ANFIS), for flood forecasting in a reach of the Yangtze River in China. Similar studies are reported by Cheng et al. (2002, 2008a,b).

Muskingum method is a hydrological flood routing technique (Chow et al., 1988) which was modified by many researchers. In the two parameter Muskingum method, there are number of ways for finding the two parameters, K (travel time) and x (weighing factor for prism and wedge storage of routing reach). These methods were discussed in detail by Singh and McCann (1980) and applied to a set of data to assess their relative efficacy. Gill (1978) proposed segmented curve method, in which least square method was used to find out the parameters of nonlinear form of Muskingum method. Stephenson (1979) demonstrated the way to calculate directly the coefficients of Muskingum method, C_0, C_1, and C_2 using Linear Programming instead of calculating the parameters, K and x.

O'Donnell (1985) considered the lateral flow factor in Muskingum two parameter model of single input single output (si-so) nature, which was converted into a three parameter model. The parameters are K, x, α (α shows the fraction of lateral flow in comparison with inflow to the reach). The least square technique is used to find out these parameters in the routing reach automatically. Khan (1993) extended the si-so flood routing model to include lateral flow to form a multi input single output (mi-so) model with lateral flow.

Tung (1985) developed state variable modeling technique for solving the nonlinear form of Muskingum method. The parameters of the model were found out by four methods of curve fitting. Yoon and Padmanabhan (1993) developed a software, MUPERS, where both linear and nonlinear relationships were dealt with. Kshirsagar et al. (1995) found parameters by a constrained, nonlinear (successive quadratic) programming. In this work, the Muskingum equation was used for routing the upstream hydrograph and the intermediate ungauged lateral inflow. The lateral inflow was calculated by an impulse response function approach. Mohan (1997) used genetic algorithm for parameter estimation of nonlinear Muskingum method and compared its performance with the approach by Yoon and Padmanabhan (1993).

Samani and Jebelifard (2003) applied multilinear Muskingum method for hydrologic routing through circular conduits. Das (2004) developed a methodology for parameter estimation for the Muskingum model of stream flow routing. Al-Humond and Esen (2006) presented two approximate methods for estimating Muskingum flood routing parameters. Geem (2006) introduced the Broyden–Fletcher–Goldfarb–Shanno (BFGS) technique, which searches the solution area based on gradients for estimation of Muskingum parameters.

Choudhury (2007) proposed a multiple inflows Muskingum model. This model appropriately extended the Muskingum philosophy to multiple inflows routing, expressed in a single inflow single outflow form. The model performance is compared with the nonlinear kinematic wave model. He applied the model to the flood events in Narmada Basin, India. Das (2007) developed a chance constrained optimization based model, for Muskingum model parameter estimation. Das (2009) developed a methodology for Muskingum model's parameter estimation for reverse stream flow routing for which a fresh calibration was found necessary. Chu (2009) applied Fuzzy Inference System (FIS) and Muskingum model in flood routing where rules of FIS were incorporated with the Muskingum formula.

As mentioned in the objectives of the present study, extended Muskingum method with multiple inflows – single outflow as developed by Khan (1993) is employed for a case study of Hirakud reservoir, Mahanadi river basin, India. Linear

Programming is adopted for arriving at the values of parameters C_0, C_1, and C_2 instead of time lag and prismoidal flow parameters. Brief description of case study is provided in the next section.

3. Case study

The project considered for the present study, Hirakud reservoir, is situated in Mahanadi basin, India. The Mahanadi basin lies between 80° 30′ and 86° 50′ East longitudes and 19° 20′ and 23° 35′ North latitudes. Area of this basin is 141,600 sq km and is broadly divisible into three distinct zones, the upper plateau, the central hill part flanked by Eastern ghats, and the delta area. Hirakud dam across Mahanadi river is located in the second zone.

Mahanadi river originates in Raipur district of Madhya Pradesh and runs for a length of 851 km and joins the Bay of Bengal. After a run of 450 km from its starting point, the Hirakud dam was built across the river. Downstream (d/s) of the dam, the river gets water mainly from two tributaries, Ong and Tel, in addition to free catchment. The river flows down to Naraj, the head of delta and finally joins the Bay of Bengal. The catchment area up to Naraj is 132,200 sq km. On the d/s of Naraj, the river divides into several branches, namely, Birupa, Chitrotpala, Devi, Kushabhadra, Bhargabi, Daya etc. and runs 80 km before discharging into Bay of Bengal. The schematic diagram of Hirakud project is shown in Fig. 1.

The multipurpose Hirakud reservoir is utilized mainly for three purposes, flood control, irrigation, and hydropower production in that order of priorities. Hirakud dam is expected

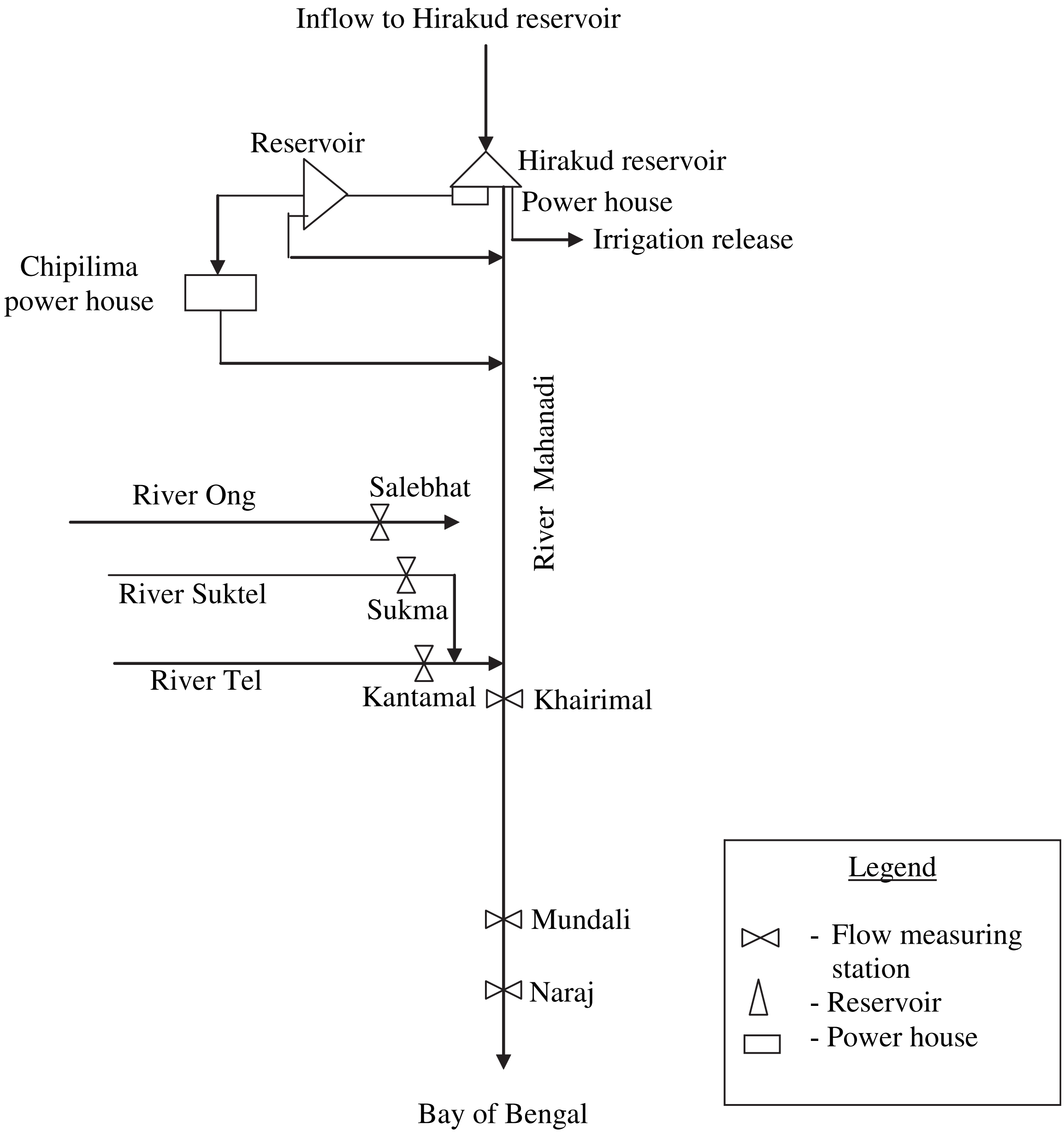

Fig. 1. Schematic diagram of Hirakud project.

to control flood in coastal delta area by limiting the flow at Naraj to be within 25,500 cumecs. There are three head regulators, which can draw 128.8 cumecs for irrigation purposes. Areas of 160,000 and 110,000 ha are irrigable under the reservoir during Kharif (June to October) and Rabi (November to February) seasons respectively. The total installed hydropower capacity of the project is 308 MW, out of which 236 MW can be produced from seven units of Hirakud hydropower station, and 72 MW from three units of Chipilima hydropower station, located further d/s of Hirakud dam. The water, used for power generation at Hirakud, flows from Hirakud hydropower station to Chipilima hydropower station through a power channel, 22.4 km long. After generating power at Chipilima, the water flows back into the river.

For this reservoir, flood control is the first priority. There is no other flood controlling structure downstream of Hirakud reservoir. During monsoon season, the coastal delta part, between Naraj and Bay of Bengal, is severely affected by floods. These flood flows comprise of releases from Hirakud reservoir for power generation and spill and runoff from the downstream catchment. Naraj where the flow of Mahanadi river is measured is situated at the head of the delta area. The flow of Mahanadi river at Naraj is considered by the Hirakud authority as the indicator of occurrence of flood in the coastal delta area. As Hirakud reservoir is on the upstream side of the delta area of the basin, it plays an important role in alleviating the severity of the flood in this area by suitable regulation of releases from the reservoir. Before making the operational decisions, it is necessary to optimize the resultant flow at Naraz by suitably routing the flood in the reach between the reservoir and Naraz. Next section discusses the data requirements and analysis for flood routing.

4. Data for flood routing

4.1. Inflows into the reach

The 320 km long reach of the river between Hirakud reservoir and Naraj is treated in this study for flood routing. Flood data from 1992 to 1995 are considered for the purpose. There is one flood each in 1992, 1993, and 1995 and six floods in 1994. The characteristics of the floods are shown in Table 1 (Patri, 1993).

The outflow from the reservoir is the combination of the spill from the reservoir and the release for power. This outflow is measured at the dam. Water released for power joins the river downstream of the reservoir after generating power at two locations as shown in Fig. 1. Both the points are 25 km apart and for simplicity, these two flows are combined into one value. Two tributaries, Ong and Tel, join Mahanadi river on the downstream of Hirakud reservoir. Flow in tributary Ong is measured at Salebhat. The flow in tributary Tel is measured at Kantamal. In the case of Suktel, the tributary of Tel, flow is measured at Sukma. Suktel joins the tributary Tel d/s of Kantamal. Location of these tributaries and measuring stations are shown in Fig. 1. The confluence points of these two tributaries with Mahanadi river are quite close, compared to the length of river reach considered for flood routing purpose. The flows at all these three stations are therefore considered together, as if these flows are combined together before joining the Mahanadi river. This combined flow at Khairimal is henceforth termed as d/s catchment contribution.

4.2. Lateral flow

The data is tested for the amount of unmeasured lateral flow in Hirakud-Naraj routing reach. Duration of each flood event is so chosen that the river level at Naraj is same at beginning and end of the event. As there is no other supporting data, it is assumed that the slope of water surface in this reach is same at the beginning and at the end of each flood event. Floods numbered 4 and 5 occurred consecutively without any time gap, and so did floods numbered 7 and 8. So these two pairs of floods are combined together to form one flood event each only for the calculation of lateral flow and are named as flood numbers 4 and 7 respectively. The volume of inflow into the reach is the combination of release for power and spill from Hirakud reservoir and d/s catchment contribution. The difference between inflow into the reach and outflow from the reach, i.e., flow at Naraj, constitutes the lateral flow into the

Table 1
Characteristics of floods under consideration for flood routing.

Flood No.	Duration of flood event	Influencing variable of flow at Naraj	Maximum flow from reservoir (10^3 cumecs)	Maximum d/s catchment contribution (10^3 cumecs)	Maximum flow at Naraj (10^3 cumecs)	Minimum flow at Naraj (10^3 cumecs)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	16/08/92−29/08/92	Both RPS[a] and DC[b]	15.52	14.11[c]	33.98[c]	08.01
2	14/08/93−27/08/93	Both RPS and DC	10.16	05.54	22.10	02.20
3	19/06/94−27/06/94	Only RPS	11.18	02.96	17.63	08.64
4	08/07/94−19/07/94	Only RPS	21.22[c]	08.04	29.04	03.11
5	19/07/94−28/07/94	Only RPS	17.99	02.52	22.93	08.12
6	30/07/94−10/08/94	Only RPS	11.68	09.74	14.52	07.03
7	17/08/94−26/08/94	Both RPS and DC	11.01	11.25	22.04	11.63
8	26/08/94−12/09/94	Both RPS and DC	16.74	13.54	30.64	12.84
9	18/07/95−30/07/95	Both RPS and DC	13.62	09.85	25.93	01.71

[a] RPS − Release for power and spill from Hirakud reservoir.
[b] DC - downstream catchment contribution.
[c] Maximum.

routing reach. These details are shown in Table 2. It is observed that the unmeasured lateral flow varies widely from one flood to other. This is not having any specific ratio to the volume of inflow or volume of outflow. The ratio of lateral flow to the volume of inflow varies from 1% to 72%, and that to the volume of outflow at Naraj varies from 1% to 42% (columns 6 and 7 of Table 2).

4.3. Data sets for calibration and testing

Data of each of the nine floods are plotted individually in Fig. 2 showing the influence of release for power and spill from reservoir and d/s catchment contribution on the flow at Naraj. It may be observed from the plots that the floods can be classified into two categories. In first category, both outflow from reservoir and d/s catchment contribution influence the pattern of flow at Naraj and in second category, only outflow from reservoir has the influence. Four floods, numbers 3 to 6 fall under second category and remaining five floods fall under first category. This characteristic is denoted in third column of Table 1. Two thirds of the data is used for calibration and the remaining is used for testing. The important factor in such classification of calibration and testing data sets is that the characteristics of testing data should be definitely present in the calibrating set. If the calibrating set is too generalized, performance during testing may not be that encouraging. This aspect is taken care of in the present study. Here, the quantity of flow at various time instances is taken as the characteristic of flood flow data. The floods of higher maximum discharge and lower minimum discharge are used for calibration, leaving the intermediate cases for testing (Baliarsingh, 2000).

Keeping these aspects in consideration, out of five floods of first category, flood numbers 1, 8, 9 are used for calibration and flood numbers 2, 7 for testing. Similarly, out of four floods in second category, flood numbers 3, 4 are used for calibration and flood numbers 5, 6 for testing. In total five floods, i.e., flood numbers 1, 3, 4, 8, and 9 are used for calibration and four floods, i.e., flood numbers 2, 5, 6, and 7 for testing.

Presently for Hirakud reservoir, the decisions of gate operation are being taken once in 12 h during flood. However, in the present study, it is proposed to evaluate the performance of gate operation once in 24 h, 18 h, 12 h, and 6 h to find out the effect of these variations on the predicted flow at Naraj. The flood routing equations with flow ordinates at the above time intervals are required to be used in the model for corresponding gate operation. Accordingly, the flood routing equations with these four routing periods by extended Muskingum method are analyzed in this study.

4.4. Performance measure

In this study, only one output viz., flood flow at the flood control station, Naraj, is predicted. So a single performance measure viz., Mean relative error is considered sufficient to evaluate the performance of the various models and is given by

$$E = \frac{1}{n}\sum_{l=1}^{n}\left|\frac{\mathrm{EY}_l - \mathrm{OY}_l}{\mathrm{OY}_l} \times 100\right| \tag{1}$$

where E is mean relative error in %; EY is the estimated outflow at Naraj; OY is the observed outflow at Naraj; n is the number of data points in the data set. Section 5 presents results and discussion.

5. Results and discussion

The extended Muskingum routing method (Khan, 1993) deals with multiple inflow – single outflow routing process. In the present study, the flood hydrograph at Naraj (Q) depends on two flood hydrographs on the upstream side of the reach; (i) hydrograph of release for power and spill from the reservoir (RPS) and (ii) hydrograph of d/s catchment contribution (DC). Downstream catchment contribution is the summation of flow at Kantamal, Sukma, and Salebhat. Downstream catchment contribution is assessed by deducting RPS from the total flow measured at Khairimal.

The relationship used in extended Muskingum method is expressed as

$$Q_{t+1} = f[\mathrm{RPS}_{t+1}, \mathrm{DC}_{t+1}, \mathrm{RPS}_t, \mathrm{DC}_t, Q_t] \tag{2}$$

where, Q_t is the flow at Naraj (outflow from the Hirakud-Naraj routing reach) at beginning of time period t; RPS_t is the release for power and spill from Hirakud reservoir at beginning of time period t; DC_t is the flow from Ong and Tel tributaries as assessed at Khairimal at beginning of time period t. t is the time period.

Table 2
Assessment of unmeasured lateral flow for each flood.

Flood No.	Duration of flood	Volume of outflow from the reach (10^9 cubic meter)	Volume of inflow into the reach (10^9 cubic meter)	Volume of lateral flow into the reach (10^9 cubic meter)	Lateral flow in percentage of inflow	Lateral flow in percentage of outflow
(1)	(2)	(3)	(4)	(5) = (3) − (4)	(6) = (5) × 100/(3)	(7) = (5) × 100/(4)
1	16/08/92–29/08/92	20.55	12.19	08.36	68	40
2	14/08/93–27/08/93	11.84	06.87	04.97	72	42
3	19/06/94–27/06/94	08.49	06.51	01.98	30	23
4	08/07/94–28/07/94	32.47	25.94	06.54	25	20
6	30/07/94–10/08/94	11.48	11.31	00.17	01	01
7	17/08/94–12/09/94	44.63	28.46	16.17	57	36
9	18/07/95–30/07/95	10.62	09.21	01.41	15	13

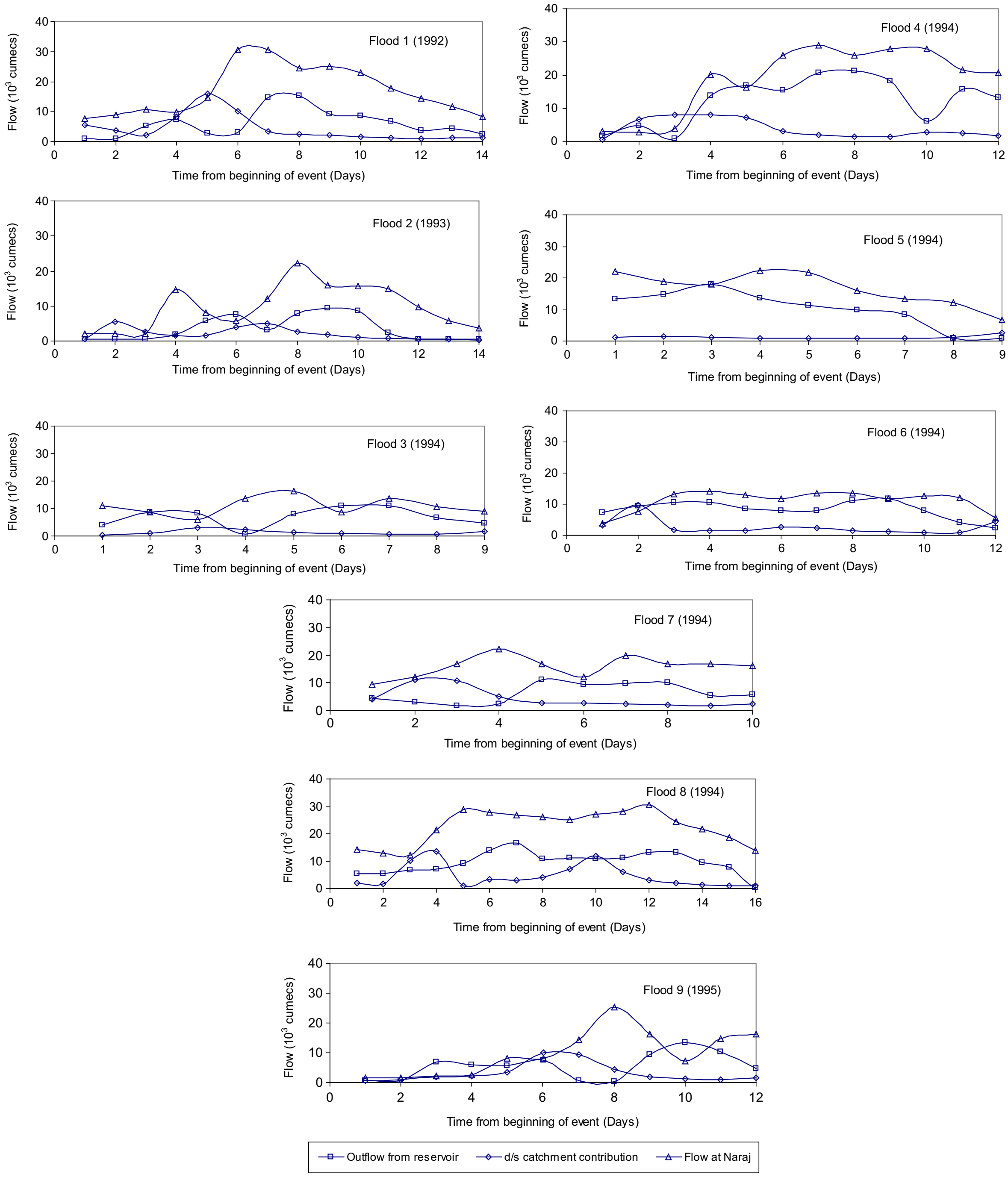

Fig. 2. Flow pattern of nine floods used in flood routing.

Khan (1993) developed the extended Muskingum method for multiple inflow – single outflow (mi-so) form of flood routing, based on the work of single inflow single outflow (si-so) form proposed by O'Donnell (1985) and it takes care of the ungauged lateral inflow also. The original equation of mi-so form of routing as given by Khan (1993) is

$$Q_{t+1} = C_0^1 i_t^1 + C_0^2 i_t^2 + \cdots + C_0^m i_t^m + C_1^1 i_{t+1}^1 + C_1^2 i_{t+1}^2 + \cdots + C_1^m i_{t+1}^m + C_2 Q_t \tag{3}$$

where, Q_t is the outflow from the reach at beginning of routing interval t; C_0 with superscript 1, 2,…, m denote the C_0 coefficients associated with inflows of tributaries 1, 2,…, m respectively at the beginning of the routing interval t; C_1 with superscript 1, 2,…, m denote the C_1 coefficients associated with inflows of tributaries 1, 2,…, m respectively at the end of the routing interval t; C_2 is the coefficient associated with outflow from the routing reach at beginning of routing interval t. i_t with superscript 1, 2,…, m denote the inflow of tributaries

Table 3
Coefficients and mean relative error for extended Muskingum method in calibrating phase.

Routing interval (h)	Coefficients of extended Muskingum equation					Objective function value (outcome of LP optimization)	Mean relative error (%)
	C_0^H	C_0^K	C_1^H	C_1^K	C_2		
24	0.513	1.138	0.082	−0.092	0.509	141.5	21.8
18	0.431	0.981	−0.070	−0.322	0.697	188.1	20.2
12	0.236	0.691	−0.164	−0.180	0.855	210.0	14.1
06	0.107	0.656	−0.100	−0.436	0.958	251.6	7.9

1, 2,…, m respectively at beginning of routing interval t. t is the routing interval.

In the present study, m is 2, one for release for power and spill from Hirakud reservoir and the second for the d/s catchment contribution. Rewriting the equation (3) for this case results in

$$Q_{t+1} = C_0^H\,\mathrm{RPS}_t + C_0^K\,\mathrm{DC}_t + C_1^H\,\mathrm{RPS}_{t+1} + C_1^K\,\mathrm{DC}_{t+1} + C_2\,Q_t \quad (4)$$

where superscript H and K of the C_0 and C_1 coefficients represent outflow from Hirakud reservoir (RPS) and d/s catchment contribution at Khairimal (DC) respectively.

According to O'Donnell (1985) and Khan (1993), the above set of C coefficients is valid for any ungauged lateral flow into the reach, if it is found by least square technique. In this study, Linear Programming (LP) is used for the purpose to minimize the sum of absolute deviations between observed and calculated flow at Naraj for determining C_0, C_1, and C_2.

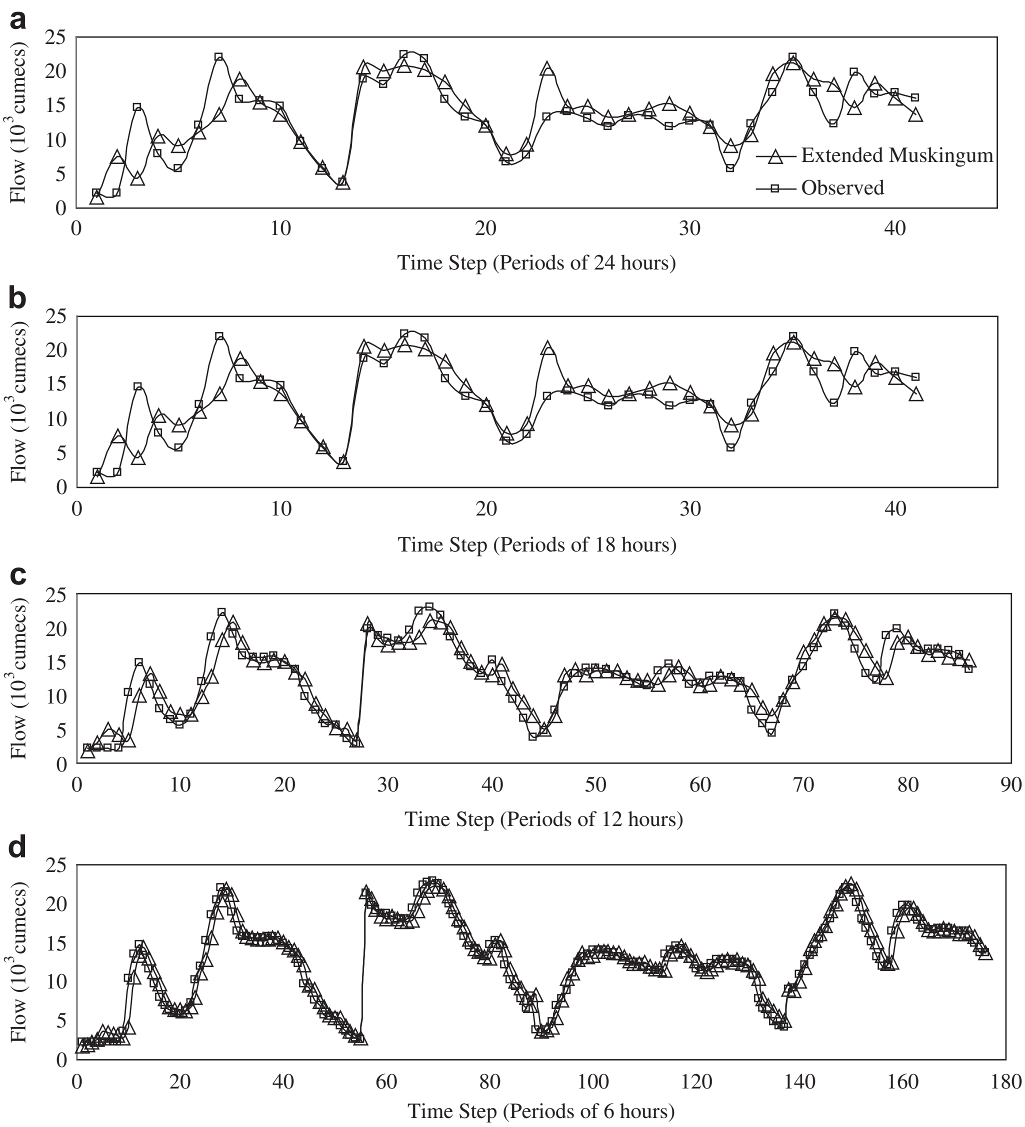

Fig. 3. (a)–(d). Performance for different routing intervals of 24, 18, 12, and 6 h in testing phase.

Table 4
Number of data sets falling within the ranges of errors (predicted versus observed flows).

Routing interval (h)	0–10%	10–20%	20–30%	30–40%	40–50%	Above 50%	Total number of data sets
24	18	11	4	2	1	5	41
18	28	14	5	4	2	4	57
12	48	20	4	8	1	5	86
6	127	29	13	1	1	5	176

$$\text{Minimize} \quad Z = \sum_{j=1}^{5} \sum_{t=2}^{n_j+1} \left[\text{pos.err}_{t,j} + \text{neg.err}_{t,j}\right] \tag{5}$$

Subject to

$$\begin{aligned} Q_{t,j} + \text{pos.err}_{t,j} - \text{neg.err}_{t,j} = {} & C_0^H \ \text{RPS}_{t-1,j} + C_0^K \ DC_{t-1,j} \\ & + C_1^H \ \text{RPS}_{t,j} + C_1^K \ DC_{t,j} \\ & + C_2 \ Q_{t-1,j} \\ & \text{for } j = 1,2,\ldots,5; \\ & t = 2,3,\ldots,n_j,n_j+1 \end{aligned} \tag{6}$$

$$\begin{aligned} & -\infty \le C_0^H \le +\infty; \quad -\infty \le C_0^K \le +\infty; \\ & -\infty \le C_1^H \le +\infty; \quad -\infty \le C_1^K \le +\infty; \quad -\infty \le C_2 \le +\infty; \end{aligned} \tag{7}$$

$$\text{pos.err}_{t,j} \ge 0; \quad \text{neg.err}_{t,j} \ge 0 \quad \textit{for } j = 1,2,\ldots,5; \quad t = 2,3,\ldots,n_j,n_j+1 \tag{8}$$

In this formulation, t denotes the time period and j is flood event. As there are five floods in calibrating set, j ranges from 1 to 5. n_j is the number of time periods in j^{th} flood event. pos.err and neg.err denote the positive and negative error values respectively. This formulation is valid for all routing intervals, namely 24 h, 18 h, 12 h, or 6 h and even any other. Only n_j values are to be changed suitably.

Linear Programming formulation is run for individual routing interval data. The resulting objective function values and values of coefficients C_0, C_1, and C_2 are presented in Table 3. It may be noticed that the C_1 values which correspond to inflows at the end of the time interval are all negative except 1 out of 8. Thus, the effect of the inflows at the end of the time interval is actually negative. The C_0^H, C_0^K values are decreasing with decrease of routing interval whereas C_2 values are increasing. In the case of values of C_1^H, C_1^K, there is no consistency of decrease or increase. It is observed that the objective function value of LP increases with decrease of routing interval. The mean relative errors with training data for the four time intervals are shown in the last column. These values i.e., 21.8, 20.2, 14.1, 7.9 are decreasing with decrease in time interval. The models, thus obtained, are used for testing to obtain mean relative errors for respective routing intervals.

The total number of testing data sets obtained on combining those of the four floods 2, 5, 6 and 7 are 41, 57, 86 and 176 for routing intervals of 24, 18, 12 and 6 h respectively. Fig. 3a–d present the comparison of discharge values predicted by extended Muskingum method and observed values. It is noted from Fig. 3a–d that flows predicted by Muskingum method are less than the observed flows for 15 data sets out of 41 (36.6%), 20 data sets out of 57 (35.1%), 38 data sets out of 86 (44.2%) and 69 data sets out of 176 (39.2%) for 24, 18, 12,6 h routing interval respectively.

Table 4 presents number of data sets falling within error ranges of 10%, 10%–20%, 20%–30%, 30–40%, 40–50% and above 50% for the predicted versus observed flows. It is observed from Table 4 that for routing interval of 24 h, 18 h, 12 h and 6 h, numbers of data sets falling in error range of 0–20% are 29, 42, 68, 156 (or 70.73%, 73.68%, 79.06%, 88.63% of the total numbers of data sets). For routing interval of 24 h, 18 h, 12 h and 6 h, numbers of data sets falling in error range of below 50% are 36, 53, 81, 171 (or 87.80%, 92.98%, 94.18%, 97.15% of the total numbers of data sets). Further the mean relative errors (in %) of testing data are computed using equation (1) and these are 23.3, 17.4, 15.9, 9.1 respectively for 24, 18, 12, 6 h. It is observed that mean relative error decreases systematically with decrease in routing interval as observed in calibration phase.

6. Conclusions

The extended Muskingum method is examined in this study for its applicability as flood routing method for the case study of Hirakud reservoir, Mahanadi river basin, India. Nine floods from 1992–1995 are analyzed for this purpose. Linear Programming is employed to determine the values of the coefficients required for the extended Muskingum method. It is observed that mean relative error decreases systematically with decrease in routing interval both for calibration and testing phase. In addition, it is observed that the flows predicted by Muskingum method are less than the observed flows for 15 data sets out of 41 (36.6%), 20 data sets out of 57 (35.1%), 38 data sets out of 86 (44.2%) and 69 data sets out of 176 (39.2%) for 24, 18, 12 and 6 h routing interval respectively.

From these results it can be concluded that the extended Muskingum method can be explored for similar reservoir configuration such as Hirakud reservoir system with suitable modifications. Limitations, suggested improvements and future directions of the present study are

1. In the present study LP is used as the basis for estimating Muskingum coefficients. Genetic algorithms, fuzzy inference system, Radial basis function and other advanced neuro computing techniques can be explored for this purpose.

2. Floods of 1992—1995 only are considered for the present study. More floods may be analyzed as and when sufficient data becomes available, to make the forecasting more robust and reliable.

References

ASCE Task Committee on Application of Artificial Neural Networks in Hydrology, 2000a. Artificial neural networks in hydrology. I: preliminary concepts. Journal of Hydrologic Engineering, ASCE 5, 115—123.

ASCE Task Committee on Application of Artificial Neural Networks in Hydrology, 2000b. Artificial neural networks in hydrology. II: hydrologic applications. Journal of Hydrologic Engineering, ASCE 5, 124—137.

Al-Humond, J.M., Esen, I.I., 2006. Approximate methods for the estimation of Muskingum flood routing parameters. Water Resources Management 20, 979—990.

Baliarsingh, F., 2000. Long-term and short-term optimal reservoir operation for flood control. Doctoral Thesis. Indian Institute of Technology, Kharagpur, India.

Chau, K.W., Wu, C.L., Li, Y.S., 2005. Comparison of several flood forecasting models in Yangtze river. Journal of Hydrologic Engineering, ASCE 10, 485—491.

Cheng, C.T., Chau, K.W., 2001. Fuzzy iteration methodology for reservoir flood control operation. Journal of the American Water Resources Association 37, 1381—1388.

Cheng, C.T., Chau, K.W., 2002. Three-person multi-objective conflict decision in reservoir flood control. European Journal of Operational Research 142, 625—631.

Cheng, C.T., Ou, C.P., Chau, K.W., 2002. Combining a fuzzy optimal model with a genetic algorithm to solve multiobjective rainfall-runoff model calibration. Journal of Hydrology 268, 72—86.

Cheng, C.T., Wang, W.C., Xu, D.M., Chau, K.W., 2008a. Optimizing hydropower reservoir operation using hybrid genetic algorithm and chaos. Water Resources Management 22, 895—909.

Cheng, C.T., Xie, J.X., Chau, K.W., Layeghifard, M., 2008b. A new indirect multi-step-ahead prediction model for a long-term hydrologic prediction. Journal of Hydrology 361, 118—130.

Choudhury, P., 2007. Multiple inflows Muskingum routing model. Journal of Hydrologic Engineering, ASCE 12, 473—481.

Chow, V.T., Maidment, D.R., Mays, L.W., 1988. Applied Hydrology. McGraw-Hill Book Co., Singapore.

Chu, H.J., 2009. The Muskingum flood routing model using a neuro-fuzzy approach. KSCE Journal of Civil Engineering 13, 371—376.

Das, A., 2004. Parameter estimation for Muskingum models. Journal of Irrigation and Drainage Engineering, ASCE 130, 140—147.

Das, A., 2007. Chance-constrained optimization-based parameter estimation for Muskingum models. Journal of Irrigation and Drainage Engineering, ASCE 133, 487—494.

Das, A., 2009. Reverse stream flow routing by using Muskingum models. Sadhana 34, 483—499.

Fernando, D.A.K., Jayawardena, A.W., 1998. Runoff forecasting using RBF networks with OLS algorithm. Journal of Hydrologic Engineering, ASCE 3, 203—209.

Fread, D.L., 1981. Flood routing: a synopsis of past, present, and future capability. In: Singh, V.P. (Ed.), Rainfall-runoff Relationship, Proceedings of the International Symposium on Rainfall-runoff Modelling. Mississippi State University, U.S.A.

Geem, Z.W., 2006. Parameter estimation for the nonlinear Muskingum model using the BFGS technique. Journal of Irrigation and Drainage Engineering, ASCE 132, 474—478.

Gill, M.A., 1978. Flood routing by Muskingum method. Journal of Hydrology 36, 353—363.

Khan, M.H., 1993. Muskingum flood routing model for multiple tributaries. Water Resources Research 29, 1057—1062.

Kshirsagar, M.M., Rajagopalan, B., Lall, U., 1995. Optimal parameter estimation for Muskingum routing with ungauged lateral flow. Journal of Hydrology 169, 25—35.

Mohan, S., 1997. Parameter estimation of non-linear Muskingum models using genetic algorithm. Journal of Hydraulic Engineering, ASCE 123, 137—142.

O'Donnell, T., 1985. A direct three-parameter Muskingum procedure incorporating lateral inflow. Journal of Hydraulic Engineering, ASCE 30, 479—496.

Patri, S., 1993. Data on Flood Control Operation of Hirakud Dam. Irrigation Department, Government of Orissa, India.

Samani, H.M.V., Jebelifard, S., 2003. Design of circular urban storm sewer systems using multilinear Muskingum flow routing method. Journal of Hydraulic Engineering, ASCE 129, 832—838.

Singh, V.P., McCann, R.C., 1980. Some notes on Muskingum method of flood routing. Journal of Hydrology 48, 343—361.

Stephenson, D., 1979. Direct optimization of Muskingum routing coefficients. Journal of Hydrology 41, 161—165.

Tung, Y.-K., 1985. River flood routing by nonlinear Muskingum method. Journal of Hydraulic Engineering, ASCE 111, 1447—1460.

Yoon, J., Padmanabhan, G., 1993. Parameter estimation of linear and nonlinear Muskingum models. Journal of Water Resources Planning and Management ASCE 119, 600—610.