A NONLINEAR HYDROLOGIC CASCADE

by

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LIST OF SYMBOLS

Symbol	Definition
p	Inflow to the storage element
q	Outflow from the storage element
s	Storage in the storage element
t	Time
n	Number of storage elements in the cascade
k	Characteristic parameter of storage element
x	Index of nonlinearity
$\dot{S}$	The vector form of rate of change of storage in the element
P	The vector form of inflow
K	The vector form of parameter k
B	Matrix
S	The vector form of storage element
Q	The vector form of outflow
C	Matrix
$\dot{S}_o$	The vector of $\dot{S}$ at $t = 0$
S_o	The vector of S at $t = 0$
S_1	The vector of S at $t = 1$
Δt	The time interval
k_1	The parameter k of the 1st storage element
k_2	The parameter k of the 2nd storage element
k_3	The parameter k of the 3rd storage element
f	Infiltration rate of Philip's equation
β	A parameter of Philip's equation for infiltration
α	A parameter of Philip's equation for infiltration
F	Objective function
Q_{p_o}	Observed hydrograph peak

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Symbol	Definition
Q_{p_e}	Estimated hydrograph peak
M	Number of rainfall-runoff events in the optimization set
SLOPE	Average watershed slope
CSLOPE	Average mainstream slope
AREA, A	Watershed area
XLR, L	Length of mainstream
WIDTH	Width of watershed
DD	Drainage density
SHAPE	Shape factor
SO	Stream order

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ABSTRACT

A nonlinear hydrologic cascade is developed for simulation of watershed runoff. The cascade consists of three nonlinear storage elements. The parameters of these elements are determined by optimization by the modified Rosenbrock-Palmer algorithm. An attempt is made to correlate these parameters with the watershed physiography but the correlations are not encouraging.

Using the optimized parameter values hydrographs are predicted for a number of events on a number of natural agricultural watersheds. The predicted hydrographs are in good agreement with the observed hydrographs and indicate that the cascade performs well in prediction of runoff.

CHAPTER 1

INTRODUCTION

1.1 General Remarks

The growing interest in the development, calibration, testing, and application of nonlinear runoff models primarily stems from recognition of the nonlinearity of rainfall-runoff process. The past two decades have witnessed evolution of a wide variety of nonlinear approaches to runoff modeling. Of them all storage models appear to be most prominent, in a large measure for their simplicity. Some of the well-known and more recent models are briefly reviewed here.

Diskin (1964) developed a model consisting of two parallel cascades, each having a number of equal reservoirs. Laplace transforms were employed to solve the equations. The nonlinearity in surface runoff was established by showing variability in the shape of the instantaneous unit hydrograph with rainfall storm characteristics.

Kulandaiswamy (1964) derived a general storage equation and applied it to natural watersheds. His results indicated that the model parameters decreased exponentially with increase in the peak discharge, thus indicating basin nonlinearity.

Singh (1964) studied the departure from actual basin behavior of the results of essentially linear or linearlized models. His approach accounted for apparent variation in instantaneous unit hydrographs with rainfall-excess characteristics.

Prasad (1967) developed a nonlinear hydrologic system response model by using the continuity equation and a nonlinear storage reservoir. The model was applied to natural watersheds and yielded good results. The model parameters were estimated from watershed and rainfall-excess characteristics by multiple linear regression and correlation analysis. Watershed characteristics such as area, length of mainstream and slope of mainstream were considered. The rainfall-excess characteristics included the volume of rainfall-excess, its duration and its average intensity.

Mein, et al (1974) developed a simple nonlinear model with a single parameter that was evaluated by fitting the recorded data. The watershed storage effects were represented in the model by nonlinear storage elements.

Reed, et al (1975) considered Nash's model (Nash, 1957) which represents a watershed as a cascade of n linear reservoirs, each with storage coefficient or lag time k. For each reservoir the lag time k was nonlinearly estimated. By applying it to natural catchments it was found that the lag model gave a much improved representation of the hydrograph, particularly its peak characteristics.

Singh (1976) developed a uniformly nonlinear hydrologic cascade. It represented a watershed by a cascade of n nonlinear storage elements or reservoirs, each with storage coefficient k. The model parameter k was estimated by optimization. Further, it was shown that it could be estimated from watershed characteristics. The model was applied to several natural agricultural watersheds and a close agreement between observed and predicted hydrographs was found.

These studies indicate that nonlinear effects are significant in the rainfall-runoff process. The present study considers a nonlinear hydrologic cascade for simulation of watershed response.

1.2 Objectives

The objectives of this study are:

(1) To develop a nonlinear hydrologic cascade for prediction of watershed runoff.

(2) To estimate the model parameters from watershed characteristics.

CHAPTER 2

NONLINEAR HYDROLOGIC CASCADE

2.1 Model Formulation

The nonlinear cascade represents a watershed by a series of unequal nonlinear storage elements or reservoirs as shown in Fig. 2.1. The governing equations for a nonlinear element are a spatially lumped continuity equation and a nonlinear storage-discharge relation, which can be written respectively as:

$$p = q + \frac{ds}{dt} \qquad (2\text{-}1)$$

$$q = ks^{x} \qquad (2\text{-}2)$$

where p = inflow to the storage element in cm/hr;

q = outflow from the storage element in cm/hr;

s = storage in the storage element in cm;

t = time in hour;

$\frac{ds}{dt}$ = rate of change of storage in the element;

k = storage characteristic parameter; and

x = an index of nonlinearity.

Equations (2-1) and (2-2) can be combined to yield a single differential equation relating inflow and storage:

$$\frac{ds}{dt} = p - ks^{x} \qquad (2\text{-}3)$$

The nonlinear cascade of Fig. 2.1 can then be represented by a system of equations:

$$\frac{ds_1}{dt} = p_1 - k_1 s_1^{x}$$

$$\frac{ds_2}{dt} = p_2 + k_1 s_1^{x} - k_2 s_2^{x}$$

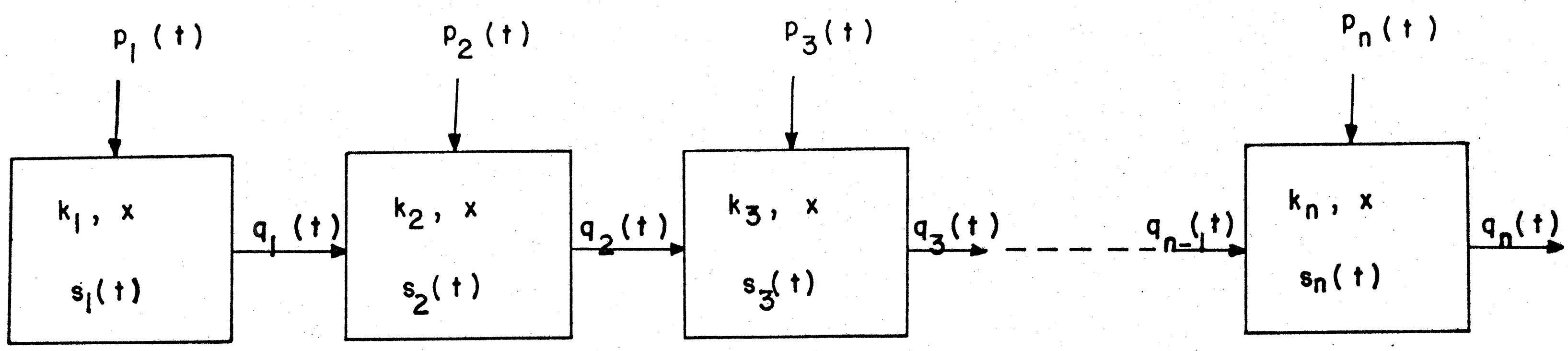

Fig. 2.1. A nonlinear hydrologic cascade with distributed input.

$$\frac{ds_3}{dt} = p_3 + k_2 s_2^x - k_3 s_3^x$$

$$\vdots$$

$$\frac{ds_j}{dt} = p_j + k_{j-1} s_{j-1}^x - k_j s_j^x \tag{2-4}$$

$$\vdots$$

$$\frac{ds_{n-1}}{dt} = p_{n-1} + k_{n-2} s_{n-2}^x - k_{n-1} s_{n-1}^x$$

$$\frac{ds_n}{dt} = p_n + k_{n-1} s_{n-1}^x - k_n s_n^x$$

where n denotes the number of elements in the cascade. Note that the parameter n does not change from one element to another whereas k does.

Equation (2-4) can be written more conveniently and concisely in the matrix form as:

$$\dot{S} = P + KBS \tag{2-5}$$

where

$$\dot{S} = \begin{bmatrix} \frac{ds_1}{dt} \\ \frac{ds_2}{dt} \\ \frac{ds_3}{dt} \\ \vdots \\ \frac{ds_j}{dt} \\ \vdots \\ \frac{ds_n}{dt} \end{bmatrix} ;$$

$$P = \begin{bmatrix} p_1 \\ p_2 \\ p_3 \\ \vdots \\ p_j \\ \vdots \\ p_n \end{bmatrix} ;$$

$$K = \begin{bmatrix} k_1 \\ k_2 \\ k_3 \\ \vdots \\ k_j \\ \vdots \\ k_n \end{bmatrix} ;$$

$$S = \begin{bmatrix} s_1 \\ s_2 \\ s_3 \\ \vdots \\ s_j \\ \vdots \\ s_n \end{bmatrix} \quad ; \quad \text{and}$$

$$B = \begin{bmatrix} s_1^{x-1} & 0 & 0 & \cdots & 0 & 0 \\ s_1^{x-1} - s_2^{x-1} & & 0 & \cdots & 0 & 0 \\ 0 & s_2^{x-1} - s_3^{x-1} & & \cdots & 0 & 0 \\ \vdots & \vdots & \vdots & & \vdots & \\ 0 & 0 & 0 & & s_{n-1}^{x-1} - s_n^{x-1} & \end{bmatrix}$$

Our main purpose is to obtain the relationship between the final outflow q_n and the lateral inflows p_1, p_2, p_3------p_j---p_n. The storage-discharge relationship for the cascade can then be expressed in the matrix form as:

$$Q = KCS \tag{2-6}$$

where

$$Q = \begin{bmatrix} q_1 \\ q_2 \\ q_3 \\ \vdots \\ q_j \\ \vdots \\ q_n \end{bmatrix};$$

$$C = \begin{bmatrix} s_1^{x-1} & 0 & \cdots & 0 & 0 \\ 0 & s_2^{x-1} & \cdots & 0 & 0 \\ \vdots & \vdots & & & \\ & & & s_{n-1}^{x-1} & 0 \\ 0 & 0 & & 0 & s_n^{x-1} \end{bmatrix}; \text{ and}$$

K and S are as defined previously.

Equations (2-5) and (2-6) provide a state variable representation of the cascade. For simulation of runoff, however, the cascade with lumped input was considered as shown in Fig. 2.2.; that is, the inflows p_2, p_3, p_4--------p_n in P are zero and p_1 is positive. The cascade operation can now be summarized as follows:

(1) Specify parameters k_1, k_2, --------k_n; x and n.

(2) Select the time interval Δt.

(3) Given the input pattern (rainfall-excess) use Eq. (2-5) to compute $\dot{S}$. At the beginning of time, t = 0, B and S are zero. The inputs p_2, p_3--------p_n in the matrix P are all zero; only p_1 is positive. So the value of $\dot{S}_o$ at t = 0 is obtained. This is the initial condition.

(4) Input into jth storage element at the beginning of a particular time interval Δt is an impulse of input, equal to the sum of input p_j, the surface inflow q_{j-1} and the surface outflow of q_j. The impulse input causes an instantaneous change in storage of the jth element. Responses of the elements to the impulse inputs determine the outputs and the states of the elements at the end of the time interval Δt; the new states are the initial conditions for the following time increment.

(5) For given initial states of the system, B_o, S_o, and P_o at the beginning of a particular time interval Δt, $\dot{S}_o$ can be obtained from Eq. (2-5). The state of the system, at the end of this time interval Δt, is the initial state at the beginning of the next time interval Δt and is given by $S_1 = S_o + \dot{S}_o$. Outflows from the n elements at the end of the time Δt_o can then be obtained from Eq.(2-6).

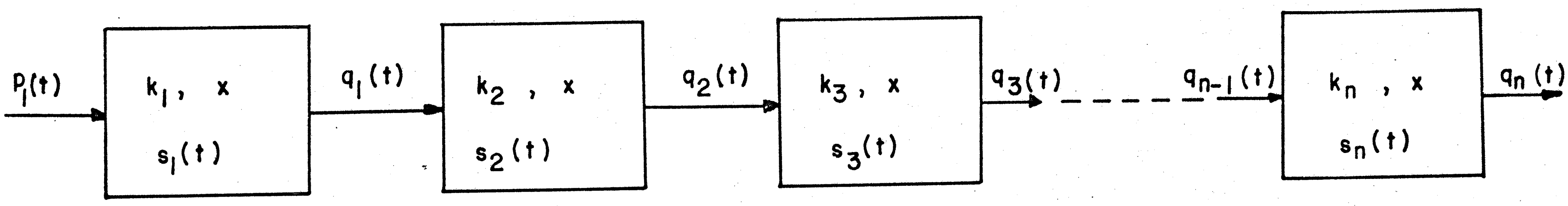

Fig. 2.2. A nonlinear hydrologic cascade with lumped input.

CHAPTER 3

APPLICATION TO NATURAL WATERSHEDS

For model application, thirty-eight natural agricultural watersheds were selected. These watersheds represent several regions of the United States. Of them, 16 watersheds are near Riesel (Waco), Texas; 5 near Hastings, Nebraska; 5 near Coshocton, Ohio; 8 near Oxford, Mississippi; and 1 each in Watkinsville, Georgia; McCredie, Missouri; Fennimore, Wisconsin; and Ralston Creek, Iowa. These are small watersheds varying in area from 0.5 ha to 3,055 ha. For complete information on watershed characteristics see Shelburne and Singh(1976) or USDA publications entitled "Hydrologic Data for Experimental Agricultural Watersheds in the United States."

3.1 Determination of mean Areal Rainfall

Rainfall-runoff data of all 38 watersheds were obtained from the USDA publications,"Hydrologic Data For Experimental Agricultural Watersheds in the United States." These publications are released almost every year and contain for each watershed one largest event a year. Although there may be more than one raingage on a watershed, data is normally available in the USDA publications for a centrally located raingage indicating that this represents the mean areal rainfall. For consistency, this practice was followed for each watershed. The guidelines for selection of hydrologic data are given by Shelburne and Singh (1976).

3.2 Determination of Rainfall-Excess

Rainfall-excess is inflow to the model and is determined by substracting infiltration from rainfall. Philip's equation (Philip, 1957) was used to estimate infiltration which can be written as:

$$f = \tfrac{1}{2}\,\beta t^{-\frac{1}{2}} + \alpha \qquad (3\text{-}1)$$

where f = infiltration rate in cm/hr;

β = a parameter, dependent on soil characteristics and initial soil moisture content;

t = time in hours; and

α = a parameter, dependent on soil characteristics.

The parameter α is related to the saturated hydraulic conductivity (Schulz, 1973) and, therefore, can be determined from soil characteristics (Gray, 1970). To account for antecedent soil moisture conditions, the parameter β was allowed to vary with each rainfall event and was determined by preserving the volume continuity of water. For a sample rainfall event on watershed 4-H, Hastings, Nebraska, rainfall-excess determination by this method is shown in Fig. 3.1.

3.3 Choice of Objective Function

The following objective function was used in this study:

$$F = \sum_{j=1}^{M} \left[Q_{p_o}(j) - Q_{p_e}(j) \right]^2 \Longrightarrow \min \qquad (3\text{-}2)$$

where F = objective function;

$Q_{p_o}(j)$ = observed hydrograph peak in cm/hr for the jth event;

$Q_{p_e}(j)$ = estimated hydrograph peak in cm/hr for the jth event; and

M = number of rainfall-runoff events in the optimization set.

The choice of this objective function is based on the findings of Singh (1975a, 1975b, 1975c, 1976) and Shelburne and Singh (1976). Because it only requires the hydrograph peak from each rainfall-runoff event, computationally it is more efficient. When F is divided by the number of events in the optimization set, the mean square error can be obtained which reflects on the average error occurring in the performing of optimization.

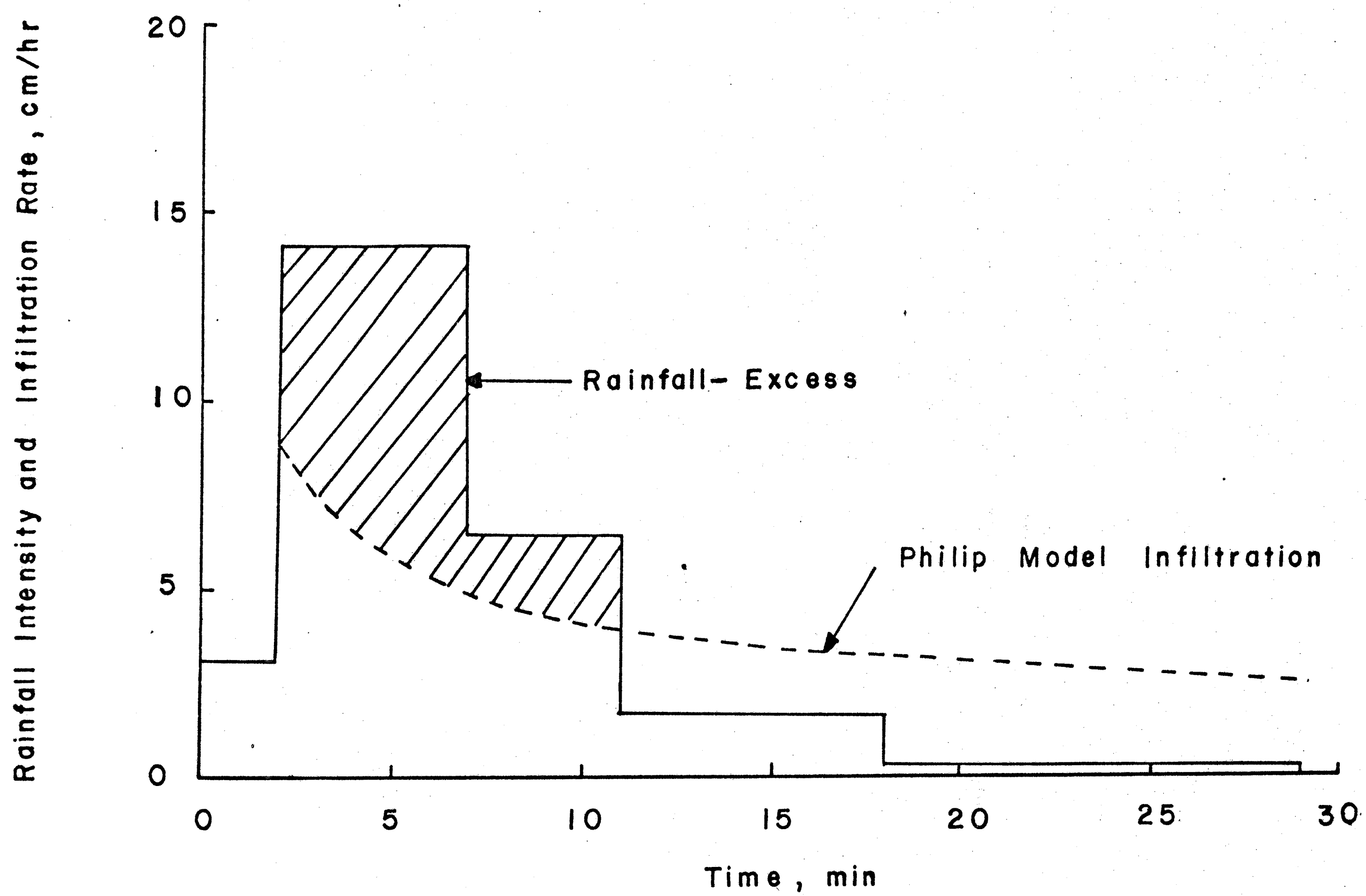

Fig. 3.1. Determination of rainfall-excess by Philip equation for the event of 8-11-1939, Watershed 4-H, Hastings, Nebraska.

3.4 Determination of Computation Time Interval

To study the effect of the computation time interval on model parameters and its predictive performance a watershed 4-H, Hastings, Nebraska, was selected. This watershed had 18 events. These events were divided into two sets; one set, called the optimization set, consisted of 10 events; the other set, called the prediction set, consisted of 8 events. These two sets did not have any events in common. The number of elements, n, in the cascade was restricted to three and x was fixed at 1.5 (Singh, 1976). Then parameters k_1, k_2, and k_3 were optimized for the optimization set of events by the modified Rosenbrock-Palmer algorithm (Rosenbrock, 1960; Palmer, 1969; Himmelblau, 1972) using the objective function of Eq. (3-2). The optimization was performed for different time intervals of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0 and 4.5 minutes. Table 3.1 shows the optimized values of k_1, k_2, k_3 and the objective function values for different time intervals. It is evident from Table 3.1 and Fig. 3.2 that the parameters k_1, k_2, and k_3 change with time interval and the objective function value increases with the increase in the time interval. To see the effect of time interval on model performance runoff hydrographs were predicted for the events in the prediction set, using optimized values of k_1, k_2 and k_3 corresponding to each computation time interval. The predicted hydrograph peak and its time are shown in Table 3.2 and Figs. 3.3-3.11. It is clear that the relative error increases with the increase in the time interval. From the above observations a computation time interval of 1.0 minute was found suitable.

3.5 Parameter Optimization

The nonlinear cascade has (n + 2) parameters: n, x, k_1, k_2, -------k_n. The parameter x reflects the degree of nonlinearity in surface

Table 3.1. Optimized model parameter values for different computation time intervals for watershed 4-H, Hastings, Nebraska.

Watershed Name	Time Interval (min)	Optimized Model Parameters			Objective Function
		k_1	k_2	k_3	F
Hastings, Nebraska					
4-H	0.5	0.3300	7.57	10.6	27.4382
	1.0	0.1800	3.78	4.3	13.9757
	1.5	0.9276	0.8545	0.5	44.4446
	2.0	0.2935	0.5670	0.6	59.8454
	2.5	0.3325	0.3173	0.3	60.9275
	3.0	0.1603	0.3786	0.4	51.6472
	3.5	0.1042	0.2133	0.4	145.4256
	4.0	0.0875	0.1351	0.3	197.1997
	4.5	0.0639	0.1128	0.2	326.8289

Table 3.2. Comparison of predicted and observed hydrograph peak and its time for different computation time intervals

		Time Interval 0.5 min					
Watershed Name	Date	Observed Hydrograph Peak (cm/hr)	Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Observed Hydrograph Peak Time (min)	Predicted Hydrograph Peak Time (min)	Relative Error (%)
Hastings							
4-H	8-11-39	4.5212	4.8725	7.7	5.0	5.5	-10.0
	6-20-42	5.8166	5.7329	1.44	8.0	10.5	-31.25
	9-5-46	3.8862	3.0823	20.69	12.0	6.5	45.83
	6-1-51	6.7564	6.8449	-1.31	123.0	130.5	-6.09
	7-3-52	9.1948	9.4789	-3.09	21.0	18.5	11.90
	6-12-58	0.9195	0.9254	-0.65	14.0	8.5	39.28
	6-12-65	9.7028	8.8887	8.39	19.0	14.5	23.68
	6-12-65	6.1468	5.8334	5.10	7.0	11.5	-64.28

Table 3.2. (continued)

Time Interval 1.00 min				Time Interval 1.5 min			
Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Predicted Hydrograph Peak Time (min)	Relative Error (%)	Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Predicted Hydrograph Peak Time (min)	Relative Error (%)
3.4254	24.24	6.0	-20.00	3.9865	11.83	7.5	-50.00
5.0194	13.71	11.0	-37.50	5.7994	0.30	10.5	-31.25
2.1212	45.42	8.0	33.33	2.6170	32.66	8.5	70.83
6.3084	6.63	131.0	-6.50	6.8863	-1.92	126.0	-2.43
9.9094	-7.77	19.0	19.52	9.6060	-4.47	19.5	7.14
0.5000	45.62	11.0	21.43	0.5426	40.98	13.5	3.57
8.2274	15.21	15.0	21.05	9.0837	6.38	15.0	5.26
4.9645	19.23	12.0	-71.42	5.9855	2.62	12.00	-71.42

Table 3.2. (continued)

Time Interval 2.00 min				Time Interval 2.5 min			
Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Predicted Hydrograph Peak Time (min)	Relative Error (%)	Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Predicted Hydrograph Peak Time (min)	Relative Error (%)
2.7216	39.80	12.0	-140.0	1.8065	60.04	15.0	-200
5.3338	8.30	12.0	-50.0	4.1275	29.04	15.0	-87.5
1.9207	50.58	14.0	-16.66	1.5414	60.34	17.5	-45.83
6.5639	2.85	128.0	-4.06	5.4795	18.90	132.5	-7.72
9.5568	-3.94	20.0	4.76	9.2300	-0.38	20.0	4.76
0.3166	65.57	20.0	-42.85	0.2163	76.47	25.0	-78.57
9.0079	7.16	16.0	15.78	8.1537	15.97	17.5	7.89
5.5478	9.75	14.0	-100	4.1666	32.22	17.5	-150.0

Table 3.2. (continued)

Time Interval 3.00 min				Time Interval 3.5 min			
Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Predicted Hydrograph Peak Time (min)	Relative Error (%)	Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Predicted Hydrograph Peak Time (min)	Relative Error (%)
1.5557	65.59	15.00	-200.0	0.8980	80.14	21.0	-320.0
3.7273	35.92	15.00	-88.88	2.6531	54.39	21.0	-200.0
1.3767	64.57	21.0	-75.00	0.9331	75.99	24.0	-104.0
6.6588	1.44	133.0	-9.87	6.2974	6.79	136.5	-10.90
9.4169	-2.42	21.0	0	8.5906	6.57	21.0	0
0.1798	80.45	27.0	-92.84	0.1177	87.20	38.5	-175.0
8.3628	13.81	18.0	5.26	6.8909	28.98	21.0	-10.52
4.1034	33.24	18.0	-155.55	2.9706	51.67	21.0	-200.0

Table 3.2. (continued)

Time Interval 4.00 min				Time Interval 4.5 min			
Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Predicted Hydrograph Peak Time (min)	Relative Error (%)	Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Predicted Hydrograph Peak Time (min)	Relative Error (%)
0.5825	87.12	28.0	-460.0	0.3779	91.64	40.5	-710.0
2.0030	65.56	24.0	-200.0	1.4477	75.11	31.5	-293.75
0.6437	83.44	32.0	-166.66	0.4421	88.62	40.5	-237.50
6.3478	6.05	140.0	-13.82	4.9579	26.62	144.0	-17.07
7.3808	19.73	24.0	-14.28	6.1893	32.69	27.0	-28.57
0.0847	90.79	48.0	-242.85	0.0611	93.35	63.0	-350.00
5.2705	45.68	24.0	-26.31	4.1076	57.67	27.0	-28.57
2.3216	62.23	24.0	-242.85	1.5443	74.88	40.5	-478.57

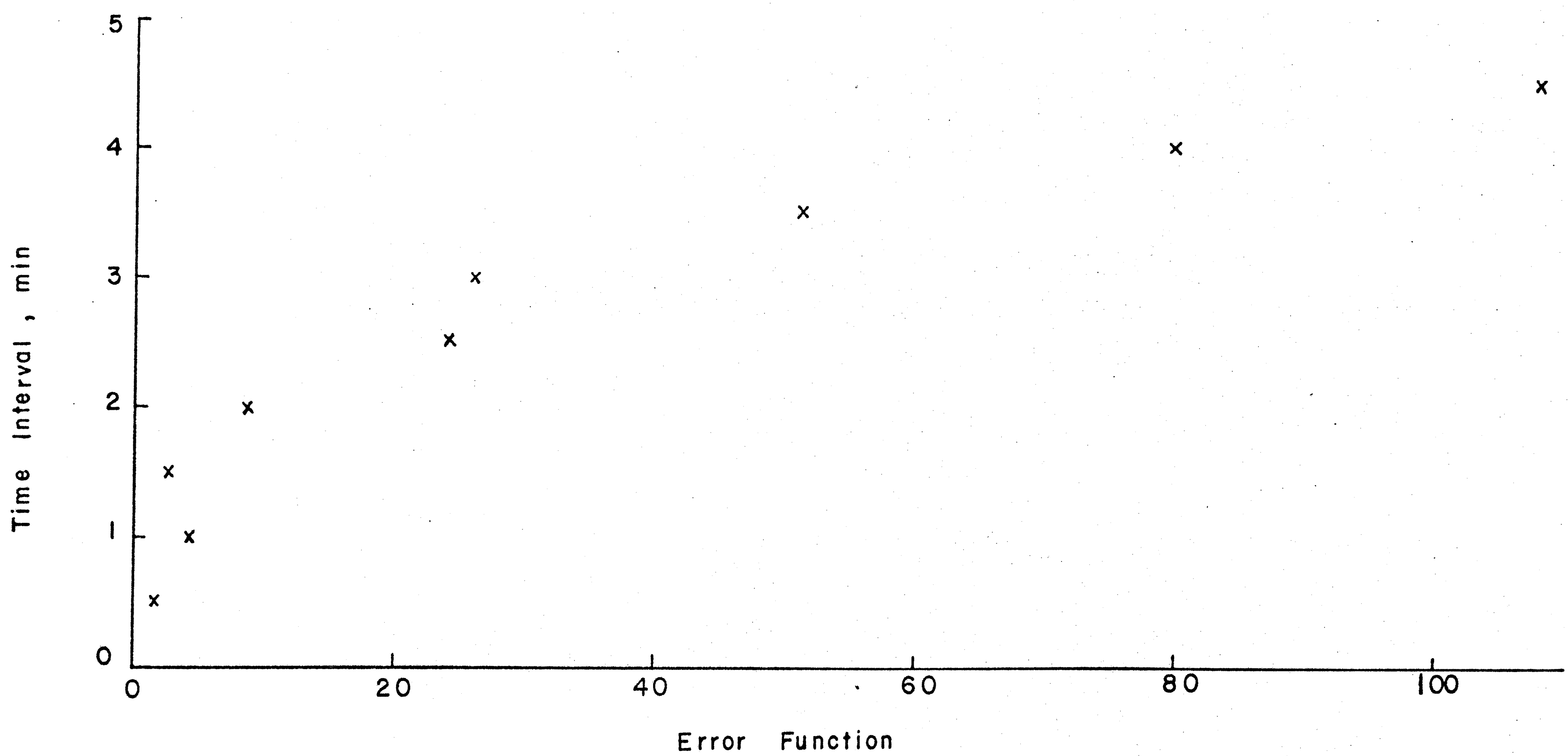

Fig. 3.2. Time interval versus error function of predicted runoff on watershed 4-H, Hastings, Nebraska.

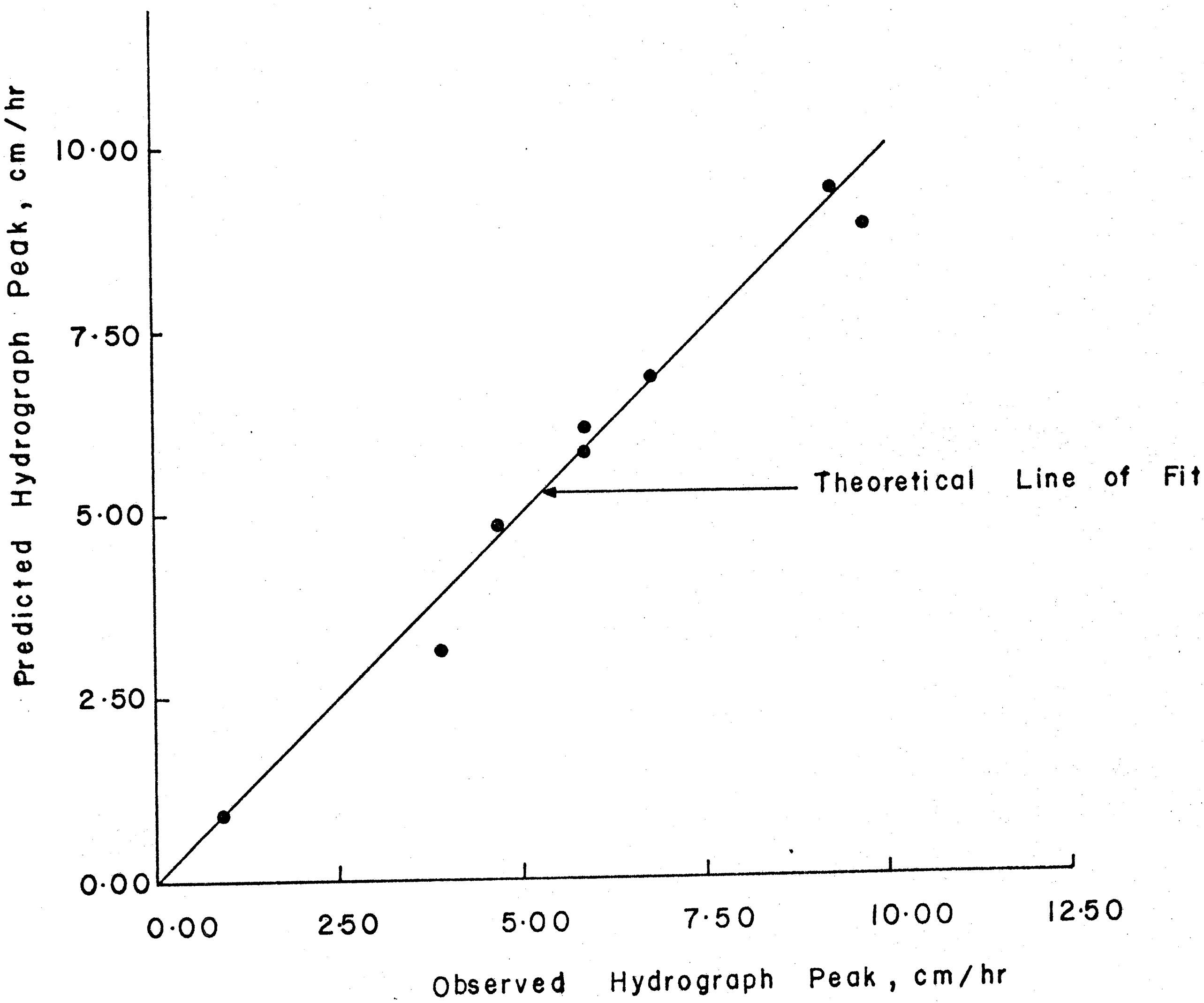

Fig. 3.3. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 0.5 min on watershed 4-H, Hastings, Nebraska.

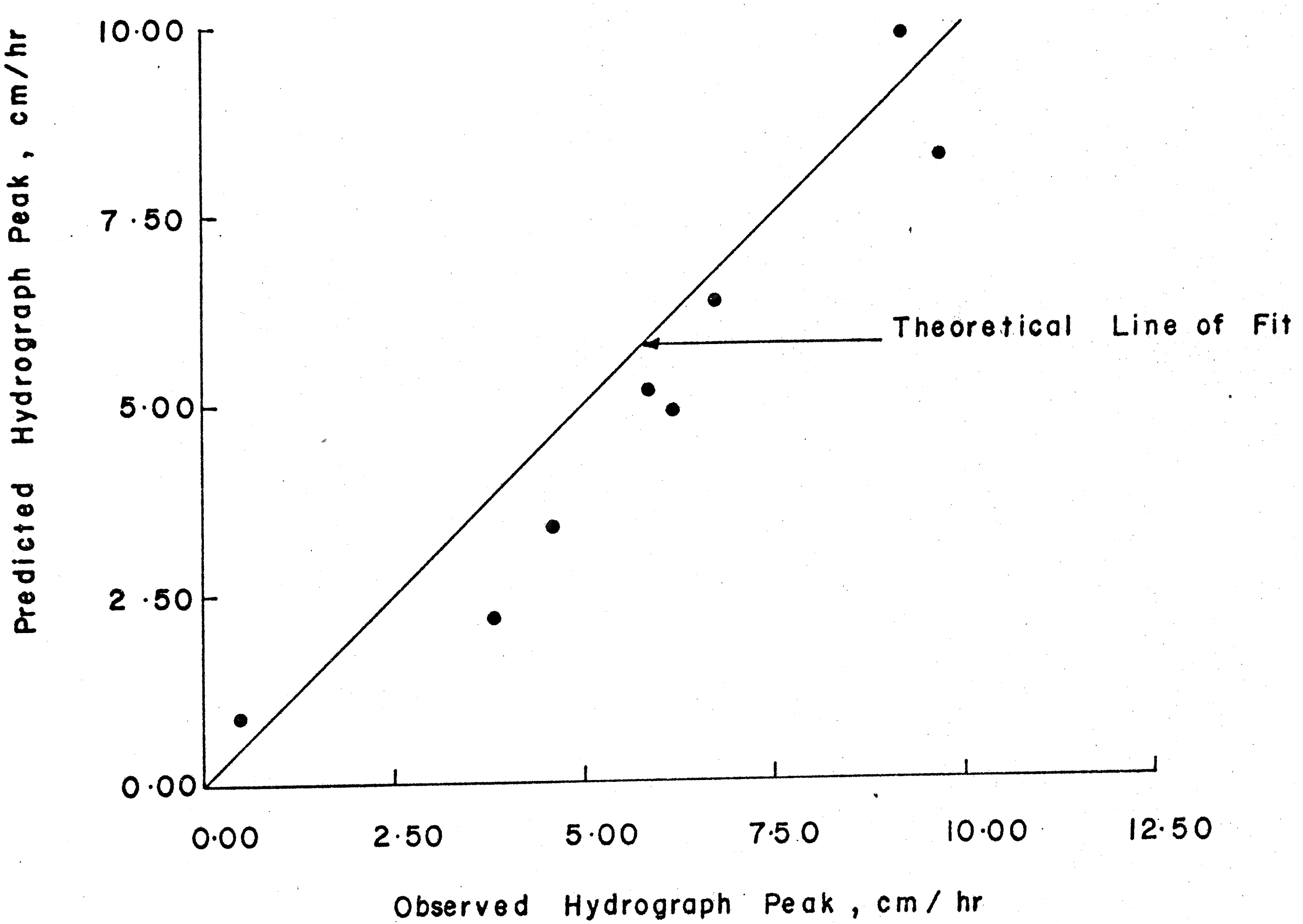

Fig. 3.4. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 1.00 min on watershed 4-H, Hastings, Nebraska.

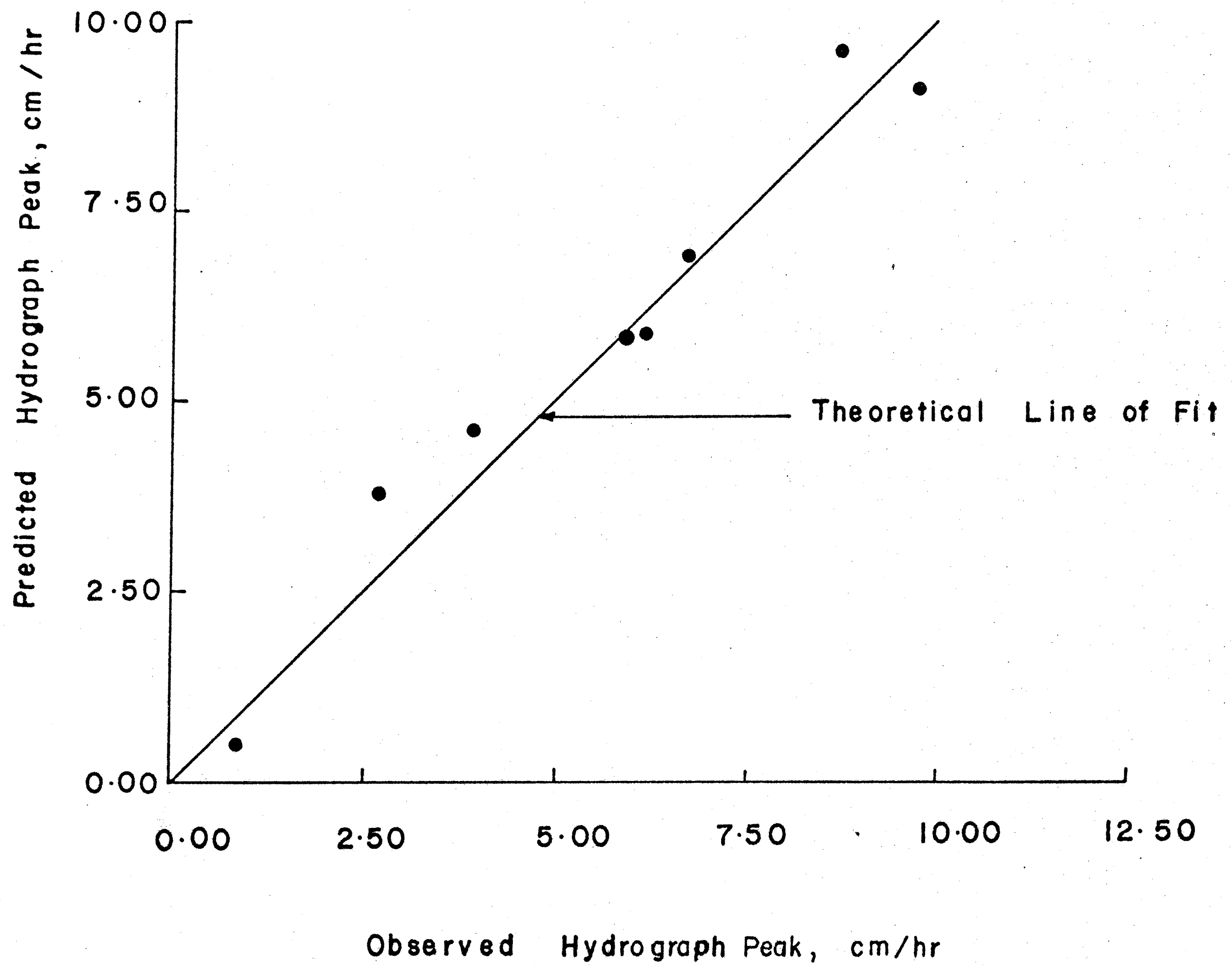

Fig. 3.5. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 1.5 min on watershed 4-H, Hastings, Nebraska.

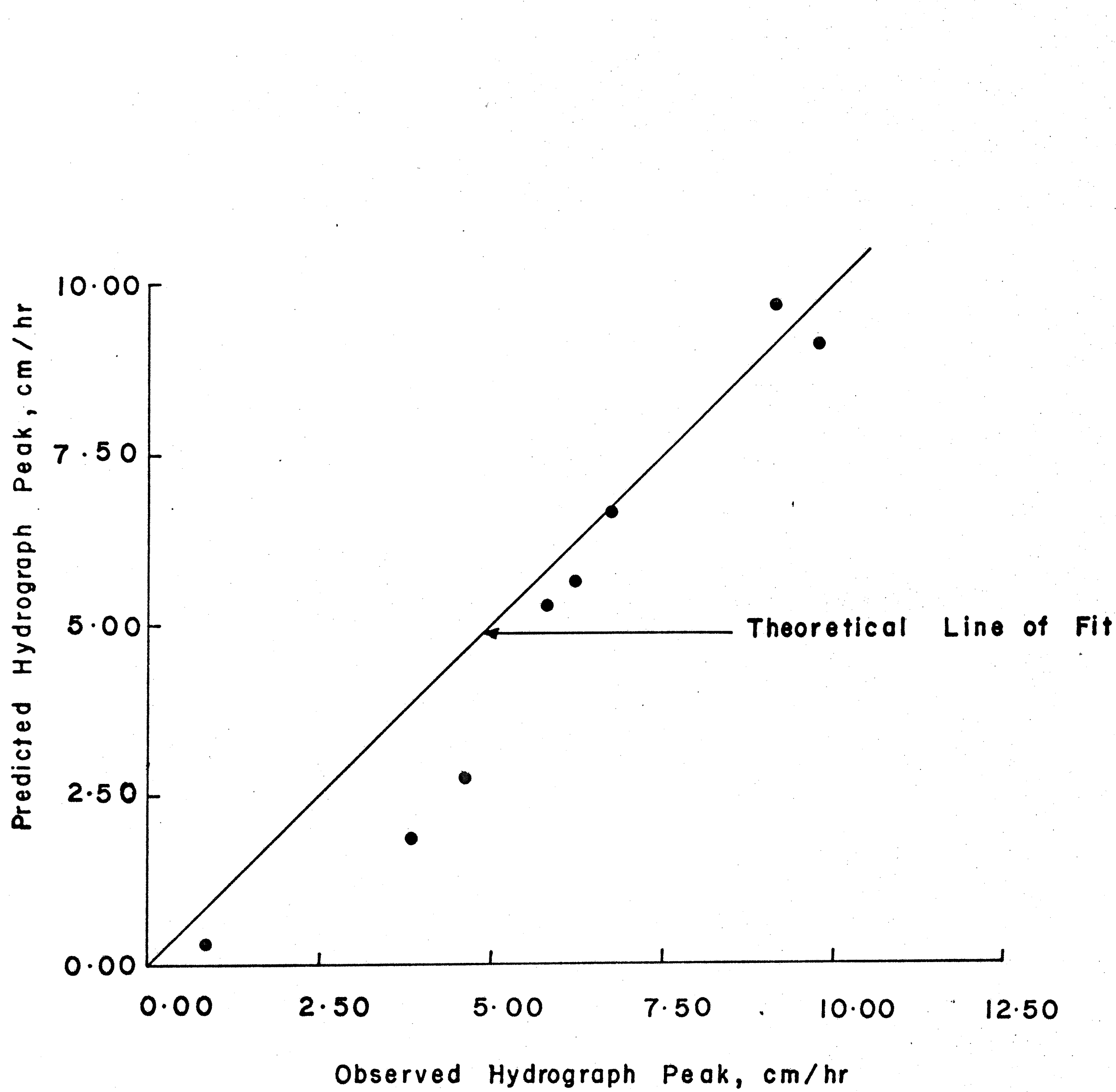

Fig. 3.6. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 2.00 min on watershed 4-H, Hastings, Nebraska.

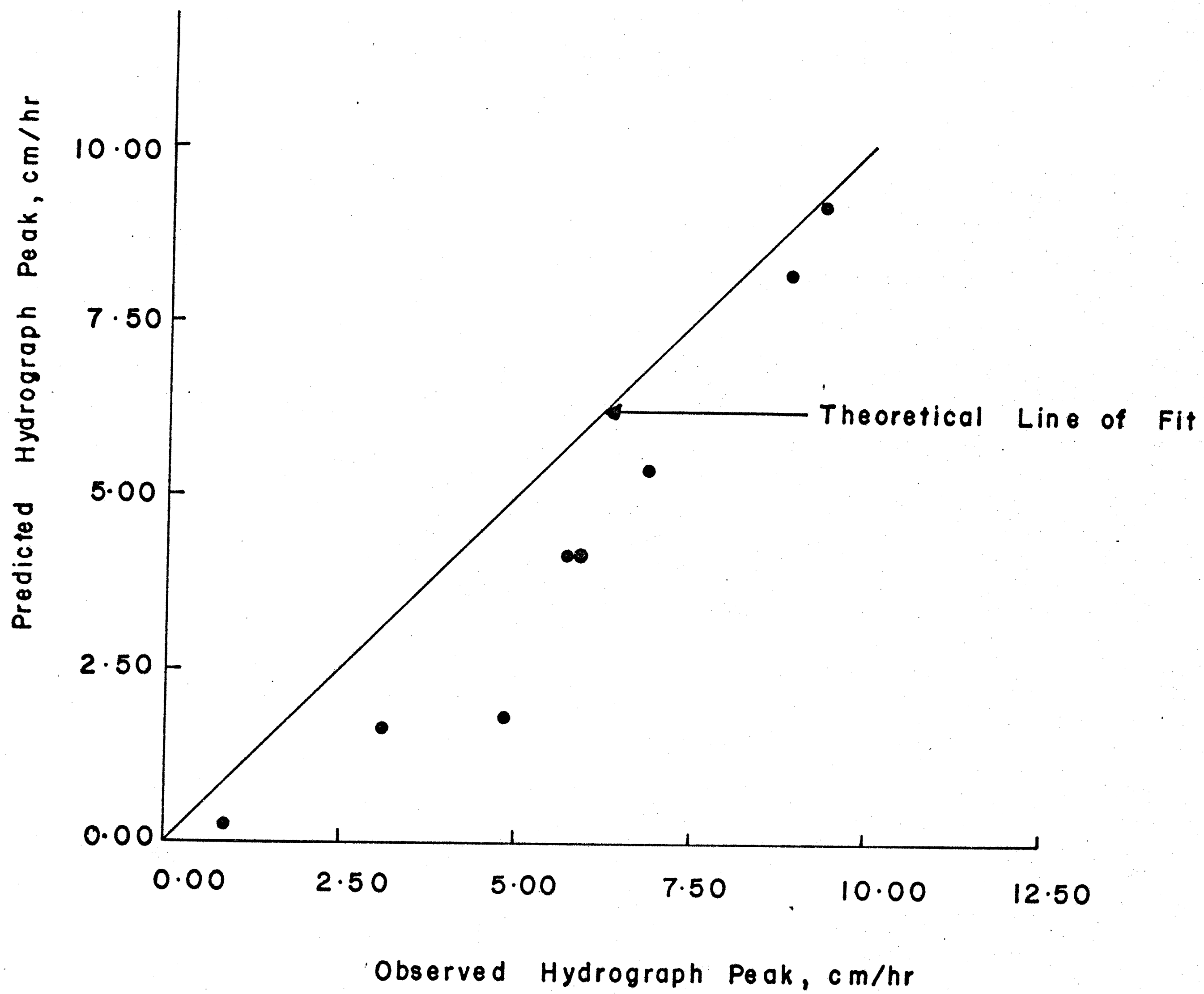

Fig. 3.7. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 2.50 min. on watershed 4-H, Hastings, Nebraska.

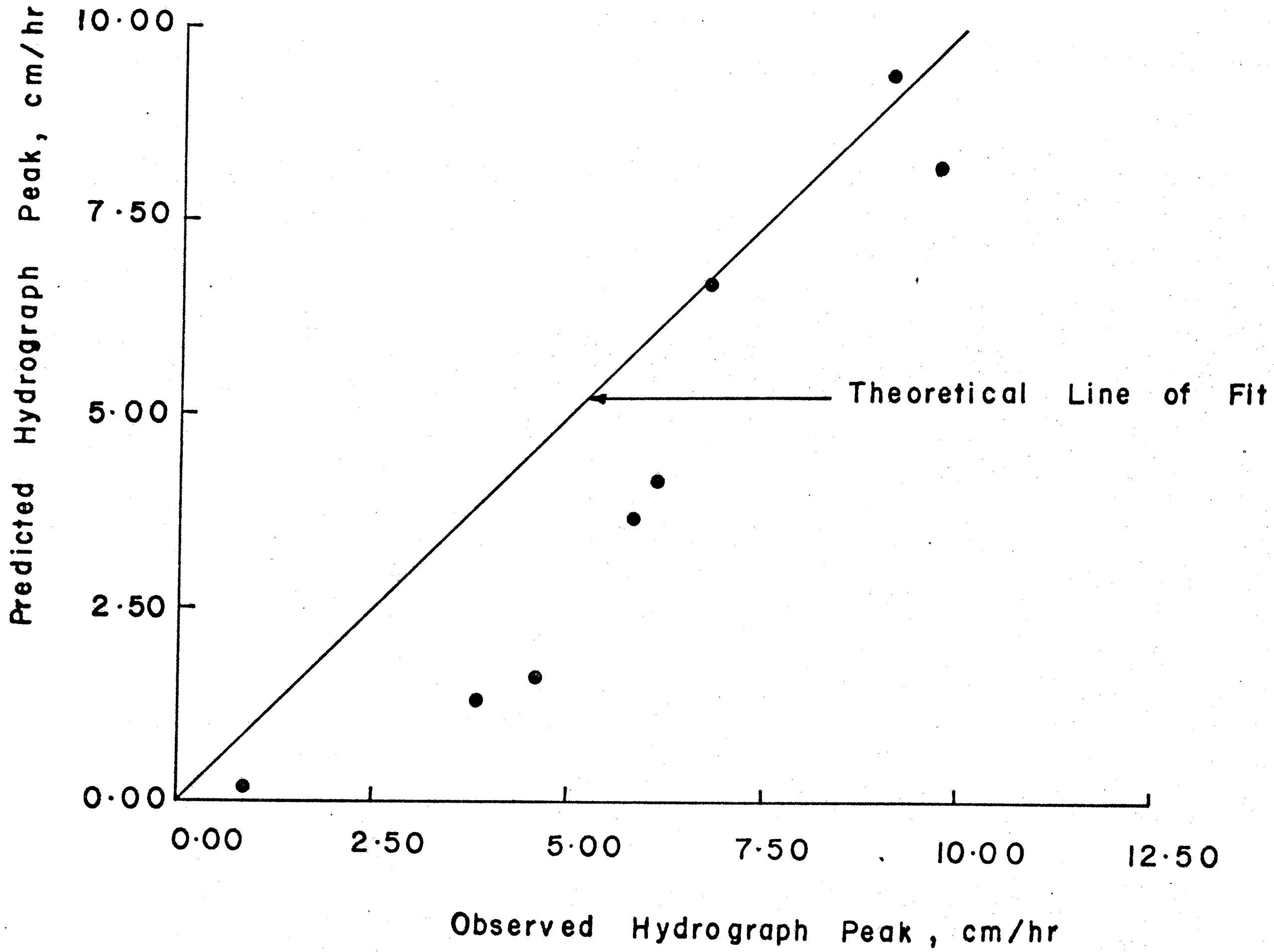

Fig. 3.8. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 3.00 min on watershed 4-H, Hastings, Nebraska.

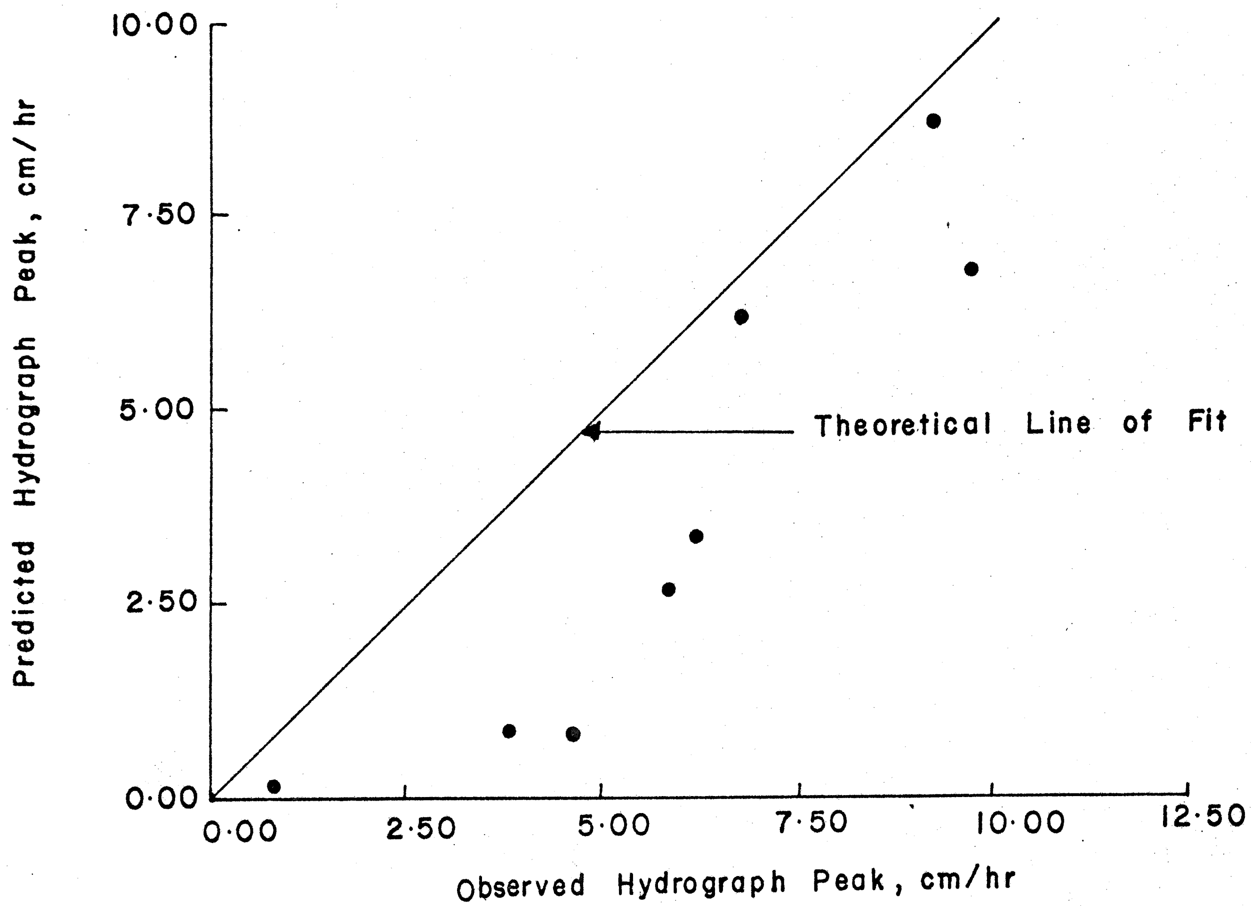

Fig. 3·9. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 3.50 min on watershed 4-H, Hastings, Nebraska.

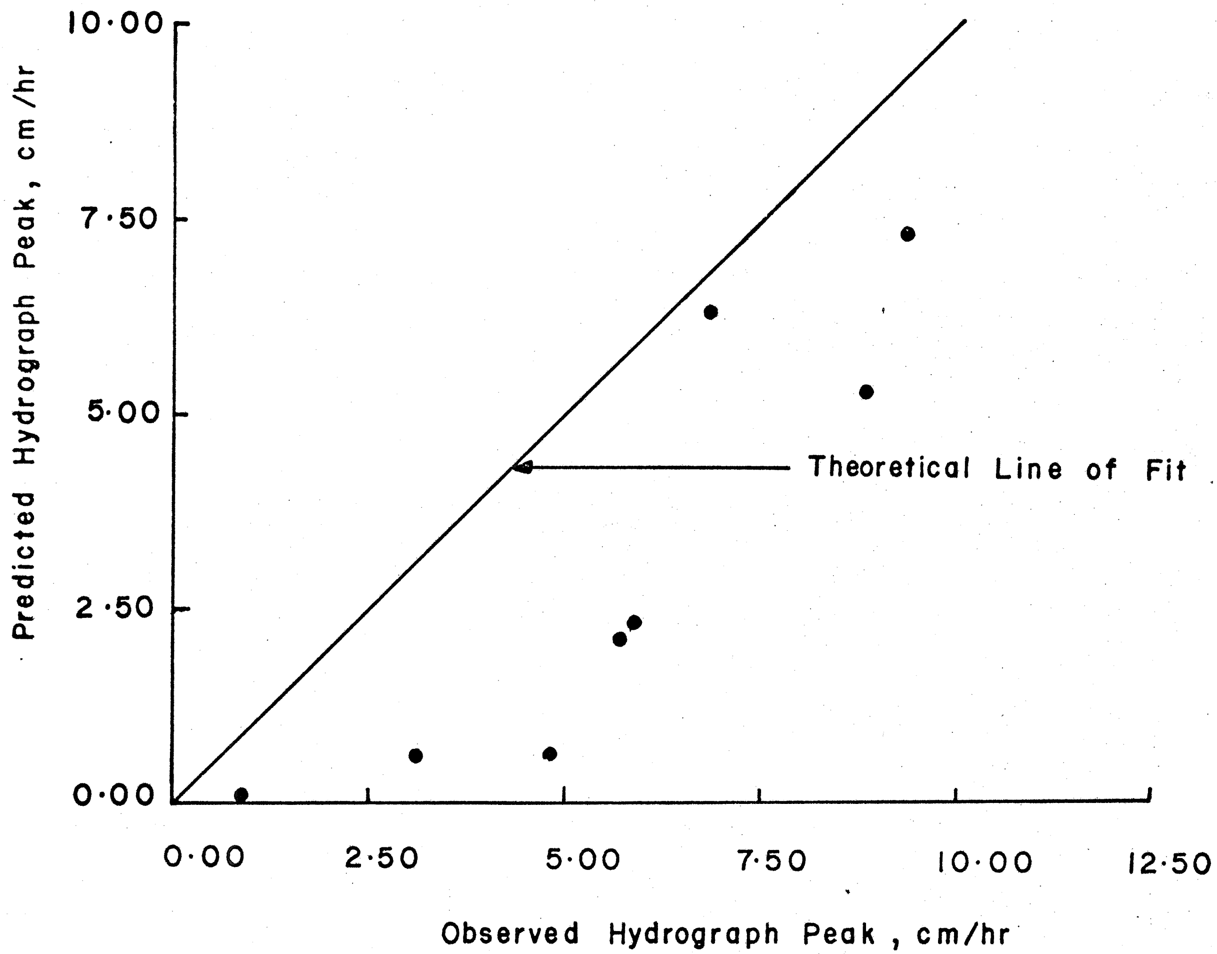

Fig. 3.10. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 4.00 min on watershed 4-H, Hastings, Nebraska.

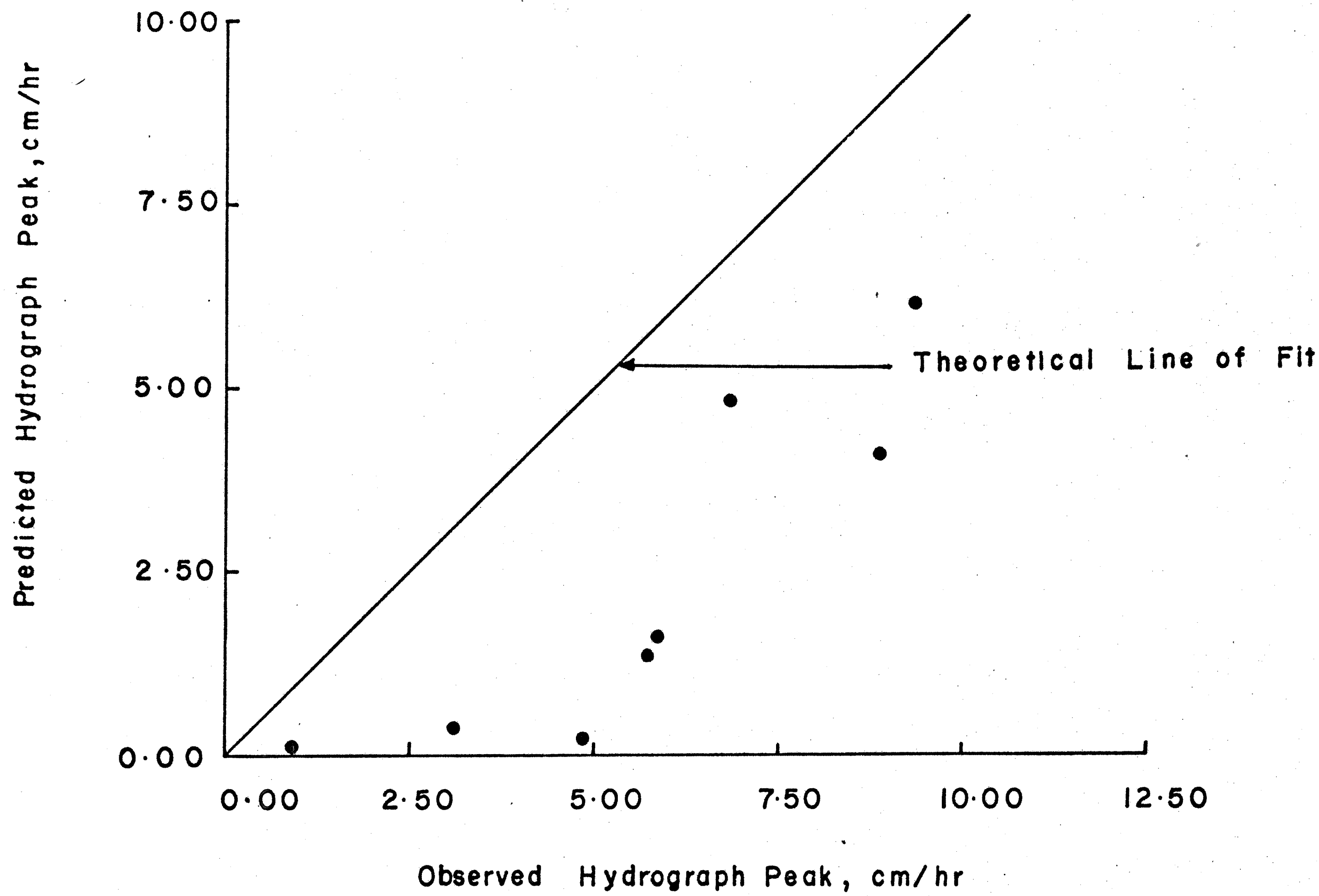

Fig. 3.11. Predicted hydrograph peak versus observed hydrograph peak for computation time interval of 4.50 min on watershed 4-H, Hastings, Nebraska.

runoff. Based on the findings of Singh (1976), x was fixed at 1.5. The parameter n denotes the number of storage elements in the cascade. The value of n must be greater than one to obtain proper hydrograph shape and will depend on the topographic complexity. Again, based on the study by Singh (1976) and Shelburne and Singh (1976) n was fixed at 3. Thus parameters for optimization are k_1, k_2, and k_3. As before, rainfall-runoff events of each of 38 watersheds were divided into an optimization set and a prediction set. Parameters k_1, k_2, and k_3 were then optimized for each optimized set of events by the modified Rosenbrock-Palmer algorithm (Rosenbrock, 1960; Palmer, 1969; Himmelblau, 1972) in accordance with the objective function of Eq. (3-2). The results of optimization are given in Table 3.3. It should be remarked that the parameters do not change very much from one watershed to another in a given region. This observation may be useful in specification of the parameters on a regional basis.

3.6 Hydrograph Prediction

Hydrographs were predicted for the events in the prediction sets of several watersheds, utilizing the optimized parameter values. For a few sample watersheds hydrograph peak characteristics are given in Table 3.4. It is clear that the predicted hydrograph peak characteristics are in good agreement with the observed peak characteristics. A comparison of observed and predicted hydrographs for some sample events is shown in Figs. 3.12-3.22. These figures indicate that the cascade can simulate the entire hydrograph well.

3.7 Discussion of Prediction Results

Table 3.4 and Figs. 3.12-3.22 indicate that in most cases hydrograph peak, its time and hydrograph shape are well predicted by the model. However, the prediction errors go high in some cases. The reason appears to be the inaccurate determination of rainfall-excess and poor synchronization between rainfall and runoff observations. Poor hydrograph prediction is particularly evident for complex rainfall distribution, indicating thereby that the model is sensitive to spatial distribution of rainfall as illustrated in Figs. 3.20 and 3.21. The best predicted hydrographs were found for the events which approximated a normal distribution and when the rainfall-excess was large.

Accurate determination of rainfall-excess is a major problem in rainfall-runoff modeling (Singh, 1976; Shelburne and Singh, 1976). In this study rainfall-excess was determined by substracting infiltration from rainfall. Philip's equation (Philip, 1957) was used to determine infiltration. There are two parameters in this equation. These parameters depend on soil characteristics and initial soil moisture content. In most cases these parameters cannot be estimated accurately.

The other sources of error might be inaccurate measurement of rainfall and inaccurate estimation of mean areal rainfall. Although a central raingage might represent the mean areal rainfall for a small watershed, this, however, is not always true. This causes errors in the optimization. The gaged stream flow, which is used in comparison with the predicted runoff, may also contain errors. Some errors may be due to fixing the values of parameters x and n and determination of k_1, k_2, and k_3 by optimization.

Table 3.3. Optimized parameters of nonlinear hydrologic cascade.

Watershed Name and Location	Watershed	Number of Events Used in Optimization	Optimized Parameters			Objective Function
			k_1	k_2	k_3	
Riesel(WACO), Texas	C	9	0.01	0.1885	0.1	0.7107
	D	8	0.0064	0.0507	0.1	2.1113
	G	8	0.002	4.6758	0.1	0.0943
	SW-12	10	0.2151	2.8752	0.2	0.2644
	SW-17	9	0.0753	2.9470	0.2	10.7572
	W-1	5	0.1107	0.1881	0.2	2.4099
	W-2	8	0.0427	0.2184	0.1	0.5318
	W-6	8	0.0452	1.1088	0.1	8.4204
	W-10	9	0.0508	2.4812	0.3	12.5929
	Y	8	0.0192	0.8077	0.6	6.5005
	Y-2	9	0.0196	4.7117	5.4	0.7334
	Y-4	9	0.0192	3.5646	0.4	0.5009
	Y-6	9	0.0150	6.0058	5.3	8.3673
	Y-7	9	0.0384	4.0105	0.9	5.0492
	Y-8	5	0.1127	0.2012	0.3	5.3155
	Y-10	5	0.1045	0.1874	0.2	2.0791
Coshocton, Ohio	5	7	3.77	7.14	0.02	0.2814
	92	8	5.6	0.19	0.02	0.1779
	95	7	0.0271	0.3119	0.03	0.2789
	97	9	1.7627	0.0551	0.01	0.4541
	177	9	8.7690	0.1205	0.04	0.1009
McCredie, Missouri	W-1	9	0.0248	2.9309	0.1	1.5632
Watkinsville, Georgia	W-1	9	0.0777	3.5816	0.2	18.6779
Fennimore, Wisconsin	W-1	10	0.0772	2.9436	0.2	0.8152
Ralston Creek, Iowa		10	0.0088	0.5955	0.5	2.1323
Hastings, Nebraska	2-H	10	0.1207	1.7874	2.2	7.7348
	4-H	10	0.1800	3.78	4.3	13.9757
	W-3	10	0.0242	0.5955	0.5	2.1323
	W-8	7	0.0040	0.7471	0.7	0.1437
	W-11	10	0.0029	0.5687	0.02	1.7775

Table 3.3,(continued)

Watershed Name and Location	Watershed	Number of Events Used in Optimization	Optimized Parameters			Objective Function
			k_1	k_2	k_3	
Oxford, Mississippi	W-5	6	0.0148	0.6162	0.1	0.1603
	W-35	4	0.0051	0.7313	1.1	0.1005
	W-10	5	0.0141	0.5850	0.4	0.0812
	W-24	5	0.0068	1.0636	0.9	0.3866
	WC-1	7	0.1510	2.0277	2.4	8.1092
	WC-2	9	0.2460	3.25	0.2	13.4011
	WC-3	9	0.1665	2.0692	2.3	20.3453
	WP-4	9	0.1480	2.6946	1.2	3.7453

Table 3.4. Comparison of observed and predicted hydrograph peak and its time.

Watershed Name	Date	Observed Hydrograph Peak (cm/hr)	Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Observed Hydrograph Peak Time (min)	Predicted Hydrograph Peak Time (min)	Relative Error (%)
Hastings 2-H	8-11-39	2.8194	0.9774	65.33	10.0	10.0	0
	5-20-49	0.7036	0.1351	80.79	3.0	20.0	-566.66
	6-12-65	8.8138	7.9924	9.32	9.0	24.0	-166.66
	6-12-65	2.1565	2.1880	-1.46	13.0	15.0	-23.08
	6-29-65	2.0676	1.5737	23.89	9.0	9.0	0
	8-7-46	3.7592	1.5174	59.64	7.0	10.0	-42.86
	8-7-42	2.5197	1.5120	39.99	10	23.0	-130.00
	6-16-50	0.1735	0.0040	97.69	2.0	43.0	-2050.0
	9-7-42	3.5306	1.8000	49.02	15.0	12.0	20.0
	9-5-46	2.2885	1.3593	40.60	9.0	12.0	-33.33
Hastings 4-H	8-11-39	4.5212	3.4254	24.24	5.0	6.0	-20.00
	6-20-42	5.8166	5.0194	13.71	8.0	11.0	-37.50
	9-5-46	3.8862	2.1212	45.42	12.0	8.0	33.33
	6-1-51	6.7564	6.3084	6.63	123.0	131.0	-6.50
	7-13-52	9.1948	9.9094	-7.77	21.0	19.0	9.52
	6-12-58	0.9195	0.5000	45.62	14.0	11.0	21.43
	6-12-65	9.7028	8.2274	15.21	19.0	15.0	21.05
	6-12-65	6.1468	4.9645	19.23	7.0	12.0	-71.42
Riesel (Waco)							
W-1	3-12-53	2.8702	3.1407	-9.42	59.0	26.0	55.93
	6-23-59	4.8006	4.6877	2.35	87.0	111.0	-27.58
	3-29-65	5.8750	7.9949	-36.08	101.0	71.0	29.70
Y-8	5-13-57	5.6642	5.5050	2.81	19.0	28.0	-47.36
	3-23-65	5.7122	6.9353	-21.41	61.0	73.0	-19.67
	6-18-61	0.1986	0.4147	-108.80	100	72.0	28.00
Y-10	4-24-57	6.8580	6.7709	1.27	35.0	36.0	-2.85
	5-13-57	4.8514	4.4727	7.81	26.0	33.0	-26.92
	6-9-62	1.0008	1.3655	-36.45	38.0	38.0	0
	3-23-65	6.9248	7.8860	-13.88	69.0	81.0	-17.39

Table 3.4. (continued)

Watershed Name	Date	Observed Hydrograph Peak (cm/hr)	Predicted Hydrograph Peak (cm/hr)	Relative Error (%)	Observed Hydrograph Peak Time (min)	Predicted Hydrograph Peak Time (min)	Relative Error (%)
Hastings W-8	6-7-42	0.2718	0.1693	37.69	64.0	48.0	25.00
	5-11-44	0.4521	0.0819	81.89	75.0	112.0	-49.33
	7-6-45	0.4267	0.2626	38.46	71.0	55.0	22.54
	6-1-51	1.1328	1.2051	-6.38	154.0	168.0	-9.09
	6-26-52	0.4491	0.4790	-6.65	106.0	88.0	16.98
	7-6-52	0.2195	0.0720	67.19	62.0	85.0	-37.10
	6-16-57	0.6579	0.9788	-48.78	212.0	149.0	29.72
Coshocton 97	4-25-61	1.3919	1.6009	-15.01	130.0	108.0	16.92
	6-18-40	0.2946	0.3684	-25.05	122.0	57.0	53.28
	3-9/10-64	0.4729	0.6060	-28.12	394.0	393.0	0.25
	6-12-57	0.6604	0.5747	12.97	150.0	80.0	46.67
	6-28/29-40	0.5182	0.6941	-33.96	150.0	113.0	24.67
	6-4-41	0.9144	0.6623	27.57	156.0	62.0	60.26
	9-23-45	0.8204	0.8292	-1.07	120.0	139.0	-15.83
	7-11-46	0.5359	0.3902	27.19	212.0	100.0	52.83
	6-28-57	1.8390	1.5310	16.75	416.0	379.0	8.89

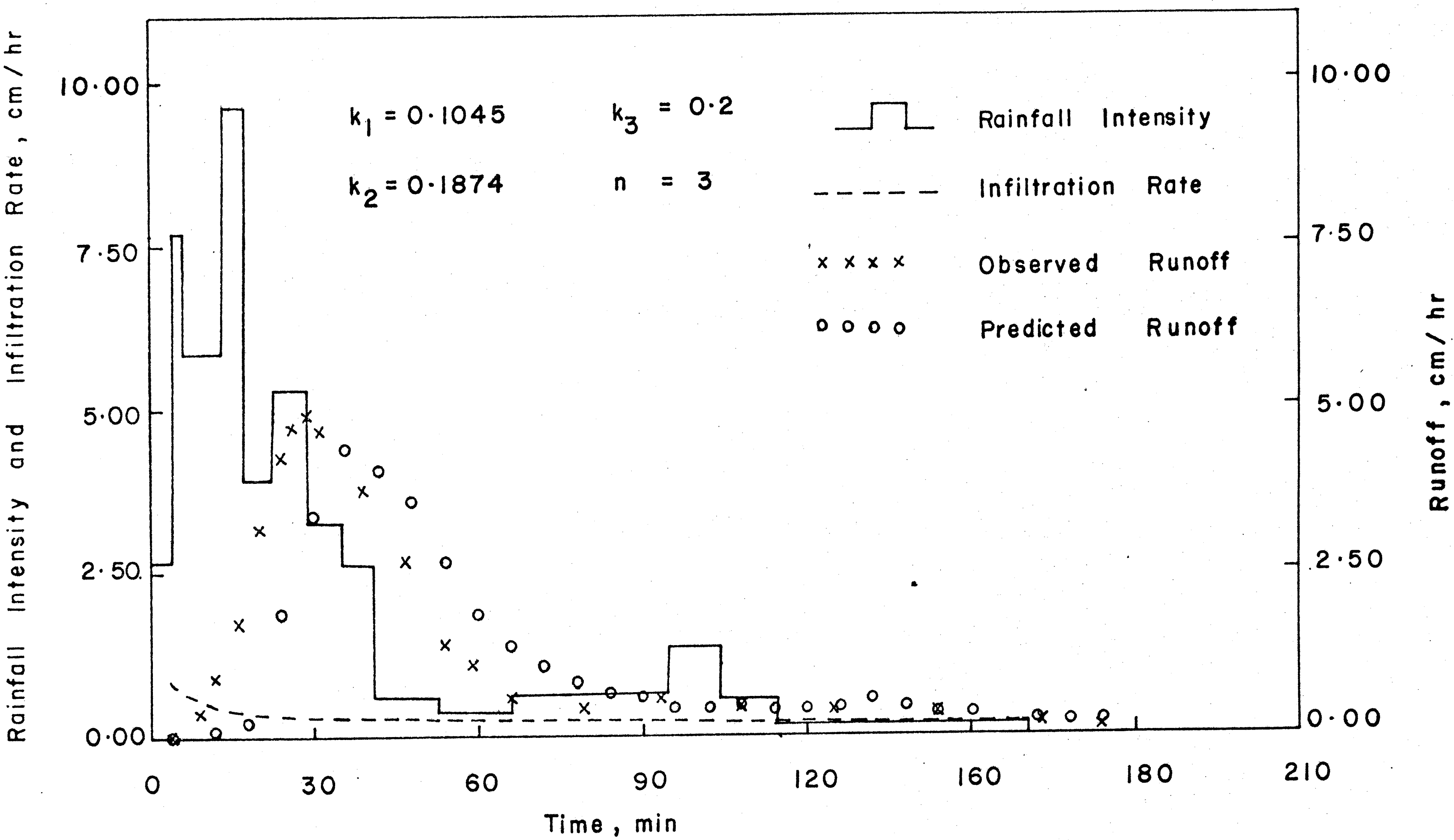

Fig. 3.12. Hydrograph prediction for rainfall event of 5-13-57 on watershed Y-10, Riesel (Waco), Texas.

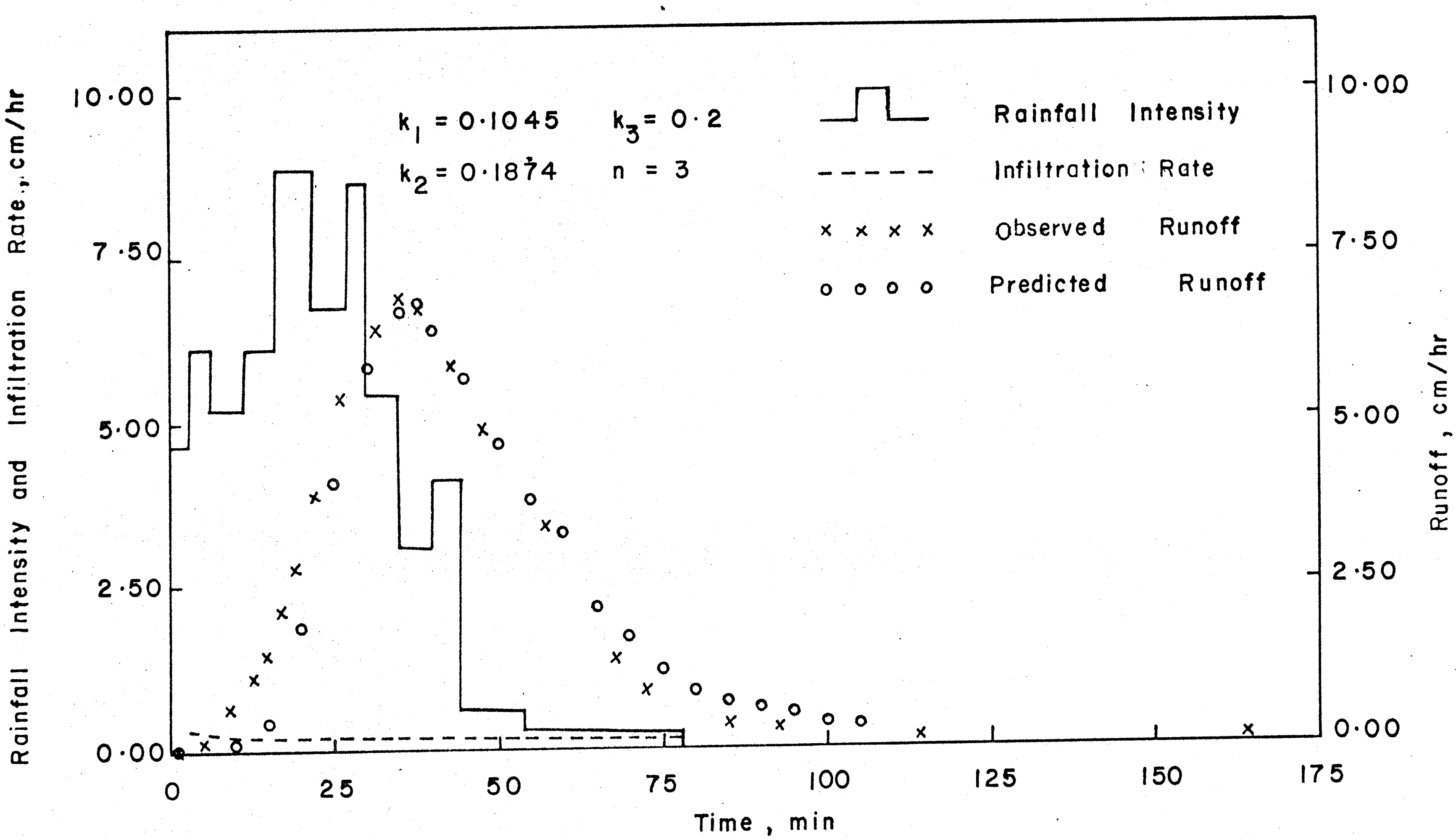

Fig. 3.13. Hydrograph prediction for rainfall event of 4-24-57 on watershed Y-10, Riesel (Waco), Texas.

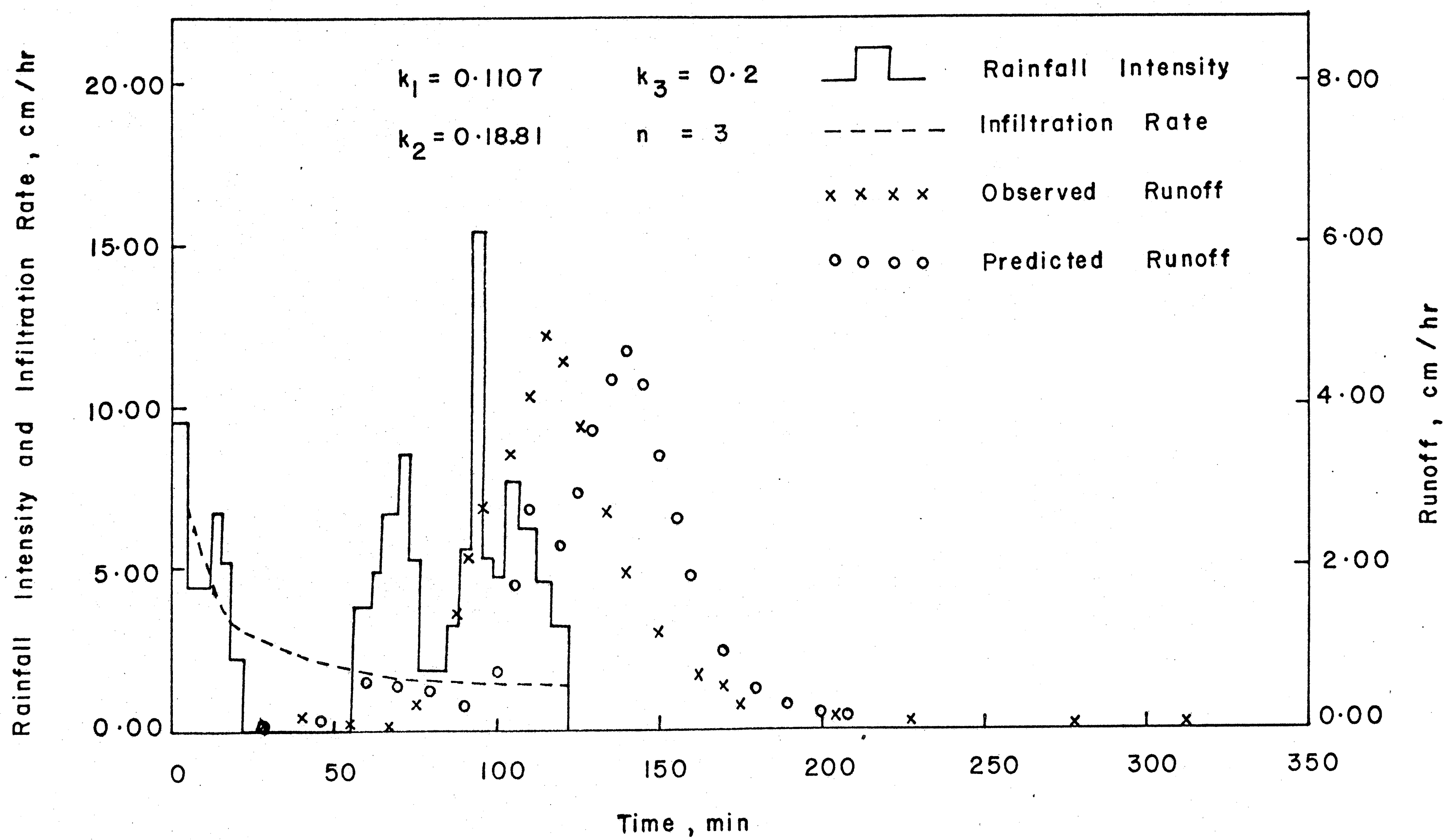

Fig. 3.14. Hydrograph prediction for rainfall event of 6-23-59 on watershed W-1, Riesel (Waco), Texas.

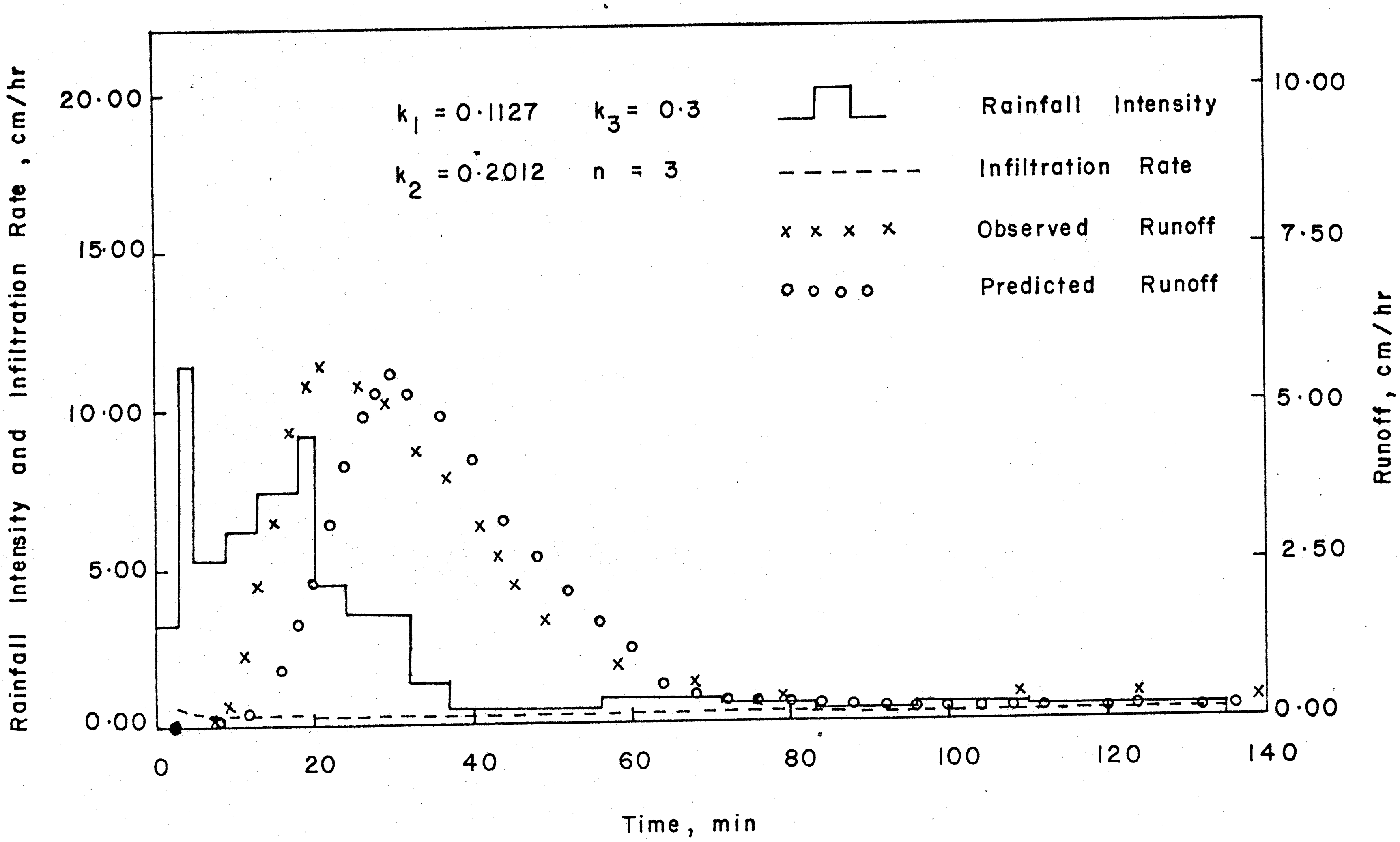

Fig. 3.15. Hydrograph prediction for rainfall event of 5-13-57 on watershed Y-8, Riesel (Waco), Texas.

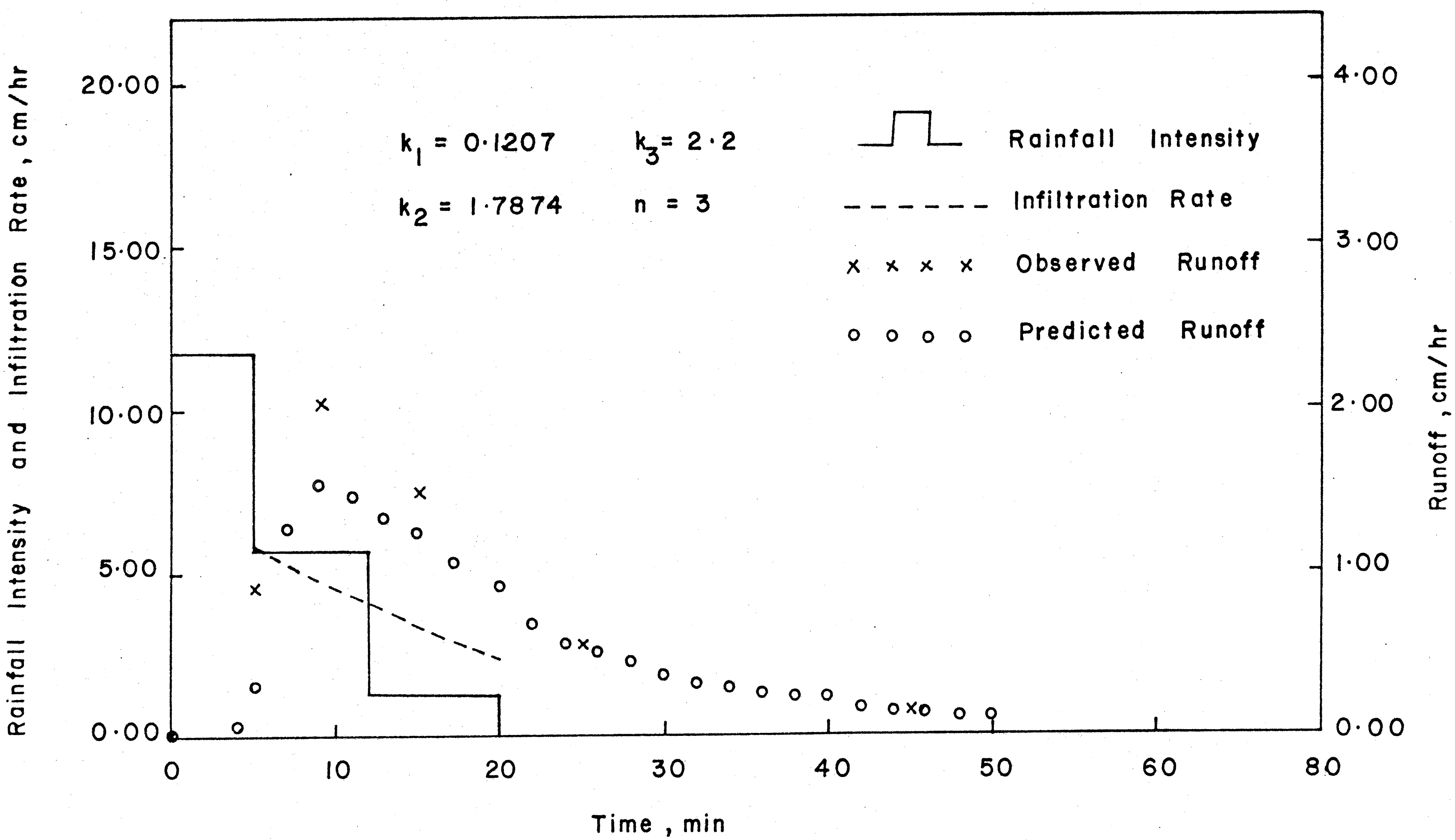

Fig. 3.16. Hydrograph prediction for rainfall event of 6-29-65 on watershed 2-H, Hastings, Nebraska.

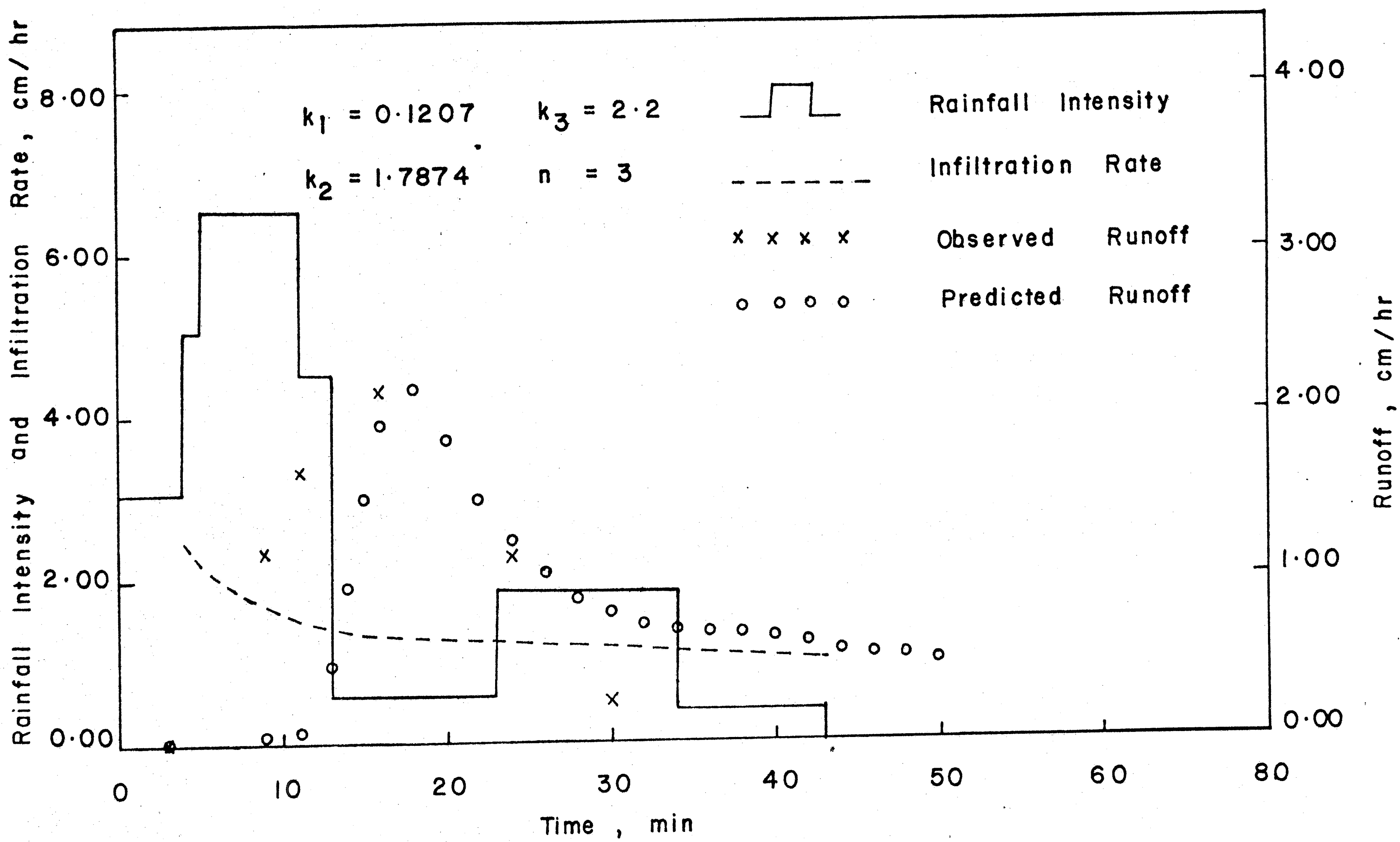

Fig. 3.17. Hydrograph prediction for rainfall event of 6-12-65 on watershed 2-H, Hastings, Nebraska.

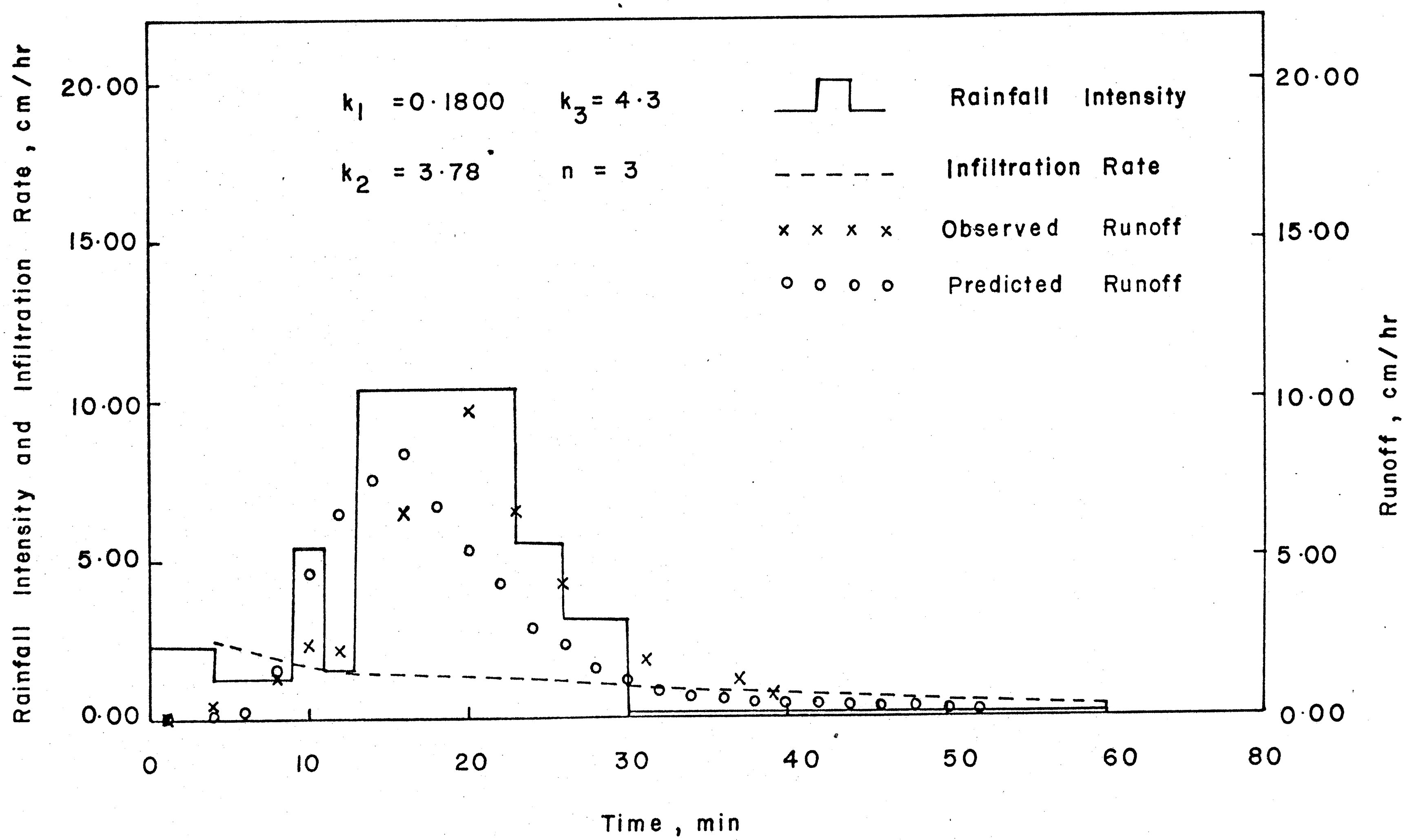

Fig. 3.18. Hydrograph prediction for rainfall event of 6-12-65 on watershed 4-H, Hastings, Nebraska.

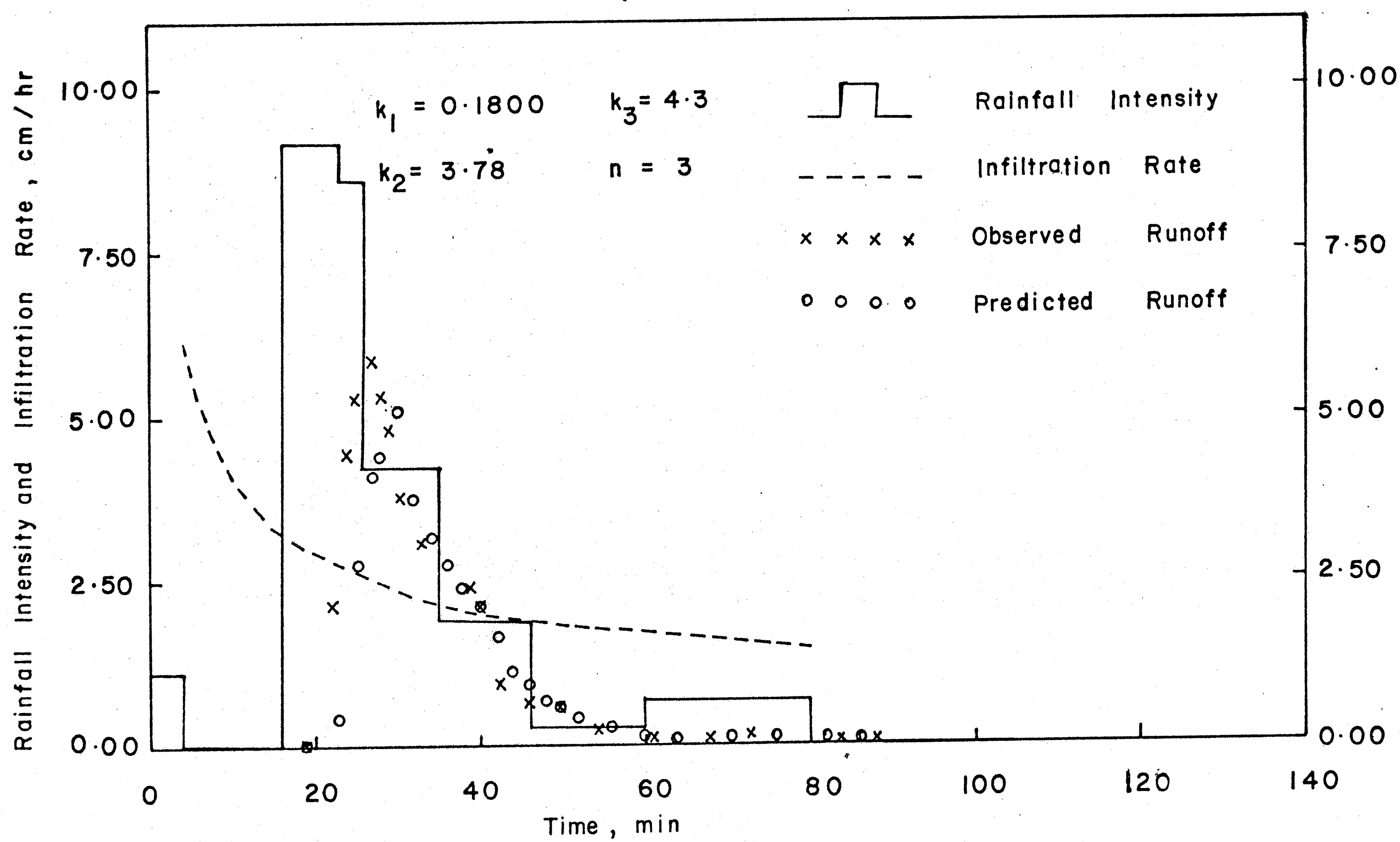

Fig. 3.19. Hydrograph prediction for rainfall event of 6-20-42 on watershed 4-H, Hastings, Nebraska.

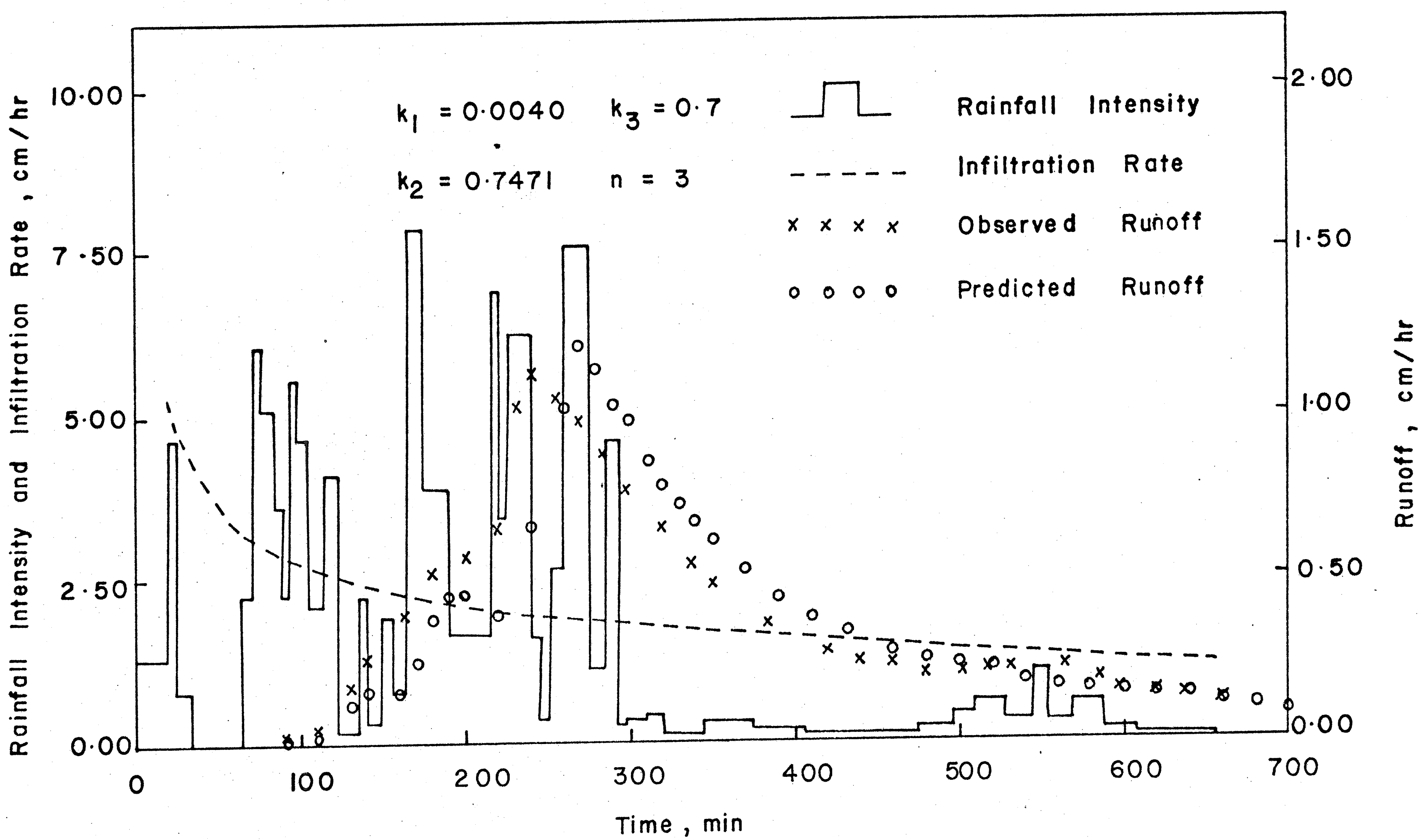

Fig. 3.20. Hydrograph prediction for rainfall event of 6-1-51 on watershed W-8, Hastings, Nebraska.

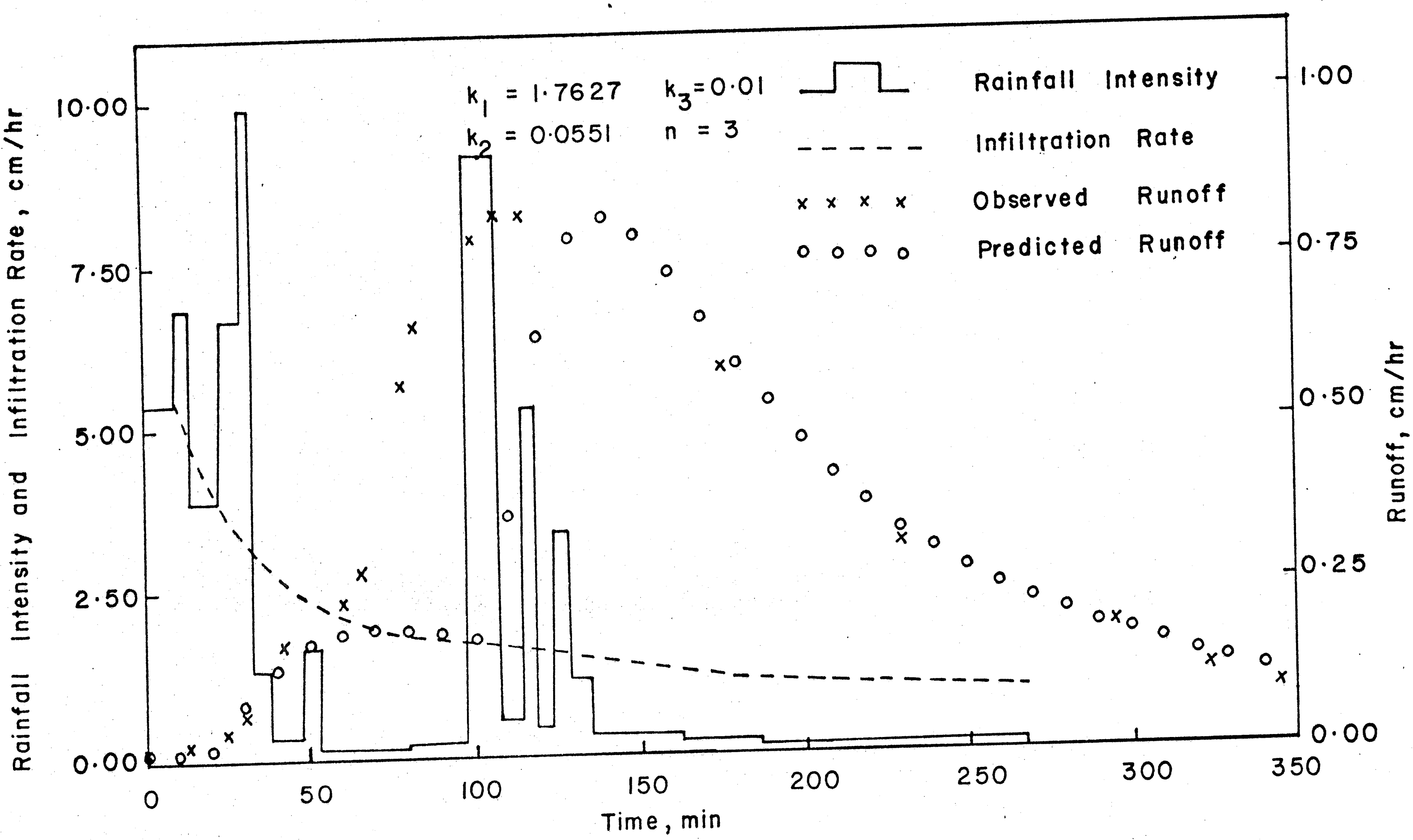

Fig. 3.21. Hydrograph prediction for rainfall event of 9-23-45 on watershed 97, Coshocton, Ohio.

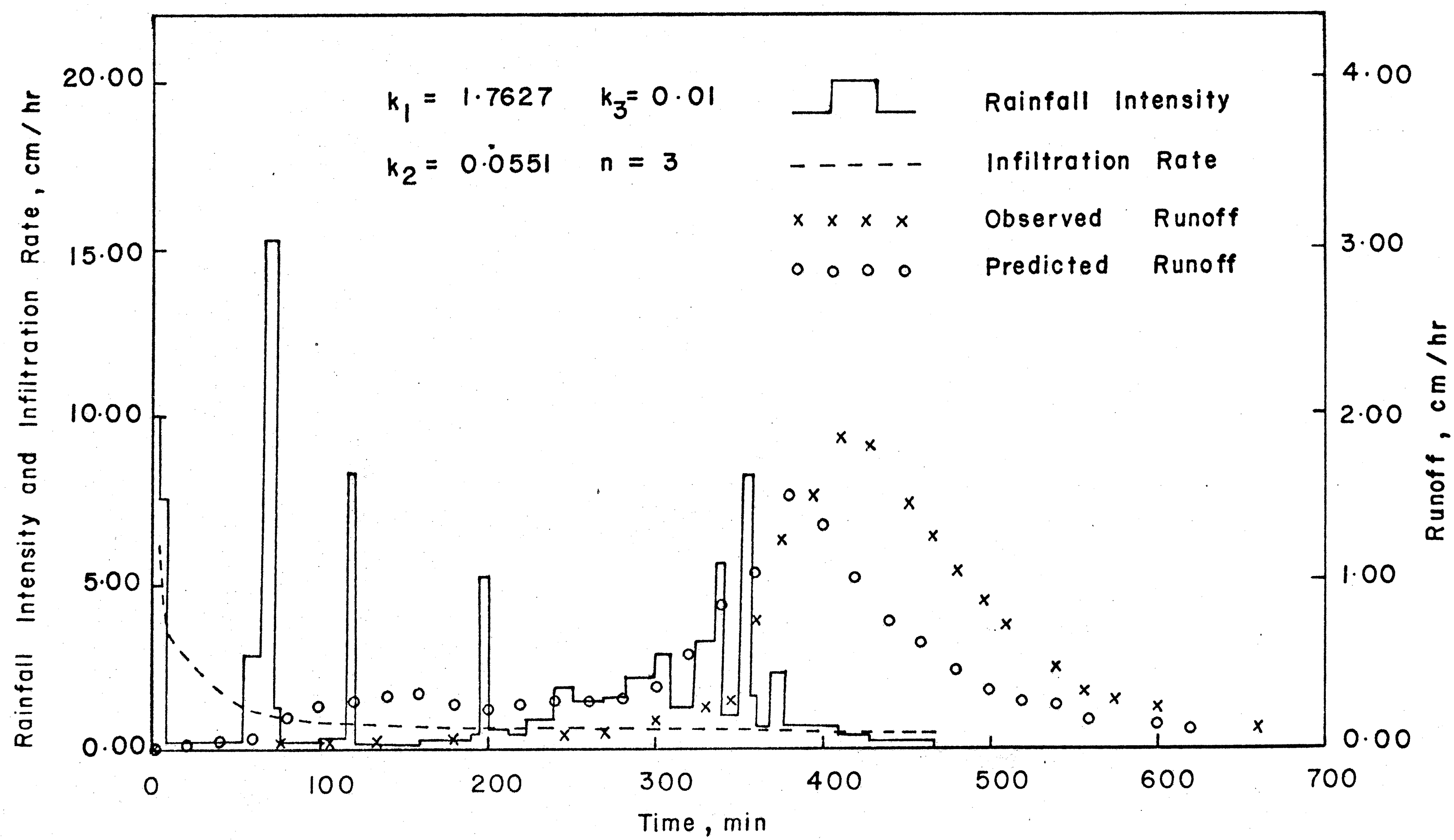

Fig. 3.22. Hydrograph prediction for rainfall event of 6-28-57 on watershed 97, Coshocton, Ohio.

CHAPTER 4

PARAMETER ESTIMATION

This chapter explores the possibility of estimating the model parameters from physically measurable watershed characteristics. A regression and correlation analysis is performed to examine this possibility.

4.1 Watershed Characteristics

The topographic characteristics of the watersheds, used in this study, included average watershed slope, slope of mainstream, watershed area, width of watershed, length of mainstream, drainage density, shape factor and stream order. These watershed characteristics were considered to affect runoff hydrograph (Sherman, 1932; Gray, 1961; Black, 1975). The topographic characteristics of all 38 watersheds and the optimized parameters are given in Table 4.1.

4.1.1 Average Watershed Slope (SLOPE)

An average watershed slope was obtained by weighting each slope with respect to the proportion of the watershed area comprising this slope. Average watershed slope for each watershed was obtained directly from USDA hydrologic data publications.

4.1.2 Average Mainstream Slope (CSLOPE)

The slope of main stream was taken as the slope of the principal drainage channel, and was obtained by dividing the difference in elevations of its upstream and downstream points by its length.

4.1.3 Watershed Area (AREA), Width (WIDTH) and Length of Mainstream (XLR)

Watershed area and its width were obtained directly from the USDA hydrologic data publications. The length of mainstream was defined as the distance from the most remote point on the watershed to the stream gauging station.

Table 4.1. Watershed characteristics and optimized parameters.

Location of Watershed	Watershed Characteristics								Model Parameters		
	Area (Hectares)	Width (Km)	Length of main-stream (Km)	Slope (%)	Shape	Drainage Density (m^{-1})	Stream Order	Slope of main-stream (%)	k_1	k_2	k_3
Coshocton, Ohio											
5	141.24	0.671	2.415	15.505	3.2431	0.0063	3	1.770	3.77	7.14	0.02
92	372.32	0.518	2.898	15.400	1.7716	0.0007	3	2.368	5.6	0.19	0.02
95	104.01	3.460	5.635	16.890	2.3973	0.0018	3	1.353	0.0271	0.3119	0.03
97	1,853.53	3.892	9.016	17.210	3.4444	0.0010	3	1.014	1.7627	0.0551	0.01
177	30.59	0.465	0.854	15.325	1.8707	0.0031	2	5.357	8.7690	0.1205	0.04
Fennimore, Wisconsin											
W-1	133.55	0.914	1.768	5.975	1.8389	0.0022	2	1.724	0.0772	2.9436	0.2
Hastings, Nebraska											
2-H	1.38	0.076	0.189	6.135	2.0395	0.0078	2	2.581	0.1207	1.7874	2.2
4-H	1.47	0.107	0.162	5.960	1.3921	0.0078	1	4.528	0.18	3.78	4.3
W-3	194.66	1.207	2.720	5.305	2.9861	0.0039	3	0.430	0.0242	0.5955	0.5
W-8	844.20	1.811	7.953	5.500	5.8850	0.0032	4	0.430	0.004	0.7471	0.7
W-11	1,412.40	2.012	11.673	5.576	7.5793	0.0032	4	0.430	0.0029	0.5687	0.02
McCredie, Missouri											
W-1	62.32	0.415	1.036	3.420	1.3541	0.0028	3	0.176	0.0248	2.9309	0.1
Oxford, Mississippi											
W-5	457.31	2.347	2.415	7.720	1.0013	0.0009	1	0.630	0.0148	0.6162	0.1
W-10	2,238.00	4.829	6.440	10.140	1.4555	0.0010	3	0.700	0.0141	0.5850	0.4
W-24	206.80	1.207	2.012	12.420	1.5382	0.0038	3	3.030	0.0068	1.0636	0.9
W-35	3,055.48	3.621	8.855	7.550	2.0155	0.0014	4	0.689	0.0051	0.7313	1.1
WC-1	1.57	0.114	0.149	7.760	1.1163	0.0248	3	6.222	0.1510	2.0277	2.4
WC-2	0.59	0.084	0.092	7.130	1.1197	0.0146	1	6.667	0.2460	3.25	0.2
WC-3	0.65	0.061	0.107	5.930	1.3725	0.0122	1	4.286	0.1665	2.0692	2.3
WP-4	1.22	0.076	0.134	9.380	1.1602	0.0138	1	5.682	0.1480	2.6946	1.2

Table 4.1. (continued)

Location of Watershed	Watershed Characteristics								Model Parameters		
	Area (Hectares)	Width (Km)	Length of main-stream (Km)	Slope (%)	Shape	Drainage Density (m^{-1})	Stream Order	Slope of main-stream (%)	k_1	k_2	k_3
Riesel(WACO), Texas											
C	234.32	1.402	2.366	2.040	1.8761	0.0023	3	0.570	0.01	0.1885	0.1
D	449.22	1.982	3.567	2.100	2.2246	0.0022	3	0.510	0.0064	0.0507	0.1
G	1,772.59	2.592	7.829	2.055	2.7160	0.0020	4	0.389	0.002	4.6758	0.1
W-1	71.23	0.610	1.646	2.180	2.9887	0.0029	2	0.833	0.1107	0.1881	0.2
W-2	52.61	0.823	0.945	2.550	1.3335	0.0034	2	1.290	0.0427	0.2184	0.1
W-6	17.12	0.457	0.445	2.020	0.9090	0.0129	2	1.370	0.0452	1.1088	0.1
W-10	7.97	0.305	0.323	1.620	1.0289	0.0041	1	1.500	0.0508	2.4812	0.3
Y	125.05	0.915	1.537	2.405	1.4830	0.0026	2	0.990	0.0192	0.8077	0.6
Y-2	53.42	0.854	1.000	2.585	1.4702	0.0027	2	1.220	0.0196	4.7117	5.4
Y-4	32.34	0.595	0.610	2.850	0.9031	0.0011	2	1.750	0.0192	3.5646	0.4
Y-6	8.46	0.259	0.338	3.225	1.0634	0.0015	1	1.804	0.0150	6.0058	5.3
Y-7	16.19	0.381	0.543	1.865	1.4289	0.0017	1	0.840	0.0384	4.0105	0.9
Y-8	8.42	0.183	0.244	1.945	0.5550	0.0016	1	1.100	0.1127	0.2012	0.3
Y-10	8.50	0.381	0.338	2.375	1.0584	0.0038	1	1.300	0.1045	0.1874	0.2
SW-12	1.20	0.119	0.116	3.950	0.8770	0.0171	1	2.894	0.2151	2.8752	0.2
SW-17	1.21	0.122	0.116	1.830	0.8712	0.0066	1	1.579	0.0753	2.9470	0.2
Ralston Creek, Iowa	781.07	1.463	5.796	10.250	3.3780	0.0085	4	0.550	0.0088	0.3365	0.1
Watkinsville, Georgia	7.77	0.274	0.381	10.625	1.4763	0.0114	2	4.161	0.0777	3.5816	0.2

4.1.4 Drainage Density (DD)

Drainage density was defined by the accumulative length of streams divided by the total drainage area:

$$DD = \frac{\Sigma L}{A} \qquad (4\text{-}1)$$

where DD = drainage density;

L = length of a stream; and

A = area of drainage basin.

4.1.5 Shape Factor (SHAPE)

The shape factor of Chorley, Malm and Pagorzelski (1957) was used in this study. It can be defined as:

$$SHAPE = \frac{\Pi L^2}{4A} \qquad (4\text{-}2)$$

where L = length of the mainstream; and

A = area of the watershed.

Shape factor is a dimensionless parameter.

4.1.6 Stream Order (SO)

The stream order developed by Strahler (1957) was applied in this study. Strahler suggested that the channel network map includes all intermittent and permanent flow lines located in clearly defined valleys. The smallest finger-tip tributaries are designated order 1. Where two channels of order 1 join, a channel segment of order 2 is formed; and so on. The mainstream, through which all discharge passes, has therefore the highest order.

4.2 Correlation of Model Parameters

To correlate parameters k_1, k_2, and k_3 with topographic characteristics of the watersheds, the UCLA Biomedical Statistical Package (BMD 02R) was used to perform regression and correlation analysis. First, a multiple linear regression was performed to correlate optimized

k_1, a dependent variable, with topographic characteristics, independent variables. The correlation coefficient was 0.7742 and standard error of estimate 1.2496. The regression equation for k_1 can be represented as:

$$k_1 = -0.30127 + 0.00113*AREA - 0.98367*WIDTH - 0.2448*XLR + 0.26045*SLOPE + 0.17566*SHAPE - 171.4197*DD + 0.28804*SO + 0.19733*CSLOPE \quad (4\text{-}3)$$

To check further the reliability of this equation the optimized k_1 was plotted against k_1 estimated from Eq. (4-3) as shown in Fig. 4.1. It shows wide scattering of points around the theoretical line of fit and thus a poor relationship between parameter k_1 and watershed characteristics.

Then the multiple linear regression was performed to correlate the parameters k_2 and k_3 with watershed characteristics. The correlation coefficient for k_2 was 0.4075 and standard error of estimate 1.8641. The correlation coefficient for k_3 was 0.4065 and standard error of estimate 1.4123. The linear regression equations for k_2 and k_3 can be respectively written as:

$$k_2 = 2.75611 + 0.00111*AREA - 0.73322*WIDTH - 0.19508*XLR + 0.02899*SLOPE + 0.11413*SHAPE - 0.14540*SO - 0.03928*CSLOPE \quad (4\text{-}4)$$

$$k_3 = 0.65929 + 0.00063*AREA + 0.17000*WIDTH - 0.30879*XLR - 0.08221*SLOPE + 0.32046*SHAPE - 52.04816*DD + 0.35307*CSLOPE \quad (4\text{-}5)$$

To check further the suitability of the above equations k_2 computed from Eq. (4-4) was plotted against optimized k_2 as shown in Fig. 4.2. The plot shows considerable scattering of points around the theoretical line of fit and therefore a poor relationship for k_2.

Figure 4.3 shows optimized k_3 versus k_3 computed from Eq. (4-5). It shows considerable scattering of points around the theoretical line of

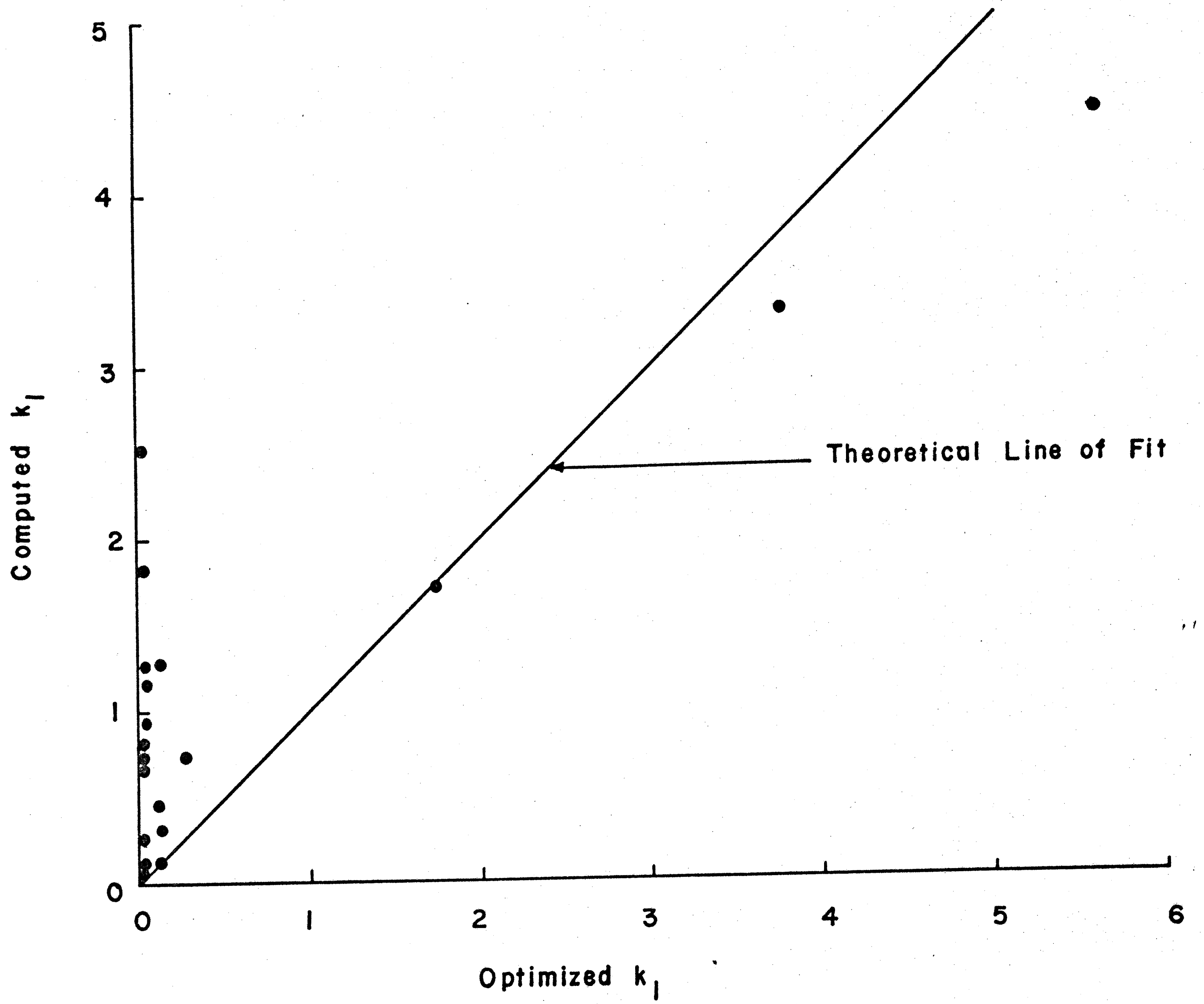

Fig. 4.1. Optimized k_1 versus computed k_1 from linear regression Eq. (4-3).

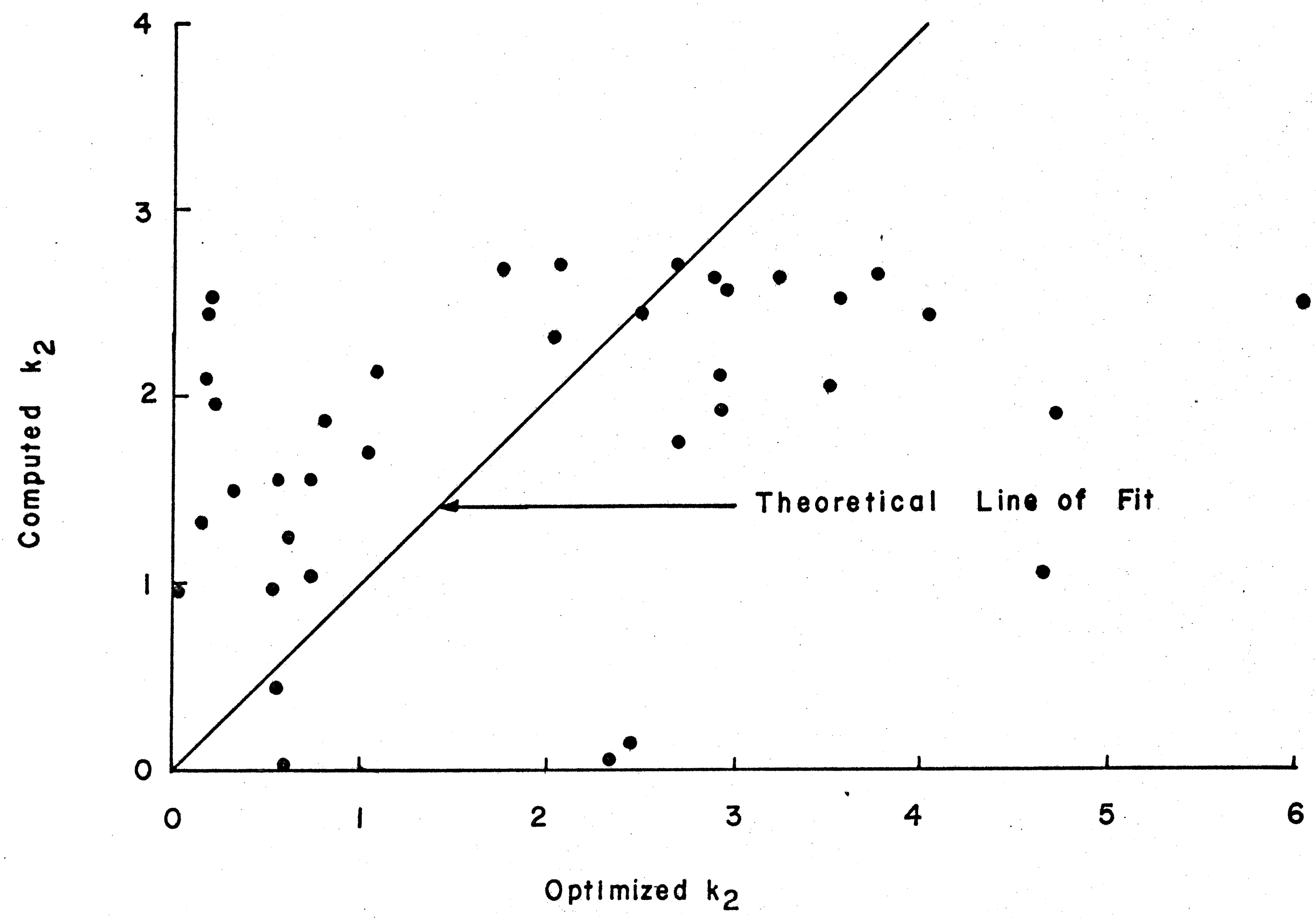

Fig. 4.2. Optimized k_2 versus computed k_2 from linear regression Eq. (4-4).

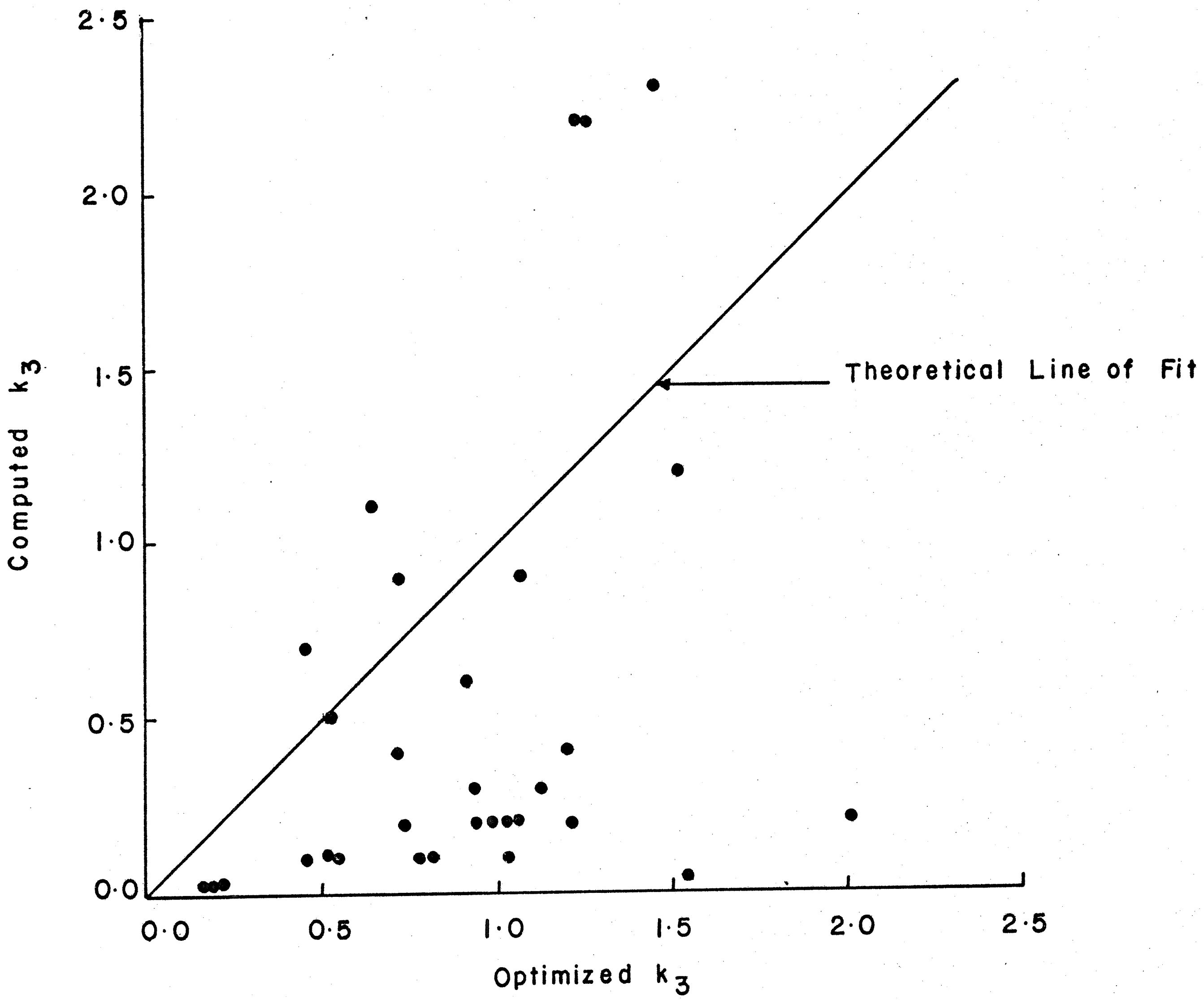

Fig. 4.3. Optimized k_3 versus computed k_3 from linear regression Eq. (4-5).

fit and therefore a poor relationship between k_3 and watershed characteristics.

In order to improve the correlation of model parameters all variables, independent as well as dependent, were transformed logarithmically to the base 10. Then the regression analyses were performed. The correlation coefficient for k_1 was 0.7264 and standard error of estimate 0.6699 where the highest correlation coefficient was contributed by stream order, then by length of mainstream, shape factor, area, drainage density, width, slope and channel slope respectively. The regression equation for k_1 can be written as:

$$\log k_1 = -5.14689 + 0.4502 \log AREA - 0.95677 \log WIDTH - 0.80706 \log LENGTH + 1.0594 \log SLOPE + 0.77707 \log SHAPE - 0.80188 \log DD - 0.19171 \log SO + 0.6892 \log CSLOPE \quad (4\text{-}6)$$

To check the improvement in the correlation log of optimized k_1 was plotted against log of computed k_1 from Eq. (4-6) as shown in Fig. 4.4. This figure shows considerable scattering of points around the theoretical line of fit. Although improved from before, the relationship is still poor.

The correlation coefficient for k_2 was 0.4802 and standard error of estimate 0.5533 where the highest correlation was with width and then with drainage density, average channel slope and shape factor respectively. Area, average length of mainstream, average slope of watershed and stream order were deleted in the analysis. The regression equation for k_2 can be written as:

$$\log k_2 = 0.65164 - 0.3860 \log WIDTH - 0.23575 \log SHAPE + 0.28078 \log DD - 0.15567 \log CSLOPE \quad (4\text{-}7)$$

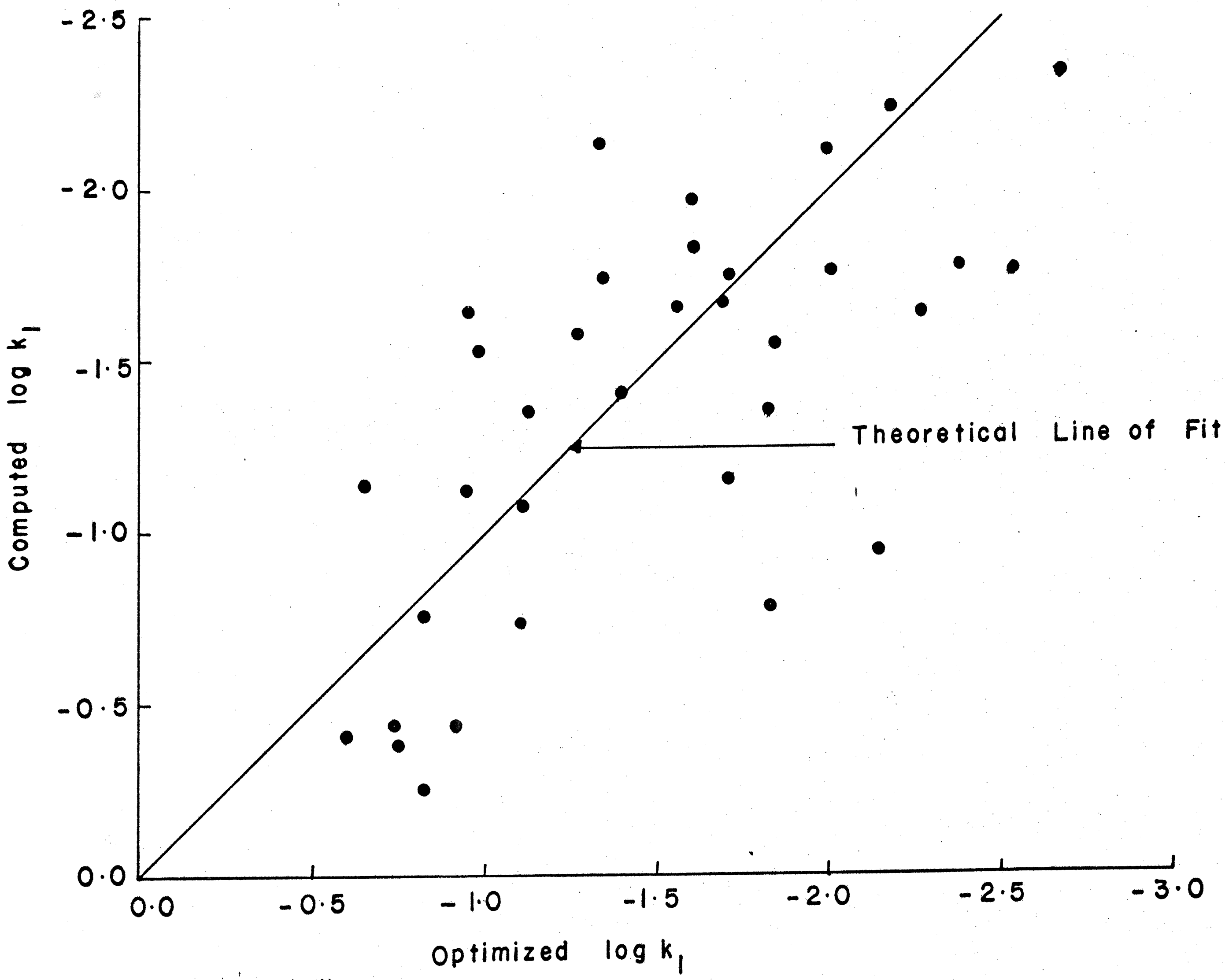

Fig. 4.4. Optimized log k_1 versus computed log k_1 from log regression Eq. (4-6).

Figure 4.5 shows the plot of optimized log k_2 versus log k_2 computed from Eq. (4-7) and indicates that the correlation is better but the scattering of points is still wide and therefore the relationship for k_2 poor.

For k_3 the correlation coefficient of 0.5371 and standard error of estimate of 0.6691 were obtained where the highest correlation was with average channel slope and then with drainage density, width, shape factor, area, stream order, average slope of watershed and length of mainstream respectively. The regression equation for k_3 can be written as:

$$\log k_3 = -2.81893 + 0.61483 \log AREA + 0.83323 \log WIDTH - 2.9685 \log XLR - 0.2086 \log SLOPE + 1.5370 \log SHAPE - 0.39749 \log DD + 0.85542 \log SO + 0.06763 \log CSLOPE \quad . \qquad (4\text{-}8)$$

The plot of optimized log k_3 versus log k_3 computed from Eq. (4-8) is shown in Fig. 4.6. The correlation for k_3 now has a small improvement over linear correlation.

After the regression analyses with watershed characteristics correlation between the parameters was considered. The optimized k_1 was plotted versus optimized k_2 as shown in Fig. 4.7. It shows considerable scattering of points. The linear regression analysis was performed to find correlation coefficient between k_1 and k_2 where k_1 was independent variable and k_2 dependent variable. The correlation coefficient was 0.0674 and standard error of estimate 1.8592. This shows that there is practically no correlation between parameters k_1 and k_2.

The plots of optimized k_2 and k_3 and optimized k_3 and k_1 are shown in Fig. 4.8 and 4.9 respectively. Linear regression analysis was then performed between k_2 and k_3 where k_2 was independent variable and k_3 dependent variable. The correlation coefficient was 0.4626 and standard

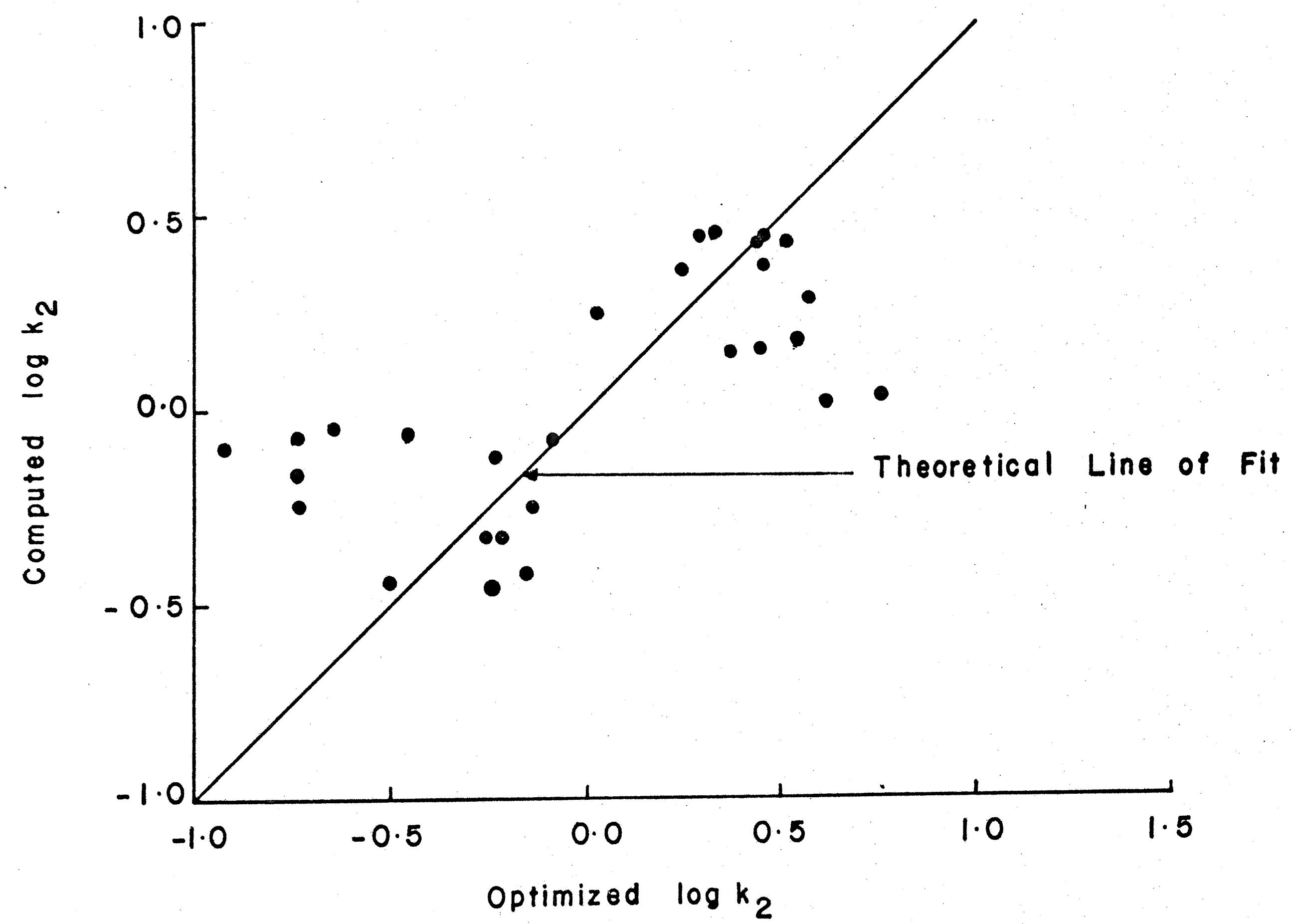

Fig. 4.5. Optimized log k_2 versus computed log k_2 from log regression Eq. (4-7).

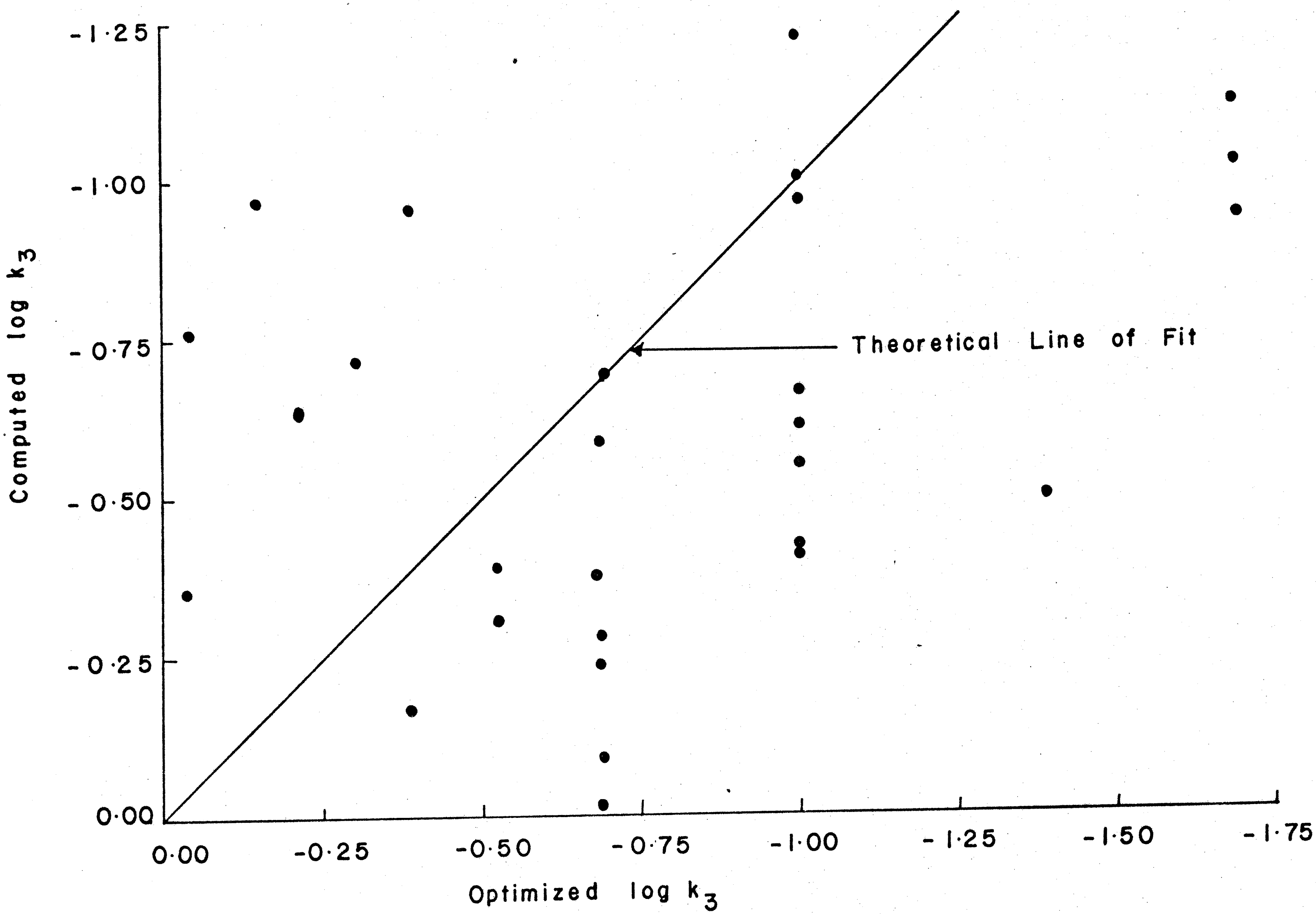

Fig. 4.6. Optimized log k_3 versus computed log k_3 from log regression Eq. (4-8).

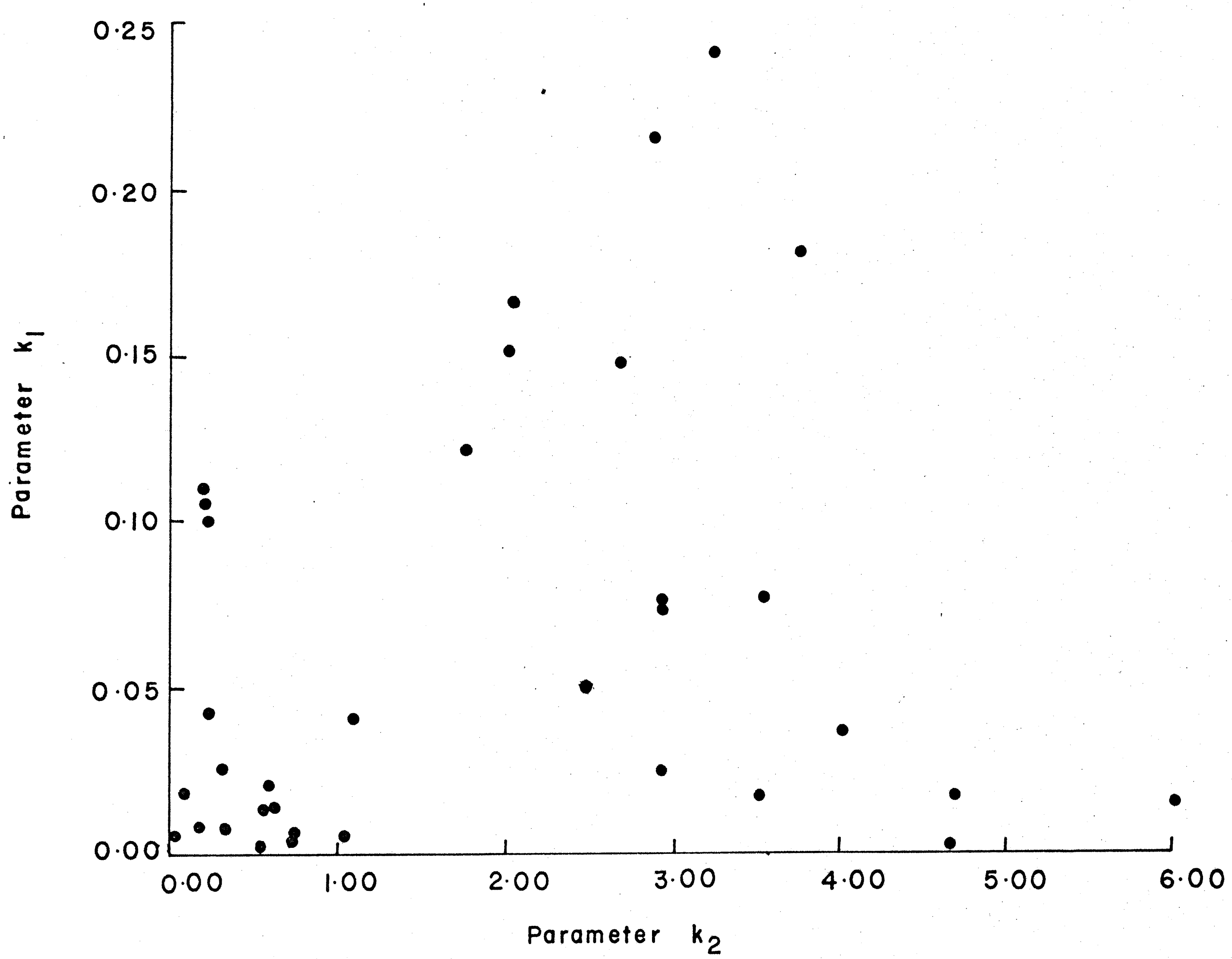

Fig. 4.7. Parameter k_1 versus parameter k_2.

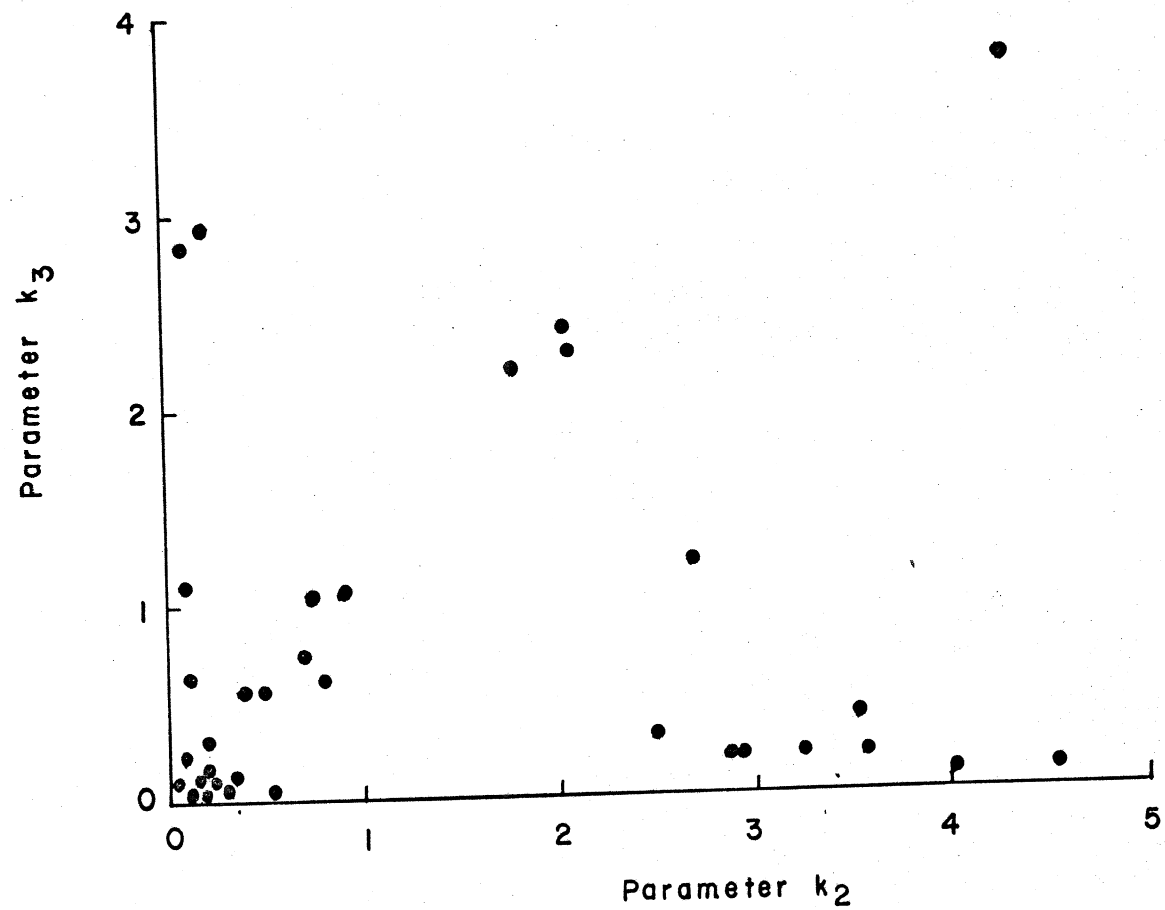

Fig. 4.8. Parameter k_2 versus parameter k_3.

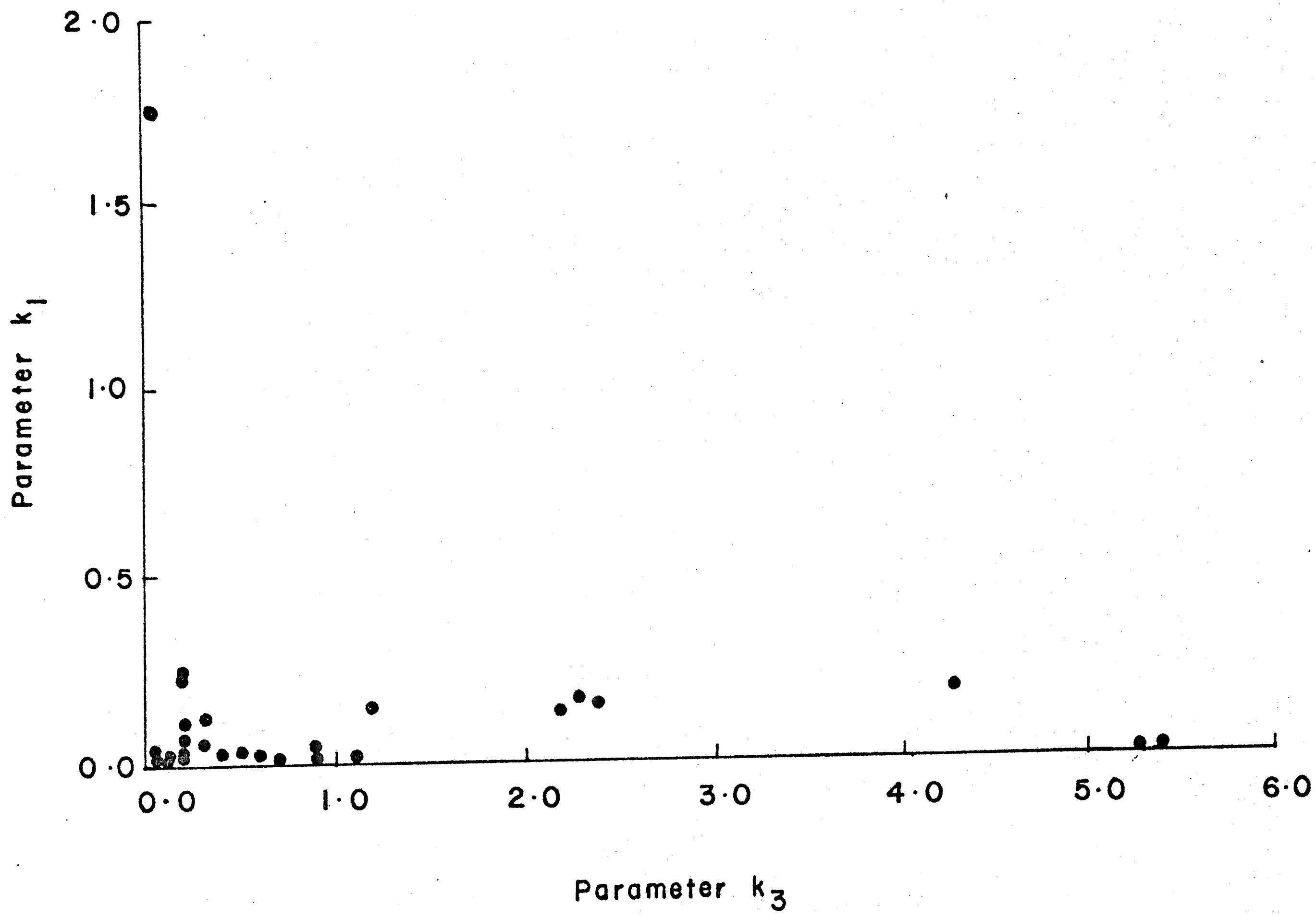

Fig. 4.9. Parameter k_3 versus parameter k_1.

error of estimate 1.2511. It indicates that k_2 and k_3 are correlated but not very strongly. Linear regression analysis was also performed between k_3 and k_1 where k_1 was dependent variable and k_3 independent variable. The correlation coefficient of 0.1699 and standard error of estimate 1.7461 were obtained. Again no significant correlation between parameters k_1 and k_3 was noticed.

4.3 Results and Discussions

Figures 4.1-4.3 show the plots of optimized k_1, k_2, and k_3 versus estimated k_1, k_2, and k_3 from linear Eqs. (4-3), (4-4) and (4-5) respectively. These figures show considerable scattering of points around the theoretical line of fit and indicate that the model parameters cannot be estimated reliably from the watershed characteristics in a linear fashion.

Figures 4.4-4.6 show the plot of the log values of optimized k_1, k_2, and k_3 versus estimated k_1, k_2, and k_3 from log regression Eqs. (4-6), (4-7) and (4-8) respectively. These figures show some improvement in correlation of parameters with watershed characteristics. The scattering of points is reduced. However, the correlations are still too poor to be used in the estimation of parameters from watershed characteristics. It appears that the model parameters not only depend on watershed characteristics but also equally depend on rainfall-excess characteristics and perhaps the interaction between them.

CHAPTER 5

CONCLUSIONS

The following conclusions can be drawn from this study.

1. The nonlinear hydrologic cascade can predict watershed runoff satisfactorily.

2. Computation time interval is important for model performance and can be taken as 1 minute.

3. Reliable correlations could not be established between the model parameters and watershed characteristics.

4. The model parameters have no correlation between themselves.

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