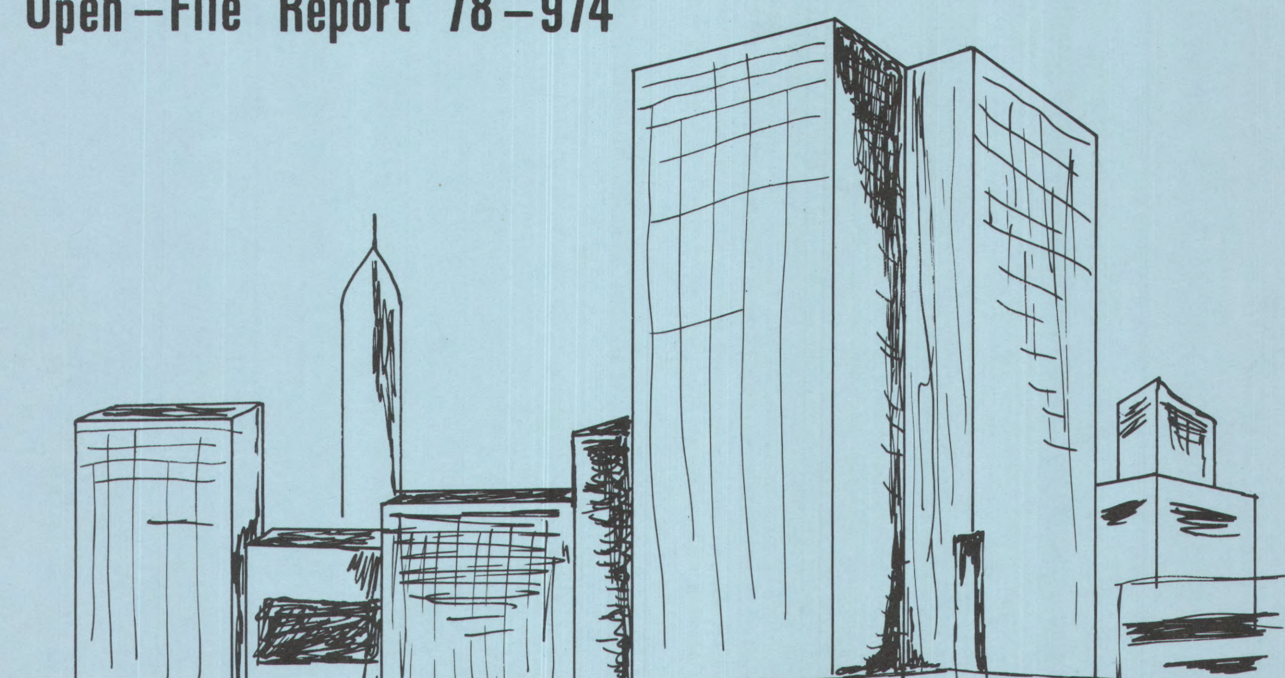
DETERMINATION OF PEAK DISCHARGE FROM RAINFALL DATA FOR URBANIZED BASINS, WICHITA, KANSAS

U.S. GEOLOGICAL SURVEY

Open-File Report 78-974



Prepared in cooperation with
the City of Wichita

UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY

DETERMINATION OF PEAK DISCHARGE FROM RAINFALL DATA FOR URBANIZED BASINS, WICHITA, KANSAS

By C. O. Peek and P. R. Jordan

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Lawrence, Kansas

October 1978

CONVERSION TABLE

The inch-pound units of measurement given in this report are listed in equivalent metric units using the following abbreviations and conversion factors:

Inch-pound unit	Multiply by	Metric unit
inch (in)	25.4	millimeter (mm)
foot (ft)	.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.509	square kilometer (km^2)
cubic foot per second (ft^3/s)	.02832	cubic meters per second (m^3/s)

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DETERMINATION OF PEAK DISCHARGE FROM RAINFALL DATA FOR URBANIZED BASINS, WICHITA, KANSAS

C. O. Peek and P. R. Jordan

ABSTRACT

Rainfall and runoff data from urbanized drainage basins in the Wichita area, Kansas, were used to evaluate the Soil Conservation Service synthetic-hydrograph method of computing flood hydrographs from rainfall data. The method was tested on six basins where the impervious surface ranged from 11 percent on the least urbanized basin to 40 percent on the most urbanized. Twenty-two of the largest storm events for which peak discharges had been observed were used in the test. After modification of the method for this particular area, results showed an average error of 20 percent, disregarding sign, with an apparent bias of 8 percent. However, uncertainties in some of the data make it impractical to adjust for bias.

Application of the modified method using data on rainfall, impervious surface, soils, land use, channel slope, length of main channel, and drainage area is described for a hypothetical basin. As an alternative to more complete and complex modeling by digital computer, a peak discharge for drainage design can be calculated by applying the SCS method to a standardized "design storm" for a specified recurrence interval. The method is sensitive to soil conditions and land use; therefore, accurate information on these factors is necessary.

INTRODUCTION

Wichita, Kansas, located in the south-central part of the State, continues to grow as other industry moves in to supplement its aircraft industry. This growth is causing increased urbanization and changes in the storm drainage. The purpose of this study is to provide peak discharges from rainfall data in small basins with varying degrees of urbanization. The peak discharges are needed for design of storm sewers, bridges, channel improvements, and for related purposes.

Wichita has had many floods in years past; the extent of the largest floods was reported by Ellis and others (1963). The city, which is located at the junction of the Little Arkansas and the Arkansas Rivers, experienced significant flooding within the corporate limits prior to the construction of the Wichita-Valley Center Floodway System, completed in the late 1950's. Other flood control and channel improvement work was performed on Chisholm Creek, Gypsum Creek, and Dry Creek. The Wichita-Valley Center Floodway System and other improvements have greatly decreased flooding from the major streams.

The continuing growth and development in the Wichita area produce significant changes in the hydrology of the small streams. As urbanization progresses, more of the area is covered with impervious surface in the form of rooftops, paved streets, paved parking lots, etc., and the quantity of runoff increases significantly from a given amount of rainfall. Heavy rainfall on impervious surfaces, coupled with the use of storm sewers, decreases the time interval between the beginning of rainfall excess and the accompanying rise in the stream channel draining the basin. As increased runoff from different parts of the basin arrives in the stream channel simultaneously, the basin's response time is shortened and higher peak discharges occur.

Anderson (1970) found that, in northern Virginia, a completely impervious surface increased the average flood-peak discharge by a factor of 2 1/2. However, an impervious surface had less effect on floods larger than the average and had an insignificant effect on the 100-year flood. Johnson and Sayre (1973) found that, in Houston, Texas, a change from a rural basin to a fully urbanized basin increased the magnitude of the 50-year flood about five times. Espey and others (1965) found that, in Austin, Texas, urbanization in a watershed produces floods with peak discharges from 100 to 300 percent greater than on an undeveloped watershed. Dempster (1974) found a much smaller effect of urbanization in Dallas than in the Austin, Texas study. The disparity of results indicates the need for basic data to define the hydrologic conditions in a particular geographic area of interest. For this reason, data have been collected at Wichita since 1964 to provide direct knowledge of the local hydrologic conditions. Through a cooperative agreement, Wichita city hydrographers and technicians accomplished nearly all the data collection.

The continuing spread of urban development in the Wichita area carries with it the problem of optimum design of storm sewers and drainage channels. Existing techniques for computation of design flows have been developed for such cities as Boston and Houston, but the techniques are not necessarily directly applicable to Wichita. In recent years numerous rainfall-runoff models have been developed for calculation by digital computer. Digital modeling will be the preferred technique when adequate data have been obtained for verification and adaptation of such a model for the Wichita area. For the present time, however, simpler methods must be chosen.

HISTORY AND DESCRIPTION OF THE DATA SYSTEM

Data collection began in 1964 at seven partial-record streamflow-gaging stations and rainfall recorders (James, 1967). The locations of these and subsequent stations are shown in figure 1 and listed, along with the periods of record, in table 1. The Dry Creek basin above Lincoln Street was extensively urbanized in 1964, and Gypsum Creek above Gilbert Street was thought to be developing rapidly. Chisholm Creek basin and Middle Fork Chisholm Creek basin were expected to develop rapidly. Big Slough was thought to be in a potential industrial area. West Branch of Chisholm Creek and Spring Creek were expected to remain essentially rural.

Data were being collected on three additional basins by 1970. The additional basins were Calfskin Tributary to Cowskin Creek at Clearwater Road, Westlink Tributary at Westfield Avenue, and Gypsum Creek at Oliver Street. The Calfskin Tributary basin was rural while Westlink Tributary at Westfield Avenue and Gypsum Creek at Oliver Street were partially urbanized and developing.

As recommended by Richards (written commun., 1971), gaging was begun in 1971 on two additional urbanized basins, Dry Creek at Pawnee Avenue and Fabrique Branch of Gypsum Creek at Harry Street. At this time, gaging was discontinued on three rural basins that indicated little tendency to become urbanized. The stations discontinued were West Branch of Chisholm Creek at 61st Street, Middle Fork of Chisholm Creek at 45th Street, and Big Slough at Ridge Road. These changes left nine basins where data were being collected.

The three most urbanized basins were Dry Creek above Lincoln Street, Dry Creek above Pawnee Avenue, and Fabrique Branch of Gypsum Creek above Harry Street. Because of the relatively short rainfall-runoff response times in these basins, it was necessary to install dual-digital recorders for rainfall and stream stage. The rainfall and stage recorders operated from a single timer, which recorded at 5-minute intervals. Operating both digital recorders from a single timer is advantageous in eliminating time discrepancies between rainfall and stream stage, but it necessitates the location of the two gages at the same site. Therefore, the rainfall is recorded only at the lower end of the basin.

In each of the rural and partially urbanized basins, equipment was installed that included a rain gage, which recorded at 15-minute intervals, near the center of the basin, and a continuous graphic recorder for stream stage at the lower end of the basin. This system was satisfactory because the basin response times were relatively long and time errors were not critical. Several non-recording rain gages, monitored by local observers, also were installed in all basins. The rainfall reported at the non-recording gages showed that the amounts recorded at the recording gages were representative of the rainfall over their respective basins. The properties of the individual basins are shown in table 2.

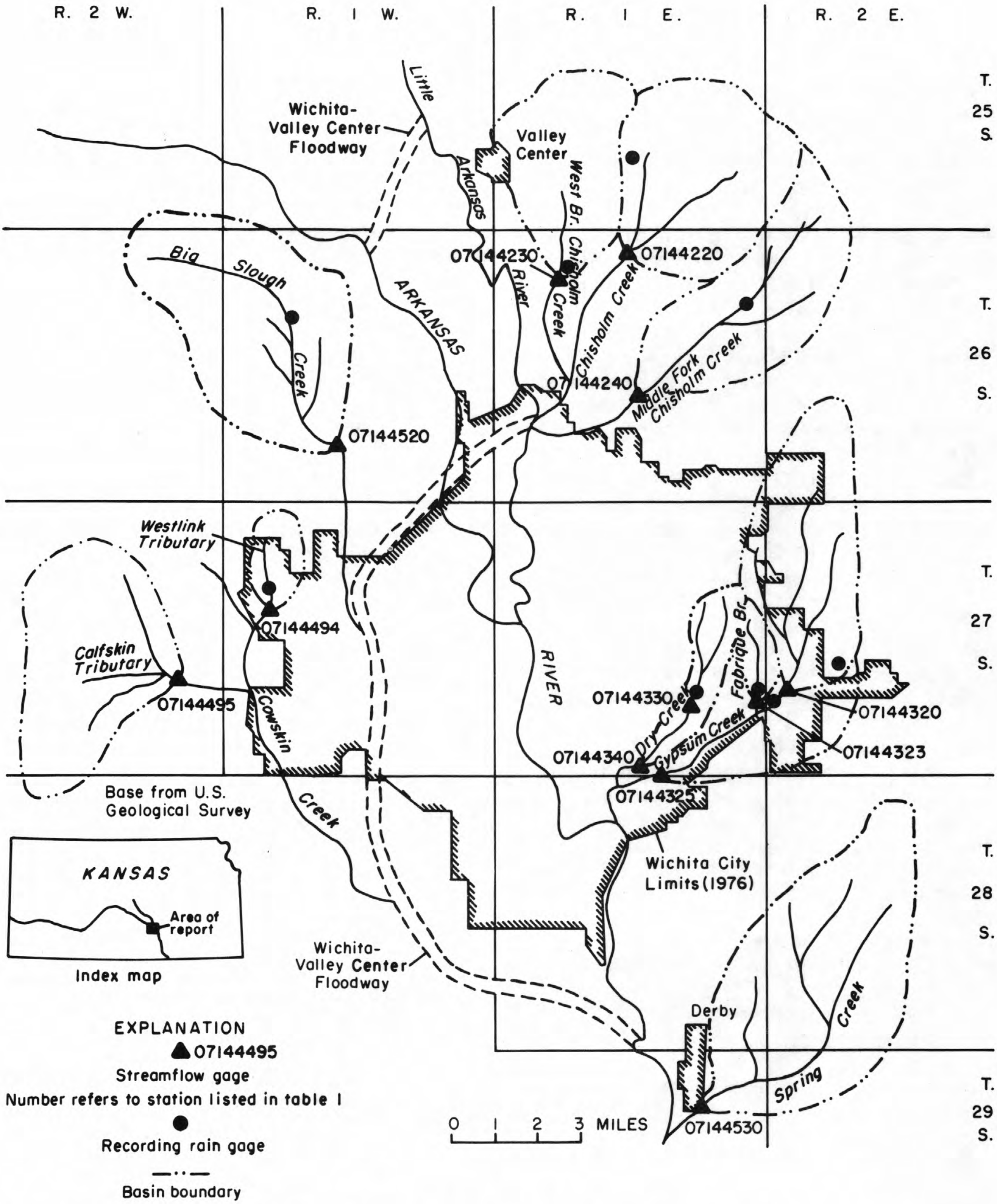

Figure 1.--Location of report area, selected drainage basins, and data-collection sites.

Table 1.--Streamflow-gaging stations and periods of record.

Station identification number	Station name	Period of record
07144220	Chisholm Creek at 69th Street	3-64 to present
07144230	West Branch Chisholm Creek at 61st Street	3-64 to 1-71
07144240	Middle Fork Chisholm Creek at 45th Street	3-64 to 1-71
07144320	Gypsum Creek at Gilbert Street	3-64 to present
07144323	Fabrique Branch of Gypsum Creek at Harry Street	3-71 to present
07144325	Gypsum Creek at Oliver Street	3-68 to present
07144330	Dry Creek at Lincoln Street	3-64 to present
07144340	Dry Creek at Pawnee Avenue	3-71 to present
07144494	Westlink Tributary at Westfield Avenue	3-68 to present
07144495	Calfskin Creek Tributary at Clearwater Road*	10-67 to present
07144520	Big Slough at Ridge Road	3-64 to 1-71
07144530	Spring Creek at Woodlawn Avenue**	10-64 to present

* No precipitation data since 10-70.
** Stage gage only.

Most of the stage gages for this study were not installed to monitor low flow but only medium-and high-flow stages. The stage-discharge relations were established by current-meter measurements because limited funding did not permit the extensive surveys and computations needed for indirect measurements. Most of the stage-discharge rating curves were only of fair accuracy owing to an insufficient number of discharge measurements, particularly at high discharges. Dry Creek at Pawnee Avenue and Fabrique Branch of Gypsum Creek at Harry Street had the least reliable rating curves because of the short period of record and the necessity of extending the rating curve.

Table 2.--Characteristics of streams studied in and near Wichita.

Station name	Drainage area (mi^2)	Length of main channel (mi)	Channel slope* (ft/mi)	Impervious area as percentage of total drainage area 1964**	1968**	1974***
Chisholm Creek at 69th Street	16.49	7.4	13.4	1.04	1.27	---
West Branch of Chisholm Creek at 61st Street	16.28	7.6	11.2	1.85	2.12	---
Middle Fork of Chisholm Creek at 45th Street	11.07	6.2	17.0	2.06	2.77	---
Gypsum Creek at Gilbert Street	8.92	5.65	16.3	9.76	11.46	12
Fabrique Branch of Gypsum Creek at Harry Street	1.14	1.26	30.7	---	36.47	40
Gypsum Creek at Oliver	16.43	8.41	13.7	---	14.70	15
Dry Creek at Lincoln Street	2.94	2.30	23.0	30.48	31.44	32
Dry Creek at Pawnee Avenue	3.86	4.29	19.1	---	29.55	31
Westlink Tributary at Westfield Avenue	3.53	3.18	10.7	---	9.72	11

Table 2.--Characteristics of streams studied in and near Wichita (concluded).

Station name	Drainage area (mi^2)	Length of main channel (mi)	Channel slope* (ft/mi)	Impervious area as percentage of total drainage area 1964**	1968**	1974***
Calfskin Tributary to Cowskin Creek at Clearwater Road	15.09	4.1	30	---	1.79	---
Big Slough at Ridge Road	20.30	13.8	3.4	0.86	1.33	---
Spring Creek at Woodlawn Blvd.	31.62	13.8	8.4	1.65	---	---

* Ratio of elevation difference to horizontal distance between points at 10 percent and 85 percent of channel length.
** Furnished by Wichita City-County Flood Control Office.
*** Estimated from aerial photos.

METHODS OF ANALYSIS

Previous investigators have used various methods of analysis to determine the effect of urbanization on high flows or to provide a method of calculating flood-frequency curves for urbanized areas. The initial method of analysis in the Wichita area was to continue gaging several streams as their drainage basins changed from mostly rural to mostly urban and to use these data in identifying the effects of urbanization on the unit hydrographs. During the data-collection phase of the project, however, the urbanization did not proceed as expected. Few of the data reflected conditions appropriate for use of the intended approach (James, 1967; Richards, written commun, 1971). Thus, the circumstances forced the consideration of alternate methods of analysis.

The alternate methods considered can be described generally as those that use (1) frequency analysis of observed peak-flow data, (2) detailed modeling of the runoff process, and (3) synthetic unit hydrographs.

Frequency analysis of observed peak-flow data has been used successfully for rural basins (Dalrymple, 1960; Jordan and Irza, 1975) where fairly long records of peak flows are available. Application of this method to the Wichita area is precluded partly by the short length of flow records available, but mainly because the available record represents a period of smaller-than-normal intense rainfall events. Figure 2 shows the deficiency of intense rainfall during 1964-76, represented by data from the Gilbert Street station, as compared with the normal (based on Herschfield, 1961) expected over a long period.

In recent years, with the availability of high-speed digital computers, most research efforts in analysis of urban runoff have been directed to detailed modeling of the runoff process. Much success has been achieved in digital modeling, but the method involves stringent requirements for data: continuity, format, and machine-readability. Data collection for this project, which began when digital modeling was in its infancy, was not designed to meet these stringent requirements. Some changes have been made in instrumentation so that, at some time in the future, calibration of a digital model may be feasible for the Wichita area. At the present time, however, application of digital modeling is impractical for the Wichita data.

Several "synthetic unit-hydrograph" methods have been developed for urban basins, but most are somewhat complex for use in routine drainage design. One method, which was developed by the U.S. Soil Conservation Service (1964; 1975; Kent, 1973), appears reasonably simple to apply, covers the essential hydrologic and hydraulic considerations, and adapts to all degrees of urbanization. Preliminary tests indicated that good results could be obtained by this method using Wichita data with simple modifications. Thus, the Soil Conservation Service (hereafter abbreviated SCS) method was tested further.

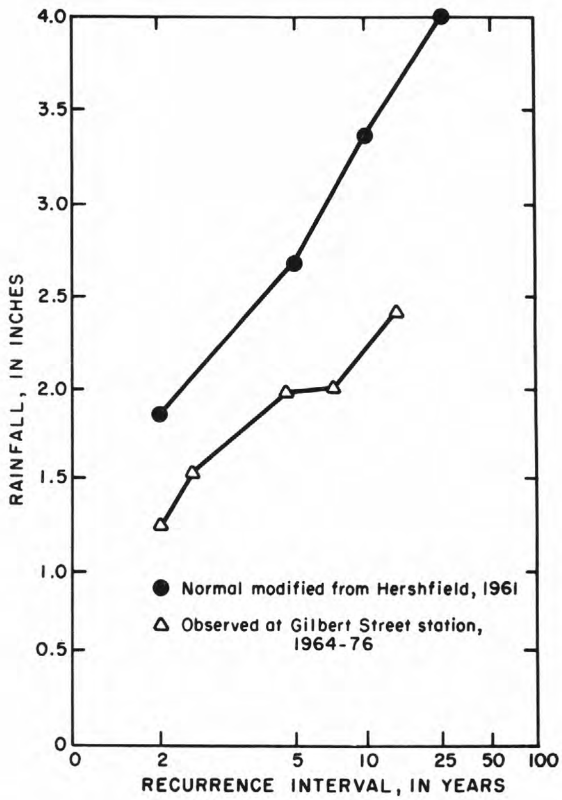

Figure 2.--Normal and observed 3-hour rainfall frequencies in the Wichita area.

TESTS OF THE SCS METHOD

The SCS hydrograph method uses a relation of runoff to rainfall and soil-cover characteristics to calculate the runoff for short time increments, then distributes each runoff increment in a triangular discharge hydrograph. The base and peak times of the hydrograph are related to characteristics of the drainage basin. Finally, the triangular hydrograph ordinates are summed to form the resulting synthetic hydrograph. Details of the method are given later in this report with a numerical example.

The calculated time distribution of discharge is governed by the lag time, which is defined by the SCS (Kent, 1973) as the time from the centroid of rainfall excess to the discharge peak. During the testing by application of the SCS method to actual rainfall events, the calculated lag times consistently exceeded the observed lag times. Examination of alternate methods of calculating lag times showed that an equation proposed by Putnam (1972) gave good results. Thus the SCS hydrograph method was modified by adopting Putnam's equation for lag time.

The modified SCS triangular-hydrograph method was tested for large rainfall events, and the resulting synthetic peak discharges were compared with observed discharges, as shown in table 3.

Table 3.--Comparison of results of triangular-hydrograph computations for largest events.

Station	Date	Observed peak flow ft^3/s	Synthetic peak flow ft^3/s	Percent error
Gypsum Creek	June 11, 12, 1970	1450	1420	-2
at Gilbert Street	May 28, 1975	1300	1830	+41
	June 2, 3, 1975	1250	1035	-17
	June 16, 17, 1975	1060	1060	0
Fabrique Branch	July 14, 1973	860	770	-10
of Gypsum Creek	May 28, 1975	1150	630	-45
at Harry Street	June 2, 3, 1975	830	900	+8
	June 17, 1976	820	393	-52
Gypsum Creek	April 20, 21, 1974	1550	1845	+19
at Oliver Street	May 9, 1974	1020	905	-11
	May 13, 14, 1974	870	895	+3
	June 16, 17, 1975	1540	1450	-6
Dry Creek	May 27, 1964	1100	1330	-21
at Lincoln Street	June 4, 5, 1965	1170	1760	+50
	May 28, 1975	960	1240	+29
	June 16, 17, 1975	1270	1040	-18
Dry Creek	July 27, 28, 1971	520	555	+7
at Pawnee Avenue	June 16, 1975	1600	1690	+6
	July 3, 1976	740	674	-9
Westlink Tributary	June 21, 1969	355	196	-44
at Westfield Avenue	October 23, 1970	640	672	+5
	June 23, 1976	195	144	-26

The average error of 20 percent, disregarding sign, shows that the estimates are reasonably accurate. The geometric mean of the ratios of synthetic to observed peak flows is 0.92, showing an apparent bias of 8 percent. No adjustment for bias is proposed here because the runoff events available for testing do not include extremely large events and because some of the apparent bias may result from inaccurate data of "observed" discharges. The results on Fabrique Branch of Gypsum Creek at Harry Street show the largest errors, but the station has a poor stage-discharge rating curve owing to a lack of large discharge measurements. Dry Creek at Pawnee Avenue also has a poor rating for the same reason, and malfunction of equipment prevented evaluation of several large events.

The results of the testing suggest that the triangular-hydrograph method provides reasonable estimates of actual events. It is assumed that the method will provide reasonable results when applied to the calculation of design flows for ungaged basins. However, continued data collection and additional flood measurements are needed either to verify the method or to provide the basis for further modifications.

APPLICATION OF MODIFIED SCS METHOD TO UNGAGED BASINS

The SCS triangular-hydrograph method of estimating peak runoff discharges for small ungaged watersheds, as outlined by U.S. Soil Conservation Service (1964) and Kent (1973) was originally designed for rural basins. The urban hydrology report by U.S. Soil Conservation Service (1975) states that the triangular-hydrograph method should be used only on basins with drainage areas of 2,000 acres (3.1 mi^2) or less. The study in the Wichita area found results were reasonable even when used on basins as large as 16 mi^2. The same SCS report states that results would be suspect if slopes are greater than 30 percent; none of the slopes in the Wichita area approach this limit.

Probability, Recurrence Interval, and the "Design Storm" Concept

For purposes of hydrologic analysis and for some economic design purposes, the degree of extremity of a flood is expressed by its "probability of exceedance." A very low probability indicates an extreme flood. A flood that has a probability of 2 percent has a 2-percent chance of being exceeded in any one year.

Although the concept of probability is the most meaningful way of classifying and comparing floods, the terminology in more common use is the "recurrence interval" in years, which is 100 times the reciprocal of the probability of exceedance when the probability is in percent. Thus, a 2-percent probability flood has a recurrence interval of 50 years. Stated in more general terms, a flood having a recurrence interval of N years is expected to be exceeded an average of once in each period of N years and is known as the "N-year flood." More than one flood exceeding that magnitude may occur in any particular period of N years, or no flood exceeding that magnitude may occur during an equivalent period. The fact that a flood of given magnitude occurs in one year does not reduce the probability of a flood of equal or greater magnitude occuring during the next year. The term "recurrence interval" will be used in this report instead of probability.

For ungaged small basins, the preferred method (Alley, 1977, p. 9) of determining peak flows for specified recurrence intervals is to use a digital-computer model to create several decades of synthetic record of high flows. A recurrence-interval analysis would then be made for the series of annual peak discharges. Until several digital-computer models have been tested with data from Wichiua and a model selected and adapted for convenient use, a peak discharge for drainage design can be calculated by applying the SCS method to a standardized "design storm". Application of the SCS method to the estimation of flood-frequency discharges depends on the assumption of equality between rainfall recurrence interval and discharge recurrence interval. A rainfall event -- the "design storm" -- having a given recurrence interval is assumed to produce a discharge peak having the same recurrence interval. It has been shown by several investigators (Mawson, 1959; Wilson, 1968; Wasson, 1969) that this assumption is not generally correct in rural areas. The antecedent moisture conditions (wetness of the soil and temporary surface storage) in rural areas may be widely different before each storm. The result is that a 5-year recurrence interval storm on a dry basin may produce 2-year recurrence interval discharge peak; whereas, the same storm on a wet basin may produce a 10-year recurrence interval discharge peak.

In drainage design and flood-hazard studies of urban areas, the problems previously described are much less severe than for rural areas. A heavy storm on an imprevious urban area will produce a significant peak discharge even if the area was previously dry; antecedent moisture conditions have little effect on the amount of runoff from impervious areas; and antecedent conditions on pervious urban areas are more uniform because of lawn watering. Thus, a peak flow calculated by the synthetic-hydrograph method should have approximately the same recurrence interval as the rainfall from which it was calculated if an average antecedent moisture condition is used in the calculation.

In the future, digital-computer models will be available for calculations of hydrographs resulting from several storms each year for a large number of years, by the SCS method or other methods. Computer models available at the present time have not been tested for application to the Wichita data and the requirements are unduly complex for application to routine design calculations. The modified SCS method presented here is simple, easily applied, and should give good results when applied to urbanized basins in and near Wichita.

The average accumulation of rainfall for 24 hours in heavy storms occurring in the United States, as given by Kent (1973), is shown in figure 3. The terms used in figure 3 are defined as follows: P_x is accumulated rainfall, in inches, for x hours, and P_{24} is total rainfall, in inches, for 24 hours. The rainfall accumulation from figure 3 can be used with the 24-hour rainfall for a selected recurrence interval from figure 4 to produce a "design storm" from which a discharge hydrograph can be calculated.

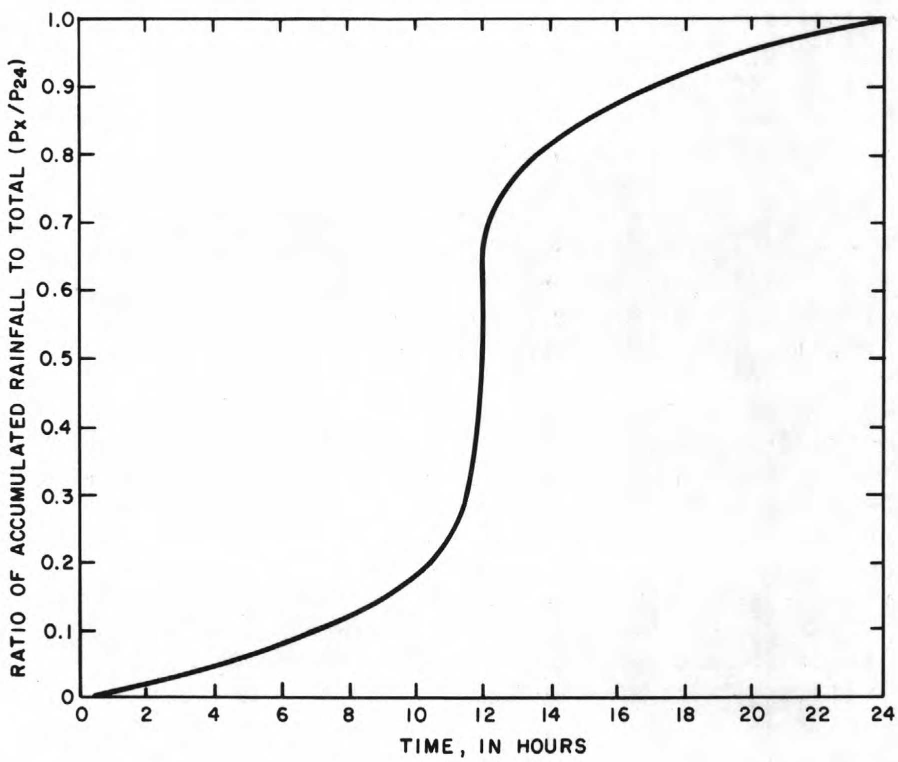

Figure 3.--Accumlation of rainfall to 24 hours for continental United States (Kent, 1973).

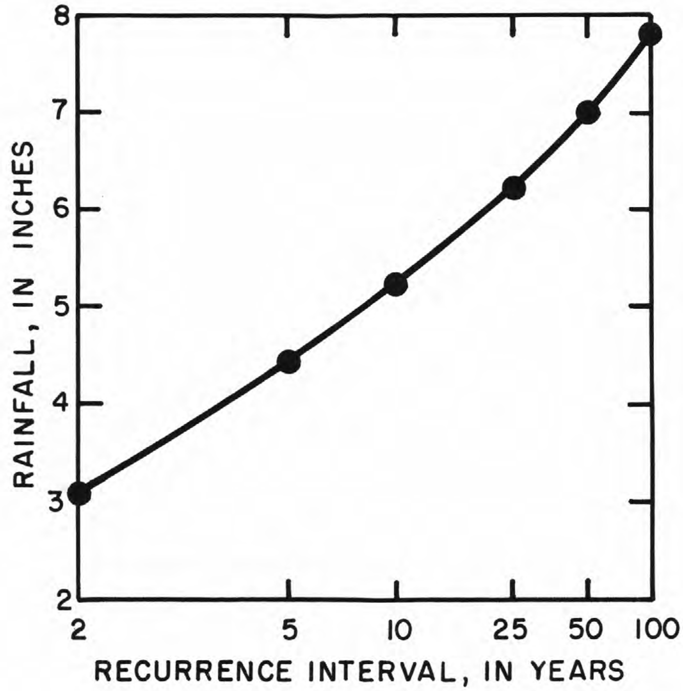

Figure 4.--24-hour rainfall frequency for the Wichita area, modified from Hershfield (1961).

Calculation of Synthetic Hydrograph

The parameters that need to be known for a given basin are: (1) length of main channel, (2) slope of channel, (3) drainage area, (4) percentage of impervious surface, (5) hydrologic soil groups, and (6) land use. Length of main channel, slope of channel, and drainage area can be determined from topographic maps supplemented with information on the storm-sewer system. The percentage of impervious surface can be measured using aerial photos, or a percentage can be based on the expected future degree of urbanization. The hydrologic soil group for a basin in Wichita (based on data from Penner, 1978) can be determined from figure 5. The soil typing, as used here, is divided into four categories according to infiltration capacity. Type A is a predominantly sandy texture with a high infiltration capacity; type D is a heavy soil with a high clay content and low infiltration capacity. Types B and C have intermediate infiltration capacities.

Using information on land use in the basin combined with the hydrologic soil group, the curve number (CN) can be determined from table 4. The curve numbers in table 4 are based on the average antecedent moisture condition (condition II), which is the condition assumed for the probability-discharge calculation. The numbered curve determined from table 4 is used in estimating the runoff resulting from a given amount of rainfall, as shown in figure 6.

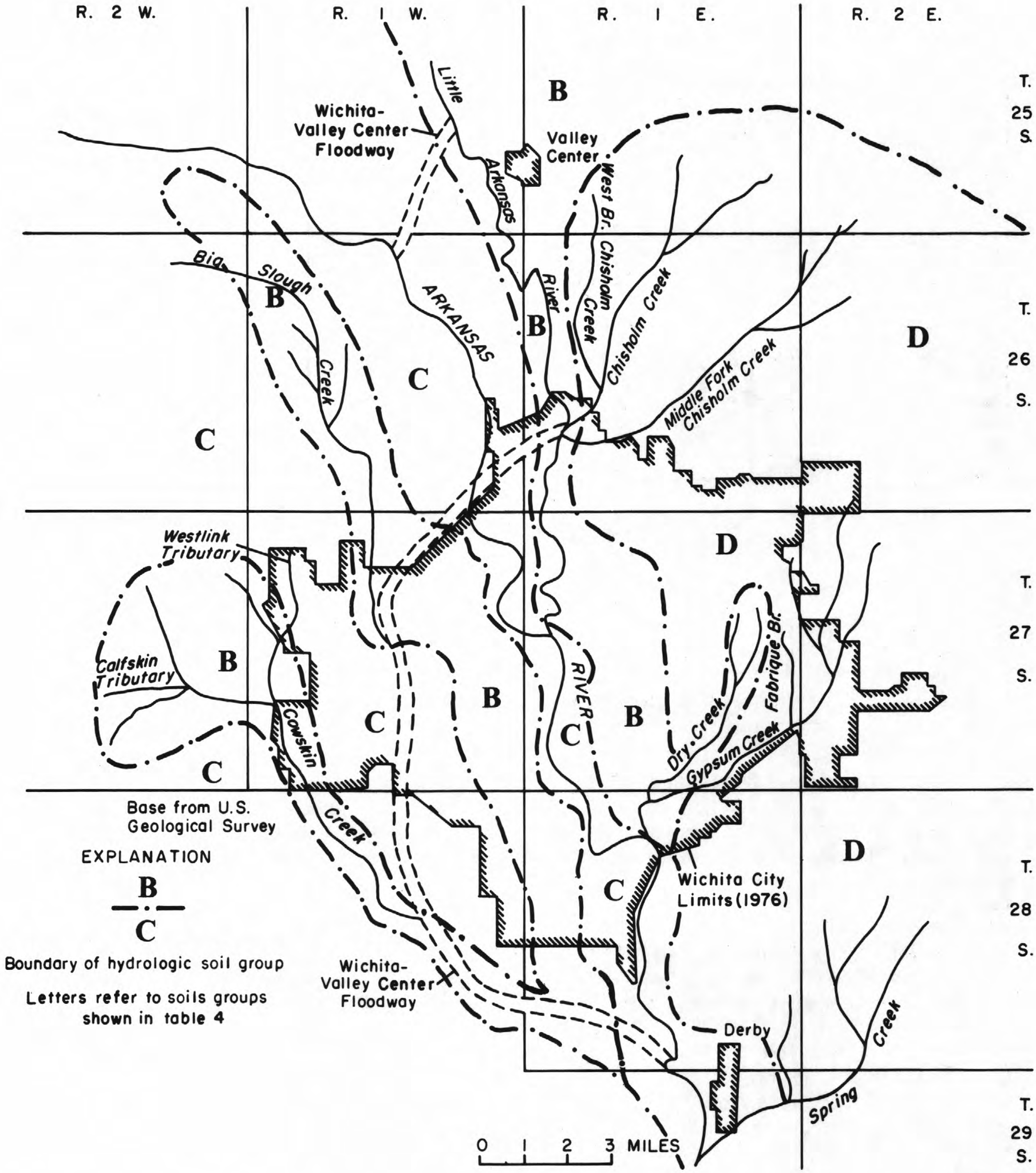

Figure 5.--Soil classification according to infiltration capacity.

Table 4.--Runoff curve numbers for selected agricultural, suburban, and urban land use. (Antecedent moisture condition II).

[From SCS Tech. Release No. 55 (1975)]

LAND USE DESCRIPTION	HYDROLOGIC SOIL GROUP			
	A	B	C	D
Cultivated land: without conservation treatment	72	81	88	91
: with conservation treatment	62	71	78	81
Pasture or range land: poor condition	68	79	86	89
: good condition	39	61	74	80
Meadow: good condition	30	58	71	78
Wood or Forest land: thin stand, poor cover, no mulch	45	66	77	83
: good cover	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc.				
good condition: grass cover on 75% or more of the area	39	61	74	80
fair condition: grass cover on 50% to 75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential:* Average lot size — Average % impervious**				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads: paved with curbs and storm sewers	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

* Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns, where additional infiltration could occur.

** The remaining pervious areas (lawn) are considered to be in good pasture condition.

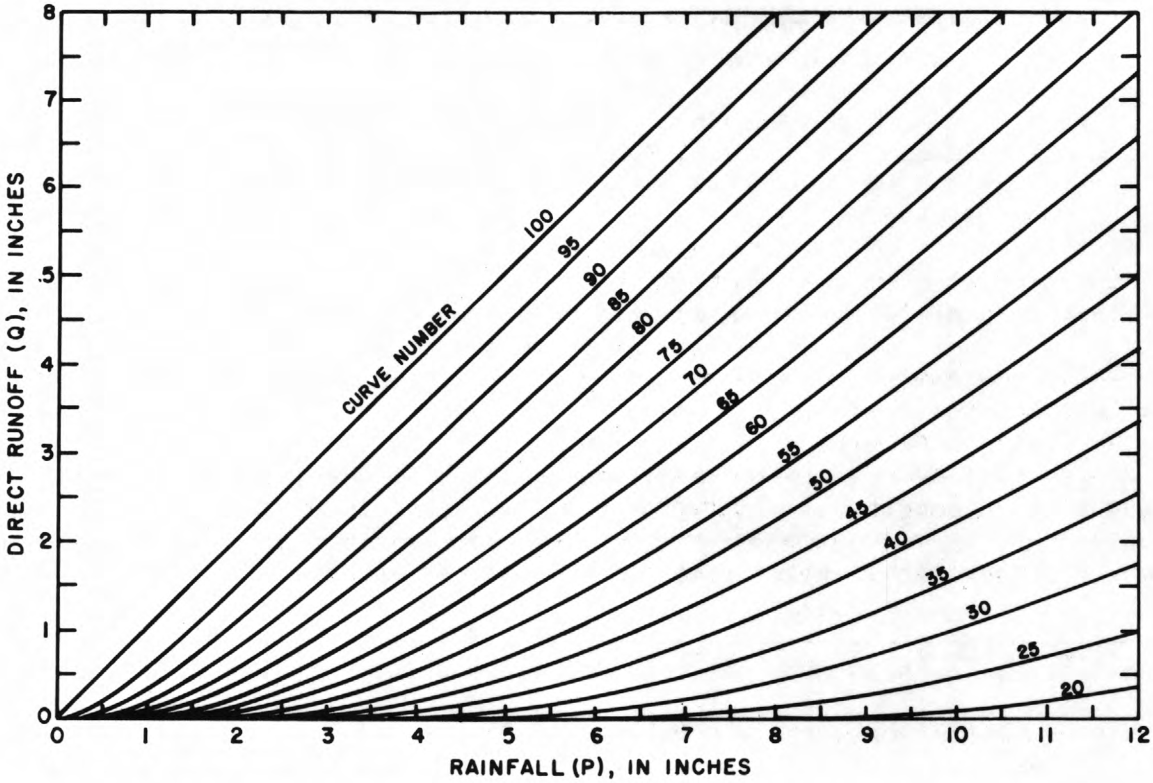

Figure 6.--Relationship of runoff to rainfall and curve number (from Mockus, 1955).

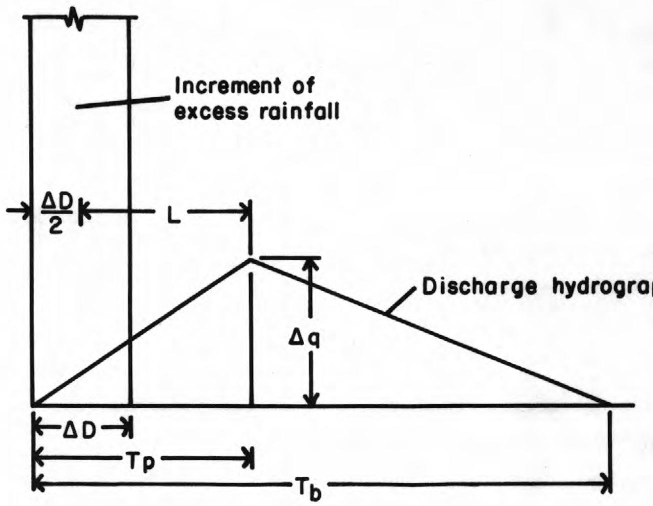

Figure 7.--Triangular hydrograph relationships.

The adjustment for lag time is determined by the equation (Putnam, 1972):

$$L = 0.49 \left[\frac{\ell}{\sqrt{S}} \right]^{0.5} (I)^{-0.57} \quad ,$$

where L = lag time, in hours
ℓ = length of main water course, in miles
S = channel slope, in feet per mile
I = ratio of impervious area to total drainage area

(Even though Putnam's definition of lag time differs slightly from that of the SCS, numerical results from the quoted equation were found to be compatible with the SCS procedure).

A synthetic hydrograph for each increment of rainfall can be developed by the use of a triangular hydrograph, as illustrated in figure 7. The first step considers the relationship between the variables in figure 7. Examination of many hydrographs has shown that the average relation is

$$\text{(Time of base) } T_b = 2.67 \left(\frac{\Delta D}{2} + L \right) \text{, and}$$

from the geometry in figure 7 it follows that:

$$\Delta q = \frac{484 \ A \ (\Delta Q)}{\frac{\Delta D}{2} + L} \quad ,$$

where ΔD = time increment, in hours
Δq = peak discharge, in cubic feet per second for an increment of runoff
ΔQ = runoff, in inches during period ΔD
A = drainage area, in square miles
L = lag time, in hours;
and 484 is a constant that includes conversion of units.

Suggested values of ΔD are 10 minutes (0.167 hour) for drainage areas under 5 mi^2 and 15 minutes (0.25 hour) for drainage areas over 5 mi^2. ΔQ is calculated from the cumulative Q, which is found by the use of figure 6.

Sample Computation

In order to demonstrate the method used, a numerical example is shown for calculating the 100-year flood for a hypothetical urbanized basin in Wichita:

Given the following basin characteristics:
drainage area (A) = 8.9 mi^2,
ΔD of 0.25 hour is used for areas larger than 5 mi^2,
main-stream length (ℓ) = 9.4 miles,
channel slope (S) = 14 ft/mile,
impervious area ratio (I) = 0.20.

1. Determine the hydrologic soil group. For this basin, assume figure 5 shows group C.

2. Determine the percentage of basin area having each land use. Using table 4, calculate the weighted curve number (CN). The following table shows the calculations:

Land Use	Percent	CN	Product
Residential, 1/8 acre	30	90	2,700
Commercial and business area	15	94	1,410
Streets, paved with curbs and sewers	10	98	980
Paved parking lots	5	98	490
Open spaces, lawns, parks, etc., good condition	40	74	2,960
	Total		8,540

Weighted CN = $\frac{8,540}{100}$ = 85.

3. Calculate the lag time (L) by Putnam's equation:

$$L = 0.49 \left[\frac{\ell}{\sqrt{S}}\right]^{0.5} (I)^{-0.57}$$

$$= 0.49 \left[\frac{9.4}{\sqrt{14}}\right]^{0.5} (0.20)^{-0.57}$$

$$= 1.94 \text{ hours.}$$

4. Calculate the time to peak (T_p) and time of base (T_b):

$$T_p = (\frac{\Delta D}{2} + L) = (\frac{0.25}{2}) + 1.94 = 2.06 \text{ hours,}$$

$$T_b = 2.67\ T_p = 2.67(2.06) = 5.51 \text{ hours.}$$

5. Calculate the numerical relationship between Δq and ΔQ:

$$\Delta q = \frac{484\ A\ (\Delta Q)}{\frac{\Delta D}{2} + L} .$$

Using 0.25 hour for ΔD,

$$\Delta q = \frac{484\ (8.9)\ \Delta Q}{0.125 + 1.94} = 2090(\Delta Q).$$

Similarly, when ΔD = 0.50 hour, the result is Δq = 1970(ΔQ).
when ΔD = 1.00 hour, the result is Δq = 1770(ΔQ).

6. Figure 4 shows that the 100-year, 24-hour rainfall is 7.8 inches.

7. Using the results of steps 5 and 6 and figures 3 and 6, prepare the results, as shown in table 5. The shortest time intervals are used during the time of most rapid accumulation of rainfall.

8. Using the Δq values from table 5 and T_p and T_b previously calculated, plot the set of triangular hydrographs with their appropriate, respective starting times, as in figure 8.

9. Construct the resulting synthetic hydrograph by summing the ordinates of the triangular hydrographs. In figure 8, the triangular hydrograph ordinates were summed at each hour line. The resulting synthetic hydrograph has a peak of 7,400 ft^3/s.

Table 5.--Rainfall accumulation and triangular-hydrograph ordinates for 100-year storm.

Time (hours)	P_x/P_{24}	P_x (inches)	Direct runoff, Q* (inches)	ΔQ (inches)	Δq (ft^3/s)
0	0	0	0	0	0
1.0	.010	.08	0	0	0
2.0	.022	.17	0	0	0
3.0	.034	.26	0	0	0
4.0	.048	.37	0	0	0
5.0	.062	.48	0	0	0
6.0	.080	.62	.01	.01	18
7.0	.100	.78	.08	.07	124
8.0	.120	.94	.16	.08	142
9.0	.147	1.15	.24	.08	142
10.0	.181	1.41	.39	.15	266
10.5	.204	1.59	.54	.15	266
11.0	.235	1.83	.68	.14	276
11.5	.283	2.20	.97	.29	571
11.75	.387	3.01	1.59	.62	1,300
12.0	.663	5.17	3.52	1.93	4,030
12.5	.735	5.73	4.07	.55	1,080
13.0	.772	6.02	4.31	.24	472
13.5	.799	6.23	4.52	.21	414
14.0	.820	6.40	4.70	.18	355
15.0	.855	6.67	4.91	.21	372
16.0	.880	6.86	5.11	.20	354
17.0	.902	7.04	5.26	.15	266
18.0	.921	7.18	5.42	.16	283
19.0	.930	7.25	5.52	.10	177
20.0	.952	7.42	5.64	.12	212
21.0	.968	7.55	5.76	.12	212
22.0	.980	7.64	5.86	.10	177
23.0	.991	7.73	5.95	.09	159
24.0	1.000	7.80	6.02	.07	124

* Using curve number of 85 from step 3.

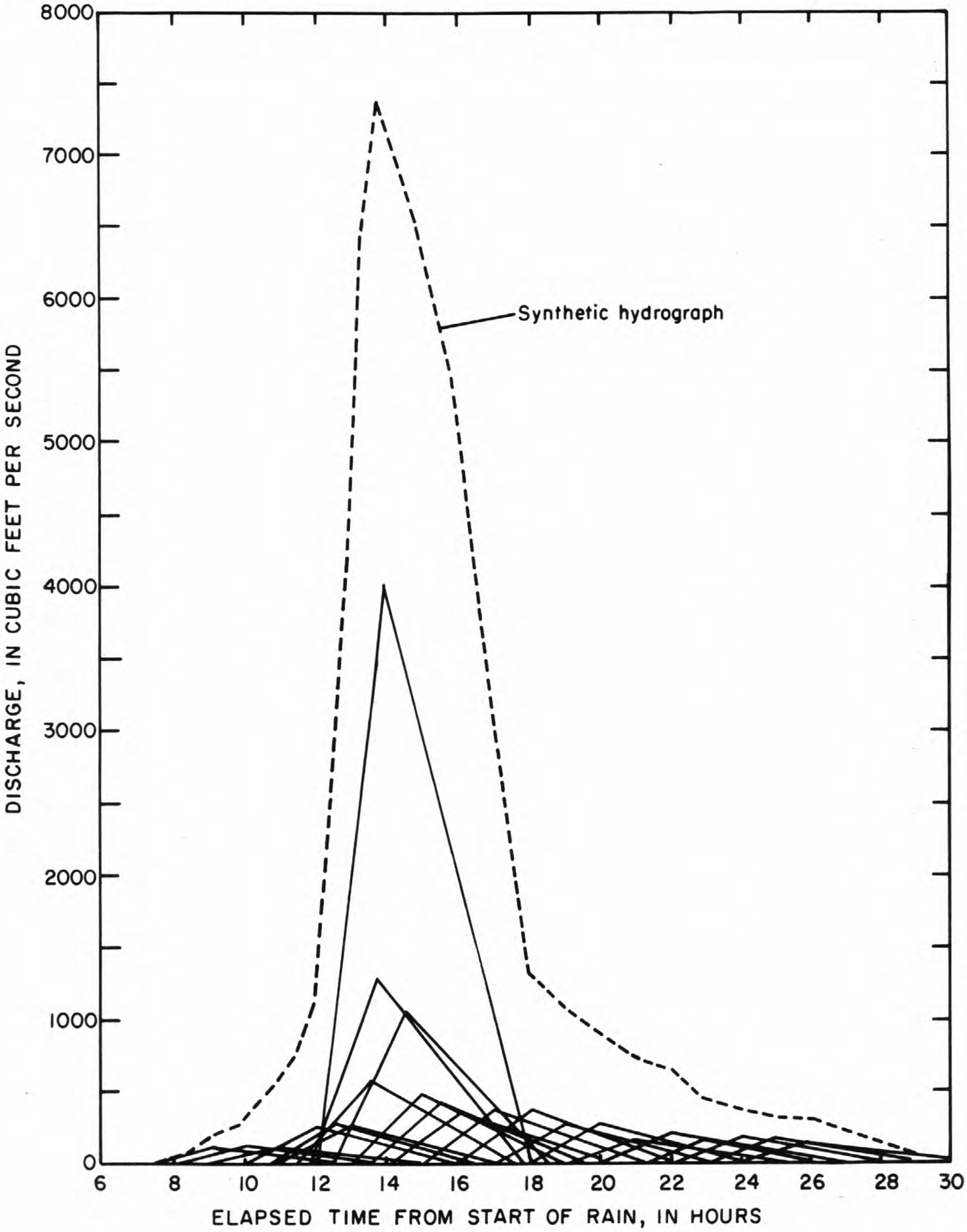

Figure 8.--Triangular hydrographs and synthetic hydrograph resulting from 100-year storm on hypothetical urbanized basin.

Shortcut Calculation

The plotting of triangular hydrographs and construction of the synthetic hydrograph are not necessary if only the peak discharge, rather than the complete hydrograph, is needed for a "design storm" having the proportional accumulation shown in figure 3. Kent (1973, p. 12-17) developed a shortcut solution using only the period of excess rainfall that directly affects the peak discharge. A relationship between ΔD and L can be chosen that enables the summation of only a single ordinate from each triangular hydrograph, which can be calculated rather than plotted. The usual choice is to make ΔD equal to one-third the time to peak (T_p), which is equivalent to choosing ΔD equal to 0.4 L. For the rainfall accumulation shown in figure 3 and high curve numbers appropriate for urbanized areas, the effective peak-producing runoff period is $7\Delta D$, and the midpoint of the most intense increment of runoff is 11.88 hours, which is the midpoint of the fifth increment of the seven (Kent, 1973, p. 13-14). Thus, the computation is started at a time of $11.88-4.5\Delta D$, and the peak discharge is:

$$q = 0.2\Delta q_1, + 0.4\Delta q_2 + 0.6\Delta q_3 + 0.8\Delta q_4$$

$$+ 1.0\Delta q_5 + \tfrac{2}{3}\Delta q_6 + \tfrac{1}{3}\Delta q_7 ,$$

in which q is the peak discharge, and Δq_1, Δq_2, etc. are the peaks of the triangular hydrographs for the first through the seventh periods of the seven used.

In the case of the hypothetical basin used in the previous numerical example, L is 1.94 hours, so

$$\Delta D = 0.4(1.94) = 0.78 \text{ hour}.$$

The starting time is

$$11.88 - 4.5(0.78) = 8.37 \text{ hours}.$$

For $\Delta D = 0.78$ hour, $A = 8.9$ mi^2, and $L = 1.94$ hours, the relation between Δq and ΔQ is

$$\Delta q = \frac{484.(8.9)\ \Delta Q}{\frac{0.78}{2} + 1.94} = 1{,}850\ \Delta Q.$$

Using figure 3, the seven required Δq values can be calculated as shown in table 6.

Table 6.--Calculation of Δq for seven increments.

Increment	Time, hours	P_x/P_{24}	P_x (inches)	Direct runoff, Q (inches)	ΔQ (inches)	Δq (ft^3/s)
	8.37	0.130	1.01	0.19		
1					0.02	37
	9.15	.150	1.17	.21		
2					.17	314
	9.93	.178	1.39	.38		
3					.22	407
	10.71	.217	1.69	.60		
4					.35	647
	11.49	.290	2.26	.95		
5					2.87	5,310
	12.27	.730	5.69	3.82		
6					.52	961
	13.05	.775	6.04	4.34		
7					.27	499
	13.83	.815	6.35	4.61		

The peak discharge is calculated by summing the appropriate fractions of the Δq as follows:

$$q = 0.2(37) + 0.4(314) + 0.6(407) + 0.8(647) + 1.0(5{,}310)$$
$$+ 0.67(961) + 0.33(499)$$
$$= 7{,}000 \ ft^3/s.$$

In the triangular- hydrograph solution the discharge was 7,400 ft^3/s, which is a 5-percent difference. Other examples yielded similar results; thus, the shortcut calculation is sufficiently accurate for practical use.

CONCLUSIONS

Data collected in and near Wichita from 1964 through 1976 have provided a basis for testing the validity of the Soil Conservation Service synthetic-hydrograph method of computing flood hydrographs from rainfall data for small streams in the Wichita urban area. Because the magnitude of intense storms was below normal in the 1964-76 period, only the largest storms for which adequate data were available were used for tests of the computation technique. The tests showed a need for modification in the calculations of lag time, which was accomplished by using Putnam's (1972) equation. With the modification, the SCS method was found to be applicable to basins varying from slightly urbanized to fully urbanized in the Wichita area. Tests indicate the synthetic-hydrograph method may give slightly biased results; however, uncertainties in some of the data render impractical any adjustment for bias.

Application of the modified SCS method is fairly simple and requires data on impervious surface, soils, land use, channel slope, length of main channel, drainage area, and rainfall in short increments of time for the selected storm. As an alternative to more complete and complex modeling by digital computer, a peak discharge for drainage design can be calculated by applying the modified SCS method to a standardized "design storm" for a specified recurrence interval.

The SCS method is sensitive to soil conditions and land use; therefore, accurate information on these factors is necessary. Continued and improved collection of data may provide for adjustment to remove the small bias from the method and also may provide adequate data for the future use of digital-computer models.

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APPENDIX

Rainfall and discharge data for selected storms

Station 07144320, Gypsum Creek at Gilbert Street

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 11, 1970				June 12, 1970 (cont.)		
1930	---	---	0	0440			1400
1945	0.01	0.01		0455			1450
2000	.01	.02		0540			1280
2015	.38	.40		0640			1040
2030	.40	.80		0740			870
2045	.34	1.14		0840			675
2100	.08	1.22		0940			500
2115	.03	1.25		1040			340
2130	.02	1.27		1140			255
2145	.01	1.28		1240			190
2200	.00	1.28		1340			150
2300				1440			112
2400				1540			92
				1640			79
	June 12, 1970			1740			67
				1840			59
0040				1940			54
0140				2040			50
0240			0	2140			45
0340			200				

Time	Inter-val rain-fall (in)	Cumu-lative rain-fall (in)	Dis-charge (ft^3/s)	Time	Inter-val rain-fall (in)	Cumu-lative rain-fall (in)	Dis-charge (ft^3/s)
	May 28, 1975				May 28, 1975 (cont.)		
0045	---	---	0	1030			370
0100	0.21	0.21		1130			305
0115	.08	.29		1230			255
0130	.26	.55	53	1330			222
0145	.46	1.01		1430			200
0200	.20	1.21		1530			177
0215	.01	1.22		1630			165
0230	.09	1.31	710	1730			155
0245	.15	1.46		1830			140
0300	.10	1.56		1930			125
0315	.10	1.66		2030			115
0330	.01	1.67	1100	2130			105
0345	.03	1.70		2230			95
0400	.05	1.75		2330			88
0415	.01	1.76					
0430	.00	1.76	1300		May 29, 1975		
0530			1150				
0630			1020	0030			81
0730			820	0130			74
0830			630	0230			62
0930			485	0330			60

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 2, 1975				June 3, 1975 (cont.)		
2315	---	---		0600			580
2330	0.10	0.10	0	0700			400
2345	1.02	1.12		0800			310
2400	.13	1.25	410	0900			255
				1000			203
	June 3, 1975			1100			172
				1200			145
0015	.14	1.39		1300			132
0030	.15	1.54		1400			120
0045	.05	1.59		1500			103
0100	.04	1.63	870	1600			87
0115	.02	1.65		1700			74
0130	.00	1.65		1800			60
0145	.01	1.66		1900			51
0200	.00	1.66	1100	2000			42
0300			1250	2100			38
0400			1070	2200			35
0500			810				

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 16, 1975				June 16, 1975 (cont.)		
1800	--	--		2330	.00	1.88	
1815	0.02	0.02		2345	.01	1.89	
1830	.01	.03		2400	.01	1.90	940
1845	.00	.03					
1900	.00	.03				June 17, 1975	
1915	.00	.03					
1930	.03	.06		0100	.00	1.90	810
1945	.00	.06		0200			590
2000	.01	.07	0	0300			400
2015	.51	.58		0400			308
2030	.21	.79		0500			260
2045	.06	.85		0600			212
2100	.00	.85	200	0700			174
2115	.35	1.20		0800			140
2130	.30	1.50		0900			118
2145	.16	1.66		1000			101
2200	.09	1.75	810	1100			88
2215	.03	1.78		1200			74
2230	.03	1.81		1300			62
2245	.02	1.83		1400			52
2300	.03	1.86	1060	1500			45
2315	.02	1.88		1600			39

Station 07144323, Fabrique Branch of Gypsum Creek at Harry Street

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	July 14, 1973				July 14, 1973 (cont.)		
0615	--	--		0850	.04	2.33	187
0630	0.05	0.05		0900	.03	2.36	170
0640	.19	.24	0	0910	.03	2.39	142
0650	.21	.45	45	0920	.22	2.61	170
0700	.28	.73	70	0930	.04	2.65	185
0710	.29	1.02	163	0940	.00	2.65	189
0720	.31	1.33	320	0950			168
0730	.39	1.72	680	1000			115
0740	.06	1.78	860	1010			82
0750	.01	1.79	500	1020			62
0800	.03	1.82	340	1030			51
0810	.18	2.00	225	1040			43
0820	.17	2.17	245	1050			38
0830	.08	2.25	287	1100			30
0840	.04	2.29	305				
	May 28, 1975				May 28, 1975 (cont.)		
0110	--	--		0330			134
0120	0.27	0.27	0	0340			86
0130	.29	.56	78	0350			79
0140	.28	.84	330	0400			96
0150	.27	1.11	1100	0410			91
0200	.03	1.14	1150	0420			75
0210	.00	1.14	840	0430			70
0220			540	0440			61
0230			310	0450			57
0240			240	0500			50
0250			200	0510			44
0300			200	0520			41
0310			160	0530			40
0320			152				

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 2, 1975		
2340	--	--	0
2350	0.80	0.80	49
2400	.40	1.20	530
	June 3, 1975		
0010	.07	1.27	830
0020	.05	1.32	680
0030	.05	1.37	530
0040	.02	1.39	285
0050	.03	1.42	155
0100	.04	1.46	165
0110	.03	1.49	130
0120	.00	1.49	115
0130			98
0140			82
0150			56
0200			51
0210			45
0220			40
0230			35
	June 3, 1975 (cont.)		
0240			30
0250			28
0300			26
0310			23
	June 17, 1976		
2110	.02	.02	
2120	.54	.56	108
2130	.14	.70	395
2140	.05	.75	820
2150	.09	.84	480
2200	.00	.84	220
2210			125
2220			91
2230			70
2240			57
2250			46
2300			40
2310			30

Station 07144325, Gypsum Creek at Oliver Street

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	April 20, 1974				April 20, 1974 (cont.)		
0600	--	--	0	1215	.16	1.30	
0615	0.01	0.01		1230	.20	1.50	
0630	.05	.06		1245	.12	1.62	
0645	.07	.13		1300	.07	1.69	1200
0700	.05	.18	44	1315	.01	1.70	
0715	.01	.19		1330	.00	1.70	
0730	.00	.19		1400			1500
0745	.00	.19		1500			1550
0800	.01	.20	57	1600			1470
0815	.01	.21		1700			1250
0830	.06	.27		1800			1020
0845	.02	.29		1900			650
0900	.04	.33	76	2000			300
0915	.05	.38		2100			187
0930	.03	.41		2200			125
0945	.05	.46		2300			94
1000	.08	.54	225	2400			75
1015	.09	.63					
1030	.06	.69			April 21, 1974		
1045	.02	.71					
1100	.01	.72	550	0100			64
1115	.00	.72		0200			56
1130	.00	.72		0300			52
1145	.06	.78		0400			46
1200	.36	1.14	800	0500			42

Time	Inter-val rain-fall (in)	Cumu-lative rain-fall (in)	Dis-charge (ft^3/s)	Time	Inter-val rain-fall (in)	Cumu-lative rain-fall (in)	Dis-charge (ft^3/s)
	May 9, 1974				May 9, 1974 (cont.)		
0245	--	--		0545	.15	1.14	
0300	0.02	0.02	0	0600	.23	1.37	665
0315	.01	.03		0650	.00	1.37	1020
0330	.10	.13		0700			970
0345	.04	.17		0800			815
0400	.15	.32	38	0900			680
0415	.12	.44		1000			330
0430	.23	.67		1100			98
0445	.04	.71		1200			60
0500	.19	.90	215	1300			50
0515	.06	.96		1400			42
0530	.03	.99		1500			37
	May 13, 1974				May 14, 1974 (cont.)		
2300	--	--	0	0200			800
2315	0.11	0.11		0300			770
2330	.09	.20		0400			550
2345	.35	.55		0500			305
2400	.47	1.02	165	0600			115
				0700			86
	May 14, 1974			0800			65
				0900			55
0015	.15	1.17		1000			48
0030	.04	1.21		1100			42
0045	.00	1.21		1200			39
0100			830	1300			32
0110			870	1400			30

Station 07144325, Gypsum Creek at Oliver Street (concl.)

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 16, 1975				June 17, 1975 (cont.)		
1945			0	0045			1500
2000	--	--		0145			1300
2015	0.18	0.18		0245			1080
2030	.34	.52		0345			900
2045	.06	.58	48	0445			550
2100	.00	.58		0545			247
2115	.22	.80		0645			150
2130	.34	1.14		0745			110
2145	.22	1.36	1060	0845			82
2200	.13	1.49		0945			67
2215	.04	1.53		1045			58
2230	.03	1.56		1145			51
2245	.03	1.59	1500	1245			46
2300	.00	1.59		1345			42
2345			1520	1445			38
				1545			33
	June 17, 1975			1645			30
0030			1540				

Station 07144330, Dry Creek at Lincoln Street

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
May 27, 1964				May 27, 1964 (cont.)			
1245	--	--		1815	.10	2.08	810
1300	0.16	0.16		1830	.02	2.10	800
1315	.02	.18	0	1845	.02	2.12	700
1330	.00	.18	30	1900	.08	2.20	660
1345	.04	.22	46	1915	.02	2.22	575
1400	.02	.24	80	1930	.04	2.26	550
1415	.06	.30	89	1945	.02	2.28	530
1430	.16	.46	74	2000	.00	2.28	510
1445	.12	.58	66	2015			470
1500	.02	.60	240	2030			410
1515	.00	.60	400	2045			245
1530	.06	.66	380	2100			165
1545	.06	.72	275	2115			115
1600	.24	.96	190	2130			90
1615	.14	1.10	240	2145			72
1630	.04	1.14	490	2200			68
1645	.16	1.30	620	2215			58
1700	.34	1.64	780	2230			44
1715	.12	1.76	810	2245			36
1730	.10	1.86	1100	2300			33
1745	.06	1.92	920	2315			30
1800	.06	1.98	840				

Station 07144330, Dry Creek at Lincoln Street (cont.)

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 4, 1965				June 4, 1965 (cont.)		
1330	--	--		2200	.08	3.14	490
1345	0.44	0.44		2215	.08	3.22	810
1400	.06	.50	0	2230	.00	3.22	1100
1415	.00	.50	60	2245	.00	3.22	1170
1430	.08	.58	118	2300	.08	3.30	920
1445	.08	.66	118	2315	.04	3.34	740
1500	.02	.68	118	2330	.06	3.40	580
1515	.04	.72	100	2345	.04	3.44	490
1530	.02	.74	104	2400	.16	3.60	410
1545	.00	.74	100				
1600	.62	1.36	88		June 5, 1978		
1615	.10	1.46	92				
1630	.00	1.46	420	0015	.10	3.70	395
1645	.00	1.46	575	0030	.02	3.72	410
1700	.02	1.48	550	0045	.04	3.76	450
1715	.00	1.48	430	0100	.02	3.78	452
1730	.00	1.48	355	0115	.02	3.80	410
1745	.02	1.50	355	0130	.00	3.80	395
1800	.02	1.52	350	0145			340
1815	.04	1.56	280	0200			325
1830	.06	1.62	245	0215			260
1845	.02	1.64	240	0230			240
1900	.02	1.66	168	0245			190
1915	.00	1.66	150	0300			165
1930	.00	1.66	168	0315			110
1945	.00	1.66	150	0330			82
2000	.18	1.84	120	0345			63
2015	.00	1.84	100	0400			53
2030	.08	1.92	96	0415			44
2045	.02	1.94	125	0430			38
2100	.00	1.94	220	0445			34
2115	.28	2.22	245	0500			29
2130	.24	2.46	224	0515			26
2145	.60	3.06	280				

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	May 28, 1975				May 28, 1975 (cont.)		
0120	--	--	0	0500			395
0130	0.33	0.33	21	0510			330
0140	.47	.80	88	0520			295
0150	.32	1.12	550	0530			260
0200	.12	1.24	740	0540			235
0210	.02	1.26	840	0550			205
0220	.00	1.26	760	0600			170
0230	.00	1.26	620	0610			130
0240	.14	1.40	550	0620			123
0250	.00	1.40	625	0630			115
0300	.01	1.41	810	0640			95
0310	.11	1.52	900	0650			77
0320	.03	1.55	960	0700			64
0330	.01	1.56	895	0710			57
0340	.00	1.56	820	0720			54
0350			765	0730			50
0400			710	0740			48
0410			623	0750			46
0420			610	0800			44
0430			580	0810			41
0440			540	0820			39
0450			470				

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
June 16, 1975				June 16, 1975 (cont.)			
2010	--	--	0	2340			280
2020	0.42	0.42	9	2350			240
2030	.18	.60	39	2400			210
2040	.01	.61	94				
2050	.01	.62	430	June 17, 1975			
2100	.00	.62	450				
2110	.19	.81	435	0010			175
2120	.30	1.11	515	0020			155
2130	.26	1.37	690	0030			135
2140	.17	1.54	1030	0040			123
2150	.15	1.69	1270	0050			108
2200	.06	1.75	1270	0100			91
2210	.02	1.77	1025	0110			70
2220	.00	1.77	980	0120			61
2230			910	0130			54
2240			790	0140			49
2250			700	0150			46
2300			560	0200			43
2310			470	0210			40
2320			400	0220			38
2330			320				

Station 0714340, Dry Creek at Pawnee Avenue

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	July 27, 1971				July 28, 1971 (cont.)		
2230	--	--		0050			520
2240	0.03	0.03		0100			500
2250	.07	.10		0110			450
2300	.16	.26		0120			392
2310	.42	.68		0130			340
2320	.25	.93		0140			293
2330	.10	1.03	0	0150			250
2340	.02	1.05	43	0200			208
2350	.00	1.05	86	0210			187
2400	.01	1.06	110	0220			164
				0230			145
	July 28, 1971			0240			114
				0250			92
0010	.03	1.09	210	0300			73
0020	.01	1.10	330	0310			56
0030	.02	1.12	440	0320			44
0040	.00	1.12	490	0330			37

Station 07144340, Dry Creek at Pawnee Avenue (cont.)

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 16, 1975				July 17, 1975		
2000	--	--		0100			510
2010	0.06	0.06		0020			450
2020	.28	.34		0030			405
2030	.03	.37		0040			362
2040	.01	.38		0050			332
2050	.01	.39		0100			307
2100	.05	.44	0	0110			280
2110	.54	.98	10	0120			263
2120	.39	1.37	70	0130			240
2130	.41	1.78	103	0140			228
2140	.26	2.04	205	0150			215
2150	.12	2.16	313	0200			200
2200	.05	2.21	430	0210			180
2210	.02	2.23	600	0220			165
2220	.02	2.25	940	0230			145
2230	.02	2.27	1200	0240			130
2240	.02	2.29	1450	0250			112
2250	.01	2.30	1600	0300			96
2300	.00	2.30	1480	0310			80
2310	.01	2.31	1300	0320			67
2320	.01	2.32	1150	0330			52
2330	.01	2.33	980	0340			41
2340	.00	2.33	860	0350			30
2350	.01	2.34	760				
2400	.00	2.34	590				

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	July 3, 1976				July 3, 1976 (cont.)		
0500	--	--		0910			600
0510	0.03	0.03		0920			540
0520	.20	.23		0930			490
0530	.17	.40		0940			445
0540	.06	.46		0950			400
0550	.17	.63		1000			370
0600	.23	.86		1010			350
0610	.21	1.07		1020			325
0620	.09	1.16	0	1030			305
0630	.05	.21	6	1040			270
0640	.02	1.23	12	1050			255
0650	.03	1.26	42	1100			230
0700	.04	1.30	100	1110			200
0710	.11	1.41	290	1120			190
0720	.03	1.44	460	1130			170
0730	.02	1.46	570	1140			160
0740	.01	1.47	680	1150			145
0750	.00	1.47	730	1200			130
0800			740	1210			110
0810			737	1220			90
0820			720	1230			75
0830			720	1240			60
0840			700	1250			42
0850			680	1300			29
0900			640				

Station 07144494, Westlink Tributary at Westfield Avenue

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 21, 1969				June 21, 1969 (cont.)		
0845	--	--		1500			62
0900	0.19	0.19		1530			61
0915	.72	.91		1600			61
0930	.36	1.27	0	1630			60
0945	.01	1.28		1700			59
1000	.00	1.28	272	1730			57
1030			355	1800			54
1100			310	1830			51
1200			235	1900			48
1230			175	1930			47
1300			135	2000			44
1330			105	2030			42
1400			81	2100			41
1430			63	2130			39
	October 23, 1970				October 23, 1970 (cont.)		
1600	--	--		2130			73
1615	0.01	0.01		2200			66
1630	.27	.28		2230			64
1645	1.10	1.38		2300			65
1700	.33	1.71	0	2330			64
1715	.48	2.19		2400			63
1730	.03	2.22	470				
1745	.03	2.25			October 24, 1970		
1800	.04	2.29	640				
1815	.04	2.33		0030			60
1830	.01	2.34	480	0100			58
1900	.00	2.34	370	0130			56
1930			235	0200			53
2000			177	0230			50
2030			128	0300			48
2100			92				

Station 07144494, Westlink Tributary at Westfield Avenue (concl.)

Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)	Time	Interval rainfall (in)	Cumulative rainfall (in)	Discharge (ft^3/s)
	June 23, 1976				June 24, 1976		
2200	--	--	0	0030			110
2215	0.02	0.02		0100			80
2230	.67	.69	57	0130			59
2245	.45	1.14		0200			46
2300	.01	1.15	195	0230			35
2315	.00	1.15		0300			31
2330	.00	1.15	178	0330			28
2345	.06	1.21					
2400	.00	1.21	144				

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