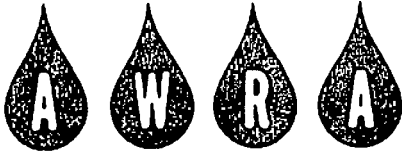


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American Water Resources Association

Water Resources Bulletin

VOL. 15, NO. 4, AUGUST 1979

Water Resources Bulletin

AMERICAN WATER RESOURCES ASSOCIATION

Volume 15

August 1979

Number 4

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AN APPLICATION RUNOFF MODEL STRATEGY FOR
UNGAGED WATERSHEDS¹*Donald L. Chery, Jr., Calvin G. Clyde, and Roger E. Smith²*

ABSTRACT: Mathematical models for predicting watershed surface flow responses are available, most of which are elaborate nonlinear numerical surface and channel flow models linked with infiltration models. Such models may be used to make predictions for ungaged areas, assuming an acceptable fitting of the model to the topography and roughness of the real system. For some application purposes, these models are impractical because of their complexity and expensive computer solutions. A procedure is developed that uses a complex model of an ungaged area to derive a simpler parametric nonlinear system model for repetitious simulation with input sequences. The predicted flow outputs are obtained with the simpler model at significant savings of money and time. The procedures for constructing a complex kinematic model of a 40 acre (161,880 m²) reference watershed and deriving the simpler system model are outlined. The results of predictions from both models are compared with a selected set of measured events, all having essentially the same initial conditions. Peak discharges ranged from 3 to 118 ft³/sec (0.085 to 3.34 m³/sec), which includes the largest event of record. The inherent limitations of lumped systems models are demonstrated, including the bias caused by their inability to model infiltration losses after rainfall ceases. Computer costs and times for the models were compared. The derived simple model has a cost advantage when repeated use of a model is required. Such an applications hydrologic model has an engineering tradeoff of reduced accuracy, and lumping bias, but is more economical for certain design purposes. (KEY TERMS: analysis; application; computer; costs; design; economics; flow hydrologic; kinematic; mathematical; models; prediction; runoff; system; ungaged; watershed.)

INTRODUCTION

Deterministic models of the hydrologic system may be divided into two general categories: physically-based models and system models. Physically-based models are those for which the transformations of input to output are derived from models of the physical processes within the watershed system. Input-output (I-O) system models employ a transfer relation derived from a sequence of inputs and outputs for the particular watershed. Models of each group have particular advantages.

In general, the physically-based, deterministic models better represent the structure of the real system. Thus, if one has confidence in the physically-based, deterministic model,

¹Paper No. 78094 of the *Water Resources Bulletin*. Discussions are open until April 1, 1980.

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it may be used to make performance forecasts for a watershed system (for which there is no sampling of the output) by measuring appropriate variables, like soil properties, surface slopes, channel dimensions, and hydraulic roughness within that system.

The I-O system models are empirically developed for a particular system by measuring the input (I) and output (O) of that particular system. They are, relatively, much simpler than the physically based models and may be used efficiently and inexpensively to predict the behavior of the particular system for which they were developed.

Deterministic surface water models have progressed from modeling and refining models of subunits of this system, to combining these subunits into more comprehensive models. Considerable effort has been directed toward the improvement of these comprehensive models, building confidence in their representation of the hydrologic system. In the process, these models have become complex and thus more difficult and expensive to use.

An applications role for a complex physical model is proposed and investigated in this study. Given an ungaged watershed the possibility of using a complex model as an intermediate step to develop parameters for a simplified model is investigated. If the simplification can be accomplished preserving a desired accuracy, then the simpler and more economical (saving both time and money) system model may be used operationally with a full spectrum of inputs for predictive purposes.

The overall scheme of this study is illustrated in Figure 1. The objective is representation of the real watershed system. The variable distributed rainfall of the real system may be sampled at several points. This greatly reduced, but still distributed, representation of rainfall is used as the input to the deterministic model. Rainfall excess is determined by an infiltration routine. The generated excess is then routed over infiltrating planes and through a channel network with provision for losses (channel infiltration) or gains in flow to produce a predicted output (q_k) that is compared with the measuring flow (q_m) to evaluate the model performance.

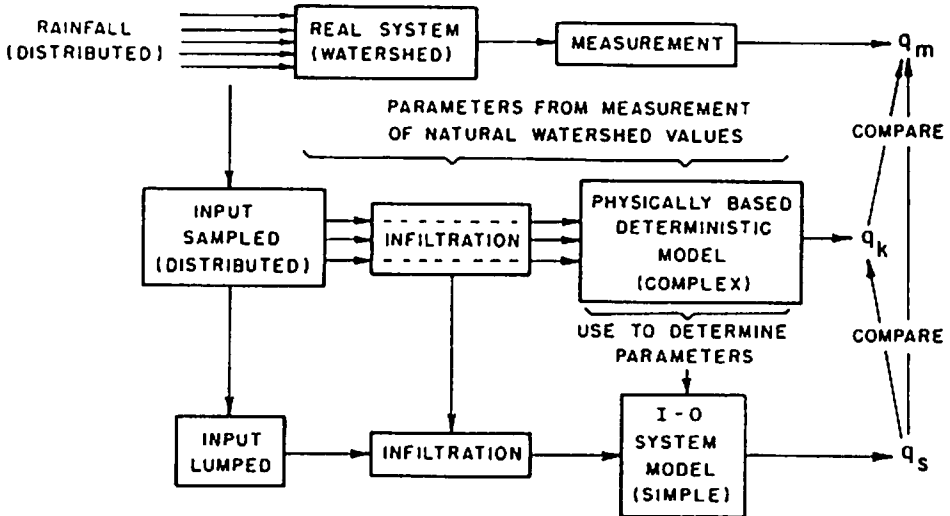


Figure 1. Scheme of Study.

The input for the I-O model is further reduced by lumping and is then processed by the same infiltration routine used for the deterministic model. The excess is convoluted with an impulse or unit pulse function (the parameters of which were derived from operation of the complex model), to produce a predicted output (q_s). The output of the system model is compared both with the output of the complex model and that of the measured real system output to evaluate its performance.

PROCEDURE FOR DEVELOPING THE APPLICATIONS MODEL

Review of the Kinematic Surface Water Model Development

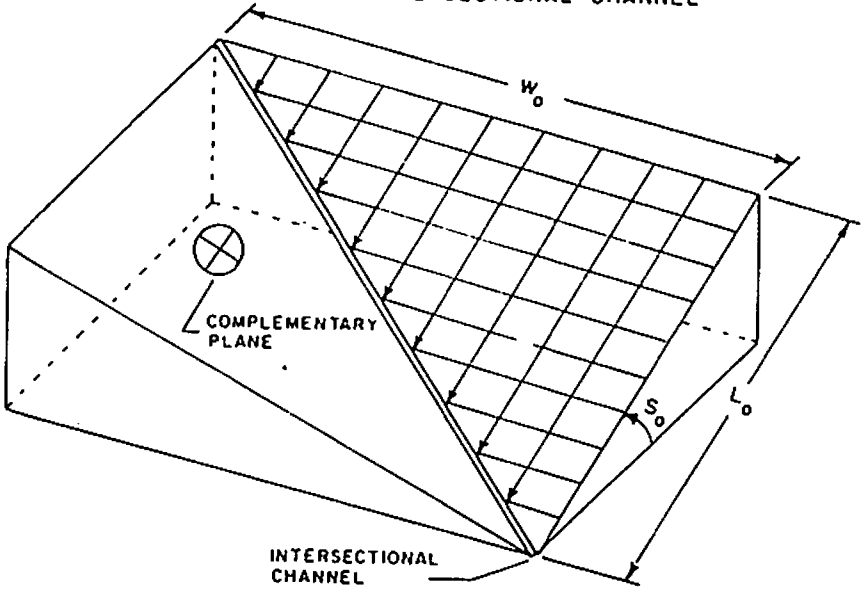
The deterministic modeling of surface water flow follows Kibler and Woolhiser's (1970) aptly described "reductionist" approach. The complex topography of natural watersheds is represented by a combination of simple planes contributing flow to channel lengths with simple cross sections. The work of many researchers, Brakensiek (1966 and 1967); Chen and Chow (1968); Eagleson (1969); Grace and Eagleson (1965); Henderson and Wooding (1964); Kibler and Woolhiser (1970), Liggett and Woolhiser (1967a and 1967b); Lighthill and Whitham (1955); Maddaus and Eagleson (1969); Overton and Brakensiek (1970); Strelkoff (1970); Wei and Larson (1971); Wooding (1965a and 1965b); Woolhiser, Hanson, and Kuhlman (1970); and Woolhiser and Liggett (1967) has contributed to selecting reductions of the complete shallow-water equations and to restricting criteria, which have led to the acceptance of the kinematic wave equations as an "adequate" model of shallow overland water flow and flow in channels. Kinematic is the term used for flow that can be described by the equation of continuity of mass and a stage discharge relation, like the Chezy or Manning friction formula.

A kinematic model is the complex physically based model used in this study. It has two features that contribute to a better representation of the real system -- continuously infiltrating planes and channels, and a transitional friction relation. The infiltration component was derived from work of Smith and Woolhiser (1971), Smith (1972), and Smith and Chery (1973). The transitional friction relation developed from works of Morgali and Linsley (1965); Morgali (1970); and Woolhiser, Hanson, and Kuhlman (1970).

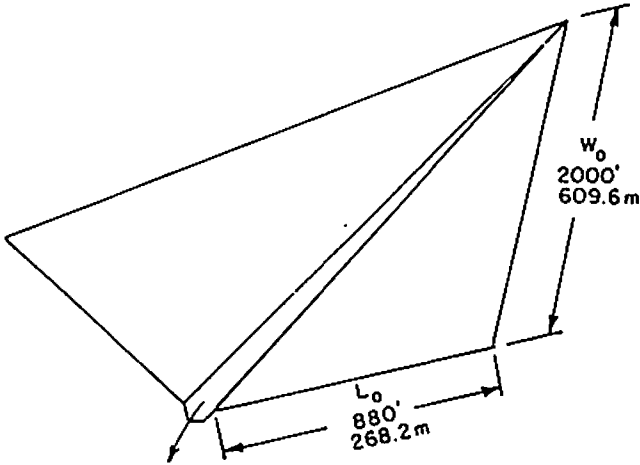
Description of Intermediate Kinematic Model

The intermediate physically based mathematical model used for this study was derived from the work discussed in the previous section. Flow from a triangular plane (Figure 2) was represented by calculating the longest length of overland flow (L_0 in Figure 2a) and then storing all the intermediate solutions made at the node of each equal length segment. These stored solutions are used later as the distributed triangular plane flow into the "intersectional" channel, illustrated in Figure 2a. The outflow from one triangular plane channel set can be doubled to give the response from the geometrical configuration as shown in the alternate configuration (Figure 2b). The program has the capabilities of calculating the dimensions of the complementary plane shown in Figure 2a and adding the distributed plane flow calculated for that plane to the distributed lateral input of the "intersectional" channel. The watershed model program makes flow calculations for "independent" channels, which may have uniform lateral input, and may accept input from one, two, or three upstream converging channels. The simple geometrical configuration of channels in the watershed model is illustrated in Figure 2c.

(a) TRIANGULAR PLANE AND INTERSECTIONAL CHANNEL



(b) ALTERATE TRIANGULAR PLANE ARRANGEMENT



(c) GENERAL CHANNEL CROSS SECTION

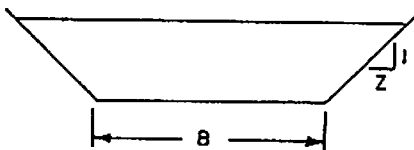


Figure 2. Geometrical Elements Used in the Physically Based Model.

A control program directs the solution to a plane or channel routine in a sequence governed by a set of coded instructions. The processing begins with reading the rainfall data and solving for the flow from the first plane. This solution is coded and stored for use in the solution of flow in channels between intersecting planes or channels on the edge of planes. Additional controls determine in which arrays the computed outflow is to be stored. Two main subroutines of the program are KINTRI and CHANNEL. The KINTRI subroutine makes the numerical kinematic solution of flow on a triangular plane. The CHANNEL subroutine solves for the flow in the defined channels by a kinematic numerical method. Details of the plane and channel flow equations were presented by Chery (1976).

Simple Derived Applications Model

For the simple applications model, a discrete nonlinear convolution model was employed (Chery, 1976). In terms of discrete time series in which the hydrologic data are sampled and manipulated in these analyses, the convolution can be expressed as a summation

$$y(nt) = \sum_{i=0}^n v(it) [U_p(t)]_i \tag{1}$$

in which $y(nt)$ = discrete time output; $v(it)$ = discrete time input; and $U_p(t)$ = unit pulse response for the specific discrete time interval used.

The process in which a different pulse response, $U_p(t)$ is used for each input excess rate or range of input rates is described as discrete linearization, and the resulting total model is described as "discrete nonlinear."

Differencing of the "S" hydrograph (Chow, 1964, p. 14-16) was used to derive the unit pulse response for a series of constant input (excess rates to the kinematic model. The constant excess rates are obtained rainfall input to the model having zero infiltration. They are applied until an equilibrium outflow is reached (Figure 3). A unit pulse response $U_p(t)$ is obtained by offsetting the equilibrium response of the system by the time interval of the desired unit pulse, and taking the difference between the two curves (6). These quantities are then normalized by dividing by the constant input rate and the area as shown in Equation (2):

$$\{U_p(t)\} = \frac{q_j - q(i - \Delta t)}{IA} \tag{2}$$

in which q_j = outflow rate; I = constant input rate; A = area of watershed; and Δt = time interval for which unit pulse is desired. The results of this operation are shown in Figure 4.

COMPLEX KINEMATIC MODEL OF A REAL WATERSHED

An evaluation of the overall scheme required a detailed kinematic model of a real watershed. The 40-acre (161,880 m²) Agricultural Research Service Watershed 47.002 (Figure 5) west of Albuquerque, New Mexico, was selected as the real reference watershed. Record collection for this watershed began in 1939 and continued to 1975. Good

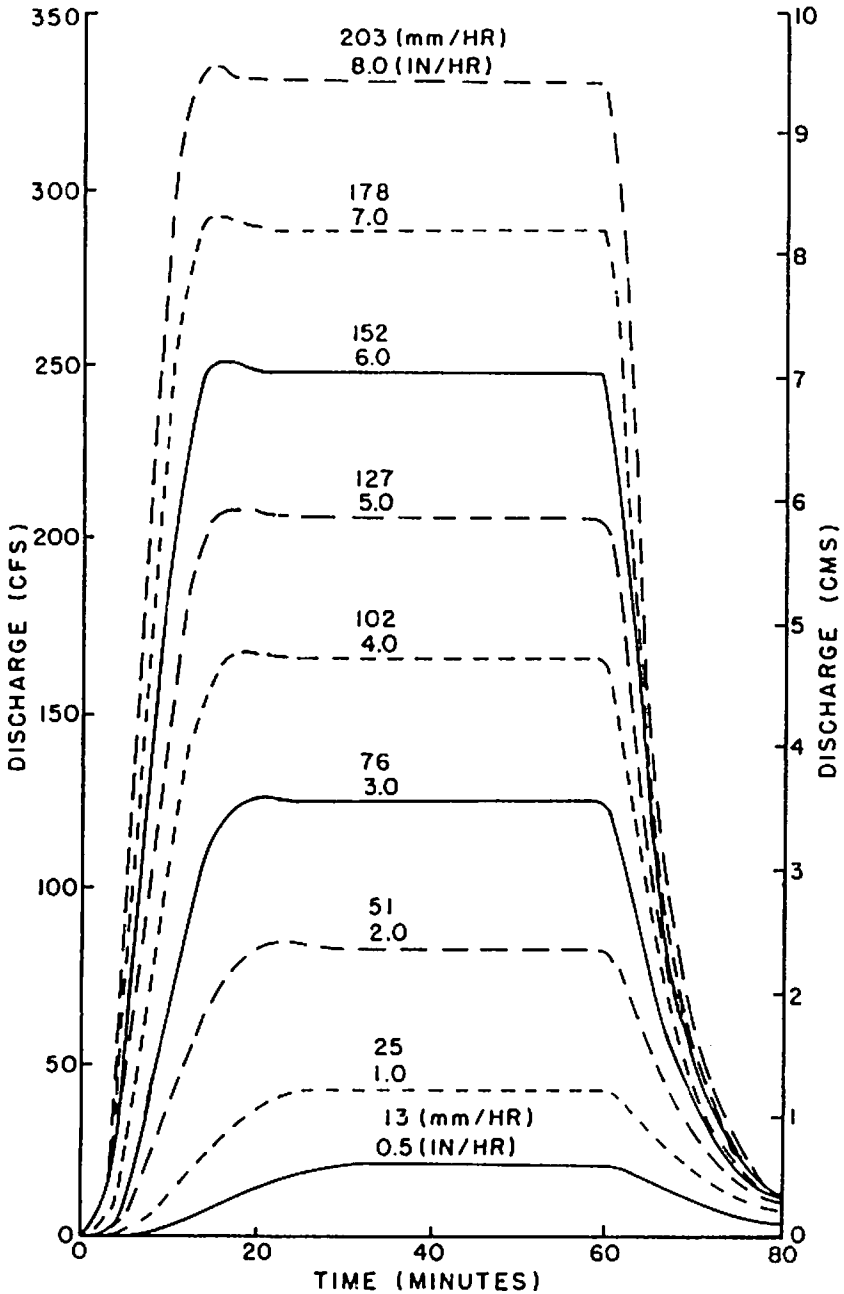


Figure 3. Responses of Kinematic Model (Triangular Planes, Figure 2b) With No Infiltration to Constant Input Rates.

measurements of rainfall and runoff were obtained for the entire period. The watershed shape is relatively simple and the soil rather uniform. Topographic details and locations of the two recording, weighing raingages are also shown on the map in Figure 5. The outflow

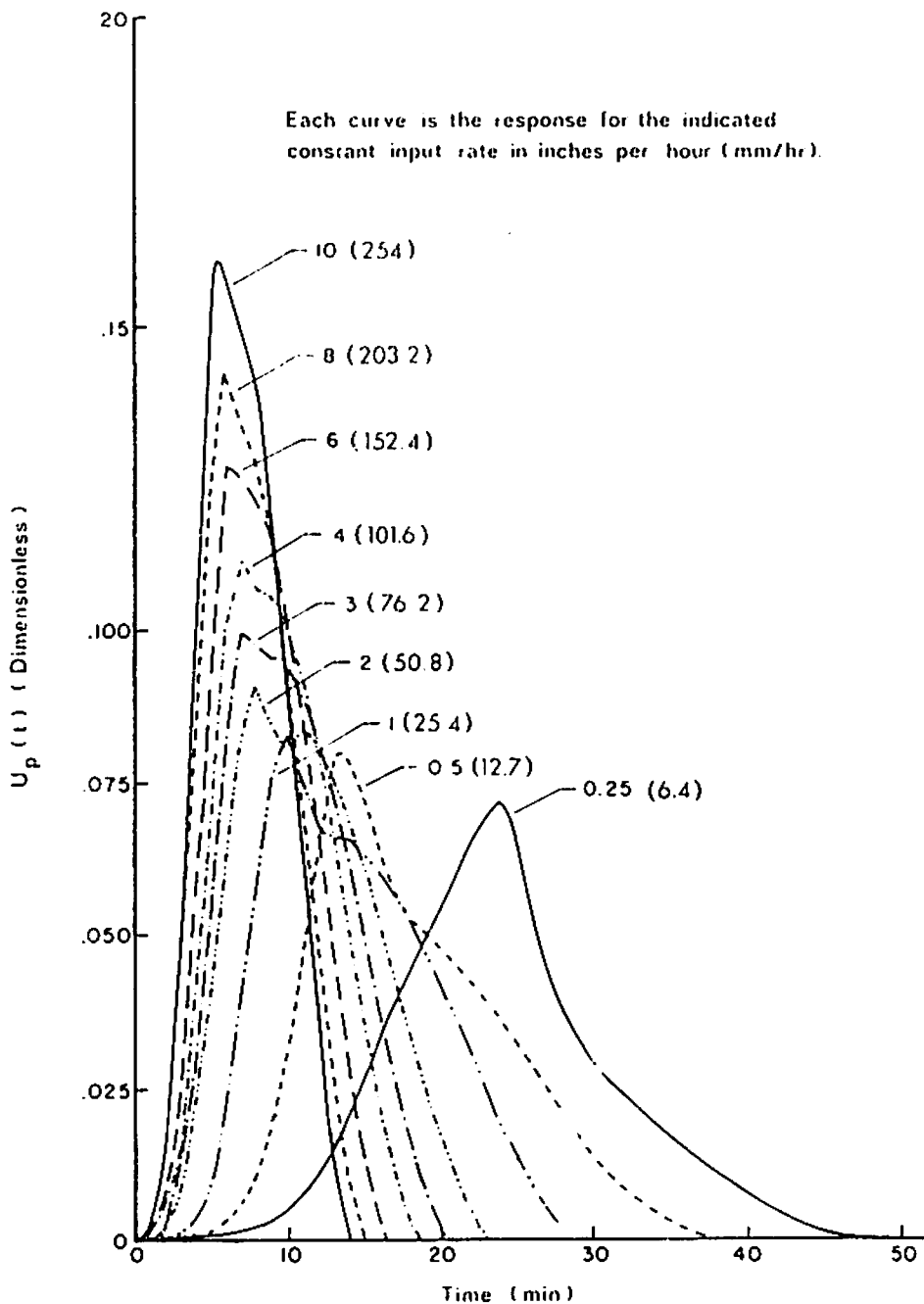


Figure 4. Derived One-Minute Pulse Responses for Triangular Planes (Figure 2b).

was measured by a 3:1 broadcrested V-notch weir. More details about the watershed were reported by Chery (1976), or may be obtained from a publication by the U. S. Department of Agriculture (1958). A previous analysis (Chery, 1972) made it possible to select runoff events having no rainfall within the previous 120 hours. Such cases were considered to have the same initial "dry" state, a necessity because a soil moisture accounting model was not readily available for this area.

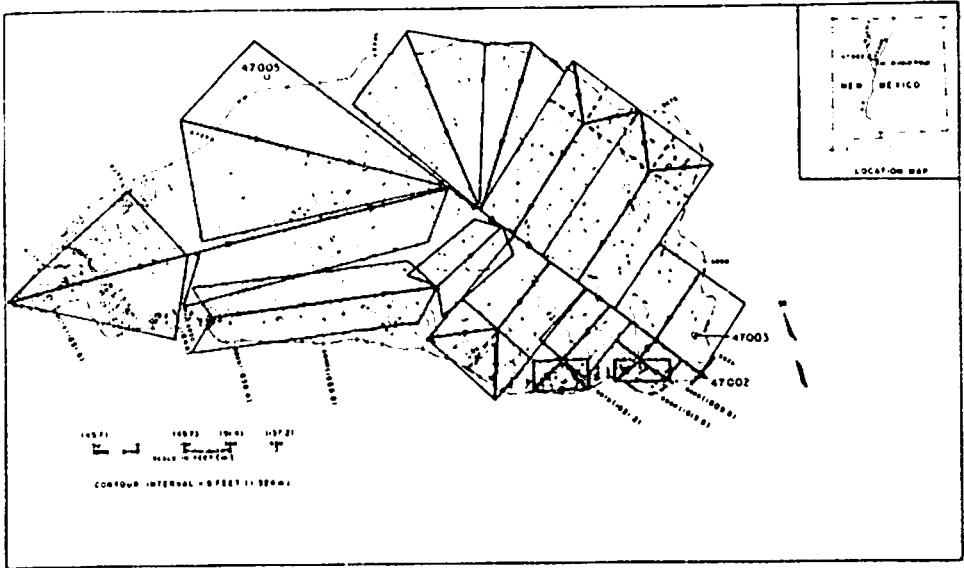


Figure 5. Topographic and Location Maps of Watershed 47.002, With Kinematic Model Planes Superimposed (1 foot = 0.3048 m).

A set of planes and channels shown superimposed on the map of Figure 5 and laid out in a sequential arrangement in Figure 6 is the designed geometrical representation of the reference watershed. The configuration of rectangular and triangular planes was selected as being sufficiently representative of the watershed topography, yet not too numerous as to make the kinematic model prohibitively expensive to operate. We realized that there is an interaction between the chosen geometry for a watershed and parameters of slope and roughness. This interaction was used in a study by Lane, Woolhiser, and Yevjevich (1975) that investigated the distortion of roughness to achieve a given model prediction accuracy as the model geometry is simplified. Some geometrical parameters of the constructed complex kinematic model and the reference watershed are compared in Table 1.

Besides establishing the geometry of the model, six infiltration and three friction (or roughness) parameters (two for planes, one for channels) are needed. The infiltration parameters are α , an exponent in the infiltration equation (Smith, 1972, p. 7):

$$f = f_{\infty} + \Lambda(t-t_0)^{-\alpha} \quad (3)$$

f_{∞} , the final infiltration rate; CIN, a coefficient in a relation for a normalizing time parameter (Smith, 1972, p. 15), which incorporates parameter A of Equation (3); SOIN, the maximum water content by volume; SIN, the initial water content by volume; and BIN, a coefficient in the functional relation between dimensionless soil water volume at time of ponding the dimensionless rainfall rate (Smith, 1972, p. 18).

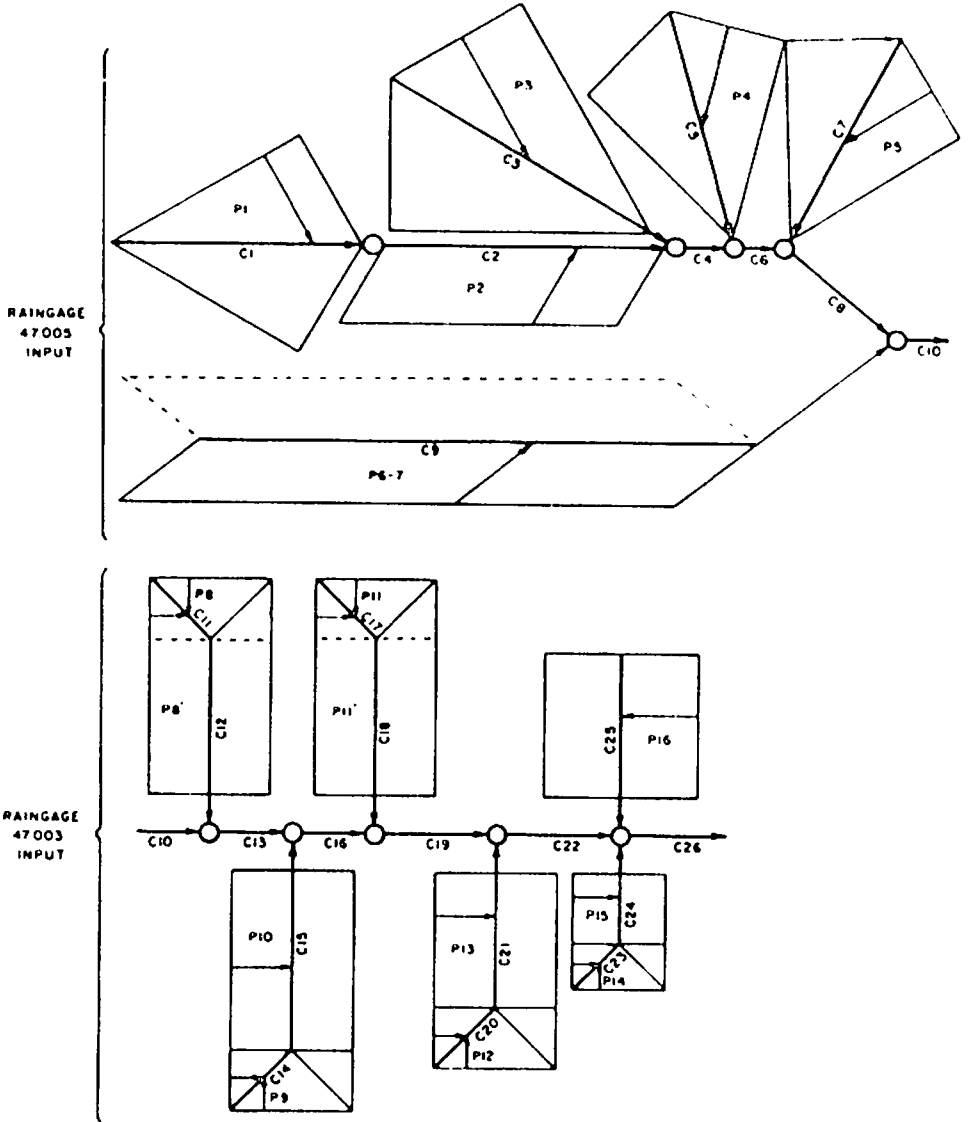


Figure 6. Assemblage of Planes and Channels for the Kinematic Model of Watershed 47.002 (P = Plane, C = Channel).

TABLE 1. Comparative Geometrical Dimensions of Kinematic Watershed Model and Reference Watershed.

Measurement (1)		Kinematic Model (2)	Dorroh and Kringold ^a (3)	Hickok and Rafferty ^b (4)
Total Area	(acres)	40.1	40.1	40.10
Average Overland Flow Length	(ft)	203.0		
Average Weighted Plane Slope (average landslope)	(%)	11.400	6.5 ^c	14.30
Range of Plane Slopes (range of landslope)	(%)	5-27.000	5-45.0	
Average Slope of Upper Half of Watershed	(%)	13.000 (Upper 60%)		16.40
Average Slope of Lower Half of Watershed	(%)	8.700 (Lower 40%)		11.90
Total Length of Channels (total length of waterways)	(ft)	8,059.8	20,600.0	
Drainage Density	(ft/acre)	204.000	509.0	86.00
Length of Principle Waterway (length of channel from outlet to end of main channel)	(ft)	2,612.000	1,635.0	1,640.0
Length of Chan. to Center of Gravity	(ft)	≈ 1,150.000		1,140.00
Length of Chan. to End of Watershed	(ft)	2,612.000		2,870.00
Main Channel Slope	(%)	4.000	4.6 ^d	4.12
Channel Slope to End of Channel	(%)	6.400		4.75
Channel Slope to End of Watershed	(%)	6.400		6.05
Range of Channel Slope	(%)	4-19.4		

NOTE: 1 acre = 4047 m², 1 ft. = 0.3048 m.

^aJ. Dorroh and D. B. Krimbold, Maps of Montano Watershed W-2 (7.002) with notations, 1941, map files, Southwest Watershed Research Center, USDA, ARS, Tucson, Arizona.

^bR. B. Hickok and B. R. Rafferty, Special eight report series on flood runoff studies, 1954, File Reports, Southwest Watershed Research Center, USDA, ARS, Tucson, Arizona.

^cPrevailing landslope.

^dChannel slope to center of gravity.

Infiltration parameters α , f_{∞} , CIN, and BIN were initially estimated from some soil sampling data published by the United States Department of Agriculture (1968) for the reference watershed. The values of these parameters were varied and several model-predicted discharges compared with those of the same measured event. From these analyses, it was concluded that the final infiltration rate, f_{∞} was not more than 0.08 in/hr (2.03 mm/hr). It was also determined that the normalizing time parameter (CIN) had values of 5000 to 6000 for early-season events (June or before), decreasing to 1000 to 2000 for July, August, and September events. For a given input rate, the model responses were little changed by a 10-fold variation of BIN values; so the originally

established value of $BIN = 0.62$ was used for this coefficient in all following tests. The initial estimate of 0.57 for α was satisfactory. "Dry" initial state for this soil was estimated as $SIN = 0.4$.

After the infiltration analysis, it was decided to avoid the influence of the dynamic soil properties by selecting the late-season events, $CIN = 1000$. The set of "dry," late-season runoff events with peak discharges greater than 1 cfs ($0.0283 \text{ m}^3/\text{sec}$) are listed in Table 2.

For the planes, the friction coefficient (F) and transitional Reynolds number (R_t) were estimated from a composite graph of friction coefficient versus Reynolds number prepared by Smith (1975). The friction coefficient at $R = 1$ was estimated to be 700 and R_t to be 200. R_t was fixed at 200 and the responses of the model compared with measured response for values of $F = 500, 700, 1000,$ and 1400 . The best fits were obtained with the initial value of $F = 700$. As shown in Figure 7, tests with distributed Chezy C (as listed in Table 1) match the measured response more closely than those with a constant channel friction coefficient of $C = 40$.

The model configuration, illustrated in Figure 6 with the following parameters, was deemed adequate for the comparative analysis of the selected set of 18 "dry" events: exponent in the infiltration equation, $\alpha = 0.57$; final infiltration rate, $f_\infty = 0.08 \text{ in/hr}$ (2.03 mm/hr); normalizing time parameter, $CIN = 1000$, coefficient $BIN = 0.62$; initial soil water content by volume, $SIN = 0.40$; maximum water content by volume, $SOIN = 0.85$; plane friction coefficient, $F = 700$; transitional Reynolds number, $R_t = 200$; distributed Chezy C for channels, $C = 10$ to 40 ; and time increment, $\Delta t = 0.5$ minute.

The set of 18 events is a rigorous test of the model, spanning ranges of responses from 2.7 to 118.0 cfs (0.076 to $3.341 \text{ m}^3/\text{sec}$) peak discharge and total event volumes of 0.0165 to 0.7695 inch (0.4 to 19.5 mm).

DERIVATION OF SIMPLE SYSTEM MODEL FROM COMPLEX KINEMATIC MODEL

Pulse responses for nine constant excess rates were obtained from the kinematic model by the method of differencing the S hydrograph. The results of this procedure are shown in Figure 8. When the derived pulse responses (Figure 8a) were convoluted with the excess rate for event No. 4 (Table 2), a large secondary peak was generated (highest peak in Figure 9). The evaluation of runoff responses from event Nos. 4 and 16, plus facts learned in the development of the discrete nonlinear model and the results of a special investigation (Chezy, 1976) showed the secondary peak response to be from planes 1, 3, and 5 of Figure 6. This information contributed to the preparation of modified single-peaked unit pulse responses (Figure 8b) that had the second peak removed. These studies indicated that the upper north side of the constructed kinematic model should be altered by either reducing flow lengths or roughness or both to produce a more rapid response from this area. The influence of the long lengths of overland flow in planes 3, 4, and 5 of Figure 6 was checked by reversing the dimension of the triangles. Maximum flow lengths were reduced from 800, 420, and 420 to 380, 280, and 150 feet. The overall influence of this change is shown in Figure 10. For the complex model with the dimensions of planes 3, 4, and 5 reversed, the rates on the rising side of the hydrograph are increased with a higher peak occurring one minute earlier, followed with less recessional rates as compared with the original hydrograph. A comparison of the derived one-minute unit pulses is shown in Figure 11. For the input rates of 10 and 4 inches per hour, the secondary response is

TABLE 2. Peak Discharge and Total Volume Comparison Between Complex Kinematic Model and Measured Events.

Event ID No.	Date ^a	Measured Event		Kinematic Model Responses				Maximum Rainfall Excess Rate		Computer Run Time In Seconds (11)
		Peak Discharge In Cubic Feet Per Second (3)	Volume In Inches (4)	Predicted Peak Discharge In Cubic Feet Per Second (5)	% Difference (6)	Predicted Volume In Inches (7)	% Difference (8)	In Feet Per Second $\times 10^{-7}$		
								P47.003 (9)	P47.005 (10)	
1	240857 ^b	118.0	0.7695	94.2	-20	0.7636000	-0.8	1152	1152	244
2	040947	97.7	0.5330	104.5	7	0.6418000	20.0	842	1117	256
3	220769	84.1	0.3818	116.2	38	0.5925000	55.0	1355	1572	159
4	030864	46.2	0.4013	52.2	13	0.4273000	6.5	635	627	194
5	270966	36.8	0.2522	47.4	22	0.3062000	21.4	615	719	162
6	081054	34.1	0.3107	13.5	-60	0.0967000	-68.9	348	159	142
7	150859	22.6	0.2445	17.0	-25	0.1956000	-20.0	139	420	250
8	110954	22.6	0.2148	11.1	-51	0.0959000	-55.0	626	72	178
9	051058	17.4 ^E	0.1034	17.0	-2	0.1254000	21.0	502	715	129
10	280739	16.0	0.1048	19.3	21	0.1592000	52.0	963	755	134
11	010866	13.4	0.1064	18.5	33	0.1656000	56.0	784	323	219
12	120759	12.3	0.0856	24.5	99	0.1686000	97.0	303	411	117
13	040967	8.0	0.0320	12.6	58	0.1079000	237.0	636	365	128
14	120740	6.5	0.0585	15.8	143	0.1741000	198.0	239	351	179
15	230752	4.4	0.0441	0.0	100	0.0000087	-100.0	11	61	128
16	191057	4.2	0.0239	3.6	-14	0.0242000	1.0	0	390	112
17	090866	3.5	0.0270	4.1	16	0.0276000	25.0	341	298	92
18	250939	2.7	0.0165	3.1	15	0.0166000	0.6	170	75	74

NOTE: 1 ft³ = 0.028 m³, 1 in = 25.40 mm.^aDate in day, month, year notation.^bRainfall record of rain gage P47.003 also used as input for rain gage P47.005.

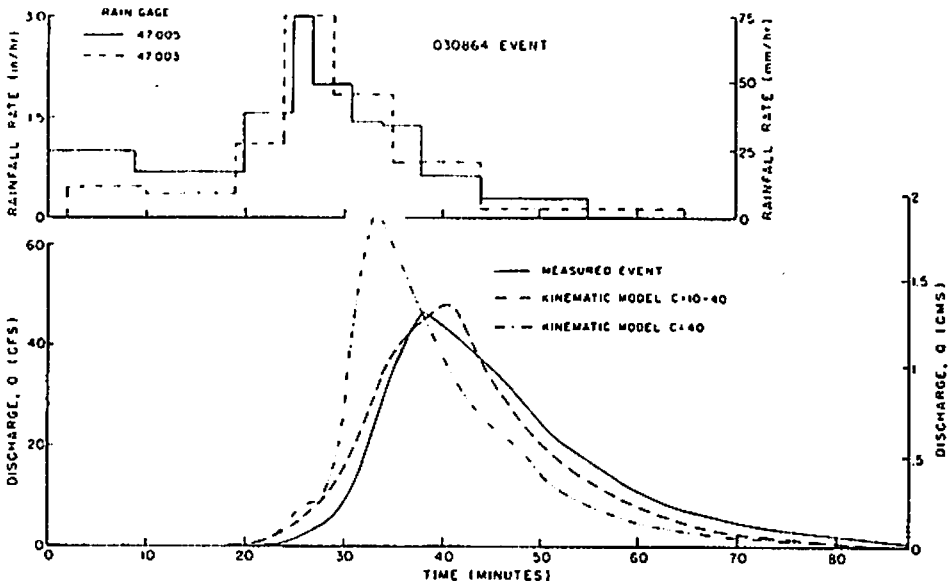


Figure 7. Comparison of Kinematic Model Response for Event No. 4 for Chezy, $C = 40$ and Distributed Chezy, $C = 10 - 40$ (Refer to Table 1 for distribution).

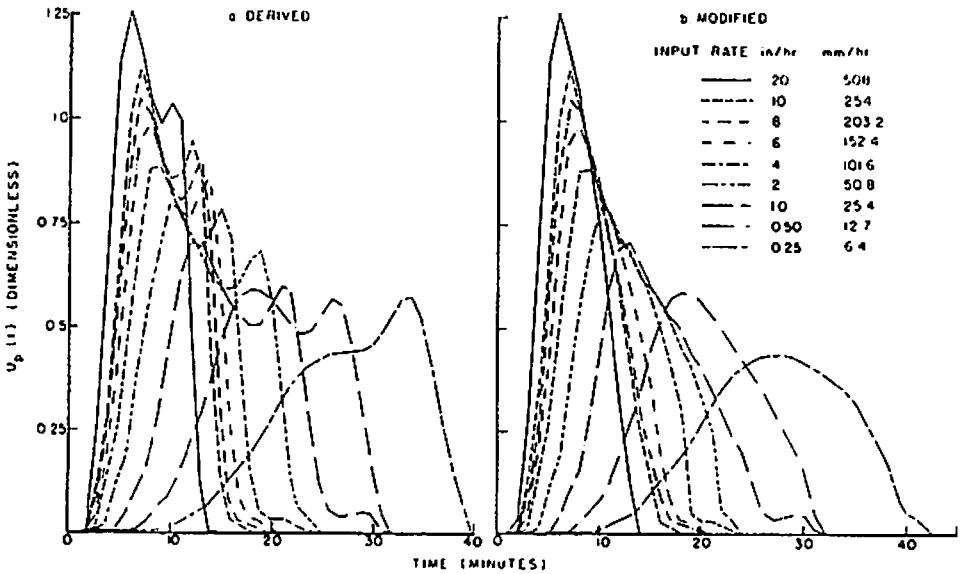


Figure 8. $U_p(t)$ Responses for Model 10.

essentially eliminated supporting the assumption of the modified responses. As the input rate decreases, the secondary response begins to develop. For the 1-inch per hour input rate, the secondary response is evident though it is not as prominent as for the lowest input rate, 0.25-inch per hour. As the flow from these three planes came off more quickly by reversing the dimensions, the total response peak was increased (Figure 10) and the peaks of the unit pulse responses were proportionately increased (Figure 11). The judgment was made at this stage of the study to proceed with development and evaluation of the procedure by using the modified responses (Figure 8b) without incurring the expense of a repeated set of runs with the kinematic model.

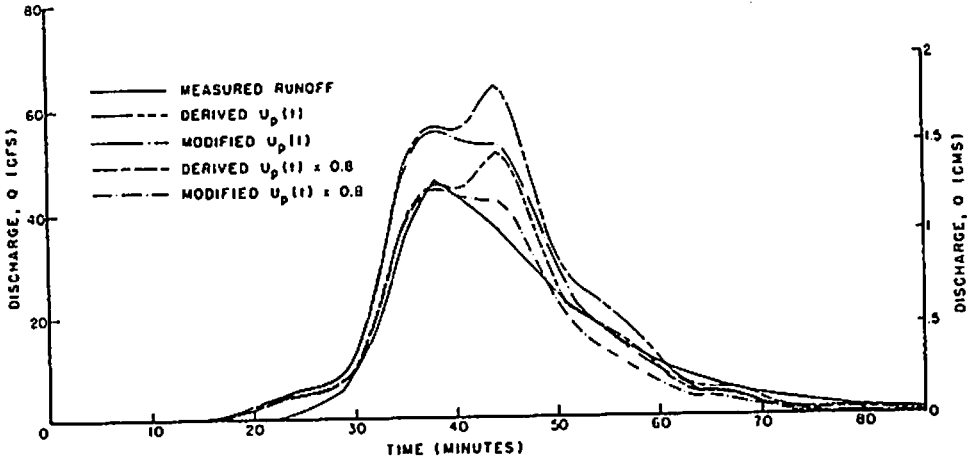


Figure 9. Fit of Convolution Model Responses for Measured Event No. 4 (Table 2).

EVENT 4, 030864

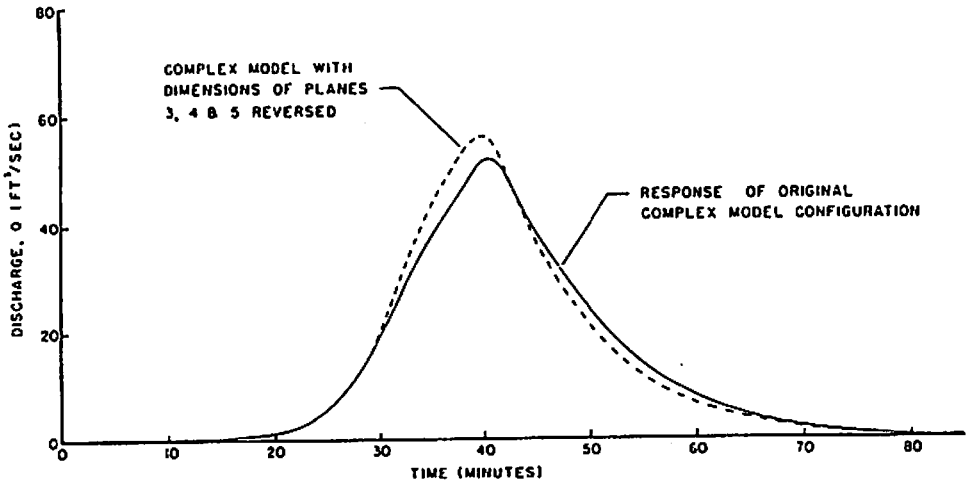


Figure 10. Comparison of Outflow Response of Complex Model, With Dimensions of Triangular Planes Reversed to Give Shorter Flow Lengths, With the Response of the Original Configuration.

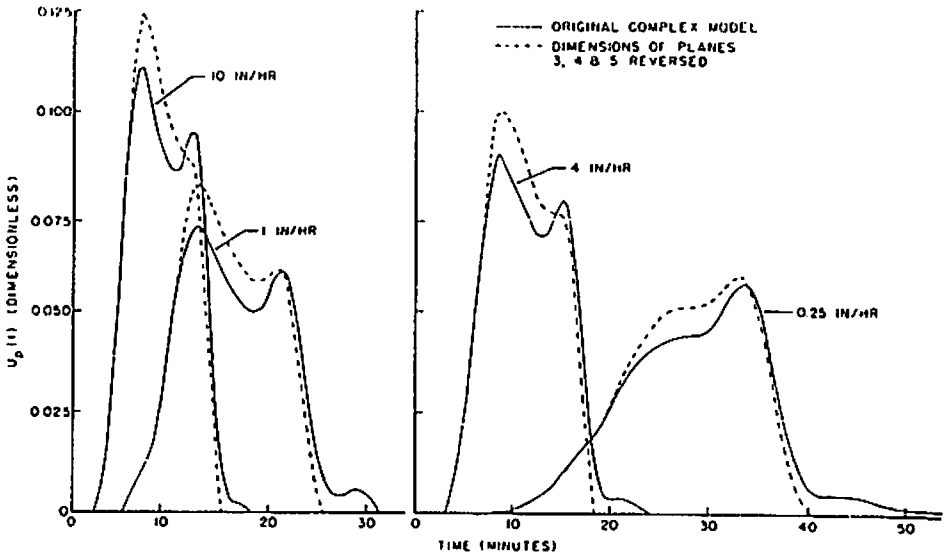


Figure 11. $U_p(t)$ Responses by Difference Method for Complex Model.

These evaluations indicated the need for further studies to determine methodologies for estimation of a reasonable effective or characteristic overland flow length. The fitting of planes (with some characteristic flow length) and channels to real watershed geometry needs some further refining for operational application to ungaged areas.

RESULTS AND COMPARISONS OF PREDICTIONS

For this study, the integral square error was selected as a measure of the goodness-of-fit between the model response and the measured watershed response of an individual event. An integral square error was defined by March and Eagleson (1965) as:

$$E = \frac{[\sum(q_m - q_p)^2]^{1/2}}{\sum q_m} \times 100 \tag{4}$$

in which q_m = measured discharge rate and q_p = predicted discharge rate. Also, a conventional expression for a linear correlation coefficient, ρ , was used as an error measure,

$$\rho = \frac{n\sum q_m q_p - \sum q_m \sum q_p}{[n\sum q_m^2 - (\sum q_m)^2] [n\sum q_p^2 - (\sum q_p)^2]^{1/2}} \tag{5}$$

in which n = number of measurement pairs, q_m = measured discharge rate, and q_p = predicted discharge rate. The measures of goodness-of-fit for the 18 test events, comparing predicted discharges of the two models with measured discharges and with each other, are shown in Table 3.

TABLE 3. Integral Square Error for Kinematic and Convolution Predictions.

Event No.	Date	Between Measured and Kinematic Model		Between Measured and Convolution		Between Kinematic and Convolution Models	
		E %	ρ	E %	ρ	E %	ρ
1	240857	8.02	0.84	9.26	0.79	3.29	0.98
2	040947	6.61	0.97	6.97	0.94	3.29	0.99
3	220769	12.99	0.99	13.03	0.98	2.27	0.99
4	030864	3.43	0.98	2.26	0.99	2.45	0.99
5	270966	4.27	1.00	28.98	0.55	23.37	0.55
6	081054	13.02	0.69	6.30	0.94	27.71	0.66
7	150859	4.24	0.96	6.28	0.91	3.99	0.99
8	110954	11.25	0.80	4.21	0.97	20.47	0.77
9	051058	7.62	0.94	11.27	0.94	5.65	0.97
10	280739	15.14	0.75	30.86	0.74	13.27	0.92
11	010866	15.93	0.96	34.75	0.84	11.35	0.91
12	020759	21.30	0.90	37.85	0.85	12.72	0.89
13	040967	38.12	0.85	41.95	0.70	7.77	0.88
14	120740	30.24	0.92	29.70	0.91	5.29	0.92
15	230752	16.01	0.65	65.93	-0.08	1000.00+	-0.13
16	191057	13.46	0.75	165.37	0.97	51.30	0.61
17	090866	8.94	0.93	43.28	0.98	32.58	0.57
18	250939	15.50	0.62	85.88	0.69	88.14	0.39

In general, both the kinematic and convolution model responses fit better for the larger events (Nos. 1 to 9). The kinematic model also predicts the overall response better than the convolution model, as would be expected, although by the integral square error test, the convolution model was a better predictor for four events (Nos. 4, 6, 8, and 14). The latter situation is not unreasonable in the sense that the errors of measurement and modeling would be so random that there may well be some instances in which the simpler model would predict as well as or better than the complex models.

As mentioned in the previous section, rainfall distribution can vary considerably over this watershed. The distributed kinematic model was constructed to represent this distribution in only a two-part way, because two rain gage sample points exist. Continuously varying rainfall could be projected for the area between the two gages, but even this would be inadequate for extremely variable events, like event No. 1 (Table 2). For this event, the model did not generate enough excess (even when the largest rainfall was used at both inputs) to match the measured response. The explanation lies in a combination of inadequate sampling of the rainfall, inadequate definition of the initial conditions, and the delayed response from planes 1, 3, 4, and 5. There also may be some variation of infiltration over the watershed, but there was no sampling to assess the differences.

The convolution procedure did not predict events that had peak discharges less than 10 to 18 cfs (0.283 - 0.510 m³/sec) as well as the kinematic model (4, p. 121). There are at least two reasons for this deficiency. One is the error introduced by representing

the distributed rainfall as one lumped input by using the record of one rain gage. The extreme of this situation is illustrated by event No. 16, for which the complex model only generated rainfall excess on the upper portion of the watershed. The convolution model when used with input from the gage with the greatest depth uses this input as if it occurred over the entire watershed. As a result, the predicted outflow response is considerably greater than it should be. Second, for the low excess rates, a considerable portion of surface flow is infiltrated as the flow moves over the surface. As an illustration of this situation, the rainfall excess of event No. 4 (Table 2) was modeled as $81,219 \text{ ft}^3$ (2300 m^3) which is equivalent to the output of the 1-0 model. This amount is greater than the total watershed discharge of $62,193 \text{ ft}^3$ (1761 m^3) predicted by kinematic model (76 percent of the excess). The loss between excess generated at a point on the surface and the watershed discharge becomes relatively greater the smaller the event. For example, excess for event No. 17 was $14,397 \text{ ft}^3$ (408 m^3), and the kinematic prediction of watershed discharge was $4,023 \text{ ft}^3$ (114 m^3) (30 percent of the excess). A partial compensation for these two factors is the empirical reduction coefficient of 0.8, illustrated in Figure 9. This coefficient was selected after comparison of results from tests with coefficient of 1.0, 0.833, 0.80, and 0.75. The infiltration model also calculates negative excess when the rainfall rate falls below the infiltration rate. These negative excess values were not used in the simple model transform of input or output, but the incorporation of negative excess values in the transfer calculation could also be a compensation in the proper direction. The meaning of transforming negative inputs must be investigated. What is the pulse response of a negative input rate of a given magnitude?

The kinematic model can represent infiltrating surface flow where the convolution model cannot. Thus, what is generated as an excess at a point is passed to output without any loss due to flow over surfaces or in channels. This situation explains the overestimation bias in all the convolution predictions but it becomes overwhelming for flow responses of less than 18-cfs peak discharge. This performance is probably acceptable for most design situations, which are concerned with the large or maximum flow events. The performance of the convolution model also suggests that the compensating coefficient could be made a function of both the excess rate and the distribution of measured rainfall depths. However, this will be investigated at a later date.

ECONOMICS OF THE APPLICATIONS PROCEDURE

The computer time required to make the convolution model discharge predictions was considerably less than that needed for the kinematic model. The computer run times for the 18 events by the kinematic model are shown in the last column of Table 2. They varied from 74 to 256 seconds, with an approximate total cost of \$175.00, an average of \$9.72 per event. The 18 convolution runs, exclusive of excess calculations and derivation of pulse response, were done in 14.5 seconds for a total cost of \$0.90. The cost to generate the unit pulse response is about \$16.00 per pulse; thus the one time fixed cost of setting up the model for the nine unit pulses used in the discrete linearization convolution was about \$144.00. This fixed initial cost amortized over the 18 test events averages \$8.00 per event, but as more events are modeled or predictions made, this average initial cost per event would decrease. Excess prediction would cost less than \$0.20 per event and the convolution about \$0.05 per event; thus, in this sample run of 18 events, the cost per event was \$8.25 for the discrete convolution versus \$9.72 for the kinematic model.

The advantage would increase for larger numbers of events and sets of events with a greater proportion of large events.

For example, the costs of applying the proposed modeling procedure can be compared with that of using only the complex model. Suppose one wishes to predict a 50-year flow sequence for an ungaged area, like the reference watershed. The geometry can be obtained from topographic maps. The infiltration parameters may be determined from soil sampling or infiltration tests, and roughness coefficient selected from a field assessment. For the area of the reference watershed, and supposedly most areas, distribution of runoff-producing rain storms can be obtained. This distribution can be used in conjunction with a joint distribution of storm rainfall amount distribution, antecedent moisture conditions, and a seasonal shift of infiltration properties to generate storm inputs to the model. Ten runoff events per year would be a reasonable average for the vicinity of the reference watershed; thus, a 50-year simulation would mean 500 runs of a model. From the experience of this study, predictions by the complex kinematic model would cost about \$4,860, whereas the proposed applications model procedure would cost about \$275. Thus, a savings of over \$4,500 is realized. Probably more relevant to the problem, it would only cost a reasonable amount (less than \$300) for the computer operation in a hydrologic engineering evaluation of an ungaged area, with accuracies satisfactory for the engineering purposes. Further, these predictions are based, in a derived sense, on the physics of the real system. Consequently, alterations or management treatments proposed for the ungaged area can be evaluated inexpensively for long sequences of inputs.

SUMMARY AND CONCLUSIONS

A kinematic watershed model with infiltrating planes and channels is assembled. This model, based on the work of many preceding researchers and from this extensive body of antecedent work, was deemed to be an acceptable representation of the natural watershed system. With some additional refinement of fitting plane and channel segments to natural topography, the kinematic model can make outflow predictions for ungaged watersheds with surface flow. However, the kinematic model is complex and computer solutions are expensive. For hydrologic design applications requiring many repetitive tests with the complex kinematic model, a procedure of deriving a simpler model from it is developed and evaluated. The lumped input and uniform rainfall excess determined for the entire area cause the simple convolution model to have an overprediction bias as the size of the event decreases. This bias is inherent in simple system models that cannot account for losses from surface water after rainfall ceases. Lumped models should be acceptable, however, in most design situations dealing with large or maximum flow events. Further investigation may develop compensating coefficients, that would be a function of both excess rate and rainfall distribution, to partially compensate for this limitation of lumped models and to refine the simple model predictions.

A hypothesized use of the procedure shows that appreciable savings could be obtained. In a complementary sense, some hydrologic design problems could be solved with a physically based evaluation for reasonable computer solution costs of only a few hundred dollars.

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