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Predicting Varied Kinematic Roughness Parameter during Storm Durations under Ungauged Catchment Situations

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ABSTRACT: Effective overland roughness happens to be the key parameter of Kinematic Wave (KW) models, which truly govern's the overall success of model application in terms of surface runoff simulations. Most of the researchers adopted certain published unique values of Manning's roughness (*N*) based upon their own subjective judgments. The focus of present study is to address the temporal variability of *N* during storm duration, that too under less favorable situations where no sufficient hydrologic records are available. Use of unit pulse response through the Soil Conservation Service (SCS) synthetic Unit Hydrograph (UH) is demonstrated for self prediction of varied (N_j) during application of KW equations at individual time steps (*j*) of simulation durations. A natural catchment (1400 ha) has been adopted for application of the approach, considering it as totally ungauged catchment. Synthetic SCS UH was derived utilizing realistic records of rainfall runoff depths, using SCS method. For same unit duration, responses of different rainfall excess depths (0.5–10 mm) were computed in the similar form of Direct Runoff Hydrographs (DRH), considering system to be linear. For each hydrograph ordinate the N_j values were computed with trial and error by running a simple KW model with ensured KW conditions. Time varied optimized values of N_j and overland flow depth (h_j) were obtained for each unit pulse hydrograph for two specific time segments viz. rising and falling limbs of h_j. Pooled set of values for these time segments were utilized for developing predictive roughness equations. Later these equations were incorporated in KW model to accommodate event based temporally varied nature of *N*. KW surface runoff responses were simulated for a few storms and found to match reasonably well with the available observed hydrographs on the catchment. Paper comprises a preliminary effort on successful application of KW approach for purely ungauged situations where not a single DRH record is available. Through this study yet another positive utilization SCS UH is projected to generate time varied nature of KW roughness.

INTRODUCTION

Prediction of surface runoff responses from variety of natural watersheds remained the key objective under majority of hydrological studies. Due to complex nature of Rainfall-Runoff relationships it still remained a difficult task for researchers as well as field managers. There exists a plethora of methods and models to estimate runoff which comprised variety of lumped and distributed models based on either or non-linear system's approach. These methods and models vary in their capabilities, constraints, data requirements, and analytical vigor. They necessarily required varied magnitudes of observed data, physiographic indicators, parameter values information in the domain of space and time. Further, the problem of surface runoff prediction remains more acute under ungauged situations where available observed data/records/parameter values are either nil or thin. This remained truer for tropical situations like India, subsists with little gauged watersheds. Reviewing available rainfall-runoff models depicts that among distributive physiographic models (based on nonlinear system approach), the simpler KW models are considered most favorable and user friendly. Similarly, among lumped conceptual models (based on linear systems approach), SCS UH still avails the status of most popular and acceptable method across the globe.

The overall success of any such KW model application depends upon the precision with which the effective roughness parameter is estimated or assigned in the model (Singh, 1997; Singh and Frevert, 2001). This is generally accomplished either by direct use of certain published values of Manning's n or through hydrograph analysis. Majority of researchers relied upon the use of certain published lumped values of Manning's *N*, ignoring its varied nature over space and time domain. Also if we see the availability of such published values, it is very much evident that enough information exists for stream and channels having natural or synthetic surfaces (Chow, 1959; Jarret, 1984; Engman, 1986). However similar values for overland flows on natural catchments continue to be under investigation till date (Gaur, 1999).

The central idea of this paper is to adopt an integrated application of KW and SCS synthetic UH based concept by viewing a 1400 ha size natural watershed as a nonlinear system. It well describes the watershed's hydrologic behavior within lighter computational burden. The hydrologic response function is viewed as a time-variant characteristic function in the study and is developed to include the effect of water movement mechanism in the output watershed hydrograph. Synthetic UH is derived by using SCS curve number approximation by relating it to prevailing realistic ranges of conditions pertaining to soil vegetation covers and input rainfalls over the study watershed. The overland surface runoff under ungauged condition has been computed by incorporating temporally varied effective overland roughnesses in a simpler KW model. Unit pulse response through the SCS method are used for estimation of temporally varied overland roughness in order to simulate the surface runoff responses.

MATERIAL AND METHODS

Study Watershed

The Ministry of Railways, Govt. of India, monitored the hydrological responses of few medium sized streams where the flash floods caused damages to its culverts and bridges. Data of one such watershed of bridge number 719 were procured and used in this work. Railway Bridge number 719 is situated on the Jalarpet-Banglore section of the Indian Railways. The location and geographic details of study watersheds is shown in Figure 1(a). The worked out physiographic parameters used for computation of effective overland roughness and other flow parameters on study watershed along with general features are given in Table 1.

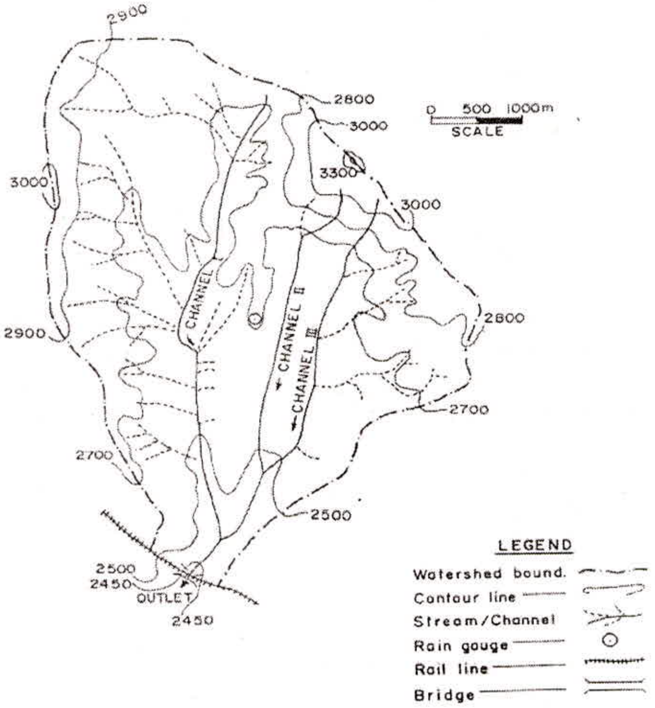

Fig. 1(a): Physiographic Map of Study Watershed (Railway Bridge No. 719)

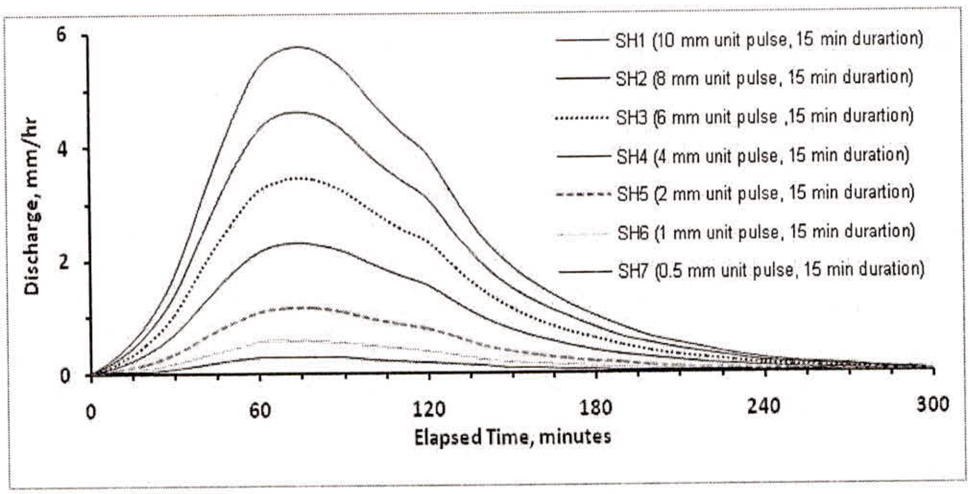

Fig. 1(b): SCS Synthetic Unit Hydrographs of different Depths on B719 Watershed

Table 1: Physiographic Details of Study Watershed

General Features	*Applied KW Configurations*		
	Parameters	*Overland Planes*	*Channel Phase*
Latitude : 12°52′ N	Lengths, m	972.2	7200
Longitude : 78°16′ E	Slope, m/m	0.068	0.021
Area : 1400 ha	Space step (Δx), m	10.0	200
Soil Group/Major Land uses:	Cross-section shape	–	Trapezoidal
Rocky outcrops Dryland cultivation + Forest	Base width (B), m	–	15
	Side Slope (Z), H:V	–	2:1
	Time Step, min	5	5

Mathematical Formulation of KW Model Used

Below given Saint Venant's equation forms basis for mathematical formulation of KW theory,

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q \qquad \ldots (1)$$

$$\frac{1}{g}\frac{\partial u}{\partial t}+\frac{u}{g}\frac{\partial u}{\partial x}+\frac{\partial y}{\partial x}+S_f-S=0 \qquad \ldots (2)$$

It is assumed that the derivatives of the energy and the velocity terms in the momentum equation (Eqn. 2) are very small in comparison to gravity and frictional forces allowing to assume that S is approximately equal to S_f. Under these conditions the discharge can be described as the function of flow area only in the following form,

$$Q=\alpha A^{m} \qquad \ldots (3)$$

Here α and m are known as KW routing parameters which are directly and closely related to catchment flow characteristics. Eqns. (1)&(3), when considered together are termed as KW equation and may be written in its general form as given below,

$$\frac{\partial A}{\partial t}+\alpha m A^{m-1}\frac{\partial A}{\partial x}=q \qquad \ldots (4)$$

Physiographic Configuration for KW Model

Application of KW equations requires certain simplifications of complex physiography of watershed. This is achieved by assuming suitable physiographic configuration to incorporate overland and channel phase of surface runoff. In the present study a simpler and most popularly adopted open book type physiographic model has been adopted. It involves a channel segment equal to that of main drainage channel of the catchment. The equal size rectangular overland plans have represented the two contributing catchments situated on either side of the channel, such that total area of planes equals catchment area. For channel configuration, trapezoidal Cross-Sections (CS) with realistic base widths (*B*) and side slopes (*Z*) are adopted.

Application of KW Model on Real Catchments

Modeling Overland Flows

Movement of water on the land surface implies to computation of flow rates at different points down the slopes. Consequently a uniform sheet flow has been considered as basis for development of overland flow equations. For a sheet flow on an overland plane of unit width the water flow area (*A*), and the hydraulic radius (*R*) can be replaced by *h*, yielding KW equations in the following form,

$$\frac{\partial h}{\partial t}+\frac{\partial q}{\partial x}=r-f=i_e \qquad \ldots (5)$$

$$q=\alpha_o\left(h\right)^{m_o} \qquad \ldots (6)$$

After combining Eqns. (5) & (6) the general form of KW equation will get transformed as under,

$$\frac{\partial h}{\partial t}+\alpha_o m_o\left(h\right)^{m_o-1}\frac{\partial h}{\partial x}=r-f=Ie \qquad \ldots (7)$$

The KW parameters α_o and m_o are obtained by using below given steady state form of Manning's roughness equation meant for computing q on unit width of overland planes,

$$q=\frac{1}{N}h^{5/3}S^{1/2}=\frac{1}{N}S^{1/2}(h^{5/3}) \qquad \ldots (8)$$

Here, Manning's *n* has been replaced by an appropriate coefficient *N*, which is termed as effective overland roughness of overland flows and incorporates multiple influences like raindrop impacts, obstacles on overland planes, tillage etc. Comparison of Eqns. (6) & (8) gives the value of KW parameters in terms of measurable physiographic factors. Accordingly α_o and

m_o can be written as $(S^{1/2}/N)$ and (5/3) respectively formulating KW equation as follows,

$$\frac{\partial h}{\partial t}+\frac{5}{3}\alpha_o h^{2/3}\frac{\partial h}{\partial x}=Ie \qquad \dots (9)$$

Here, h is the only dependent variable and can be easily worked out for its substitution in Eqn. (6) to compute q, which remains the input for channel flows.

Modeling Channel Flows

The ultimate forms of the governing equations for channel face of runoff were as follows,

$$\frac{\partial A}{\partial t}+m_c\alpha_c(A)^{m_c-1}\frac{\partial A}{\partial x}=q \qquad \dots (10)$$

$$Q=\alpha_c(A)^{m_c} \qquad \dots (11)$$

It has been established by previous researchers (DeVries and McArthur 1979) that values of m_c varies from 4/3 (triangular channel CS) to 5/3 (wide rectangular CS). However for present study it was considered appropriate, to apply the average values of KW parameters (α_c and m_c) which shall remain valid for the entire range of discharge passing through the adopted trapezoidal channel sections on study watershed.

Numerical Solutions of KW Equations

There are no direct solutions to unsteady partial differential equations described above. These were solved with the help of numerical solution techniques, where the solution advanced point by point from one time line to the next. Initial values and boundary conditions were utilized for solution of governing differential equations at successive nodal points. In the present study following two first order explicit numerical schemes were adopted,

(i) Scheme-I Forward in time and Backward in space
(ii) Scheme-II Backward in time and Forward in space.

These schemes have been successfully utilized by earlier researchers (Ponce, 1989, Hossain 1989; Gaur, 1999) and reported as always converging but conditionally stable. These two schemes are in fact the mirror image computational schemes whose discretization on finite difference grids of space and time is shown in Figure 1. Scheme-I was found stable under the situations where Courant number (i.e. ratio of average KW celerity, $\bar{c}$ and the grid celerity $(\Delta x/\Delta t)$, is less then or equal to one. Alternatively Scheme-II was stable for conditions where Courant number was exceeding one. While seeking numerical solutions for governing differential equations for overland as well as channel flows, the Courant number was computed first on each time step and depending upon its magnitude either Scheme-I or Scheme-II was applied through computer program written in advance version (1995) of FORTRAN language. It is considered out of scope to describe the details of the finite difference numerical formulation of the KW equations. The initial conditions (IC) and Boundary Conditions (BC) applied were as follows:

	Overland Flows	*Channel Flows*
IC	$h_{(x,0)}=0$ (for all x at $t=0$); $q_{(x,0)}=0$ (for all x at $t=0$);	$A_{(x,0)}=0$ (for all x at $t=0$) $Q_{(x,0)}=0$ (for all x at $t=0$)
BC	$h_{(0,t)}=0$ (for all t at $x=0$); $q_{(0,t)}=0$ (for all t at $x=0$);	$A_{(0,t)}=0$ (for all t at $x=0$) $Q_{(0,t)}=0$ (for all t at $x=0$)
KW Celerity	$\bar{c}=\alpha_o m_o\left\|\bar{h}\right\|^{m_o-1}$;	$\bar{c}=\alpha c_c m_c\left\|\bar{A}\right\|^{m_c-1}$

Computation of Temporally Varied Roughness (N_j) Through Unit Pulse Responses

In one of his study the authors (Gaur, 1999) reported that the KW model failed to produce the desired responses when the effective overland roughness were given a unique (i.e. constant) value through out the storm duration. Various aspects of temporal variations in the effective overland roughness during the storm period were reported in details. These optimized flow parameters were derived by adhering KW flow conditions keeping Frouds number values well within acceptable limits (2.0 or less) and also producing best matched simulation (at each time step as well as in totality) in regard to match it with synthesized SCS UHs under similar input rainfall condition. Further details in this regard are shown by Gaur and Mathur (2003), where effective overland roughness prediction equations were developed for variety of natural watersheds. All these relationships (i.e. the DRH based roughness prediction equations) turned out to be of the following mathematical shapes,

$$n_j=\alpha_1(h_j)^{\beta 1} \text{ for } h_j=0\rightarrow h_{\max} \qquad \dots (12)$$

$$n_j=\alpha_2(h_j)^{\beta 2} \text{ for } h_j=0\rightarrow \text{end point} \qquad \dots (13)$$

where, N_j is value of temporally varied effective overland roughness at j^{th} time step during the given storm period, h_j is mean overland flow depth in mm corresponding to j^{th} time step, α_1 & α_2 are coefficients for rising and depleting phases of overland flow depths, and β_1 & β_2 are exponents for rising and depleting phases of overland flow depths.

The Reynolds number (R_e = V.R./ν; R_e is Reynold number, V is mean flow velocity in m/Sec, R is hydraulic radius in meter, and ν is Kinematics viscosity of water in m^2/Sec, taken as 10^{-6} m^2/Sec) quantifies the relative significance of the inertia and viscosity and has been considered in present study while computing various flow computations though KW model applications. In case of overland flows on a unit width of overland plane, R equals to h (i.e. the overland flow depth) and accordingly on the basis of Reynolds number of overland flows efforts are made to compare the influences of different overland roughness conditions on it.

In case of completely ungauged watersheds with no flow records available for the entire region, there is no way out except to generate representative unit pulse responses (i.e. synthetic direct runoff hydrographs due to different rainfall excess depths of unit duration). Thus a unit hydrograph will be a particular case of a unit pulse response generated by say 10 mm of the rainfall excess function. It may be noted that these unit pulse responses are mostly computed by considering the rainfall excess and the generation of runoff as a linear function. It is assumed that the end product i.e. the unit response to be a satisfactory indicator of watershed behavior, the same has been used irrespective of the methodologies adopted for its derivation. In India following two methods (Snyder's Synthetic UH and SCS Synth UH) are widely adopted for generating synthetic unit hydrographs for the ungauged watersheds.

The SCS method better suits to the present concept, because it takes into account the different hydrological conditions which may be considered sufficiently appropriate for its use in estimation of effective overland roughnesses. Also in addition of this the SCS synthetic unit hydrograph encompasses several other positive features in caparison to the Snyder's synthetic unit hydrograph. Unlike the Snyder's method, the SCS method uses a constant ratio of actual time base to time to peak (i.e. T_b/T_p = 5), and also it uses a dimensionless hydrograph function to provide a standard unit hydrograph shape. Accordingly, in present study the study watershed (B719) is considered as ungauged and unit pulse response corresponding to 15 min unit duration have been worked using SCS method with following steps:

Step 1: Using 10 storm events and their daily rainfall and runoff amounts (i.e. depths) the runoff Curve Numbers (CN) were worked out through standard curve for estimation of CN from observed data as described by Ponce (1989). Accordingly value of average representative CN was computed for B719 watershed.

Step 2: Since the watershed is well within 16 Km^2 in area, the watershed lag is computed by using the below given formula,

$$T_l = \{L^{0.8}(2540\text{–}22.86\,CN)^{0.7}\}/\{14104\,(CN)^{0.70}\,S^{0.5}\} \quad \text{... (14)}$$

Where, T_l is watershed lag in hrs, L is hydraulic length (along main drainage channel) in meters, CN is runoff curve number, and S is *Av.* watershed slopes in m/m.

Step 3: Time to peak and the peak rate of runoff were computed by using the below given relationships,

$$T_p = (10/9)\,T_l \quad \text{... (15)}$$

$$Q_p = \{\,2.08\,A\}/T_p \quad \text{... (16)}$$

Where, T_p is the time to peak in hours, T_l is watershed lag in hours, Q_p is the peak rate of runoff in m^3/sec, and A is the watershed area in Sq. Km.

Step 4: After computing the Q_p and T_p, the standard dimensionless unit hydrograph was used to compute the synthetic unit hydrograph ordinates. In SCS method the ratio of time to peak and the unit duration of the unit hydrograph are fixed and given by following relationship in usual way,

$$T_p / T_r = 5 \quad \text{... (17)}$$

Where T_r is the unit duration of the unit hydrograph in hours. The real computations for deriving the SCS synthetic unit hydrograph on B719 watershed are illustrated in Table 2.

Step 5: Using the above SCS Synthetic unit hydrograph (10 mm, 15 min) seven other unit pulse hydrographs (different depths but same unit duration i.e. 15 min) were computed following Sherman's UH approach. These Unit pulse hydrographs were then used for deriving optimized flow parameters through computer programe described by Gaur and Mathur, (2003).

Table 2: Computations for Deriving the SCS Synthetic Unit Hydrograph on B719 Watershed

<table>
<tr><th colspan="4">Average Curve Number (CN)</th><th>Computation of T_l, T_p, Q_p, and T_r</th></tr>
<tr><td>Date</td><td>P inches</td><td>Q inches</td><td>Runoff CN Using Standard Curve Figure*</td><td rowspan="10">Given, $L = 7200$ m; $S = 0.068$ m/m; $CN = 88$
Using Eqn. (14), Tl for watershed is computed as follows,
$T_l = \{(L)^{0.8}(2540–22.86\text{CN})^{0.7}\}/\{14104\ (CN)^{0.7}(S)^{0.5}\}$
= 1.162 hours or 69.71 minutes
Now using Eqn. (15) and Eqn. (16), the Tp and Qp may be worked out as follows while substituting the value of Tl and area A,
$T_p = 1.291$ hours; $Q_p = 22.556\ \text{m}^3/\text{sec}$
As per Eqn. (4) Unit duration of SCS synthetic UH,
$T_r = t_p/5 = 0.2582$ hrs say 15 min
Further using values of Qp, Tp and the dimensionless S-Hydrograph function the ordinates of representative SCS synthetic UH were computed as given in Table 3</td></tr>
<tr><td>25–7–64</td><td>1.02</td><td>0.69</td><td>95</td></tr>
<tr><td>27–7–64</td><td>1.30</td><td>1.08</td><td>95</td></tr>
<tr><td>3–9–64</td><td>0.74</td><td>0.34</td><td>90</td></tr>
<tr><td>4–8–65</td><td>1.81</td><td>1.03</td><td>90</td></tr>
<tr><td>18–9–65</td><td>0.94</td><td>0.22</td><td>85</td></tr>
<tr><td>6–7–66</td><td>1.13</td><td>0.24</td><td>80</td></tr>
<tr><td>16–9–66</td><td>1.78</td><td>0.75</td><td>85</td></tr>
<tr><td>20–9–66</td><td>1.3</td><td>0.22</td><td>80</td></tr>
<tr><td colspan="4">Average $CN = 87.5$ (say 88)</td></tr>
</table>

(P = Daily Rainfall; Q = Daily Runoff Depth; L = Length; S = Slope; * Ref. Ponce (1989))

Table 3: Computed Ordinates of 15 min 10 mm SCS Synthetic UH for B719 Watershed

Time (min.)	*Time (hours)*	*T/Tp*	*Q/Qp*	*Q (mm/hr)*
0	0	0	0	0
15	0.25	0.2	0.1	0.55
30	0.5	0.4	0.31	1.77
45	0.75	0.6	0.66	3.8
60	1	0.8	0.93	5.37
75	1.25	1	1	5.77
90	1.5	1.2	0.93	5.37
105	1.75	1.4	0.78	4.49
120	2	1.5	0.67	3.86
135	2.25	1.7	0.466	2.67
150	2.5	1.9	0.329	1.88
165	2.75	2.1	0.24	1.36
180	3	2.3	0.175	0.98
195	3.25	2.5	0.125	0.69
210	3.5	2.7	0.091	0.5
225	3.75	2.9	0.065	0.35
240	4	3.1	0.047	0.24
255	4.25	3.3	0.034	0.17
270	4.5	3.5	0.025	0.12
285	4.75	3.7	0.018	0.07
300	5	3.9	0.013	0.04
315	5.25	4.1	0.01	0.03
330	5.5	4.3	0.009	0.02
345	5.75	4.5	0.005	0.01
360	6	4.6	0.003	0

RESULTS AND DISCUSSION

The unit pulse responses corresponding to seven different rainfall excess depths starting from 0.50 mm to 10 mm over 15 minute duration are shown in Figure 1(b). These ranges of the rainfall excess rates were realistic, keeping in view the prevailing range of rainfall excess depths recorded over this particular watershed (1–40 mm/hr). Authors (Gaur, 1999; Gaur and Mathur, 2003) have synthesized computer programs to run KW model for estimation of time

Table 4: Ranges of Variations in Computed Flow Parameters for Observed DRHs on B719

Storm Date	*Optimized Overland Flow Parameters (Ranges within the Storm Duration)*					
	N_j	*h_j*	*v_j*	*R_e*	*Froud No*	*KW No. (10000)*
24–7–64	.007–.353	0.1–13.65	.051–.46	2–1753	.12–2.0	1.01–33.10
3–9–64	.006–.28	0.1–7.50	.047–.337	2–811	.13–2.0	1.40–43.35
11–8–65	.01–.72	0.1–24.81	.027–.519	10–2880	.06–2.0	0.79–93.76
18–9–65	.007–.393	0.1–5.41	.022–.33	2–724	.09–2.0	2.452–147.44
6–7–66	.006–.135	0.1–5.80	.05–.354	4–830	.26–2.0	2.017–24.79
16–9–66	.007–.458	0.1–18.12	.04–0.41	9–2909	.09–2.0	1.52–39.45
20–9–66	.006–.169	0.1–5.43	.04–.389	4–829	.21–2.0	1.536–45.65

varied nature of effective overland roughness parameter over variety of natural watersheds utilizing their observed event based rainfall runoff records. Same computer program was utilized herein to compute values of the time varied effective overland roughnesses corresponding different ordinates. The ranges of these variations (as computed by real observed DRHs) are reflected in Table 4, were quite wider in contrast to similar values derived and described for purely ungauged situation where certain synthetic SCS UHs are utilized for the same watershed.

Time Varied Nature of Overland Roughness and Associated Flow Parameters

Utilizing the methodologies (Gaur, 1999; Gaur and Mathur, 2003) the variations in the flow parameters in real time domain were computed for the set of synthetic SCS UHs and are shown in Figure 2. The temporal variations are clear and can be better understood through various parts of the said Figure (2a through 2b), where temporal variations (i.e. within the storm duration) in the values of the following overland flow parameters are shown encompassing two representative (randomly selected) Synthetic SCS UHs adopted in study:

- Overland flow depths v/s time
- Overland flow velocities v/s time
- Overland flow Reynolds numbers v/s time
- Effective overland roughness v/s time.

The effective overland flow depths (h_j) and the corresponding effective overland roughness (N_j), produced hysterisis loops, suggesting different trends of relationships between them for rising and recession periods of overland flow depths. Distinct relationship in the shape of power functions were found to exist in between h_j and N_j for rising as well as depleting segments of overland flow depth profiles. The values of effective roughnesses were found to be increasing with increased overland flow depths. This was opposite to what reported by some previous researchers in case of channel flows. The possible reason may be the very low flow depth encountered during overland flows in comparison to channel flows. The values of effective overland roughness were generally higher during the period when there is input rainfall excess, as compare to values during non-rainfall excess periods. Additionally the trend of temporal variations in effective overland roughnesses were of more unevenness during rainfall excess periods. While the roughness values during non-rainfall excess period showed smoother trends. This altogether depicted the effect of rainfall impacts on effective roughnesses.

The real time variations of overland flow parameters viz. flow depths, velocities, Reynolds number, effective overland roughness etc. suggested somewhat similar patterns over all the events adopted in the present research work.

As evident from results shown in Figure 2, considerable temporal variations were encountered in these parameters within an individual storm of a certain duration. Overland flow depths keeps rising till the end of rainfall excess and then subsequently declined. So long the overland flow depth profile keeps developing the overland flow velocities had comparatively small values. After the end of rainfall excess the overland flow depths depleted rapidly with increased flow velocities. A final declining trend in flow velocities were seen only near the end portion of the runoff hydrographs.

Results showed that the maximum value of overland flow Reynolds number is mostly attained when peak rate of runoff reaches. The decline in Reynolds number values corresponding to recession part of the hydrographs showed more or less uniform rate. The maximum roughness attains at a time corresponding to close vicinity of the peak overland flow depth at which Reynolds number need not be or generally has not attained the peak value. Sudden increase in Reynolds number value on the rising limb of the overland flow depth profile may be due to concentrated overland flows as well as increases in the overland flow velocities.

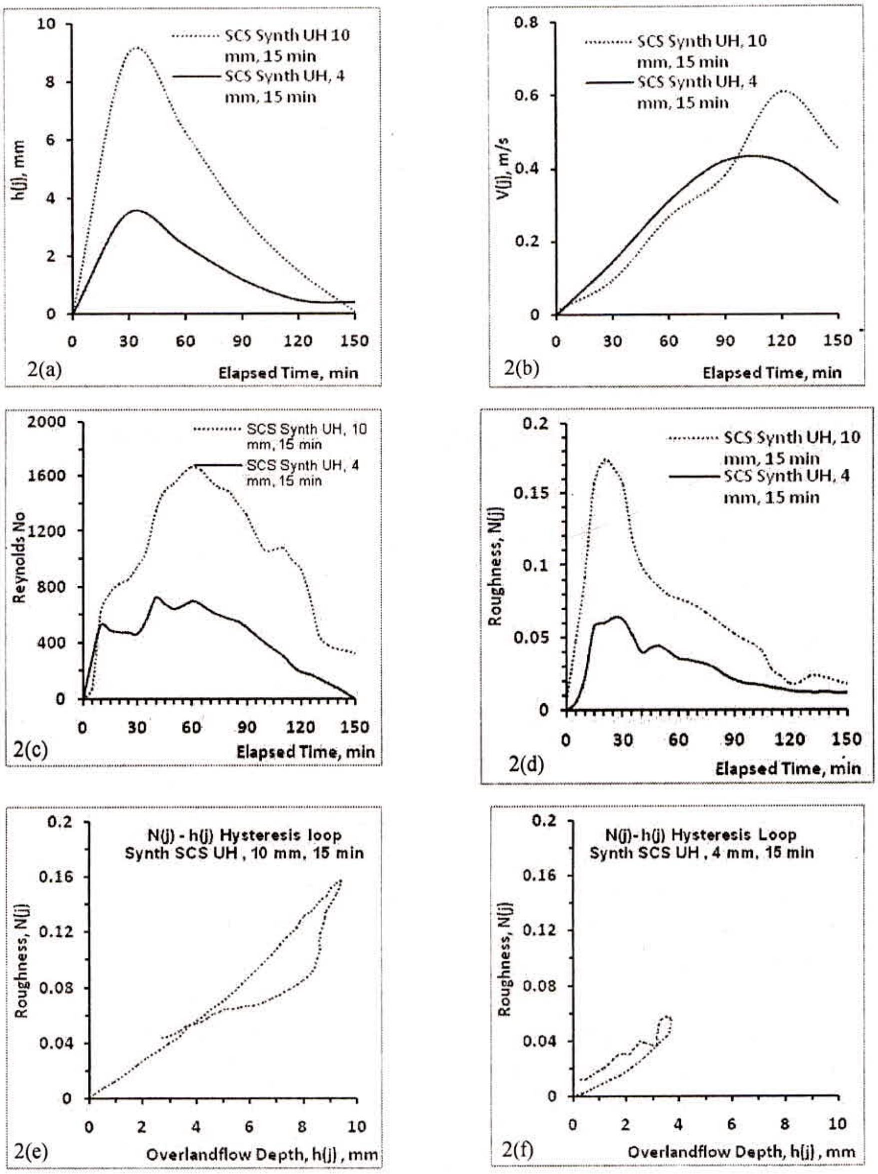

Fig. 2: Time Varied Nature of Overland Roughness and Associated Flow Parameters

Predictive Roughness Equations

The relationship between the overland flow depth and the effective overland roughness in the form of $h_j - N_j$ hysterisis loops, is synthesized as visible in Figure 2 (E&F). In order to develop a regression relationship for the time period $0 \rightarrow h_{\max}$, the seven sets of values belonging to overland flow depth and their corresponding effective overland roughnesses (i.e. from $0 \rightarrow h_{\max}$) were pooled together. Its plot is shown in Figure 3a. A linear regression model fitted best and the relationship is given as under,

$$N_j = -0.01 + 0.01798\, h_j \qquad \ldots (18)$$

Similarly all the seven sets of $h_j - N_j$ values for the recession portion of the overland flow depths were pooled and their plot is shown in Figure 3b. The linear best fit for regression equation is given as under,

$$N_j = 0.0076 + 0.0125\, h_j \qquad \ldots (19)$$

The above mentioned two equations have been successfully used to simulate the direct runoff hydrographs under various given input rainfall conditions. KW physiographic model (Gaur and Mathur, 2003) incorporating parameters described in Table 1 were used for the application of KW theory with self prediction of time varied values of effective roughness parameter (*Nj*). Above described predictive equations (Eqns. 18 & 19) were found quite effective to deliver acceptable simulations.

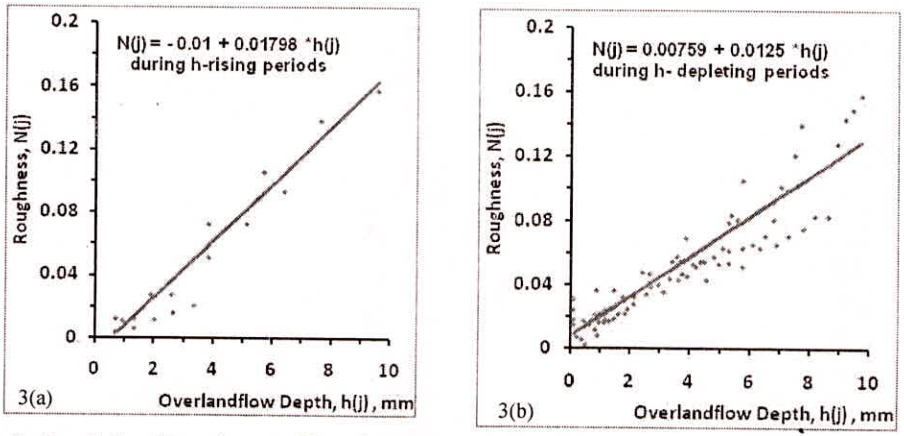

Fig. 3: Predictive Roughness Equations Based on SCS Synthetic UHs of Different Depths

Simulation Results

The study watershed have some observed rainfall runoff records, and it was studied was examined by author (Gaur and Mathur, 2003) to develop time varied roughness predictive equations based on real DRH ascertained at the watershed. Figure 4 shows the composite comparison of the real observed hydrographs with the simulated ones i.e. using the DRH based roughness prediction equations as well as with the synthetic UH based roughness prediction equations (Eqns. 18 & 19). It may be noted that the SCS synthetic unit response based relationships have provided quite satisfactory answers. Results illustrated in Figure 4 have proved that in a completely ungagued situation the SCS synthetic unit hydrograph may be an alternate choice to develop simpler roughness prediction equations which were found capable enough to accurately simulate many direct runoff hydrographs on a given watershed.

Above comparisons indicate that the simulated peak flows were in close proximity with the observed ones. However, the time to peaks for the simulated runoff differed to a considerable extent in few cases. When minutely compared with observed records, the KW simulations through synthetic SCS UH based roughness prediction equations gave a bit higher (but acceptable) values of peak runoff rates in general, in contrast to DRH based roughness equations. The visual comparison of the simulated and observed hydrographs (Figure 4) accounting the temporal variations in the overland roughnesses reveal that except in a few cases the comparison was very good and the simulated hydrographs matched reasonably well with the observed ones. This altogether has clearly vindicated the postulation of temporally varied overland roughness and its close relationship with the overland flow depths.

Beside the visual comparison of the simulated and observed hydrographs the numerical comparisons of simulated results are shown in Table 5, which is self explanatory to show the acceptability of end results. The key hydrograph parameters viz. the peak discharge rates (*Qp*) and, time to occurrence of peak flows (*Tp*) are simultaneously compared in above cited table. Results are self explanatory to prove the validity of approach adopted in the present study.

Table 5: Numerical Comparison of Observed and Simulated Hydrograph Key Parameters

Storm Dates	*Observed*		*Simulated Using Synthetic SCS UH Based Nj Eqns*		*Simulated Using Real DRH Based Nj Eqns*	
	Qp	*Tp*	*Qp*	*Tp*	*Qp*	*Tp*
24–07–64	6.12	180	6.2	165	5.54	195
03–09–64	2.65	180	3.27	150	2.95	180
11–08–65	9.90	150	9.4	135	7.98	165
18–09–66	2.39	120	2.59	90	2.06	120
06–07–66	2.64	120	2.78	105	2.29	120
20–09–66	2.66	90	2.65	90	2.15	120

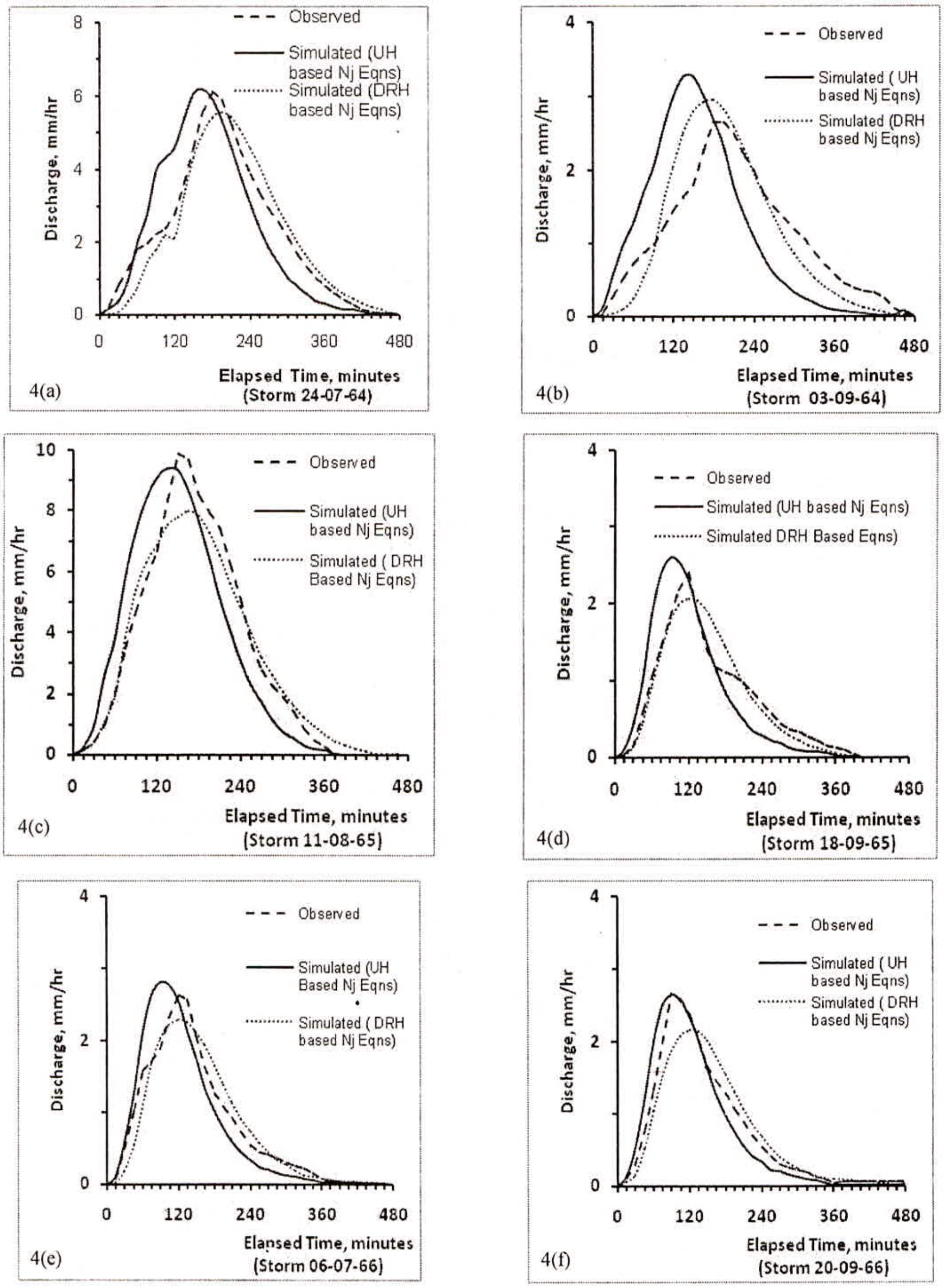

Fig. 4: Comparison of Simulation Performances of Roughness Predictive Equations Synthesized by Using Synthetic SCS UHs as well as Observed DRHs on B719 Watershed

CONCLUSION

For overland flow modeling we need basically three types of data (i) excess rainfall rates, (ii) effective roughness characteristics which indeed is governed by vegetation or the land use patterns, and (iii) watershed physiographic parameters. Success of any overland flow modeling depends upon the precision with which these three groups of input information are estimated. Out of these the effective roughness happens to be the weakest and most unexplored filed of research. Therefore the present study concentrated more towards the hydraulic resistance to flows offered by different soil-vegetation-cover complexes on variety of natural small watersheds in Indian tropical situations. The main objective was to study the temporal variation of effective overland roughness parameter used in KW models for simulating watershed runoff responses. It involved development of roughness prediction equations utilizing the synthetic SCS UHs on a 1400 ha natural watershed considered it as purely ungauged. The concept of temporally varied overland roughness has been successfully demonstrated for ungauged watershed situations by developing suitable roughness prediction equations and verifying them for accurate direct runoff simulations.

Results revealed that there exists a sound correlationship in between the effective overland roughness and the corresponding mean overland flow depths during storm duration. Computed values of overland roughnesses as they vary with the overland flow depths for different storm events are generated for the rising portions of h_j as well as during depletion of h_j. The plots of h_j *v/s* N_j are shown which conceded an interesting curves forming hysterisis loops showing vast non-linearity in their relationship for different time segments.

In above cited hysterisis loops, for the same overland flow depth two different values of roughnesses are obtained. One corresponds to the rising limb, up to the attainment of the maximum overland flow depth $h_{\max}$ where as the other one corresponds to the depleting overland flow depths. It advocates that the prevailing method of assigning a single unique value of Manning's roughness coefficient in KW modeling approach needs to be discouraged and the distributed nature of this effective roughness parameter is to be researched more for betterment of hydrological modeling on natural watersheds. The estimation of temporally varied effective overland roughness through the approach studied herein needs to be replicated over multi-locational watersheds and more generalized prediction equations may be developed for estimation of time distributed roughnesses for different regions.In future, more generalized roughness prediction equations may be attempted by incorporating area specific crops, land use systems and watershed physiographic factors. It can be achieved by considering more number of watersheds for application of the hyptothesis conceived in this research work.

The findings of present study offers a food for thought for future researches to seek certain simple and alternative methods, keeping in view the thought provoking statements by renowned researchers (like Woolhiser, 1996) that "*Complex models may give equally bad answers as that of simpler models, but at higher costs*".

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