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OKLAHOMA CITY URBAN STORM RUNOFF QUANTITY: COMPARISON AND CALIBRATION OF PREDICTIVE METHODS

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THE UNIVERSITY OF OKLAHOMA

GRADUATE COLLEGE

OKLAHOMA CITY URBAN STORM RUNOFF QUANTITY:
COMPARISON AND CALIBRATION OF PREDICTIVE METHODS

A DISSERTATION

SUBMITTED TO THE GRADUATE FACULTY

in partial fulfillment of the requirements for the

degree of

DOCTOR OF PHILOSOPHY

BY

KEITH KIM WILLIAMS

OKLAHOMA CITY, OKLAHOMA

1979

OKLAHOMA CITY URBAN STORM RUNOFF QUANTITY:

COMPARISON AND CALIBRATION OF PREDICTIVE METHODS

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ABSTRACT

OKLAHOMA CITY URBAN STORM RUNOFF QUANTITY: COMPARISON AND CALIBRATION OF PREDICTIVE METHODS

BY: KEITH KIM WILLIAMS

MAJOR PROFESSOR: REGENTS PROFESSOR GEORGE W. REID

Six discrete event urban rainfall-runoff quantity models commonly used by federal agencies were calibrated on twenty-three events recorded by the U.S. Geological Survey on three urban basins during 1974-1975 in Oklahoma City. The models were the Rational Method (Department of Housing and Urban Development), TR-20 (Soil Conservation Service), HEC-1 (Corps of Engineers), Urban Flood Hydrograph Synthesis Model (Geological Survey), SWMM (Environmental Protection Agency), and MINICAT (National Weather Service, River Forecast Center). All the models were calibrated for peak discharge on the recorded floods, and all except the Rational Method were calibrated for runoff volume. It was found during the calibration process that antecedent soil wetness was not an influence on runoff from the basins and storms used in the study. The calibrated models were compared on how accurately they reproduced the recorded hydrographs, engineering applications, and relationships between various hydrograph parameters. It was found that each model calibrated nearly as well as the others, except that HEC-1 was a little more reliable in reproducing the recorded events than the other models, and TR-20 tends to bias, making the larger floods too large and the smaller hydrographs too small.

It was found that the models vary greatly in complexity, resource requirements, and usefulness to various applications. The Rational Method and regression equation developed by the Geological Survey from its model are simplest and most suitable as aids in sizing small numbers of hydraulic structures, such as individual roadway culverts. The Geological Survey's regression equations are best suited for flood plain boundary studies, provided the basin is not regulated by reservoirs. TR-20 and HEC-1 are computer models requiring more resources and are suitable for use in flood control project design, while SWMM and MINICAT are the largest models requiring the most resources and are suitable for analyzing and designing large complex sewer systems.

Formulas, tables, and graphs were derived so that if one knows the unit hydrograph shape parameters for one of the models such as Snyder's Unit Hydrograph, Clark's Unit Hydrograph (as computed by the Corps of Engineers), Clark's Unit Hydrograph (as computed by the Geological Survey), or the Soil Conservation Service Unit Hydrograph, then he can convert to another model with its parameters such as to get the same shape unit hydrograph.

ACKNOWLEDGMENTS

I wish to acknowledge the advice and colleague review of Mr. Will Thomas, U.S. Geological Survey during the early phases of this report, and Professor George Reid, Dr. Arthur Bernhart, Dr. Leale Streesin, and Dr. Jimmy Harp for serving on my committee and giving helpful criticism.

This research was conducted without any grants, research assistantships, scholarships, or any other funds from the taxpayers of the United States and Oklahoma, except for about $2,000 in computer expenses.

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LIST OF SYMBOLS

This list of symbols includes in parenthesis the agency or model using the symbol, and the page number where the symbol is first used in the text of this report. If the symbol is associated with a parameter in a computer program and the parameter's identification is different from the symbol, the parameter and model are identified after the symbol's description.

A	drainage area, acres (Rational Method pg. 29).
A	area as a ratio to the total basin area (Corps of Engineers pg. 60).
A	subcatchment area, acres (EPA pg. 73). Same as WAREA in SWMM.
A	conduit cross sectional flow area, square feet (EPA pg. 75).
a_i	planimetered ordinate of the time-area histogram at the end of time period i, in square miles (Corps of Engineers pg. 61).
A_f	conduit cross sectional area when flowing full, square feet (EPA pg. 75).
α	decay rate of infiltration, 1/second (EPA pg. 72). Same as DECAY in SWMM.
α	ratio of a conduit's wetted cross sectional area to its area when full (EPA pg. 75).
C	runoff coefficient, dimensionless (Rational Method pg. 29).
C	dimensionless routing constant (Corps of Engineers pg. 61).
CFS	cubic feet per second.
cfs	cubic feet per second.

CN curve number (SCS pg. 35).

CP Snyder's coefficient for hydrograph storage attenuation (Corps of Engineers pg. 65).

C_t surface water depth at time t after adding rainfall, inches (EPA pg. 72).

d constant drainage rate for redistribution of soil moisture, inches per hour (USGS pg. 43). Same as DRN in G824.

d initial accumulated rain loss during which k_t is increased, inches (Corps of Engineers pg. 56). Same as DLTKR in HEC-1.

D_d surface detention, inches (EPA pg. 73). Same as WSTORE in SWMM.

D_t increase in the loss rate coefficient at time t corresponding to ten inches more of accumulated loss (Corps of Engineers pg. 56).

D_t surface water depth at time t before adding rainfall, inches (EPA pg. 72).

d_t depth of surface water at time t after subtracting infiltration, inches (EPA pg. 72).

Δt computational time step interval, minutes (USGS pg. 44).

Δt computational time step interval, hours (Corps of Engineers pg. 61).

Δt rainfall input interval, hours (EPA pg. 72).

E exponent of precipitation that reflects the influence of rainfall rate on basin-average loss characteristics (Corps of Engineers pg. 56). Same as ERAIN in HEC-1.

e daily pan evaporation, inches per day (USGS pg. 43).

E_i rainfall excess in inches at time increment i (SCS pg. 38).

e_p potential evapotranspiration, inches per day (USGS pg. 43).

F infiltration rate, inches per hour (USGS pg. 44).

f_i maximum infiltration rate, inches per hour (EPA pg. 72). Same as WLMAX in SWMM.

f_o minimum infiltration rate, inches per hour (EPA pg. 72). Same as WLMIN in SWMM.

g gravitational acceleration.

i rainfall intensity, inches per hour (Rational Method pg. 29).

I_i ordinate of time-area runoff at the end of time period i, cubic feet per second (Corps of Engineers pg. 61).

I_t infiltration at time t, inches per hour (EPA pg. 72).

$I(t)$ inflow to a storage reservoir at time t, cubic feet per second (USGS pg. 49).

$i(t)$ cumulative infiltration since beginning a calibration, inches (USGS pg. 43).

K peak rate factor, dimensionless (SCS pg. 37).

k minimum (saturated) hydraulic conductivity used to determine soil infiltration rates, inches per hour (USGS pg. 43). Same as KSAT in G824.

k_p coefficient to convert pan evaporation to potential evapotranspiration (USGS pg. 43). Same as EVC in G824.

k_s linear storage coefficient, hours (USGS pg. 49).

k_t basic loss coefficient at time t, dimensionless (Corps of Engineers pg. 56).

L length of a basin's main watercourse, miles (Corps of Engineers pg. 67).

LAG Snyder's unit hydrograph lag, hours (Corps of Engineers pg. 66).

Lag time from the center of mass of rainfall excess to the peak rate of runoff of a unit hydrograph, hours (SCS pg. 36).

Lag time from the center of mass of rainfall excess to the centroid of a unit hydrograph, hours (USGS pg. 54).

L_{ca} length along a basin's main watercourse from the basin's centroid to the outfall, miles (Corps of Engineers pg. 67).

L_t rainfall loss from all causes at time t, inches per hour (Corps of Engineers pg. 56).

m_c soil moisture storage volume at field capacity, inches (USGS pg. 43). Same as BMSM in G824.

m_o initial soil moisture content at the beginning of a storm event, inches (USGS pg. 44).

n Manning's surface roughness coefficient, dimensionless (EPA pg. 73).

O_i instantaneous unit hydrograph ordinate, cubic feet per second (Corps of Engineers pg. 61).

P cumulative rainfall, inches depth uniformly distributed over a basin (SCS pg. 35).

P_o observed flood peak, cubic feet per second (USGS pg 50).

P_s combined effect of initial moisture content and suction at the wetted front at field capacity, inches (USGS pg. 43). Same as PSP in G824.

P_s synthetic flood peak, cubic feet per second (USGS pg. 50).

P_t rainfall intensity at time t, inches per hour (Corps of Engineers pg. 57). Same as PRCP in HEC-1.

ψ ratio of a conduit's discharge to its full capacity discharge (EPA pg. 75).

Q peak discharge, cubic feet per second (Rational Method pg. 29).

Q cumulative runoff, inches depth uniformly distributed over a basin (SCS pg. 35).

Q flow rate, cubic feet per second (EPA pg. 75).

q lateral inflow rate of overland flow, per unit length of channel (MINICAT pg. 82).

Q_i outfall hydrograph ordinate at time increment i, cubic feet per second (SCS pg. 38).

Q_i unit hydrograph ordinate averaged for the time step i, cubic feet per second (Corps of Engineers pg. 61).

Q_f conduit discharge when flowing full under gravity and friction influence alone, cubic feet per second (EPA pg. 75).

q_L rate of overland flow, cubic feet per second per foot width (EPA pg. 71).

Q_p Snyder's unit hydrograph peak discharge, cubic feet per second (Corps of Engineers pg. 66).

q_p unit hydrograph peak discharge per square mile, cubic feet per second per square mile (Corps of Engineers pg. 67).

Q_t outflow rate at time t, cubic feet per second (EPA pg. 73).

Q(t) outflow hydrograph ordinate at time t, cubic feet per second (USGS pg. 49).

R daily rainfall, inches per day (USGS pg. 43).

R hydrograph storage coefficient, hours (Corps of Engineers pg. 59).

r ratio of the suction at the wetted front for soil moisture at wilting point to that at field capacity (USGS pg. 43). Same as RGF in G824.

r ratio of the rain loss coefficient on an exponential recession curve to that corresponding to ten inches more of accumulated loss (Corps of Engineers pg. 56). Same as RTIOL in HEC-1.

R_e rainfall excess during a time step, inches (USGS pg. 44).

r_i proportion of daily rainfall that infiltrates into the soil (USGS pg. 43). Same as RR in G824.

R_t rainfall intensity at time t, inches per hour (EPA pg. 72).

r^2 coefficient of determination.

S maximum potential rainfall loss, inches (SCS pg. 35).

S rainfall supply rate during a time step, inches per hour (USGS pg. 44).

s value at the beginning of a storm of the rainfall loss coefficient, inches per hour (Corps of Engineers pg. 56). Same as STRKR in HEC-1.

s slope of a basin's main watercourse, feet per mile (Corps of Engineers pg. 67).

s	subcatchment ground slope, feet per foot (EPA pg. 73). Same as WSLOPE in SWMM.
S_f	water surface friction slope, feet per foot (EPA pg. 75).
S_o	conduit invert slope, feet per foot (EPA pg. 74).
σ	standard deviation.
T	time as a ratio to time of concentration (Corps of Engineers pg. 60).
t	time, seconds (EPA pg. 74).
t	time during period being calibrated, days (USGS pg. 43).
TC	time of concentration, hours (Corps of Engineers pg. 59).
T_c	basin time of concentration, hours (SCS pg. 36).
t_c	basin time of concentration, in minutes (USGS pg. 49).
T_j	jth ordinate of a translation hydrograph (USGS pg. 49).
T_p	volume under the rising limb of a basin'a unit hydrograph, cubic feet (SCS pg. 37).
T_r	volume under the decending limb of a basin's unit hydrograph, cubic feet (SCS pg. 37).
t_r	duration of unit rainfall excess, hours (Corps of Engineers pg. 66).
U_i	unit hydrograph ordinate at time increment i, cubic feet per second (SCS pg. 38).
v	velocity, feet per second (EPA pg. 74).
V_o	observed runoff volume, in inches uniformly distributed over a basin (USGS pg. 50).
V_s	synthetic runoff volume, in inches uniformly distrigüted over a basin (USGS pg. 50).
V_t	velocity at time t, feet per second (EPA pg. 73).
W	subcatchment width, feet (EPA pg. 73). Same as WIDTH in SWMM.

x	longitudinal distance down a conduit, feet (EPA pg. 74).
$\bar{X}$	mean value.
y	water depth in a conduit, feet (EPA pg. 74).

OKLAHOMA CITY URBAN STORM WATER RUNOFF QUANTITY:

COMPARISON AND CALIBRATION OF PREDICTIVE METHODS

CHAPTER I

INTRODUCTION

Judging from references to inundations in ancient literature, such as the Bible, floods have been a problem to us humans as long as we have been on Earth. They have meant loss of life, damage to buildings, crops, transportation, and commerce. In the United States in 1974 there was approximately one billion dollars in property damage from flooding, and loss of about 80 lives. Now much of this loss is sustained in cities and towns, from local flooding, in the sense that the water is contributed by rainfall on basins of only a few square miles in size. No scientific way yet exists to predict years in advance precisely the day, hour, and minute of floods, but methods do exist to predict the depth and velocity of water, and area flooded, provided one knows certain geometric properties of the area flooded, and its friction resistance to the water flow, and the flow-rate of the water. This latter problem, the flood flow-rate, is the subject of this research.

There are many methods of estimating the flood flow-rate that would result from a particular rainfall distribution over a certain basin. An old and simple, yet still popular one, is the "Rational" method--in fact, it is the only method allowed by city ordinance in Oklahoma City, and many nearby cities, for basins 500 acres or less in size, so it is treated in this research. With the appearance on the scene of computer technology, there has been development of many computer programs using many mathematical concepts addressed to the issue of predicting urban flood flow-rates. Some Federal government agencies have used their technical expertise along that line, and their programs have been used by some local government and private engineers. Such Federal agencies and their programs are:

U. S. Corps of Engineers: Flood Hydrograph Package (HEC-1)

U. S. Soil Conservation Service: Project Formulation - Hydrology (TR20)

U. S. Geological Survey: Urban Flood Hydrograph Synthesis Model (G824)

U. S. Environmental Protection Agency: Storm Water Management Model (SWMM)

U. S. River Forecast Center: Deterministic Urban Runoff Model (MINICAT)

The effort of this research has been to calibrate those five Government computer programs and the Rational method on three urbanized basins in Oklahoma City, Oklahoma, using some twenty-three observed flood events and their rainfall distributions that occurred during 1974 and 1975, and to compare the reliability of those programs in reproducing the observed floods. The basis of comparing the computer results to the observed events has been with respect to the peak discharge, the volume of storm water runoff, and the time the peak flow occurred, except that the Rational method was not compared on the basis of volume of runoff, because that method as traditionally used cannot deal with water volume, only peak discharge.

The results presented here are the culmination of some three or four hundred computer runs on the University of Oklahoma's IBM 370, amounting to a cost of about two thousand dollars.

CHAPTER II

THE COLLECTION OF RAINFALL-RUNOFF RECORDS AND THE TEST BASINS

This chapter describes the procedures used by hydrologists of the U. S. Geological Survey to collect the rainfall-runoff records at the three flood gage sites used in calibrating the models of this study, and describes the physical properties of the three basins contributing flow to those sites.

The Collection of Rainfall-Runoff Records at the Flood Gage Sites

During 1973 hydrologists of the Water Resources Division of the U. S. Geological Survey, under the direction of Mr. Wilbert O. Thomas, Jr., and Mr. Robert K. Corley, installed flood recording gages in Oklahoma City, Oklahoma, on Deep Fork Creek at Portland Avenue above Will Rogers Park (Figure 1), on Bluff Creek at Northwest Highway (Figure 2), and on Deep Fork Creek at Eastern Avenue (Figure 3). Those were Model SR Recorders, as pictured in Figure 4. That model is a graphic recorder which records both flood stages and rainfall (1). The recorder sits on a perforated

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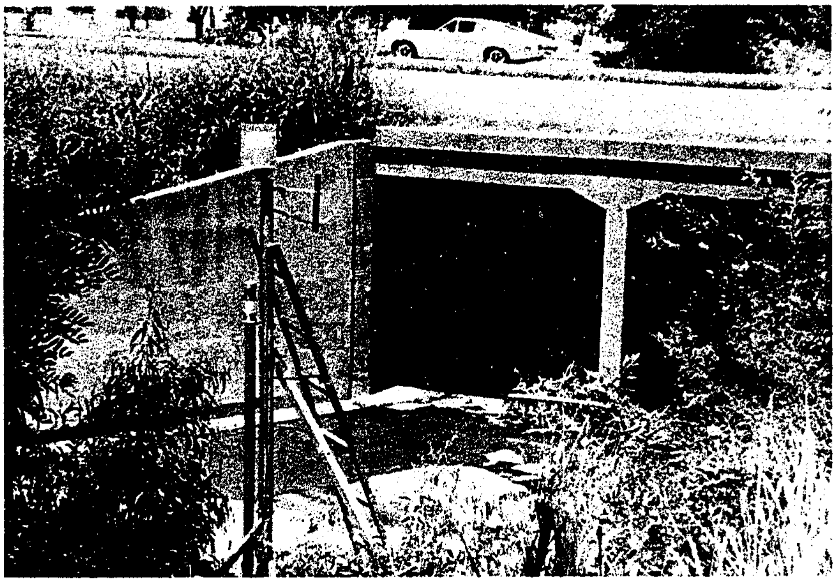
1. Buchanan, Thomas J., and William P. Somers, "Stage Measurement at Gaging Stations," U. S. Geological Survey Techniques of Water-Resources Investigations. 1968.

Looking down toward the gage from near the top of the culvert, on the East side of Portland Avenue.

Looking West, upstream, toward the gage, which is on the right culvert wingwall.

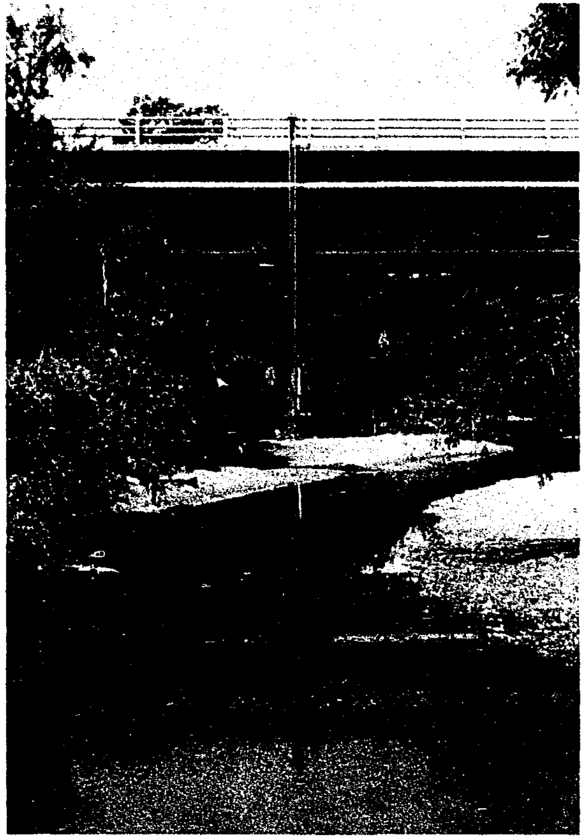
Figure 1. Flood gage on Deep Fork Creek at Portland Avenue.



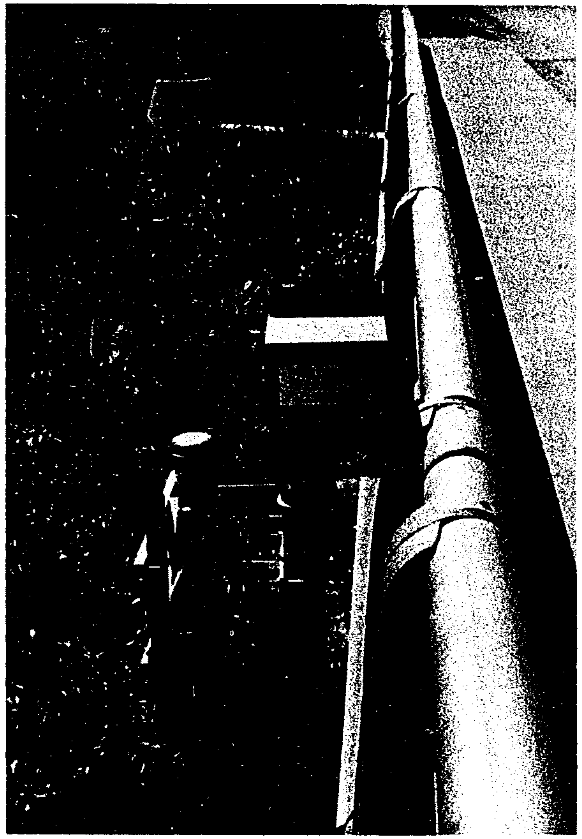
Looking North, downstream, toward the gage and the culvert under Northwest Highway.

Looking South, upstream, toward the gage from Northwest Highway.

Figure 2. Flood gage on Bluff Creek at Northwest Highway.



Looking West, upstream, toward the gage.



Looking down at the gage on the East side of Eastern Avenue.

Figure 3. Flood gage on Deep Fork Creek at Eastern Avenue.

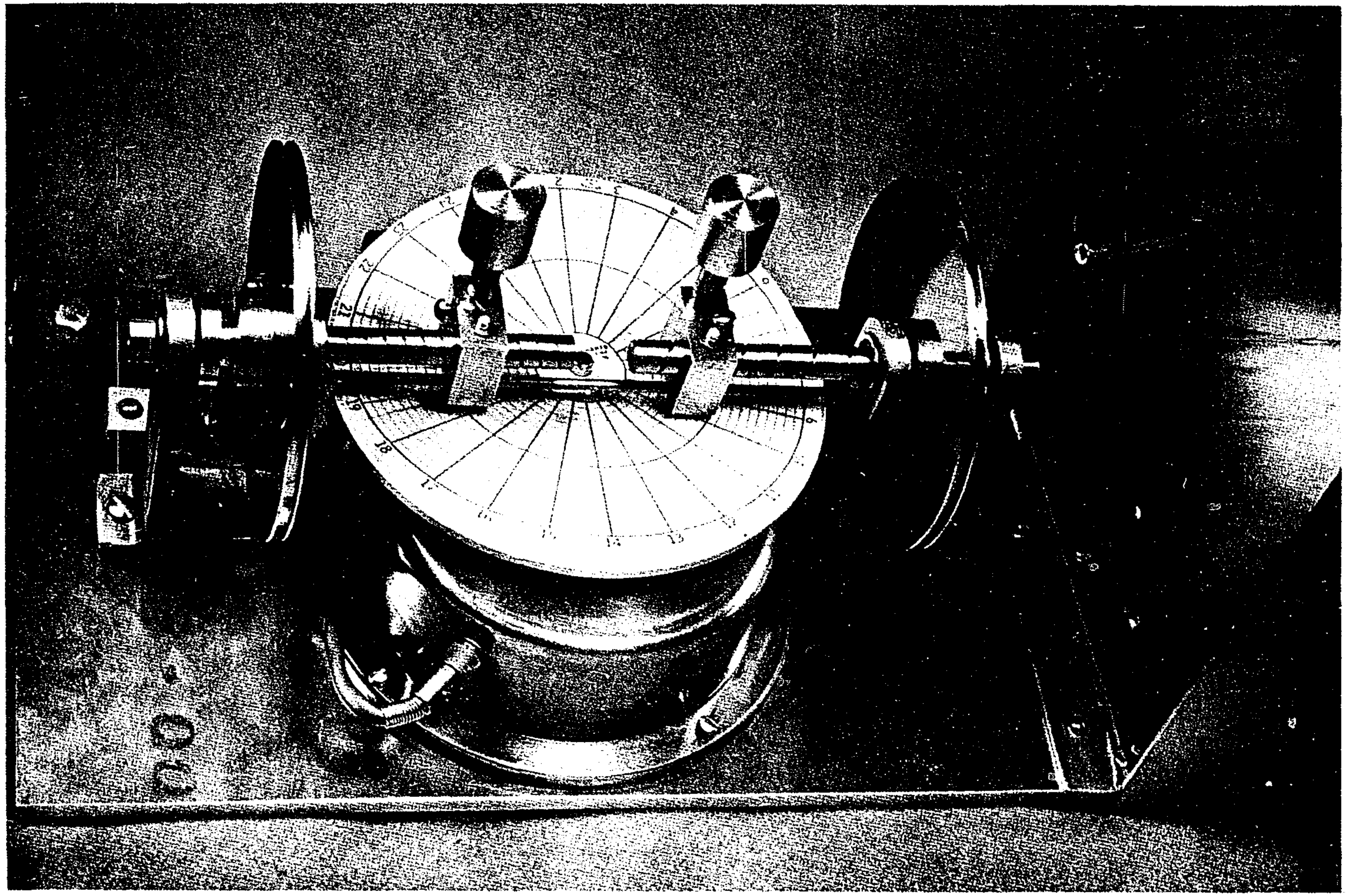


Figure 4. Flood gage, Model SR Stage-Rainfall recorder.

pipe which is securely fixed in place with its bottom surveyed to a known datum. Inside is a float which moves up and down as the stream water rises and falls. The float is attached by string to a worm wheel which moves a pencil mark across a mylar disc as the float moves up and down, thus recording the "stage," or height of the stream water above the known datum. A battery-wound clock rotates the 5-inch disc one revolution every twenty-four hours. A rainwater catchment atop the recorder feeds rainfall to another pipe sealed at the bottom. It has a float which rises as rainfall accumulates and moves a worm wheel leaving a pencil mark recording total rainfall as the disc rotates. Incremental rainfall is determined by subtracting the cumulative rainfall at the beginning and end of each time increment. Figure 5 is a typical sample of a storm-flood recording with the report the observer fills out upon retrieving the record after a storm.

The raw field recordings of pencil lines for each storm are converted to digital data and tabulated on standard "Stage-Rainfall Gage Record" forms. Figure 6 is a typical example, reduced in size for inclusion in this report. The raw chart is read for time and tabulated in Column 2. The total rainfall depth at each time step is read from the chart and tabulated in Column 4, and end and beginning of period values are subtracted and tabulated in Column 6 as incremental rainfall. The stream's stage at each time increment is read from the chart and recorded in Column 10. Stage is converted to discharge by a process described in the following paragraphs.

From time to time hydrologists measure the stream's discharge and stage during flood events. Typically this is done by stretching a tag line across the stream near the flood gage or along a bridge, and

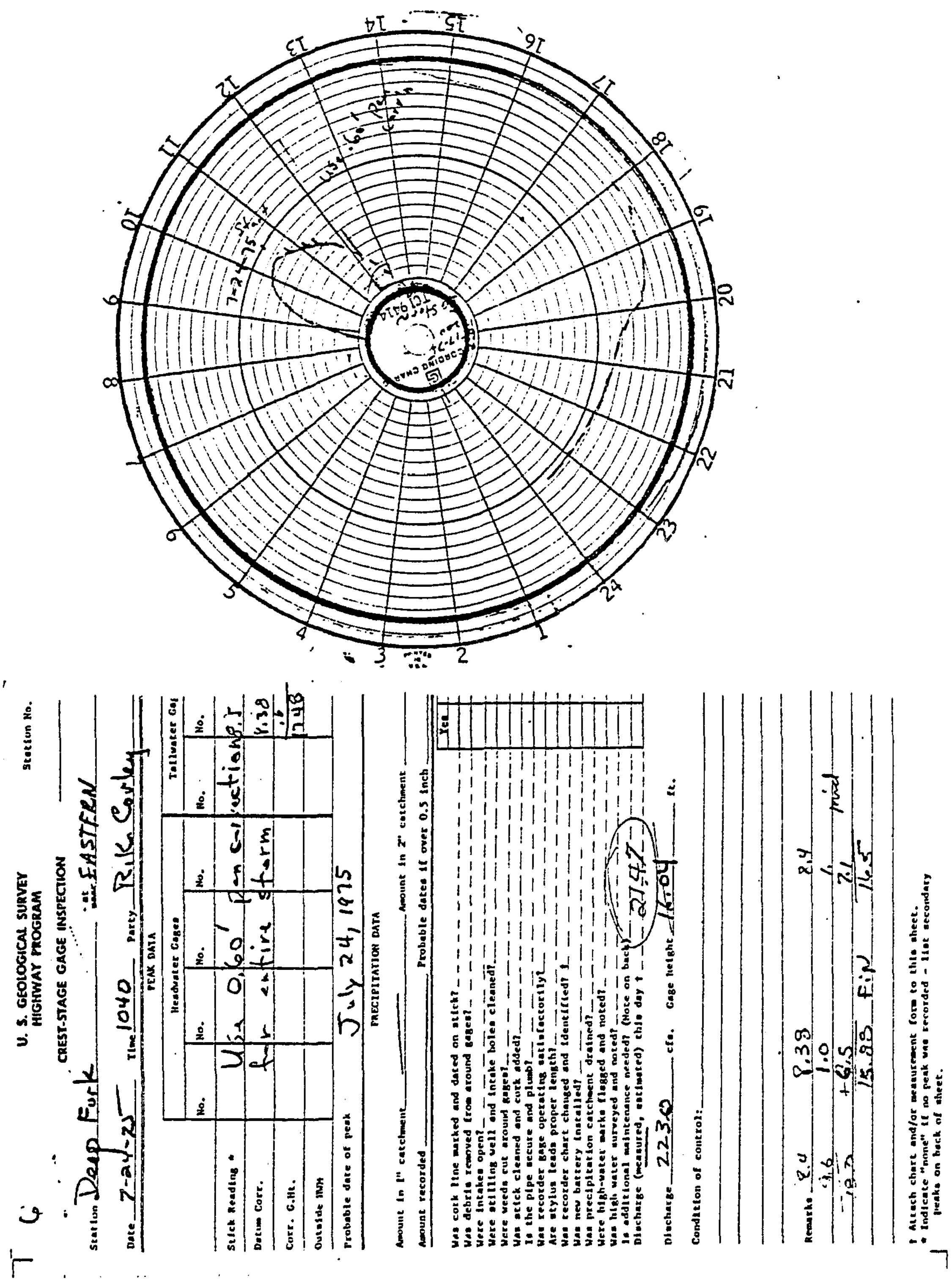


Figure 5. Typical stage-rainfall trace from a flood gage.

U. S. Geological Survey Sta. No.________

STAGE-RAINFALL GAGE RECORD

Crest-stage partial-record station Deep Fork @ Eastern Ave-OKC-#10

Storm of 10-14-15-75

Date	Time	Time inter-val	Accumulated depth: Re-corded	Accumulated depth: Cor-rected	Depth for each time interv.			Gage height: Chart	Gage height: Corrected 8.38	Discharge
1	2	3	4.	5	6	7	8	9	10	11
	Hours	Min.	In.	In.	In.	In.	In.	Feet	Feet	CFS
10-14-75	2300									
			2.4		0					
	30		2.57		.17					
			2.74		.17					
	2400		2.90		.16					
10-15-75			2.93		.03					0
	30		2.95		.02					20
			2.97		.02					40
	100		3.00		.03					90
			3.20		.20					170
	30		3.22		.02					200
			3.23		.01					333
	200		3.25		.02				8.59	476
			3.29		.04				9.48	656
	30		3.32		.03				10.18	803
			3.36		.04				10.41	875
	300		3.40		.04				10.38	851
			3.45		.05				10.28	817
	30		3.50		.05				9.98	756
									9.88	736
	400				1.10				9.78	716
									9.68	696
	30								9.58	676
									9.48	656
	500								9.38	636
									9.29	616
	30								9.18	586
									9.08	576
	600								8.78	536
									8.43	442
	30									[illegible]
	700									

Tabulated by WRD. Date 3-8-76 Checked by______ Date______
Computed by______ Date______ Checked by______ Date______

Figure 6. Typical stage-rainfall gage record.

dividing the cross section into twenty to thirty partial sections, as described in (2), and the area and mean velocity of each is determined separately, and the measurements recorded in the field at the time on "Discharge Measurement Notes," as shown in Figure 7. As the measurer proceeds along the tag line, he stops in the middle of each partial section and records his distance from initial point. He figures the width of each partial section by subtracting the means between successive distances. He measures the water's depth with a Price or pygmy current meter, and measures the veolocity at one or two points in the vertical by counting the meter's revolutions per unit time by listening in a headset, and converts revolutions per unit time to velocity by means of a conversion table. If he reads two velocities he does so at 0.2 depth and 0.8 depth and averages the two for mean velocity. If he takes one velocity he does so at 0.6 depth. He then computes the partial section's area by multiplying depth by width, and multiplying by mean velocity to get discharge. The sum of the discharges of the partial sections is the stream's total discharge at the time of measurement.

Results of such measurements made from time to time (augmented in the case of the Bluff Creek and Deep Fork Creek at Portland Avenue, with culvert capacity calculations) are tabulated on a form such as in Figure 8, which was reduced for this report, and plotted on a graph, such as Figure 9, stage versus discharge. From sucha graph a stage-discharge,

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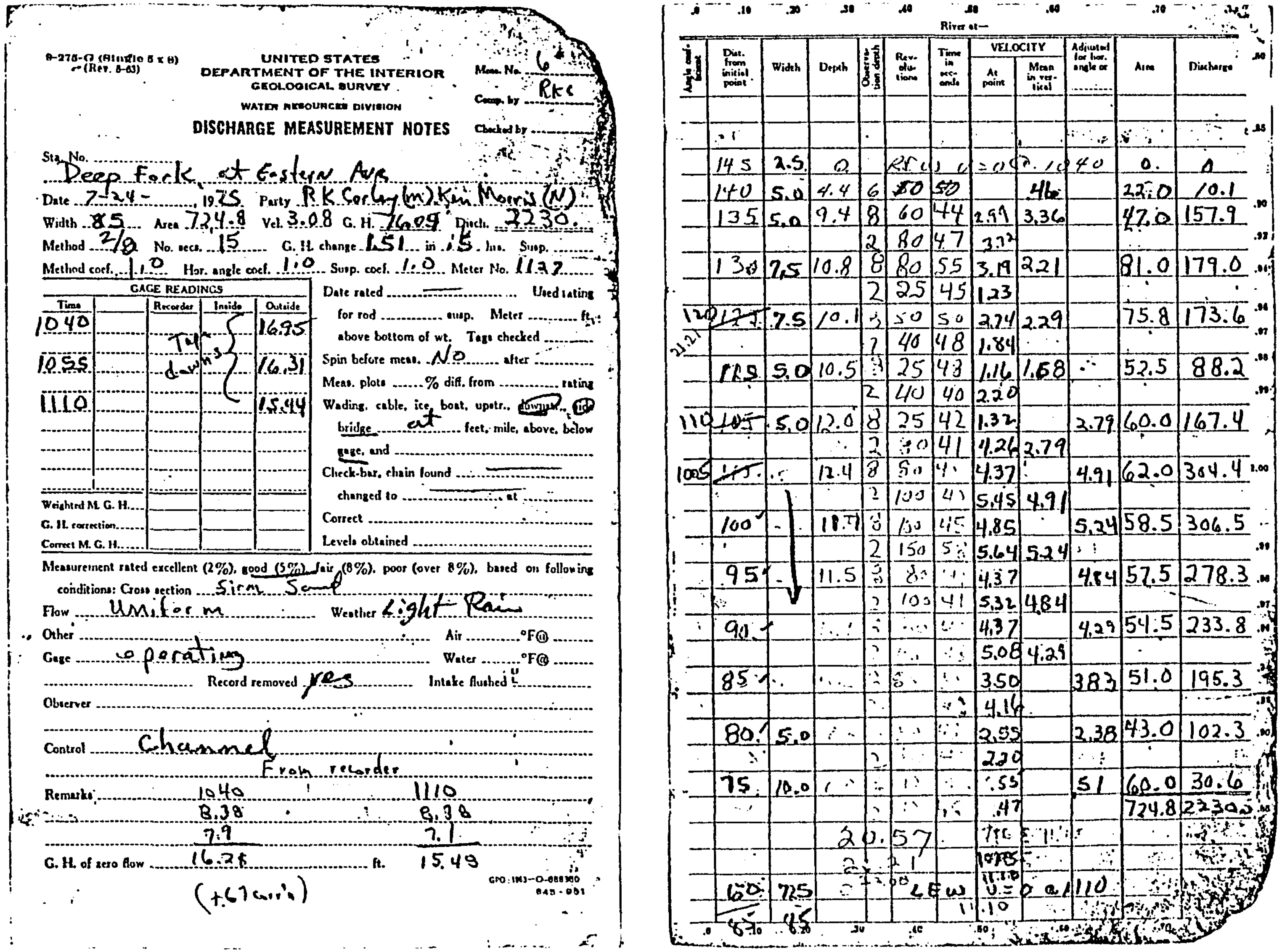
2. Carter, R. W., and Jacob Davidian, "General Procedure for Gaging Streams," U. S. Geological Survey Techniques of Water-Resources Investigations, 1968.



9-275-G (Single 8 x 8)
(Rev. 8-63)

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
WATER RESOURCES DIVISION

DISCHARGE MEASUREMENT NOTES

Meas. No. 6
Comp. by RKC
Checked by

Sta. No. Deep Fork at Eastern Ave
Date 7-24- 1975 Party R K Corley (m) Kim Morris (N)
Width 85 Area 724.8 Vel. 3.08 G. H. 16.09 Disch. 2230.
Method 2/8 No. secs. 15 G. H. change 1.51 in 1.5 hrs. Susp.
Method coef. 1.0 Hor. angle coef. 1.0 Susp. coef. 1.0 Meter No. 1137

GAGE READINGS				
Time		Recorder	Inside	Outside
1040		Tape down		16.95
1055				16.31
1110				15.44
Weighted M. G. H.				
G. H. correction				
Correct M. G. H.				

Date rated .... Used rating for rod .... susp. Meter .... ft. above bottom of wt. Tags checked ....
Spin before meas. NO after ....
Meas. plots .... % diff. from .... rating
Wading, cable, ice, boat, upstr., downstr., side bridge at feet, mile, above, below gage, and ....
Check-bar, chain found ....
changed to .... at ....
Correct ....
Levels obtained ....

Measurement rated excellent (2%), good (5%), fair (8%), poor (over 8%), based on following conditions: Cross section Silty Sand
Flow Uniform Weather Light Rain
Other .... Air .... °F @ ....
Gage operating Water .... °F @ ....
Record removed yes Intake flushed
Observer
Control Channel

From recorder

Remarks	1040	1110
	8.38	8.38
	7.9	7.1
G. H. of zero flow ft.	16.28	15.49

(+.67 cur's)

GPO: 1963—O-688360
845-951

River at—

Angle coefficient	Dist. from initial point	Width	Depth	Observation depth	Revolutions	Time in seconds	Velocity At point	Velocity Mean in vertical	Adjusted for hor. angle or	Area	Discharge
	145	2.5	0		REW		=0 @ 1040			0.	0
	140	5.0	4.4	6	50	50		.46		22.0	10.1
	135	5.0	9.4	8	60	44	2.99	3.36		47.0	157.9
				2	80	47	3.72				
	130	7.5	10.8	8	80	55	3.19	2.21		81.0	179.0
				2	25	45	1.23				
21.21	120 ~~125~~	7.5	10.1	8	50	50	2.74	2.29		75.8	173.6
				2	40	48	1.84				
	115	5.0	10.5	8	25	43	1.16	1.68		52.5	88.2
				2	40	40	2.20				
	110 ~~105~~	5.0	12.0	8	25	42	1.32		2.79	60.0	167.4
				2	90	41	4.26	2.79			
	105 ~~115~~		12.4	8	90	41	4.37		4.91	62.0	304.4
				2	100	41	5.45	4.91			
	100		11.7	8	100	45	4.85		5.24	58.5	306.5
				2	150	58	5.64	5.24			
	95		11.5	8	80	[illegible]	4.37		4.84	57.5	278.3
				2	100	41	5.32	4.84			
	90		[illegible]	8	[illegible]	[illegible]	4.37		4.29	54.5	233.8
				2	[illegible]	[illegible]	5.08	4.29			
	85		[illegible]	8	8	[illegible]	3.50		3.83	51.0	195.3
				2		[illegible]	4.16				
	80	5.0	[illegible]		[illegible]	[illegible]	2.55		2.38	43.0	102.3
				2	[illegible]	[illegible]	2.20				
	75	10.0	[illegible]	[illegible]	[illegible]	[illegible]	.55		.51	60.0	30.6
				2	[illegible]	[illegible]	.47			724.8	2230.0
			20.57				1110				
			21.1				1075				
	60	7.5			LEW		11.10 V = 0 @ 1110				
	~~65~~	15					11.10				

Figure 7. Discharge measurement notes.

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(Rev. 1-65)

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY (WATER RESOURCES DIVISION)

Station No. 07242200

Discharge measurements of Deep Fork at Okla. City, Okla., during the year ending Sept. 30, 19....

No.	Date	Made by—	Width	Area	Mean velocity	Gage height	Discharge	Rating: Shift adj.	Rating: Percent diff.	Method	Number meas. sections	Gage height change	Time	Meas. rated	REMARKS
			Feet	Sq ft	Fps	Feet	Cfs	Feet				Feet	Hr		
1	9-4-73	WOT, RKC, RWB	24.0	240	14.2	12.4	2900								Culvert comp.
2	9-12-73	WOT, RWB	3.5	1.68	2.39	4.04*	2.01		* Using	U.S.		chisel	D	for tape down	Wading 150' below culvert
3	9-27-73	RKC, RWB	17.0	36.7	2.39	5.64	87.8								Wading 50' above culvert
4	9-27-73	RKC, RWB	16.0	35.9	2.27	5.48	81.7								Wading 50' above culvert
5		WOT				3.9	5.6								Wading d.s. from culvert
6	5-23-74	WOT, RKC	24.0	129	3.64+	8.70	469		+ }	Surface	velocities				} Using wading rod D.S.S. culvert
7	5-23-74	do.	24.0	114	3.18+	8.07	362		}						}
8	5-31-74	RKC, DRS	21.0	31.6	2.38	5.26	75.2								Wading d.s. from culvert
9	5-31-74	do.	24.0	28.5	2.29	5.08	65.3								Wading d.s.s. culvert
10	6-8-74	RKC, TEC	24.0	77.4	2.13	6.11	164								Handline dss culvert
11	6-8-74	do.	24.0	65.6	1.88	5.84	123								do.
12	10-30-74	WOT, RKC	24.0	69.7	3.63	6.57	253								do
13	10-30-74	do.	24.0	50.1	3.25	5.76	163								Wading d.s.s. culvert

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Figure 8. Record of discharge measurements.

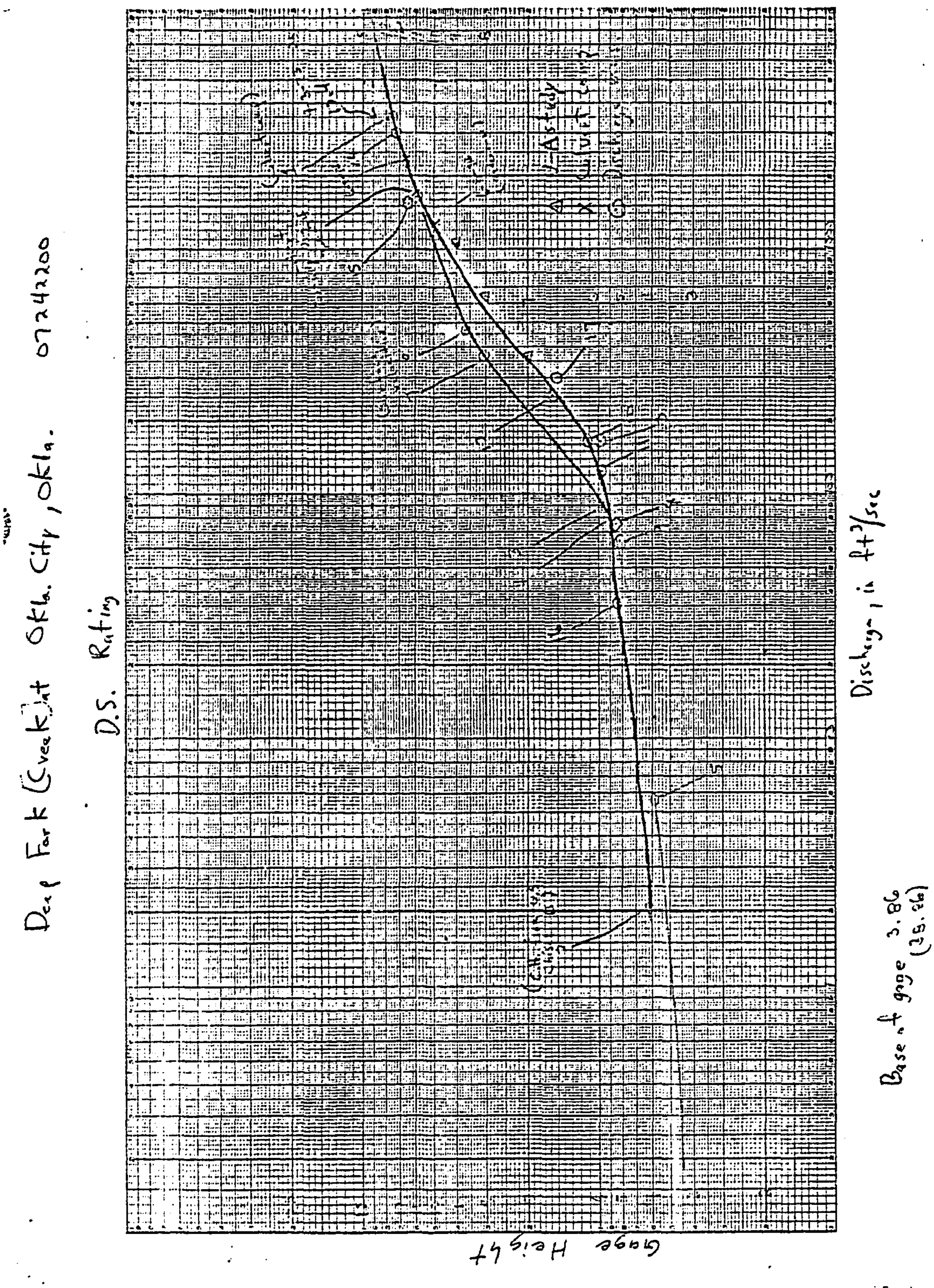

Figure 9. Stage-discharge curve.

or rating table is made, as in Figure 10. In the case of Deep Fork Creek at Portland Avenue, a downstream channel change made a change in the rating information, and in the case of Bluff Creek, a downstream backwater condition made it necessary to use two rating tables, separate for the rising and falling limbs of the hydrographs. After developing the rating tables, the hydrologist can return to the Stage-Rainfall Gage Records (Figure 6), Column 11, and tabulate the discharge by time increment for each flood.

Some twenty-three rainfall-runoff events were used in this study, recorded during 1974 and 1975 on the three basins to be described in the following sections. Appendix B has graphs of each event's rainfall hyetograph and recorded discharge hydrograph as tabulated on Stage-Rainfall Gage Records like Figure 6. During the storm of November 2, 1974, the rain gage at Bluff Creek at Northwest Highway malfunctioned, so for this study the rainfall used was the recorded at Deep Fork Creek at Portland Avenue, some two and a half miles to the Southeast.

Deep Fork Creek at Portland Avenue

This drainage basin of 2.98 square miles located in western Oklahoma City is virtually completely urbanized. Figure 11 is a map of the basin, showing also the manner it was divided into subcatchments and streams for modeling on two of the computer programs to be described in Chapter III. Table 1 presents the physical characteristics of each of the subcatchments and streams. Subcatchment areas were planimetered. Average subcatchment slope was usually determined by superimposing a grid on a topographic map of the subcatchment, measuring the downsloping

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(Rev. 2-67)

Rating No. 2

UNITED STATES DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY (WATER RESOURCES DIVISION)

Sta. No. 07242200
Table No. 03
Begin ___ ___ ___ ___
YR. MO. D. HR.

Rating table for Deep Fork at Okla. City

from Nov. 2, 1974 to March 1, 1975, from ______ to ______, from ______ to ______

Gage height	Discharge	Difference	Gage height	Discharge	Difference	Gage height	Discharge	Difference	Gage height	Discharge	Difference	Gage height	Discharge	Difference	Gage height	Discharge	Difference	Gage height	Discharge	Difference
Feet	Cfs PZF 3.4	Cfs	Feet	Cfs	Cfs	Feet	Cfs	Cfs	Feet	Cfs	Cfs	Feet	Cfs	Cfs	Feet	Cfs	Cfs	Feet	Cfs	Cfs
4.00			6.00	90	10	8.00	360	20	10.00	900	40	12.00	1900	100	.00			.00		
.10			.10	100		.10	380		.10	940		.10	2000		.10			.10		
.20			.20	110		.20	400		.20	980		.20	2100		.20			.20		
.30	3		.30	120		.30	420		.30	1020		.30	2200		.30			.30		
.40	5	3	.40	130		.40	440		.40	1060		.40	2300		.40			.40		
.50	8	3	.50	140		.50	460	20	.50	1100		.50	2400		.50			.50		
.60	11	4	.60	150	10	.60	480	30	.60	1140		.60	2500		.60			.60		
.70	15		.70	160	10	.70	510		.70	1180		.70	2600		.70			.70		
.80	19		.80	170	15	.80	540		.80	1220		.80	2700		.80			.80		
.90	23		.90	185	15	.90	570		.90	1260	40	.90	2800	100	.90			.90		
5.00	27	4	7.00	200		9.00	600	30	11.00	1300	60	13.00	2900		.00			.00		
.10	31	5	.10	215		.10	630		.10	1360		.10			.10			.10		
.20	36	5	.20	230		.20	660		.20	1420		.20			.20			.20		
.30	41	6	.30	245		.30	690		.30	1480		.30			.30			.30		
.40	47	6	.40	260		.40	720		.40	1540		.40			.40			.40		
.50	53	7	.50	275		.50	750		.50	1600		.50			.50			.50		
.60	60	7	.60	290		.60	780		.60	1660		.60			.60			.60		
.70	67	7	.70	305	15	.70	810		.70	1720		.70			.70			.70		
.80	74	8	.80	320	20	.80	840		.80	1780		.80			.80			.80		
.90	82	8	.90	340	20	.90	870	30	.90	1840	6-	.90			.90			.90		

This table is applicable for open-channel conditions. It is based on 2 discharge measurements made during Nov. 1974 ______ and is ______ well defined between ______ cfs and ______ cfs.

Use on recession of flood of 11-2-74 and thereafter

Comp. by WST date 11-19-74
Ckd. by ______ date ______

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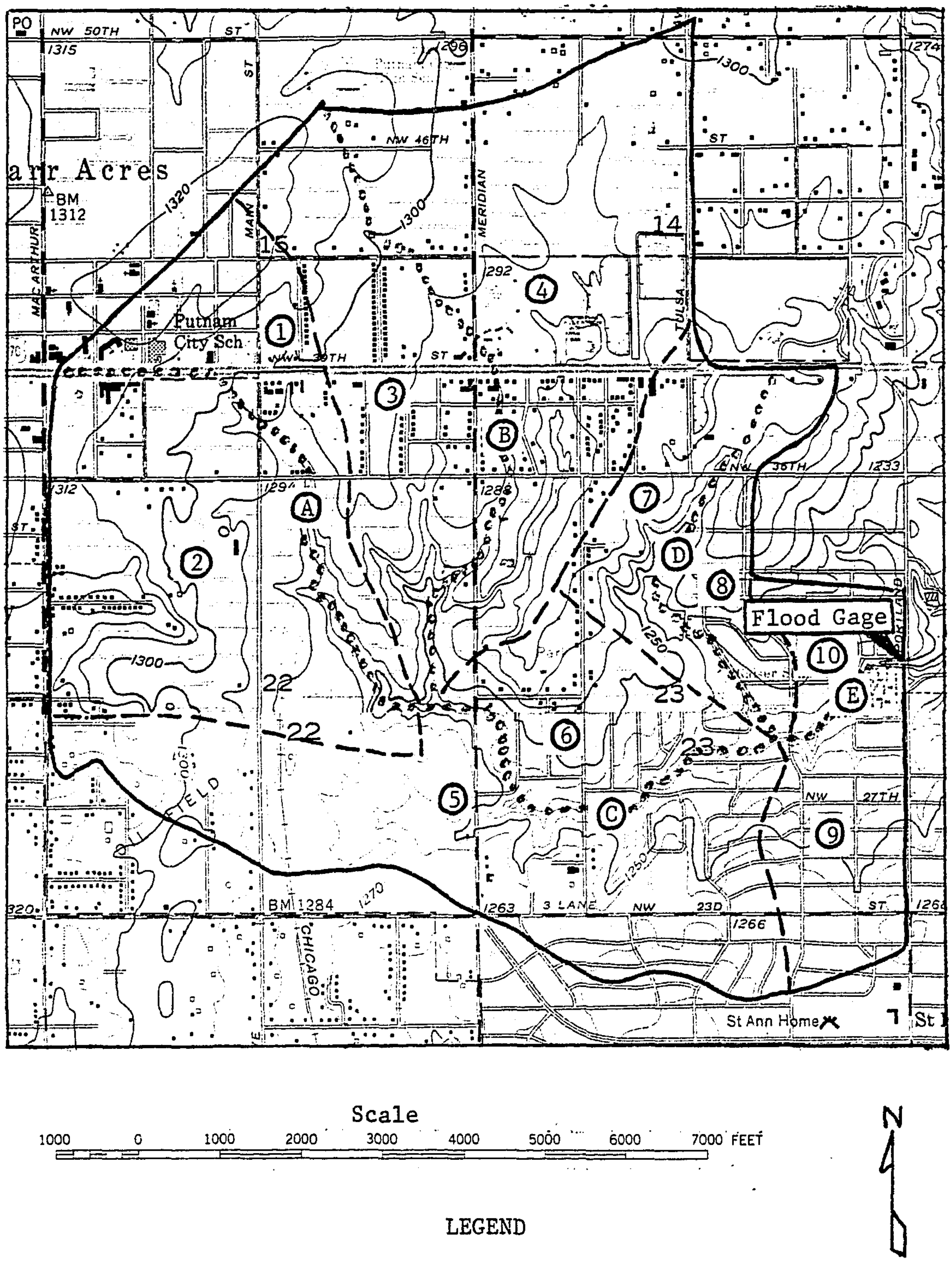
Figure 10. Stage-discharge table.



LEGEND

Basin boundary

Subcatchment boundary

Stream course used in modeling

(1) Subcatchment number

(A) Stream letter

Figure 11. Basin above Deep Fork Creek at Portland Avenue.

TABLE 1

PHYSICAL CHARACTERISTICS OF THE BASIN UPSTREAM

OF DEEP FORK CREEK AT PORTLAND AVENUE.

SUBCATCHMENTS

NO.	AREA, ACRES	MEAN SLOPE, PERCENT	LENGTH, FEET
1.	160	4.5	930
2.	340	3.3	2040
3.	240	2.7	1220
4.	310	3.5	1090
5.	360	3.3	2780
6.	130	3.5	850
7.	100	3.3	660
8.	80	3.3	2530
9.	120	3.3	2530
10.	60	2.7	1310

NOTE: Average 45% impervious cover.

STREAMS

NO.	LENGTH, FEET	FLOWLINE SLOPE, PERCENT	BOTTOM WIDTH, FEET
A	3700	1	5
B	8400	1	5
C	5600	0.6	10
D	5300	1.2	5
E	2000	0.5	10

NOTE: Average stream side slope 1.5, horizontal/vertical.

distance between two contour lines near each grid point, dividing it into the vertical contour interval to get the local slope at each grid point, then averaging those slopes. When it appeared from the spacing of contour lines on the topographic map that the overland slope does not vary much across a subcatchment, the slope may have been determined by only two or three samplings of local slope. Length refers to the average overland distance sheet runoff water must flow across the subcatchment to reach the receiving stream, and is the subcatchment's area divided by the length of the reach of the stream receiving the subcatchment's overland flow. The basin's percentage of impervious cover, 45 percent, was determined by hydrologists of the U. S. Geological Survey (3), by superimposing a grid on areal photographs of the basin, counting the number of grid points falling on roof tops, streets, parking lots, driveways, and other impervious surfaces, and dividing by the total number of grid points on the basin.

Stream lengths were determined by map measure. Stream slopes were determined by dividing the difference between the channels upstream and downstream elevations by the stream length. Bottom widths, for use in the computer program SWMM to be described in the next chapter, and side slope ratio, to be used in the River Forecast Center's model to be described also in the next chapter, were determined by estimations based on visual observations at a few points on the streams. At the time the rainfall-runoff records were collected for this study, somewhat more than

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3. Thomas, W. O. Jr., and R. K. Corley, "Techniques for Estimating Flood Discharges for Oklahoma Streams," U. S. Geological Survey, Water Resources Investigation 77-54, 1977.

half the channel lengths of this basin had been concrete lined and straightened.

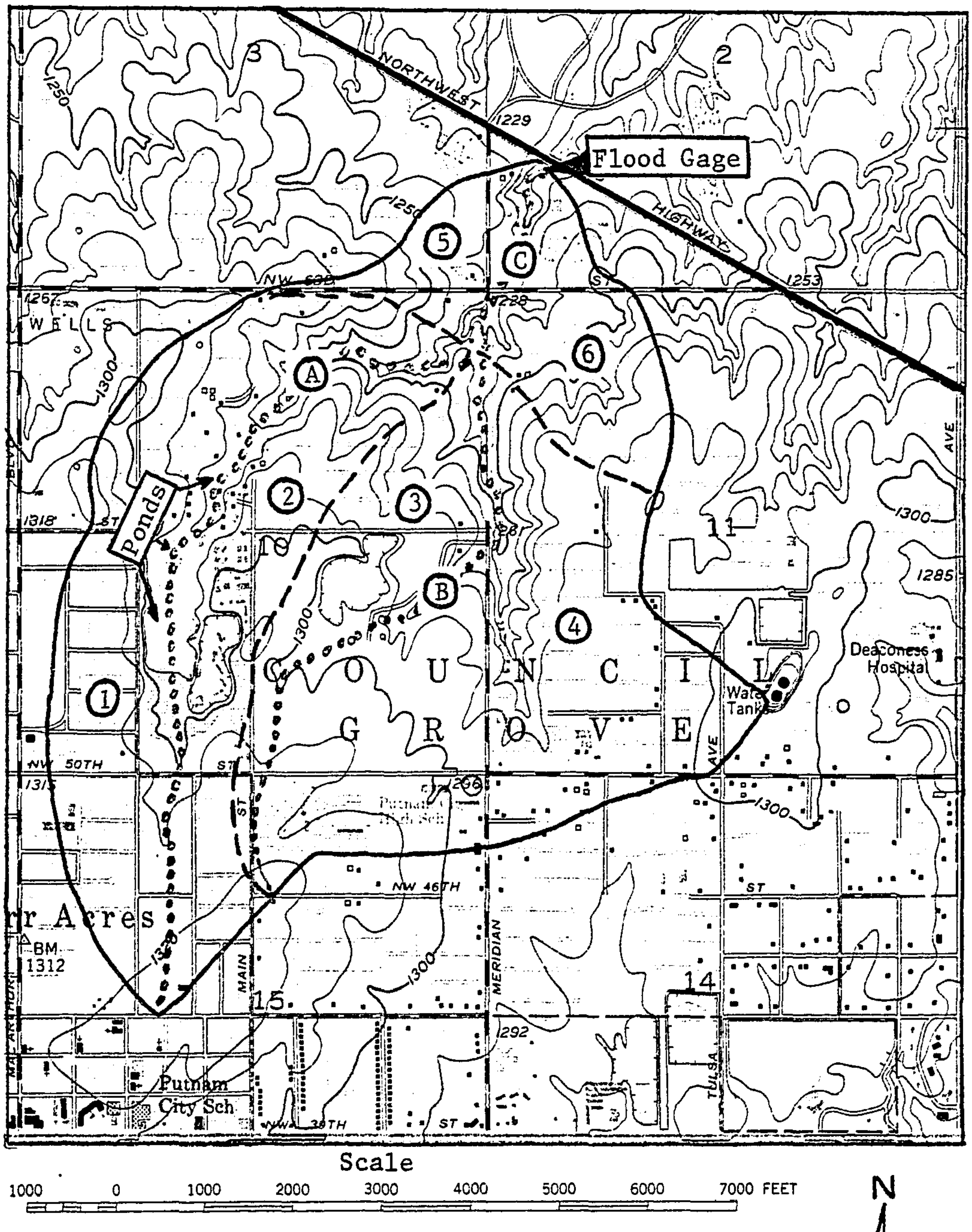
For use in the Geological Survey's model to be discussed in Chapter III, the basin's area had to be broken down into a "time-area histogram," which was done by drawing concentric circles on a map of the basins, centered on the flood gage site as illustrated in Figure 18, and planimetering the cumulative area between each ring and the flood gage site. Table 4 has the results.

Bluff Creek at Northwest Highway

This drainage basin of 1.64 square miles is immediately north of the Deep Fork basin, and is shown in Figure 12, along with the manner it was divided into subcatchments and stream segments. Table 2 has the physical characteristics of the subcatchments and stream segments. This basin is not completely urbanized, having about eighty to one hundred acres of vacant land in the south center of its area. Stream A has three ponds on it totaling about thirty-two acres, and they are simulated in the models as a wide, flat conduit because they are unregulated flow-through ponds without outlet control. The physical properties for this basin presented in Table 2 were determined the same way as those described in the previous section on Deep Fork Creek. The basin's time-area histogram is tabulated in Table 4.

Deep Fork Creek at Eastern Avenue

This is the largest drainage basin considered in this study, being 28.2 square miles, as shown in Figure 13, along with the manner it



LEGEND

——— Basin boundary

— — — Subcatchment boundary

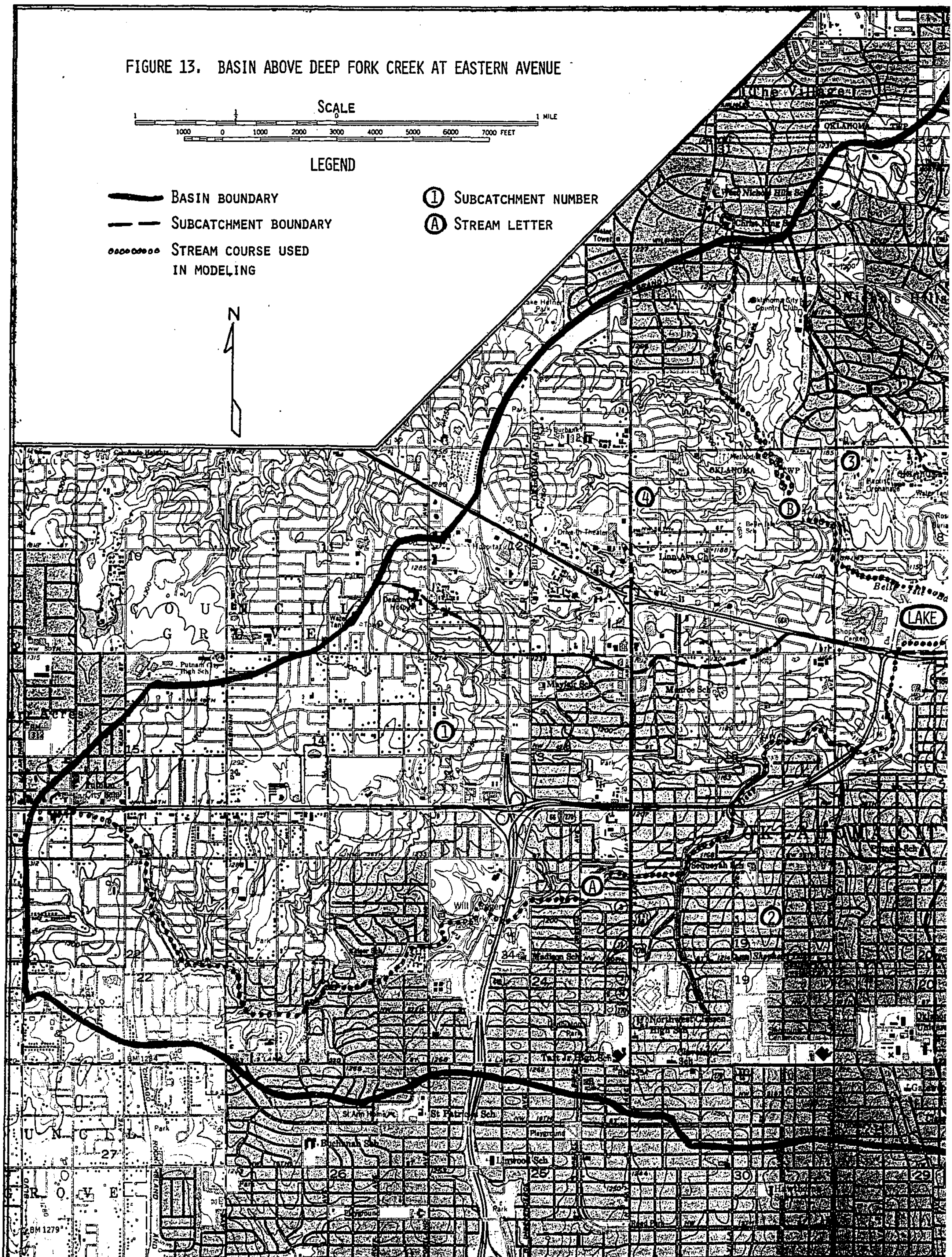
Stream course used in modeling

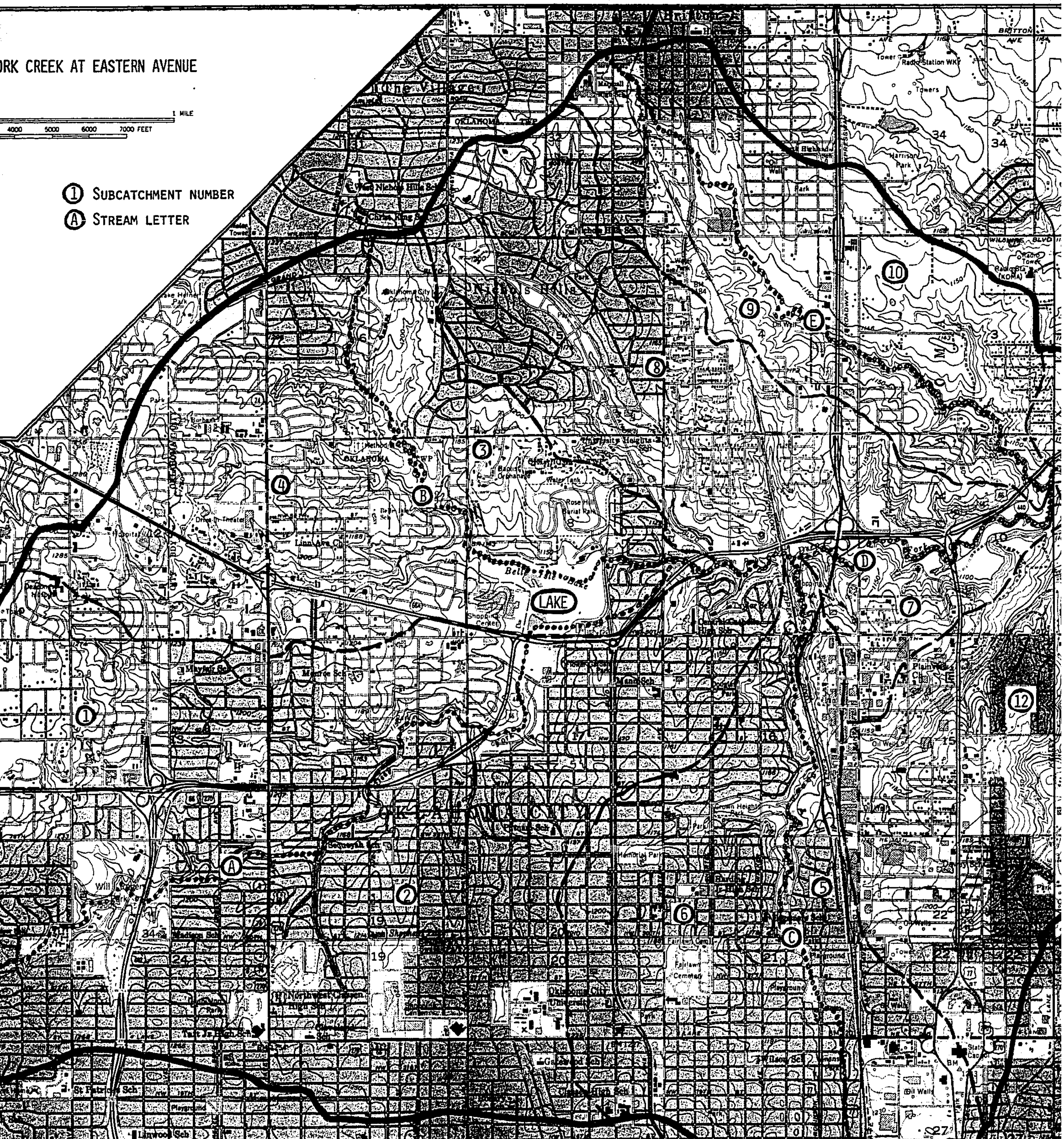
① Subcatchment number

Ⓐ Stream letter

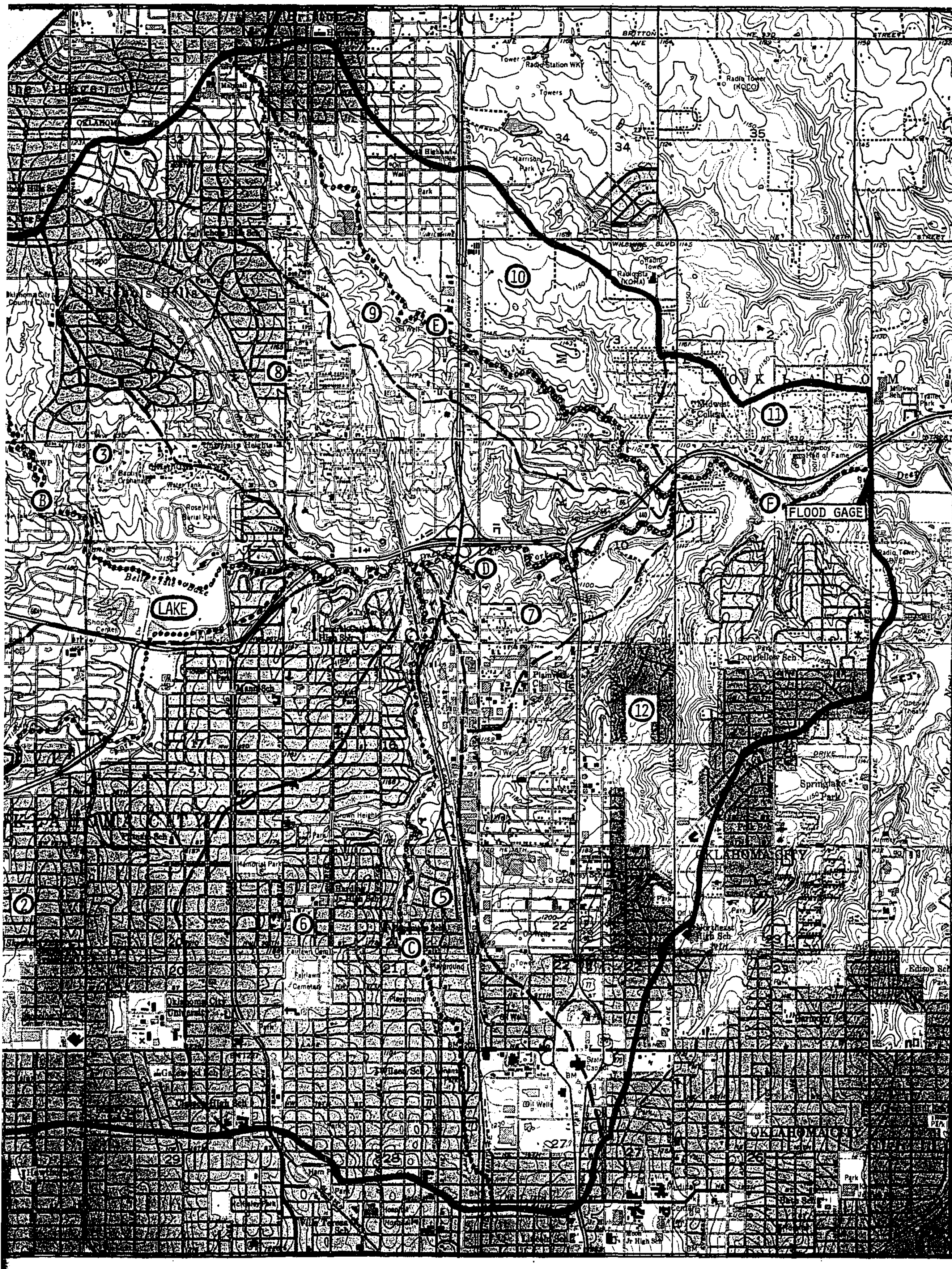
Figure 12. Basin above Bluff Creek at Northwest Highway.

FIGURE 13. BASIN ABOVE DEEP FORK CREEK AT EASTERN AVENUE





ORK CREEK AT EASTERN AVENUE
1 MILE
4000 5000 6000 7000 FEET
① SUBCATCHMENT NUMBER
Ⓐ STREAM LETTER
LAKE



FLOOD GAGE
LAKE
Midwest College
Cowboy Hall of Fame
Springlake Park
Harrison Park
Rose Hill Burial Park
Fairlawn Cemetery
Radio Station WKY
Towers
BRITTON
10
9
E
8
11
F
D
7
12
B
2
6
5
C

TABLE 2

PHYSICAL CHARACTERISTICS OF THE BASIN UPSTREAM

OF BLUFF CREEK AT NORTHWEST HIGHWAY.

SUBCATCHMENTS

NO.	AREA, ACRES	MEAN SLOPE, PERCENT	LENGTH, FEET
1.	270	3	1200
2.	173	4	770
3.	130	3	770
4.	320	4	1910
5.	58	4	1100
6.	90	3	1700

NOTE: Average 42% impervious cover.

STREAMS

NO.	LENGTH, FEET	FLOWLINE SLOPE, PERCENT	BOTTOM WIDTH, FEET
A	6800	0.8	2
PONDS	3000	---	500
B	7300	1.1	2
C	2300	1	8

NOTE: Average stream side slope 2, horizontal/vertical.

was divided into subcatchments and stream reaches. Table 3 has the physical characteristics of the basin's elements. All the subcatchments' overland slopes are set at three percent because that was the average local slope found by the grid point method discussed previously, taking section corners as grid points, and there was not much variation from subcatchment to subcatchment, as may be seen in the tables for Deep Fork Creek at Portland Avenue and Bluff Creek. Belle Isle Lake, covering fifty acres, receives flow from Stream B, and it has flood gates, but it was modeled as a flow-through uncontrolled pond (wide flat conduit segment) for this study because the flood gates were not operated during any of the rainfall events used in this study. Of the total length of stream segments modeling this basin, at most twenty-five percent had been concrete lined and straightened at the time of the rainfall events used in this study.

TABLE 3

PHYSICAL CHARACTERISTICS OF THE BASIN UPSTREAM

OF DEEP FORK CREEK AT EASTERN AVENUE.

SUBCATCHMENTS

NO.	AREA, ACRES	LENGTH, FEET
1.	2960	3220
2.	3880	4220
3.	670	1530
4.	1870	4300
5.	670	1460
6.	1680	3660
7.	370	1280
8.	1880	6560
9.	670	1420
10.	930	1970
11.	370	2460
12.	2110	14140

NOTE: Average 35% impervious cover.
Average 3% overland slope.

STREAMS

NO.	LENGTH, FEET	FLOWLINE SLOPE, PERCENT	BOTTOM WIDTH, FEET
A	40,000	0.5	9
B	15,000	0.7	5
LAKE	4,000	-	2000
C	20,000	0.7	5
D	12,500	0.3	20
E	20,500	0.8	5
F	6,500	0.15	30

NOTE: Average stream side slope 3, horizontal/vertical.

TABLE 4

TIME-AREA HISTOGRAMS OF THE THREE BASINS.

Cumulative Percentage of Each Basin Under its Time-Area Ordinates.

TIME-AREA ORDINATE	DEEP FORK CREEK AT PORTLAND AVE.	BLUFF CREEK AT NORTHWEST HIGHWAY	DEEP FORK CREEK AT EASTERN AVE.
1.	1	1	2
2.	2	2	3
3.	5	5	7
4.	8	8	10
5.	14	13	15
6.	20	17	21
7.	26	23	28
8.	33	30	36
9.	39	36	45
10.	45	43	55
11.	51	51	62
12.	57	60	70
13.	64	68	76
14.	70	76	82
15.	77	83	86
16.	83	90	90
17.	89	94	93
18.	94	98	96
19.	97	99	98
20.	100	100	100

CHAPTER III

URBAN RAINFALL-RUNOFF MODELS

This chapter is a description of the models calibrated on the observed records of rainfall and runoff as described in the previous chapter. These are by no means all of the models available for use in urban hydrology--these were selected because they are commonly used by Federal agencies (in the case of the River Forecast Center's model, it may soon be in common use by that agency). There are many models sponsored by states, cities, universities, and private interests, but a limit must be set somehow in testing the dozens of models available, and it was decided to limit this study to Federally sponsored and readily available programs. This is by no means a complete description of each model, for some of the descriptions, as published by the sponsoring agencies, are whole books. Only a small portion of some of the models was used in this study, and this chapter describes the theory of that portion of the respective model used here, with no attempt to prove the theory by a derivation from "first principles."

Rational Method

The model still apparently most used by urban design engineers for generating peak runoff rates from small basins is the Rational Method, almost one hundred years old (4). It is used by some Federal agencies,

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4. Clark, John S., Warren Viessman, Jr., and Mark J. Hammer, Water Supply and Pollution Control, 2n Ed. Scranton, Penn: International Textbook Company, 1971, pp. 207-210.

such as the Department of Housing and Urban Development, in reviewing engineering plans. Its formula is

$$Q = CiA \tag{1}$$

where

Q is the peak runoff rate, in cubic feet per second (cfs).

C is a dimensionless coefficient between 0 and 1.

i is rainfall intensity, in inches/hr.

A is drainage area, in acres.

Equation (1) is almost dimensionally correct in English units, for 1 inch of rainfall applied at a uniform rate for 1 hour onto 1 acre is 1.008 cfs.

The drainage area, A, is the size of the basin contributing runoff to the particular spot where the formula is being used to estimate the peak runoff rate.

The coefficient, C, is called the "runoff coefficient," and in practice is taken as a function of the percentage of impervious cover (streets, rooftops, driveways, sidewalks, parking lots) on the basin, and the infiltration rate (surface porosity) of the ground of that portion of the basin being pervious. The American Society of Civil Engineers has a policy of design guidelines on C (5), while Oklahoma City and most surrounding cities have by ordinance set a minimum C of 0.7 (6). Baltimore County has a design chart relating C to percentage of impervious cover

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5. Design and Construction of Sanitary and Storm Sewers, American Society of Civil Engineers and the Water Pollution Control Federation, 1970, pg. 51.

6. "Methods for Calculating Stream Flow and Runoff," Oklahoma City Code, Chapter 15A-4, 1975.

and ground slope (7). There has also been research toward relating C to time since the beginning of the storm under study (8, 9), and to the rainfall intensity (10, 11).

The rainfall parameter, i, is the rainfall rate at the peak of a storm, the duration of that peak being the "time of concentration," or "response time" of the basin at the point under study. This time of concentration, t_c, is defined as the longest time it takes any drop of water falling on the basin to reach the point in the basin where the peak flow-rate is being computed. In practice, it is the length of time it takes water to travel from the most removed point in the basin to the outfall where the peak flow is being computed. In an urban environment, that time is broken down into "inlet time," the length of time it takes the water to travel overland across yards, parking lots, and streets, from the most removed point in the basin, to the first storm sewer inlet it encounters, and "storm sewer time," being the length of time it takes the water to flow from that upstream inlet, through the storm drain pipes, conduits, and open channels, down to the point where the peak runoff rate

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7. Clark, Water Supply, pg. 210.

8. Design and Construction, pg. 52.

9. Chien, Jong-Song, and Krishan K. Saigal, "Urban Runoff by Linearized Subhydrographic Method," Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 100, No. HY8, pp. 1141-1157.

10. Design and Construction , pg. 52.

11. Schaake, John C., Jr., John C. Geyer, and John W. Knapp, "Experimental Examination of the Rational Method," Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 93, No. HY6, pp. 5607-5614.

is being computed. The author is familiar with nine different graphs or equations for estimating inlet time (12-19), various ones employing length of overland travel, surface slope, nature of the surface (paving, grass, etc.), rainfall intensity, and the runoff coefficient. In engineering design, inlet time is taken from five to about thirty minutes. Storm sewer time is usually obtained by computing the full-flow velocity of the elements of the storm sewer, applying the respective segment's velocity to its length to get incremental travel time in the respective elements, and adding those times together as the water would progress downstream through the system. For purpose of this study, rainfall and runoff records were available for each basin as discussed in Chapter II, so the time of concentration for each basin was found by solving for it, using the optimization routines in HEC-1 and G824, computer programs to be described in following sections.

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12. Clark, Water Supply.

13. Ragan, R.M. and J.O. Duru, "Kinematic Wave Nomograph for Times of Concentration," Journal of the Hydraulics Division, American Society of Engineers, Vol. 98, No. HY10, pp. 1765 - 1771.

14. "Residential Storm Water Management--Objectives, Principles, & Design Consideration," The Urban Land Institute, American Society of Civil Engineers, and National Association of Home Builders, 1975, pg. 30.

15. "Overland Flow Time Chart," Oklahoma City Engineering Department, no date.

16. "Technical Manual, "Oklahoma Highway Department, 1970, pg. 6356.

17. Eagleson, Peter S., Dynamic Hydrology, McGraw-Hill Book Company, 1970, pg. 340.

18. Mockus, Victor, National Engineering Handbook, Section 4, Hydrology, U.S. Soil Conservation Service, 1972, pg. 15-8 and 15-10.

19. Schaake, "Rational Method."

Once a basin's time of concentration is determined, it is possible to find the rainfall intensity for given storm events. In engineering design, one reads a chart, displaying return-frequency rainfall time-durations, for the particularly intensity for a storm return-frequency for the time of concentration of his particular basin of interest. The author is familiar with two such graphs for metropolitan Oklahoma City, one by the Oklahoma City Engineering Department (2), and the other by the Oklahoma Department of Transportation (21). The merits or demerits of neither chart will be discussed here, for this research dealt with twenty-three observed rainfall-runoff events, and not hypothetical rainfalls. For this study, a rainfall event's intensity over a basin was taken as the maximum average intensity during a period equal to the basin's time of concentration, as illustrated in Figure 14.

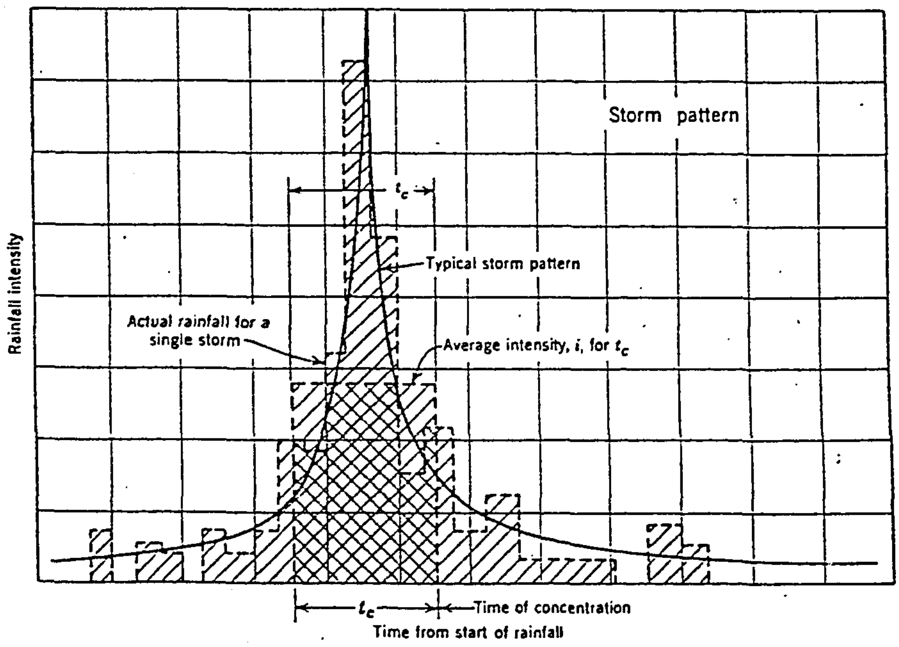

Figure 14. Rainfall hyetograph and averaging for time of concentration. From (5).

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20. "Rainfall Rate Intensity Frequency Curve," Oklahoma City Engineering Department, no date.

21. "Technical Manual," Oklahoma Highway Department, 1970, pg. 637.

The Rational Method as traditionally employed has many disadvantages compared to models to be discussed later. It predicts only the peak rate of runoff, not runoff volume, and cannot account for flood attenuation or storage on the flood plan or in lakes or retention ponds, so it gives results too high downstream of retarding structures and on basins so large that attenuation is a factor on the runoff--those over a few hundred acres in size. In spite of its shortcomings on basins over a few hundred acres in size, it is much used by designers on basins of several square miles in size, such as the drainage study in (22), so it is calibrated on the basins used in this study by entering Equation (1) with the basin's drainage area, the respective storm's intensity as determined above, the storm's observed peak runoff rate, and solving for the runoff coefficient, C.

Many attempts have been made to modify the Rational Method to overcome its shortcomings, particularly its failure to account for rainfall volume (23), and a particularly attractive method recently presented turns Equation (1) into a linearized subhydrograph with a methodology capable of hand calculation to generate complete outflow hydrographs from observed or synthetic storms (24).

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22. Rea Engineering & Associates, Inc., "Deep Fork," Oklahoma City, 1965. (Report to the Oklahoma City Engineer.)

23. "Practices in Detention of Urban Stormwater Runoff," American Public Works Association, National Technical Information Service Publication PB-234-554, 1976.

24. Chien, "Urban Runoff."

U. S. Soil Conservation Service: Project Formulation Hydrology (TR-20)

In 1965 the Soil Conservation Service implemented its computer program for story water runoff, intended as a design tool for flood detention/prevention structures on agricultural basins. The program computes a complete hydrograph for surface runoff resulting from any synthetic or natural rainstorm. It can take into account conditions having a bearing on runoff, and will route the hydrograph through stream channels and reservoirs. It can combine the routing hydrographs with those from other tributaries and print out the resulting hydrograph, and the water surface elevations for the hydrograph points, at any desired cross section or structure. For purpose of this research, only part of the program's capability was utilized--that of producing a runoff hydrograph from a rainfall storm assumed to be uniformly distributed in area over a basin assumed to have an areally uniform rainfall loss equation. The paramters entered into the computer model in order to generate a synthetic outflow hydrograph from a basin for a particular storm are

1) The cumulative rainfall hydrograph.
2) The basin's surface area.
3) The basin's time of concentration.
4) The basin's dimensionless unit hydrography shape.
5) The basin's curve number, relating to rainfall loss, at the time of the storm.

Parameters 3), 4), and 5) are explained in later paragraphs. A complete description of the computer program is contained in its users' manual (25).

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25. "Computer Program for Project Formulation--Hydrology--TR-20," U.S. Soil Conservation Service, 1965.

The program's model for dealing with rainfall infiltration, transpiration, surface evaporation, and other losses, is expressed in the following equation:

$$Q = \frac{(P - 0.2S)}{P + .08S} \tag{2}$$

where

Q is the cumulative runoff, expressed as inches depth uniformly distributed over the basin, at any instant.

P is the cumulative rainfall, expressed as inches depth uniformly distributed over the basin, at the same instant.

S is the maximum potential rainfall loss by infiltration, etc., in inches, and is expressed by the equation,

$$S = \frac{1,000}{CN} - 10 \tag{3}$$

so that

$$CN = \frac{1,000}{S + 10} \tag{4}$$

This variable, CN, is called the "curve number," which is entered into the computer program. Thus all rainfall losses may be expressed by this one parameter, the curve number. The curve number for a particular storm over a particular basin depends on the basin's hydrologic soil types, nature of the vegetative cover, percent impervious cover, and antecedent rainfall preceeding the storm in question. References 26 and 27 have the step by step procedure to follow in getting a curve number for an event

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26. "Urban Hydrology for Small Watersheds--TR-55," U. S. Soil Conservation Service, 1975.

27. Mockus, Hydrology.

when no rainfall-runoff records are available. However, for this study such records were available for each storm over each basin, so Equations (2), (3), and (4) were solved to find the curve numbers for the calibration process.

The rainfall which does not soak into the ground or evaporate or remain attached to the vegetation and group covers, results in a direct runoff, and is known as rainfall excess. This computer program's model for generating a runoff hydrograph from the rainfall excess is a dimensionless unit hydrograph, which is the hydrograph of the direct runoff that would be observed at the downstream outfall of a drainage basin one unit in area as a result of one unit of rainfall except occurring within a one unit time interval. See Figure 15 for the shape of the dimensionless unit hydrograph as built into the computer program, and the meaning of the terms used in describing the unit hydrograph. The program user may modify the standard unit hydrograph and enter any shape of his choosing.

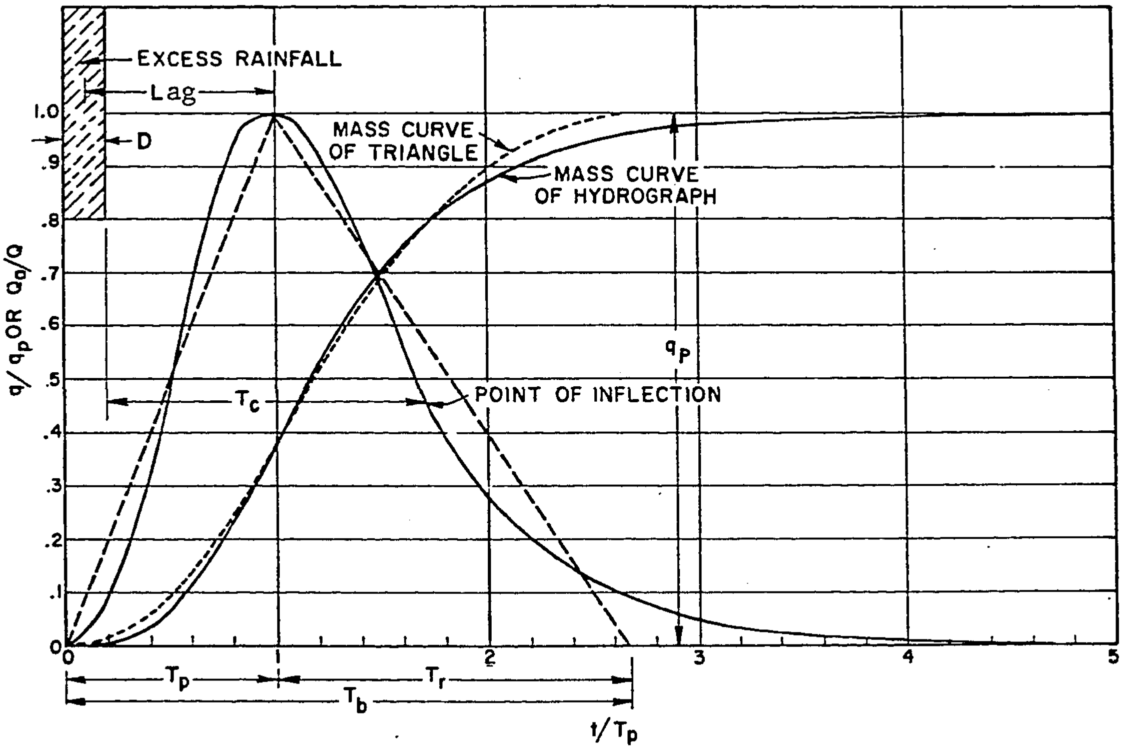

Figure 15. Dimensionless curvilinear unit hydrograph and equivalent triangular hydrograph. From (27).

Note that the time of concentration, T_c, is the time from the end of excess rainfall to the point of inflection of the unit hydrograph. When using the standard dimensionless unit hydrograph, the time duration of the unit rainfall excess is equal to $.1333\ T_c$. For the standard dimensionless unit hydrograph built into the program, 37.5 percent of the volume of the hydrograph is on its ascending side, between the beginning of runoff and the peak, while the remainder of the volume is on the descending limb, occurring after the peak discharge. That ratio is converted to a "peak rate factor," K, by the equation

$$K = 645.33 \frac{2}{1 + T_r/T_p} \qquad (5)$$

where

T_r/T_p is the ratio of the volume under the descending limb of the unit hydrograph, to the volume under the rising limb, and 645.33 is a conversion factor of one inch of rainfall per one hour over one square mile, to cubic feet per second.

This "peak rate factor," which is 484 for the standard unit hydrograph built into the program, will be discussed in later sections in the chapter for its relation to other unit hydrograph models.

Few basins meet the criteria for the standardized unit hydrograph--one square mile area and one hour time-to-peak. Unit hydrographs for real basins are generated in the program by multiplying the ordinates of the standard dimensionless hydrograph by the area of the basin in square miles, and multiplying the abscissa by the time-to-peak of the basin in hours. The area of the basin is planimetered from a map, and when modeling an ungaged basin, the hydrologist figures the basin's time of concentration or lag time using standard Soil Conservation Service procedures presented in (28), which computes the basin's time-to-peak for figuring the particular basin's unit hydrograph. For this study, observed rainfall-runoff records were available, so each basin's time of concentration was found by a trial-and-error process, stopping when the mean error between observed time of peak flow and synthetic time of peak agreed within two or three minutes, over the storms used in each basin's calibration process.

None of the storms used in the study meet the definition of the unit storm, producing one inch of runoff uniformly distributed over the basin during one unit time interval--they are complex storms covering several unit time intervals producing various amounts of runoff excess during those several time intervals. The theory of the unit hydrograph is that the outflow hydrograph of the excess runoff of an individual unit time interval is a direct multiple of that amount of excess, so

$$Q_i = U_i E_i \tag{6}$$

where

Q_i is the outfall hydrograph ordinate for any unit time increment, i.

U_i is the unit hydrograph ordinate for that same time increment, i.

E_i is the rainfall excess runoff for that same time increment, i.

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28. Mockus, Hydrology.

Graphically, the outflow hydrograph is the same as the unit hydrograph with the ordinates scaled up or down as the excess exceeds or falls short of one inch. For complex storms producing excess runoff over several time intervals, the individual hydrographs resulting from the various time intervals are computed in sequence and the individual components are added, as in the example Figure 16. The computer program performs the calculation by a convolution

$$Q_i = \sum_{j=1}^{n} U_j \; E_{i-j+1} \tag{7}$$

where

i is the sequence number of time interval.

Q_i is the outflow hydrograph ordinate for time interval period i.

U_j is the jth unit hydrograph ordinate.

n is the number of unit hydrograph ordinates, or rainfall excess time intervals, whichever is smaller.

E_{i-j+1} is the rainfall excess runoff in inches, in reverse order from i through i - n.

This model as implemented by the Soil Conservation Service is an engineering design tool and has no automatic calibration capabilities, so it had to be calibrated "by hand"--trial and error. The parameters were total volume of runoff, peak flow rate, and time of peak discharge. Calibration of volume of runoff was easily done by adjusting the curve number, CN. That was terminated and the model considered calibrated when the mean difference between observed and synthetic volumes agreed within about 5 percent, which was well within the 50 percent confidence interval,

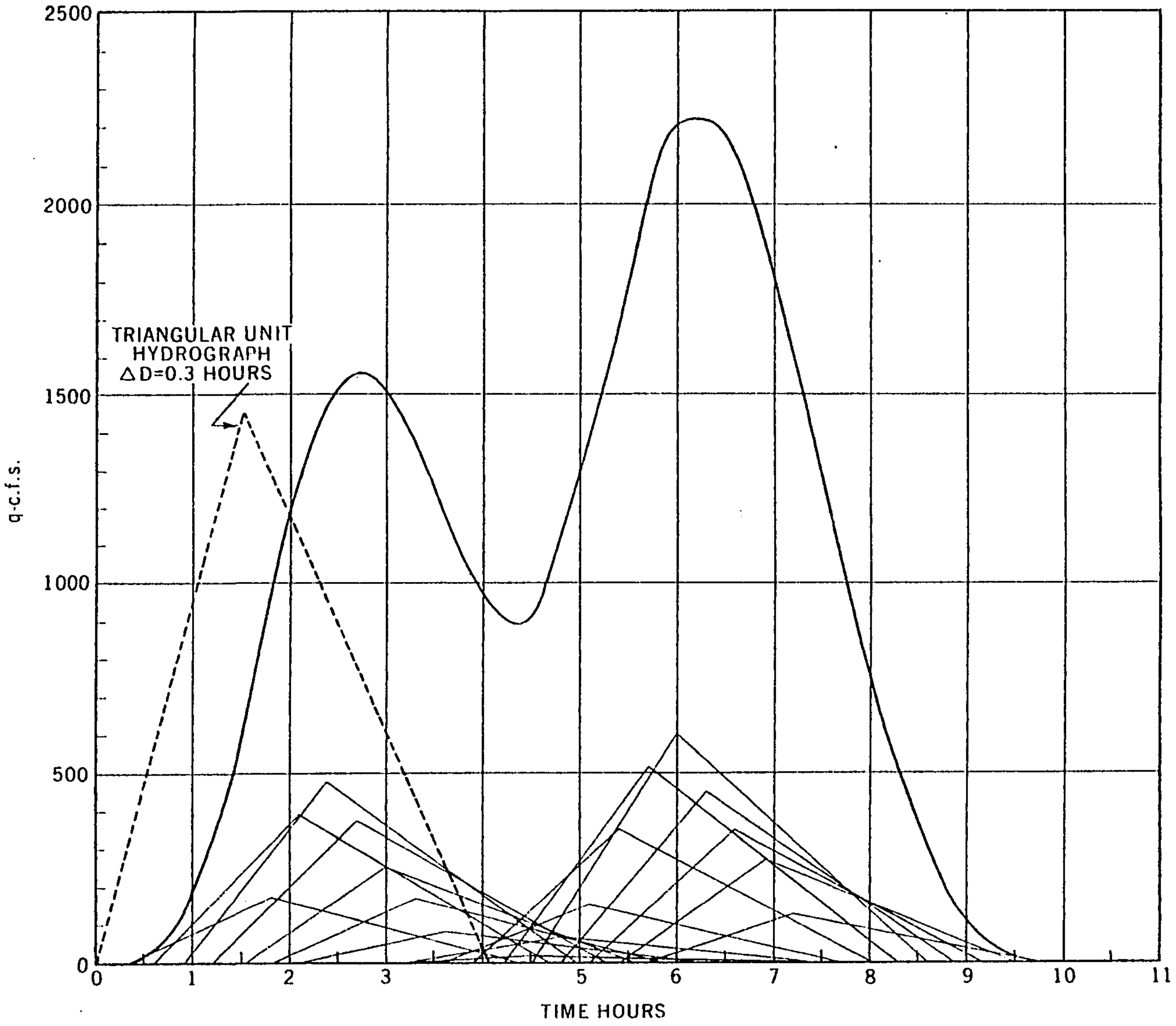

Figure 16. Composite flood hydrograph. From (28).

as will be discussed in the following chapter. Calibration of peak flow rate was done by adjusting the "peak rate factor," or shape of the unit hydrograph, until the mean of the ratios of synthetic peak to observed peak was unity plus or minus 5 percent, which is the U. S. Geological Survey's estimate of the accuracy of its measurements. It was found necessary to make that adjustment on only one basin's unit hydrograph--Bluff Creek, probably because of the influence of storage in upstream ponds, as discussed in the previous chapter. A parameter found to have greater impact on peak flow rate was the unit hydrograph's time of concentration. It was discovered that adjusting the time of concentration for peak discharge also gave a synthetic time of peak that was very close to the observed time of peak on the average.

U. S. Geological Survey Urban Flood Hydrograph Synthesis Model (G824)

In 1972 and 1973 research hydrologists with the Geological Survey published documentation for a computer program calibrating a rainfall-runoff model for natural basins (29, 30), and that model was later modified to one used in this study, calibrating urban basins, being capable or considering multiple rain gages on the basin, each with its own rainfall record, and capable of accounting for different areas of the

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29. Dawdy, David R., Robert W. Lichty, and James M. Bergmann, "A Rainfall-Runoff Simulation Model for Estimation of Flood Peaks for Small Drainage Basins," U. S. Geological Survey Professional Paper 506-B, 1972.

30. Carrigan, P. H., Jr. "Calibration of U. S. Geological Survey Rainfall/Runoff Model for Peak Flow Synthesis--Natural Basins," U. S. Geological Survey, National Technical Service publication PB-226-217, 1973.

basin having different percentages of impervious cover (31). This program is capable of long-term accounting of antecedent soil moisture over ten years of record, and is the only program considered in this study capable of such an internal accounting. The user may treat as many as twenty-five rainfall-runoff events, and each may have as many as three distinct peaks. As many as five rain gages may be used in the calibration, each with its own rainfall and pan evaporation data. The program takes daily rainfall totals and daily pan evaporation at each rain gage, and for those days having hydrographs to be used in the calibration process, the observed outfall hydrograph for the basin in unit time intervals of usually five to thirty minutes, and the observed rainfall records at the various rain gages again in short time duration increments. Smaller basins usually have shorter response times to rainfall, and their hydrographs are less regular over time, so their time steps used in modeling are usually shorter than the time steps used for larger basins.

This model has seven parameters to calibrate for rainfall excess resulting in runoff, more than any other model used in this study, and is the most sophisticated in its treatment of rainfall losses. In the following discussion, capital letters in parentheses after the description of a variable identify the variable's name as it is used in the computer program, and is done only for program data input parameters, so that model

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31. Carrigan, P. H., Jr., George R. Dempster, Jr., and David E. Bower, "User's Guide for U. S. Geological Survey Rainfall-Runoff Models - Revision of Open-File Report 74-33," U. S. Geological Survey Open-File Report 77-884, 1977. Chapter 14, "Calibration of Urban Basin Model."

users may quickly relate to the discussion. Let

d = constant drainage rate for redistribution of soil moisture, in inches per hour (DRN).

r_i = proportion of daily rainfall that infiltrates into the soil (RR).

k_p = coefficient to convert pan evaporation to potential evapotranspiration values (EVC).

m_c = soil moisture storage volume at field capacity, in inches (BMSM). It is a function of the depth of the soil.

k = minimum (saturated) hydraulic conductivity used to determine soil infiltration rates, in inches per hour (KSAT).

P_s = combined effect of initial moisture content and suction at the wetted front at field capacity, in inches (PSP).

r = ratio of the suction at the wetted front for soil moisture at wilting point to that at field capacity (RGF).

The above parameters are optimized by the computer model. Also let

t = a point in time during the period being calibrated, in days.

R = daily rainfall at time t, in inches per day.

e = daily pan evaporation rate at time t, in inches per day.

$i(t)$ = cumulative infiltration since the beginning of the period being calibrated, in inches.

Convert observed pan evaporation to potential evapotranspiration by setting

$$e_p = k_p e. \tag{6}$$

Then account for soil moisture during the time between flood events by

$$i(t+\Delta t) = \begin{cases} i\ (t) + r_i R - \Delta t\ (e_p+d), & i\ (t) + r_i R \geq \Delta t\ (e_p+d) \\ 0, & \text{otherwise} \end{cases} \qquad (7)$$

where Δt is one day.

In order to compute the soil moisture at the beginning of a storm event, two variables need to be defined. Let

$$e_p{*}\Delta t = e_p\Delta t - [i(t) - i\ (t+\Delta t)] \qquad (8)$$

and

m_o = initial soil moisture content at the beginning of one of the storm events used in the calibration, in inches.

Then when $d = 0$,

$$m_o(t+\Delta t) = \begin{cases} m_o(t) - e_p{*}\Delta t, & m_o(t) > e{*}_p\Delta t \\ 0, & \text{otherwise} \end{cases} \qquad (9)$$

But if $d > 0$,

$$m_o(t+\Delta t) = \begin{cases} m_o(t) + d\Delta t, & m_o(t) < m_c \\ m_c, & \text{otherwise.} \end{cases} \qquad (10)$$

When modeling one of the storm events used in the calibration, the computer goes to a Δt time step of five to thirty minutes, whatever is the time step employed to read in the observed hydrograph and rainfall amounts. In order to compute the runoff from a storm used in the calibration, three more variables must be defined. Fix a time t during the event, and let

R_e = rainfall excess during Δt, in inches. This is surface runoff used by the model to compute the outfall hydrograph.

S = rainfall supply rate during Δt, in inches per hours, obtained from the rainfall data fed to the model.

$F = \frac{di}{dt}$ the infiltration rate, in inches per hour.

Then from the modified Phillip equation for infiltration (32, 33),

$$F = \frac{di}{dt} = K \{ 1 + [rP_s - P_s(r-1)\frac{m_o}{m_c} /i] \} \tag{11}$$

and

$$R_e = \begin{cases} \Delta t^{1/2} \frac{S^2}{F}, & S < F \\ \Delta t \ (S - F/2), & \text{otherwise} \end{cases} \tag{12}$$

for the pervious parts of the basin. During the rainfall the cumulative infiltration is accounted by

$$i(t+\Delta t) = i(t) + \Delta t \ (S-R_e). \tag{13}$$

For the impervious area of the basin

$$R = \begin{cases} R -.05, & R > .05 \\ 0, & \text{otherwise} \end{cases} \tag{14}$$

The pervious soil of a basin is considered homogenous, and the seven parameters d through r are fixed for that basin--the computer program merely solves for their value by an optimization technique to be mentioned later. The variable m_o changes between each storm, but remains fixed during each particular storm used in the calibration. Variables S, e, and R are independent, being input data, and the rest are dependent.

It has been found through the experience of others using the model that it is most sensitive to the parameters k and P_s. The same three basins calibrated in this report were also calibrated using the version of this computer program for natural basins--it can simulate homgenous urban basins by assuming a uniform percentage of impervious cover--and the parameter d was set at 1.0 because past experience indicated the model is

32. Dawdy, "Rainfall-Runoff Model."

33. Carrigan, "Calibration."

virtually insensitive to that parameter. That calibration was conducted by Mr. W. O. Thomas, Jr. and Mr. Robert Corley of the U. S. Geological Survey (34).

For those storm events being calibrated and simulated, the rainfall excess R_e for each time increment Δt is converted to runoff volume by multiplying by the basin area, and converted to a translation hydrograph represented by a time-area histogram reflecting the effect of varying travel times in the basin. See Figure 17. That histogram is then routed through a storage element, and the output is the flood hydrograph. The procedure just described is known as the Clark method (35).

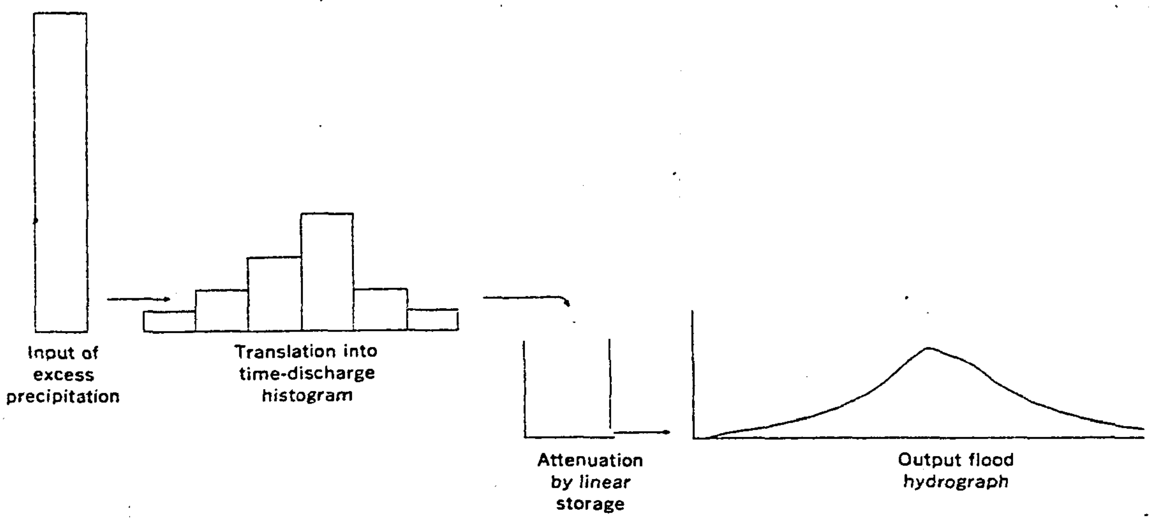

Figure 17. Schematic of the Clark unit hydrograph. From (29).

34. Thomas, "Flood discharges."

35. Clark, C. O., "Storage and the Unit Hydrograph," Transaction of the American Society of Civil Engineers, Vol. 110, pp. 1419-1488, 1945.

In following the description of the way this model implements the Clark method, refer to Figure 18. First a drawing of the boundary of the basin is partitioned into subbasins based on Theissen polygons about the rain gage sites. The Theissen method is common, and is described in other literature, such as (36), but for purpose of this report, let it be said that the boundaries of the Theissen polygons are perpendicular bisectors of the lines connecting the locations of adjacent rain gages. Thus a "subbasin" is not a drainage subbasin whose boundaries follow ridge and drainage divides, but an area whose rainfall is idealized and considered uniform over the area and represented by the point rainfall at the rain gage centered in the polygon. Figure 18 is divided into subbasins by rain gages I and II.

Next the time-area histogram is prepared. That is achieved by marking the basin into time-area bands by marking isochronic contours on the basin--lines representing equal travel time to the basin outfall, so that it takes rain falling on any point on a line the same time to reach the outfall stream gage as any other point on the same line. For using the Urban Flood Hydrograph Synthesis Model, there must be twenty such time-area bands. In practice the marking is usually done by placing the point of a dividing compass at the basin outfall point, and marking off concentric circles, as in Figure 18. Next each band is planimetered for its area.

Finally, each time-area band is divided into a pervious portion and an impervious portion. That is facilitated by dividing the basin up into land use categories each with its homogeneous percentage of impervious

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36. Linsley, Ray K., Jr., Max A. Kohler, and Joseph L. H. Paulhus, Hydrology for Engineers, Second Edition, McGraw-Hill Book Company, 1975, pg. 82.

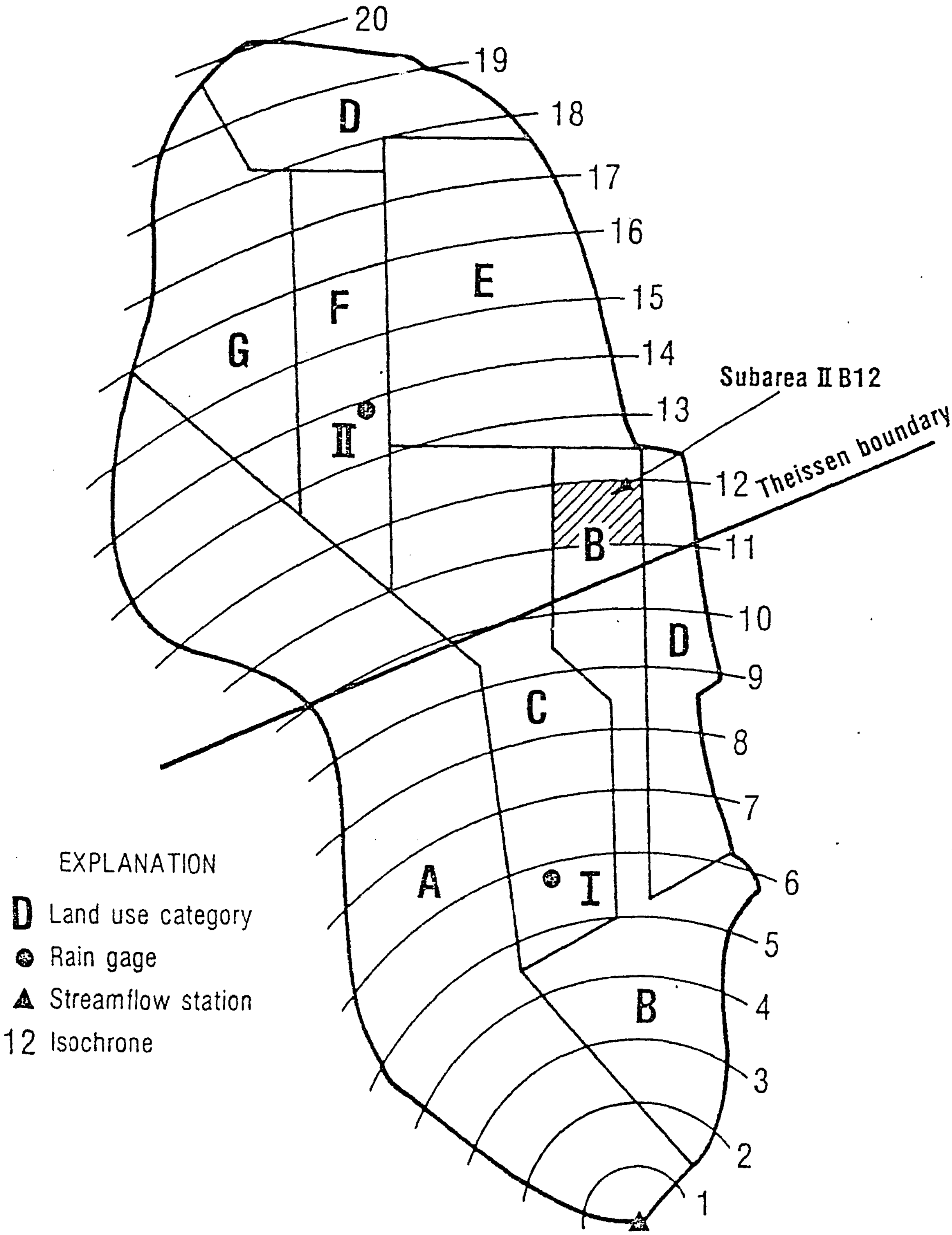

Figure 18. Sketch showing division of drainage area into subareas according to location of raingages, time of travel, and degrees of imperviousness. From (31).

cover, such as areas A through G in Figure 18. Next the area of each land use type in each time-area band is planimetered, multiplied by its respective percentage of impervious cover, and accumulated over the land use types. The result that is fed into the computer model is two time-area histographs per rain gage subbasin, one for impervious area, and other for pervious area.

The time-area histograms, or translation hydrographs, are convolved with the rainfall excesses determined by Equations (12) and (14) to produce the input into the attenuating linear storage reservoir. In particular,

$$I(t) = \sum_{j=1}^{n} T_j R_{e_{(t-j+1)}} \tag{15}$$

where

$I(t)$ = inflow to the storage reservoir at time t.

T_j = jth ordinate of the translation hydrograph.

$R_{e_{(t-j+1)}}$ = rainfall excess during time interval t-j+1.

Let k_s be the linear storage coefficient, then the ordinates of the basin's outflow hydrograph are

$$Q\ (t+\Delta t) = I(t) - [I(t) - Q(t)]e^{-\Delta t/k_s} \tag{16}$$

where

$Q(t)$ = outflow at time t.

The parameter k_s is the slope of the graph of $\log_e Q(t)$ versus t. It is called the linear storage coefficient because it means that outflow is a linear function of channel storage (37).

In this model the basin's "time of concentration," t_c, is defined as the time it takes inflow to the storage reservoir to cease, which would

37. Mitchell, William D., "Effect of Reservoir Storage on Peak Flow," U. S. Geological Water-Supply Paper 1580-C, 1962, pg. 5.

result from one unit time length of rainfall excess. Since the computer program user inputs twenty time-area ordinates, the time of concentration is divided by twenty to find the time length of each ordinate. That determines how many time-area ordinates are in each translation hydrograph ordinate. For example, suppose a certain basin's time of concentration is 100 minutes, so each time-area ordinate covers a time interval of five minutes, but the rainfall records, discharge records, and routing steps are in ten minutes, then it takes two time-area ordinates for each translation histogram ordinate to enter Equation (16).

Time of concentration and the linear storage reservoir coefficient are the last two parameters available for calibration in the model. They determine the shape of the outflow hydrograph. This model can calibrate its nine parameters by minimizing the three following objective functions;

$$U_1 = \sum_{i=1}^{n} (\log_e V_{o_i} - \log_e V_{s_i})^2 \tag{17a}$$

$$U_2 = \sum_{i=1}^{n} (\log_e P_{o_i} - \log_e P_{s_i})^2 \tag{17b}$$

$$U_3 = \sum_{i-1}^{n} [\log_e (P_{s_i} \frac{Vo_i}{Vs_i}) - \log_e P_{o_i}]^2 \tag{17c}$$

where

n = number of floods used in the calibration

Vo_i = observed runoff volume for event i

Vs_i = synthetic runoff volume for event i

Po_i = observed flood peak for event i

Ps_i = synthetic flood peak for event i.

The method this model uses for calibration is a modification of Rosenbrock's technique (38), which minimizes Equations (17a-17c) by means of a pattern search of the orthonormal vectors of the functions.

Other researchers (39,40) have added another aspect to the automatic calibration built into the program by doing trial-and-error adjustments to the real percentage of impervious cover to reduce the standard deviation of error of the estimate between the synthetic and observed volumes and peaks. It was done on the theory that not all an urban basin's actual physical impervious cover is really effective. For example, rain falling on roof tops of family dwelling have no roof gutters and spouts, merely falls off the roof onto the ground and has a chance to soak into the pervious soil. Such roof tops and sidewalks not connected to street gutters are called "disconnected impervious cover." It was found for the basins they sampled in Oklahoma City and Dallas that typically only about half an urban's impervious cover is actually effective in the sense of yielding almost total runoff, and probably represents "connected impervious cover." Following their lead, each of the basins in this study were also calibrated on the basis of percentage of impervious cover.

This model does not use directly a "unit hydrograph," such as does the previously described model, TR-20. Instead, this model can be thought

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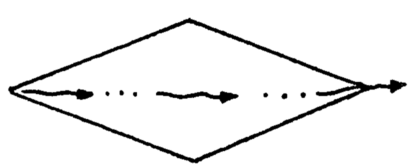
38. Carrigan, P.H., Jr., "Rosenbrock Technique for Determining Greatest or Least Value of A Function," Arlington, Va., U.S. Geological Survey Computer Contribution, 31 p.; available only from U.S. Department of Commerce, Nat'l. Tech. Inf. Service, Springfield, Va. 22151 as report PB-214 350, 1972.

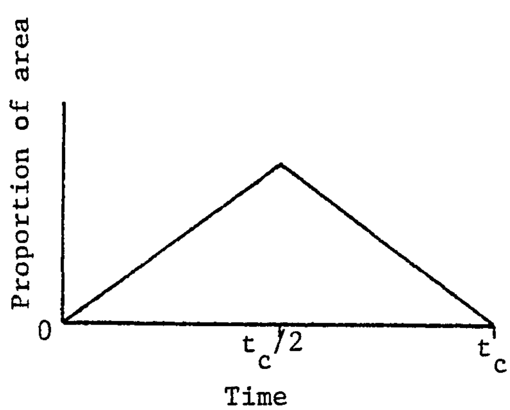
39. Thomas, "Flood Discharges."

40. Dempster, George R., Jr., "Effects of Urbanization on Floods in the Dallas, Texas Metropolitan Area," U.S. Geological Survey Water Resources Investigations 60-73 (Published by the National Technical Information Service as PB-230 188), 1974, pg. 13.

of as an assemblage of incremental unit hydrographs, each relating to a subarea and each having characteristics (lag time, storage coefficient) dependent on its location relative to the total basin outlet. However, it does use two parameters of the unit hydrograph--time of concentration and the linear channel storage coefficient--and a unit hydrograph for a basin can be calculated knowing those two parameters and the translation hydrograph. If exactly one unit of rainfall excess is input to Equation (15), then the input to Equation (16) is exactly the translation hydrograph, and the outfall hydrograph is the basin's unit hydrograph. Each basin has its own time-area histogram, but if one assumes a standardized histograph, such as in Figure 19, which would result from an idealized basin shape, such as in Figure 19, and assumes a basin area of one unit area and sets the time of concentration as one unit time, then the outflow



Idealized basin shape



Standardized time-area histogram

Figure 19. USGS standardized time-area histogram.

from Equation (16) is a dimensionless unit hydrograph, whose shape then depends on the storage coefficient. As part of this study, that was done by means of a computer program listed in Appendix A. That program was run for a wide range of storage coefficients, then Equation (5) used on the results to find the relation between the Soil Conservation Service's peak rate factor k, and the Geological Survey's ratio of storage coefficient to time of concentration. The result is displayed in Figure 20 and can be expressed by the equations

$$K = 590 - 229\ k_s/t_c \tag{18a}$$

$$k_s/t_c = 2.58 - K/229. \tag{18b}$$

It is seen that $k_s/t_c = 0.47$ corresponds to the standard Soil Conservation Service unit hydrograph with K = 484. That USGS hydrograph is compared to the SCS hydrograph in Figure 21 after adjustment so that the peak occurs at unit time.

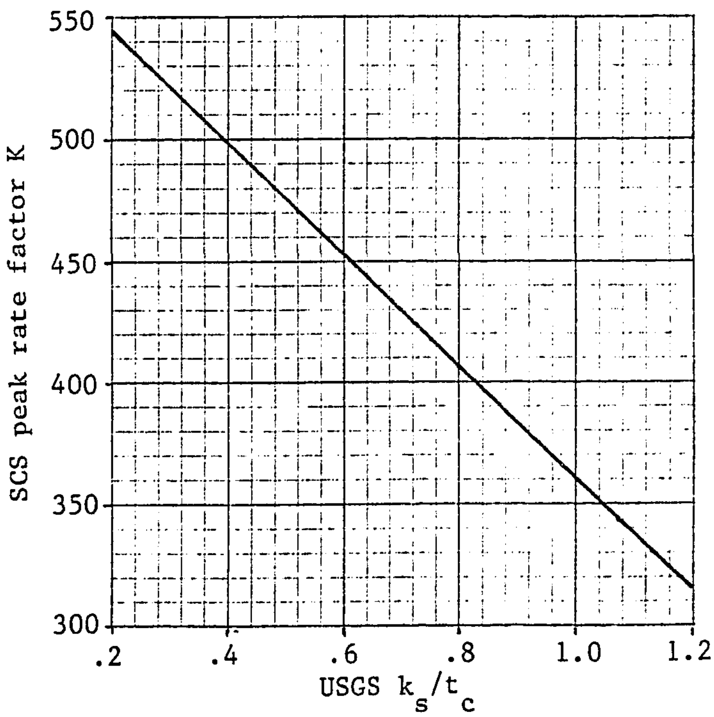


Figure 20. Comparison of USGS and SCS unit hydrograph shape factors.

Note a difference in computational procedure between the Geological Survey and the Soil Conservation Service. The SCS begins runoff with the beginning of unit rainfall excess, but the USGS begins runoff only after the end of rainfall excess in its model, which means both models are sensitive to the size of the input rainfall time interval step size, in different ways. Equations (18a) and (18b) and Figure 20 were computed for instantaneous rainfall excess, and are aproximations subject to variations of the length of the time step increments of the input rainfall. For the same basin, changing the time step size changes its computational time of concentration.

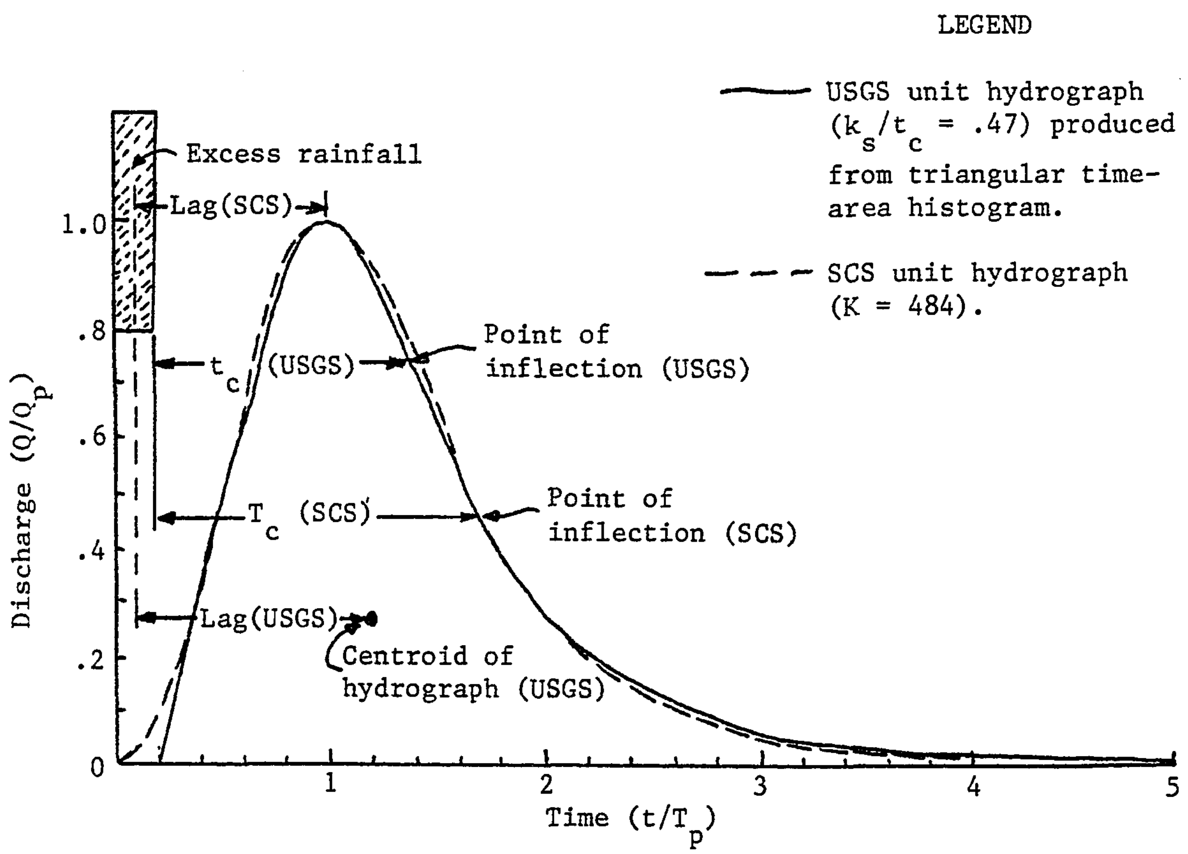


Figure 21. Relation between unit hydrographs of the USGS and SCS.

The Geological Survey conducted a study in Oklahoma on sixty basins calibrating its model for natural basins, which is a companion to the model used in this study, and assuming each basin's translation hydrograph to have the shape in Figure 19 (41), and it found the mean ratio k_s/t_c to be 0.77, not 0.47, with a standard deviation of 0.35, so that the SCS standard shape hydrograph fell within one standard deviation of the mean. For the standard conditions given in Figure 21, the SCS time of concentration is twenty-seven percent longer than that for the USGS, and lag for the USGS is ten percent longer than that for the SCS.

U. S. Army Corps of Engineers Flood Hydrograph Package (HEC-1)

In 1968 the Hydrologic Engineering Center of the Corps of Engineers published its computer program developed under the direction of Leo Beard for hydrograph computations, with the last published update being in 1973 (42). Like the Soil Conservation Services' TR-20, it was intended as a design tool for flood control and water resource projects, and has many capabilities. The types of jobs it can do are generalized precipitation, runoff, routing, and combining operations to simulate a watershed and its stream network; computations for specified precipitation depth-area storm relationships for an entire watershed; specialized precipitation streamflow network simulation relative to multiple floods for multiple plans of basin development and the economic analysis of flood damages; and otpimization of routing parameters. Those capabilities were not used in this study, because the only part of the computer program's ability used here was its routine to optimize unit hydrograph and rainfall loss rate parameters in calibrating them to observe outflow hydrograph records. However, unlike the Geological Survey's model, this one automatically optimizes on only one runoff event at a time, so it does not conduct a calibration on many years of continuous record. Thus each storm event generates its own set of parameter values which differ from storm to storm, but when modeling a basin from several storms there is a process to follow to

41. Thomas, "Flood Discharges."

42. U. S. Army Corps of Engineers, "HEC-1 Flood Hydrograph Package," Davis, California: Hydrologic Engineering Center, 1973.

resolve the values and get one number for each parameter. That process will be discussed after an explanation of each parameter and its function.

In the following discussion, capital letters in parentheses after the description of a variable identify the variable's name as it is used in the computer program, and is done only for program data input parameters, so that model users may quickly relate to the discussion. Observed rainfalls and outfall hydrographs are read into the computer program in incremental time steps 1, 2, 3, ..., t, beginning at the start of each storm event. To compute the infiltration, transpiration, and evaporation losses, make the following definitions:

L_t = loss from all causes during time increment t, in inches per hour.

k_t = basic loss coefficient in the same increment t, dimensionless.

d = initial accumulated rain loss during which k_t is increased, in inches (DLTKR).

D_t = increase in the loss rate coefficient during increment t corresponding to ten inches more of accumulated loss.

s = value at the beginning of a storm of the rainfall loss coefficient, inches per hour (STRKR).

r = ratio of the rain loss coefficient on an exponential recession curve to that corresponding to ten inches more of accumulated loss (RTIOL). Always exceeds 1.

E = exponent of precipitation that reflects the influence of rainfall rate on basin-average loss characteristics, and never exceeds 1.0 (ERAIN).

P_t = rainfall intensity during time interval t, in inches per hour (PRCP).

Each of the above parameters must be positive. The model is expressed by

$$D_t = 0.2d[1 - (\sum_{i=1}^{t-1} L_i)/d]^2 \tag{19a}$$

and

$$k_t = s/(0.1r \sum_{i=1}^{t-1} L_i) \tag{19b}$$

and

$$L_t = (k_t + D_t)\, P^E \quad . \tag{19c}$$

Figure 22 illustrates a graphical representation of the parameters. No provision is made for recovery of loss rate potential during periods of no rainfall.

The computer program optimizes parameters d, s, r, and E for each individual storm event by the univariate gradient search method to minimize the weighted standard deviation between computed and observed flows at each ordinate of the hydrographs input to the program. Errors associated with high flows are weighted more than low flow errors so as to improve the reproduction of peaks. The user then selects one value of E for each basin. In this study, the average value obtained from optimizing all the storms used over the basin was selected as the fixed value of E. Then the computer program is rerun, with E fixed for each basin, and the used selects a value to fix r for each basin. Again for this study, it was the average of the r for the several storms after E had been fixed. Next the user reruns the computer program with E and r fixed and optimizes s for each storm, then fixes a representative value for s for each basin.

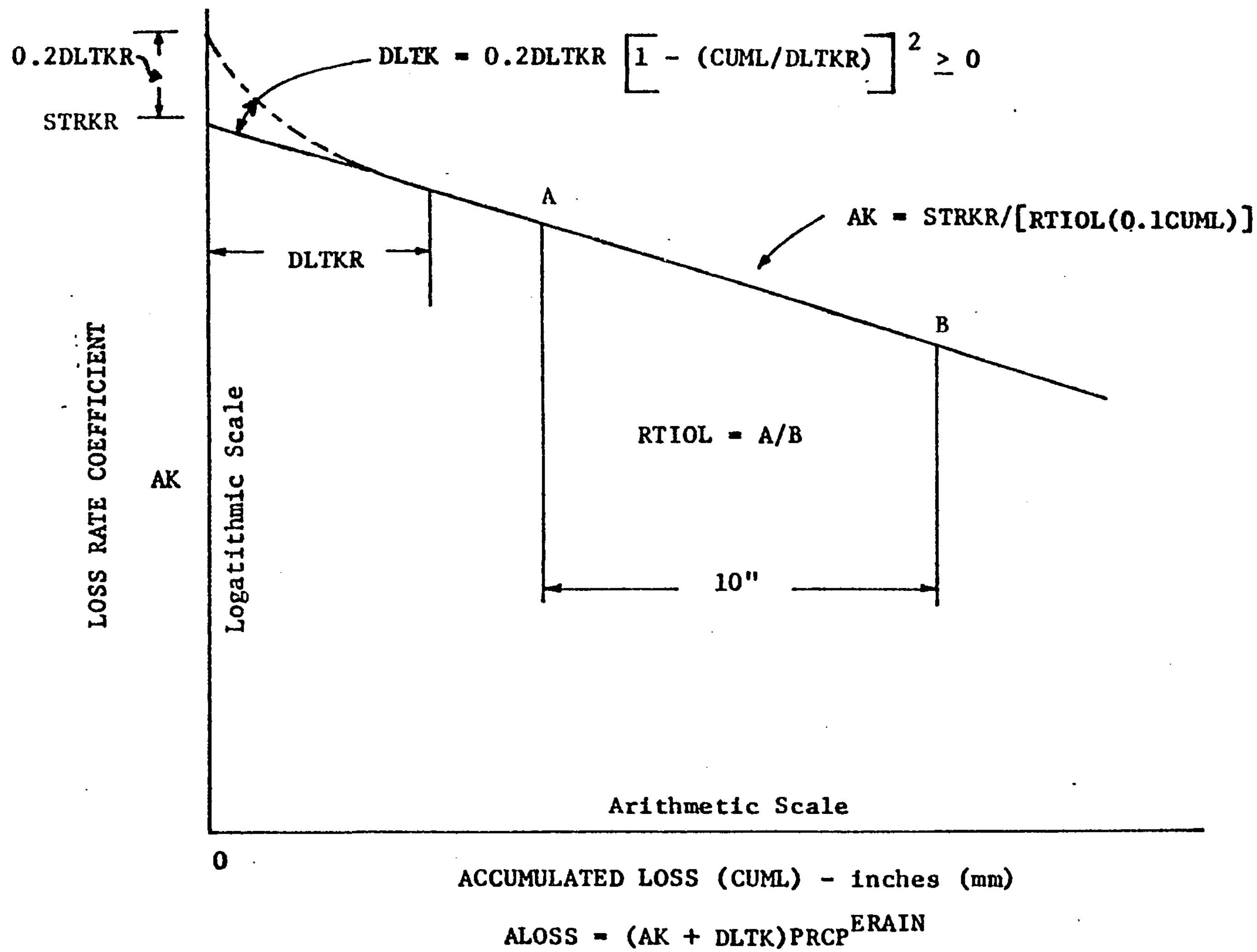


Figure 22. HEC-1 general loss rate function. From (42).

Again for this study, that was set at the average value of s. Finally, with E, r, and s fixed at single values for each basin, the user reruns the computer program. Now d is thought to vary from storm to storm depending on antecedent soil saturation, so the user develops a scheme for getting a value for d for each storm, which may be, say, a regression equation based on recorded antecedent rainfall/evaporation. For this study a fixed value of d was selected (again the mean) because it was found to be independent of antecedent rainfall/evaporation, as will be discussed in the following chapter.

Records of more than one rain gage may be read into the model for any storm, but the computer program merely averages the rainfall by time-step increment to get a basin wide average, so HEC-1 cannot spacially vary the rainfall inputs nearly so well as the Geological Survey's urban model during calibration. Also the optimization process in HEC-1 cannot account for impervious cover as a separate parameter--it is swallowed by the other parameters--but there is another Corps program not so widely known in which imperviousness for urban basins is an input and which can optimize on several storms at once instead of going through the sequential process described in the previous paragraph (43).

After optimizing the rainfall loss parameters for volume as described in the previous paragraphs, the model optimizes two more parameters to route the runoff and calibrate to the observe outflow hydrograph. Those two are Clark's time of concentration (TC) and storage coefficient (denoted R in HEC-1), as discussed in the Clark method in the

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43. U. S. Army Corps of Engineers, "Hydrologic Engineering Methods for Water Resources Development: Volume 4, Hydrograph Analysis," Davis, California: Hydrologic Engineering Center (Available from National Technical Information Service as Document AD-774 261), 1973, App. 3.

previous section on the Geological Survey's model. The user of HEC-1 may opt to use a standard time-area histogram built into the computer program, or he may input a single time-area histogram had by planimetering bands across the basin as described in the explanation of Figure 18. Only one histogram may be input per basin being calibrated, as opposed to several when using the Geological Survey's urban model, because this model does not admit imperviousness as a parameter in optimization, and only one rainfall hyetograph is used in optimizing even if several rain gage records are read in (they are averaged). The standard inbuilt histogram is different from the Geological Survey's standard displayed in Figure 19. The HEC-1 standard is expressed by

$$A = T^{1.5} / 0.707 \qquad (0 < T < .5) \tag{20a}$$

$$1 - A = (1 - T)^{1.5} / 0.707 \quad (.5 < T < 1.) \tag{20b}$$

where

A = area as a ratio to the total basin area.

T = time as a ratio to time of concentration.

Figure 23 shows the relation between the Corps' standard time-area histogram and the Geological Survey's.

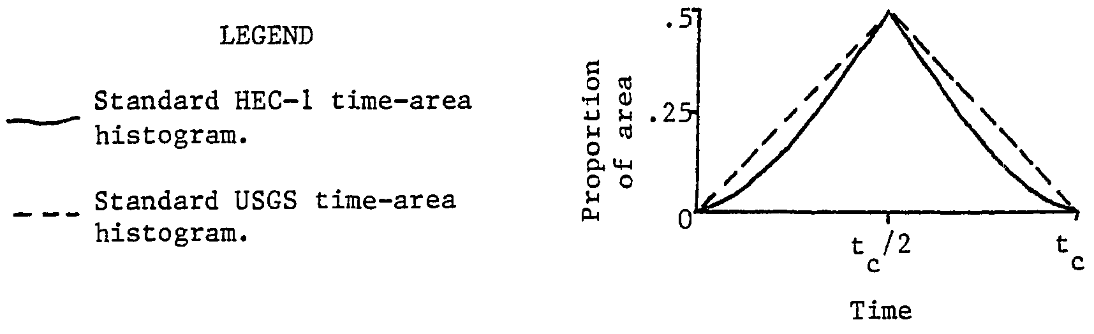

Figure 23. Comparison of USGS and HEC-1 standard time-area histogram.

At this point in the calculation the Corps and the Geological Survey diverse in their procedures, even though both are using Clark's method. The USGS convolves the runoff with the time-area histogram by means of Equation (15) and routes the resulting inflow through the storage reservoir using k_s in an exponential expression, Equation (16), to get the storm's outflow hydrograph without using directly a unit hydrograph. HEC-1, on the other hand, does use a true unit hydrograph. Translation through the time-area histogram is accomplished by

$$I_i = 645\ a_i/\Delta t$$

where

I_i = ordinate in cubic feet per second of the time-area runoff at the end of time period i, which must be between 0 and TC, the time of concentration.

a_i = planimetered ordinate of the time-area histogram, or from the standard histogram in Equation (20), at the end of period i.

Δt = time period of computational interval, in hours.

Whereas the USGS routes the translated hydrograph through a channel storage reservoir expressed as an exponential equation, in HEC-1 the attenuation is done by a convex equation,

$$O_i = CI_i + (1 - C)\ O_{i-1} \tag{21}$$

where

O_i = instantaneous unit hydrograph ordinate, in cfs.

C = dimensionless routing constant, expressed by

$$C = \frac{2\Delta t}{2R + \Delta t}$$

where

R = attenuation (storage) coefficient, in hours.

The parameter R is approximately equal to the ordinate of the unit hydrograph at the point of the time of concentration, divided by the slope of the unit hydrograph at that same point.

The instantaneous unit hydrograph expressed by Equation (21) is converted to a unit hydrograph for rainfall excess of duration Δt by averaging ordinates of the instaneous unit hydrograph at interval Δt apart,

$$Q_i = 0.5\ (O_i + O_{i-1}) \tag{22}$$

Because of the computational difference between USGS Equation (16) and HEC-1 Equations (21) and (22), there is usually a difference in parameter values between t_c and k_s (USGS) and TC and R (HEC-1) for the same basin and the same rainfall-runoff events. Based on a limited sample size of four watersheds (the three in this study plus one other) the following pattern seems to have emerged: Time of concentration is nearly the same for both models for the same basin, but k_s and R are related by the equations

$$R = k_s - .5 \tag{23a}$$

$$k_s = R + .5 \tag{23b}$$

and that relation is graphed in Figure 24.

As part of this study, the relationship between SCS unit hydrograph parameters and HEC-1 unit hydrograph parameters was investigated. Based on studies of eleven watersheds (the three in this report, five in the HEC-1 users' manual, and three others performed by the author), the

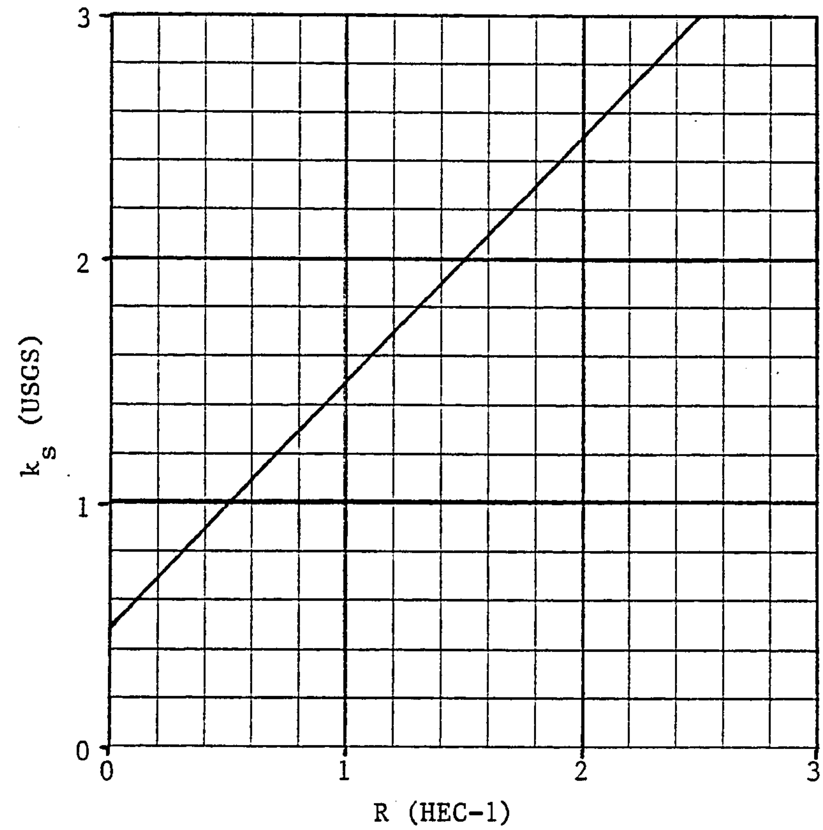

Figure 24. Comparison of USGS and HEC-1 unit hydrograph shape factors.

relation between the SCS unit hydrograph shape factor, K, and the HEC-1 unit hydrograph parameters, t_c and R, can be expressed by the equations,

$$K = 427.5\ (R/TC)^{-.48} \tag{24a}$$

$$R/TC = 296720\ K^{-2.08} \tag{24b}$$

which are graphed in Figure 25. The coefficient of determination, r^2, was 0.89 for the eleven samples.

The relationship between the standard SCS dimensionless unit hydrograph with K = 484, and that same hydrograph input into HEC-1 and optimized for t_c and R, is displayed in Figure 26. The standard time-

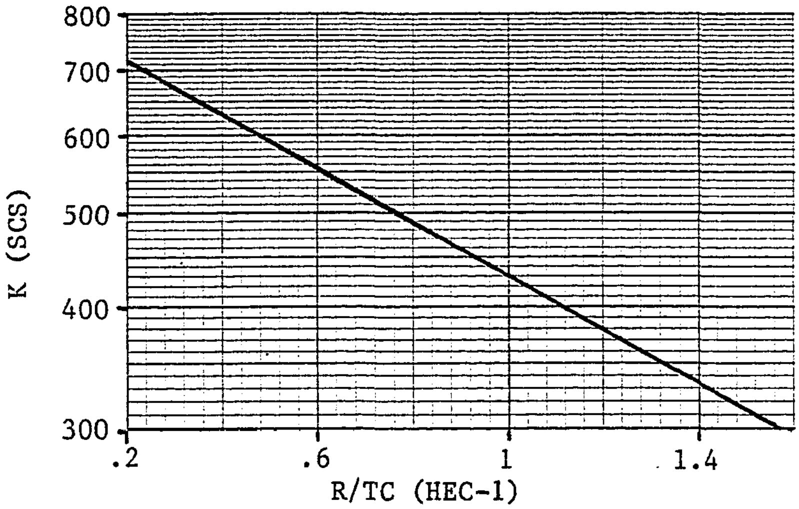

Figure 25. Comparison of SCS and HEC-1 unit hydrograph shape factors.

area histogram built into HEC-1 and displayed in Figure 23 was used. Notice that due to slight differences in shape that the points of inflection

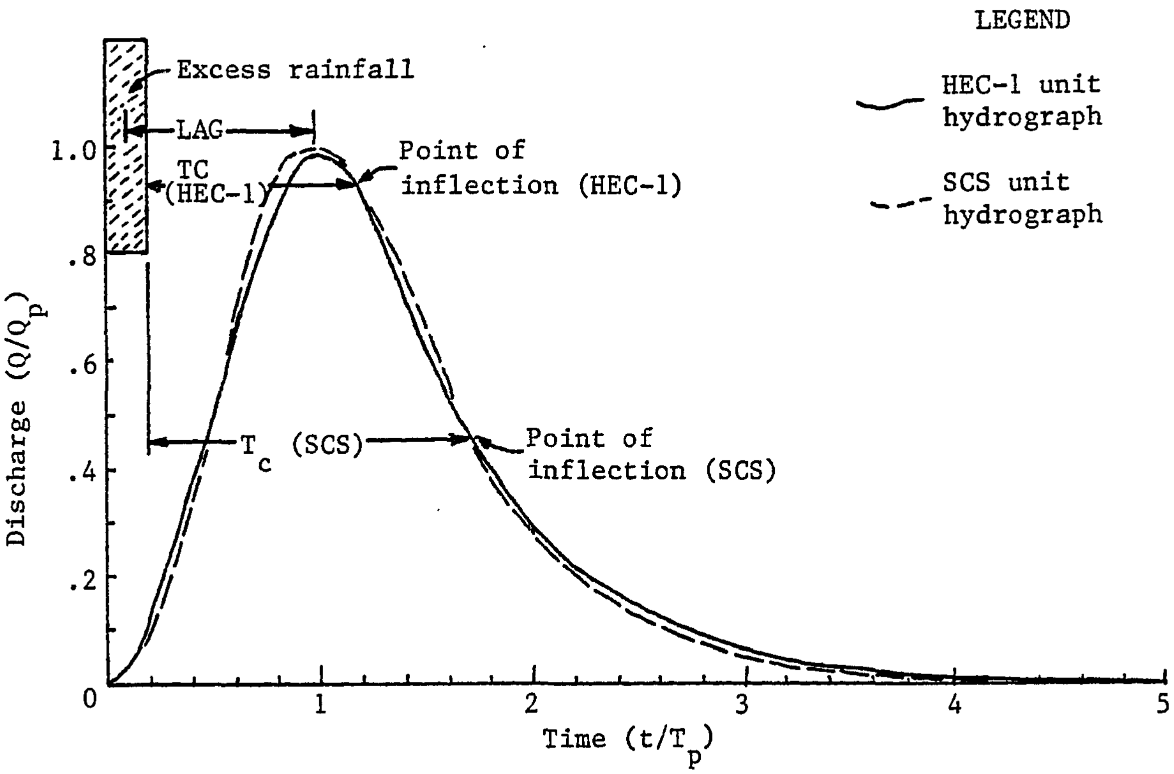

Figure 26. Relation between unit hydrographs of SCS and HEC-1.

are located differently, and therefore the times of concentration are different. Note that TR-20 and HEC-1 both produce runoff during the period of rainfall excess, and are unlike the USGS which has zero discharge at the precise end of rainfall excess, as shown in Figure 21.

For unit hydrographs not of standard shape, $K \neq 484$, TR-20 was programmed so that its time of concentration (Figure 15) is very nearly 1.67 times the unit hydrograph's lag (Figure 15), but is slightly influenced by the size of the computational time step interval, Δt. It has been found from the experience of this study, to be summarized in Chapter IV, that HEC-1's time of concentration is usually ten to twenty percent longer than the time to peak and that its time of concentration is usually slightly shorter than TR-20's time of concentration for the same basin.

Leo Beard recently completed a study of urban hydrology using HEC-1, including two of the basins in this report, and developed regression equations and graphs for TC + R, TC, and R, relating them to size of the drainage area (44).

In addition to optimizing parameters of Clark's unit hydrograph for each basin, HEC-1 also optimizes parameters of Snyder's unit hydrograph (45). Two parameters, LAG and CP, are used in Snyder's method to

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44. Beard, Leo R., and Shin Chang, "An Urban Runoff Model for Tulsa, Oklahoma," Austin, Texas: Center for Research in Water Resources, the University of Texas, 1978, pg. 61.

45. "Engineering and Design: Flood-Hydrograph Analysis and Computations," U. S. Army Corps of Engineers Manual EM 1110-2-14-5, 1959, para. 19.

describe a unit hydrograph's time of peak and peak discharge. LAG is much like the SCS lag in Figure 15, except that the standard unit rainfall duration is not twenty percent of the time to peak, like in the SCS model, it is seventeen percent in Snyder's method. The relation between the parameters is defined by,

$$LAG = 5.5\ t_r \tag{25}$$

$$Q_p = \frac{640 \times CP \times A}{LAG} \tag{26}$$

where

t_r = duration of unit rainfall excess, like D in Figure 15.

A = drainage area, in square miles.

Q_p = peak discharge, in cfs.

Some adjustments are made inside the computer program when the relationship between the length of the rainfall input interval and LAG is different from that expressed in Equation (25).

The shape of a unit hydrograph, whether it is lean and sharp-crested, or fat and broad-crested, is determined by the relationship between LAG and CP in Equation (26), just as K in Equation (5) in the SCS dimensionless unit hydrograph, and the relation between t_c and k_s for the USGS hydrograph, and the relation between TC and R for Clark's hydrograph in HEC-1, determine the shape of the unit hydrograph. The relation between Clark's TC and R in HEC-1 and LAG and CP has recently been investigated (46, 47), and will not be repeated here.

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46. Beard, "Tulsa," pp. 15, 41.

47. Russell, Samuel O., Bruce F. I. Kenning, and Greg J. Sunnell, "Estimating Design Flows for Urban Drainage," Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 105, No. HY1, pp. 43-52.

In 1977 Mr. Dale Reynolds of the Tulsa District of the Army Corps of Engineers conducted a study relating LAG to drainage basin main channel length and slope, and relating q_p to LAG, where

$$q_p = Q_p/A$$

so that q_p is peak discharge per square mile of basin (48). His study was based on rainfall-runoff analysis by HEC-1 of twenty-three basins, including five urban basins, of which three were those of this report. His graphs have not been published, and are included in Figures 27 and 28 because they are not readily available. As part of this study, a regression was done on his data on rural basins, and it was found that the following equations describe his curves:

$$LAG = 1.32 \left(\frac{LL_{ca}}{\sqrt{s}}\right)^{.41} \tag{27}$$

$$q_p = 395(LAG)^{-.98} \tag{28}$$

where

L = length of the basin's main watercourse, in miles, from upper tip of the basin to the outfall.

L_{ca} = length along the main watercourse from the basin's center of mass to the outfall, in miles

s = slope of the main water course, in feet per mile

The respective coefficients of determination (r^2) of Equations (27) and (28) are 0.97 and 0.99, which are quite good.

U.S. Environmental Protection Agency Storm Water Management Model (SWMM)

In 1971 the Environmental Protection Agency released its Storm Water Management Model (SWMM), a comprehensive mathematical model, capable of representing urban storm water runoff, to aid in planning, evaluating, and managing overflow abatement alternatives (49). SWMM is by far the

48. Reynolds, Dale, personal communication, June 1978.

49. "Storm Water Management Model, Volume I--Final Report," U.S. Environmental Protection Agency, 1971.

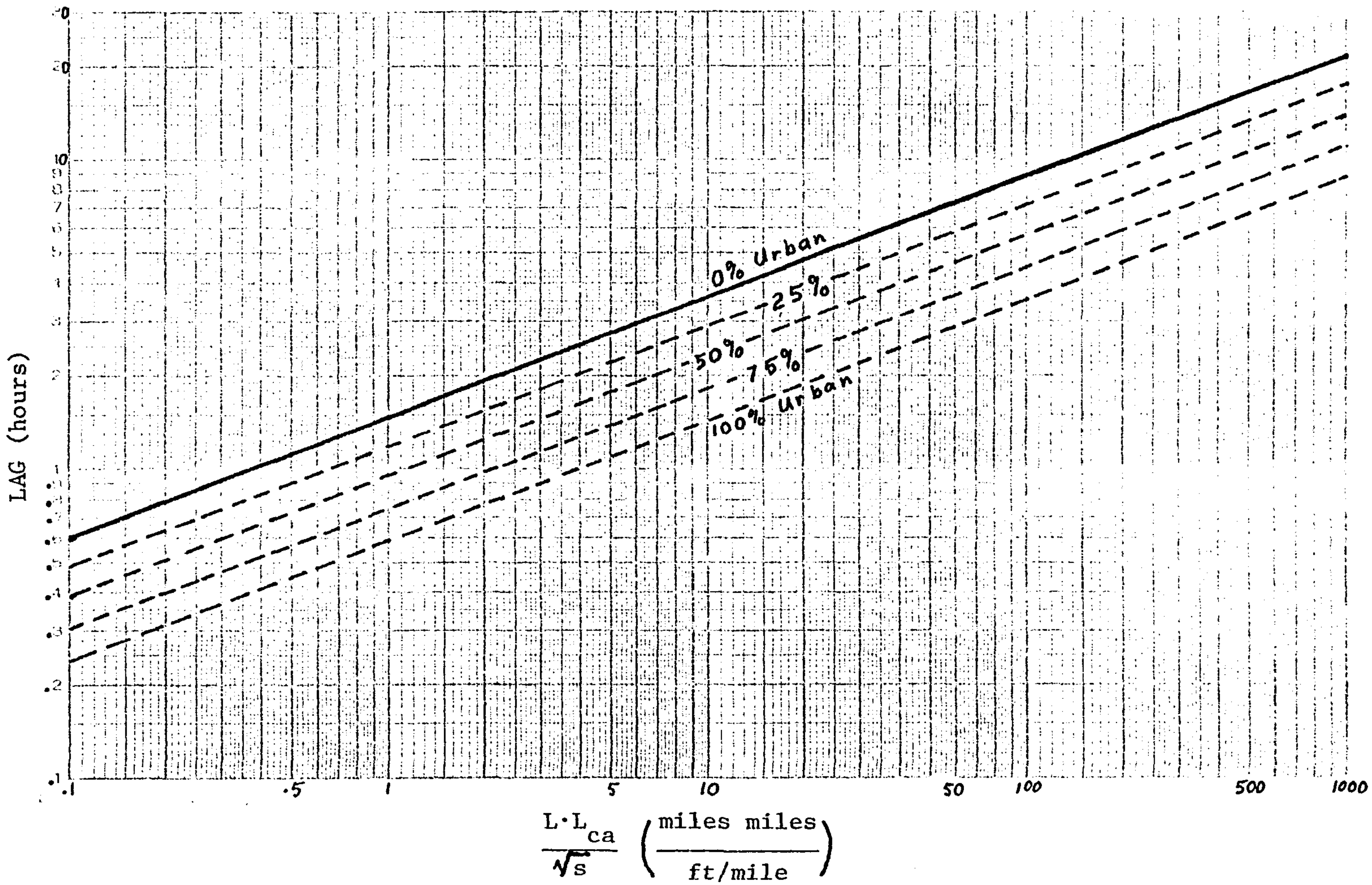


Figure 27. Snyder's unit hydrograph LAG.

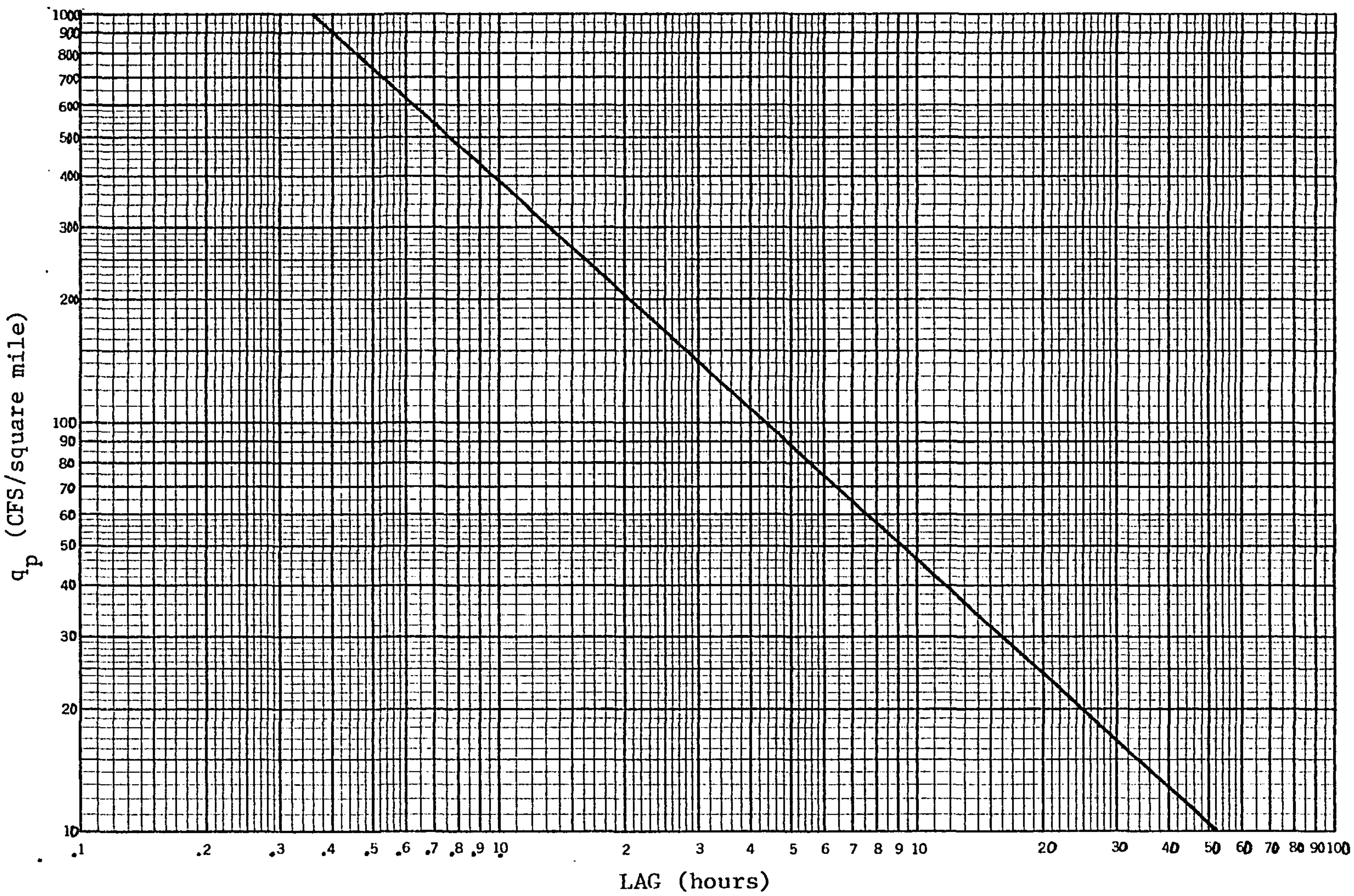


Figure 28. Snyder's unit hydrograph q_p.

largest model used in this study, and has the greatest capability. It can generate hydrographs and pollutographs for real storm events and systems from points of origin in real time sequence to points of disposal, including travel in receiving waters, with user options for intermediate storage and/or treatment facilities. Both combined and separated sewerage systems may be evaluated. Cost routines aid in estimating the economics of installation and maintenance. As is the case with several of the other models used in this study, only a small part of the capability of SWMM was calibrated on the test basins, and only the RUNOFF and TRANSPORT blocks, used in this study, are discussed on the following pages.

The RUNOFF block generates surface runoff based on rainfall hyetographs, antecedent soil moisture conditions and infiltration rates, land use, and drainage basin topography. The results are hydrographs and pollutographs at inlets to the main storm sewer system, computed as overland and gutter flow. A drainage basin is geometrically represented to the computer as overland and gutter flow. A drainage basin is geometrically represented to the computer as one or more subcatchments which contribute storm water runoff to their receiving drainage pipes, channels, or inlet manholes. Subcatchments must be represented as rectangular in surface shape with uniform ground slope, percentage of impervious cover, detention depth (representing rainfall which clings to the surface grass, leaves, and ground, and collects in shallow surface depressions), and roughness factor resisting the overland (surface sheet) flow of water, such as Manning's coefficient. This means averaging the values for natural irregular, nonhomogeneous watersheds. Rectangularity is achieved by dividing the subcatchment's area by the total width of overland flow contributing to the main drainage conduit. For natural subcatchments with overland inflow to both sides of the main channel, the width is usually twice the length of the main channel, as illustrated in Figure 29.

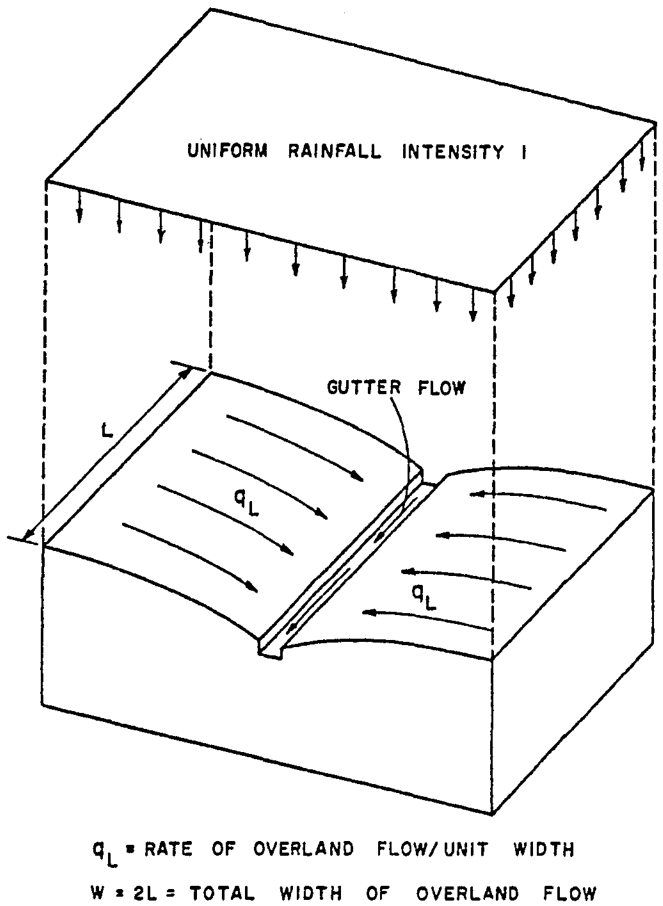


Figure 29. Idealized subcatchment-gutter arrangement. From (49).

In the following discussion, capital letters in parentheses after the description of a variable identify the variable's name as it is used in the computer program input, and is done only for program data input parameters, so that model users may quickly relate to the discussion. Once the rainfall hyetograph and data describing the subcatchments have been entered into the computer program, computations proceed as follows:

1. Rainfall is added according to the hyetograph,

$$C_t = D_t + R_t \cdot \Delta t$$

where

C_t = surface water depth at time t, after adding rainfall.

D_t = water depth before adding rainfall.

R_t = hyetograph rainfall intensity, inches per hour, at time t.

Δt = rainfall input interval.

2. Infiltration is computed by Horton's function,

$$I_t = f + (f_i - f_o)e^{-\alpha t}$$

where

I_t = infiltration, in inches per hour, at time t since the beginning of the rainfall.

f_o = minimum infiltration rate (WLMIN), in inches per hour.

f_i = maximum infiltration rate (WLMAX), in inches per hour.

α = decay rate of infiltration (DECAY), 1/second.

Infiltration is subtracted from water depth to find the depth of water on the pervious part of the subcatchment surface,

$$d_t = C_t - I_t$$

where

d_t = depth of surface water at time t, after sutracting infiltration.

If the difference is negative, because there is more infiltration than available rainfall, the result is set to zero, so there will never be negative water on the surface. No such subtraction is made for the impervious portion of the subcatchment surface, so all its rainfall contributes to the following calculations.

3. A thin layer of water is subtracted from the surface water depth, to account for wetting the surface, clinging to grass and trees, and filling shallow depressions, and an overland flow rate is computed by Manning's formula,

$$V_t = \frac{1.49}{n} (d_t - D_d)^{2/3} S^{1/2}$$

and

$$Q_t = V_t W (d_t - D_d)$$

where

V_t = velocity at time t

n = Manning's surface roughness coefficient, entered into the computer as parameter W5 for impervious surface, and W6 for pervious surface.

s = average subcatchment ground slope (WSLOPE), ft/ft.

D_d = surface detention (WSTORE), inches.

Q_t = outflow rate, cubic feet per second, at time t.

W = subcatchment width, (WWIDTH), meaning width of the overland flow front, usually twice the length of the main watercourse for natural subcatchments.

Again as in step 2 negative differences are set to zero.

4. The continuity equation is solved to determine the water depth on the subcatchment, resulting from rainfall, infiltration, and outflow,

$$D_{t+\Delta t} = d_t - (Q_t/A)\Delta t$$

where

A = subcatchment area (WAREA).

Steps 1 through 4 are repeated for each time step for each subcatchment to determine the surface sheet flow running off to the channels, drain inlets, gutters and conduits conducting the water out of the basin. The program does the proper conversions, such as inches to feet, to maintain proper volume and area units.

The RUNOFF block is also capable of simulating runoff water quality and gutter flow, but they are not discussed here because they were not used in the study.

The TRANSPORT block receives the surface sheet and gutter flow from the RUNOFF block and routes it through the basin's channels and conduits. The computational procedure basically follows a kinematic wave approach in which disturbances are allowed to propagate downstream as unsteady, non-uniform free-surface flow. By ignoring abrupt hydraulic changes such as hydraulic jumps, shock waves, and bore waves, it is possible to represent velocity and flow area relationships in a sewer system by the St. Venant equations, one the momentum equation,

$$\frac{\partial y}{\partial x} + \frac{v \partial v}{g \partial x} + \frac{\partial v}{g \partial t} = S_o - S_f \tag{30}$$

and the other the continuity equation,

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{31}$$

where

y = Depth.

v = Velocity.

x = Longitudinal distance.

t = Time.

g = Gravitational acceleration.

S_o = Invert slope.

S_f = Friction slope.

Q = Flow rate.

A = Flow area.

In SWMM, Equations (30) and (31) are solved by finite difference schemes. Referring to Figure 30, let the subscript j denote the upstream conditions of flow, Q , and area, A, and subscript j + 1 denote the downstream conditions. The subscript n denotes conditions at the previous time step, and the subscript n + 1 denotes conditions at the new time step.

The computational process is facilitated by normalizing flow area for each time step to a dimensionless ratio,

$$\alpha_{j,n} = \frac{A_{j,n}}{A_f}$$

where A_f is conduit area when flowing full, and normalizing discharge to

$$\psi_{j,n} = \frac{Q_{j,n}}{Q_f}$$

where again Q_f denotes conduit discharge when full flowing under gravity and friction influence alone. Then Equation (31) is written as a finite difference, terms collected as discussed in (50), and finally expressed as

$$\psi_{j+1,n+1} + C_1\alpha_{j+1,n+1} + C_2 = 0 \tag{32}$$

where

$$C_1 = \frac{\Delta x A_f}{\Delta t Q_f}$$

and

$$C_2 = \frac{\Delta x A_f}{\Delta t Q_f}\left[0.82\ (\alpha_{j,n+1} - \alpha_{j,n}) - \alpha_{j+1,n}\right] +$$

$$.82\ (\psi_{j+1,n} - \psi_{j,n}) - \psi_{j,n+1}$$

50. Ibid, pp. 121-127.

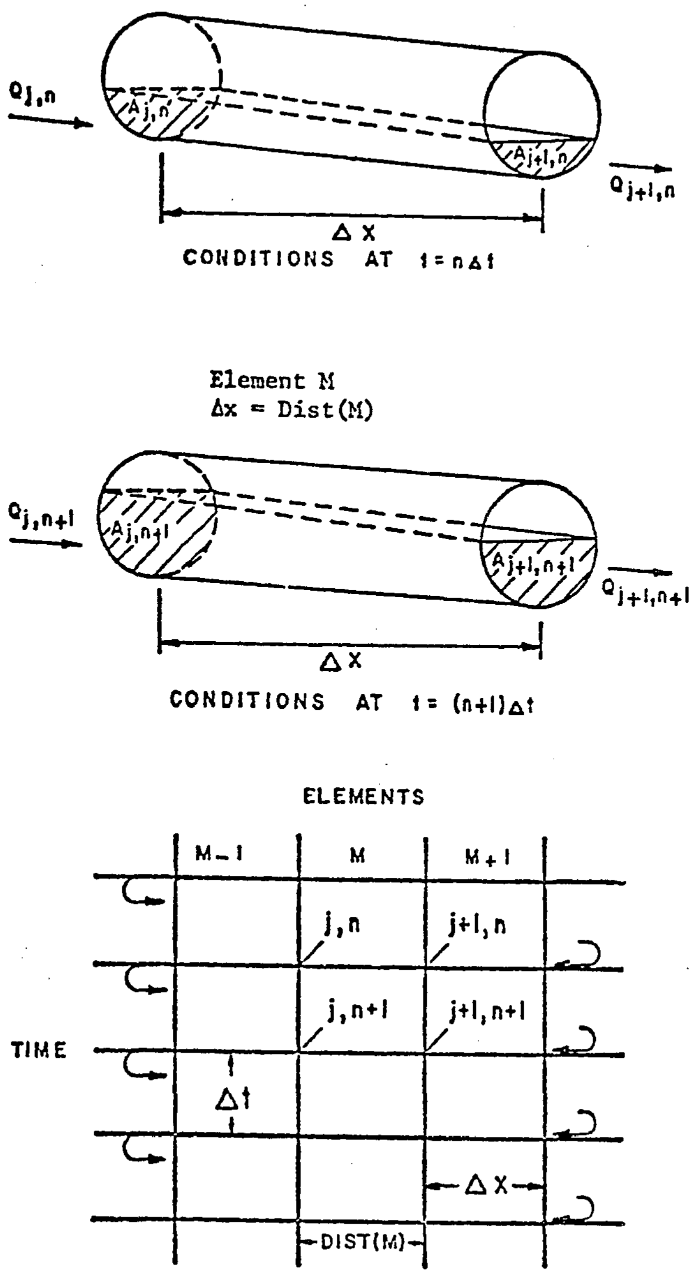


Figure 30. Finite difference definition for element M, routing through all elements at each time-step. From (40).

with 0.82 being a ratio to insure numerical stability and peak attenuation. In Equation (32) the only unknowns are $\alpha_{j+1,n+1}$ and $\psi_{j+1,n+1}$, the rest having been solved in the previous time or distance step, but it is still one equation in two unknowns, and a second equation is needed. That is provided by Equation (30). By neglecting the third term on the left side, solving for the friction slope S_f, inserting the solution into the slope variable of Manning's equation, and evaluating the partial derivatives in finite difference form, one obtains

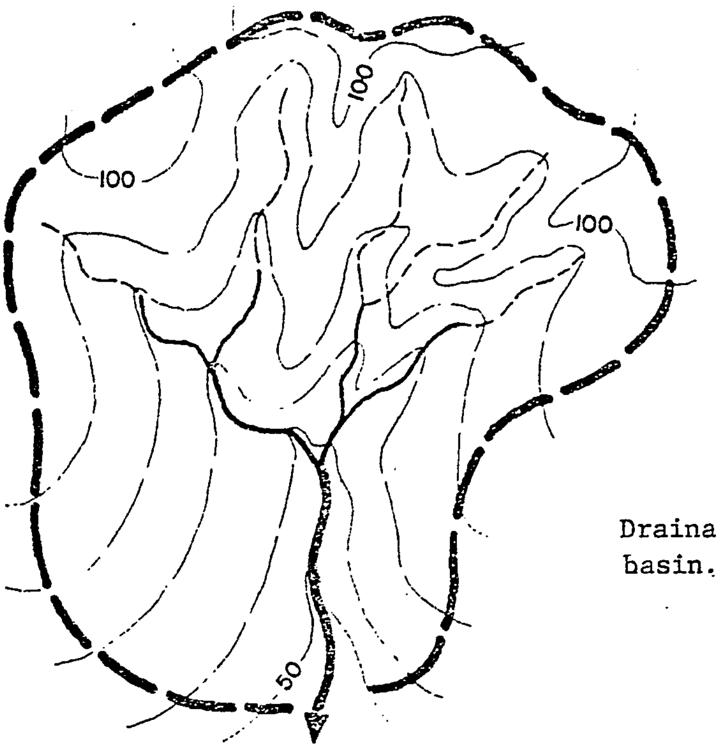
$$Q_f = \frac{1.49}{n} A_f R_f^{2/3} \left(S_o + \frac{y_{j,n} - y_{j+1,n}}{\Delta x} + \frac{v^2_{j,n} - v^2_{j+1,n}}{2g\Delta x}\right) \quad (33)$$

for full conduit flow, where n in the part $\frac{1.49}{n}$ is Manning's friction factor, not the time step. In order to remove undesirable numerical oscellations in conduits with low slopes, Equation (33) is solved as many as four times at each step, starting first with the previous step's values of velocity and depth, with each successive iteration averaging the previous ones. At each time step, the part-full flow is solved by the uniform flow equation, Manning's formula, with the invert slope S_o as the energy slope, and substituting into Equation (32) along with Equation (33). Then Equation (32) is solved by Newton-Raphson techniques. The process of Equations (30) through (33) is repeated for each conduit.

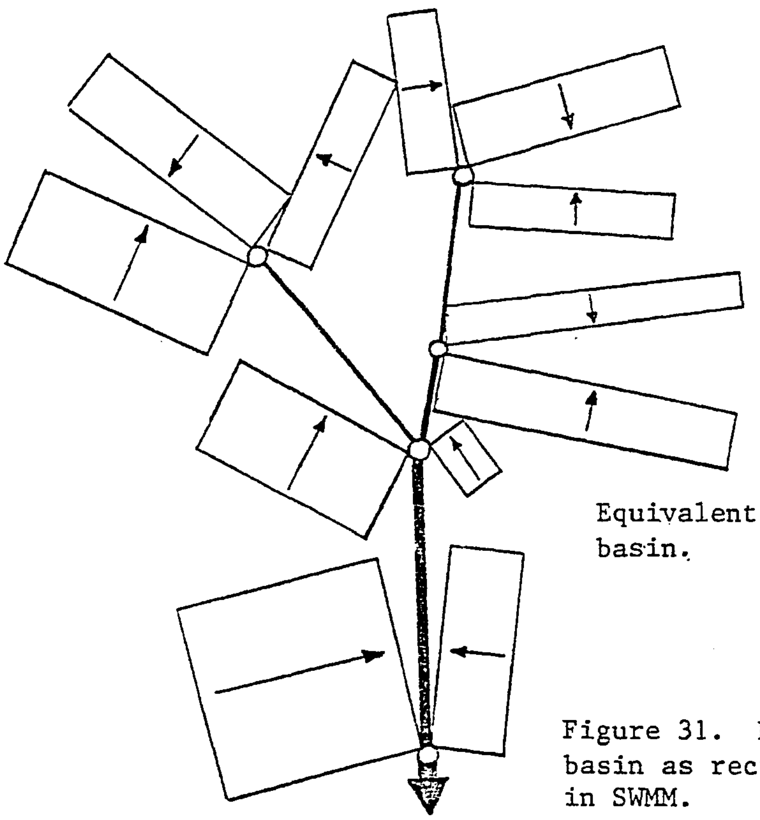
The TRANSPORT block is also capable of routing pollutant loads and concentrations, and to a limited extent, backwater effects, storage ponds, and hydraulic flow diverters. None of those were used in this study, and are not described here. Ponds in the study basins are flow-through ponds with uncontrolled outlets, and were modeled as wide, long conduits with low flow-line slopes.

In using SWMM to compute hydrographs, the physical drainage system of a basin must be represented by mathematical abstractions, subcatchments for overland sheet flow, gutters, sewers, open channels, storage ponds, pumps, etc. The first step in modeling a basin is to delineate the boundaries of the subcatchments. At one extreme a basin may be modeled as many subcatchments, each small in size, say the individual lots in a city subdivision. Such a modeling of basins such as the sizes of those in this study, one to fifteen square miles, would indeed be very tedious. At the other extreme, the entire basin may be represented as only one subcatchment. Most work is done with SWMM using at least four or five subcatchments to model a basin. If most of the travel time used by runoff to reach the outfall is spent as overland sheet flow, then it is best to model the basin as few subcatchments. On the other hand, if the water concentrates quickly into gutters, gullies, riverlets, drains, and channels, and there it spends most of its travel time, then it is best to model the basin as many small subcatchments of overland flow in the RUNOFF block, and give careful attention to modeling the channels and conduits in the TRANSPORT block. Such a basin can be modeled as a few subcatchments, but the physical width of the subcatchments must be adjusted in the model so as to give proper representation of the distance the water travels in sheet flow, and to get a proper ratio between overland travel time and channel travel time. That is where calibration becomes important to match the time of the flood peaks and shape of the hydrographs. Figure 31 displays a hypothetical basin and its mathematical abstraction.

The literature has many reports of previous trial and error



Drainage network of a typical basin.



Equivalent "block" diagram of the basin.

Figure 31. Representation of natural basin as rectangular subcatchments in SWMM.

calibrations of SWMM (51 through 58). In calibrating for runoff volume, previous research has reported the most important parameters to which the model is sensitive are the subcatchments' percentage of impervious cover, and the minimum infiltration rate of the pervious area in Horton's formula (51, 52, 54, and 57). Percentage of impervious cover may be considered a variable because some of the rain falling on house and building roofs may merely run off the roofs onto pervious ground next to the roofs, in which case the roofs may actually act as pervious area.

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51. "Storm Water Management Model User's Manual Version II," U. S. Environmental Protection Agency, EPA-670/2-75-017, 1975, pp. 101-102.

52. J. D. Sharon, "CSO Facilities Planning in Cincinnati Using SWMM (A Case Study)," in "Proceedings Stormwater Management Model (SWMM) Users Group Meeting, November 13-14, 1978.

53. Lorant, F. I., and C. Doherty, "Verification and Calibration of the Illinois Urban Area Drainage Simulator (ILLUDAS), in "Proceedings Stormwater Management Model (SWMM) Users Group Meeting May 4-5, 1978" U. S. Environmental Protection Agency, 1978.

54. Tang, Charles, Gary Kemp and Jeff Yarne, "Application of SWMM in an Urban Drainage Study," in "Meeting 3-4 November 1977." U. S. Environmental Protection Agency, no date.

55. James F. Mac Laren Ltd. "Review of Canadian Design Practice and Comparison of Urban Hydrologic Models," Canada Centre for Inland Waters, 1973, pp. 65-69.

56. Diniz, E. V., "Modifications to the Storm Water Management Model and Application to Natural Drainage Systems" in <u>Urban Storm Drainage</u>. John Wiley & Sons, 1978.

57. Jewell, Thomas K., Thomas J. Nunno, and Donald Dean Adrign, "Methodology for Calibrating Stormwater Models," <u>Journal of the Environmental Engineering Division</u>, American Society of Civil Engineers, June 1978, pp. 485-501.

58. Jess Abbott, "Testing of Several Runoff Models on an Urban Watershed," Davis, California: Corps of Engineers, the Hydrologic Engineering Center, 1978.

Other parameters do not have nearly so much impact on runoff volume as percentage of impervious cover and minimum infiltration rate. The parameter in the RUNOFF block having the most impact on hydrograph timing and shape is the subcatchment width, especially if the basin is modeled by a few large subcatchments, as discussed in the previous paragraph. Of lesser influence are surface slope and Manning's roughness for the surface. In the TRANSPORT block, conduit roughness may be considered a parameter for calibration for natural channels, assuming their shape, length, and slope are known from maps or measurements.

National Weather Service, River Forecast Center Deterministic Urban Runoff Model (MINICAT)

In 1970 John C. Schaake, Jr., first published a description of his urban runoff model based on a kinematic wave approximation of the St. Venant equations (59). The computer program has been expanded and improved by a group at the Massachusetts Institute of Technology, and has come to be known as the MIT Catchment Model or MITCAT, and is now a proprietary computer program (60). However, the older version is now in the public domain, and modifications are being used on an experimental basis by the U. S. Geological Survey (61) and the National Weather Service, River Forecast Center, which is the one used in this study.

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59. Schaake, John C., Jr. "Deterministic Urban Runoff Model," Institute on Urban Water Systems. Colorado State University, 1970.

60. "MITCAT Catchment Simulation Model, Description and Users Manual, Version 6" Cambridge: Resource Analysis, Inc., 1975.

61. Dawdy, David R., John C. Schaake, Jr., and William M. Alley, Users Guide for Distributed Routing Rainfall-Runoff Model" U. S. Geological Survey, Water-Resources Investigations 78-90, 1978.

References 62 and 63 present brief histories of the kinematic wave theory, but it is sufficient to say there that it is rather new compared to the rational method or the unit hydrograph theory, and has been in much use at all only since the late 1960's, barely ten years.

The theory of the kinematic wave is to replace the momentum equation of the St. Venant equations, Equation (3), by an approximation,

$$Q = \alpha A^{m} \tag{34}$$

where Q and A are discharge and water cross sectional area, as previously defined and α and m are paramenters dependent on whether the computation is being done for rectangular or pipe conduits, or open channels, or overland sheet flow. Also Equation (31) is modifed to

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{35}$$

for computing open channels with lateral inflow from their sides, where q is the lateral inflow rate of overland flow, per unit lenght of channel. Equation (34) is differentuated and substracted into Equation (35) to yield

$$\frac{\partial A}{\partial t} + \alpha\, m\, A^{m-1} \frac{\partial A}{\partial X} = q \tag{36}$$

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62 . Rovey, Edward W., David A. Woolhiser, and Roger E. Smith, "A Distributed Kinematic Model of Upland Watersheds," Hydrology Paper No. 93, Colorado State University, 1977.

63 . Eagleson, Dynamic Hydrology.

which has only one dependent variable. Equations (36) and (35) are solved in the computer program by rewriting them as finite difference equations, finding initial boundary conditions for each time and length step, setting up a Lax-Wendroff scheme based on time and distance downstream, much as illustrated in Figure 30, and obtaining convergence by the Newton-Raphson technique. References 64, 65, and 66 give a more detailed description of the mathematics.

The program user sees MITCAT as similar to SWMM in many respects in modeling basins--subcatchments of overland sheet flow are represented as rectangular blocks, and an open channel is represented by one typical cross section. However,the computational schemes are somewhat different, and MITCAT permits lateral inflow to the sides of an open stream, as portrayed in Figure 32, whereas SWMM does not, and MITCAT permits an overland flow subcatchment to contribute water to another overland flow subcatchment, again illustrated in Figure 32, whereas SWMM again does not. The input data requirements for the model are as follows:

1) Rainfall Hyetographs, which may change with several of the subcatchments.
2) Percentage of imperviousness and infiltration parameters for each subcatchment, which may be SCS curve number as previously described under the section on TR-20, or Horton's equation, as

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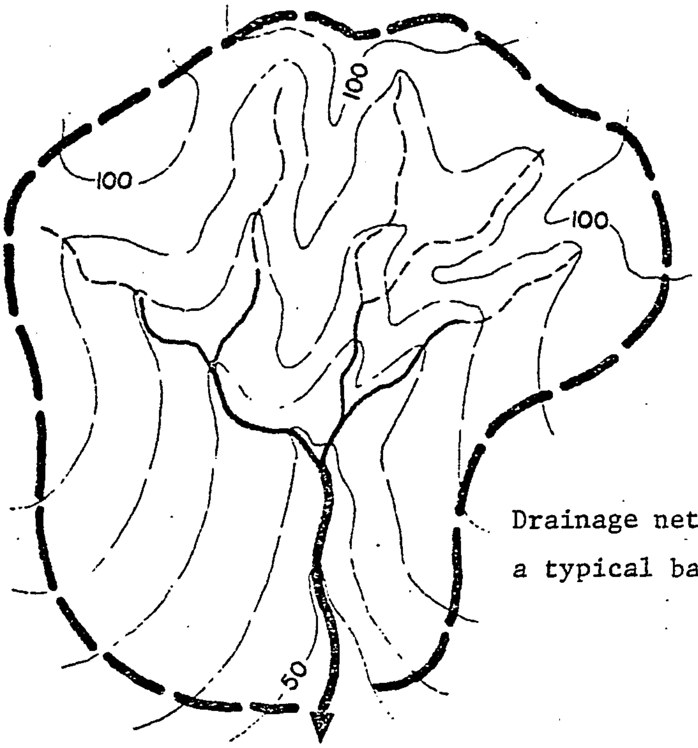
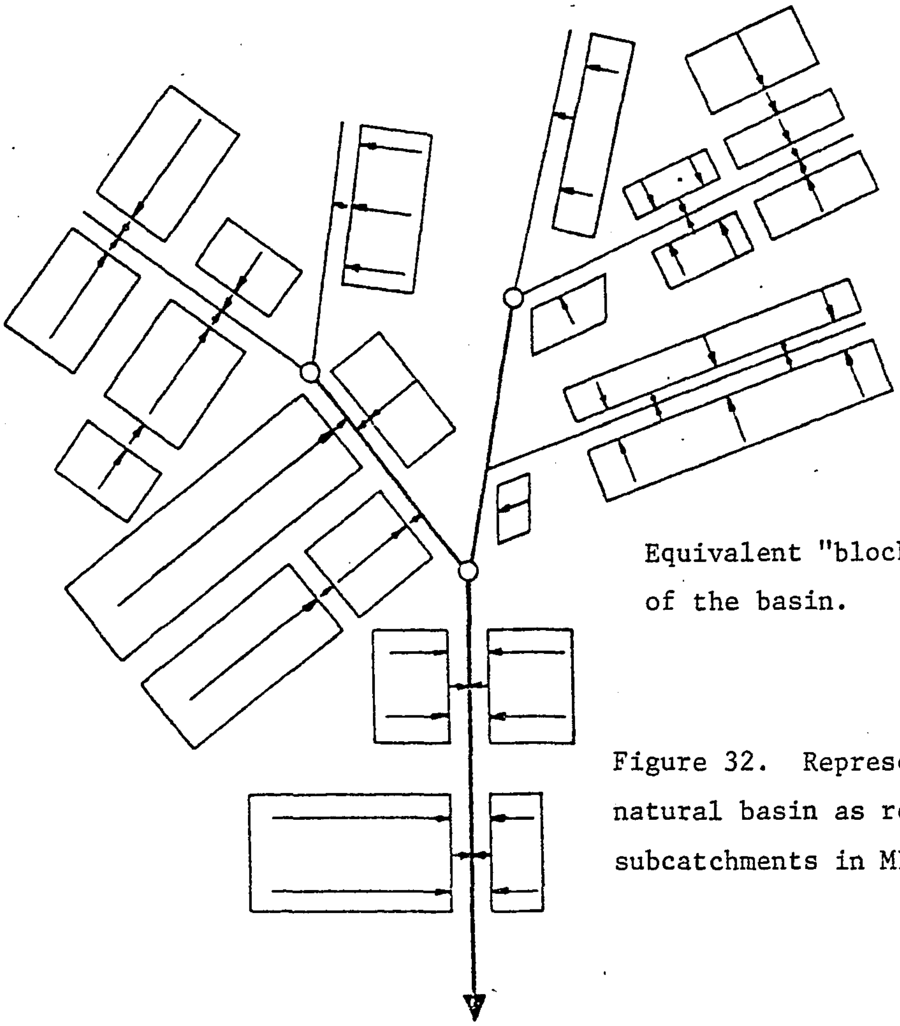
64 . Wilson, Charles, and Lqnacio Rodriquez - Iturbe. "Joint Usage of Rainfall-Runoff Models and Rainfall Generation Models" Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Massachusetts Institute of Technology, 1975.

65 . Schaake, "Urban Runoff Model."

66 . Eagleson, Dynamic Hydrology.



Drainage network of
a typical basin.



Equivalent "block" diagram
of the basin.

Figure 32. Representation of natural basin as rectangular subcatchments in MINICAT.

described under the section of this chapter on SWMM.

3) Average slope, Manning's roughness, and length of typical overland flow (which is important in the calibration process) for each subcatchment.

4) Length, slope, Manning's roughness, and typical cross sections of streams.

5) Specification of what subcatchments/streams flow into which subcatchments/streams.

6. Time step and distance steps, which affect stability of the computational process.

Calibration of this model is also by trial and error following the same process of adjusting the same variables as described in the last paragraph of the section on SWMM in this chapter.

Table 5 summarizes the mathematical foundations of the models discussed in this chapter.

TABLE

MATHEMATICAL FOUNDATIONS OF THE MODELS

		Rational Method	TR-20 Soil Conservation Service	HEC-1 Corps of Engineers	C824 Geological Survey	SWMM Environmental Protection Agency	MINICAT River Forecast Center
Infiltration	SCS Curve Number $CN = \frac{1000}{2 + 10}$		X				X
	Beard's Equation $L_t = (k_t + D_t)\ P^E$			X			
	Horton's Equation $I_t = f + (f_i - f_o)e^{-\alpha t}$					X	X
	Phillips Equation $F = K\ \{1 = [rP_x - P_x(r-1)\frac{M_o}{M_c}\ /i]\}$				X		
Runoff Calculations	Rational Formula $Q = CIA$	X					
	SCS Dimensionless Unit Hydrograph $K = 645.33\ \frac{2}{1 + Tr/Tp}$		X				
	Clark's Unit Hydrograph $I(t) = \sum_{j=1}^{n} T_j\ R_{e(t-j+1)}$ $Q(t+\Delta t) = I(t) - [I(t) - Q(t)]\ e^{-\Delta t/k_s}$				X		
	Clark's Unit Hydrograph $I_i = 645\ a_i/\Delta t$ $O_i = CI_i + (1-C)\ O_{i-1}$			X			
	Snyder's Unit Hydrograph $Q_p = \frac{640 \cdot CP \cdot A}{LAC}$			X			
	Shallow Flow Equation $V_t = \frac{1.49}{n}\ (d_t - D_d)^{2/3}\ S^{1/2}$ $Q_t = V_t\ W\ (d_t - D_d)$					X	
	Kinematic Wave $\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q$ $Q = \alpha A^m$						X
Stream Routing	Kinematic Wave $\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q$ $\frac{\partial y}{\partial x} + \frac{v \partial v}{g \partial x} + \frac{\partial v}{g \partial t} = S_o - S_f \qquad (Q = \kappa A^m)$					X	X
	Other Methods (Muskingum, working R and D, convex, reservoir) available but not used in this study.		X	X		X	X

CHAPTER IV

THE CALIBRATION PROCESS AND RESULTS

Each of the six models used in this study has its own set of parameters to use in calibrating to observed rainfall-runoff records, leading to six distinct calibration processes, and each will be discussed separately in this chapter. At this point it may be interesting to give the number of parameters used in calibrating each of the five computer models used in this study, and that is done in Table 6. More variables may have been used in some of the models in the early stages of calibration, then chopped after it was found they are of little influence to the outflow hydrographs. For example, subcatchment slope

TABLE 6

NUMBER OF PARAMETERS USED AND METHOD OF CALIBRATION OF THE COMPUTER MODELS.

	TR-20	G824	HEC-1	SWMM	MINICAT
No. of parameters used in calibrating volume	1	7	4	2	2
No. of parameters used in calibrating hydrograph shape and time.	2	2	2	1	1
Automatic or trial-and-error calibration	T&E	Auto	Semi-Auto	T&E	T&E

and roughness were used in the early stages of calibrating SWMM, but they were held constant in the final calibrations because it was found even large changes in those variables had only minimal changes to the basin outflow hydrographs.

In Table 6 parameters calibrating volume are listed first because they are always calibrated first. Volume of runoff has a considerable impact on peak discharge. In the case of SWMM and MINICAT, volume was adjusted first, then some refinements to the infiltration rates affective volume had to be refined as hydrograph shapes and times were calibrated. Note in Table 6 that the model with the most parameters for optimizing (the Geological Survey's G824) fortunately has automatic calibrating on all parameters--except one, percentage of impervious cover.

The process of calibration used in this study was to calibrate each of the three basins on each of the six models individually, leading to eighteen calibration efforts. For each of these efforts, total volume of runoff per flood event was calibrated (except in the Rational method, which cannot compute volume) by rationing each event's synthetic total volume of runoff to the same events' observed runoff volume, and calibrating by adjusting the volume parameters until the mean of those ratios by events is well within one standard deviation of being unity.

That is, when a perfect model using perfect input date gives perfect prediction,

$$\frac{\text{Synthetic volume}}{\text{Observed volume}} = 1.$$

But these are imperfect models using imperfect input data, so the ratio rarely comes out exactly unity for any flood, so the object was to get the average as close to unity as possible, and of the five models, the one is supposedly best in predicting volume of runoff when its scatter about unity, meausred by the standard deviation, is smallest of the five. Time measures of good hydrograph shape were computed for this study--ratio of synthetic peak discharge to observed, and absolute difference between synthetic peak time and observed. For each of the six models, for each basin, for each storm event, the ratio,

$$\frac{\text{Synthetic peak discharge}}{\text{Observed peak discharge}}$$

was formed. Each model was calibrated by changing parameter values until the average of those ratios, grouped by basin, was near unity, meaning well within one standard deviation of being unity. That model is best at predicting peak discharges which then had the smallest standard deviation. For the five computer models, in calibrating for time of hydrograph peak, the difference, time of synthetic peak discharge minus time of observed peak was subtracted for each basin for each storm event. In calibrating, model parameter values were adjusted until the mean of those difference grouped by basin approached zero, meaning within five minutes of being zero. That model is best at predicting time of hydrograph which had the smallest standard deviation about its mean. In order to determine if any model is biased toward low or high volumes or discharges, the regression lines of synthetic versus observed were computed and the intercepts tested for statistically significant difference from zero. The statistical analysis for this study was done on the Statistical Analysis System (67).

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67. Barr, Anthony J. James H. Goodnight, John P. Sall, and Jane T. Helwig, "A User's Guide to SAS 76," Raleigh, North Carolina: SAS Institute Inc., 1976.

Rational Method

This simple hand calculated model,

$$Q = CIA \qquad (1)$$

was also the easiest to calibrate for this study, since one parameter, "C" in Equation (1), is the only unknown for each storm-runoff event. For each of the three study basins, an average "C" was computed as displayed in Table 7. As described in Chapter III, each storm's peak intensity was determined by finding each storm's peak amount of rainfall occurring during the basin's time of concentration, then dividing that rainfall by the time of concentration. Each basin's time of concentration had been determined previously by HEC-1 and G824.

An attempt was made to see if it would be possible to improve its performance by varying the parameter "C" with rainfall intensity. A plot was made of each basin of each storm event's peak intensity as used in Equation (1) and its "C" as found by solving Equation (1) for "C" for each storm. As shown in Figure 33, for Deep Fork Creek at Portland Avenue there was a very definite positive correlation between "C" and intensity, but there is no meaningful correlation of Bluff Creek, and indeed a strong negative correlation on Deep Fork Creek at Eastern Avenue, which is illogical. Therefore each basin's mean "C" was used in computing the statistics in Table 7 and the graphs in Figure 34.

The Rational Method displayed no statistically significant bias on any of the three study basins used in this study, in a regression analysis of its synthetic peak discharges against the observed peak discharges.

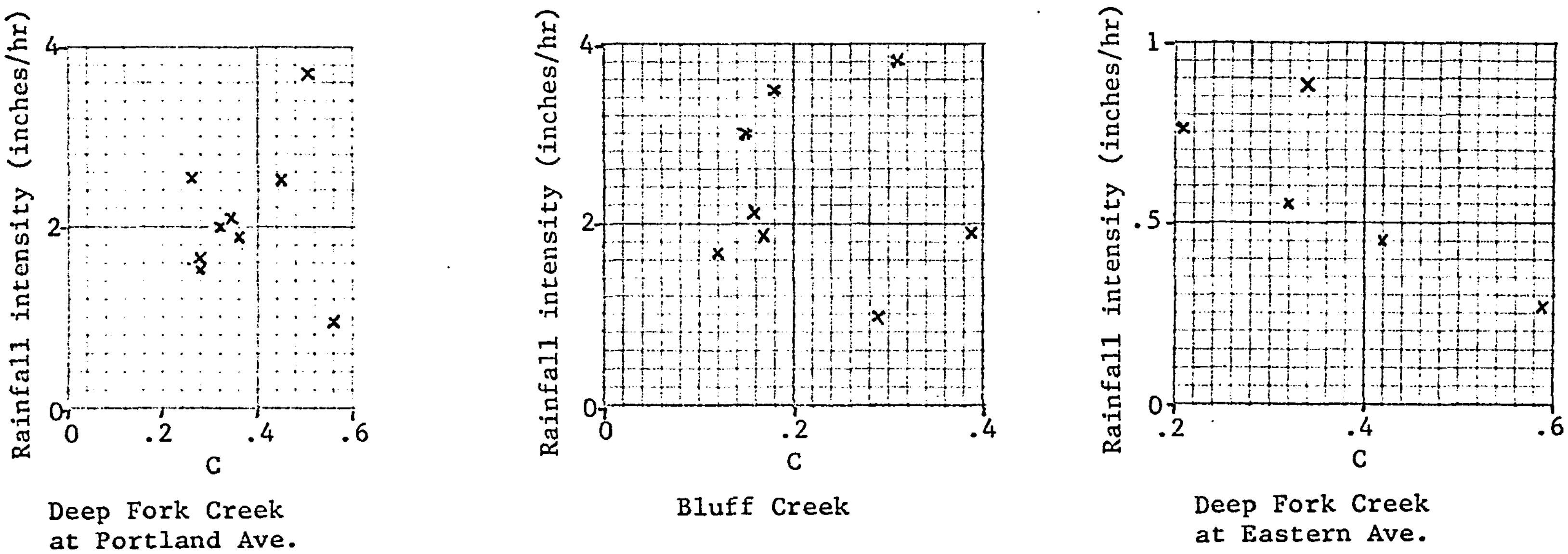


Figure 33. Rational Method runoff coefficients for the three study basins.

TABLE 7

COMPARISON BETWEEN OBSERVED AND

RATIONAL SYNTHETIC PEAK DISCHARGES

	Storm Date	Observed Peak Discharge, CFS	Rational Method Synthetic Peak Discharge, CFS
Deep Fork Creek At Portland Ave.	4-29-74	1384	1544
	5-23-74	1204	1428
	6-08-74(I)	2148	1812
	6-08-74(II)	1284	1848
	11-02-74	3600	2681
	5-13-75	1304	1377
	5-22-75	820	1123
	6-05-75	896	1210
	6-06-75	1000	674
	8-14-75	2340	1964
Bluff Creek	3-08-74	675	804
	3-10-74	336	430
	5-21-74	288	222
	5-23-74	460	693
	11-02-74	1250	878
	5-14-75	357	485
	6-16-75	214	393
	8-14-75	782	439
Deep Fork Creek At Eastern Ave.	5-02-75	2879	1852
	5-13-75	3198	3773
	5-22-75	3450	3086
	7-24-75	2883	5212
	8-14-75	5450	6104

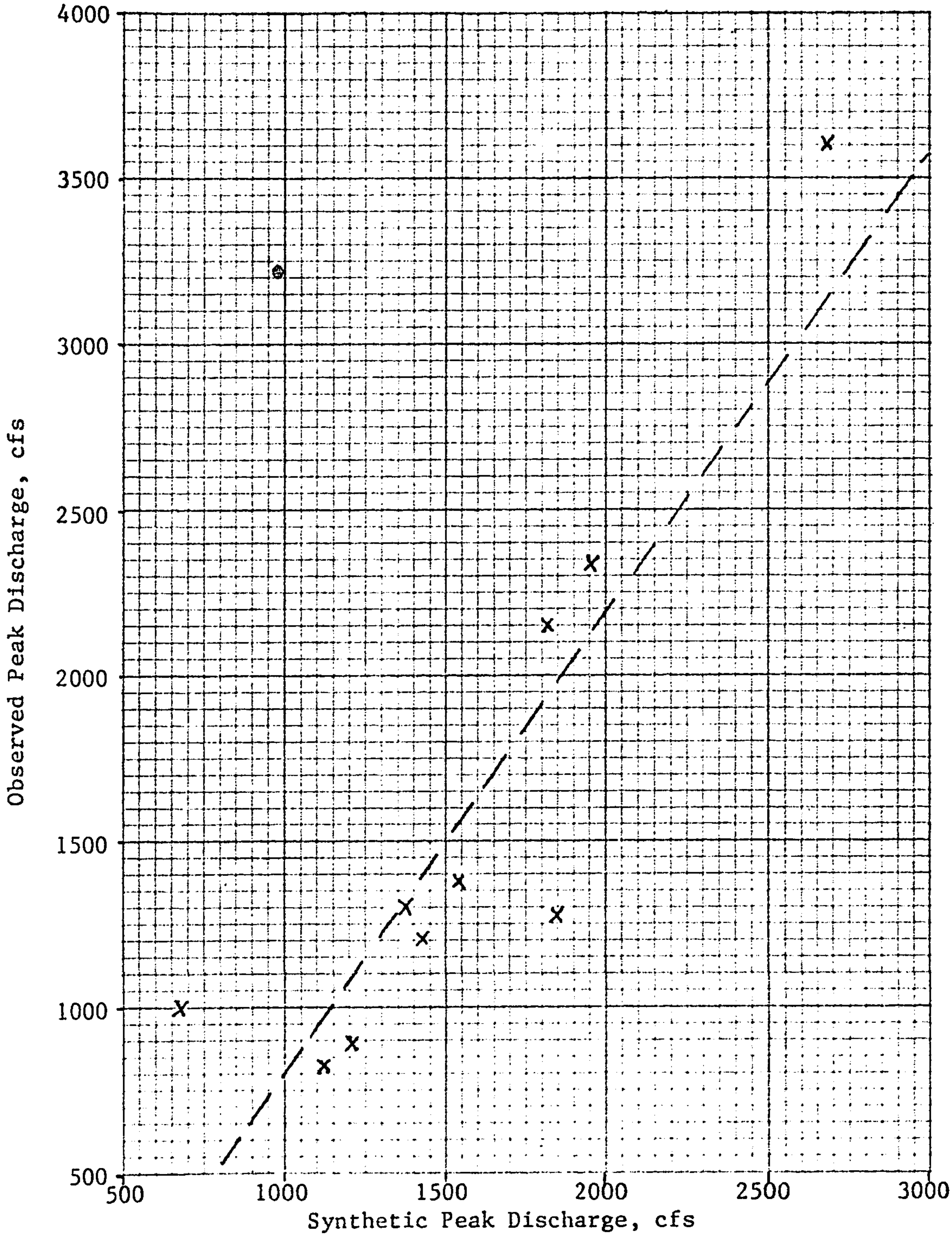
TABLE 7 - CONTINUED

STATISTICS

		Deep Fork Creek At Portland Ave. Peak Discharge	Bluff Creek Peak Discharge	Deep Fork Creek At Eastern Ave. Peak Discharge
Synthetic / Observed	Mean	1.06	1.15	1.13
	Standard Deviation	0.28	0.44	0.43
Regression Lines	Coefficient of Determination (r^2)	0.80	0.54	0.44
	Slope	1.41	1.12	0.42
	Std. Error of Est. Slope	0.25	0.42	0.28
	Intercept	-603.3	-65.0	1885
	Std. Error of Est. Intercept	409.6	246.9	1184
	Bias	No	No	No

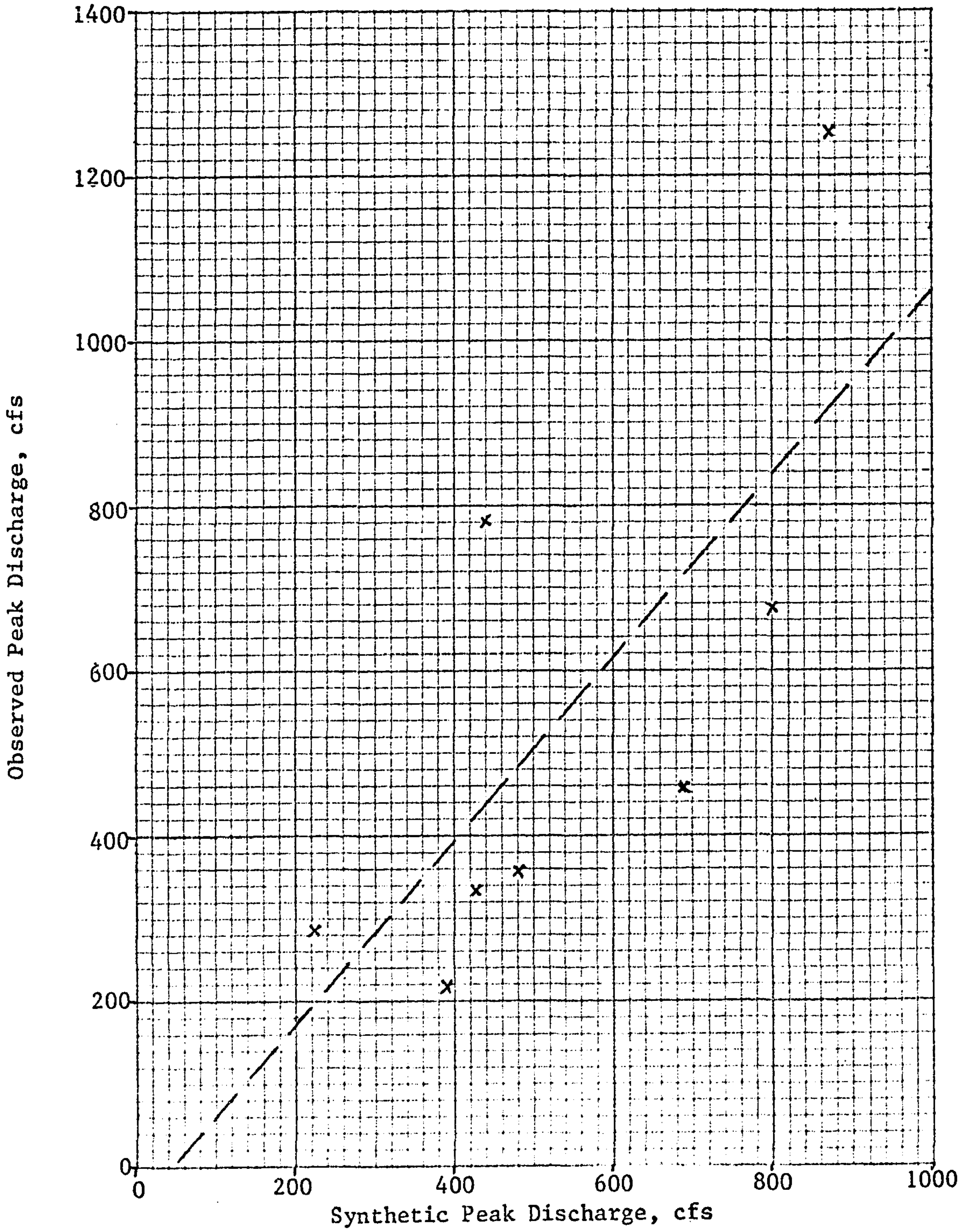
CALIBRATION RESULTS

	Deep Fork Creek At Portland Ave.	Bluff Creek	Deep Fork Creek At Eastern Ave.
Runoff Coefficient "C"	0.38	0.22	0.38



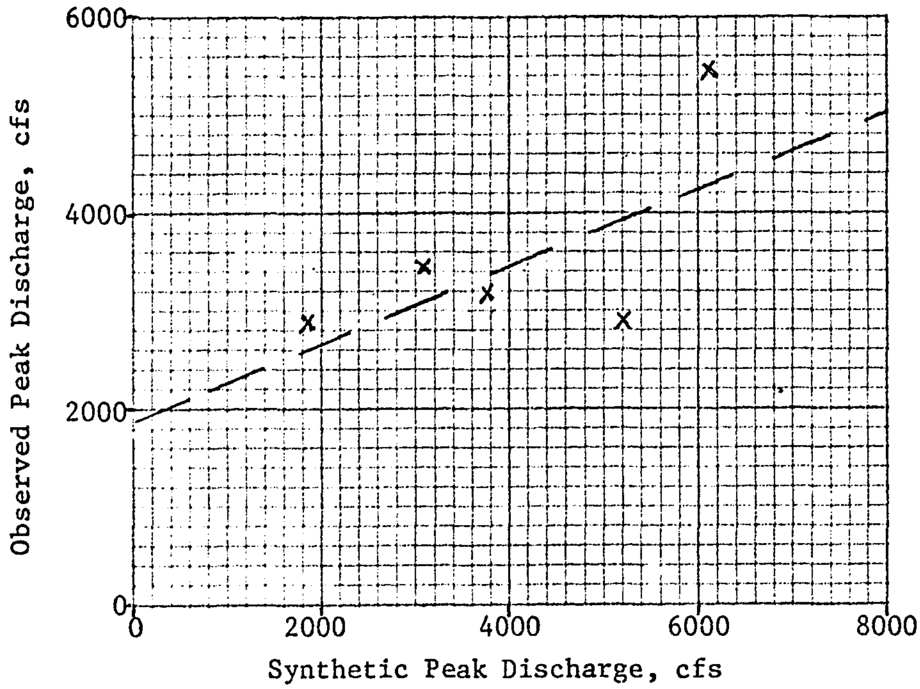
Deep Fork Creek at Portland Avenue

Figure 34. Comparison between observed and Rational Method synthetic peak discharges.



Bluff Creek

Figure 34--CONTINUED.



Deep Fork Creek at Eastern Avenue

Figure 34--CONTINUED.

U. S. Soil Conservation Service: TR-20

Of the computer models used in this study, this was the easiest to calibrate for volume. Using Equation (4) in Chapter III, each storm-runoff event's curve number, CN, was determined from the observed rainfall and observed runoff volume records, then an average CN was computed for each basin to use in the computer simulation. An attempt was made on Deep Fork Creek at Portland Avenue to change CN for each storm so as to account for the influence of antecedent soil moisture conditions. As part of their study, hydrologists of the U. S. Geological Survey had collected daily rainfall and lake evaporation data (68). That data was put into a computer program written for this study and styled after the technique developed by Williams and LaSeur (69) for the Agricultural Research Service, but the results proved no better than fixing the average CN for the basin as constant and not subject to the antecedent soil moisture conditions. Then a sign test (70) was run of antecedent rainfall against the observed CN by storm event, and it was found that variations in observed CN were completely independent of the antecedent rainfall.

Each of the three basins used in this study was modeled as one catchment on TR-20, so there was no channel or reservoir routing. The standard SCS dimensionless unit hydrograph was used on both basins on Deep Fork Creek, with time of concentration being found by trial and error

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68. Thomas, "Discharges "

69. Williams, Jimmy R., and William V. LaSeur, "Water Yield Model Using SCS Curve Numbers," Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 102, No. HY9, pp. 1241 - 1253.

70. Johnson, Robert R., Elementary Statistics, 2nd Ed. North Scituate, Mass: Duxbury Press, 1976. pp. 514 - 521.

computer runs, but the Bluff Creek basin proved impossible to model with the standard shaped unit hydrograph. Calibrating for time of peak gave a peak discharge much too high above the observed, and calibrating for peak discharge gave a time of peak much earlier than the observed. Then the unit hydrograph derived in the HEC-1 calibration, to described in the following section, was used with satisfactory results. That is a flatter hydrograph, owing probably to the attenuation due to the ponds on the West half of the basin (Figure 12), which influence almost half the water flowing through the basin. No such attenuation is apparent in the hydrograph of Deep Fork Creek at Eastern Avenue because Belle Isle Lake is downstream of such a small portion of that total basin.

No attempt was made to model the basins' impervious cover, other than the average CN for each basin which is the procedure recommended in (71). However, it could have been possible to model each basin as two catchments, one for the impervious area being modeled, and the other catchment representing the basin's pervious area, with a low CN, and area equal to the basin's actual area minus the impervious area, then adding the two catchments' flows to get the basin discharge.

Table 8 shows the results of the calibrations, Figure 35 has plots of observed versus synthetic values for each flood's peak discharge and volume of runoff water, and Figure 40 has each basin's unit hydrograph obtained by using TR-20, and Appendix B has plots of each storm's rainfall hyetograph and outfall flood hydrograph used in this study.

In a regression analysis of TR-20's synthetic volumes versus

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71. "Urban Hydrology--TR-55."

TABLE 8

COMPARISON BETWEEN OBSERVED AND TR-20 SYNTHETIC PEAK DISCHARGES AND RUNOFF VOLUMES

	Storm Date	Observed Peak Discharge, CFS	TR-20 Synthetic Peak Discharge, CFS	Observed Volume, Acre-Feet	TR-20 Synthetic Volume, Acre-Feet
Deep Fork Creek at Portland Ave.	4-29-74	1384	1705	191.65	212.52
	5-23-74	1204	916	124.96	64.84
	6- 8-74(I)	2148	2343	387.23	253.23
	6- 8-74(II)	1284	1618	125.58	128.27
	11- 2-74	3600	5076	617.70	566.74
	5-13-75	1304	1064	112.66	131.77
	5-22-75	820	744	152.89	246.64
	6- 5-75	896	671	61.68	54.50
	6- 6-75	1000	1001	89.89	121.91
	8-14-75	2340	3050	301.27	293.98
Bluff Creek	3- 8-74	675	749	71.53	71.50
	3-10-74	336	180	45.73	16.45
	5-21-74	288	195	23.28	19.66
	5-23-74	460	557	50.63	50.75
	11- 2-74	1250	1959	240.32	287.75
	5-14-75	357	355	35.45	36.24
	6-16-75	214	172	10.65	21.89
	8-14-75	782	834	100.55	94.86
Deep Fork Creek at Eastern Ave.	5- 2-75	2879	2270	1013.7	568.58
	5-13-75	3198	2328	1184.6	884.90
	5-22-75	3450	4280	2097.0	1890.47
	7-24-75	2883	6291	1729.3	2573.88
	8-14-75	5450	7532	1609.9	2195.11

TABLE 8 - CONTINUED

STATISTICS

		Deep Fork Creek At Portland Ave.		Bluff Creek		Deep Fork Creek At Eastern Ave.	
		Peak Flow	Volume	Peak Flow	Volume	Peak Flow	Volume
Synthetic / Observed	Mean	1.05	1.01	1.00	1.04	1.26	1.01
	Standard Deviation	0.24	0.32	0.33	0.50	0.59	0.40
Regression Lines	Coefficient of Determination (r^2)	0.97	0.88	0.96	0.97	0.46	0.63
	Slope	0.62	1.09	0.57	0.81	0.31	0.40
	Std. Error of Est. Slope	0.04	0.14	0.05	0.05	0.19	0.18
	Intercept	471.3	-7.2	191.8	12.2	2168	877
	Std. Error of Est. Intercept	82.9	35.7	38.2	6.1	975	318
	Bias	Yes	No	Yes	Yes	Yes	Yes

CALIBRATION RESULTS

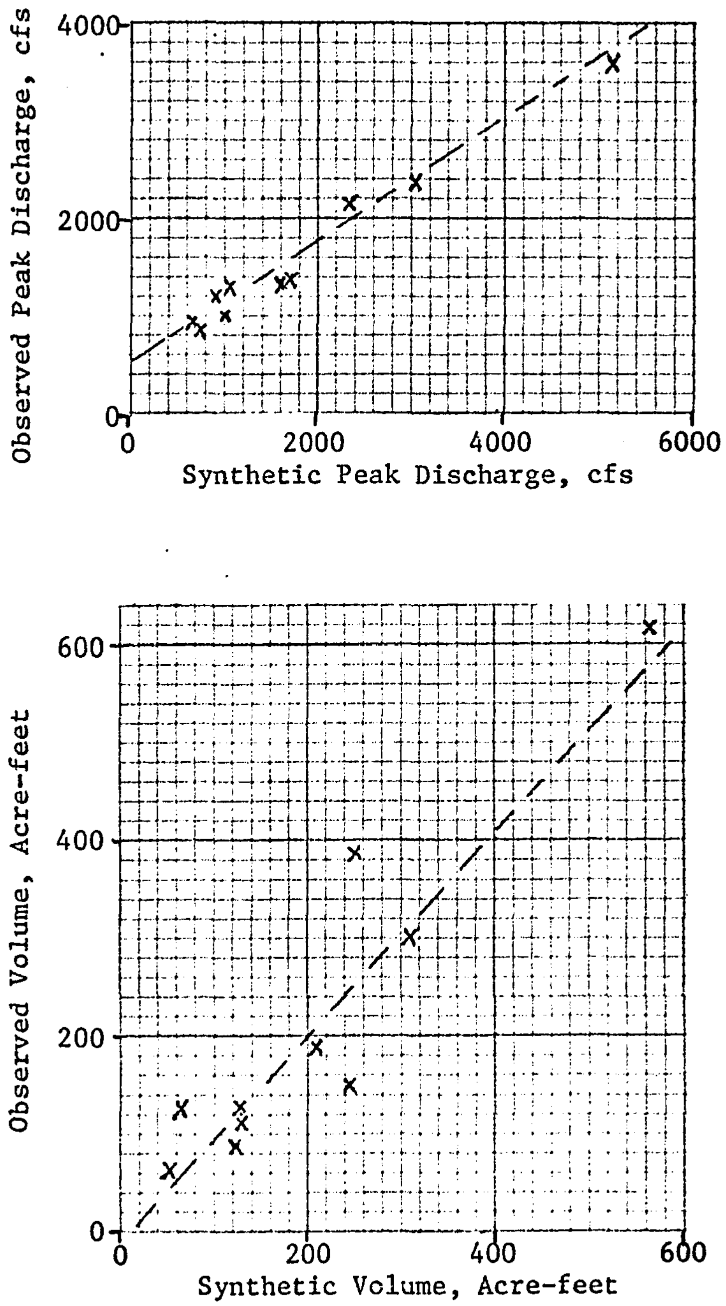
	Deep Fork Creek At Portland Ave.	Bluff Creek	Deep Fork Creek At Eastern Ave.
Time of Concentration Hr.	0.70	0.46	2.83
K	484	205	484
CN	88	86	85

the observed volumes, no bias was found for Deep Fork Creek at Portland Avenue or Bluff Creek, but bias was found on Deep Fork Creek at Eastern Avenue, with TR-20 tending to give too much volume on the higher volume floods, and too low volume on the lower volume floods. A regression for peak discharges found that TR-20 is biased on all three of the basins, tending to give too high a peak for the higher peak floods, and too low a peak on the lower peak floods, as may be seen in Figure 35.

U. S. Corps Of Engineers: HEC-1

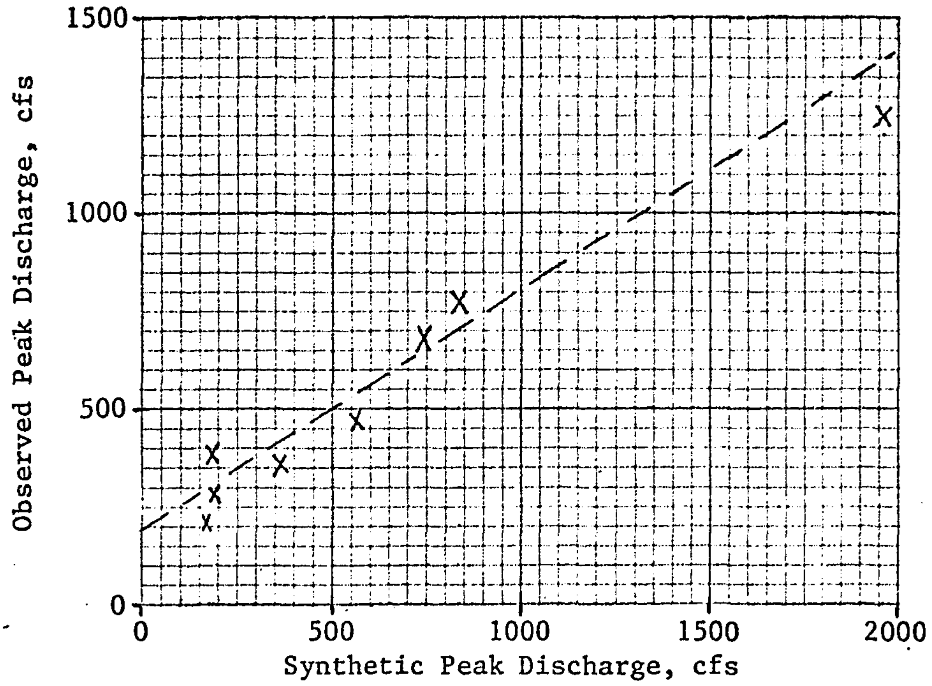
Again in calibrating this model, each basin was modeled as one catchment, primarily to make use of the computer program's automatic optimization routines, which can be used only by taking a basin as a whole without breaking it into subcatchments and connecting channels. The calibrations process has previously been described in Chapter III. As with TR-20, an attempt was made to find any influence of each rainfall-runoff event's antecedent soil moisture condition and previous rainfall. An attempt was made at multivariate regression, with the dependent variable being the HEC-1 parameter DLTKR, which is supposed to be influenced by antecedent soil moisture conditions, and the independent variables being the previous several days' rainfall amounts. No statistically significant regression could be achieved on Deep Fork Creek at Portland Avenue, and none was even attempted on the other two basins, so a fixed value of DLTKR was used for each basin's calibration.

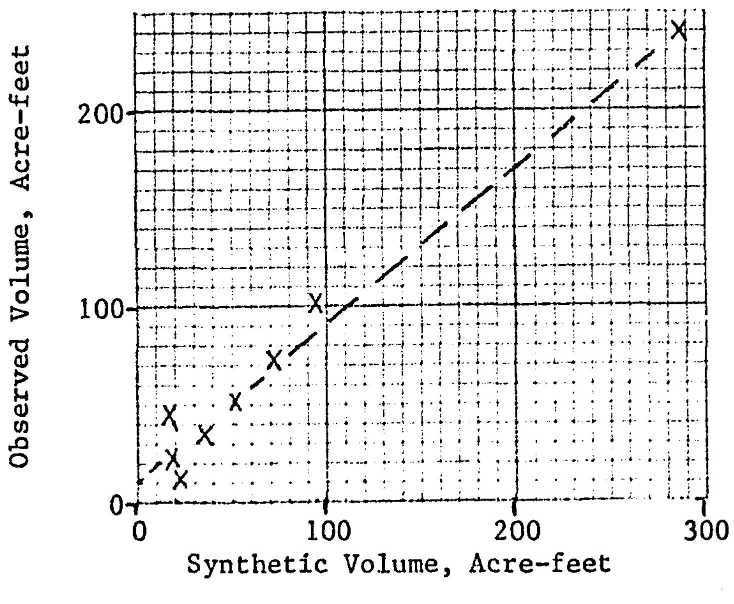
Table 9 has the results of the calibration, and Figure 40 shows each basin's unit hydrograph, and Figure 36 has plots of each event's observed versus synthetic peak discharge and runoff volume and Appendix B has each event's observed versus synthetic hydrograph.



Deep Fork Creek at Portland Avenue

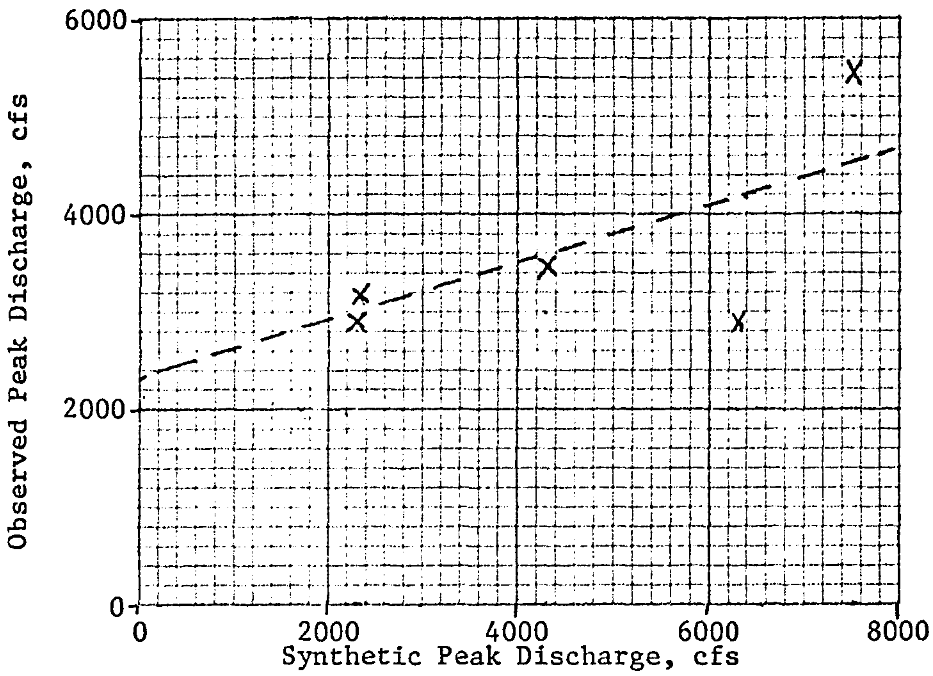
Figure 35. Comparison between observed and TR-20 synthetic peak discharges and runoff volumes.

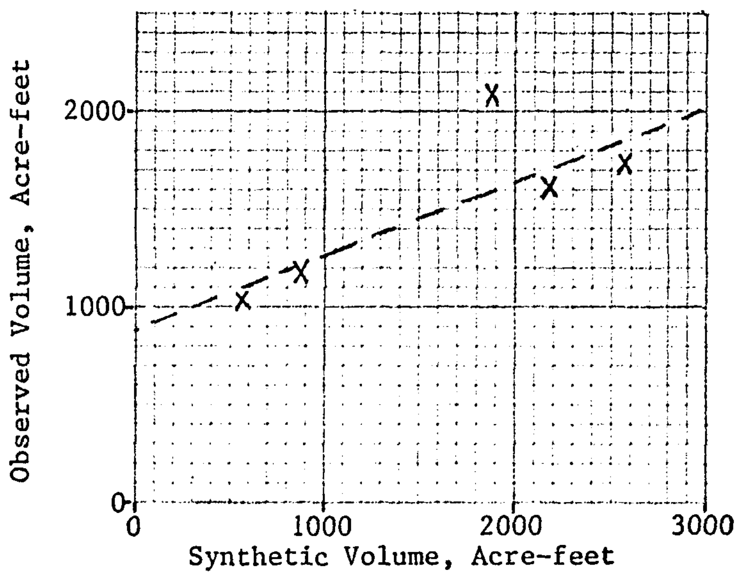




Bluff Creek

Figure 35--Continued.





Deep Fork Creek at Eastern Avenue

Figure 35--Continued.

TABLE 9

COMPARISON BETWEEN OBSERVED AND HEC-1 SYNTHETIC PEAK DISCHARGES AND RUNOFF VOLUMES

	Storm Date	Observed Peak Discharge, CFS	HEC-1 Synthetic Peak Discharge, CFS	Observed Volume, Acre-Feet	HEC-1 Synthetic Volume, Acre-Feet
Deep Fork Creek at Portland Ave.	4-29-74	1384	1528	191.65	210.41
	5-23-74	1204	1249	124.96	104.93
	6- 8-84(I)	2148	2116	387.23	256.89
	6- 8-74(II)	1284	1872	125.58	165.18
	11- 2-74	3600	3952	617.70	514.61
	5-13-75	1304	1237	112.66	123.88
	5-22-75	820	1208	152.89	184.28
	6- 5-75	896	972	61.68	81.35
	6- 6-75	1000	675	89.89	113.94
	8-14-75	2340	2526	301.27	289.75
Bluff Creek	3- 8-74	675	695	71.53	71.68
	3-10-74	336	379	45.73	41.45
	5-21-74	288	145	23.28	22.30
	5-23-74	460	463	50.63	50.58
	11- 2-74	1250	1510	240.32	229.67
	5-14-75	357	332	35.45	35.41
	6-16-75	214	97	10.65	9.82
	8-14-75	782	665	100.55	96.32
Deep Fork Creek At Eastern Ave.	5- 2-75	2879	1879	1013.7	794.0
	5-13-75	3198	3051	1184.6	1185.2
	5-22-75	3450	2708	2097.0	1631.6
	7-24-75	2883	3990	1729.3	1843.5
	8-14-75	5450	4172	1609.9	1606.9

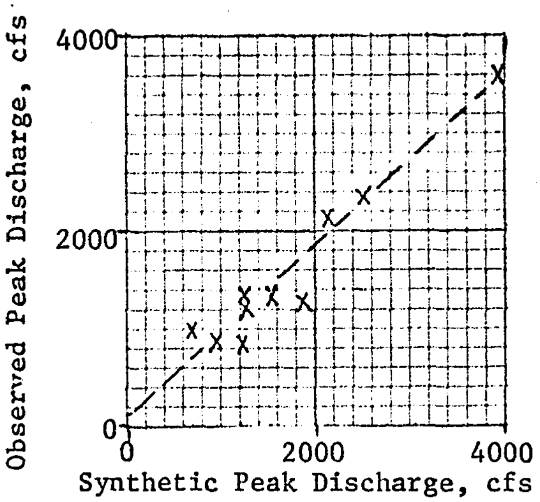
TABLE 9 - CONTINUED

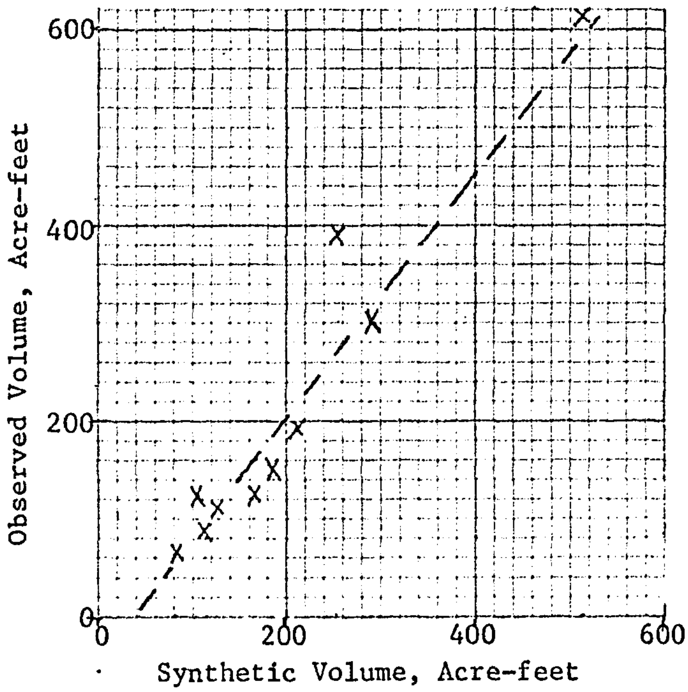
STATISTICS

		Deep Fork Creek At Portland Ave.		Bluff Creek		Deep Fork Creek At Eastern Ave.	
		Peak Flow	Volume	Peak Flow	Volume	Peak Flow	Volume
Synthetic / Observed	Mean	1.09	1.06	0.89	0.96	0.91	0.93
Synthetic / Observed	Standard Deviation	0.23	0.23	0.28	0.04	0.29	0.13
Regression Lines	Coefficient of Determination (r^2)	0.93	0.94	0.96	1.00	0.32	0.73
Regression Lines	Slope	0.87	1.31	0.75	1.05	0.65	0.88
Regression Lines	Std. Error of Est. Slope	0.08	0.12	0.06	0.01	0.54	0.31
Regression Lines	Intercept	88.5	-50.7	141.2	-0.6	1525	279
Regression Lines	Std. Error of Est. Intercept	166.1	28.3	42.0	1.0	1770	452
Regression Lines	Bias	No	No	Yes	Yes	No	No

CALIBRATION RESULTS

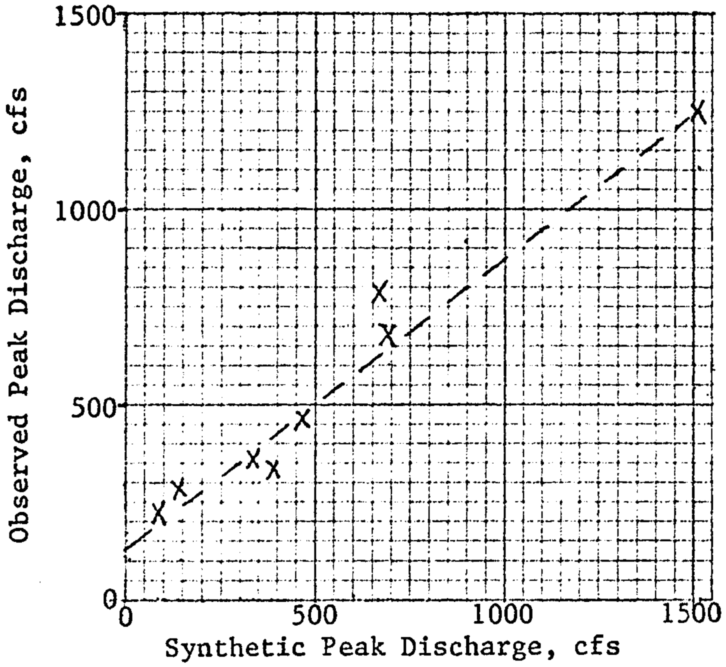
	Deep Fork Creek At Portland Ave.	Bluff Creek	Deep Fork Creek At Eastern Ave.
TC, Hr.	0.58	0.46	2.82
R, Hr.	0.20	0.89	2.40
LAG, Hr.	0.42	0.47	2.55
CP	0.78	0.39	0.62
STRKR	0.39	0.68	0.42
DLTKR	1.69	3.58	0.83
RTIOL	11.0	2.16	1.28
ERAIN	0.57	0.53	0.63

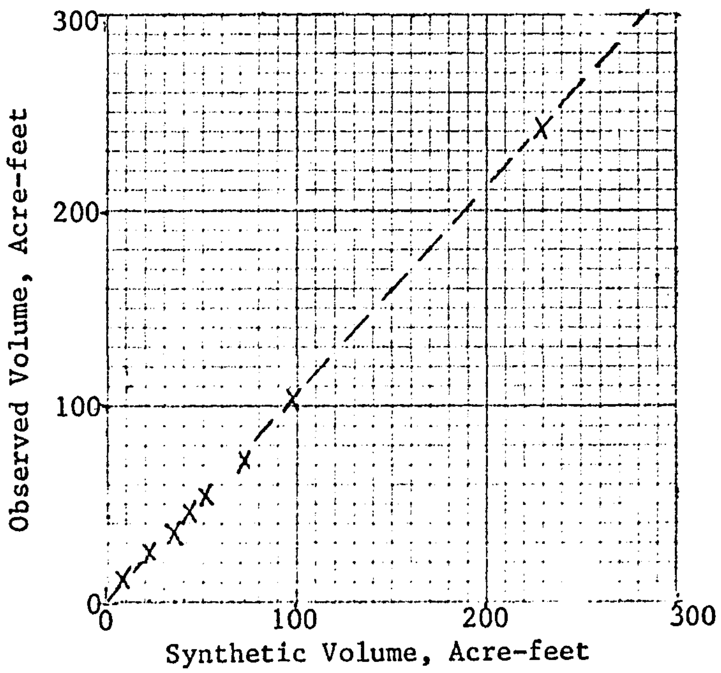




Deep Fork Creek at Portland Avenue

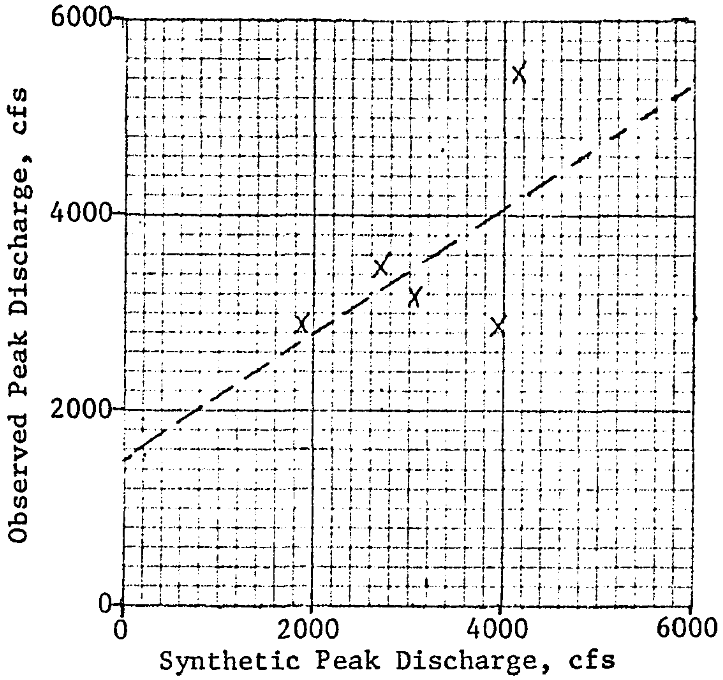
Figure 36. Comparison between observed and HEC-1 synthetic peak discharges and volumes of runoff.

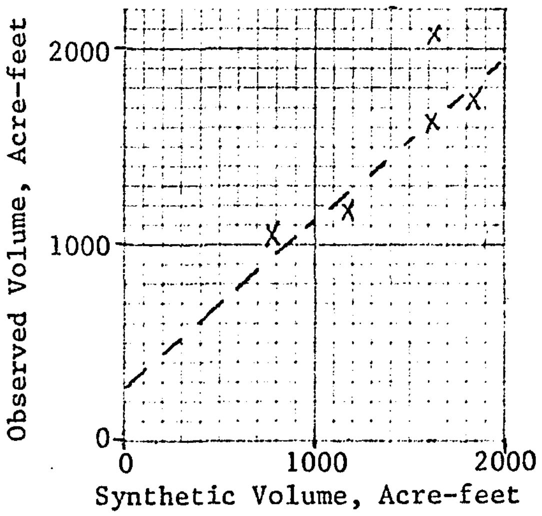




Bluff Creek

Figure 36--Continued





Deep Fork Creek at Eastern Avenue

Figure 36--Continued.

U. S. Geological Survey: G824

As with the two previous unit hydrograph models, this computer program was calibrated for this study by treating each basin as one drainage unit. As described in Chapter III, this model has a computer program within it to account for pre-storm soil wetness. Each storm's antecedent soil moisture condition is optimized from daily rainfall records (in this case the records of the National Weather Service Station at Will Rogers Airport in Oklahoma City) and daily lake evaporation records (from Canton Reservoir about 70 miles west of Oklahoma City) which are inputs to the computer model. However, this model did not perform significantly better than the previous two models, when an attempt was made to have them account for antecedent soil moisture conditions, as previously described, so it was concluded antecedent rainfall and evaporation were not influences on the discharge hydrographs used in this study. However, percentage of effective impervious cover was considered a variable for each basin in this study, and successive computer runs were made accordingly, the thought being that some of the rain falling onto impervious roofs and sidewalks runs onto pervious ground and has a chance to soak in.

Table 10 has the statistical results of the calibration, Figure 37 has a plot of each rainfall-runoff event's observed versus synthetic peak discharge and runoff volume, and Figure 40 shows each basin's derived unit hydrograph, and Appendix B has each event's observed and synthetic hydrographs.

U. S. Environmental Protection Agency: SWMM

Unlike the three previous models, this is not a unit hydrograph model, it is a kinematic wave model, and each basin was modeled as a system of subcatchments and channels, as shown in Figures 11, 12, and 13,

TABLE 10

COMPARISON BETWEEN OBSERVED AND G824 SYNTHETIC PEAK DISCHARGES AND RUNOFF VOLUMES

	Storm Date	Observed Peak Discharge, CFS	G824 Synthetic Peak Discharge, CFS	Observed Volume, Acre-Feet	G824 Synthetic Volume, Acre-Feet
Deep Fork Creek at Portland Ave.	4-29-74	1384	1596	191.65	216.09
	5-23-74	1204	1148	124.96	96.63
	6- 8-74(I)	2148	2423	387.23	279.15
	6- 8-74(II)	1284	2449	125.58	208.06
	11- 2-74	3600	4445	617.70	532.59
	5-13-75	1304	1087	112.66	116.47
	5-22-75	820	708	152.89	156.67
	6- 5-75	896	860	61.68	68.09
	6- 6-75	1000	602	89.89	104.19
	8-14-75	2340	2671	301.27	278.66
Bluff Creek	3- 8-74	675	1099	71.53	97.85
	3-10-74	336	295	45.73	28.62
	5-21-74	288	189	23.28	25.15
	5-23-74	460	581	50.63	53.87
	11- 2-74	1250	1742	240.32	231.69
	5-14-75	357	379	35.45	36.09
	6-16-75	214	260	10.65	27.23
	8-14-75	782	605	100.55	75.65
Deep Fork Creek at Eastern Ave.	5- 2-75	2879	2061	1013.7	831.2
	5-13-75	3198	3081	1184.6	1124.3
	5-22-75	3450	2670	2097.0	1599.2
	7-24-75	2883	5054	1729.3	2056.1
	8-14-75	5450	6723	1609.9	2313.1

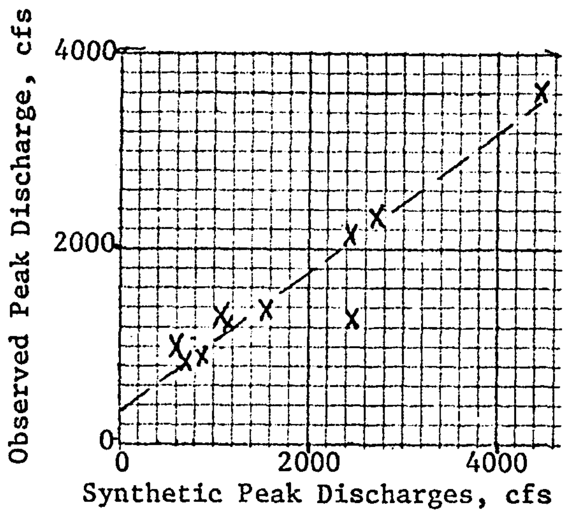
TABLE 10 - CONTINUED

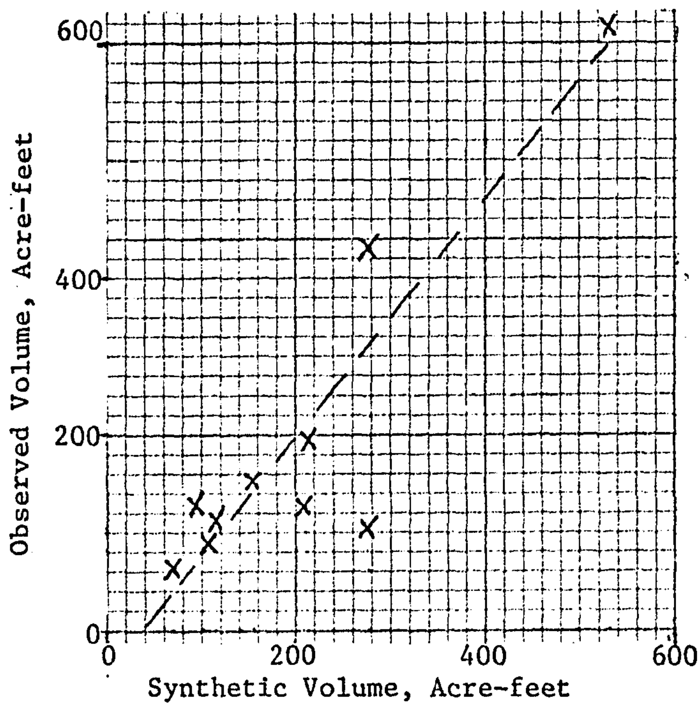
STATISTICS

		Deep Fork Creek At Portland Ave.		Bluff Creek		Deep Fork Creek At Eastern Ave.	
		Peak Flow	Volume	Peak Flow	Volume	Peak Flow	Volume
Synthetic Observed	Mean	1.08	1.04	1.11	1.18	1.09	1.03
	Standard Deviation	0.35	0.26	0.33	0.60	0.42	0.28
Regression Lines	Coefficient of Determination (r^2)	0.89	0.93	0.88	0.95	0.56	0.43
	Slope	0.68	1.22	0.61	1.03	0.42	0.46
	Std. Error of Est. Slope	0.08	0.12	0.09	0.10	0.21	0.30
	Intercept	382.7	-33.8	153.3	-1.8	1928	794
	Std. Error of Est. Intercept	180.2	38.6	76.2	9.5	911	511
	Bias	Yes	No	Yes	No	No	No

CALIBRATION RESULTS

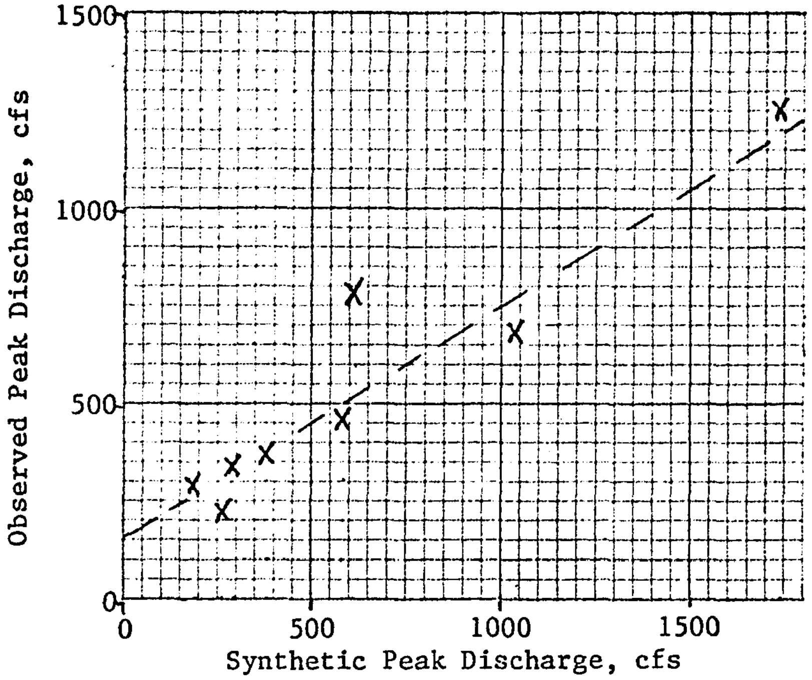
	Deep Fork Creek At Portland Ave.	Bluff Creek	Deep Fork Creek At Eastern Ave.
Percent Impervious	25	25	25
TC, Min.	36	34	210
KSW, Hr.	0.73	1.2	2.8
PSP	2.12	1.98	1.71
KSAT	0.12	0.16	0.10
RGF	6.3	14.6	9.06
BMSM	16.2	39.0	39.0
EVC	0.6	1.4	1.1
RR	0.7	1.3	0.8

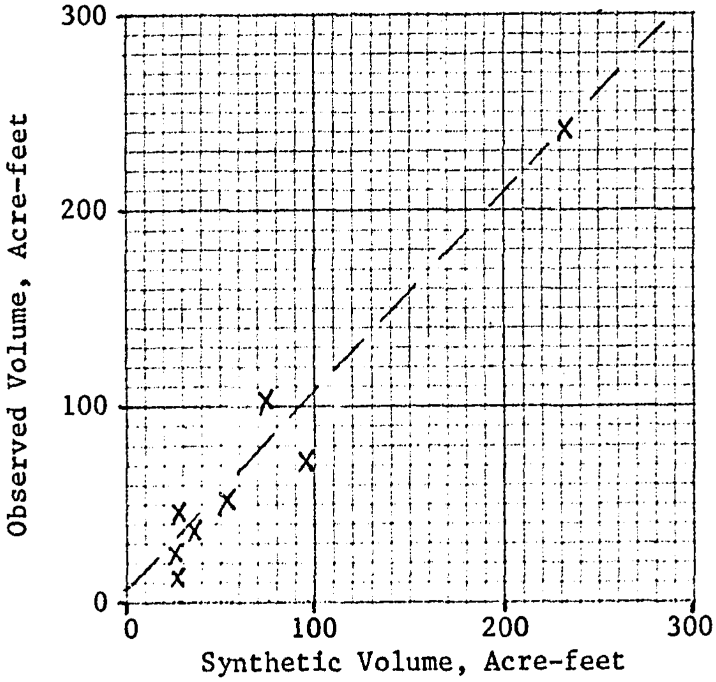




Deep Fork Creek at Portland Avenue

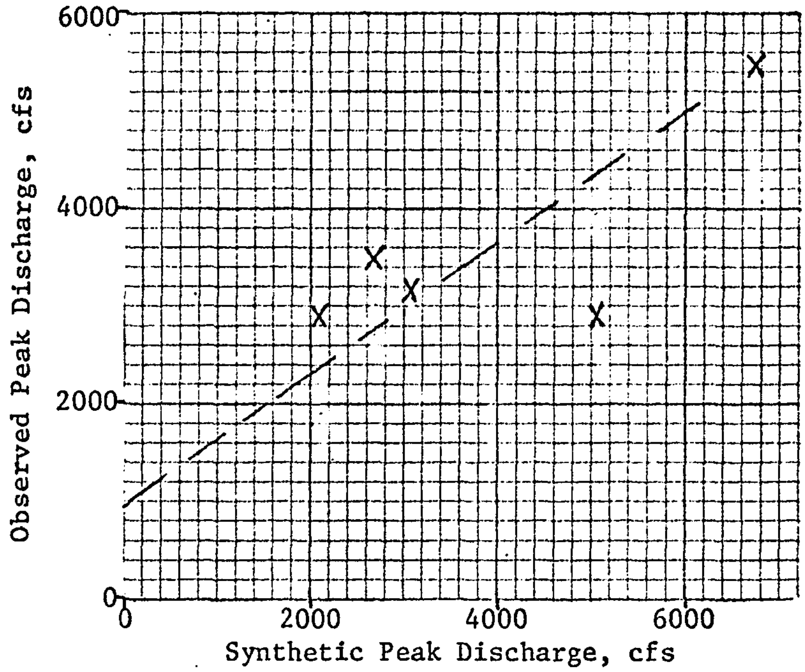
Figure 37. Comparison between observed and G824 synthetic peak discharges and runoff volumes.

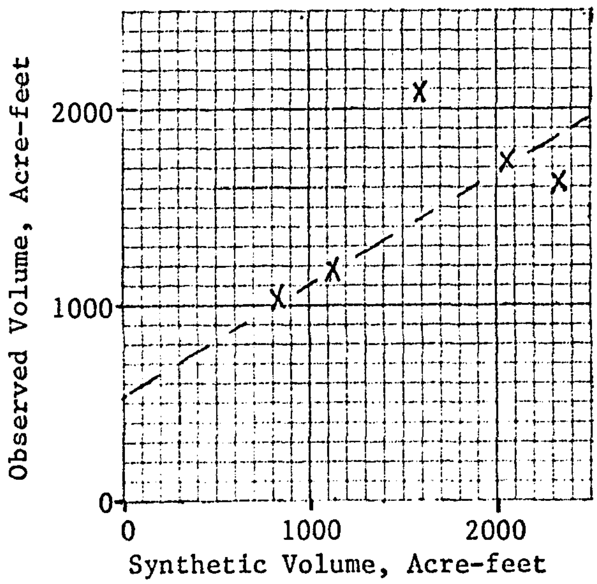




Bluff Creek

Figure 37--Continued.





Deep Fork Creek at Eastern Avenue

Figure 37--Continued.

and Tables 1, 2, and 3. During the early stages of calibration, subcatchment surface slope and roughness were treated as variable parameters in influencing hydrograph shape and timing, but they were found to be of ineffective impact, and were left constant in the later phases of calibration. Instead, it was found the most significant influence on hydrograph shape and timing is the subcatchment's length that runoff must travel overland, which is adjusted in the computer program by controlling the WIDTH variable.

This study offered a particular opportunity for comparison due to its nature. In SWMM water is not contributed uniformly to a stream along all its length from its adjoining subcatchments, instead, all the subcatchment's water is contributed to a stream at one point, its head, as shown in Figure 31 Chapter III. Thuse, the modeler must decide which of two options to take.

First, the subcatchment's outfall flow may be put at the head of the next channel reach downstream, for example, in Figure 11, Chapter II the outflow of Subcatchments 1 and 2 would be sent directly into Stream Reach C, and not A. The problem is, that approach may not sufficiently attenuate and lag the hydrograph because it has no influence of Stream Reach A, and the modeler must increase the distance the length the water must sheet flow across Subcatchments 1 and 2 in order to get the proper lag and attenuation affect. That was the approach used in this study for modeling Deep Fork Creek at Portland Avenue, so that Stream Reaches A, B, and D were not in the SWMM model. It was necessary to cut WIDTH about half its actual physical value for most of the subcatchments, which was equivalent to doubling the actual distance water must flow overland to reach the channels.

Secondly, the subcatchment's outfall flow may be put at the head of the stream reach flowing through the subcatchment, for example, in Figure 12, the outflow of Subcatchments 1 and 2 would be sent into the head of Stream Reach A, then routed and attenuated through that reach. This was the approach used in modeling Bluff Creek and Deep Fork Creek at Eastern Avenue, and it proved more satisfactory, in that it required little or no adjustment of WIDTH from the values measured from the basin's maps.

Effective percentage of impervious cover was considered a variable in calibrating SWMM, just as it was in G824, and successive computer runs were made balancing percentage of impervious cover against pervious infiltration rates to match as closely as possible the observed outflow volumes. However, no attempt was made to adjust parameters for antecedent soil moisture conditions because trying to do so on the models TR-20, HEC-1, and G824 had proved unsuccessful. Instead, for each basin, the infiltration parameters were held fixed for each flood event.

Since SWMM is not a unit hydrograph model, no true unit hydrograph could be computed for any one of the basins. This is a kinematic wave model, and is supposed to have an advantage in that it accounts for the flood's variable travel time, shallow water usually moving slower than deep water in the streams and overland. Thus, runoff from more intense rainstorms moves to the outfall and has a sharper peak hydrograph than runoff from slow steady rainfalls. This investigator has seen channel travel times double during water surface profile calculations by increasing discharge several fold. However, a "quasi-unit hydrograph" was obtained by routing an instantaneous one inch rainfall over each basin with total runoff, and the results are graphed in Figure 40.

A Mann-Whitney U test was made to determine if infiltration is influenced by peak rainfall intensity (72). It was found on Deep Fork Creek at Portland Avenue that infiltration rate tends to decrease as rainfall intensifies, but the opposite takes place on Bluff Creek, and the relationship is random on Deep Fork Creek at Eastern Avenue. Therefore the test was judged inconclusive and on each basin the infiltration rates were not changed from storm to storm.

Table 11 has the statistics of the calibration results, and Figure 38 has a plot of each rainfall-runoff event's observed versus synthetic peak discharge and runoff volume.

National Weather Service, River Forecast Center: MINICAT

This is also a kinematic wave model, and it was calibrated by very nearly the same process as used on SWMM, except for two differences. This model can compute infiltration losses by either the Soil Conservation Service curve number, Equations (2) through (4) in Chapter III, or Horton's function, Equation (21) in Chapter III. It became clear during the calibration that Horton's function gave superior results, and use of the SCS curve number was abandoned.

A second difference between this model and SWMM is that each subcatchment's discharge is contributed to its receiving stream as lateral inflow uniformly distributed along its length, so there is no question as to how to model each catchment-stream relationship.

Table 12 has the statistics for the calibration results,

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72. Johnson, Statistics, pp. 522-528.

TABLE 11

COMPARISON BETWEEN OBSERVED AND SWMM SYNTHETIC
PEAK DISCHARGES AND RUNOFF VOLUMES

	Storm Date	Observed Peak Discharge, CFS	SWMM Synthetic Peak Discharge, CFS	Observed Volume, Acre-Feet	SWMM Synthetic Volume, Acre-Feet
Deep Fork Creek at Portland Ave.	4-29-74	1384	1543	191.65	219.67
	5-23-74	1204	1236	124.96	85.10
	6- 8-74(I)	2148	2258	387.23	256.56
	6- 8-74(II)	1284	1960	125.58	141.94
	11- 2-74	3600	3881	617.70	554.57
	5-13-75	1304	1492	112.66	135.58
	5-22-75	820	934	152.89	222.60
	6- 5-75	896	1117	61.68	61.69
	6- 6-75	1000	663	89.89	124.04
	8-14-75	2340	2680	301.27	290.98
Bluff Creek	3- 8-74	675	1035	71.53	72.74
	3-10-74	336	223	45.73	24.68
	5-21-74	288	145	23.28	24.49
	5-23-74	460	468	50.63	42.45
	11- 2-74	1250	2369	240.32	242.37
	5-14-75	357	294	35.45	32.34
	6-16-75	214	236	10.65	27.01
	8-14-75	782	551	100.55	67.08
Deep Fork Creek at Eastern Ave.	5- 2-75	2879	1546	1013.7	636.0
	5-13-75	3198	2976	1184.6	1296.0
	5-22-75	3450	3374	2097.0	2306.3
	7-24-75	2883	3539	1729.3	1619.3
	8-14-75	5450	7821	1609.9	2700.6

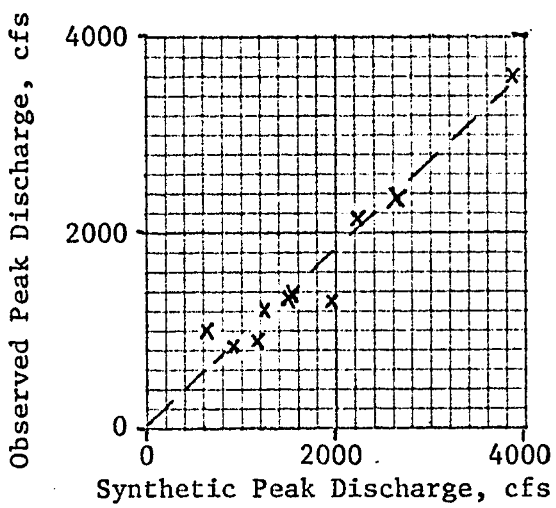
TABLE 11 - CONTINUED

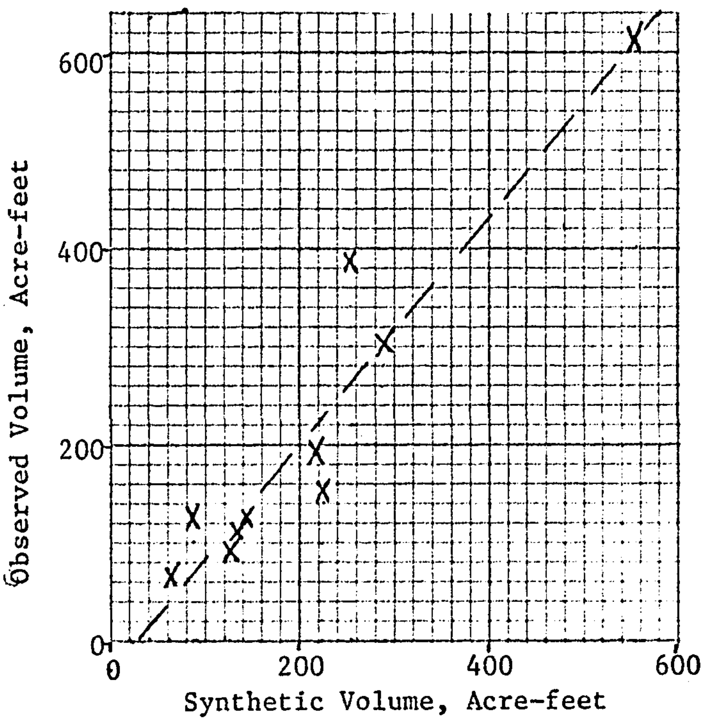
STATISTICS

		Deep Fork Creek At Portland Ave.		Bluff Creek		Deep Fork Creek At Eastern Ave.	
		Peak Flow	Volume	Peak Flow	Volume	Peak Flow	Volume
Synthetic / Observed	Mean	1.11	1.05	1.03	1.07	1.02	1.09
	Standard Deviation	0.21	0.26	0.47	0.62	0.34	0.38
Regression Lines	Coefficient of Determination (r^2)	0.93	0.91	0.87	0.96	0.91	0.60
	Slope	0.87	1.16	0.43	0.98	0.44	0.41
	Std. Error of Est. Slope	0.08	0.13	0.07	0.09	0.08	0.19
	Intercept	56.7	-25.2	257.7	7.1	1895	824
	Std. Error of Est. Intercept	161.7	32.8	65.3	8.2	354	362
	Bias	No	No	Yes	No	Yes	No

CALIBRATION RESULTS

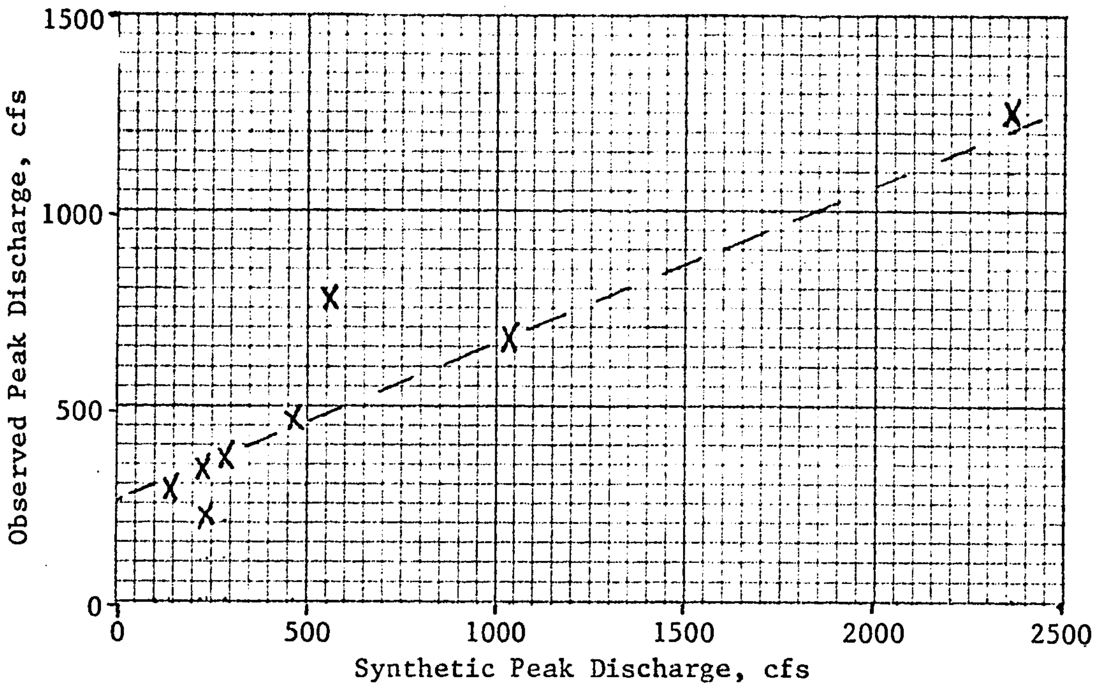
	Deep Fork Creek At Portland Ave.	Bluff Creek	Deep Fork Creek At Eastern Ave.
Percent Impervious	45	25	25
Upper Streams Modeled	NO	YES	YES
Ratio Actual Width to WWIDTH	1.0	1.0	1.3
Max. Infiltration Rate (inches/Hr)	3	5	1
Min. Infiltration Rate (inches/Hr)	.15	.55	.1

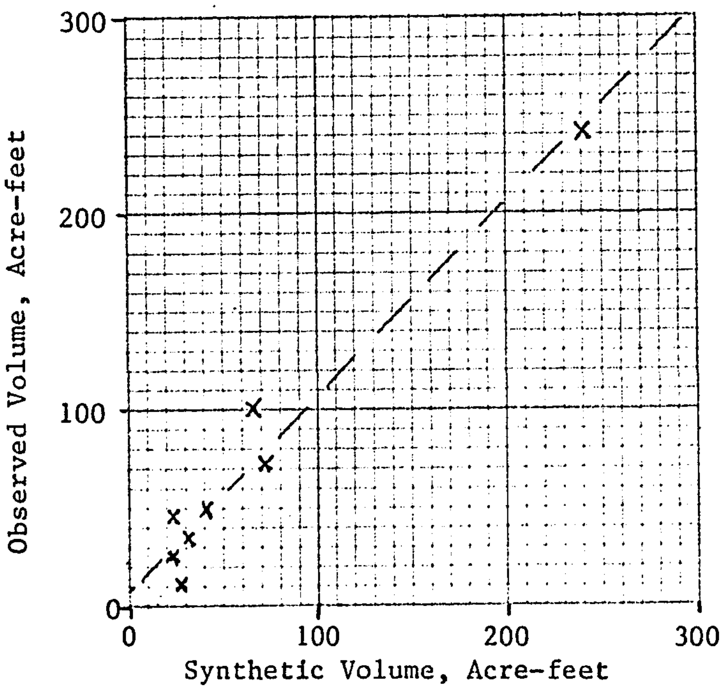




Deep Fork Creek at Portland Avenue

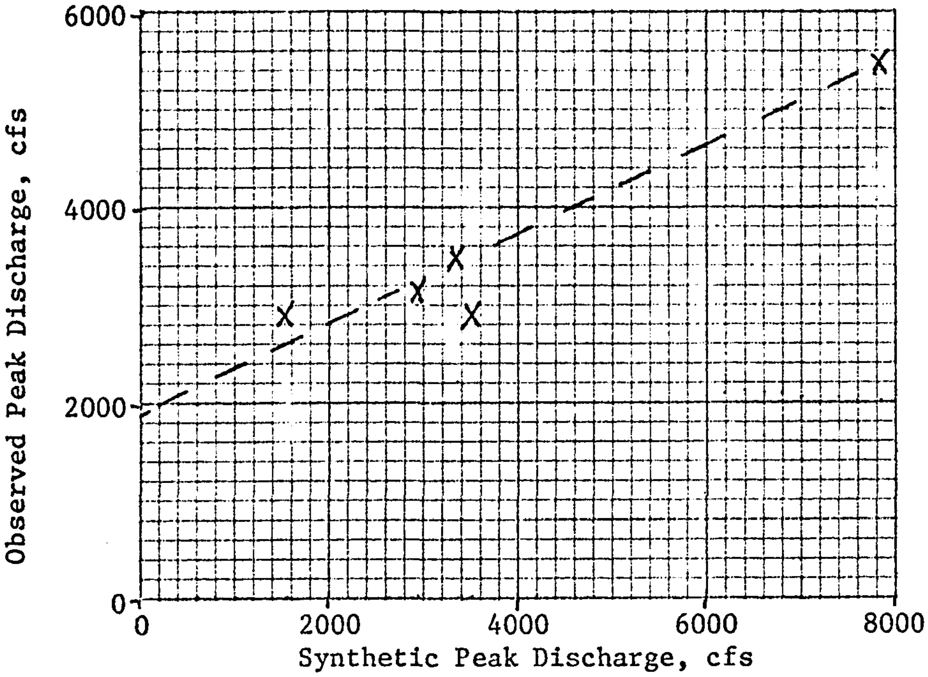
Figure 38. Comparison between observed and SWMM synthetic peak discharges and runoff volumes.

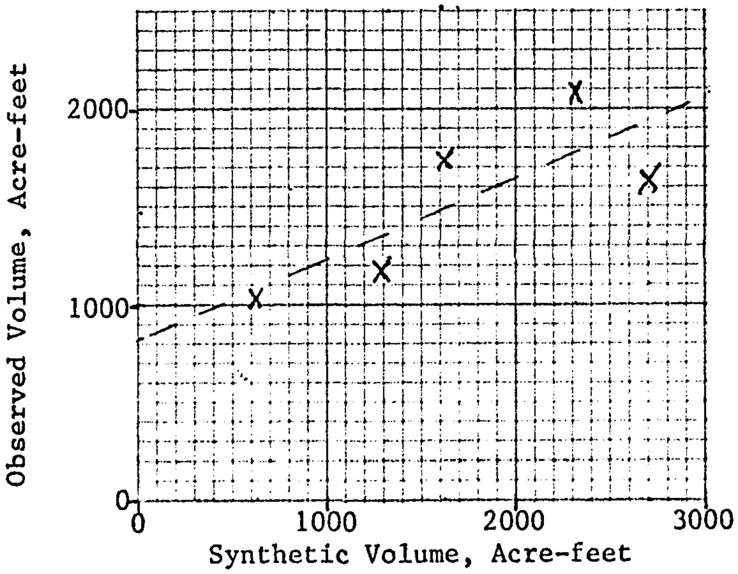




Bluff Creek

Figure 38--Continued.





Deep Fork Creek at Eastern Avenue

Figure 38--Continued.

TABLE 12

COMPARISON BETWEEN OBSERVED AND MINICAT SYNTHETIC PEAK DISCHARGES AND RUNOFF VOLUMES

	Storm Date	Observed Peak Discharge, CFS	MINICAT Synthetic Peak Discharge, CFS	Observed Volume, Acre-Feet	MINICAT Synthetic Volume, Acre-Feet
Deep Fork Creek at Portland Ave.	4-29-74	1384	1896	191.65	252.49
	5-23-74	1204	829	124.96	78.90
	6- 8-74(I)	2148	2528	387.23	277.44
	6- 8-74(II)	1284	1839	125.58	167.11
	11- 2-74	3600	5040	617.70	555.56
	5-13-75	1304	1004	112.66	120.12
	5-22-75	820	707	152.89	166.76
	6- 5-75	896	610	61.68	63.02
	6- 6-75	1000	466	89.89	77.41
	8-14-75	2340	3309	301.27	308.21
Bluff Creek	3- 8-74	675	1169	71.53	73.09
	3-10-74	336	222	45.73	15.18
	5-21-74	288	129	23.28	9.71
	5-23-74	460	897	50.63	56.58
	11- 2-74	1250	2312	240.32	231.17
	5-14-75	357	408	35.45	28.06
	6-16-75	214	256	10.65	17.91
	8-14-75	782	898	100.55	75.19
Deep Fork Creek at Eastern Ave.	5- 2-75	2879	1483	1013.7	720.2
	5-13-75	3198	2548	1184.6	1129.7
	5-22-75	3450	2940	2097.0	1975.9
	7-24075	2883	5070	1729.3	2164.5
	8-14-75	5450	5659	1609.9	1877.9

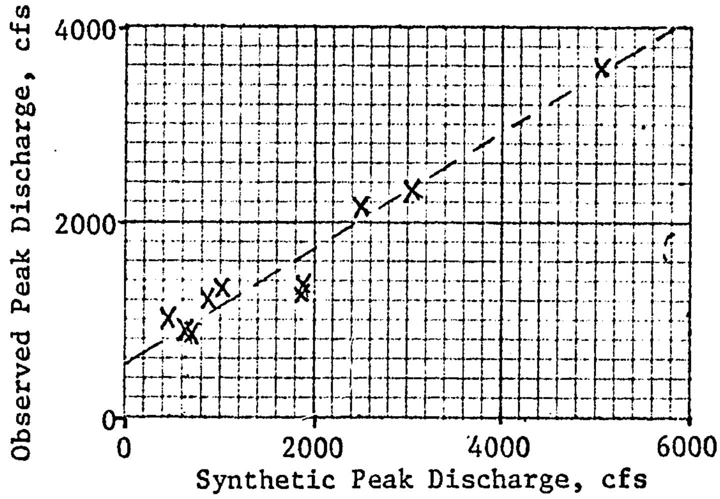
TABLE 12 - CONTINUED

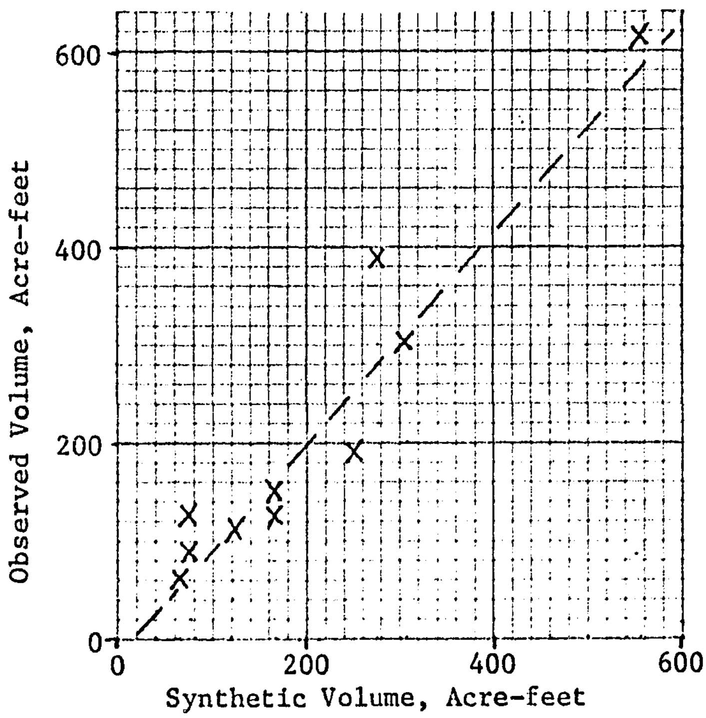
STATISTICS

		Deep Fork Creek At Portland Ave.		Bluff Creek		Deep Fork Creek At Eastern Ave.	
		Peak Flow	Volume	Peak Flow	Volume	Peak Flow	Volume
Synthetic / Observed	Mean	1.03	1.00	1.27	0.88	0.97	1.00
	Standard Deviation	0.37	0.23	0.55	0.43	0.43	0.21
Regression Lines	Coefficient of Determination (r^2)	0.95	0.92	0.92	0.96	0.40	0.81
	Slope	0.57	1.11	0.46	0.99	0.39	0.63
	Std. Error of Est. Slope	0.05	0.11	0.05	0.08	0.26	0.18
	Intercept	552.2	-12.0	186.6	9.37	2203	535
	Std. Error of Est. Intercept	108.1	28.3	89.0	7.26	923.6	223.5
	Bias	Yes	No	Yes	No	No	No

CALIBRATION RESULTS

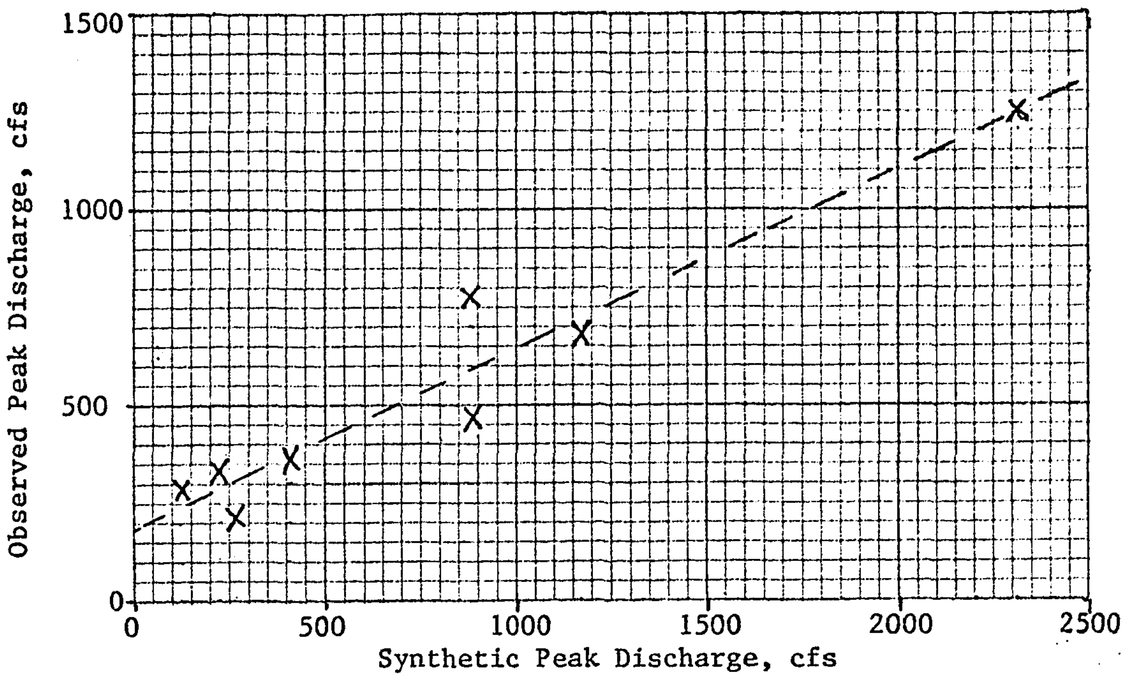
	Deep Fork Creek At Portland Ave.	Bluff Creek	Deep Fork At Eastern Ave.
Percent Impervious	45	45	45
Average divisor of overland length	2	2	3
Initial Loss Rate (Inches/Hr)	4	5	1
Steady Loss Rate (Inches/Hr)	.35	.6	.07

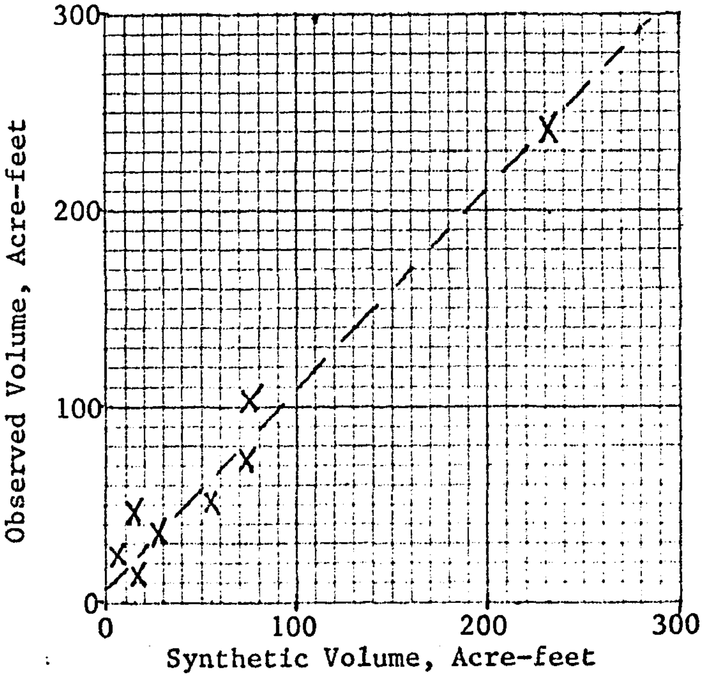




Deep Fork Creek at Portland Avenue

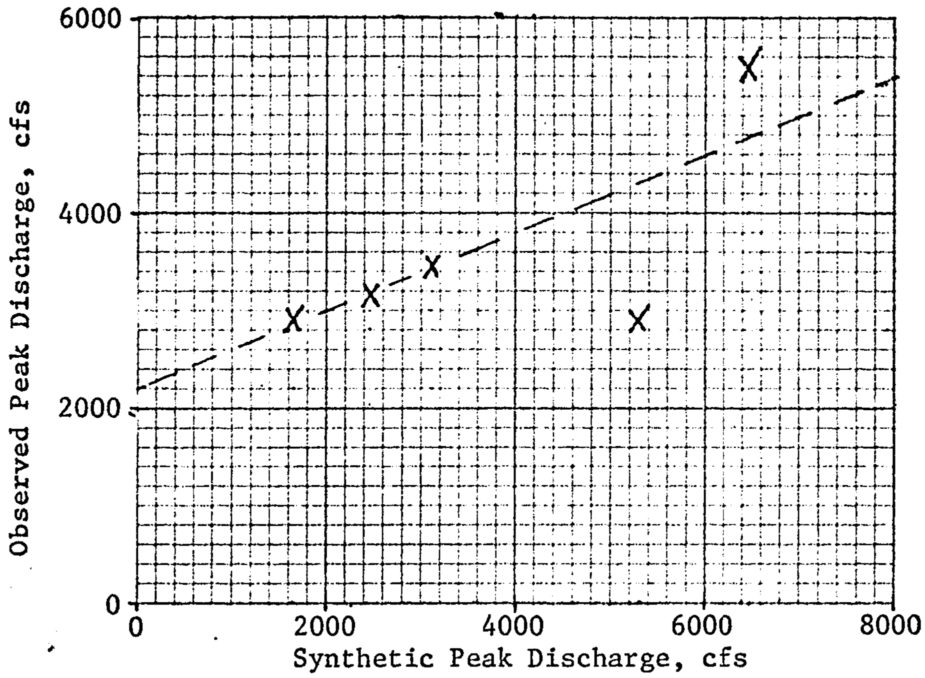
Figure 39. Comparison between observed and MINICAT synthetic peak discharges and runoff volumes.

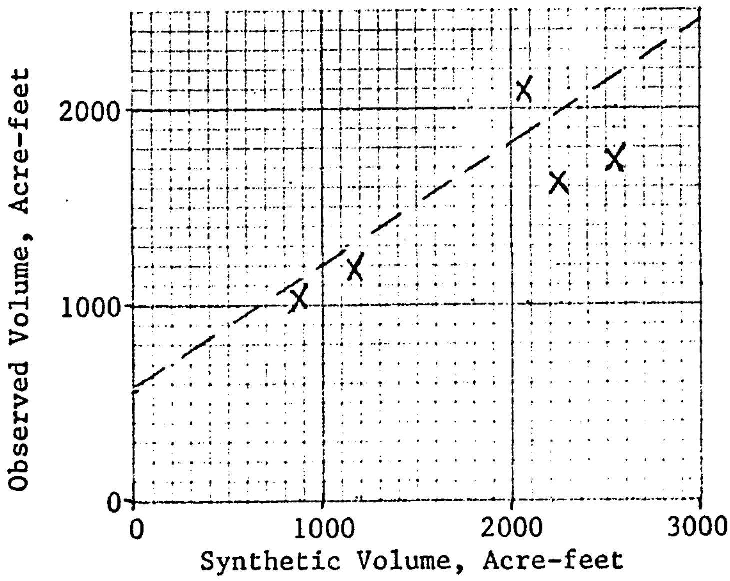




Bluff Creek

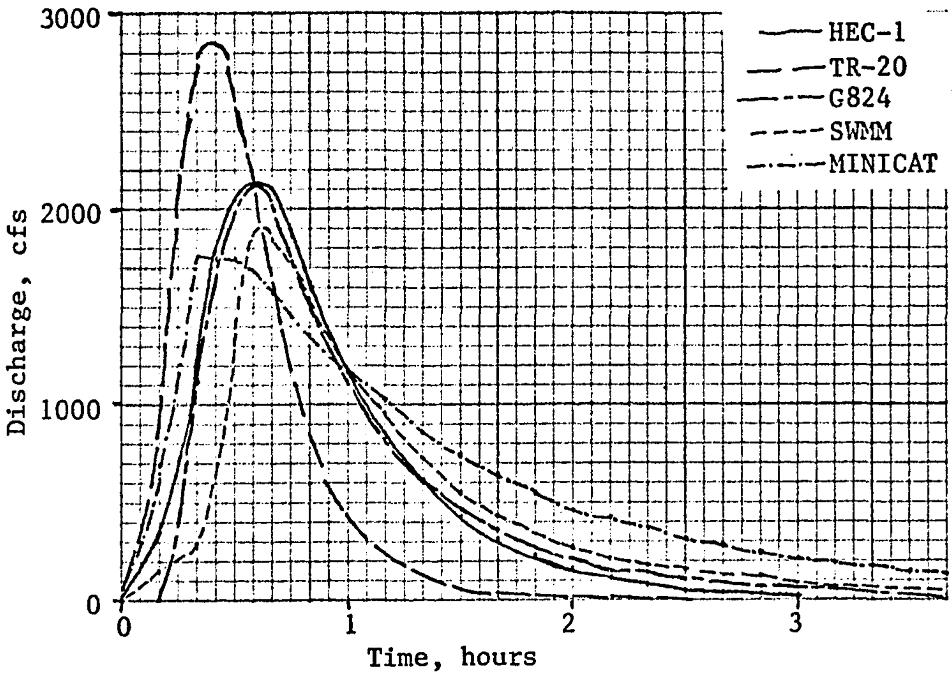
Figure 39--Continued.



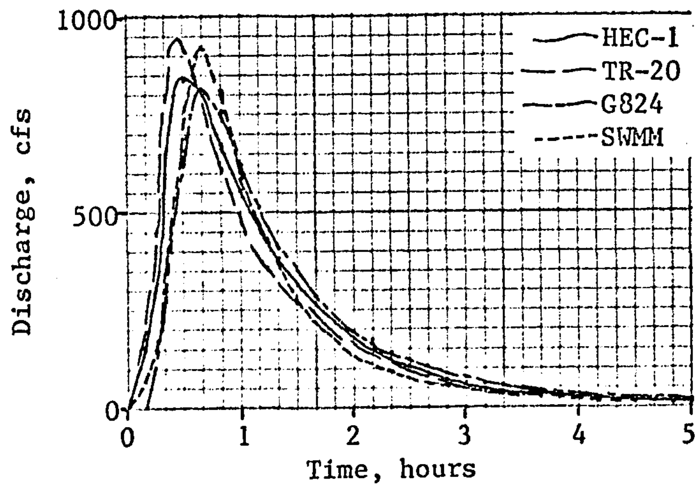


Deep Fork Creek at Eastern Avenue

Figure 39--Continued.

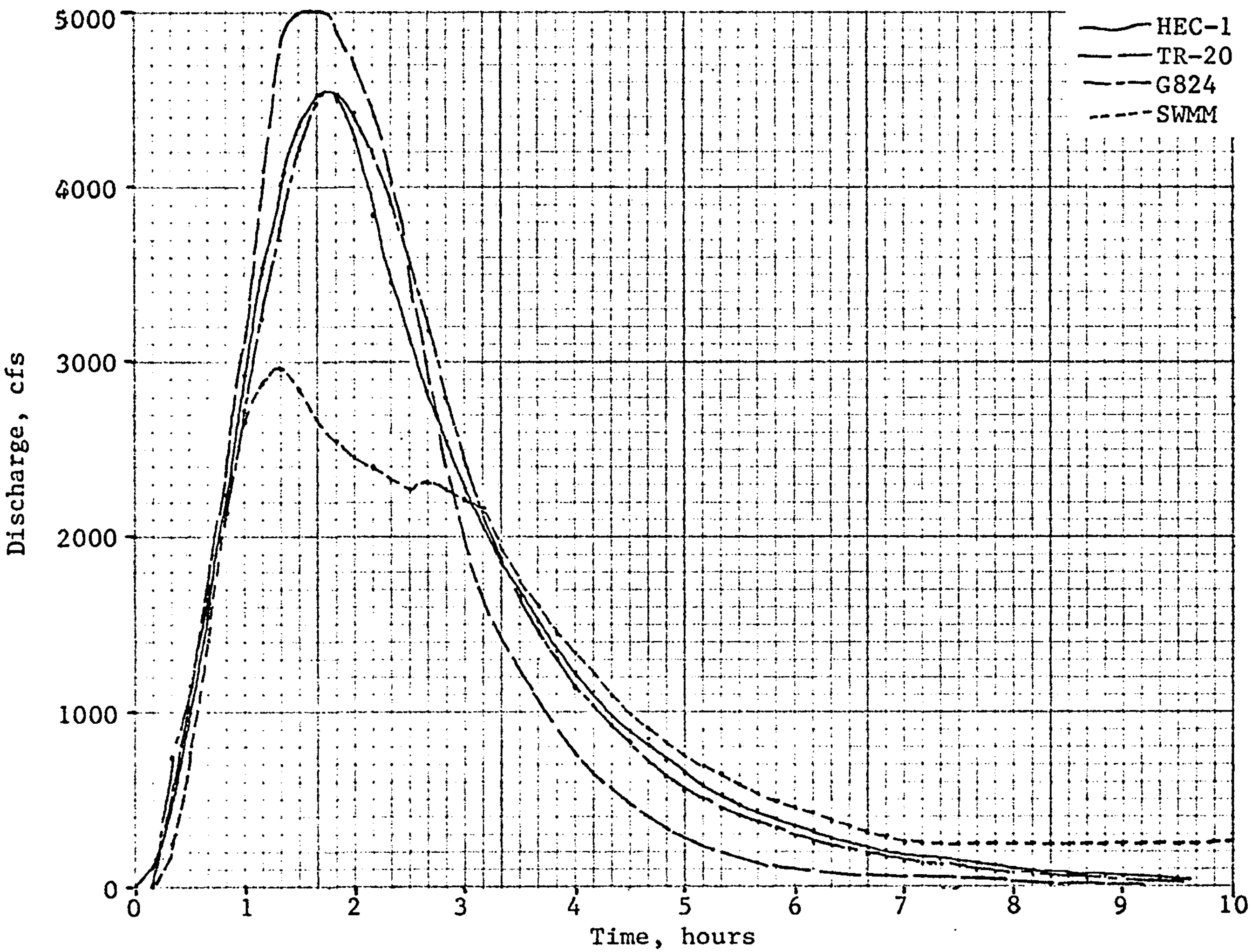


Deep Fork Creek at Portland Avenue



Bluff Creek

Figure 40. Unit hydrographs for the study basins.



Deep Fork Creek at Eastern Avenue

Figure 40--Continued.

Figure 39 has plots of each rainfall-runoff event's observed versys synthetic peak discharge and volume of runoff, Figure 40 shows a "quasi-unit hydrograph" for each basin as derived in a manner described in the previous section of SWMM, and Appendix B has a plot of each event's hydrograph.

Discussion

The results of the calibration given in Tables 7 thru 12 have rainfall loss rates and unit hydrograph shapes that are similar to others reported in the literature for urban basins (73-83), except for the unit hydrograph for Bluff Creek, which is flatter than typical, probably due to a series of ponds that attenueate the floods upstream of the flood gage.

Table 13 summarizes the calibration results for comparing one model's reliability to another's. The ratio $\sigma/\overline{X}$ (standard deviation divided by the mean), or coefficient of variations is shown because it was

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73. Diniz, "Modifications to SWMM," pg. 266.
74. Jewell, "Methodology for Calibrating Models," pp. 491-493.
75. Abbott, "Testing of Models," pg. 38.
76. Beard, "Urban Model," pp. 26-40.
77. Thomas, "Flood Discharges," pp. 30, 31.
78. Lorant, "Planning Using SWMM," pp. 151-164.
79. James F. MacLaren, Ltd., "Comparison of Models," pg. 66.
80. "Urban Hydrology--TR-55," pg. 2-5.
81. Reynolds, Dale, personal communication, June 1978.
82. Schaake, "Rational Method," pg. 364.
83. Russell, "Design Flows," pg. 50.

TABLE 13

SUMMARY OF STATISTICS

Site	Parameter	Statistic	Rational	TR-20	HEC-1	G824	SWMM	MINICAT
Deep Fork Creek At Portland Ave.	Peak Discharge	$\sigma / \bar{X}$	0.26	0.23	0.21	0.32	0.19	.36
		r^2	0.80	0.97	0.93	0.89	0.93	.95
		Bias	No	Yes	No	Yes	No	Yes
	Volume of Runoff	$\sigma / \bar{X}$	-	0.31	0.22	0.25	0.25	.23
		r^2	-	0.88	0.94	0.93	0.91	.92
		Bias	-	No	No	No	No	No
Bluff Creek	Peak Discharge	$\sigma / \bar{X}$	0.38	0.33	0.31	0.30	0.46	0.43
		r^2	0.54	0.96	0.96	0.88	0.87	0.92
		Bias	No	Yes	Yes	Yes	Yes	Yes
	Volume of Runoff	$\sigma / \bar{X}$	-	0.45	0.04	0.51	0.58	0.49
		r^2	-	0.97	1.00	0.95	0.96	0.96
		Bias	-	Yes	Yes	No	No	No
Deep Fork Creek At Eastern Ave.	Peak Discharge	$\sigma / \bar{X}$	0.38	0.47	0.32	0.39	0.33	.44
		r^2	0.44	0.46	0.32	0.56	0.91	.40
		Bias	No	Yes	No	No	Yes	No
	Volume of Runoff	$\sigma / \bar{X}$	-	0.39	0.15	0.27	0.35	.21
		r^2	-	0.63	0.73	0.43	0.60	.81
		Bias	-	Yes	No	No	No	No

TABLE 14

RANKING OF MODELS BY ACCURACY

			Rational	TR-20	HEC-1	G824	SWMM	MINICAT
PEAK DISCHARGES	Deep Fork Creek at Portland Ave.	Coefficient of variation	4	3	2	5	1	6
		Coefficient of determination	6	1	4	5	3	2
		Bias	4	5	2	3	1	6
	Bluff Creek	Coefficient of variation	4	3	2	1	6	5
		Coefficient of determination	6	1	2	4	5	3
		Bias	1	4	2	3	6	5
	Deep Fork Creek at Eastern Ave.	Coefficient of variation	3	6	1	4	2	5
		Coefficient of determination	4	3	6	2	1	5
		Bias	3	6	1	4	2	5
		Total Ranking	35	32	22	31	27	42
					$\bar{x} = 31.5$		$\sigma = 6.8$	
VOLUME OF RUNOFF	Deep Fork Creek at Portland Ave.	Coefficient of variation	-	5	1	3	4	2
		Coefficient of determination	-	5	1	2	4	3
		Bias	-	1	5	4	3	2
	Bluff Creek	Coefficient of variation	-	2	1	4	5	3
		Coefficient of determination	-	2	1	5	4	3
		Bias	-	5	4	2	1	3
	Deep Fork Creek at Eastern Ave.	Coefficient of variation	-	5	1	3	4	2
		Coefficient of determination	-	3	2	5	4	1
		Bias	-	5	1	3	4	2
		Total Ranking	-	33	17	31	33	21
					$\bar{x} = 27$		$\sigma = 7.5$	

Lower numbers imply greater accuracy

found during the calibration process that the ratio changed very little for each model for each basin as the mean approached unity, so there was really little gained by fine tuning each model until the mean was exactly unity.

A scheme of ranking was devised to rate the computer models, and the results displayed in Table 14. Using Table 13, the model's performance was rank-ordered by row, by coefficient of variation, coefficient of determination, and bias of the regression line. For example, in Table 13, under Bluff Creek, Peak Discharge, coefficient of variation, G824 ranks first because it has the lowest coefficient of variation, and SWMM ranks fifth as shown in Table 14 because it has the highest. Coefficient of determination was ranked as the highest coefficient having the highest rank. A model was considered biased with respect to the regression line if its slope differs from unity by more than three standard errors of the estimate of the slope, or if the intercept differs from zero by more than three standard errors of the estimate, as tabulated in Tables 7 through 12. It was found in conducting the study that the Soil Conservation Service model, TR-20, tends to be biased, and gives too high a figure for the big floods, and gives too small a number for the lesser floods. After rank-ordering by rows, the ranks were summed by column in Table 14, that is, by model, and the models rank-ordered. As may be seen in Table 14, HEC-1 has a slightly better performance than the other models on the data used in this study.

This is the place to discuss the great uncertainty in the input data that probably accounts for the size of the coefficients of variation in Table 13. All of the rainfall records used in this study are of one rain gage in each basin, at its outfall end, except two rainfall-runoff events on Deep Fork Creek at Eastern Avenue when rainfall data was also recorded on the Portland Avenue gage. That one rain gage's record in each basin was probably not representative of the rainfall over the entire basin, considering the size of basins (1.64 to 28.2 square miles) used in this study. Errors in basin-wide rainfall data of course lead to errors in runoff volumes which lead to errors in peak discharges. The Soil Conservation Service has a series of charts in Figure 4.6 of (84), which are useful in estimating the probable error between recorded total rainfall per event at the gage sites, and basin average total rainfall. It was found by going through the charts that rainfall volume probable errors for Deep Fork Creek at Portland Avenue run five to fifteen percent, depending on the storm, run five to ten percent of Bluff Creek, and twenty to fifty percent on Deep Fork Creek at Eastern Avenue. Some additional data error creeps in through the discharge hydrographs, due to uncertainties in reading the incremental stage height on the gage recorder, and uncertainties in the discharge measurements, as revealed in the small scatter about the stage-discharge line in Figure 9, Chapter II. It may be that antecedent soil moisture conditions do have an influence on the outflow hydrograph, contrary to the findings in Chapter IV, but it is masked by uncertainties and errors in the input data.

Neither is this study able to shed any light on the controversy between proponents of unit hydrographs and proponents of kinematic wave models, which has to do with the kinematic wave modelers accusing the unit hydrograph of moving water through the watershed at the same speed,

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71. Mockus, Hydrology, pg. 4.20.

whether it is a big flood or a small flood. The time increments in the input rainfall-runoff data on Deep Fork Creek at Portland Avenue and Bluff Creek were ten minutes, one-third of their times of concentration, too large to detect any small differences in the performance of the two theories, and the small number of storms used on Deep Fork Creek at Eastern Avenue is too small a sample and the rainfall data too uncertain to draw any conclusions from the basin's performance.

A study needs to be done on more basins with more storms and small data time increments in order to determine any influence of antecedent soil moisture conditions, and determine the reliability of unit hydrographs versus kinematic waves.

CHAPTER V

ENGINEERING APPLICATIONS

The hydrology models presented in Chapter III and the calibration results discussed in Chapter IV are not intended as mere research curiosities for ivory tower dreamers. Indeed, they are intended for use as tools for addressing real world problems of flood hazard prediction, flood control, flood damage reduction, and stream pollution. This chapter is a comparison of the usefulness of the various models in aiding to solve the above problems, and in particular, possible uses of the calibration results of this study. In addition this chapter contains a comparison of cost, resource needs, and ease of using the various models for applications.

It must be noted that most of the models used in this study are hydrologic aids for engineering design/evaluation, and provide only discharges to be used as input data for other techniques or computer programs to determine the design size or evaluate hydraulic structures, or determine water surface profile elevations. SWMM has built within it limited automatic capability to design size storm sewers and determine water surface elevations in manholes in storm sewers, but no other model used in this study has any capability to solve for final engineering design sizing of structures automatically. Indeed, the U.S. Geological Survey's Urban

Flood Hydrograph Synthesis Model (G824) is strictly a calibration and research tool, and design engineers do not use it directly, but use regression equations for discharge return frequency and Clark's unit hydrograph coefficients developed from the model for natural basins discussed in Chapter III (85). For that reason, in the following discussion, reference will not be made to the computer program G824, but to the "USGS regression equations" generated from the use of the hydrograph synthesis model.

This chapter includes discussions and ranking of preference of the various models on the basis of the uses to be made of the models, the level of technical training necessary to use the models, the costs of using the models, and the size of computer required to use the models.

Model Uses

The engineering design uses to which these models can be placed are many and varied, but are consolidated to four categories for this study: culvert sizing, storm sewer system design, flood control project design (small detention ponds, large reservoirs, levee systems), and flood plain management (flood insurance studies; flood hazard studies for zoning, land use, and building permits). Table 15 ranks the six models according to suitability for the four categories of uses. The rankings are based on the experience gained in conducting the research for this report and on the author's use of most of the models for engineering projects.

The simpler, quicker, hand calculated models are preferable for

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85. Thomas, "Flood Discharges."

sizing culverts because only peak discharge is needed, and it is an overkill to use an expensive computer consuming time to code for such a simple task, like using a shotgun to kill a single housefly. The Rational Method is ranked first for sizing culverts because it is the most commonly used for small basins, which most culverts service. It is not known as the most reliable; indeed, there has been little calibration effort of the Rational Method compared to the other models discussed in this study, and the American Society of Civil Engineers considers it an "approximate" method (86), and recommends against its use for drainage basins larger than a few square miles (87). However, it is well suited to designing minor structures, where approximations are tolerable, where a change of one pipe size can change a small culvert's hydraulic capacity by 25 to 50 percent. The USGS regression equations are of known reliability, being derived from years of record on over one hundred flood gages in Oklahoma, and are now endorsed by the Oklahoma Department of Transportation for sizing culverts and bridges, but give discharges biased too high for basins under 300 acres, so the regression equations are ranked second in preference.

A storm sewer system in the context of this report means a network of at least a few dozen connected drainage conduits, inlets, manholes, and even lined open channels. This author gives preference to SWMM for design due to its powerful capability to compute the inflow hydrographs and automatically size the conduits in one computer run for the entire system. It is the most expensive and largest computer program

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86. "Residential Storm Water Management," pg. 26.

87. Design and Construction, pg. 43.

TABLE 15

ENGINEERING USES OF THE MODELS

	Culvert Sizing	Storm Sewer System Design	Flood Control Project Design	Flood Plain Management	Total of Rank Standings
Rational Method	1	3	6	6	16
USGS Regression Equations	2	6	5	1	14
HEC-1 (Corps of Engineers)	5	6	1	2	14
TR-20 (Soil Conservation Service)	5	6	2	2	15
SWMM (Environmental Protection Agency)	6	1	3	4	14
MITCAT	6	2	3	4	15

Explanation:
1 The model is very well suited to this use.
2 The model is well suited to this use.
3 The model can be employed in this use, but other models are more suitable.
4 The model is not well suited to this use, but can still be employed.
5 The model is very poorly suited to this use.
6 The model is inapplicable, and should not be employed in this use at all.

used in this study as will be mentioned in a later section of this chapter, but its automatic results are far cheaper than hand calculations. It is especially attractive in that it can test a proposed design for larger than design discharges, or check an existing system for surcharging, backwater, overflowing manholes and inlets, reverse flows, and even route excess flows overland through streets, parking lots, and gutters. It can provide water surface elevations of the results, and is the most nearly complete model used in this study, because it can compute a sewer system's hydrology, hydraulics, and water quality parameters in one computer run. It has even been adapted for natural stream hydraulics and water surface profiles for urban streams with culverts and road overflows (88, 89).

The MIT Catchment Model runs a close second to SWMM because it can also automatically size storm sewer pipes. The Rational Method is rated third because it is now the most commonly used method for deriving the peak inflows to route through sewer systems, which is presently mostly done by hand calculations. The USGS regression equations are rated low because the equations derived for Oklahoma are biased giving discharges too large for drainage areas less than 300 acres, which is far larger than the drainage areas of the upstream inlets of storm sewer systems. The Corps of Engineers and SCS hydrology computer programs are rated low because they cannot route flow through underground conduits modeling for surcharging, backwater, looped systems, or split flows. The user can by

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88. Diniz, "Modifications to SWMM," pg. 260.

89. Eicher, Christian W., "Applications of SWMM-EXTRAN for the Evaluation of Existing Urban Drainage Systems," in "Proceedings Storm Water Management Model (SWMM) Users Group Meeting, May 4-5, 1978," U.S. Environmental Protection Agency, 1978.

those programs derive the upstream inlet inflow hydrographs, but must then route through the conduits by means of other computer programs or hand calculations. They are much more tedious to use and require a great deal more work than the Rational Method to get the peak inlet inflow, and therefore are rated below it in usefulness for sewer design.

In order to aid in the design of a flood control project with detention ponds, reservoirs, improved channels, and levees, a computer program should be able to develop subarea hydrographs, add them, and route them through reservoirs and channels. HEC-1 is rated preferable because it has several different routing techniques available to the designer, and can conduct an economic analysis of alternative designs. TR-20 is not quite as flexible, but is used by the agency (the SCS) that has built more flood control projects in Oklahoma than anyone else (almost 2,000 lakes larger than ten acres surface area). HEC-1 and TR-20 are often used in design situations in conjunction with backwater (water surface profile) computer models in order to derive channel storage-discharge or discharge-area-elevation relations for the channel routings. SWMM is capable of a wider variety of uses than any other model considered in this study, including modeling stream quality, but it is ranked third for usefulness for flood control project design because it is somewhat more cumbersome to use than HEC-1 and TR-20, it is less familiar to people designing flood control systems of reservoirs, levees, and improved channels, and the computer program requires some modification to model natural channels of irregular cross sections as well as HEC-1 and TR-20 (90). The Rational Method and the USGS regression equations are ranked low for

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90. Diniz, "Modifications to SWMM," pg. 260.

this use because both are hand calculated methods only providing discharge information for inflow to a drainage system at points, with no provision for flood routing such as is in the computer models. The Rational Method does not even provide the complete hydrograph for routing the volume of a flood, but the USGS methodology does have regression equations for unit hydrographs to generate complete hydrographs for flood and reservoir routing (91).

For flood plain management applications, such as flood hazard and flood insurance studies, flood plain use studies, and studies for flood-water surface elevations for such purposes as building permits, this author prefers the USGS regression equations. They are of known reliability, being derived from flood gage records, are easy to use and quick, not being a computer model. Like the other models, it provides peak discharges at critical points on a stream for input into any of the common water surface profile (backwater) computer models, such as the Corps of Engineers HEC-2, the SCS WSP-2, or the USGS Step-Backwater E431. The only case in which that the USGS regression equations are not directly suitable, is if a reservoir, lake, or detention pond has an influence on peak discharge and a flood routing sould be performed. HEC-1 and TR-20 are both rated a close second because both can handle the routing problems, both are quite commonly used by federal agencies and consultants for that type of work, but both are computer models requiring time to set up the input data and analyze the results. SWMM and MITCAT are both capable if desired of computing water surface elevations without recourse to an external backwater model, but only for channel reaches of uniform

91. Thomas, "Flood Discharges," pg. 41.

geometry. Neither is nearly so commonly used as the three previously mentioned methods, but MITCAT was used in a Tulsa, Oklahoma study (92), and SWMM was used in a Canadian study with the Extended Transport Version, and was very expensive (93). The Rational Method is not suitable for use on basins larger than a few square miles in size (94-96).

As may be seen in Table 15, the different models are suited to different uses, but also as may be seen in the totals rating, none is far and away more useful to a variety of applications than another, and likewise none is completely useless for engineering design.

Model Use Costs and Resource Needs

The costs of using a model could be broken down into categories of direct computer cost, and salary costs to pay people to derive the raw data, such as planimetering the size of the drainage basins, and to process the data for input to computers, such as punching coding cards, if they are used. Usually salary costs far exceed direct computer costs, and the Rational Method and USGS regression equations do not even require the use of large computers. Few of the computer runs made for this study exceeded six dollars in direct computer cost on the IBM 370 at the University of Oklahoma. The outstanding exception is SWMM, which can cost

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92. Wright-McLaughlin, "Vensel Creek."

93. Eicher, "Application of SWMM."

94. Viessman, Warren, Jr., John W. Knapp, Gary L. Lewis, and Terence E. Harbaugh, Introduction to Hydrology, Second Edition, New York: Harper & Row, 1977, pg. 512.

95. Clark, Water Supply, pg. 210.

96. Design and Construction, pg. 43.

one hundred to six hundred dollars per run when using the EXTRAN Block. Outside that exception, the greatest cost is consumed by how much time it takes a person to use a model, and how much salary that person is paid, which depends on the level of training and experience that should be expected of that person in order to properly use the model.

The relative comparison of costs given in Table 16 is strictly subjective, not supported at all by any time study, and is merely the judgement of the author based on his experience in using each of the models discussed in this chapter, except MITCAT, on engineering projects.

TABLE 16

MODEL USE COSTS AND RESOURCE NEEDS

Rational Method	Low
USGS Regression Equations	Low
HEC-1	Moderate
TR-20	Moderate
MITCAT	High
SWMM	High
SWMM with EXTRAN	Very High

The Rational Method and the USGS regression equations are low-cost models to use because they require no computer time, are simple, easy to use, and can be used by a lesser paid technician after little training. This is assuming that the model will be used for what it is suited as noted in Table 15. HEC-1 and TR-20 are moderate cost because they are computer models, requiring data preparation, keypunch time, and a specially trained engineer or skilled and experienced technician to code the data and interpret the results. SWMM and MITCAT are moderate to high cost because they require more data preparation for large basins than do TR-20 and HEC-1 in order to break large basins into several subcatchments and connecting channels. For large basins, channel travel time is a large part of the basin's time of concentration, and HEC-1 and TR-20 have been used so much and calibrated so many times that there are equations and charts, such as Figures 27 and 28 to relate model parameters for hydrograph time and shape to basin physical characteristics. However, for SWMM and MITCAT, overland flow and channel flow are separate components of the models, and presently the user must break basins into enough subcatchments and connecting channels so as to get the proper interaction between overland flow time and channel flow time so as to achieve the basin's actual response time as nearly as possible. The present SWMM User's Manual recommends subdividing each basin into at least five subcatchments (97). As time passes and calibrations are reported in the literature of basins modeled as single catchments, it may be possible to find patterns, such as have been derived for unit hydrograph parameters, to adjust basin physical characteristics, such as length, Manning's roughness, and slope, for the overland flow portion of the models, so as to represent basins as single catchments and get realistic results. SWMM using the EXTRAN Block is extremely expensive due to the extremely high direct computer cost. However, it is cheaper

97. "Storm Water Management Model User's Manual Version II," U.S. Environmental Protection Agency, 1975, pg. 48.

than hand calculations using the dynamic wave equations for complete flood hydrographs and elevations at several points in complex large sewer systems.

The resources needed to use the various models are exactly parallel to the costs. The Rational Method and the USGS regression equations do not require computers, require little input data, and can be used by less skilled technicians. TR-20 and HEC-1 require computers of moderate capacity, (less than 200K bytes) more input data, and more trained personnel to use. The government and some universities conduct courses to train people how to use them. SWMM and MITCAT require larger computers (over 300K bytes), more input data as discussed in the previous paragraph, and more experienced personnel. The author is familiar with one consultant in Oklahoma who has used MITCAT, and one person in addition to himself who has made much use of SWMM in Oklahoma.

Application of the Calibration Results

The calibration results listed in Tables 7 through 12 of Chapter IV have direct application to the basins studied in this report. The rainfall loss rates presented are basin-wide for the respective models, and could be used directly by those employing the respective models on areas within the basins calibrated for this study. Indeed, the Corps of Engineers has already made extensive use of the calibrated HEC-1 loss rates for Bluff Creek (the calculations were completed about two years ago, and the author gave the Corps the results), for a Flood Plain Information Report, part of the Oklahoma City Flood Insurance Study, and some Dam Safety Inspection Reports on the Bluff Creek Basin (98). The USGS

98. "Flood Plain Information, Bluff Creek and Tributaries, Oklahoma City, Oklahoma," Tulsa District, U.S. Corps of Engineers, 1977.

used a companion computer program to the one used in this study to develop its regression equations for Oklahoma (99). The Corps of Engineers has used the HEC-1 results of this study in developing regression equations for Snyder's unit hydrograph for Central and Northeastern Oklahoma, as previously discussed in Chapter IV and presented in Figures 27 and 28 of this study. The unit hydrograph parameters developed for this study at the sample gage sites can be adjusted to derive unit hydrographs for other points in the same basins.

The times of concentration for TR-20 and Snyder's coefficients for HEC-1 cannot be transferred directly to other points in the study basins. The user must employ a weighting procedure incorporating Figures 27 and 28 of this study and Figures 3-3, 3-4 and 3-5 of Reference 84. For a location other than one in this report, use Figures 27 and 28 of this study or Reference 100 to get unit hydrograph parameters, then adjust them by factors weighted by the ratio of the size of the new basin to the size basin at the location of this report. For example, if the user wants Snyder's lag time for a point 75 percent of the distance between the upper end of the basin and one of gage sites used in this study, and that point drains 65 percent of the drainage area of the gage site, then get a chart lag time from Figure 27, multiply the gage's lag time from Table 10 by 75 percent, and add 35 percent of that product to 65 percent of the chart lag time. The sum is the lag time to use for the new point.

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99. Thomas, "Flood Discharges," pp. 26-27.

100. "Urban Hydrology - TR-55."

Application of Hydrograph Parameter Comparisons

In Chapter III, several graphs and equations were presented which relate various parameters of the different unit hydrograph theories. That information can be used to convert from one unit hydrograph to another. For example, suppose the user is working with a basin and derives its Clark's coefficients, time of concentration t_c, and storage coefficient k_s, from the USGS regression equations 34 and 33 in Reference 101, but he wishes to use the SCS computer model TR-20 to model the basin. Then he can use Equation (18a) or Figure 20 of this report to get the SCS unit hydrograph shape factor K, and multiply the USGS time of concentration by 1.27 (see page 55 of this report) to get the SCS time of concentration to input to TR-20.

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101. Thomas, "Flood Discharges," pg. 41.

CHAPTER VI

SUMMARY AND CONCLUSIONS

During 1974 and 1975, the Water Resources Division of the U.S. Geological Survey operated three recording flood gages with rain gages on urban basins in Oklahoma City. Twenty-three of the largest floods were used to calibrate six discrete event urban rainfall-runoff models used by federal government agencies, the models being the Rational Method (Department of Housing and Urban Development), TR-20 (Soil Conservation Service), G824 (Geological Survey), HEC-1 (U.S. Corps of Engineers), SWMM (Environmental Protection Agency), and MINICAT (an old version of MITCAT, and used by the River Forecast Center). Table 5 summarized the mathematical foundations of the models as they were discussed in Chapter III. The calibration processes gave rainfall loss rates and hydrograph shapes that are similar to those reported by other researchers for urban basins, with the exception of Bluff Creek, which had a flatter unit hydrograph than typical, probably due to a series of ponds that attenuate the floods upstream of the gage site.

The Geological Survey had also collected data on daily pan evaporation and daily rainfall to use to determine the impact of antecedent soil wetness on runoff volume. This research found that antecedent evaporation and rainfall are not influences on the rainfall-runoff of the three urban basins analyzed in this study. That conclusion is probably due to a combination of random uncertainties in the field data, and the nature of the cover and soils of the watersheds, which are silt and clay loams with rather low infiltration rates, covered 35 to 45 percent by impervious concrete and asphalt, producing high volumes of runoff whether they are previously wetted or not.

The six calibrated models were compared three ways: how accurately they reproduce the observed flood events, engineering applications, and relationships between various hydrograph parameters. To compare how well they reproduced the observed floods, the coefficient of variation, coefficient of determination, and slope and intercept of the regression line, of observed volume of rainfall-runoff versus synthetic volume, and of observed flood peak discharge versus synthetic flood peak discharge, for each basin for each model (except the Rational Method, which does not compute runoff volume). It was found for floods on these urban basins, when properly calibrated, that HEC-1 has slightly more reliability than the others; and that the Soil Conservation Model, TR-20, tends to bias, and give big floods larger values than observed, and give lesser floods smaller values than observed.

A look at engineering applications revealed drastic differences between the models. The Rational Method and USGS regression equations are inexpensive to use (they do not require computers or highly trained technicians to use), but are applicable to entirely different types of projects from SWMM and MITCAT, which are expensive, requiring large computers and highly trained personnel to use. The Rational Method is best suited to designing the size of individual drainage culverts and

TABLE 17

TABLE FOR TRANSPOSING FROM ONE UNIT HYDROGRAPH TO ANOTHER

MODEL WHOSE UNIT HYDROGRAPH PARAMETERS ARE KNOWN (columns); MODEL WHOSE UNIT HYDROGRAPH PARAMETERS ARE TO BE COMPUTED (rows)

Model to be computed	Parameter	SOIL CONSERVATION SERVICE	GEOLOGICAL SURVEY (CLARK'S)	CORPS OF ENGINEERS (CLARK'S)
SOIL CONSERVATION SERVICE	T_c		$T_c = 1.2\ t_c$	$T_c = 1.2\ TC$
	K		$K = 590 - 229\ k_B/T_c$	$K = 427.5\ (R/TC)^{-.48}$
GEOLOGICAL SURVEY (CLARK'S)	t_c	$t_c = .8\ T_c$		$t_c = TC$
	k_B	$k_B = .5 + 267048\ T_c\ K^{-2.08}$		$k_B = R + .5$
CORPS OF ENGINEERS (CLARK'S)	TC	$TC = .8\ T_c$	$TC = t_c$	
	R	$R = 26048\ T_c\ K^{-2.08}$	$R = k_B - .5$	
CORPS OF ENGINEERS (SNYDER'S)	LAG	$LAG = .6\ T_c$	$LAG = .514\ (t_c + k_B - .5)^{.971}$	$LAG = .514\ (TC + R)^{.971}$
	CP	$CP = .319\left[\frac{1}{\left(\frac{K}{427.5}\right)^{2.08} + 1}\right]^{-.568}$	$CP = .319\left(\frac{k_B - .5}{t_c + k_B - .5}\right)^{-.568}$	$CP = .319\left(\frac{R}{TC + R}\right)^{-.568}$

TABLE 17

TABLE FOR TRANSPOSING FROM ONE UNIT HYDROGRAPH TO ANOTHER

MODEL WHOSE UNIT HYDROGRAPH PARAMETERS ARE KNOWN

SOIL CONSERVATION SERVICE	GEOLOGICAL SURVEY (CLARK'S)	CORPS OF ENGINEERS (CLARK'S)	CORPS OF ENGINEERS (SNYDER'S)
	$T_c = 1.2\ t_c$	$T_c = 1.2$ TC	$T_c = 1.67$ LAG
	K = $590 - 229\ k_s/T_c$	K = $427.5\ (R/TC)^{-.48}$	K = $427.5\ (7.47\ CP^{1.76} - 1)^{.481}$
$c = .8\ T_c$		$t_c =$ TC	$t_c = 1.52$ LAG
$k_s =$ $67048\ T_c\ K^{-2.08}$		$k_s = R + .5$	$k_s =$ $\frac{.794\ LAG\ (CP)^{-.568}}{1 - .523\ (CP)^{-.568}} + .5$
$C = .8\ T_c$	TC $= t_c$		TC = 1.52 LAG
R = $8\ T_c\ K^{-2.08}$	$R = k_s - .5$		R = $\frac{.794\ LAG\ (CP)^{-.568}}{1 - .523\ (CP)^{-.568}}$
AG = $.6\ T_c$	LAG = $.514\ (t_c + k_s - .5)^{.971}$	LAG = $.514\ (TC + R)^{.971}$	
CP = $\left[\frac{1}{(\ .5)^{2.08} + 1}\right]^{-.568}$	CP = $.319 \left(\frac{k_s - .5}{t_c + k_s - .5}\right)^{-.568}$	CP = $.319\ (\frac{R}{TC + R})^{-.568}$	

very small storm sewer systems. The USGS regression equations are best suited for application to flood plain studies for water surface profiles and flood boundaries, for flood plain management and zoning. HEC-1 and TR-20 are best suited to design of flood control projects where the volume of the flood hydrograph is involved, such as in routing floods through reservoirs and detention ponds. SWMM and MITCAT are most useful in designing and analyzing large urban storm sewer systems.

The newest techniques, and probably the most useful results of this research, are comparisons between hydrograph parameters for some of the models, so that users may transpose from one unit hydrograph to others having the same shape. Table 17 summarizes the equations in Chapter III that give the relationships between parameters for the SCS unit hydrograph, Clark's unit hydrograph as used by the USGS, Clark's unit hydrograph as used by the Corps of Engineers, and Snyder's unit hydrograph. If a user goes through the SCS procedures to derive SCS unit hydrograph parameters for a basin, but has no access to TR-20, and wishes to use HEC-1 with Snyder's unit hydrograph parameters to simulate the basin floods, he may enter Table 17 under the column headed "Soil Conservation Service" and proceed down to the row headed "LAG" beside "Corps of Engineers (Snyder's)" to find the equation to solve for Snyder's unit hydrograph parameter LAG, and down to the next row headed "CP" to find the equation for Snyder's parameter CP. He will then have a unit hydrograph described in Snyder's parameters, and very nearly the same shape as the one described by the SCS parameters.

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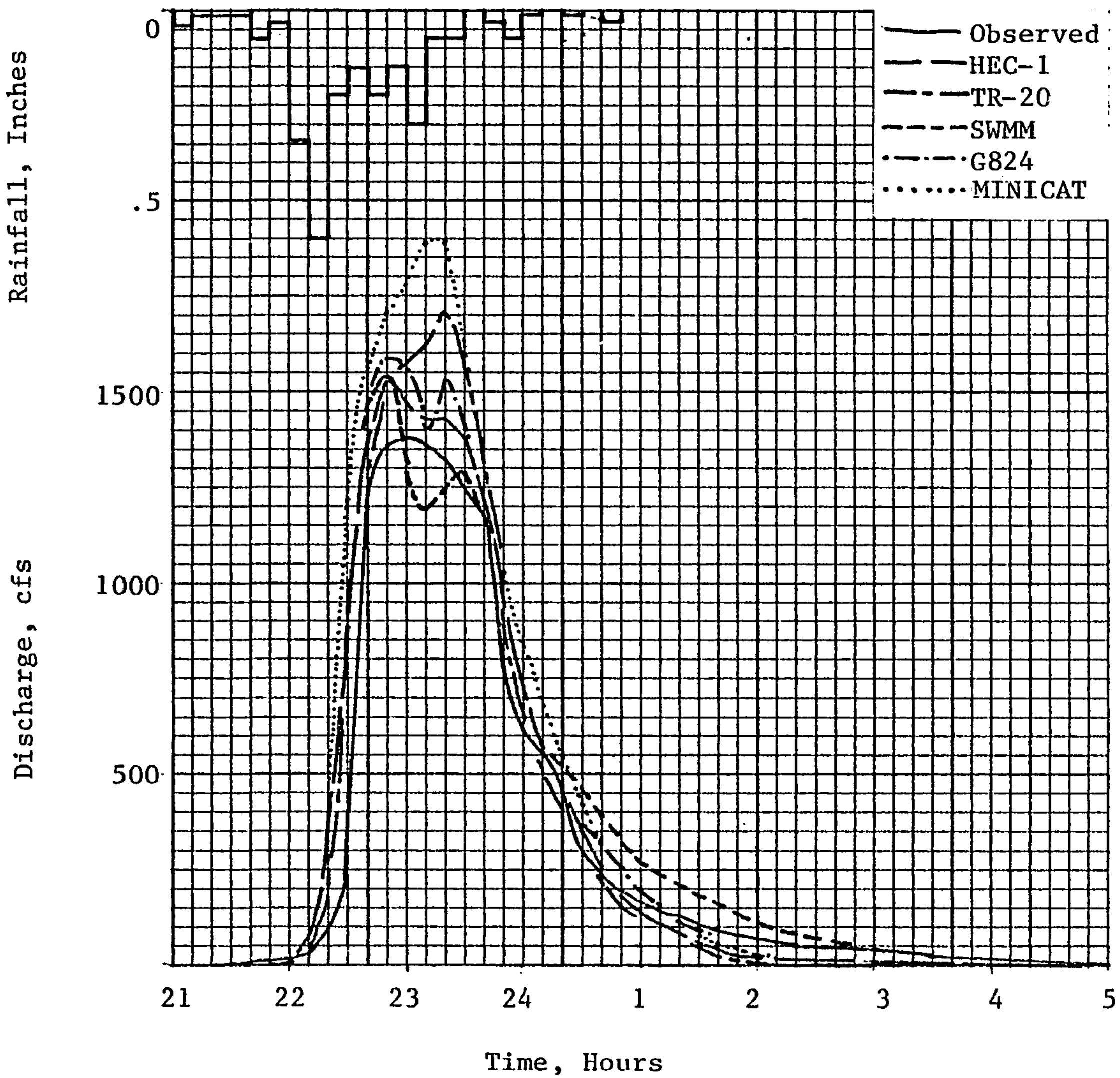
APPENDIX A

The following computer program was used to derive the data for Figure 20.

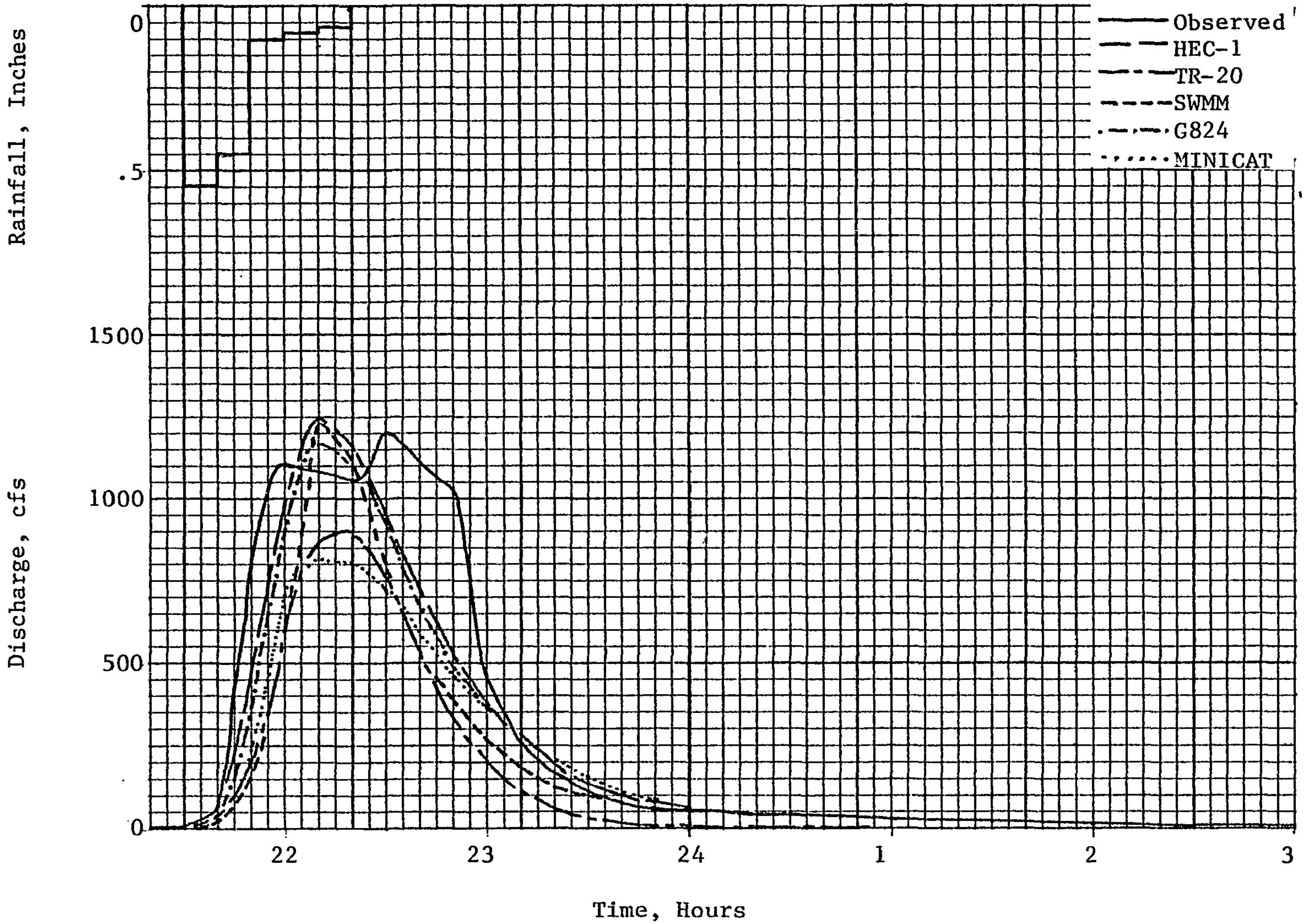
```
C       PROGRAM BY KEITH WILLIAMS TO COMPUT UNIT HYDROGRAPHS.
C       REFERENCE: USGS RAINFALL/RUNOFF MODEL USERS MANUAL.
C       R IS THE STORAGE COEFFICIENT.
C       Q IS THE OUTFLOW HYDROGRAPH ORDINATE.
        R = .2
        DO 5 J=1,10
        WRITE (6,7) R
      7 FORMAT (//'TC=1  R=',F4.2)
        WRITE (6,8)
      8 FORMAT ('    TIME     Q     TOTAL Q')
        TIME = 0.
        TOTQ = 0.
        Q1 = 0.
        DO 4 I=1,50
        IF (TIME.GT. .5) GO TO 1
        Q2=TIME-((TIME-Q1)*EXP(-.05/R))
        GO TO 3
      1 IF (TIME. GT. 1.) GO TO 2
        Q2=1.-TIME-((1.-TIME-Q1)*EXP(-.05/R))
        GO TO 3
      2 Q2=Q1*EXP(-.05/R)
      3 Q = Q2
        TOTQ = TOTQ+Q
        Q1 = Q2
        WRITE (6,6) TIME, Q, TOTQ
      6 FORMAT (3X,F4.2,3X,F5.3,3X,F5.3)
        TIME = TIME +.05
      4 CONTINUE
        R = R+.2
      5 CONTINUE
        STOP
        END
$EXEC
```

APPENDIX B

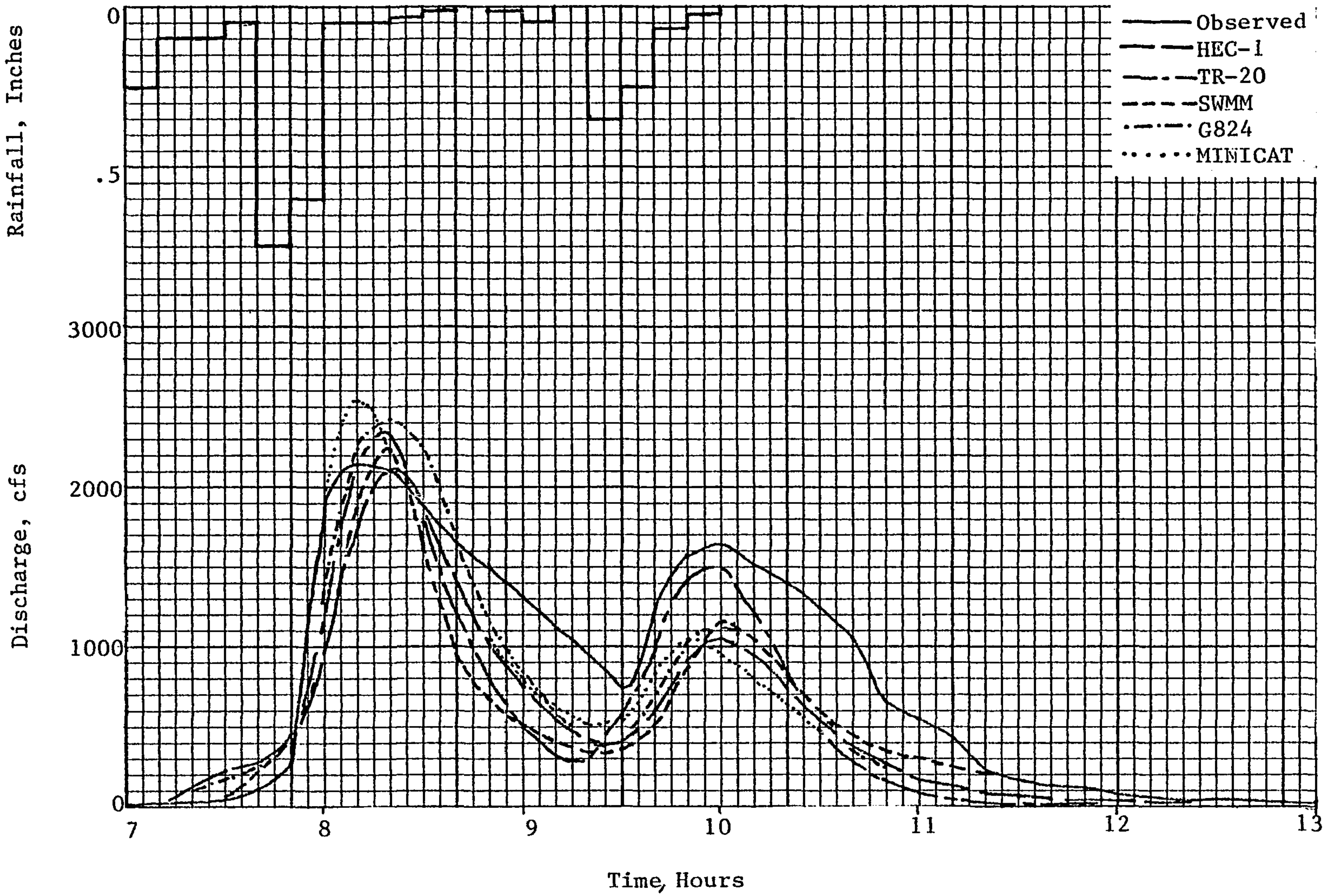
The following pages contain the recorded rainfall hyetographs and flood hydrographs as observed by the U. S. Geological Survey for the twenty-three rainfall-runoff events used in this study, and the synthetic computer hydrographs.



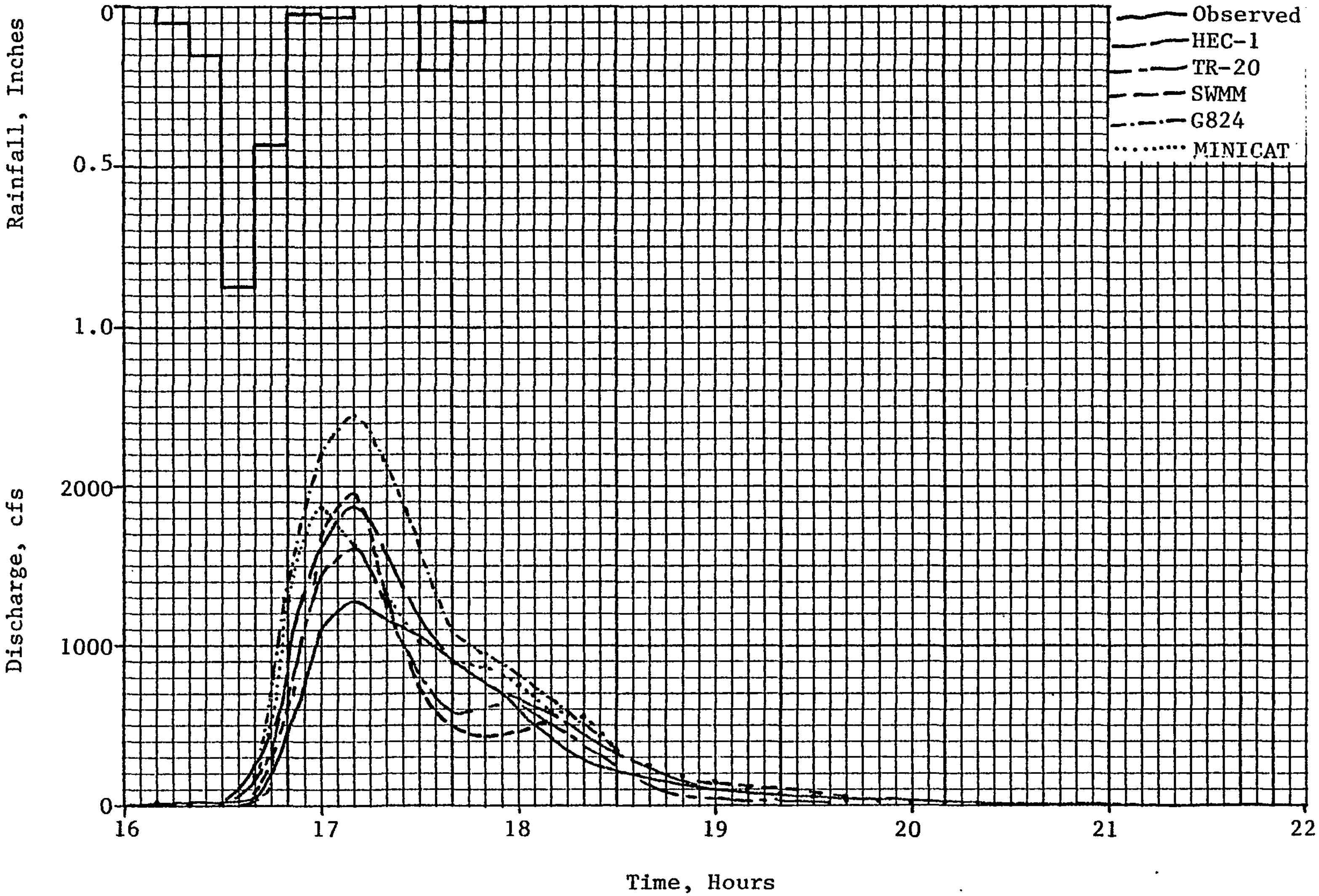
DEEP FORK CREEK AT PORTLAND AVENUE, APRIL 29, 1974



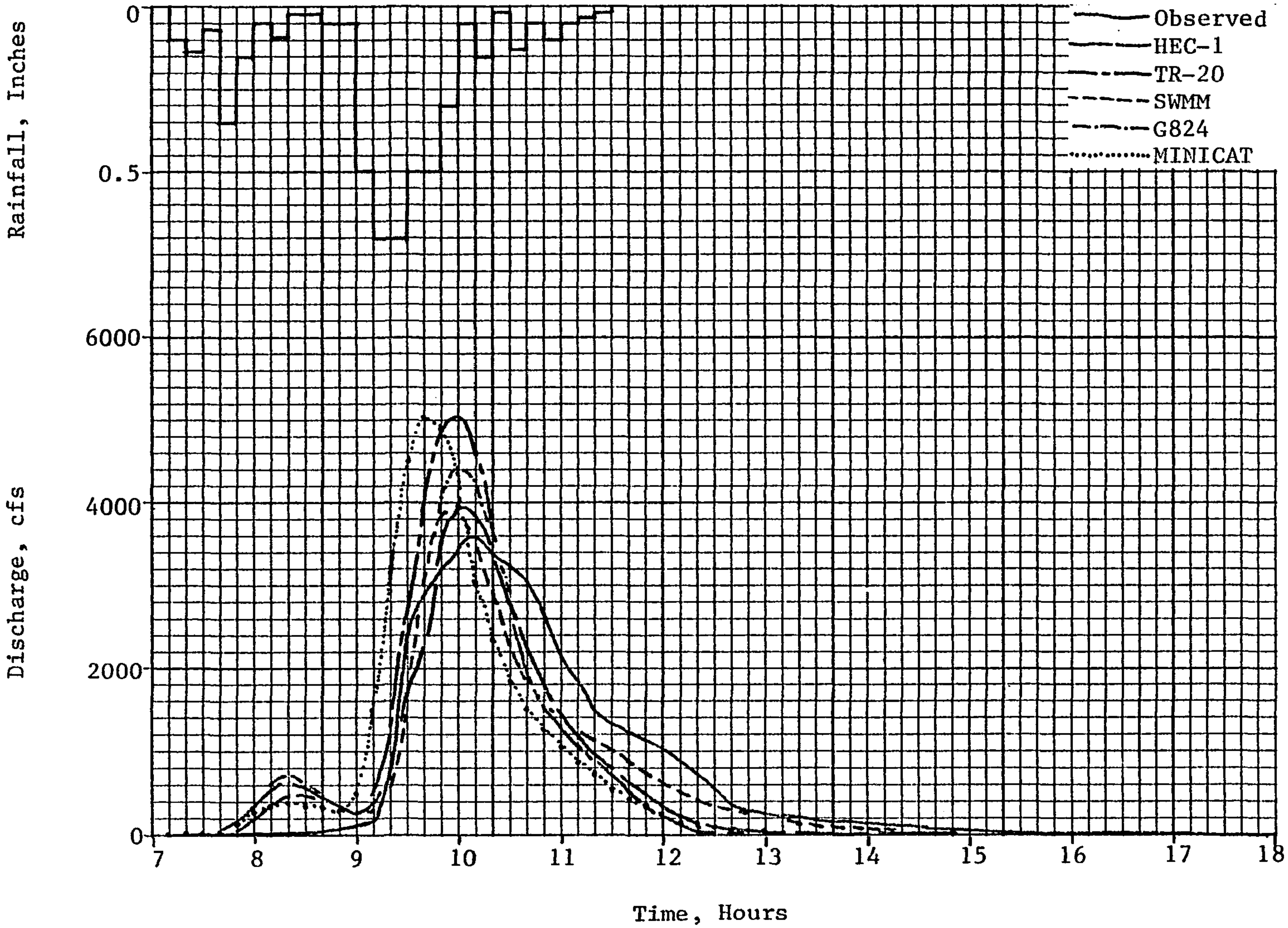
DEEP FORK CREEK AT PORTLAND AVENUE, MAY 23, 1974



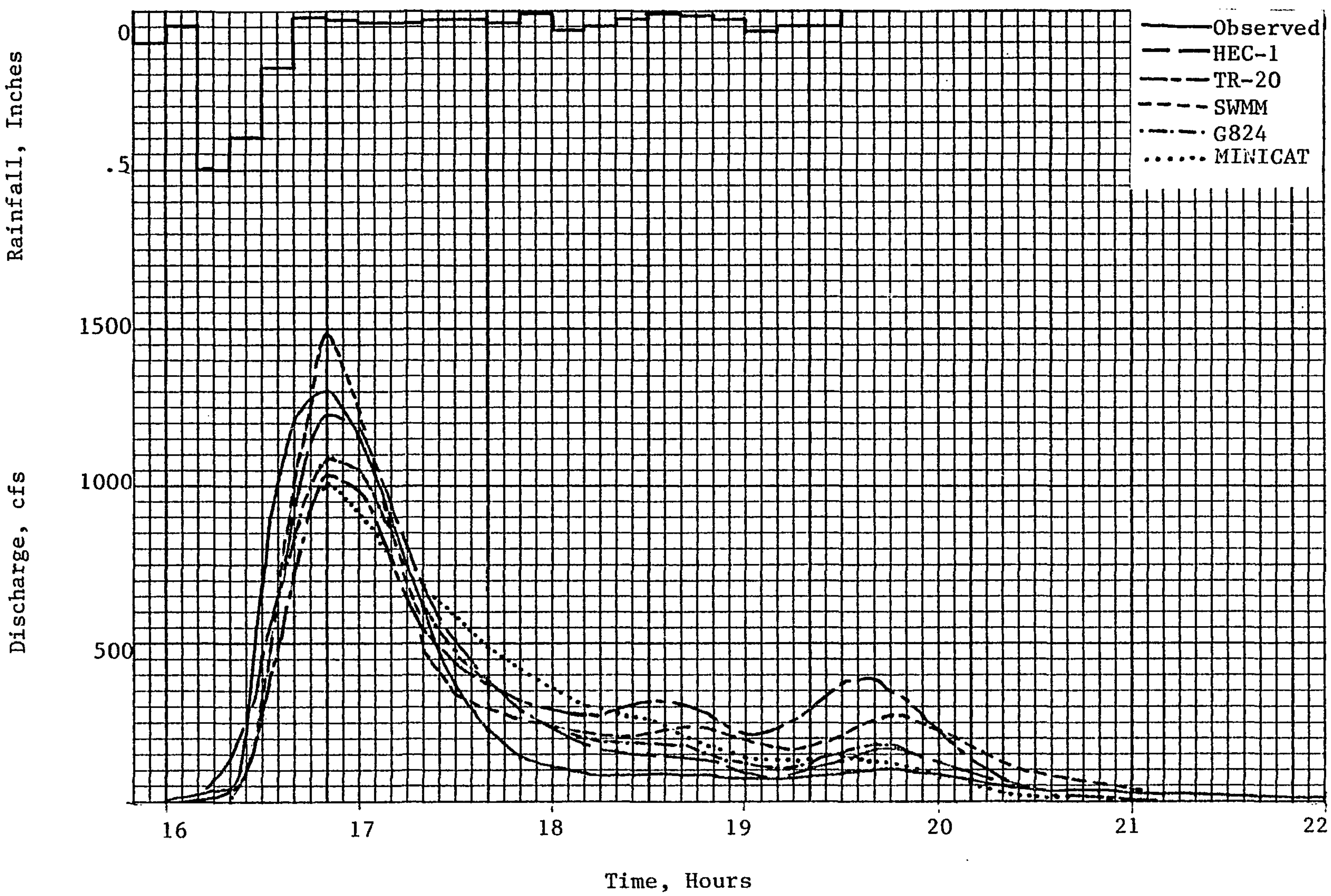
DEEP FORK CREEK AT PORTLAND AVENUE, JUNE 8, 1974 (STORM I)



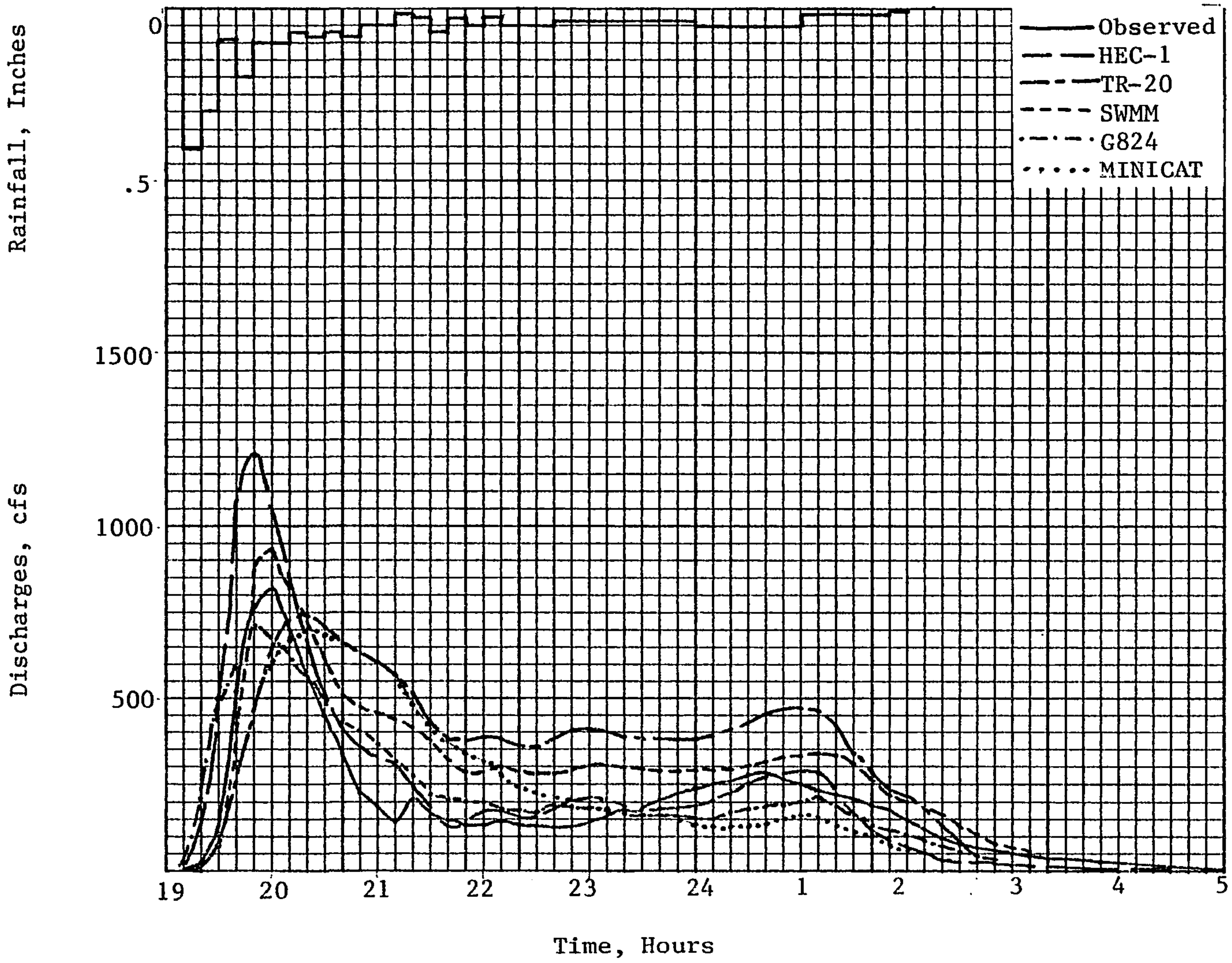
DEEP FORK CREEK AT PORTLAND AVENUE, JUNE 8, 1974 (STORM II)



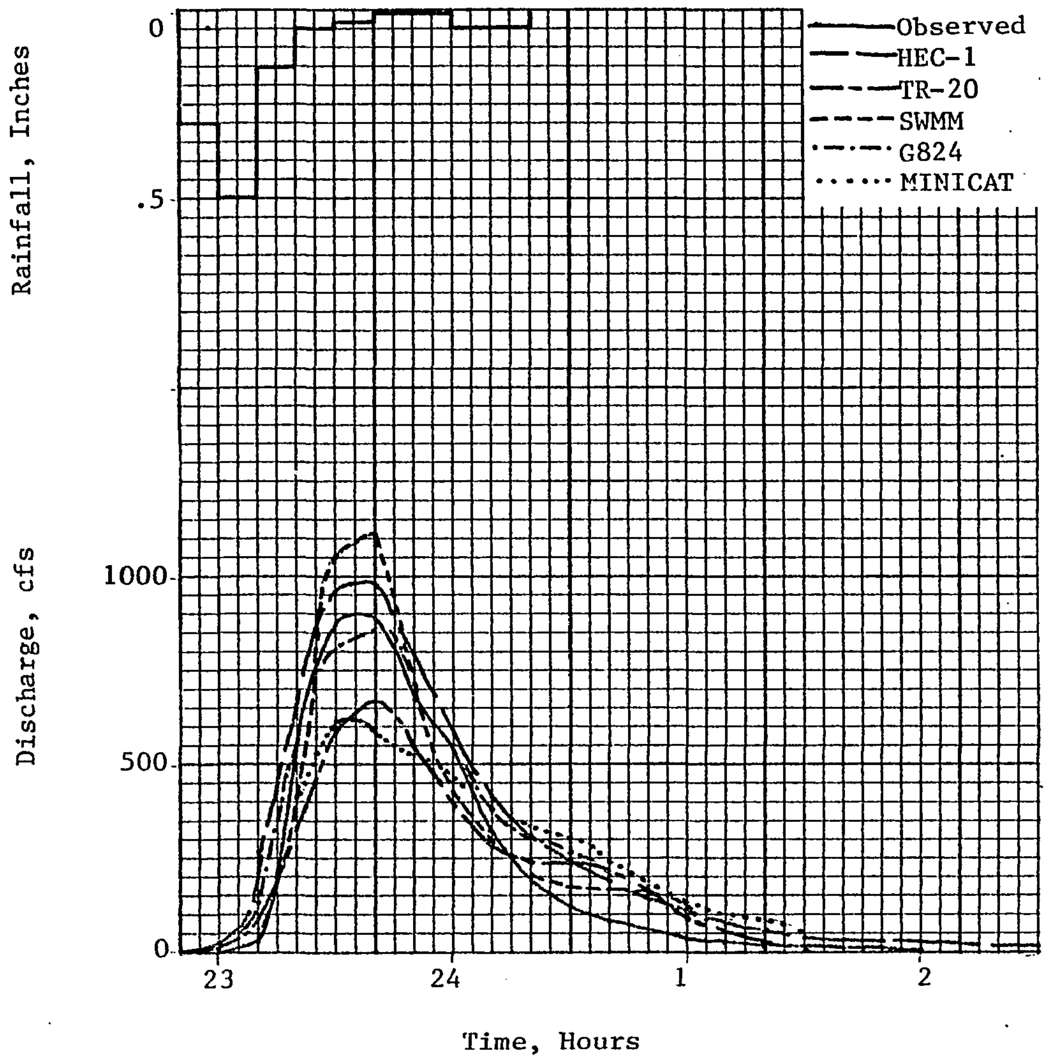
DEEP FORK CREEK AT PORTLAND AVENUE, NOVEMBER 2, 1974



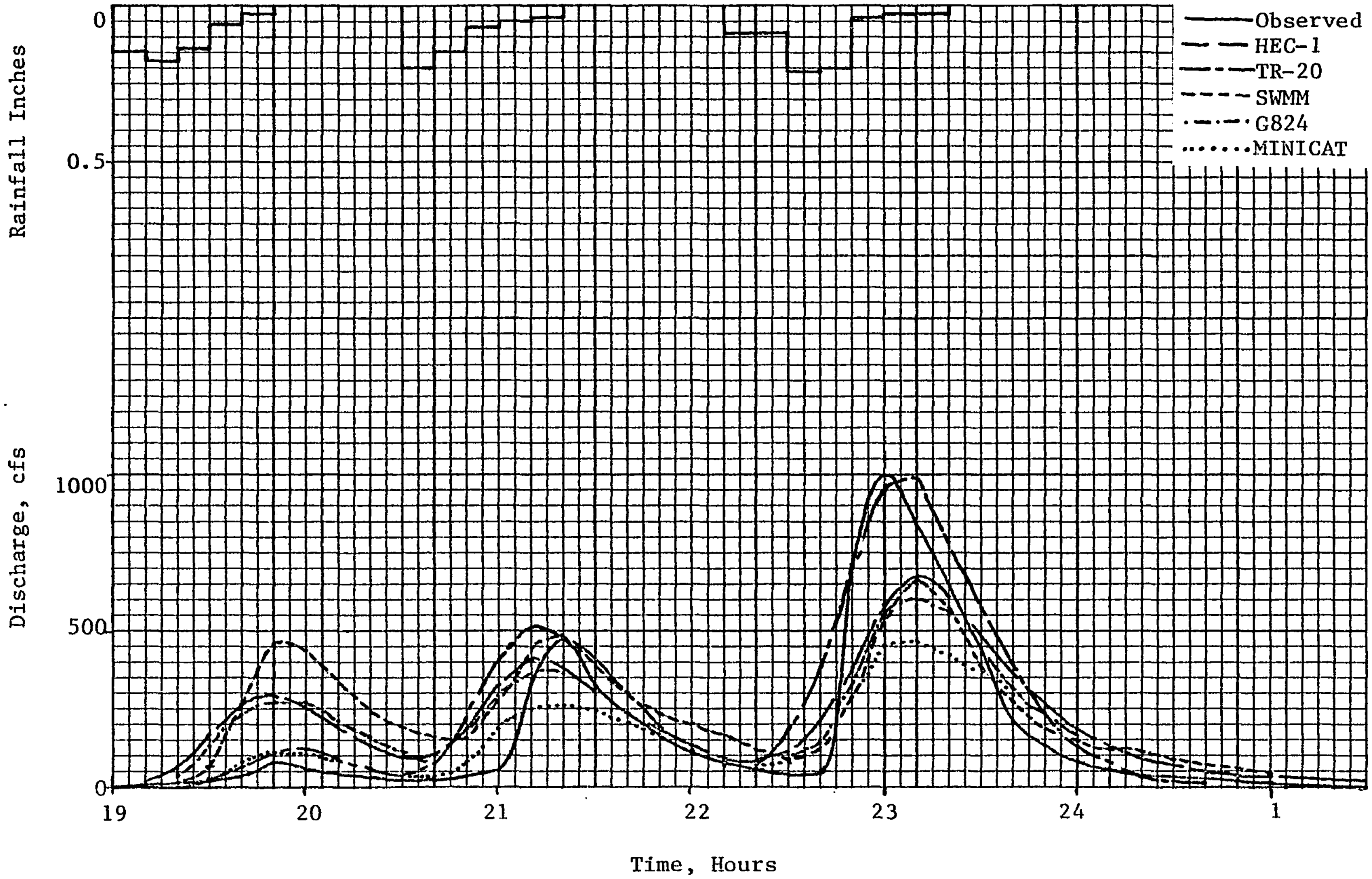
DEEP FORK CREEK AT PORTLAND AVENUE, MAY 13, 1975



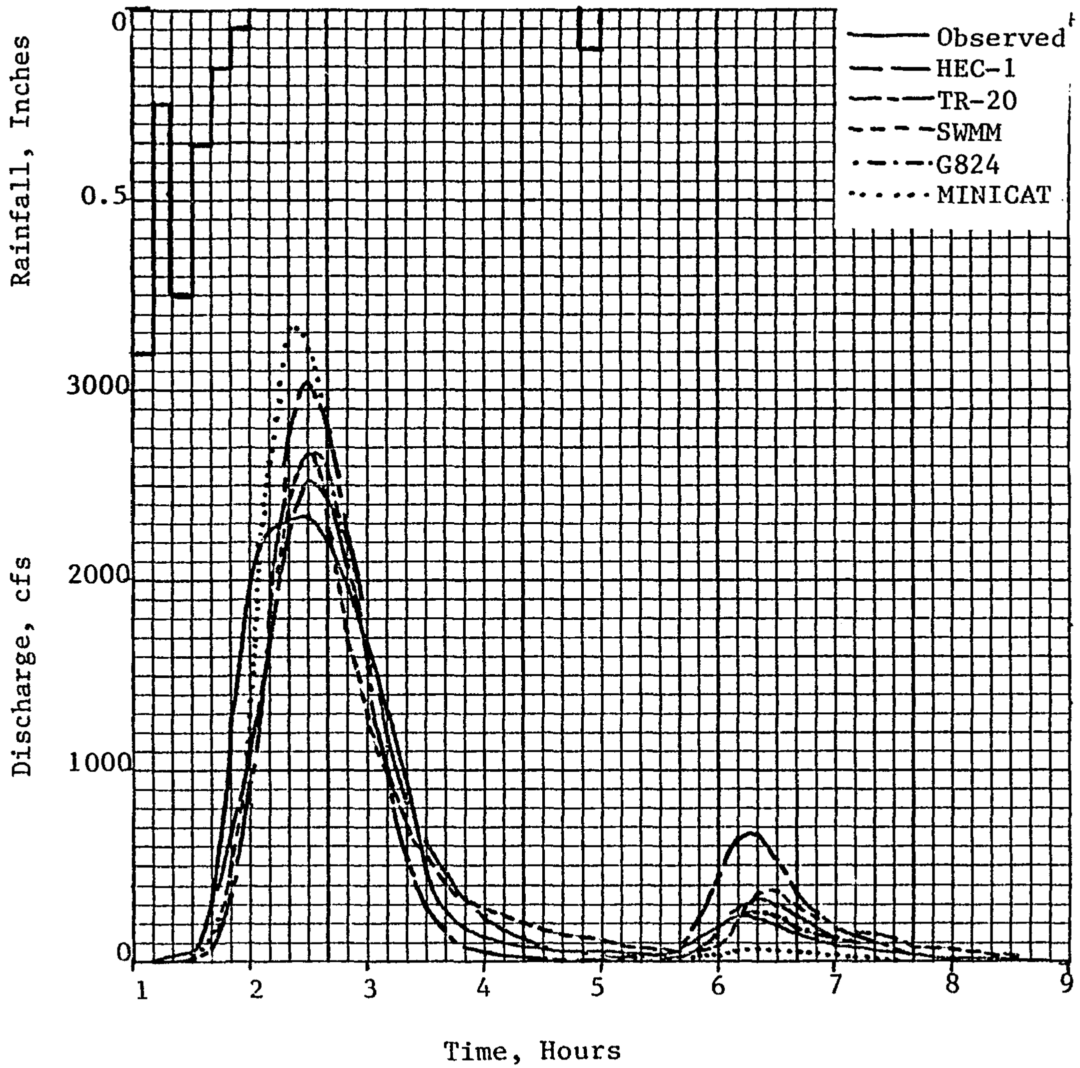
DEEP FORK CREEK AT PORTLAND AVENUE, MAY 22, 1975



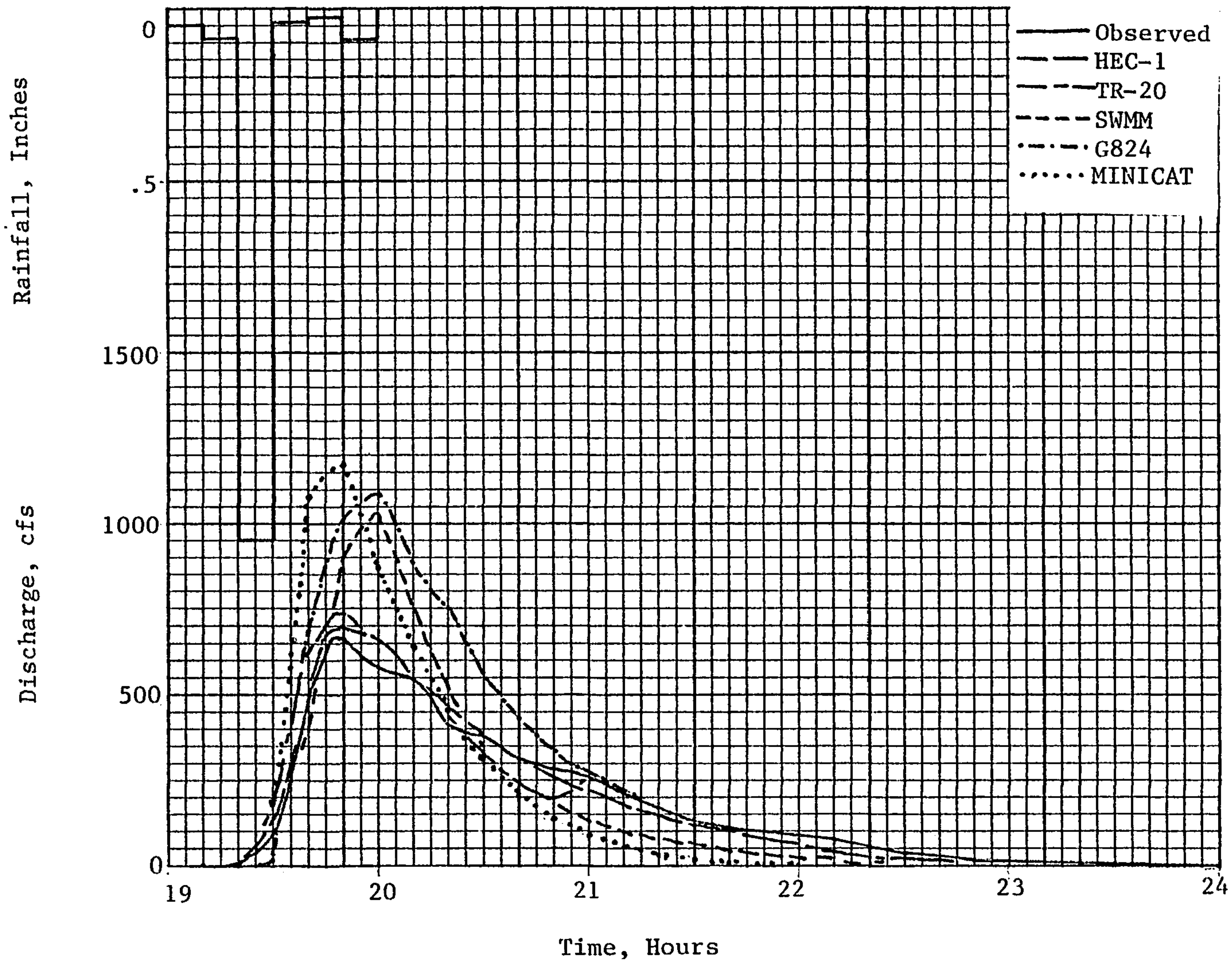
DEEP FORK CREEK AT PORTLAND AVENUE, JUNE 5, 1975



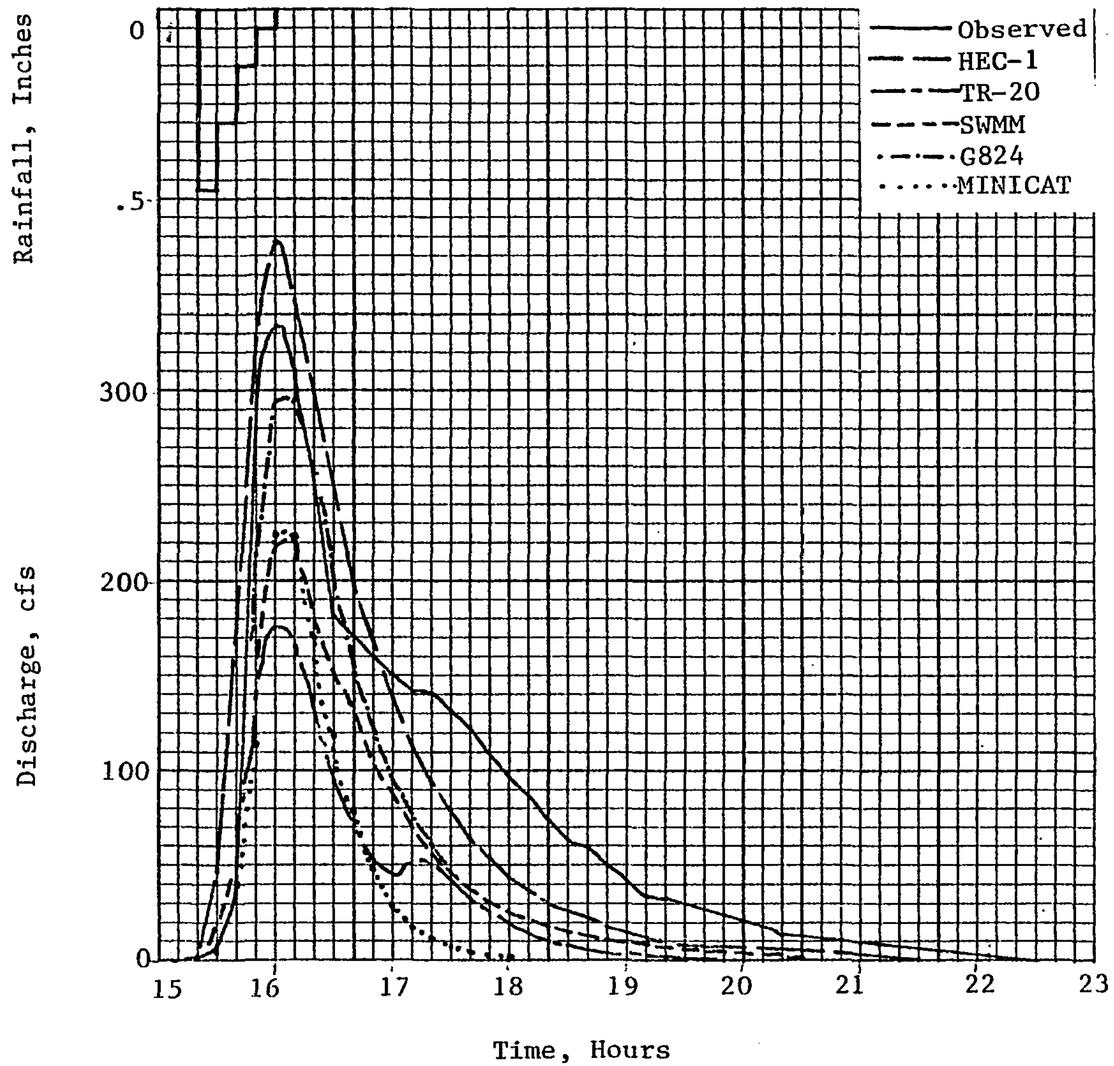
DEEP FORK CREEK AT PORTLAND AVENUE, JUNE 16, 1975



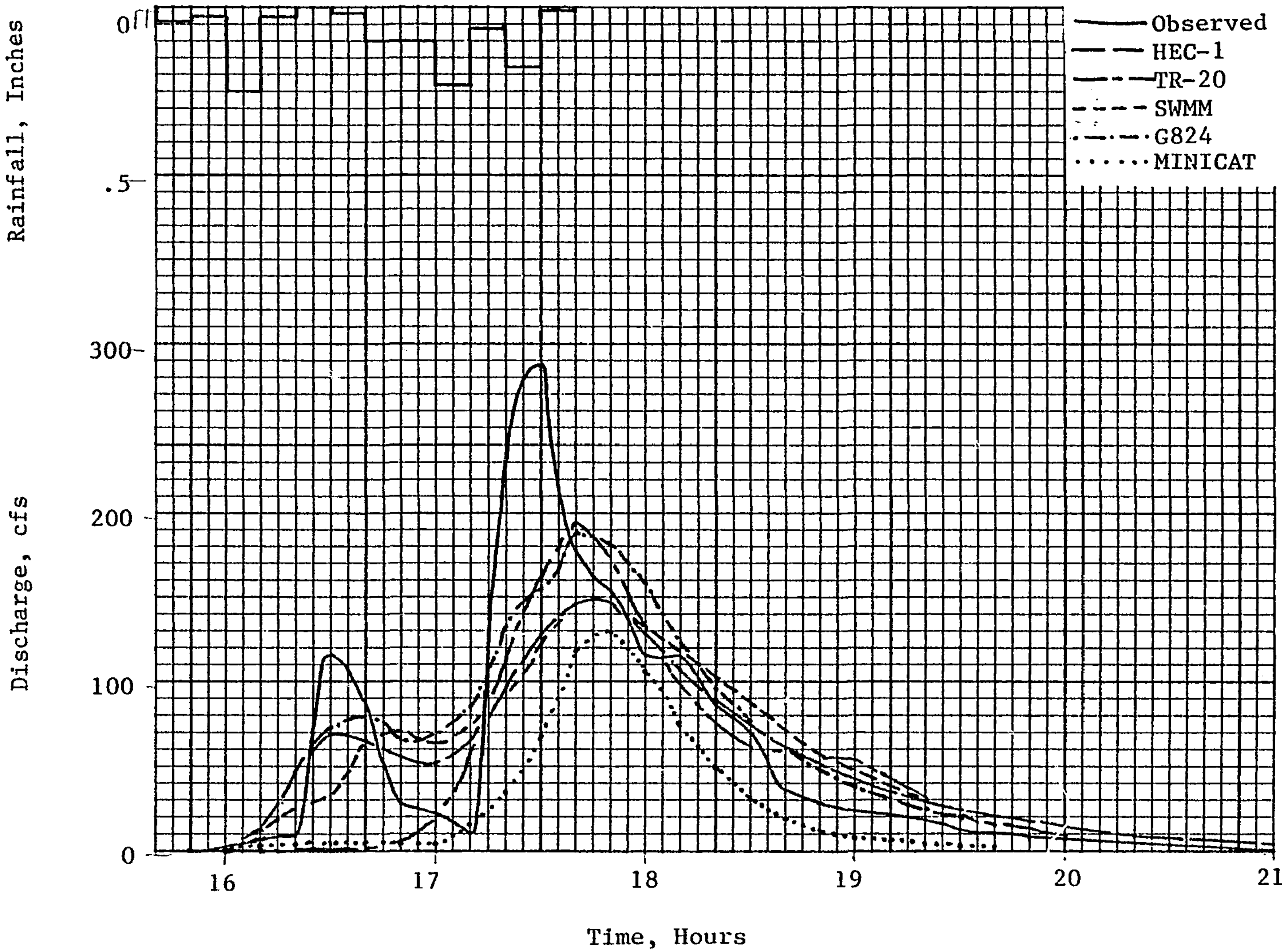
DEEP FORK CREEK AT PORTLAND AVENUE, AUGUST 14, 1975



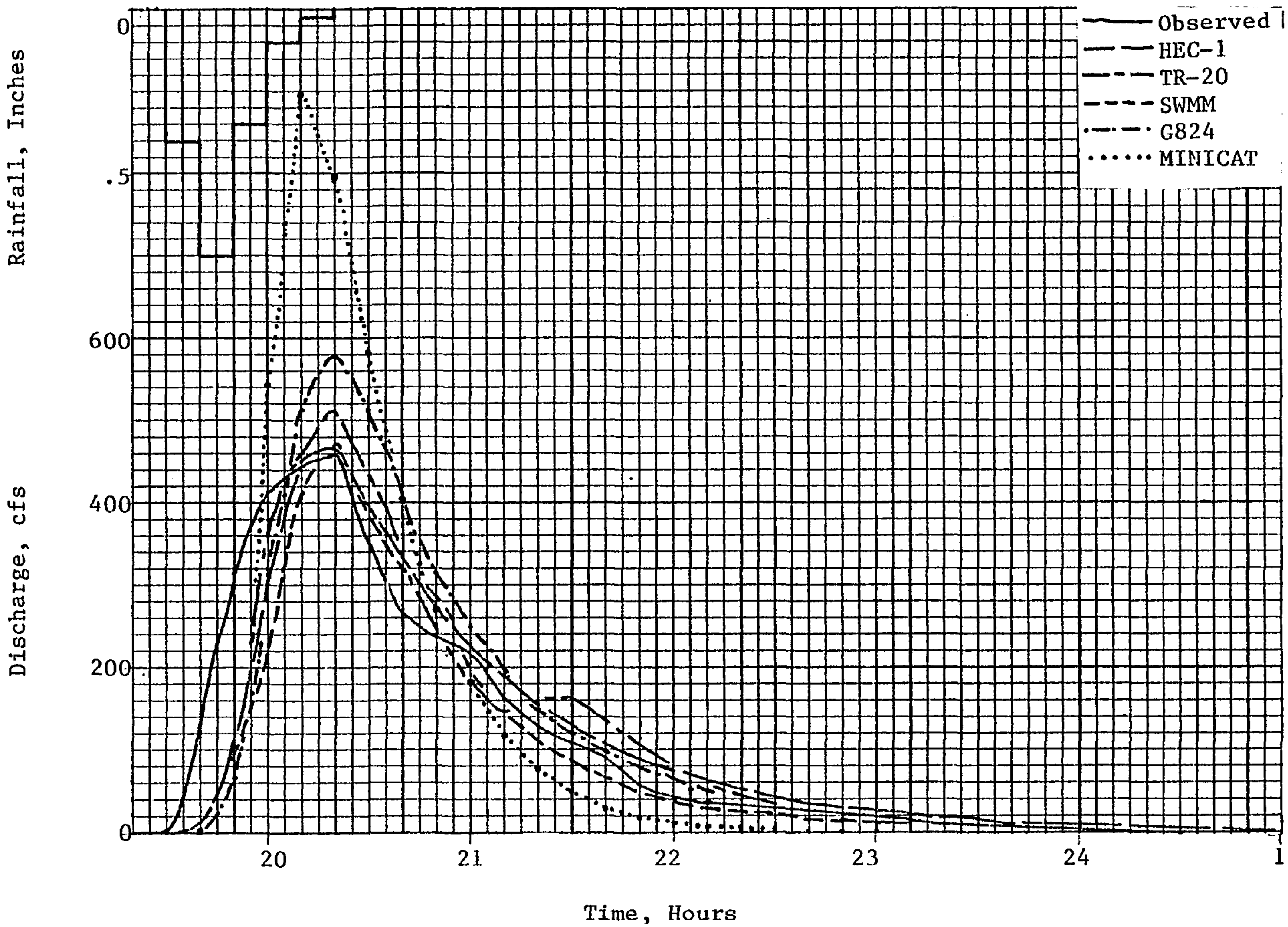
BLUFF CREEK, MARCH 8, 1974



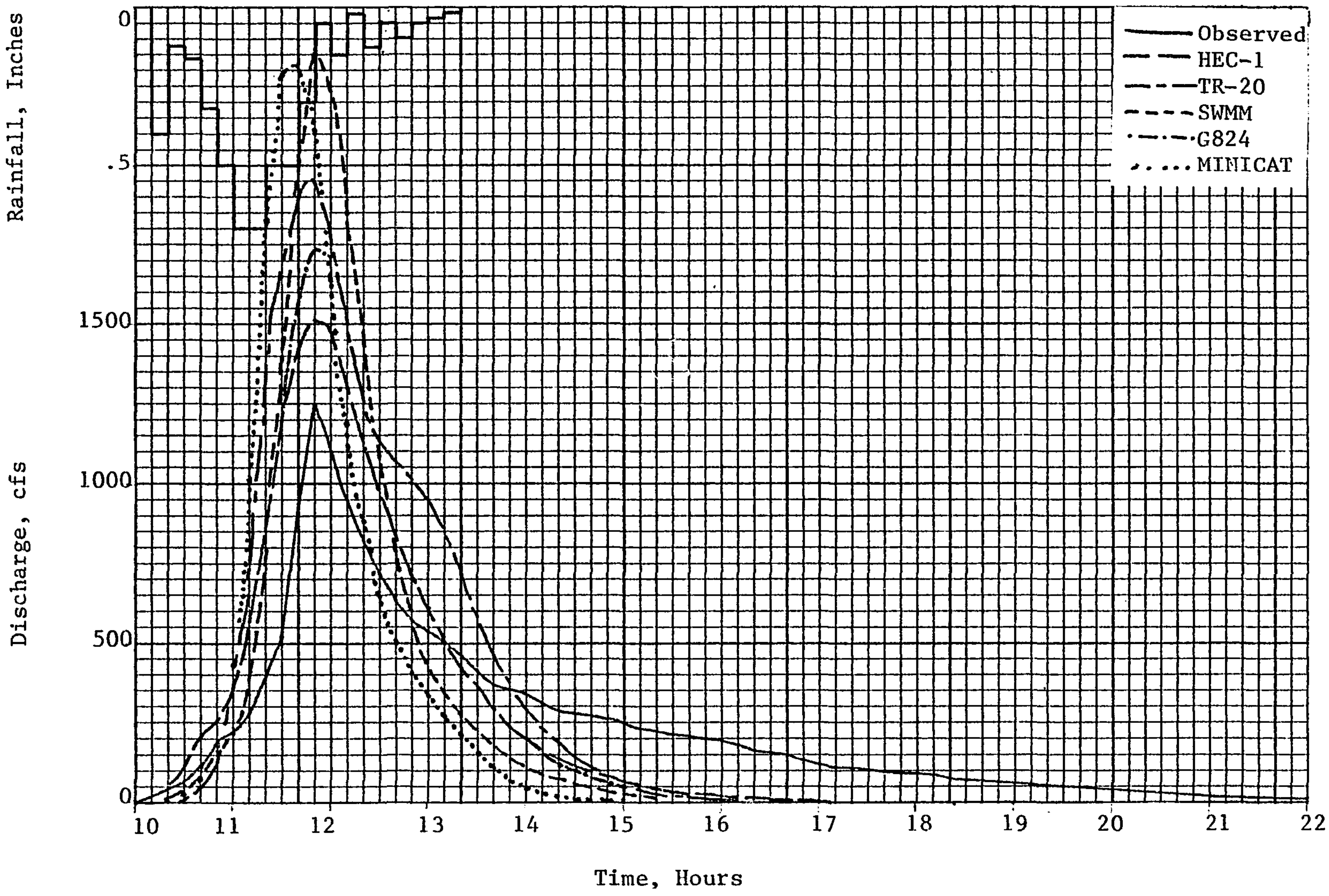
BLUFF CREEK, MARCH 10, 1974



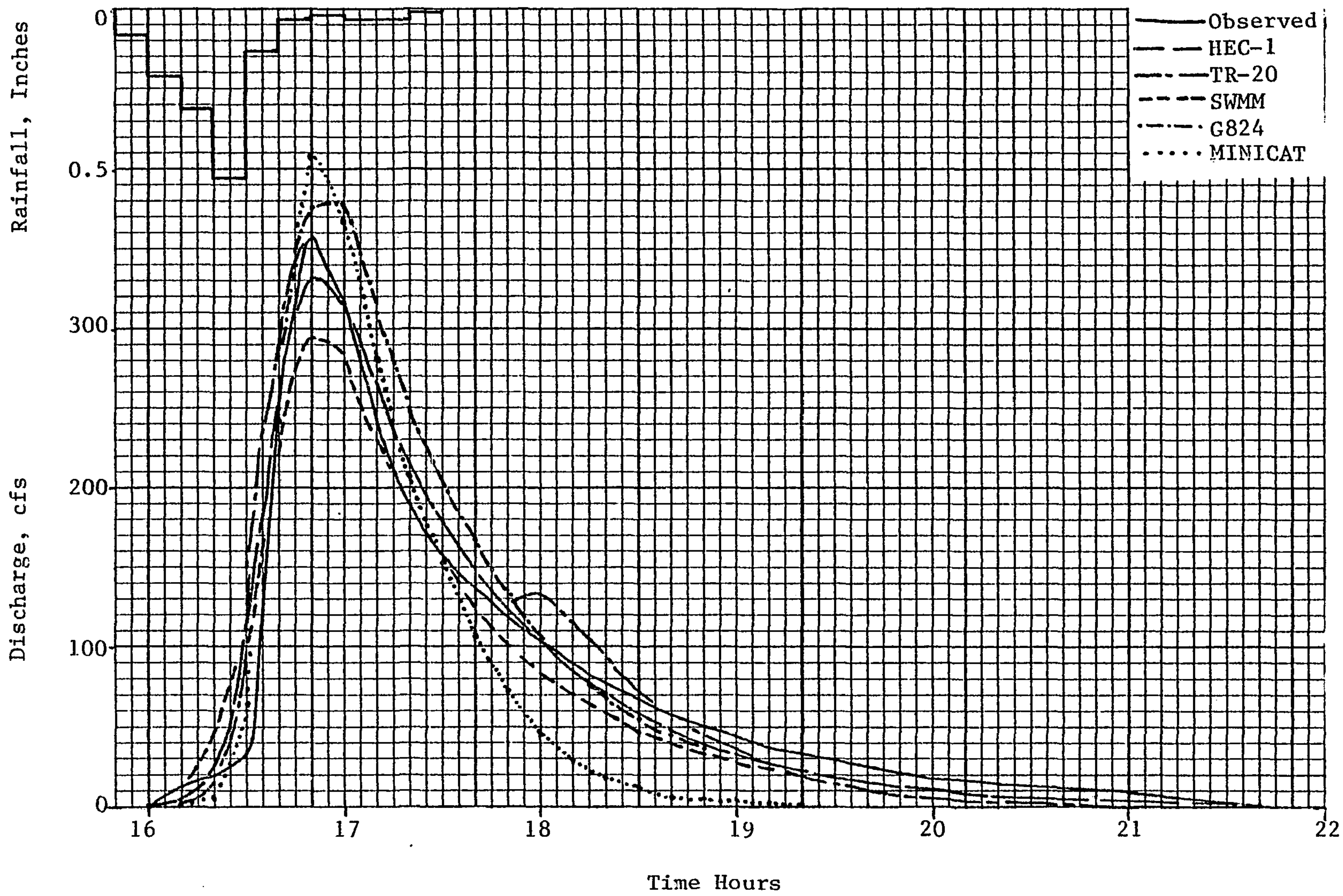
BLUFF CREEK, MAY 21, 1974



BLUFF CREEK, MAY 23, 1974



BLUFF CREEK, NOVEMBER 2, 1974



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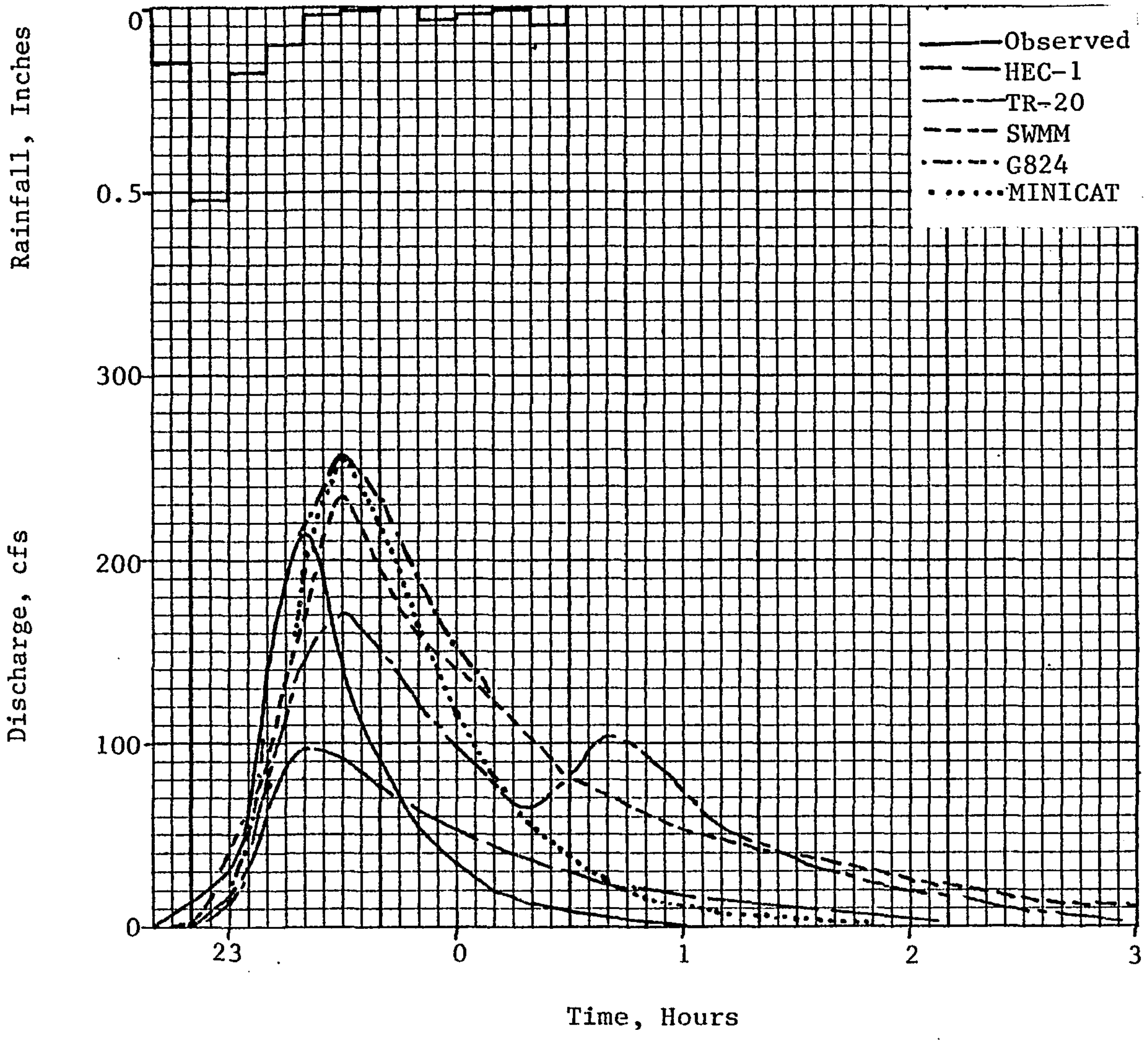

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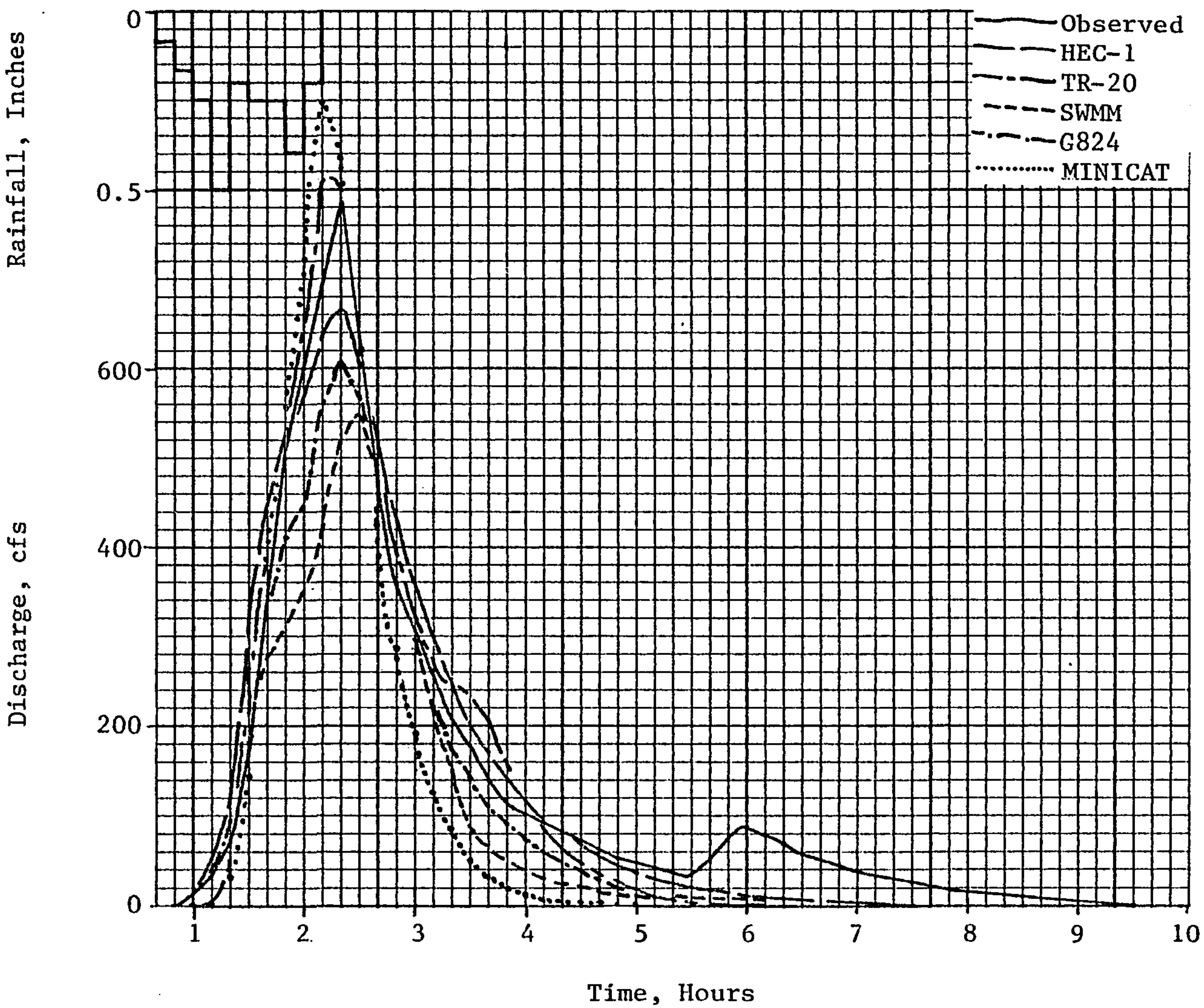

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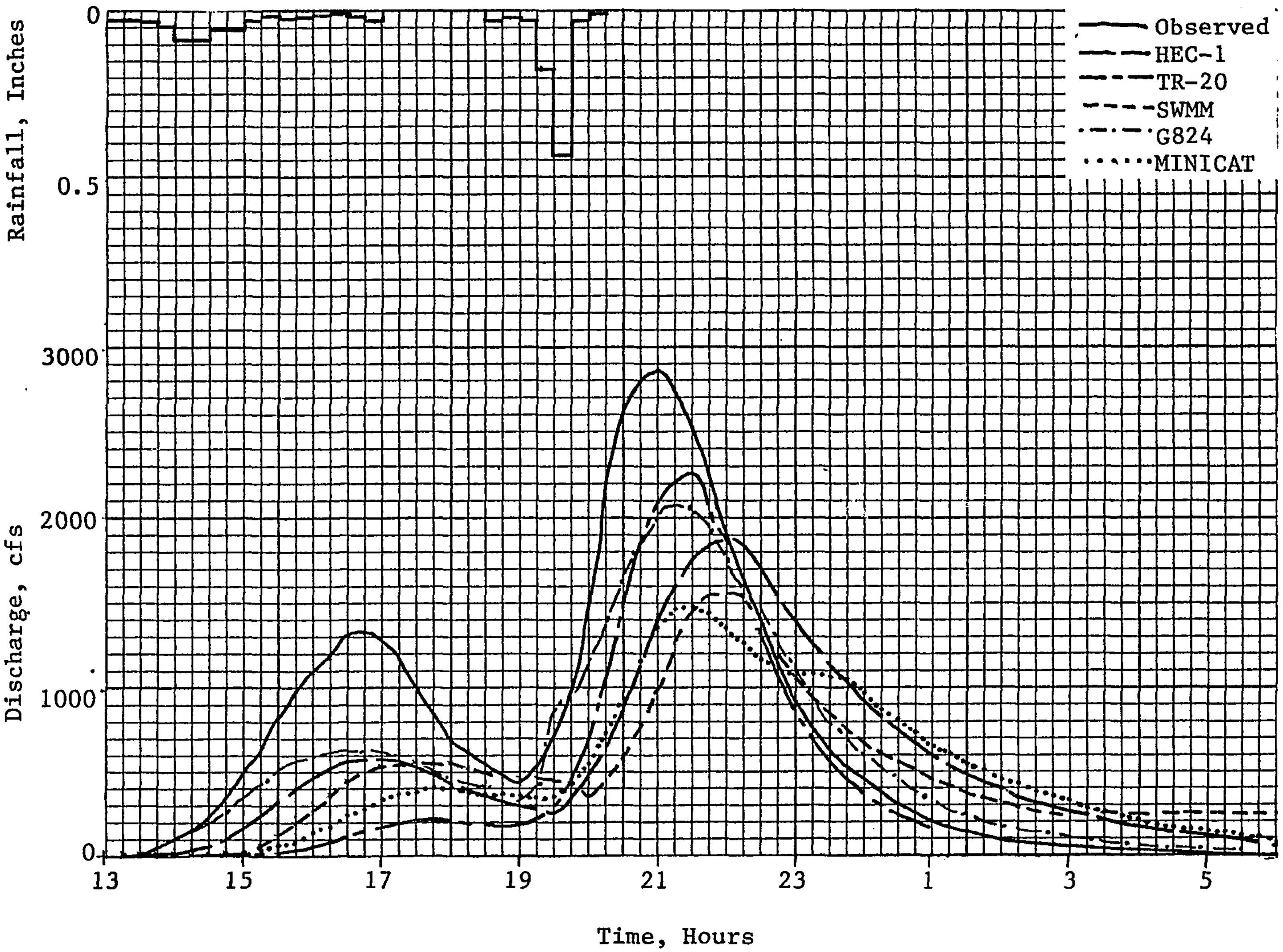
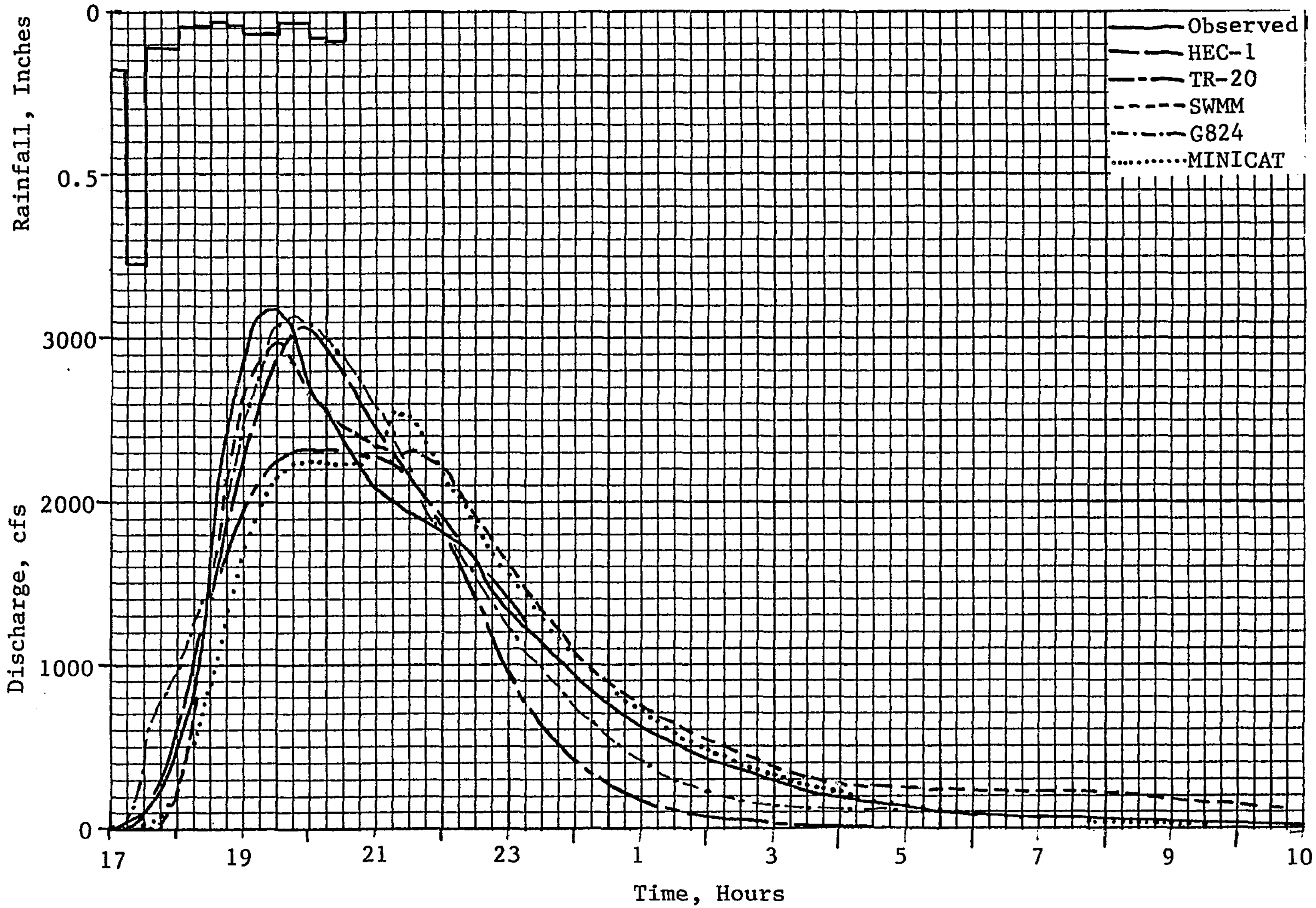


DEEP FORK CREEK AT EASTERN AVENUE, MAY 2, 1975



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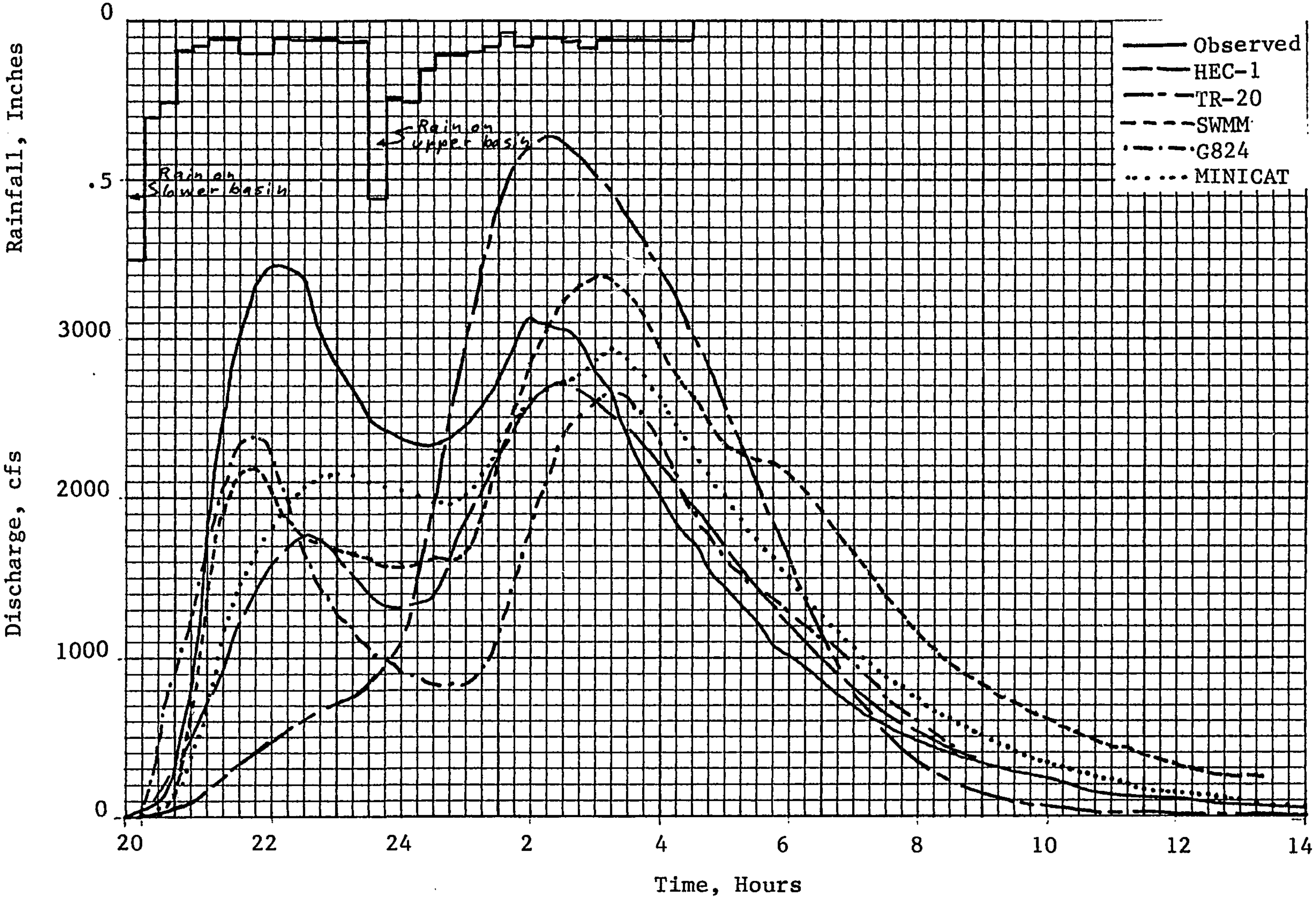
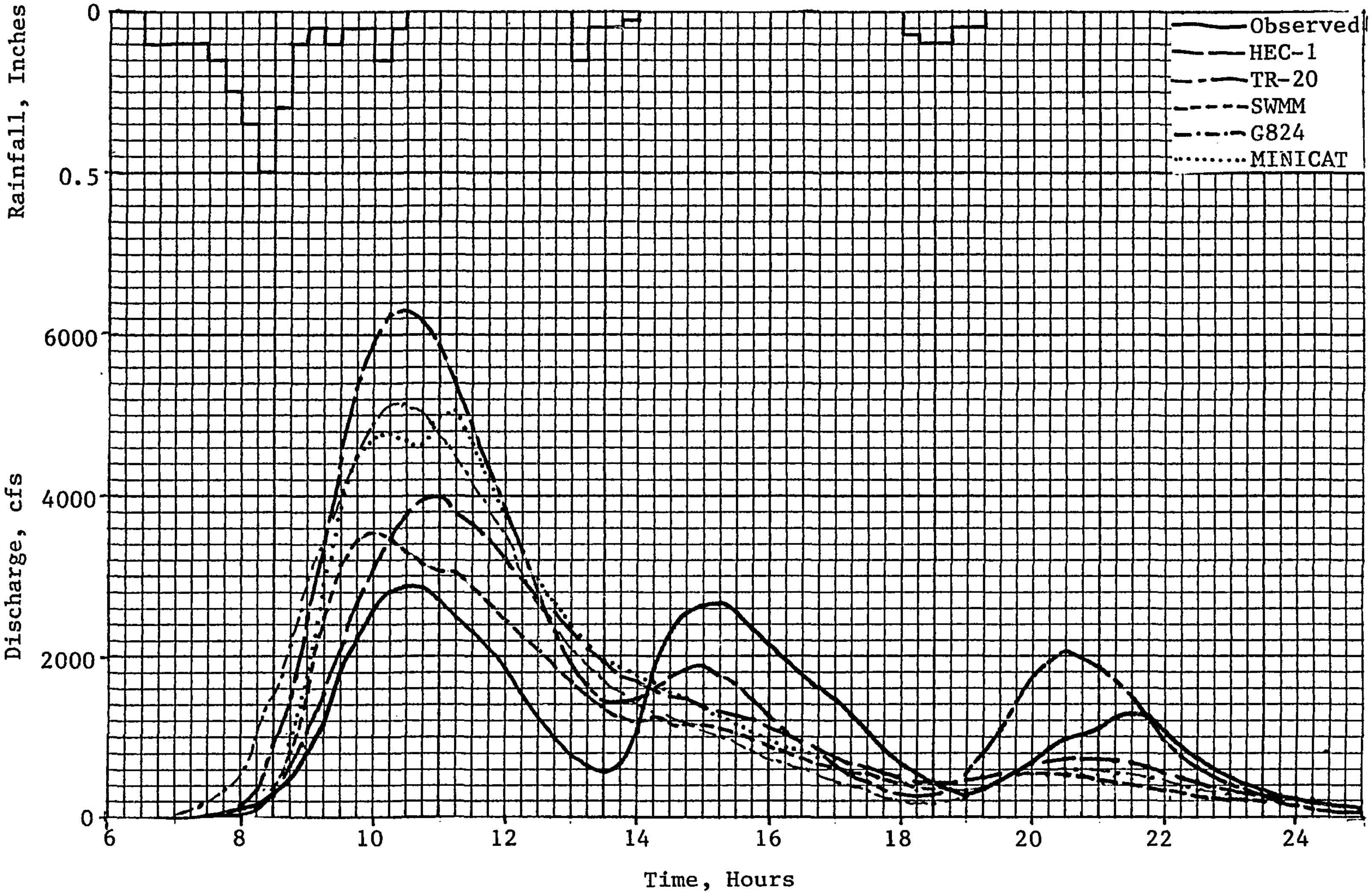


DEEP FORK CREEK AT EASTERN AVENUE, MAY 22, 1975



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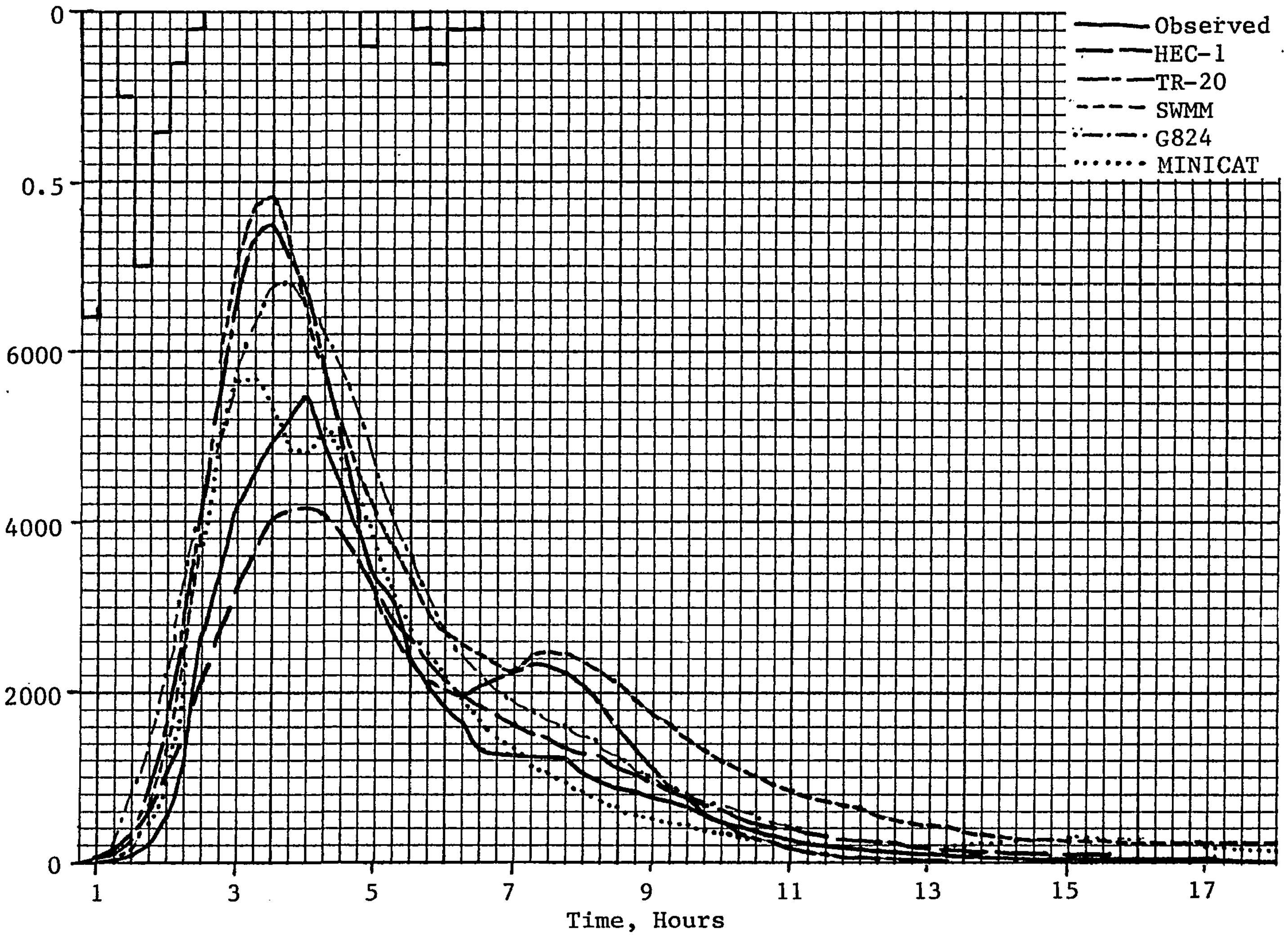

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