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Watershed Bounded Network Model (WBNM) for Runoff Prediction of Large Basins

K. Venugopal, T. V. Gopalakrishnan and R. Sakthivadivel,
Centre for Water Resources, Anna Univ., Madras, India

Applicability of a nonlinear version of the watershed bounded network model (WBNM) for large basins namely Shellbyville (1,246 km^2), Columbia (3,129 km^2), Centreville (5,304 km^2) and Hurricane Mills (6,536 km^2) of Duck River basin, to predict flood hydrograph based on rainfall excess hyetograph is investigated. Boyd et al. (1979) have applied this model for basins with areas ranging from 0.4 to 251 km^2 and recommended a constant value for the model parameter namely, c. In this investigation, c is found to vary from basin to basin and in a given basin it varies with rainfall excess volume. The effect of subdividing the basin into a finer mesh on the reproduction of the observed flood hydrograph is also investigated and the results discussed.

Introduction

Prediction of the flood hydrograph for any given storm event over a basin is an important task for hydrologists for planning, design and operation of water resources systems. For a given storm, estimating rainfall excess and transforming it to a runoff hydrograph is an accepted approach. There exist many models starting from unit hydrograph theory proposed by L. Sherman (1932) for conversion of rainfall excess to runoff.

The watershed bounded network model (WBNM) proposed by Boyd et al., (1979) takes into account geomorphological and hydrological characteristics of the basin for conversion of rainfall excess to runoff. Earlier models proposed by Clark (1945), Nash (1958), Dooge (1959), Singh (1962), Diskin (1964) and Kulandaiswamy (1964) etc. involve several parameters to be evaluated whereas WBNM needs a single parameter and realistically represents the catchment structure and flow of water on the catchment surface. As such this model can be used by practising engineers with ease. Linear and nonlinear versions of WBNM are available. In the linear version, the lag is constant for all events and is related to the size of the sub-area whereas in the nonlinear version the lag is related to both sub-area and instantaneous stream discharge. It is suggested that the nonlinear model is more versatile for practical applications. Boyd et al. (1979) have investigated the applicability of WBNM for drainage areas ranging from 0.4 to 251 km^2 in Eastern New South Wales, Australia. The value of c is taken to be an average value in their study for the purpose of prediction.

In the present paper, nonlinear WBNM is applied to Shellbyville, Columbia, Centreville and Hurricane Mills (Duck River Basin) having drainage areas 1,246, 3,129, 5,304 and 6,536 km^2 respectively, using eight storm events. Parameter c of the model is related to rainfall excess volume. For all the four basins the effect of fineness of subdivisions is investigated and the results are discussed. The predicted flood hydrograph using this model compares fairly accurately with the observed one.

Description of the Model

WBNM advocates division of the catchment into a number of sub-areas along watershed lines, each representing a storage element namely an ordered basin or an inter-basin area. Ordered basins are complete sub-catchments and no water flows into them across any boundaries. The rainfall excess occurring over each of these sub-areas is transformed into a direct runoff hydrograph at the outlet. Inter-basin areas are sub-catchments with a stream flowing through them draining upstream sub-areas. Outflow from each inter-basin area consists of both runoff from upstream areas that has been transmitted through the inter-basin by its main stream, and rainfall excess that has been transformed to runoff by the same processes occurring in ordered basins.

Each sub-area of the basin (ordered or inter-basin) is represented by a concentrated storage. This storage is linked to the other storages representing adjacent sub-areas in the same network topology as the streams in the basin.

In Fig. 1(a), (1) and (2) are ordered basins and (3) is an inter-basin. Rainfall excess in the ordered basin (1) is transformed to runoff first. Then the ordered basin (2) is taken up for transformation of rainfall excess to runoff. With the runoff of the ordered basin (2) the runoff from ordered basin (1) computed earlier is added before taking up the next sub-basin. Because next sub-basin is an inter-basin the flow from sub-basin (2) is treated as inflow to sub-basin (3) and is routed to get the outflow. Now the rainfall excess in sub-basin (3) is transformed to runoff

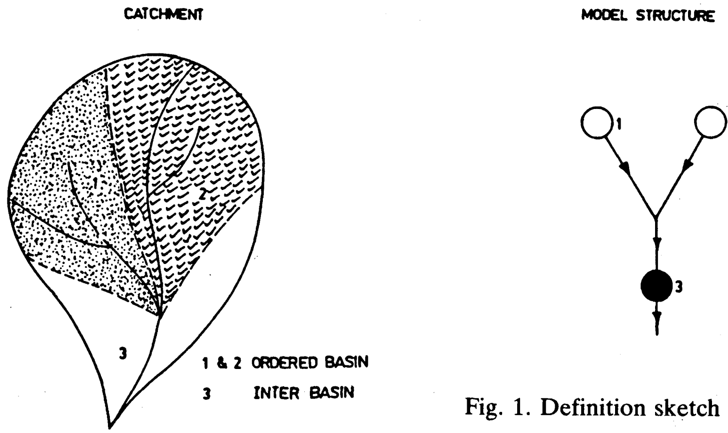

Fig. 1. Definition sketch for WBNM.

and is added to the routed outflow at the downstream point. This outflow forms inflow to the next sub-basin and this process is repeated till the last sub-basin is reached to obtain the direct surface runoff hydrograph of the entire basin.

The transformation lag time, K_B (hrs) both for ordered, inter-basin, and transmission lag time, K_I (hrs) for inter-basin are evaluated (Boyd et al. 1979) using

$$K_B = c\,A^{0.57}\,q^{-0.23} \tag{1}$$

$$K_1 = 0.6\,c\,A^{0.57}\,q^{-0.23} \tag{2}$$

where c – a parameter
A – the size of the sub-area (km^2)
q – the instantaneous discharge (cumecs)

For each sub-area, the rainfall excess hyetograph is transformed into an inflow hydrograph. The lag parameters K_B and K_I for each sub-area as appropriate are calculated using Eqs. (1) and (2). Inflow hydrographs are routed through the nonlinear storage element using a numerical solution of the continuity and storage-discharge relation to produce the nonlinear storage routing equation

$$q_2 = \frac{(i_1 + i_2)\Delta t + q_1 (2K_1 - \Delta t)}{(2K_2 + \Delta t)} \tag{3}$$

where i_1 – inflow hydrograph value at start of routing period (cumecs)
i_2 – inflow hydrograph value at end of routing period (cumecs)
q_1 – outflow hydrograph value at start of routing period (cumecs)
q_2 – outflow hydrograph value at end of routing period (cumecs)
K_1 – value of lag parameter at start of routing period (hrs)
K_2 – value of lag parameter at end of routing period (hrs)
Δt – time period (hrs)

Value of K_1 is determined from Eq. (1) for transformation of rainfall excess to runoff for both an ordered basin and an inter-basin. For transmission of runoff through an inter-basin the value of K_1 is computed using Eq. (2). The routing period is the same as the time interval used to define the hyetograph.

At each time increment, values of i_1, q_1 and K_1 at the start of the routing period are known and the inflow i_2 at the end of the routing period is also known. Eq. (3) is solved to yield at value of q_2. However, this requires a value of K_2 which in turn depends on q_2 (as shown by Eqs. (1) and (2)). An iterative solution of Eq. (3) is used at each time increment. Initially K_2 is equal to K_1 and Eq. (3) is solved for q_2. A new value of K_2 is calculated from Eqs. (1) or (2) as appropriate, and used in Eq. (3) to calculate a new value of q_2. The iteration is continued until successive calculated values of q_2 differ by less than 1 percent. A nominal value of $q = 0.01$ is used at the start of hydrograph rise.

Modelling Details

Fig. 2 shows the details of the basins under study indicating the rainfall recording stations and stream gauging stations. Table 1 shows the details of the four catchments taken up for this study. Each basin is divided into number of sub-areas (Fig. 2) as indicated in Table 1. For all the non-recording rainfall stations, mass curves were constructed based on observed rainfall at recording stations. For all the eight storms considered base flow separation is done using the straight line method from the runoff hydrograph and the corresponding rainfall excess obtained for a time period of 5 hours. The rainfall excess is assumed uniform over the entire basin for each time period. Model network of storages for all the basins is shown in Fig. 3. Parameter c is obtained for seven storms by search technique by starting from an arbitrary value so as to get maximum correlation between the observed and the computed hydrographs. The range of rainfall excess volumes utilised and the range of c values and correlation coefficients obtained in the calibration phase is indicated in Table 2.

Table 1 – Details of the Catchments

No.	Catchment	Area (km^2)	No. of rain gauges		No. of sub-areas
			Recording	Non-recording	
1.	Shellbyville	1,246	1	3	22
2.	Columbia	3,129	2	9	69
3.	Centreville	5,304	4	14	121
4.	Hurricane Mills	6,536	4	16	149

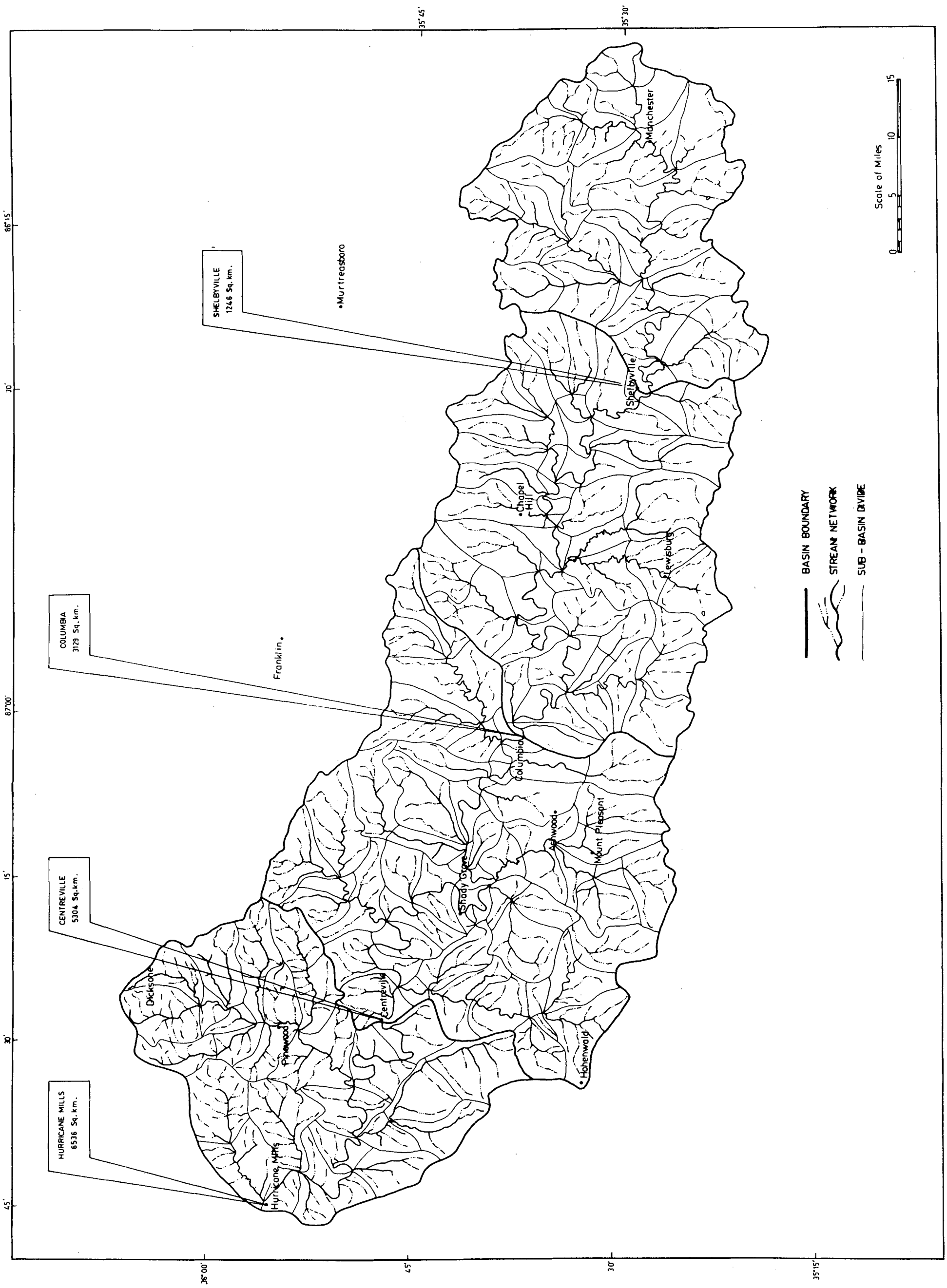

Fig. 2. Duck River basin.

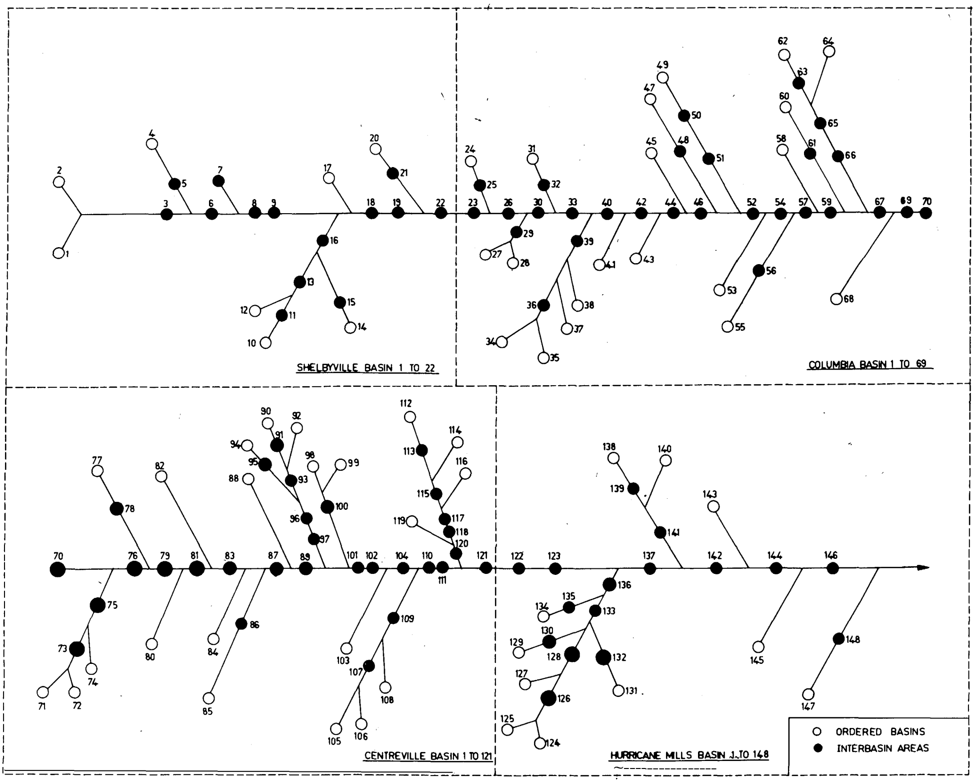

Fig. 3. Model structures.

Fig. 4 shows a typical comparison of computed and observed runoff hydrographs in the calibration phase for all the basins. Boyd et al. (1979) have suggested a constant value of *c* for a basin after investigating catchments with areas between 0.4 and 251 km^2. It is evident from Table 2 that the value of *c* varies considerably.

Table 2 – Results of Calibrations

No.	Catchment	Rainfall excess volume (cms)	Parameter, *c*	Correlation coefficient
1.	Shellbyville	3.33-6.43	1.89-3.43	0.9734-0.9846
2.	Columbia	3.23-17.0	2.36-4.77	0.9507-0.9841
3.	Centreville	2.80-20.0	2.33-4.68	0.9618-0.9904
4.	Hurricane Mills	2.27-13.0	2.28-3.76	0.9691-0.9859

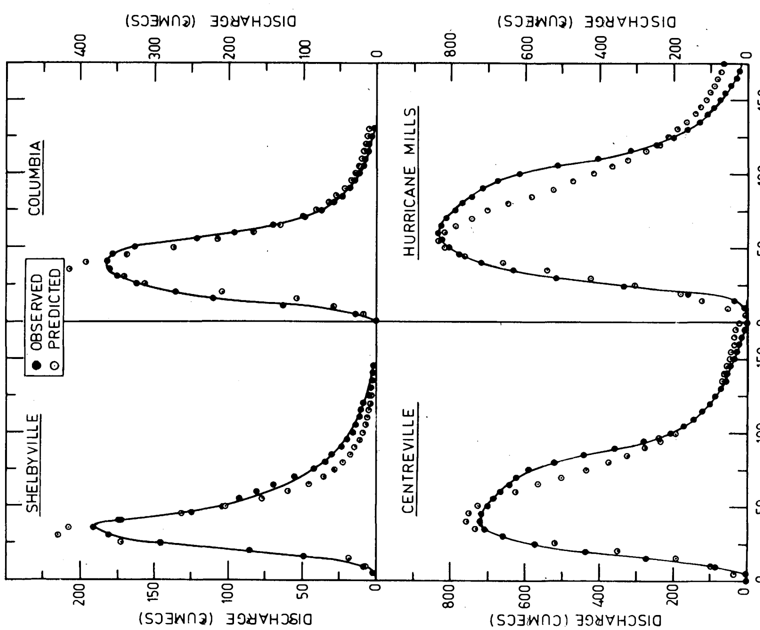

Fig. 4. Surface runoff hydrograph – 1952 (calibrated).

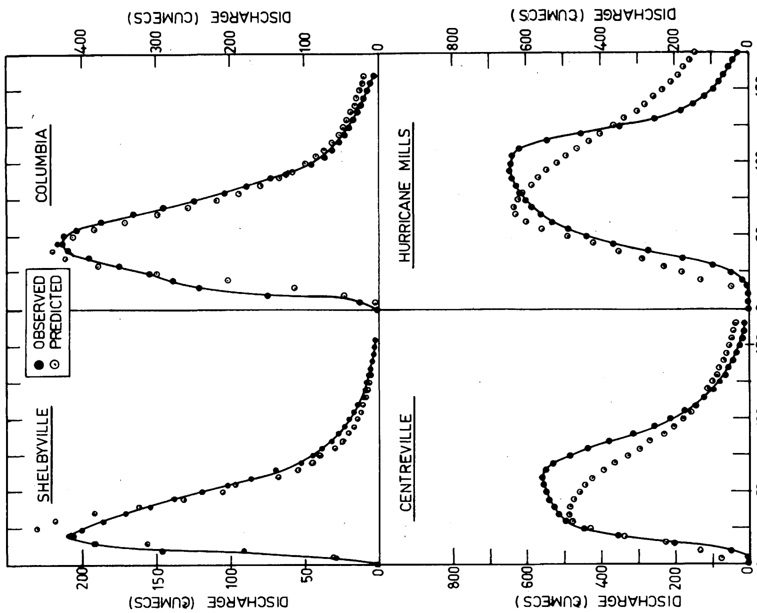

Fig. 6. Surface runoff hydrograph – 1954 (predicted).

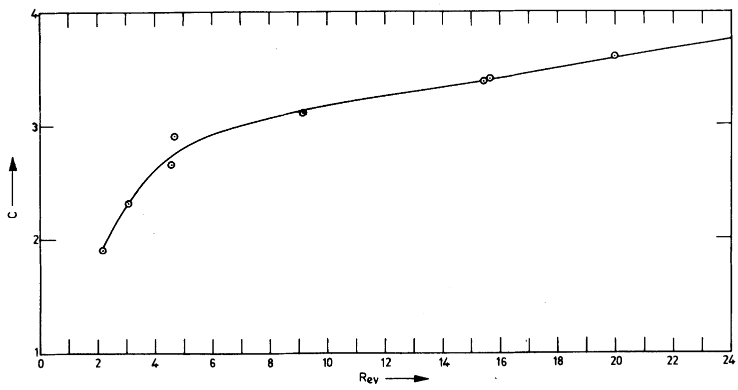

Fig. 5. Variation of parameter *c* with rainfall excess volume.

Parameter *c* is related to rainfall excess volume (R_{ev}) and this relation for the Shellbyville basin is shown in Fig. 5. Similar pattern of curves was obtained for other basins.

For each basin, rainfall vs. rainfall excess (R_{ev}) graphs are prepared with the storms used for calibration. The rainfall excess for the storm of 1954, the one not used for calibration, is estimated from the above mentioned graphs and the value of *c* from the curves *c* vs. R_{ev} for each basin. With the rainfall excess ordinates and the corresponding values of *c* for each basin the surface runoff hydrographs were predicted and compared with observed surface runoff hydrographs as indicated in Fig. 6.

Effect of Fineness of Subdivisions

The number of sub-areas used for analysis are 22, 69, 121, 149 for Shellbyville, Columbia, Centreville, Hurricane Mills respectively which are subjective. The effect of varying the number of sub-areas on model response was examined. For each model structure the number of elements were progressively reduced forming a combined sub-area of larger size. From the predicted surface runoff hydrographs four response measures namely the peak discharge (*QPS*), the time to peak (*TPS*) and the lag time (*LAG*) and the correlation coefficients (*R*) were determined. The set of results shown in Fig. 7 is typical of those for all the catchments. *TPS, R* and *LAG* get stabilised beyond a minimum number of divisions.

The results indicate that the number of sub-areas selected for any catchment is not critical as long as it is sufficiently large. For the four catchments studied the minimum number of sub-areas required ranges from 10 to 20, with larger values

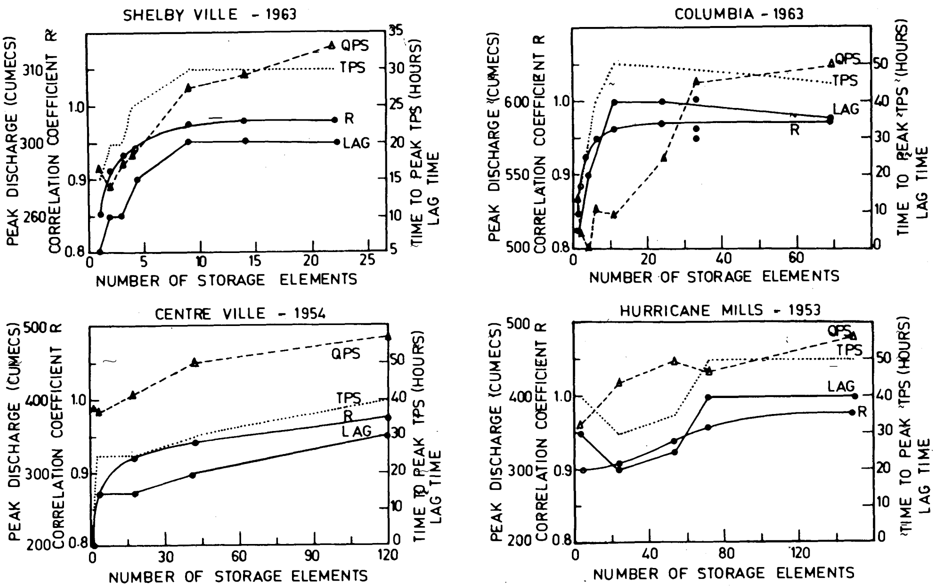

Fig. 7. Effect of model fineness on model response.

required for bigger catchments. The value of c changes rapidly for coarser divisions and gets stabilised for finer divisions. The pattern of variation of c for various storms was found to be similar and a typical variation for Hurricane Mills is shown in Fig. 8.

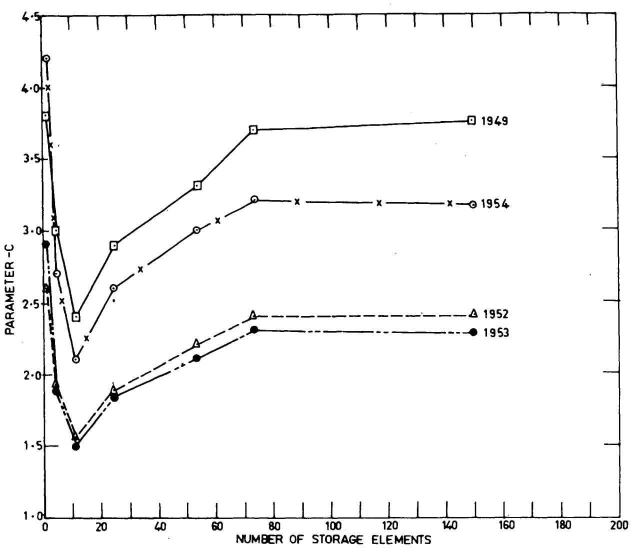

Fig. 8.
Variation of parameter c with model fineness.

Results and Conclusions

Boyd et al. (1979) have used a constant value of c for modelling basins with areas ranging from 0.4 to 251 km^2. It is observed that the value of c varies considerably even with storms over the same basin apart from one basin to another. The value of c related to the rainfall excess volume. The predicted surface runoff hydrograph with estimated value of c compares well and the error in peak discharge varies from (–) 12.72 to + 15 percent.

From the sensitivity analysis it was found that the minimum number of subareas required varies from 10 to 20 with larger values required for bigger catchments. It is observed that the computer time (CPU in IBM 370/155) utilised can be considerably reduced by choosing the minimum number of elements recommended. Sine WBNM involves only one parameter, namely c, and takes into account realistically the geomorphological and hydrological characteristics of the basins, this model can be used with ease by practising hydrologists.

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Address: Centre for Water Resources,
College for Engineering, Guindy,
Anna University,
Madras-600 025. India.