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Research Paper

STUDY OF RAINFALL-RUNOFF RESPONSE OF A CATCHMENT USING SCS CURVE NUMBER COMBINED WITH MUSKINGUM ROUTING TECHNIQUES

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In this study, a hydrological model is developed by partitioning a river basin into many sub-basins by stream-network approach. Soil Conservation Service's (SCS) Curve Number (CN) method in combination with Muskingum routing technique is applied to route the surface runoff from different sub-basins. Hydrologic Modeling System (HEC-HMS) simulation tool is successfully applied for processing the input rainfall data recorded over four rainfall stations namely, Simulia, Rangagora, Kharidwar and Tusama. The performance of the developed model is checked for two severe storms occurred over the catchment. Results indicate that, the predicted runoff volumes obtained from developed model are in good agreement with observed runoff value for both the cases.

Keywords: SCS curve number method, Muskingum routing, Rainfall-runoff, Design flood

INTRODUCTION

The need for the 'Estimation of runoff corresponding to a rainfall event and routing the runoff to downstream through a river network' has become important today as increasing numbers of researchers are called upon to conduct flood hydrology studies for economical planning of river basin projects. The rainfall-runoff process is extremely a complex process and many hydro meteorological parameters are interconnected in a very complicated way. In many cases the developed models offer satisfactory performance when data on the physical characteristics of the watershed are available (Line *et al*., 1997; Colby 2001; Miller *et al.,* 2002). With the advancement of technology and research scope Remote Sensing (RS) and Geographic Information System (GIS) made it easier to extract many land surface properties. Estimating direct runoff depths from storm rainfall by the United States

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Department of Agriculture (USDA) by curve number (CN) method (SCS, 1972, 1985) probably the most widely used techniques. Greene and Cruise (1995) and Ponce and Hawkins (1996) worked on the applicability of curve number and considered the CN method as one of the useful tool for calculating runoff depths. Many other researchers (Blanchard, 1975; Jackson *et al*., 1977; Ragan and Jackson, 1980; Bondelid *et al.,* 1982) considered hydro-meteorological properties of the watershed and nature of land derived from satellite data and integrated them with GIS to estimate SCS CNs and runoff. Among many routing technique in river network modeling procedures the Muskingum method (Nash, 1959; Overton, 1966) probably the popular for flood routing. In Muskingum-methods of channel routing lesser number of data are required compared to other routing techniques like distributed kinematic wave flow routing. Gill (1978) and Luo (1993) conducted a study to ascertain the impact of land use and management practices on rainfall-runoff relationship and used GIS techniques to route runoff through a watershed. Olivera and Maidment (1999) used a grid network for flood routing by employing the first-passage-time response function. In another study carried out by Swensson (2003) showed that Muskingum flow routing perform much better than distributed models such as Kinematic wave flow routing, when storage within the watershed is taken into consideration. Das (2004) developed an algorithm for parameter estimation that iteratively solves the governing equations to identify the Muskingum model parameters.

STUDY AREA

The Kangsabati river originates from Jabarband in the Hill of Chotanagpur range, about 48 km north-west of Purulia district. The river traverses a length of 116.5 km up to the dam site through the district of Purulia and Bankura of West Bengal. River Kumari is the main tributary of the Kangsabati and joining the river on the right bank near Ambika Nagar in district Bankura, West Bengal. There are two other minor tributaries namely, Bhairabbanki and Tarafeni, which also meet the river on the right bank. On the left bank there is practically no tributary. At the dam site, catchment area of Kangsabati river is 1657.6 km^2 and that of Kumari river is 1968.4 km^2 totaling 3626 km^2. The dam is situated at Mukutmanipur on river Kangsabati and Kumari (Longitude = 86^o 45' 30" N, Latitude = 22^o 7'30" E) and shown in Figure 1. The slope of the Kangsabati river basin is very steep. At the dam site the slope is about 1.14 meters per km where as the upper portion is much steeper, about 7.58 meters per km. The river is practically dry for most periods of the year. But during rainy season, it experiences huge discharge with high velocity. This flow condition continues up to Midnapur anicut. It is observed that the river traverses topographies of various characters from barren, eroded and hilly catchment in the upper valley to alluvial tracts and ultimately into the deltaic region. A part of this deltaic zone is under the tidal influences. Four rainfall stations are shown in Figure 2. The mean annual rainfall in the Kangsabati valley is about 140 cm, the maximum being 182 cm observed in the year 1946, and the minimum was 96 cm occurred in 1947. 83% of annual precipitation takes place during the

Figure 1: Kangsabati Upper Catchment

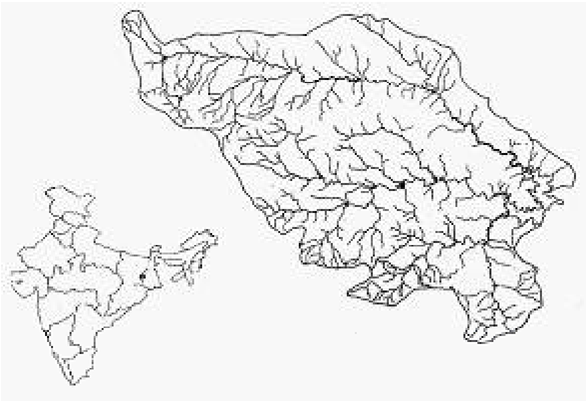
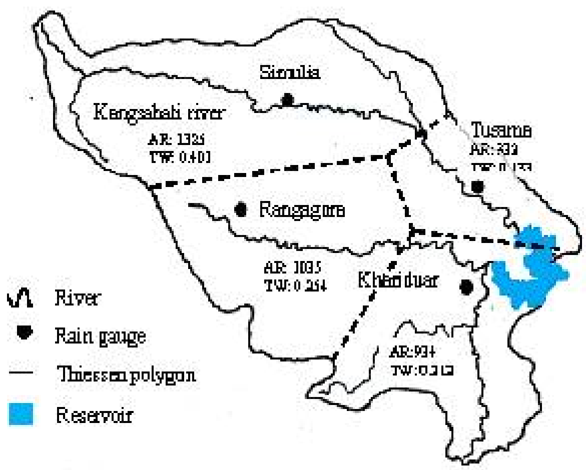


Figure 2: Kangsabati Upper Catchment and Rain-Gauge Locations Along with Thiessen Polygon



monsoon months. The average monthly rainfall during June, July, August, September and October are 21 cm, 29.6 cm, 33 cm, 21.7 cm, and 10.6 cm respectively. Analysis of rainfall during a few major storm followed by floods showed that rainfall exceeding 2.5 cm a day, generally occurs on 3 or 4 consecutive days and only in a few cases on 5 days. The floods in the Kangsabati basin are flashy in nature and generally last for a short duration. During heavy storm, there may be two or more spells of rainfall in the same storm. On such occasion, flood may prolong and may have a high peak followed or preceded by lower peak or peaks. From the records it is observed that, the flood has duration of 3-days and discharge more than 566 m^3/s is considered as a flood in the study area. Also, frequency analysis shows that flood of 7590 m^3/s with volume of 7,70,653 km^3 has a returned period of 100 years, while flood of 6,788 m^3/s with a volume of 4,80,870 km^3 has a returned period of 50 years. The design flood for Kangsabati dam which is 10,620 m^3/s has a higher returned period and has not been exceeded even in 1978 during which peak discharge found to be 9,912 m^3/s and it is highest flood on record till date.

THEORETICAL CONSIDERATIONS

The Hydrologic Engineering Centers Hydrologic Modeling System (HEC-HMS) is a widely used simulation tool for rainfall-runoff processes of watershed systems. In general, HEC-HMS watershed model is constructed by delineating boundaries around the river basin and sub-basin of interest. Several model options are available for representing various hydrological steps. Numerous options are available for representing infiltration, transforming excess rainfall into runoff and for simulating flow in open channels. Making the correct option requires knowledge of the watershed, the goals of the hydrologic study, and engineering judgment.

The Soil conservation service's (SCS) curve number (CN) method combined with Muskingum routing technique is widely used for estimating floods on a small to medium sized ungauged drainage basin. In the present study, SCS CN method is adopted due to the scarcity of the data and the simplicity of this method for determining peak rate of runoff. The runoff depth, detention storage, and initial abstraction for rainfall event are related by

$$P_e = \frac{(P-0.2S)^2}{(P+0.8S)} \quad ...(1)$$

$$S = \frac{25400}{CN} - 254 \quad ...(2)$$

$$I_a = 0.2S \quad ...(3)$$

where, P_e= runoff depth in mm; P = rainfall depth in mm; S = detention storage and; Ia = 0.2S = initial abstraction of rainfall by soil and vegetation. The CN value is a function of land use, soil type, and antecedent moisture. Using the tables published by the SCS, knowledge of the soil type and land use, the single-valued CN can be determined. But for a river basin that consists of several soil types and land uses, a composite CN can be calculated by

$$\mathrm{CN}_{composit} = \frac{\sum A_i \mathrm{CN}_i}{\sum A_i} \quad ...(4)$$

where, $\mathrm{CN}_{composite}$ = the composite CN used for runoff volume computations; i = an index of sub divisions of uniform land use and soil type; CNi = the CN for subdivision i; Ai = the drainage area of sub division i. In the present study, the methodologies adopted to convert the rainfall excess to the runoff hydrograph are (i) Snyder's Unit hydrograph Model and (ii) SCH Unit hydrograph Model. It is pertinent to mention that the choice of direct runoff model mainly based on the availability of hydro-meteorological information for calibration.

The Muskingum routing model is based on the continuity or storage equation in a river or channel and can be expressed by

$$\left(\frac{I_{t\text{-}1}+I_t}{2}\right) - \left(\frac{O_{t\text{-}1}+O_t}{2}\right) = \left(\frac{S_{t\text{-}1}+S_t}{\Delta t}\right) \quad ...(5)$$

where, I_t and I_{t-1} represent the inflow discharges, O_t and O_{t-1} the outflow discharges at section 1 and 2. S_{t-1} and S_t represent channel storages and ΔS is the increment or change in storage over time interval Δt. Storage in the reach is modeled as the sum of prism storage and wedge storage. Prism storage is the volume defined by a steady-flow water surface profile, while wedge storage is the additional volume under the profile of the flood wave. During rising stages of the flood, wedge storage is positive and is added to the prism storage. During the falling stages of a flood, the wedge storage is negative and is subtracted from the prism storage.

The volume of prism storage is the outflow rate, O multiplied by the travel time through the reach, K. The volume of wedge storage is a weighted difference between inflow and outflow, multiplied by the travel time K. Thus, the Muskingum model defines the storage as

$$S_1 = \mathrm{K}O_t + \mathrm{KX}(I_t - O_t) = \mathrm{K}\left[\mathrm{X}I_t + (1-\mathrm{X})O_t\right] \quad ...(6)$$

where, K = travel time of the flood wave through routing reach; and X = dimensionless weight and ranges from 0 to 0.5. If storage in the channel is controlled by downstream conditions, such that storage and outflow are highly correlated, then X= 0.0. In that case, eq. (6) resolves to S = KO; If X = 0.5, equal weight is given to inflow and outflow, and the result is a uniformly progressive wave that does not attenuate as it moves through the reach. If eq. (5) is substituted into eq.(6) and the result is rearranged to isolate the unknown values at time t, the result is

$$O_t = \left(\frac{\Delta t - 2\mathrm{KX}}{2\mathrm{K}(1-\mathrm{X}) + \Delta t} \right) I_t$$

$$+ \left(\frac{\Delta t + 2\mathrm{KX}}{2\mathrm{K}(1-\mathrm{X}) + \Delta t} \right) I_{t-1}$$

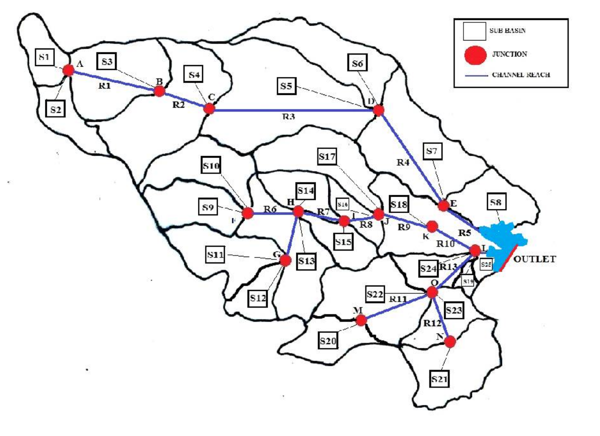
$$\left(\frac{2\mathrm{K}(1-\mathrm{X}) - \Delta t}{2\mathrm{K}(1-\mathrm{X}) + \Delta t} \right) I_{t-1} \qquad ...(7)$$

HEC-HMS solves eq. (7) recursively to compute ordinates of the outflow hydrograph, given the inflow hydrograph ordinates (for all *t*), an initial condition, and the parameters, K and X. As pointed out earlier, values of X vary from 0 to a maximum of 0.5 and the value of K is determined from eq. (7).

RIVER NETWORK MODEL BY HEC-HMS

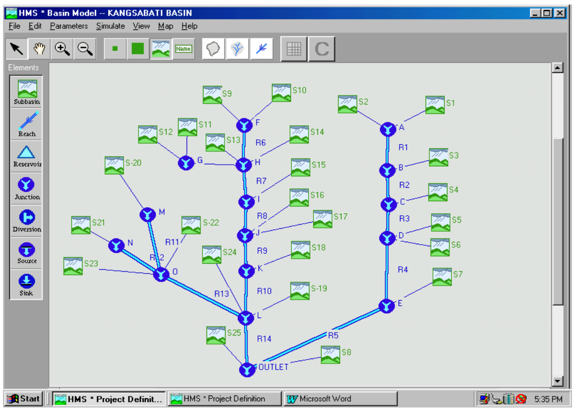
In the present study, the catchment boundary is traced from the topographical maps (sheet no. -73 J and 73 I) of scale 1:50,000 (i.e., 1 cm = 500 m) collected from Survey of India. After tracing the river network the catchment area is delineated considering the drainage density, number, and length of each tributary and also the adjacent drainage basin. The delineated catchment area is then compared with the catchment area map obtained from department of Irrigation and Waterways, Govt. of West Bengal and adjusted accordingly. Total area of the upper catchment is measured by a digital planimeter with a scale of 1:50,000 and found as 3626 km^2. As there are no storage structures or control structures in the upper catchment of Kangsabati reservoir, only two network components are considered: the sub-basin component and the channel component. Based on the topographical features of the area, drainage density, land use pattern, soil type and rain gauge locations, the total upper catchment is divided into 25 (twenty five) sub-basins as shown in Figure 3. Selection of the outlet points for each sub-basin is an important task and is done following the general stream network pattern traced from the toposheet. Once the outlet point is fixed, delineation of these sub-basin boundaries are performed following the procedure used for delineating the entire catchment, and with the help of other land marks like roads, canal layouts, etc. After defining the sub-basins, the streams and the junctions, the schematic network is then developed considering the hydraulics of flow for HEC-HMS model simulation. In this network, Sub basin components are numbered as S-1, S-2, S-3 …, S-25 (for 25 sub-basins); reach components are numbered as R-1, R-2, R-3, …, R-14 (for 14 reaches), and junction components are numbered as A, B, C, …, O

Figure 3: Stream Net-Work of Kangsabati Upper Catchment



(for 15 junctions). Figure 4 shows the schematic network of the Kangsabati upstream catchment.

Figure 4: Data Input Window for HEC-HMS Model



RESULTS AND ANALYSIS

Daily rainfall data for the upper catchment of the Kangsabati Reservoir are collected from the site office at Mukutmanipur of Irrigation and waterways department, Govt. of West Bengal, for all the four rain gauge station for the year 1997. Analysis of rainfall-runoff process developed in HEC-HMS as shown in Figure 3 is performed for a number of cases. The analysis starts with the calibration of the model parameters. As this model is being developed for the upper catchment for the first time, it is essential to calibrate the model parameters with reference to an observed outflow hydrograph in the study area. The next important step is to validate this model. That is, with the developed model, another rainfall event is processed and corresponding outflow hydrograph is compared with the observed outflow hydrograph for the same storm. Similar nature of the two hydrographs in terms of peak flow, time to peak and volume of flow ascertain the satisfactory performance of the model, from the present study objective point of view. In the present study, validations are performed for two major rainfalls on 23^{rd} July and 6^{th} August of 1997. It is pertinent to mention that, only the depths of rainfall and corresponding inflow flood volumes of the study area are available for the processing the data. But, the basic input for a rainfall event in the HEC-HMS model is the corresponding observed hyetograph. Some pre-processing was needed to obtain the hyetograph and flood peak value. For the storm occurred on 6^{th} August 1997, the depths of rainfall recorded by the four rain gauges are shown in Table 1. The network model developed in HEC-HMS is used with the rainfall event on 6^{th} August 1997. Both the Snyder's UH and SCS lag methods are used separately. The trial values for model parameters are selected from the available topographical and land use maps. Outflow hydrograph at the catchment outlet as obtained from the first run is then compared with the observed value. The model parameters are then suitably modified for the observed discrepancies. The results of the final calibrated model are shown in Figures 5. A comparison study is given in Table 2. For the storm occurred on 23^{rd} July 1997, the depths of rainfall recorded by the four rain gauges are shown in Table 3. The calibrated network model is used with the rainfall event on 23^{rd} July 1997. Here also, analysis is made using both SCS lag and Snyder's UH method. The result of the validation is shown in Figure 6. A comparison study is given in Figure 7 and Table 4. The comparison study indicates a close similarity between the model results and corresponding observed values. Hence, the calibrated model is well accepted for future predictions.

Table 1: Recorded Rainfall on 6th August, 1997

Station	Simulia	Rangagora	Kharidwar	Tusama
Rainfall (mm)	101.0	82.4	57.4mm	86.8mm

Table 2: Comparison of Simulated Flood Peak and Flood Volume with Observed Values

Case 1: 6th August, 1997

Method	Flood Peak (m^3/s)		Flood Volume ($K.m^3$)	
	Model	Observed	Model	Observed
SCS Lag	1161.2	1126.0	87904	87543
Snyders UH	1103.6	1126.0	87356	87543

Table 3: Recorded Rainfall on 23rd July, 1997

Station	Simulia	Rangagora	Khariduar	Tusama
Rainfall (mm)	127.40	188.8	237.0mm	140.6mm

Table 4: Comparison of Simulated Flood Peak and Flood Volume with Observed Values

Case 2: 23rd July, 1997

Method	Flood Peak (m^3/s)		Flood Volume (km^3)	
	Model	Observed	Model	Observed
SCS Lag	3450.6	3508.76	226119	227812
Snyders UH	3320	3508.76	227213	227812

Figure 5: Hydrograph at the Catchment Outlet for Storm of 6th August 1997: a) SCS UH Method and b) Snyder UH Method

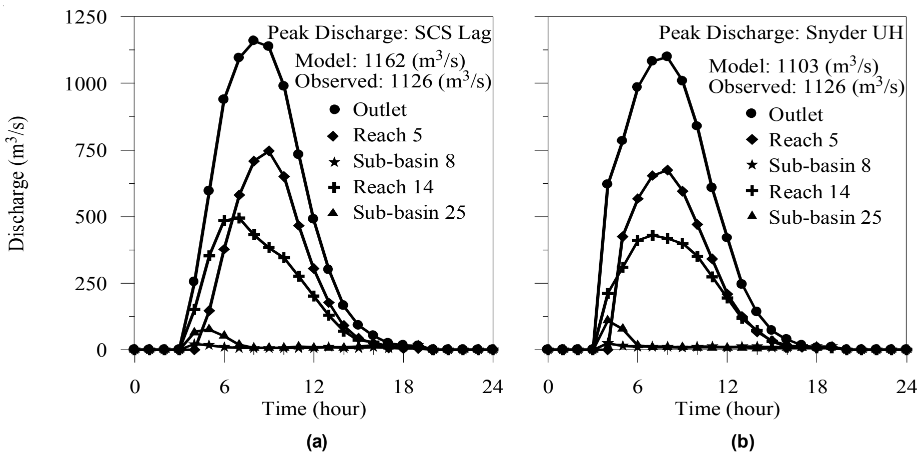


Figure 6: Hydrograph at the Catchment Outlet for Storm of 23rd July, 1997: a) SCS UH Method and b) Snyder UH Method

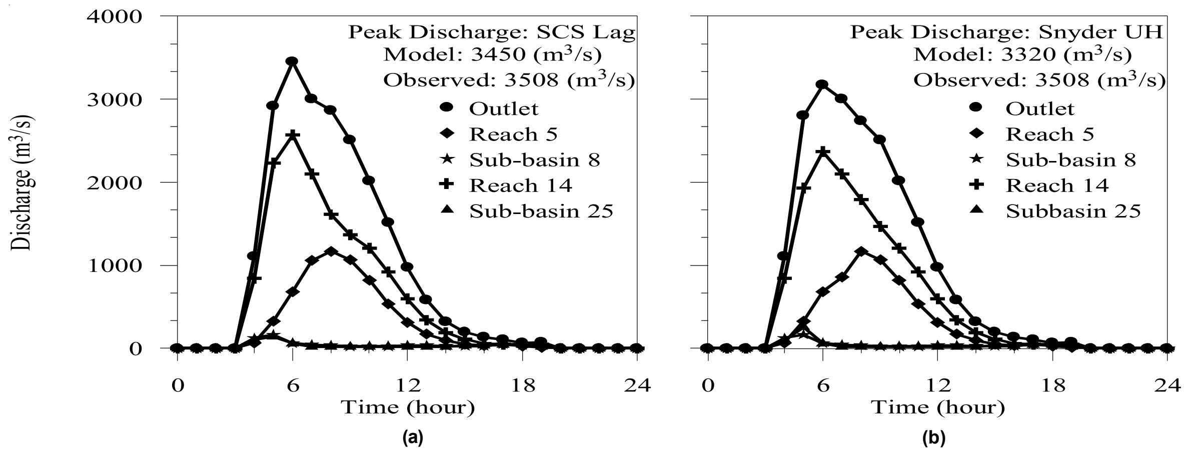


Figure 7: Comparison of SCS-UH and Snyder UH at the Catchment Outlet with the Observed Peak Discharges for Storm of 23rd July and 6th August, 1997

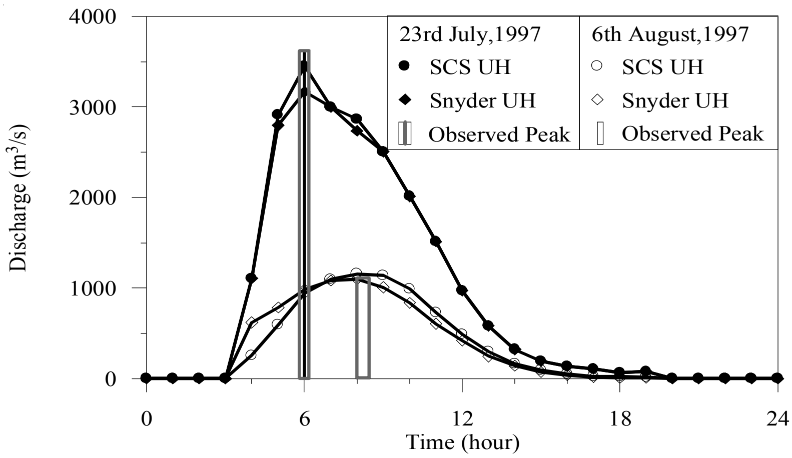


Figure 8: Variation of Runoff and Curve Number

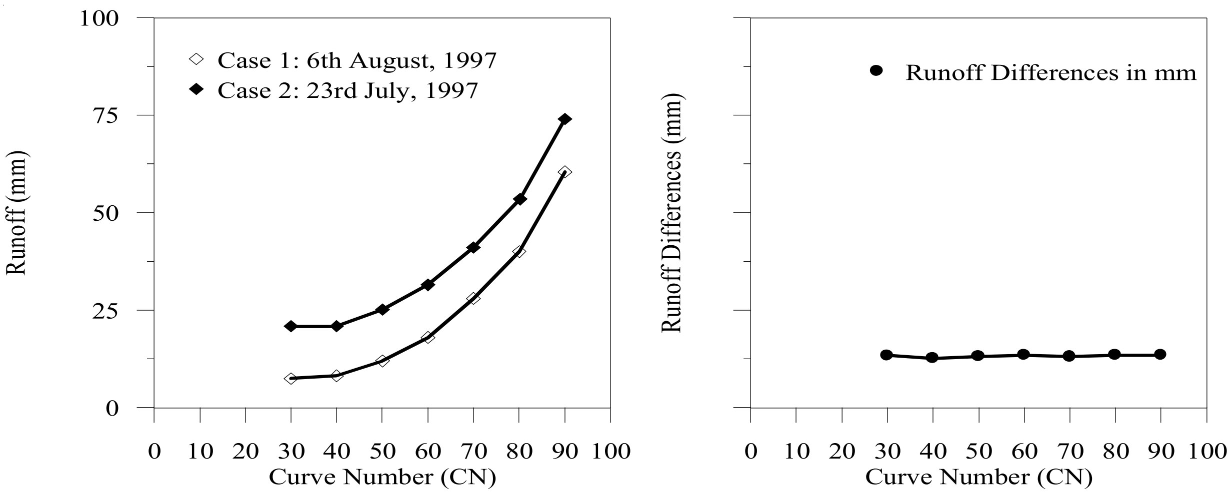


Figure 9: Variation of Peak Discharge and Curve Number

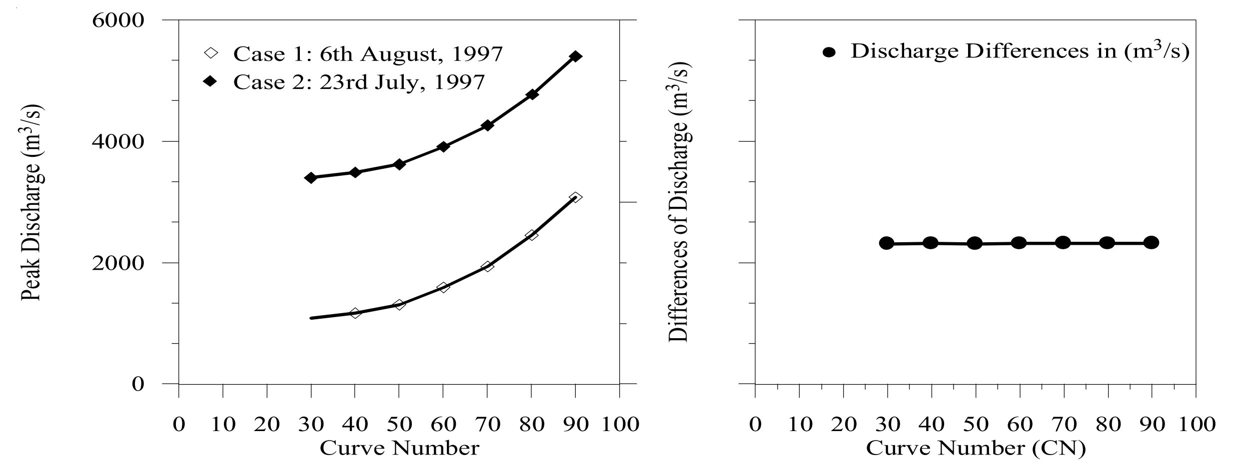


Figure 10: Variation of Outflow and Curve Number

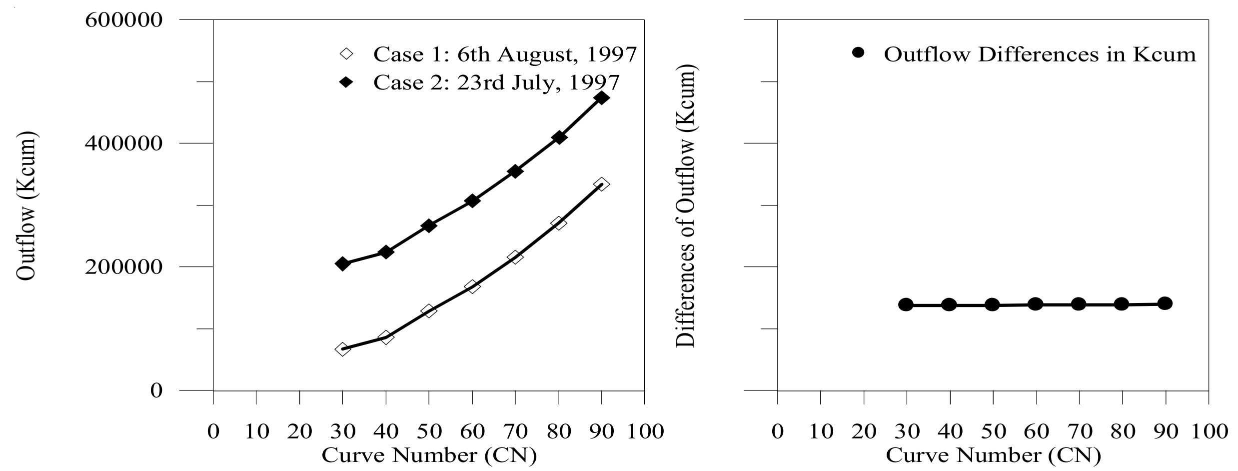


Figure 11: Rainfall-Runoff Correlation for Case1 (6th August, 1997)

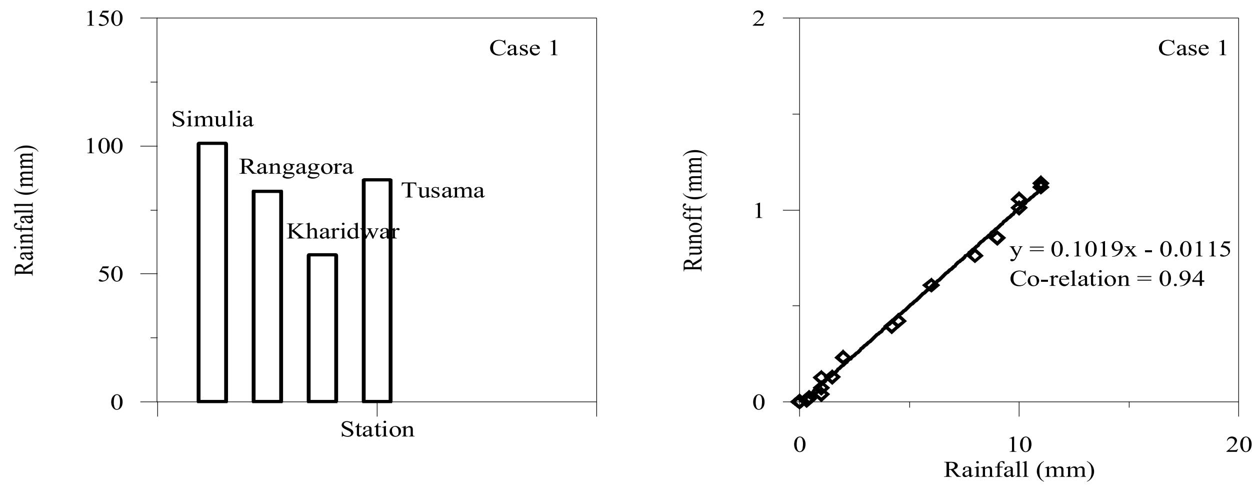
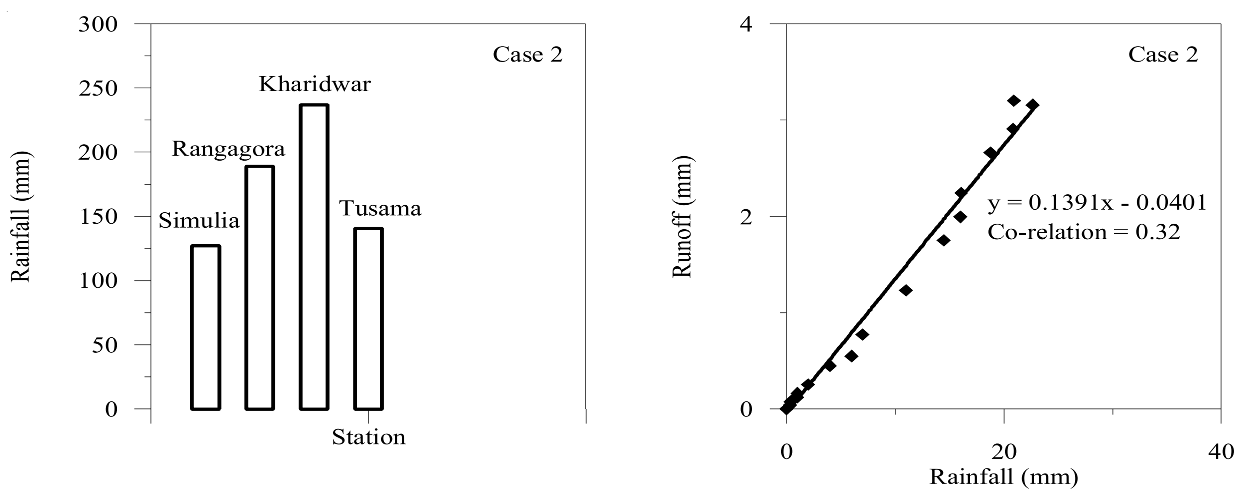


Figure 12: Rainfall-runoff Correlation for Case 2 (23rd July, 1997)



SENSITIVITY ANALYSIS

The main uncertain parameter that affects the result of simulation process is the CN value. Selection of this value depends on judgement about the watershed physical characteristics as well as its antecedent moisture conditions. A sensitivity analysis is performed to assess the effect of changing the SCS-CN values on the peak flow values. To support the results found from the simulation, both the calibration and validation model results were evaluated. The graphs for variation of runoff, peak discharge and outflow with CN value for both the rainfall event for the calibration period are shown in Figures 8 to 10. The variation of all the three parameters by the developed model for Kangsabati upstream catchment for both the rainfall events are found to indicated satisfactory results and percentage variation are also shown in Figure 8 to 10. Also the rainfall runoff correlations are 0.92 and 0.32 for the case1 and case 2 respectively and found to be satisfactory and shown in Figures 11 and 12.

CONCLUSION

Based on the above flood analysis and discussion, the following conclusions can be made.

1. The comparisons of peak flow and flow volume indicate that the network model developed here for the upper catchment is quite acceptable for developing outlet hydrographs, in terms of flood peak and flood volume. Hence, in general, methodology used here is quite acceptable for practical purposes of water resource planning project. However, this model requires various observed data for several parameters including rainfall and discharge values. The present study suffered from the scarcity of such data. Although, this no way limits the capability of the model or HEC-HMS to produce appreciable results. When reliable observed rainfall and runoff data are available, the same can simply be used as input to the model without any problem. However, more reliable observed data would have yielded better calibration, particularly for CN and K, X parameters.

Hence, for obtaining better results following suggestions are incorporated here.

2. Reliable discharge data at several important locations on major streams, particularly at the junction points are very much required to develop any river network model. These values are particularly required to calibrate the Muskingum parameters.

3. As the model deals with flood hydrograph, it is desired to have rainfall records in durations of 15 min at least, during flood season. Records of actual durations of the storms are very important. Hence, recording type rain gauges and corresponding recorded rainfall mass curve is very much needed. This will give the precise duration of the storm event and information on variation of rainfall intensity.

4. As the calibration and validation process is dependent on the observed inflow hydrograph at the reservoir, proper measures are very much needed to get reliable inflow data, at the dam site.

5. Soil characteristics and information regarding infiltration are also very important. Reliable soil test reports may help is assessing proper values for CN.

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