# RUNOFF HYDROGRAPHS FROM SMALL TEXAS BLACKLANDS WATERSHEDS



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# Runoff Hydrographs from Small Texas Blacklands Watersheds[1]

By

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Need for a method to estimate runoff hydrographs from ungaged areas became apparent when flood-routing studies were begun on the Brushy Creek Watershed near Riesel, Tex. Differences were observed between measured hydrographs and those computed by flood routing. It was impossible to determine whether the differences were because of the flood-routing procedure or errors in developing hydrographs from ungaged areas. So, data from gaged watersheds were studied to develop a more accurate procedure for estimating hydrographs from ungaged watersheds. Peak discharge rates were related to watershed factors, precipitation characteristics, and runoff volume. Time to peak and the recession constant were related to measurable watershed characteristics. The peak discharge, time to peak, and recession constant determined from these relationships were used with the two-parameter gamma distribution to compute the hydrograph.

## DATA

The data were taken from five subwatersheds of the Brushy Creek Watershed for the period 1940 through 1965. Typical of the Blacklands of Texas, the soils of the watershed are mainly deep montmorillonite clays, primarily Houston Black clay on three subwatersheds. The relief is gently rolling, with 2- to 3-percent slopes. Main stem stream slopes in the five subwatersheds range from 0.3 to 1.2 percent.

The drainage areas of these five subwatersheds range from 0.275 to 6.84 square miles. In general, the flood plain of Brushy Creek is broad and flat with a small meandering channel. The location of a typical valley cross section on the main stem is shown as VS-2 in figure 1. A plotting of this valley cross section is shown in figure 2.

The present land use, also typical of the Blacklands of Texas, is divided into three approximately equal parts: (1) Cropland planted to cotton, corn, grain sorghum, and oats; (2) pasture; and (3) idle land that is mostly in the conservation reserve. Land use has changed considerably since 1940. In 1940, 71 percent of the land was used for crops, 16 percent for pasture, and 13 percent was idle. The change in land use has not caused an important difference in the runoff hydrographs from the Brushy Creek watersheds. Therefore, the floods for the entire period of record were grouped for analysis.

Average annual precipitation is 33.06 inches and average annual runoff is 5.6 inches. Storms of short duration and very high intensities are common, particularly during the spring and summer. Since these storms usually produce the highest discharge on small watersheds, they

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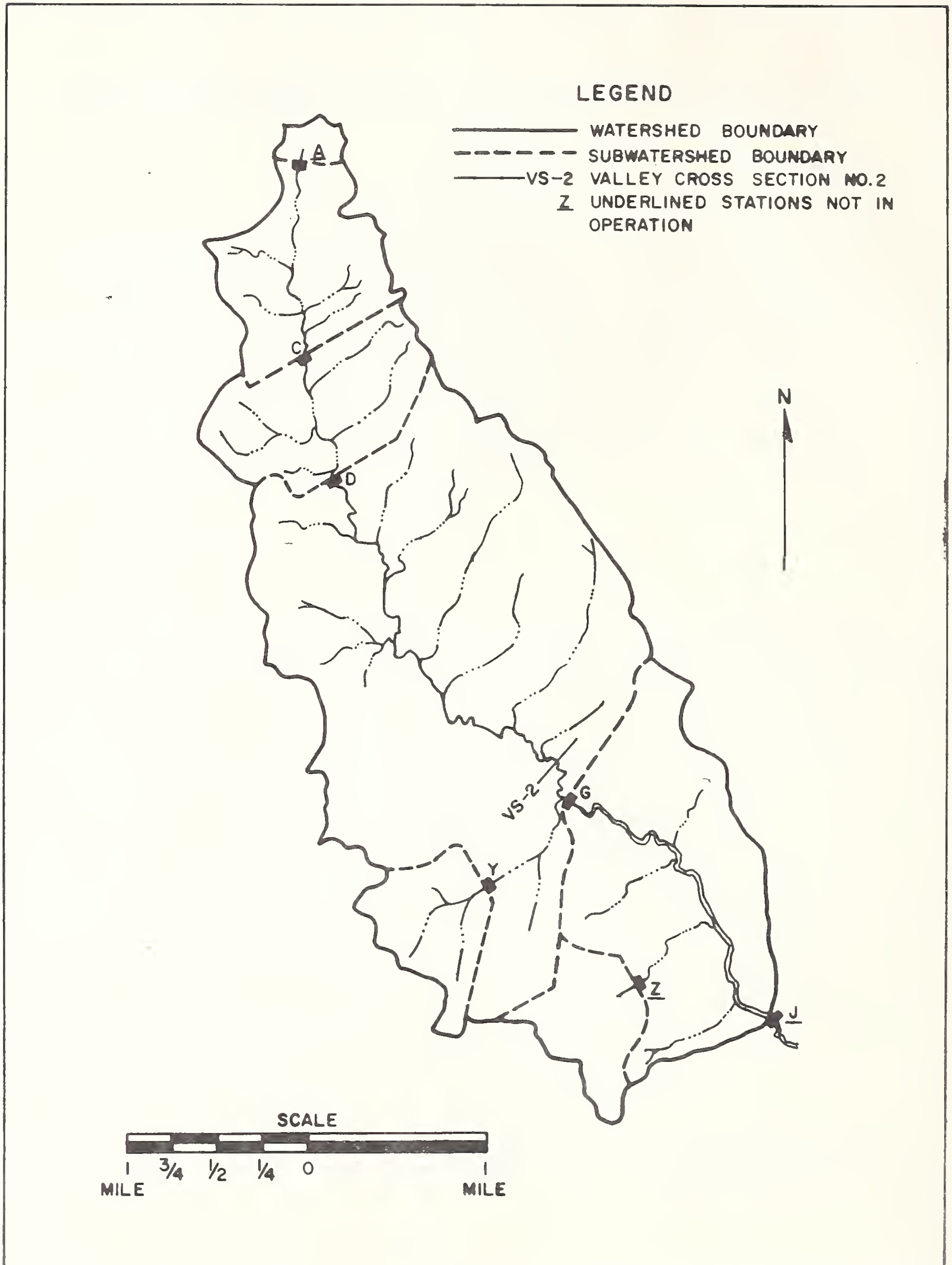


Figure 1.--Brushy Creek Watershed.

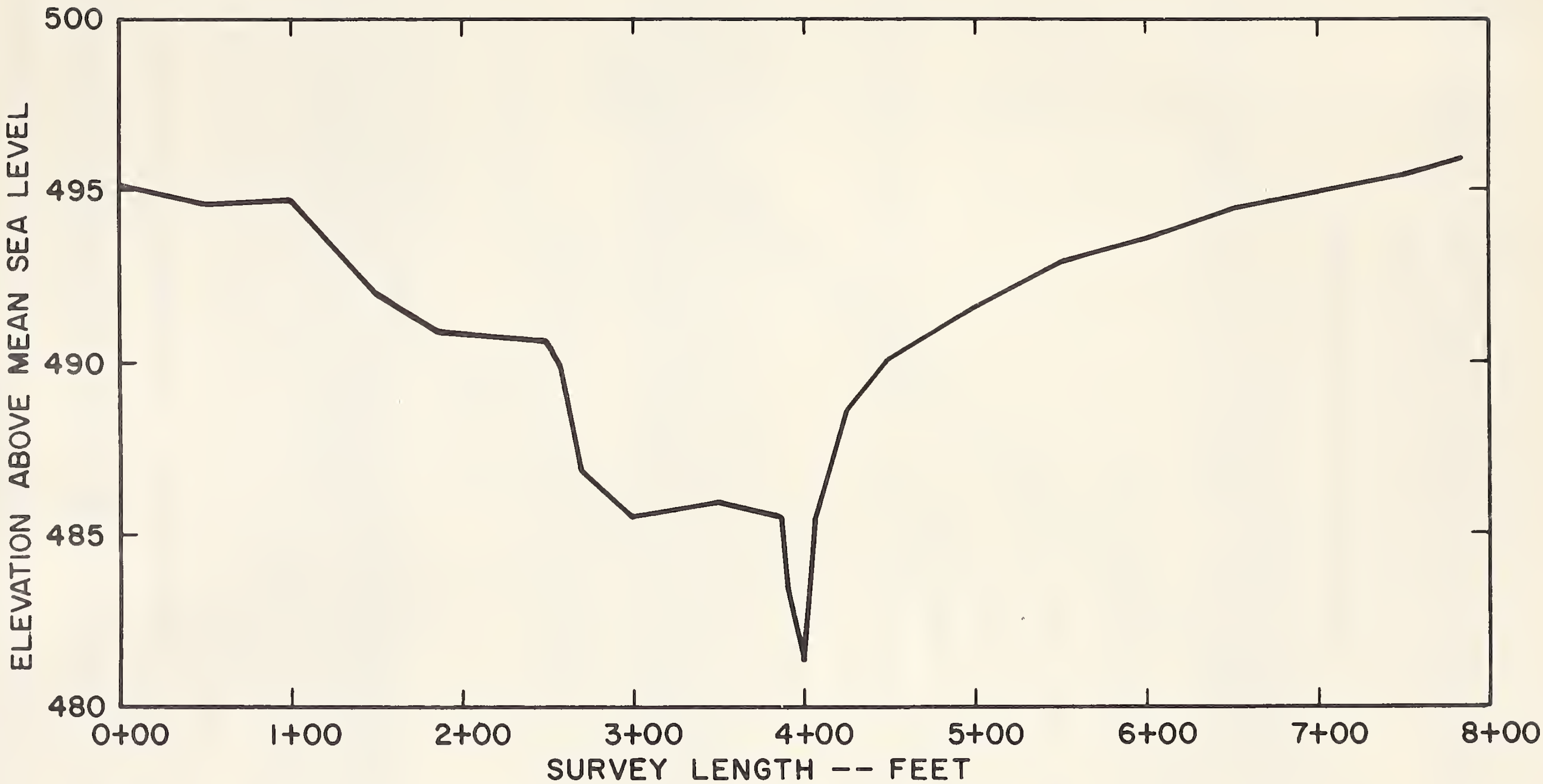


Figure 2.--Typical valley cross section on Brushy Creek.

are of primary interest. The 73 storms selected for this study had single-peak hydrographs with no rainfall excess after the peak. All of these storms produced runoff volume of 1 inch or more.

## DEVELOPMENT OF THE PROCEDURE

To adequately define a runoff hydrograph, the peak discharge, the time to peak, and the shape of the rising and recession limbs must be determined.

The major factors affecting peak discharge rate are runoff volume, rainfall intensity, drainage area, and a time factor. A multiple regression analysis of the 73 selected storms produced the equation:

$$q_p = 369\ Q^{0.686}\ A^{0.787} I^{0.225}\ K^{-0.412} \tag{1}$$

$q_p$ = peak discharge rate in c.f.s.

Q = runoff volume in inches

A = drainage area in square miles

I = rainfall intensity during a time interval equal to the recession constant (K) in inches per hour

K = recession constant in hours

Equation (1) has a coefficient of multiple correlation of 0.96. Figure 3 is a plotting of measured peak discharge versus peak discharge computed with equation (1) for the 73 selected storms.

The recession constant K was determined by plotting the recession for each of the 73 storms on semilog paper. A straight line drawn through the points of the recession curve permits determination of K. The recession curve is defined by the equation

$$K = \frac{t_1 - t_0}{2.3 \log \frac{q_0}{q_1}} \tag{2}$$

in which $q_0$ equals the flow at any time $t_0$; $q_1$ represents the flow at time $t_1$. For each watershed the value of K was almost constant from storm to storm.

K was related to certain measurable physical characteristics of the watersheds and multiple regression analysis yielded the prediction equation

$$K = 0.002044\ L^{0.520}\ S^{-1.263}\ E^{1.780} \tag{3}$$

K = recession constant in hours

L = length of the main stem along the channel from the most distant point on the watershed to the outlet in miles

S = the slope of the main stem channel from the most distant point on the watershed to the outlet, feet per foot.

E = elongation ratio--the ratio of the diameter of a circle with the same area as the watershed to the maximum length of the watershed.

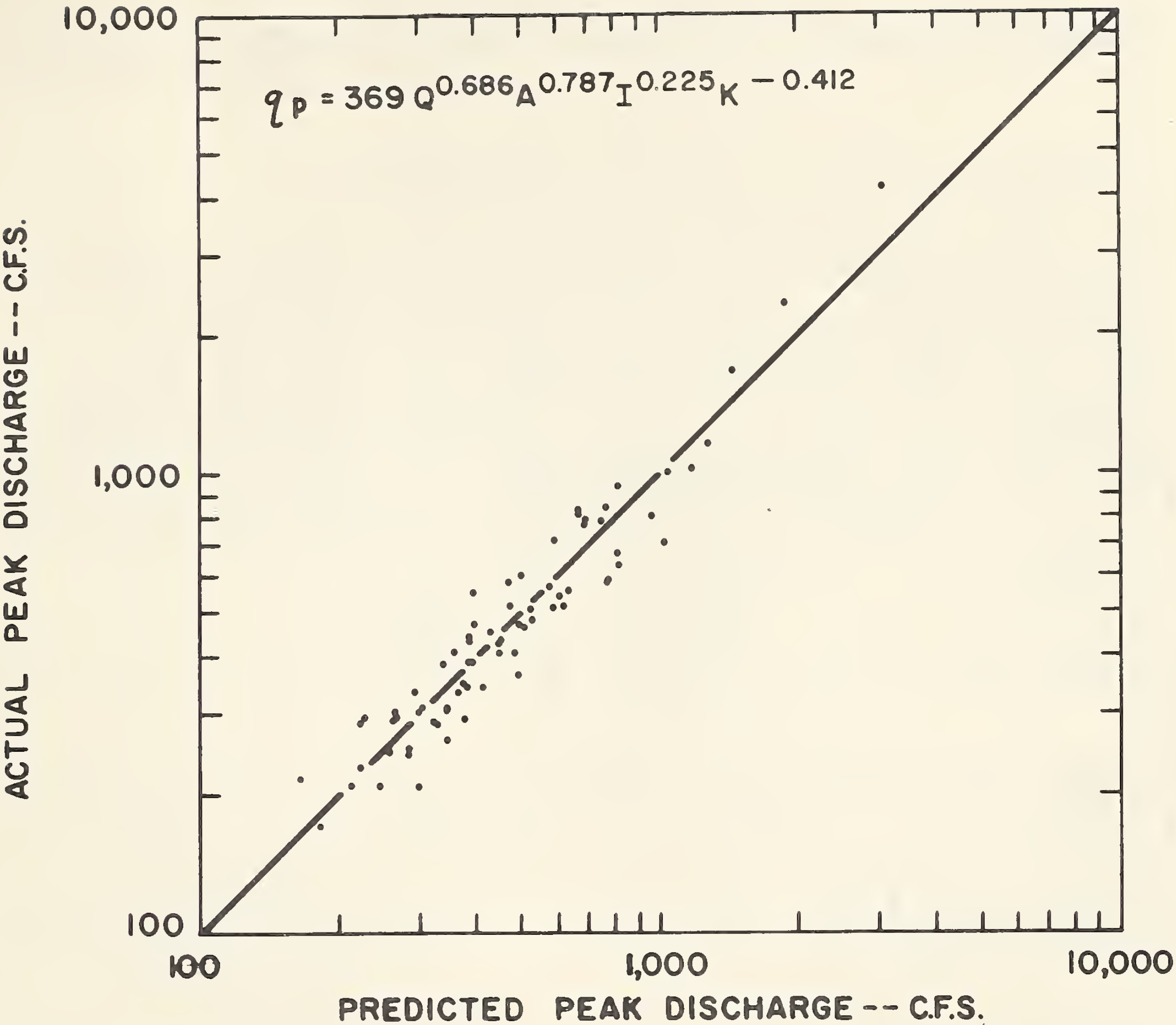


Figure 3.--Measured peak discharge versus peak discharge computed with equation 1 for Brushy Creek subwatersheds.

The coefficient of multiple correlation of equation (3) is 0.99.

Other watershed characteristics that were tried as predictors of K included drainage area, average land slope, length-width ratio, drainage density, circularity ratio, and hypsometric integral. None of these factors improved equation (3).

In developing equation (3), the data from the five Brushy Creek subwatersheds were supplemented by data from four other small Blackland watersheds. Table 1 contains descriptions of the physical characteristics of all nine watersheds used in this analysis.

TABLE 1.--Description of physical characteristics of nine watersheds

| Characteristics | Brushy Creek Sta. C | Brushy Creek Sta. D | Brushy Creek Sta. G | Brushy Creek Sta. Y | Brushy Creek Sta. W-1 | Cow Bayou Site No. 4 | Pin Oak Creek | Honey Creek Site No. 12 | Honey Creek Site No. 11 |
|---|---|---|---|---|---|---|---|---|---|
| Drainage area ---------------- sq. mi.- | 0.905 | 1.73 | 6.84 | 0.483 | 0.275 | 5.25 | 17.6 | 1.26 | 2.14 |
| Main stem length--------------miles-- | 1.59 | 2.54 | 6.52 | .97 | 1.02 | 4.11 | 8.41 | 2.01 | 2.05 |
| Main stem slope ---------------ft./ft.-- | .00738 | .00565 | .0034 | .0119 | .00944 | .0167 | .00473 | .0150 | .0110 |
| Elongation Ratio E ------------------- | .77 | .72 | .60 | .89 | .65 | .79 | .73 | .70 | .92 |
| Houston Black clay -------------pct.--- | ----- | 1 | 42 | 66 | 67 | 3 | 15 | 47 | 21 |
| Heiden clay-------------------pct.--- | ----- | ----- | 2 | 23 | 33 | 36 | 18 | 13 | ----- |
| Austin silty clay ----------------pct.--- | ----- | ----- | 2 | 10 | ----- | 12 | ----- | 30 | 70 |
| Austin-Eddy gravelly clay loam---pct.--- | ----- | ----- | ----- | ----- | ----- | 6 | ----- | ----- | ----- |
| Eddy gravelly clay loam ---------pct.--- | ----- | ----- | ----- | ----- | ----- | 34 | ----- | ----- | ----- |
| Sumter clay ------------------pct.--- | ----- | ----- | ----- | ----- | ----- | 4 | ----- | ----- | ----- |
| Crockett loam -----------------pct.--- | 7 | 3 | 1 | ----- | ----- | ----- | 13 | ----- | ----- |
| Crockett fine sandy loam --------pct.--- | ----- | ----- | ----- | ----- | ----- | ----- | 14 | ----- | ----- |
| Wilson clay loam---------------pct.--- | 73 | 66 | 29 | ----- | ----- | ----- | 15 | ----- | ----- |
| Wilson fine sandy loam ----------pct.--- | ----- | ----- | ----- | ----- | ----- | ----- | 18 | ----- | ----- |
| Trinity clay -------------------pct.--- | ----- | ----- | 1 | 1 | ----- | ----- | 5 | ----- | ----- |
| Lewisville clay -----------------pct.--- | ----- | ----- | ----- | ----- | ----- | ----- | ----- | 10 | ----- |
| Stephen silty clay ---------------pct.--- | ----- | ----- | ----- | ----- | ----- | ----- | ----- | ----- | 9 |
| Burleson-Heiden clay------------pct.--- | 17 | 24 | 20 | ----- | ----- | ----- | ----- | ----- | ----- |
| Frio clay loam-----------------pct.--- | 3 | 4 | 3 | ----- | ----- | ----- | ----- | ----- | ----- |
| Burleson clay------------------pct.--- | ----- | 2 | ----- | ----- | ----- | ----- | ----- | ----- | ----- |
| Miscellaneous soil types ---------pct.--- | ----- | ----- | ----- | ----- | ----- | 5 | 2 | ----- | ----- |

The data from these nine watersheds were also used to develop an equation for predicting the time to peak, using the same watershed characteristics that were used to predict K. Multiple regression analysis produced the equation

$$t_p = 0.144\ L^{0.935}\ S^{-0.369}\ E^{1.486} \tag{4}$$

This equation has a coefficient of multiple correlation of 0.99. Equation (4) was not improved by the use of drainage density, drainage area, circularity ratio, average land slope, length-width ratio, or hypsometric integral.

Time to peak may vary considerably from storm to storm. The only storms used to develop equation (4) were those with the shortest $t_p$'s for each watershed. Approximately 50 percent of the hydrographs originally selected for analysis were used. This tends to yield a more constant $t_p$. It also gives maximum peak discharges.

With the peak discharge and the time to peak determined, the only problem remaining is to define the shape of the hydrograph. The shape of the hydrograph can be computed by use of the two-parameter gamma distribution, as shown by Wu,[3] in the following equation

$$\frac{q}{q_p} = \left[\frac{t}{t_p}\right]^{(n-1)} \left[e^{-(n-1)}\right]^{\left(\frac{t}{t_p} - 1\right)} \tag{5}$$

in which

$q$ = discharge rate at any time t in c.f.s.

$q_p$ = peak discharge rate in c.f.s.

$t_p$ = time to peak in hours

$e$ = the base of the natural logarithm

$n$ = dimensionless parameter

Table 2 contains the dimensionless instantaneous hydrograph, which is computed with equation (5).

Wu, Delleur, and Diskin[4] have shown that n can be determined by integrating equation (5) to find the volume. The volume equation is then written in dimensionless form as follows

$$\frac{645.3\ AQ}{q_p\ t_p} = \frac{e^{(n-1)}\ \Gamma(n)}{(n-1)^n} \tag{6}$$

[3] Wu, I-Pai. Design hydrographs for small watersheds in Indiana. Jour. Hydraulics Div., Proc. Amer. Soc. Civil Engin. 89 (HY6): 35-66. 1963.

[4] Wu, I-Pai, Delleur, J. W., and Diskin, M. H. Determination of peak discharge and design hydrographs for small watersheds in Indiana. Indiana Flood Control and Water Resources Commission, Indiana State Highway Commission, and Purdue University. 106 pp. 1966.

TABLE 2.--The dimensionless instantaneous hydrograph

$q/q_p$

| $t/t_p$ | n = 2 | n = 3 | n = 4 | n = 5 | n = 6 | n = 7 | n = 8 | n = 9 | n = 10 | n = 11 | n = 12 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0.0 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 | 0.000 |
| 0.1 | .246 | .060 | .015 | .004 | .001 | .000 | .000 | .000 | .000 | .000 | .000 |
| 0.2 | .445 | .198 | .088 | .039 | .018 | .008 | .004 | .002 | .001 | .000 | .000 |
| 0.3 | .604 | .365 | .220 | .133 | .080 | .049 | .029 | .018 | .011 | .006 | .004 |
| 0.4 | .729 | .531 | .387 | .282 | .206 | .150 | .109 | .080 | .057 | .043 | .031 |
| 0.5 | .824 | .680 | .560 | .462 | .381 | .314 | .259 | .213 | .160 | .145 | .121 |
| 0.6 | .895 | .801 | .717 | .642 | .575 | .514 | .460 | .412 | .368 | .327 | .295 |
| 0.7 | .945 | .893 | .844 | .797 | .753 | .712 | .672 | .636 | .596 | .567 | .531 |
| 0.8 | .977 | .955 | .933 | .912 | .891 | .870 | .850 | .831 | .811 | .796 | .773 |
| 0.9 | .995 | .989 | .984 | .979 | .974 | .968 | .963 | .958 | .952 | .947 | .942 |
| 1.0 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 | 1.000 |
| 1.1 | .995 | .991 | .986 | .981 | .977 | .972 | .968 | .963 | .961 | .953 | .949 |
| 1.2 | .982 | .965 | .948 | .932 | .915 | .899 | .884 | .868 | .857 | .842 | .826 |
| 1.3 | .963 | .928 | .893 | .860 | .828 | .798 | .768 | .740 | .716 | .690 | .662 |
| 1.4 | .938 | .881 | .826 | .776 | .728 | .683 | .641 | .602 | .567 | .532 | .502 |
| 1.5 | .910 | .828 | .753 | .685 | .623 | .567 | .516 | .469 | .431 | .391 | .353 |
| 1.6 | .878 | .771 | .677 | .594 | .522 | .458 | .402 | .354 | .310 | .275 | .242 |
| 1.7 | .844 | .713 | .602 | .508 | .429 | .362 | .306 | .258 | .218 | .184 | .163 |
| 1.8 | .809 | .654 | .529 | .428 | .346 | .280 | .226 | .183 | .149 | .121 | .097 |
| 1.9 | .772 | .597 | .461 | .356 | .275 | .212 | .164 | .127 | .098 | .076 | .058 |
| 2.0 | .736 | .541 | .398 | .293 | .216 | .159 | .117 | .086 | .064 | .046 | .034 |
| 2.2 | .663 | .439 | .291 | .193 | .128 | .085 | .056 | .037 | .024 | .016 | .011 |
| 2.4 | .592 | .350 | .207 | .123 | .073 | .043 | .025 | .015 | .009 | .005 | .003 |
| 2.6 | .525 | .276 | .145 | .076 | .040 | .021 | .011 | .006 | .003 | .002 | .001 |
| 2.8 | .463 | .214 | .099 | .046 | .021 | .010 | .004 | .002 | .000 | .000 | .000 |
| 3.0 | .406 | .165 | .067 | .027 | .011 | .004 | .002 | .001 | | | |
| 3.5 | .287 | .082 | .024 | .007 | .002 | .001 | .000 | 0.000 | | | |
| 4.0 | .199 | .040 | .008 | .002 | .000 | .000 | | | | | |
| 4.5 | .136 | .018 | .002 | .000 | | | | | | | |
| 5.0 | .092 | .008 | .001 | | | | | | | | |
| 5.5 | .061 | .004 | .000 | | | | | | | | |
| 6.0 | .040 | .002 | | | | | | | | | |
| 6.5 | .027 | .001 | | | | | | | | | |
| 7.0 | .017 | .000 | | | | | | | | | |
| 7.5 | .011 | | | | | | | | | | |

Figure 4 shows a plotting of the above relationship. The value of n thus determined is placed into equation (5) and the entire hydrograph can now be defined. However, equation (5) did not properly define the recession curves of the hydrographs from the Brushy Creek watersheds.

Figure 5 is a comparison of a measured hydrograph with one computed with equation (5). It is evident that the predicted recession curve falls too sharply. DeCoursey[5] had the same problem when he attempted to fit a similar equation to the hydrographs from the subwatersheds of the Washita River Watershed.

[5] DeCoursey, Donn G. A runoff hydrograph equation. U.S. Dept. Agr., Agr. Res. Serv., ARS 41-116, 23 pp. 1966.

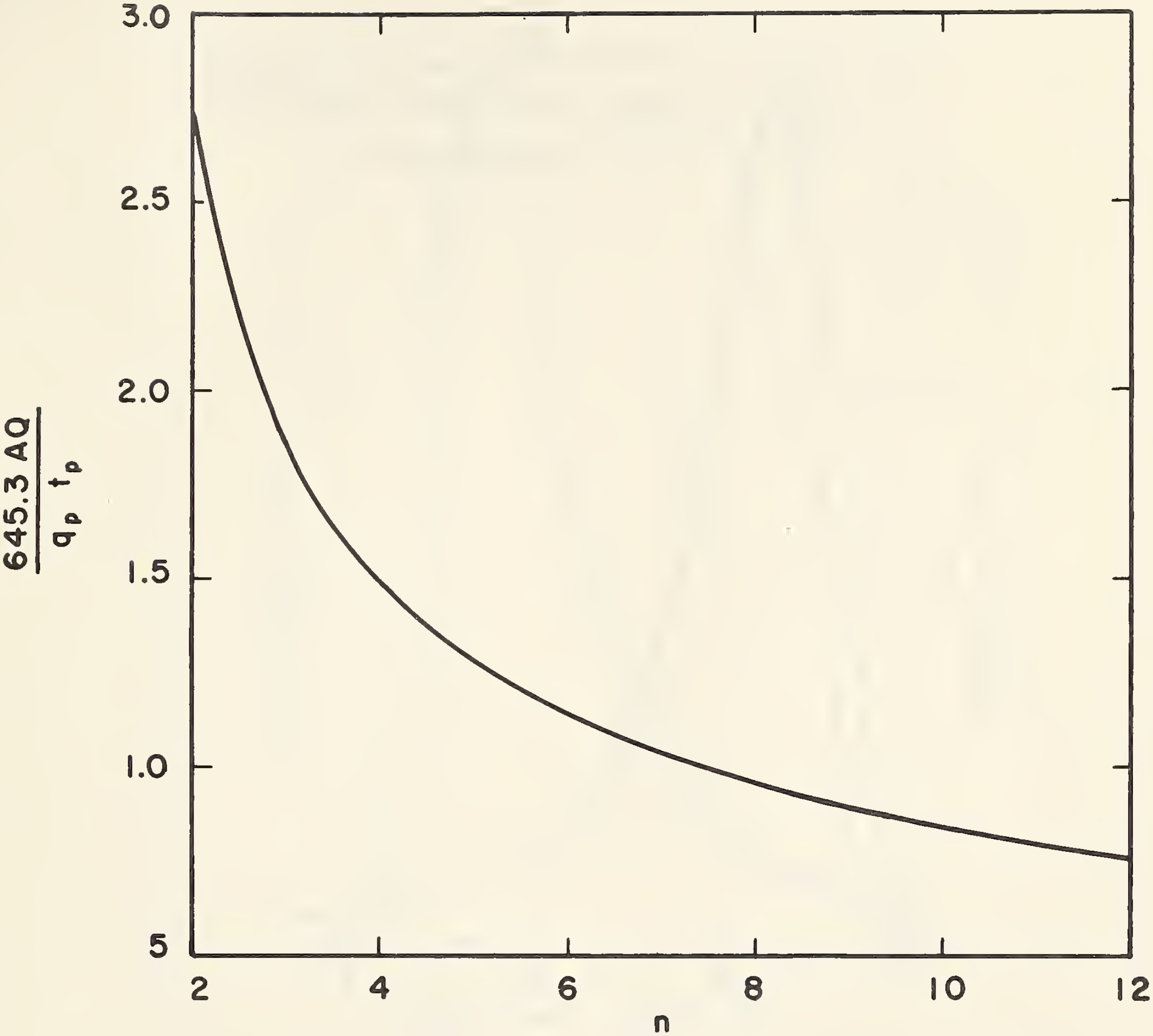


Figure 4.--Plotting of $\frac{645.3\ A_Q}{q_p\ t_p}$ $\frac{e^{(n-1)}\Gamma(n)}{(n-1)^n}$ relationship.

Equation (2) defines the slope of the recession curve and can be used to calculate the discharge rate at any time during recession when written as

$$q = q_o\ e^{-\left(\frac{t\ -\ t_e}{K}\right)} \tag{7}$$

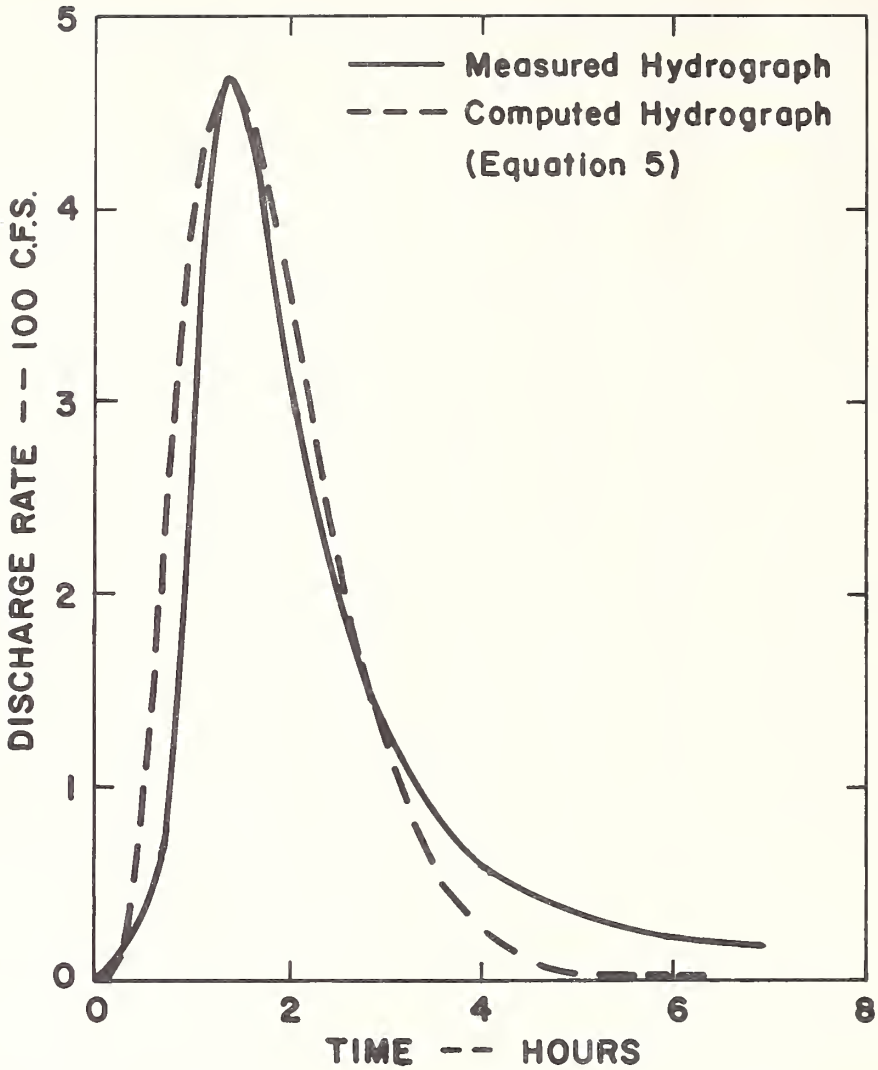


Figure 5.--Measured versus computed hydrograph for storm of February 9, 1966, subwatershed C, Brushy Creek.

When equation (7) was used to compute recession curves, the results were quite satisfactory (fig. 6). DeCoursey[6] used a similar approach to define the hydrographs from the subwatersheds of the Washita River Watershed.

This completes the solution of all the factors necessary to define a runoff hydrograph. However, this hydrograph has not been required to contain the same volume of runoff that was used

[6] See footnote 5.

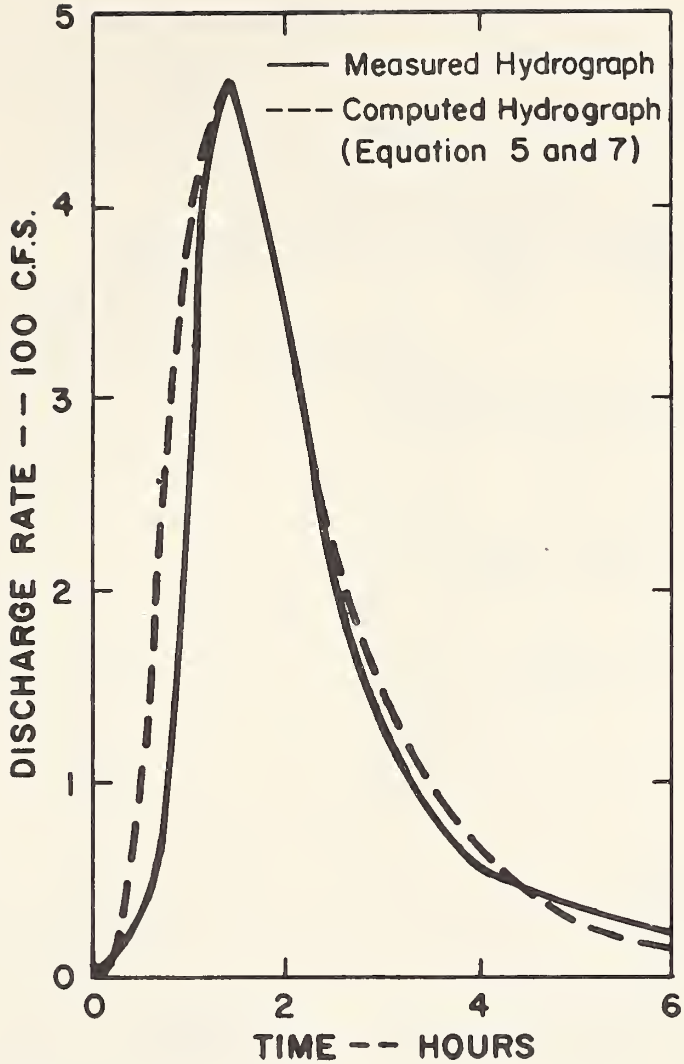


Figure 6.--Measured versus computed hydrograph for storm of February 9, 1966, subwatershed C, Brushy Creek.

in equation (1). The volume can be determined by integrating the discharge rate with respect to time

$$V = \int_0^{\infty} q \, dt$$

Equations (5) and (7) represent the values of q as follows:

$$q = q_p \left[\frac{t}{t_p}\right]^{(n-1)} \left[e^{-(n-1)}\right]^{\left(\frac{t}{t_p} - 1\right)} \qquad 0 \leqq t \leqq t_o \tag{5}$$

$$q = q_o e^{-\left(\frac{t - t_o}{K}\right)} \qquad t_o \leqq t \leqq \infty \tag{7}$$

Thus,

$$V = \int_0^{t_o} q_p \left[\frac{t}{t_p}\right]^{(n-1)} \left[e^{-(n-1)}\right]^{\left(\frac{t}{t_p} - 1\right)} dt + \int_{t_o}^{\infty} q_o\, e^{-\left(\frac{t - t_o}{K}\right)} dt \tag{8}$$

To integrate the first integral in equation (8),

let $S = \frac{t}{t_p} \quad \therefore S_o = \frac{t_o}{t_p}$

$$q = q_p (S)^{n-1} e^{-(n-1)(S-1)}$$

$$q = q_p e^{n-1} S^{n-1} e^{-(n-1)S}$$

Let $Z = q_p e^{n-1}$

$$q = Z e^{-(n-1)S} S^{n-1}$$

$$V_1 = \int_0^{S_o t_p} Z S^{n-1} e^{-(n-1)S} t_p \, dS \tag{9}$$

If n is an integer $> 1$, then a repeated integration by parts will give

$$V_1 = Z t_p e^{-(n-1)S} \left[B_o + B_1 S + B_2 S^2 + \ldots + B_{n-1} S^{n-1}\right] \Bigg|_0^{S_o t_p} \tag{10}$$

Substituting original values for S and Z,

$$V_1 = q_p t_p \left\{ e^{-(n-1)\left(\frac{t_o}{t_p} - 1\right)} \left[B_o + B_1 \left(\frac{t_o}{t_p}\right) + B_2 \left(\frac{t_o}{t_p}\right)^2 + \ldots + B_{n-1} \left(\frac{t_o}{t_p}\right)^{n-1}\right] - e^{n-1} B_o \right\} \tag{11}$$

$B_k$'s can be found by differentiating equation (10)

$$\frac{d\ V_1}{d\ S} = Z\ t_p\ e^{-(n-1)S} \left\{ -(n-1)\left[B_0 + B_1 S + B_2 S^2 + \ldots + B_{n-1} S^{n-1}\right] + \left[B_1 + 2B_2 S + \ldots + (n-1) B_{n-1} S^{n-2}\right] \right\} \tag{12}$$

By equating coefficients of powers of S, it is seen that

$$-(n-1)\ B_0 + B_1 = 0$$

$$-(n-1)\ B_1 + 2B_2 = 0$$

until finally,

$$-(n-1)\ B_{n-2} + (n-1)\ B_{n-1} = 0$$

$$-(n-1)\ B_{n-1} = 1$$

$$\therefore B_{n-1} = -\frac{1}{n-1}$$

$$B_{n-2} = B_{n-1} = -\frac{1}{n-1}$$

The volume of the hydrograph from $t_o$ to $\infty$ is obtained by integrating the second integral in equation (8).

$$V_2 = \int_{t_o}^{\infty} q_o\ e^{-\left(\frac{t-t_o}{K}\right)}\ dt$$

Let $x = t-t_o$ $\qquad dx = dt$

$$V_2 = q_o \int_{t_o}^{\infty} e^{\frac{-x}{K}}\ dx$$

$$V_2 = -K\ q_o\ e^{\frac{-x}{K}} \Big|_{t_o}^{\infty}$$

$$V_2 = -K\ q_o\ e^{-\left(\frac{t-t_o}{K}\right)} \Big|_{t_o}^{\infty}$$

$$V_2 = K\ q_o \tag{13}$$

Now adding equations (11) and (13) gives an expression for the total runoff volume in cubic feet per second-hours.

$$V = q_p\, t_p \left\{ e^{-(n-1)\left(\frac{t_o}{t_p} - 1\right)} \left[ B_o + B_1 \left(\frac{t_o}{t_p}\right) + \ldots + B_{n-1} \left(\frac{t_o}{t_p}\right)^{n-1} \right] - e^{n-1} B_o \right\} + K\, q_o \tag{14}$$

The runoff volume in inches, Q, is obtained by the equation

$$Q = \frac{V}{645.3A} \tag{15}$$

Substituting equation (15) into equation 14), gives

$$Q = \frac{q_p\, t_p \left\{ e^{-(n-1)\left(\frac{t_o}{t_p} - 1\right)} \left[ B_o + B_1 \left(\frac{t_o}{t_p}\right) + B_2 \left(\frac{t_o}{t_p}\right)^2 + \ldots + B_{n-1} \left(\frac{t_o}{t_p}\right)^{n-1} \right] - e^{n-1} B_o \right\} + K\, q_o}{645.3A} \tag{16}$$

Equation (16) can be simplified by assuming that the transition from equation (5) to equation (7) occurs at the same ratio of q to $q_p$ for all watersheds. Since equations (5) and (7) give comparable results during the beginning of the recession curve, the exact location of $t_o$ is not critical; thus the assumption can be made with very little sacrifice of accuracy. Equation (16) can now be written

$$Q = \frac{q_p\, t_p\, C + K\, q_o}{645.3A} \tag{17}$$

in which

$$C = e^{-(n-1)\left(\frac{t_o}{t_p} - 1\right)} \left[ B_o + B_1 \left(\frac{t_o}{t_p}\right) + B_2 \left(\frac{t_o}{t_p}\right)^2 + \ldots + B_{n-1} \left(\frac{t_o}{t_p}\right)^{n-1} \right] - e^{n-1} B_o \tag{18}$$

If it is assumed that $t_o$ occurs during recession at a point where $q/q_p$ is equal to 0.75, then the value of C is constant for any particular value of n. These values and the corresponding values of $t_o/t_p$ are shown in table 3 and plotted in figure 7.

TABLE 3.--Relation between C, n, and $t_o / t_p$

$$C = e^{-(n-1)\left(\frac{t_o}{t_p} - 1\right)} \left[ B_o + B_1 \left(\frac{t_o}{t_p}\right) + B_2 \left(\frac{t_o}{t_p}\right)^2 + \cdots + B_{n-1} \left(\frac{t_o}{t_p}\right)^{n-1} \right] - e^{n-1} B_o$$

Assuming $q_o = 0.75\ q_p$

| n | C | $t_o / t_p$ |
|---|---|---|
| 2 | 1.585 | 1.961 |
| 3 | 1.176 | 1.636 |
| 4 | .983 | 1.504 |
| 5 | .863 | 1.429 |
| 6 | .781 | 1.378 |
| 7 | .720 | 1.342 |
| 8 | .670 | 1.314 |
| 9 | .630 | 1.292 |
| 10 | .599 | 1.276 |
| 11 | .570 | 1.261 |
| 12 | .544 | 1.245 |

Equation (17) may be rearranged to

$$C = \frac{645.3\ AQ - Kq_o}{q_p\ t_p} \tag{19}$$

This provides a means for determining n such that the volume is required to be the same as that used in equation (1). Therefore, the runoff hydrograph from natural storms can be defined.

At this point an example will help clarify the procedure. The flood of August 12, 1966, on subwatershed G was chosen as the example. This flood was not included in the 73 floods selected to develop the procedure, because it occurred after the selection had been made.

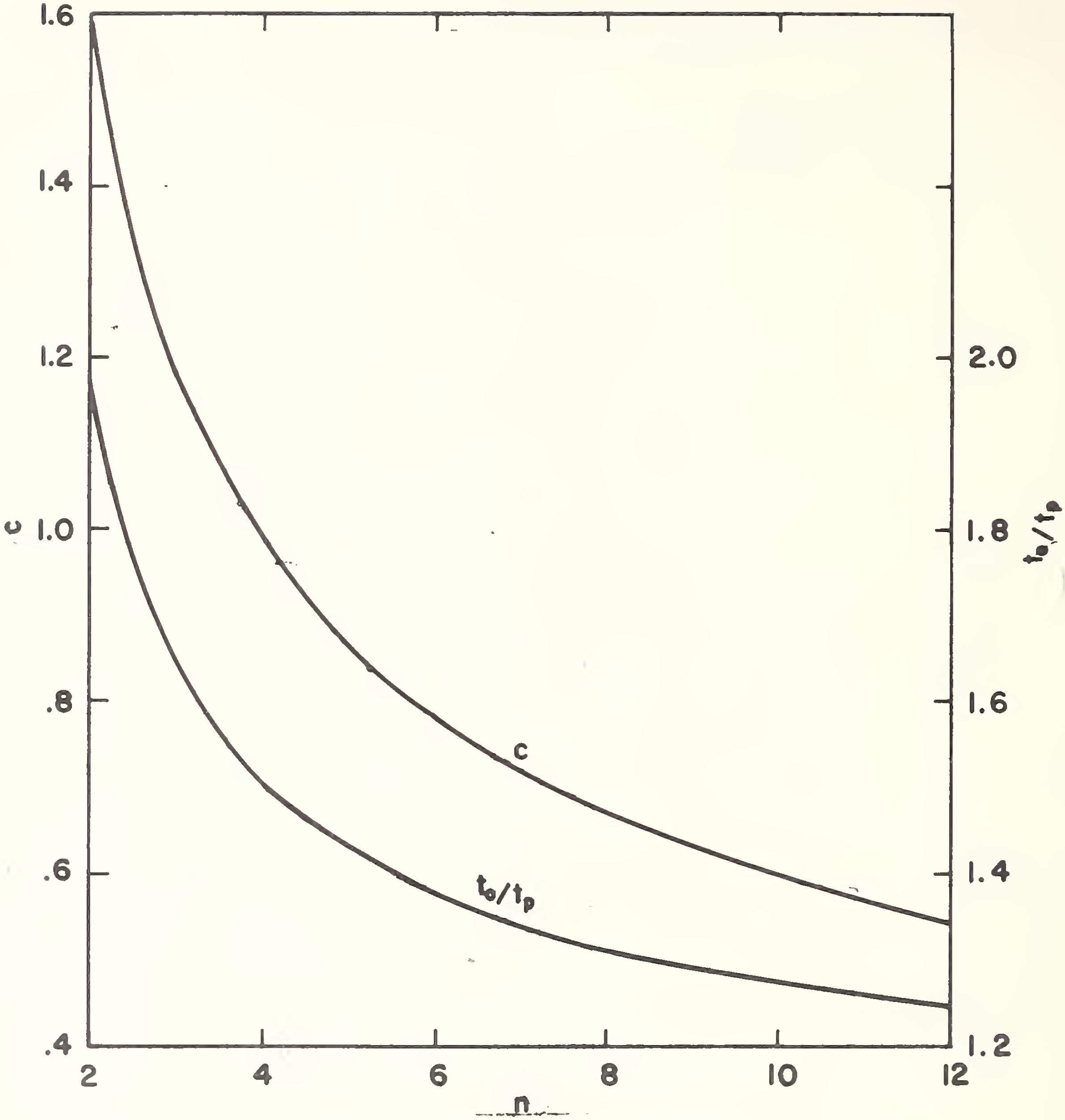


Figure 7.--Relationship of n to c and $t_o/t_p$.

The information necessary to compute the runoff hydrograph follows:

Subwatershed G, Brushy Creek, Riesel, Tex.

Drainage area = 6.84 square miles

Length of main stem = 6.52 miles

Slope of main stem = 0.0034 ft./ft.

Elongation ratio = 0.60

Rainfall intensity = 1.19 in./hr.

Runoff volume = 2.10 in.

The recession constant K is computed with equation (3).

$$K = 0.002044\, L^{0.52}\, S^{-1.263}\, E^{1.780}$$

$$K = 0.002044\, (6.52)^{0.52}\, (0.0034)^{-1.263}\, (0.60)^{1.780}$$

$$K = 2.86 \text{ hours}$$

The peak discharge is computed with equation (1).

$$q_p = 369\, Q^{0.686}\, A^{0.787}\, I^{0.225}\, K^{-0.412}$$

$$q_p = 369\, (2.10)^{0.686}\, (6.84)^{0.787}\, (1.19)^{0.225}\, (2.86)^{-0.412}$$

$$q_p = 1890 \text{ c.f.s.}$$

The time to peak is computed with equation (4).

$$t_p = 0.144\, L^{0.935}\, S^{-0.369}\, E^{1.486}$$

$$t_p = 0.144\, (6.52)^{0.935}\, (0.0034)^{-0.369}\, (0.60)^{1.486}$$

$$t_p = 3.16 \text{ hours}$$

The rising limb and the crest of the hydrograph are defined by equation (5).

C is determined by equation (19).

$$C = \frac{645.3\, AQ - Kq_o}{q_p\, t_p}$$

$$C = \frac{(645.3 \times 6.84 \times 2.10) - (2.86 \times 0.75 \times 1890)}{1890 \times 3.16}$$

$$C = 0.873$$

Figure 7 is used to find an n value of 4.9, and a corresponding $t_o/t_p$ value of 1.435.

Equations (5) and (7) can now be solved with an n value of 4.9, $q_p$ = 1890 c.f.s., $t_p$ = 3.16 hours, $t_o$ = 4.53 hours, and K = 2.86 hours.

This completes the computations necessary to define the hydrograph. Figure 8 shows a comparison of the measured hydrograph to the computed hydrographs for the storm of August 12, 1966, for subwatershed G, Brushy Creek.

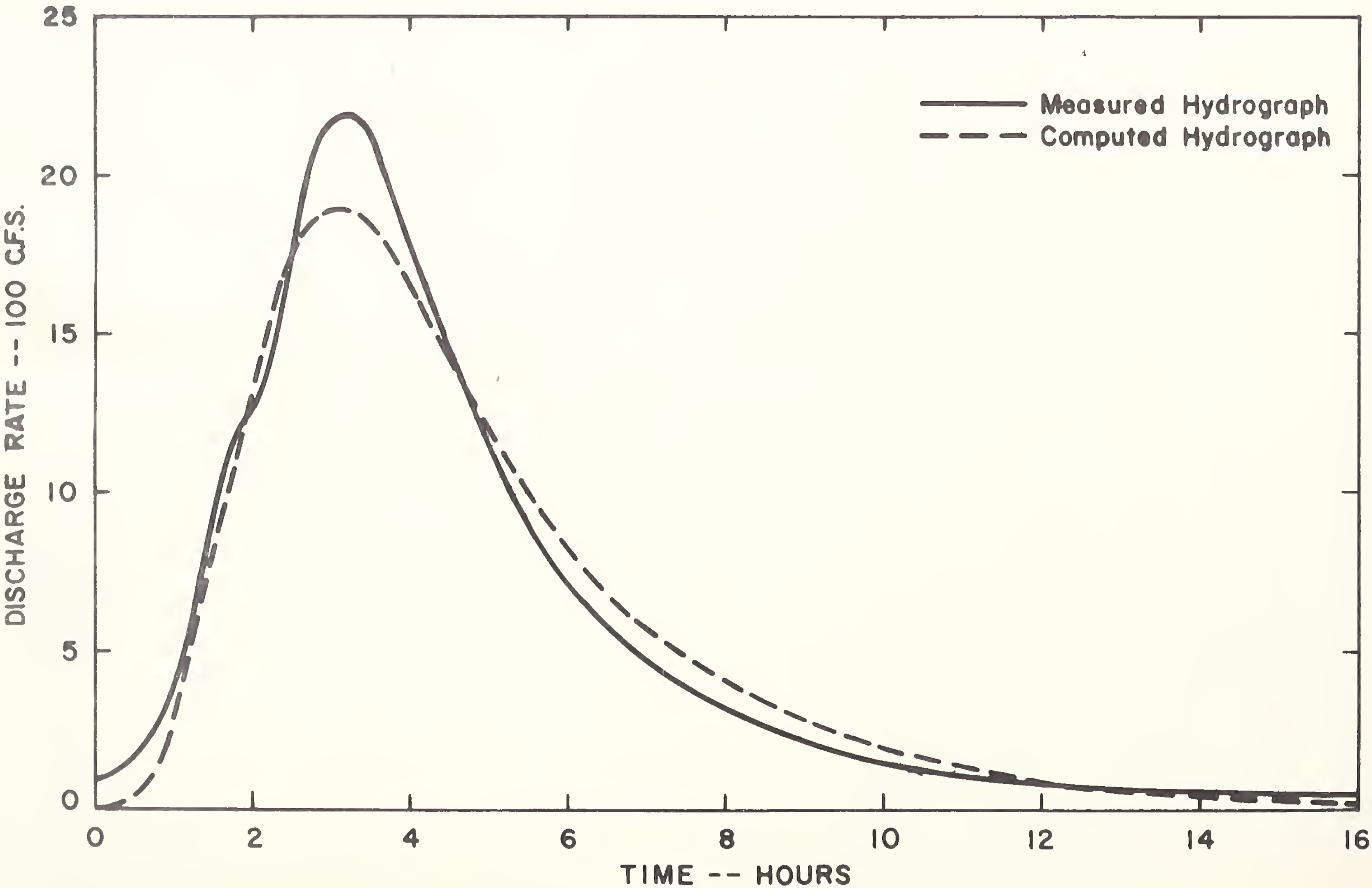


Figure 8.--Measured versus computed hydrograph for storm of August 12, 1966, for subwatershed G, Brushy Creek.

## APPLICATION OF EQUATION 1 TO OTHER WATERSHEDS

Four small Blackland watersheds were used to test equation (1). These watersheds and the periods of record studied are Honey Creek Site No. 11, 1957-64; Honey Creek Site No. 12, 1957-64; Cow Bayou Site No. 4, 1957-59; and Pin Oak Creek, 1957-62.

The peak discharges computed with equation (1) compared satisfactorily with the measured peak discharges on these four watersheds. However, it was felt that an equation for predicting peak discharge based on data from all nine watersheds would have wider application. A multiple regression analysis of data from all nine watersheds yielded the equation

$$q_p = 366\, Q^{0.695}\, A^{0.774}\, I^{0.220}\, K^{-0.416} \tag{20}$$

The close comparison between equations (20) and (1) indicates that equation (1) is applicable to other small Blackland watersheds. Figure 9 shows a plotting of actual peak discharge versus peak discharge computed with equation (20) for all nine watersheds.

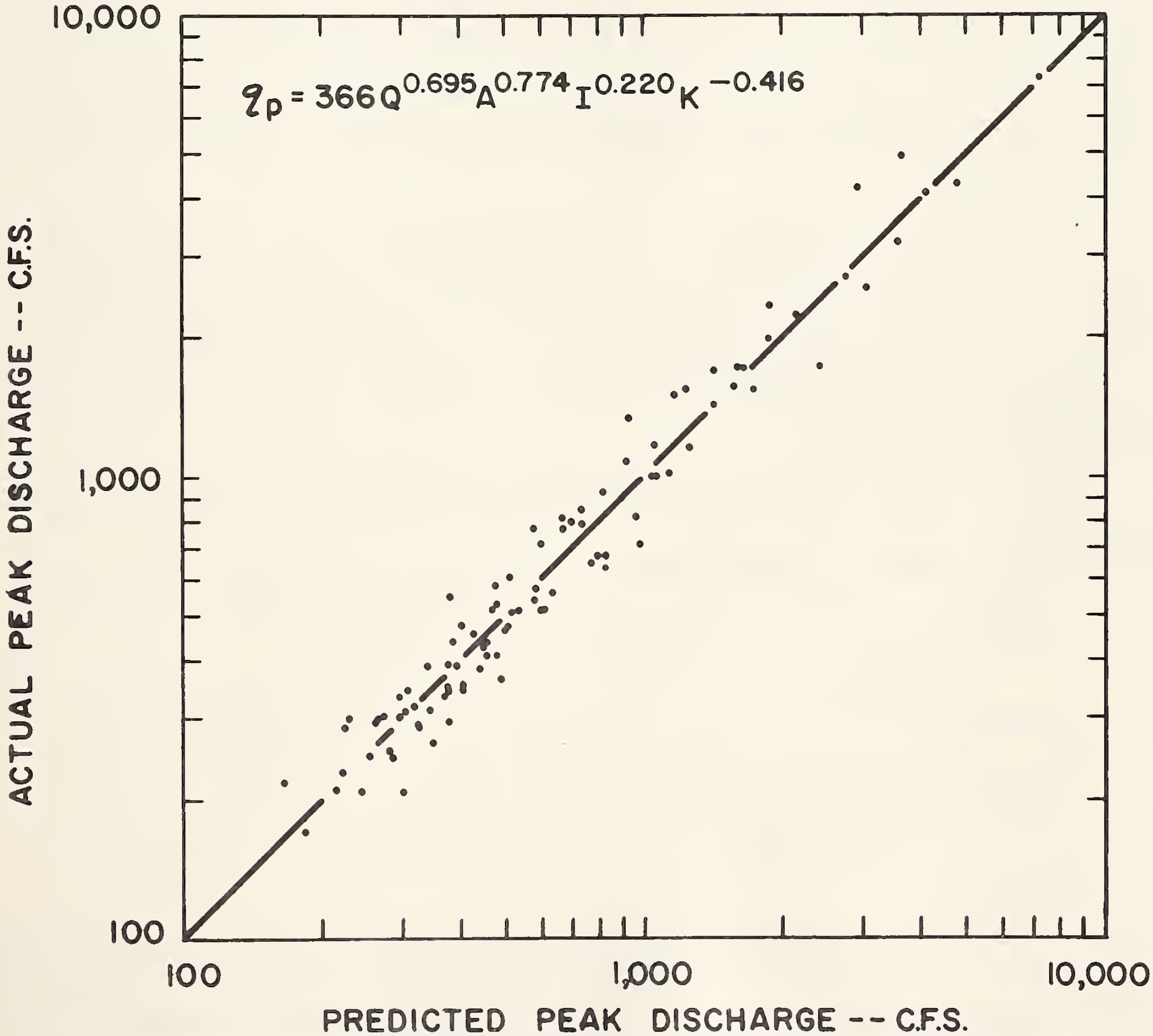


Figure 9.--Measured peak discharge versus peak discharge computed with equation (20) for nine small Blackland watersheds.

Flood control structures must be designed to control the most extreme flood. This is the flood that produces the highest peak discharge for a particular runoff volume from a watershed.

The runoff hydrographs computed by the procedure described in this paper will closely approximate most single-peak floods occurring on small Blackland watersheds. Therefore, this procedure is quite useful in developing hydrographs for flood routing natural storms. However, since equations (1) and (20) were developed by multiple regression, they tend to give average values of peak discharge rather than maximum values.

Equation (17) may be rewritten in the form

$$q_p = \frac{645.3\ AQ - Kq_o}{t_p\ C} \tag{21}$$

This equation gives maximum values of peak discharge for any particular runoff volume from a watershed, since the equations for computing K and $t_p$ were developed from extreme hydrographs. $q_p$ is computed by the same relationship used to calculate C in equation (19). Therefore, some other means must be used to determine C in equation (21). Wu, Delleur, and Diskin[7] have shown that n is related to the ratio of K to $t_p$ for design hydrographs for small watersheds in Indiana. Their relationship, shown in figure 10, can be used to determine the value of n, which in turn is used to determine C from figure 7. A comparison of predicted peak discharge to measured peak discharge for the nine watersheds is shown in figure 11. In general, equation (21) predicts higher than the measured peaks. This is simply because most of the hydrographs are not extreme events.

As an example of the development of a design hydrograph, consider the storm of April 24, 1957, on the Cow Bayou watershed. The ratio of the peak discharge to the runoff volume of this flood was one of the highest observed on this watershed. Therefore, the hydrograph could be used as a design hydrograph if the runoff volume agreed with the design frequency runoff volume. In order to check the design procedure with this hydrograph, assume that the 1.90 inches of measured runoff is the runoff volume for the design frequency.

With this assumption, proceed with the computations. The recession constant K, computed with equation (3), is 0.49 hour.

A time to peak of 1.72 hours is computed with equation (4).

Figure 10 is used to determine an n value of 12. When this value of n is entered into figure 7, C is found to be 0.544.

The peak discharge is computed with equation (21).

$$q_p = \frac{645.3\ AQ - Kq_o}{t_p\ C}$$

$$q_o = 0.75\ q_p$$

$$q_p = \frac{645.3 \times 5.25 \times 1.9 - 0.49 \times 0.75\ q_p}{1.72 \times 0.544}$$

$$q_p = 4938 \text{ c.f.s.}$$

The shape of the hydrograph is computed with equations (5) and (7) as shown in the earlier example.

A comparison of the measured to the computed hydrograph is shown in figure 12.

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[7] See footnote 4.

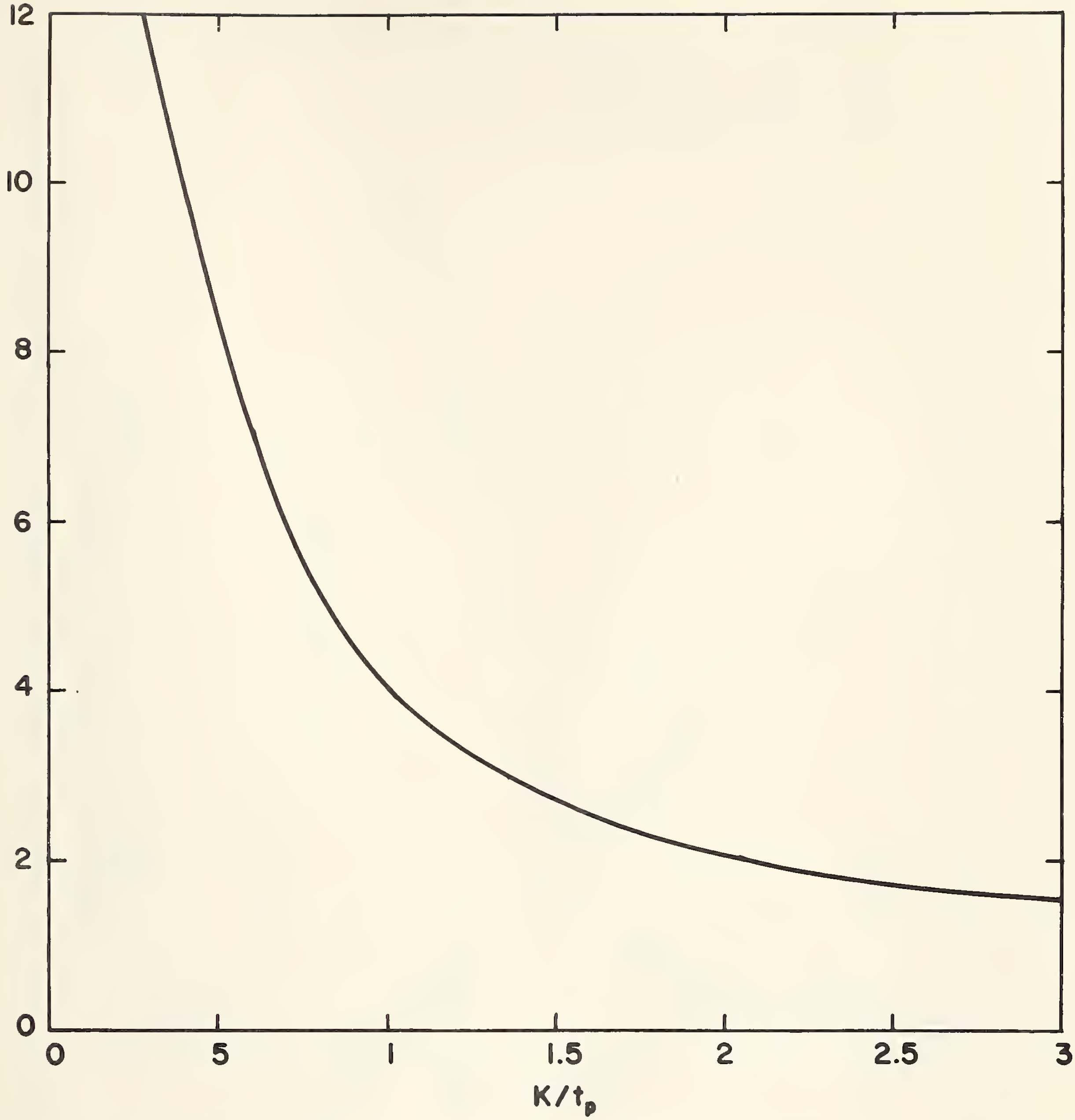


Figure 10.--Relationship between dimensionless recession constant and hydrograph parameter.

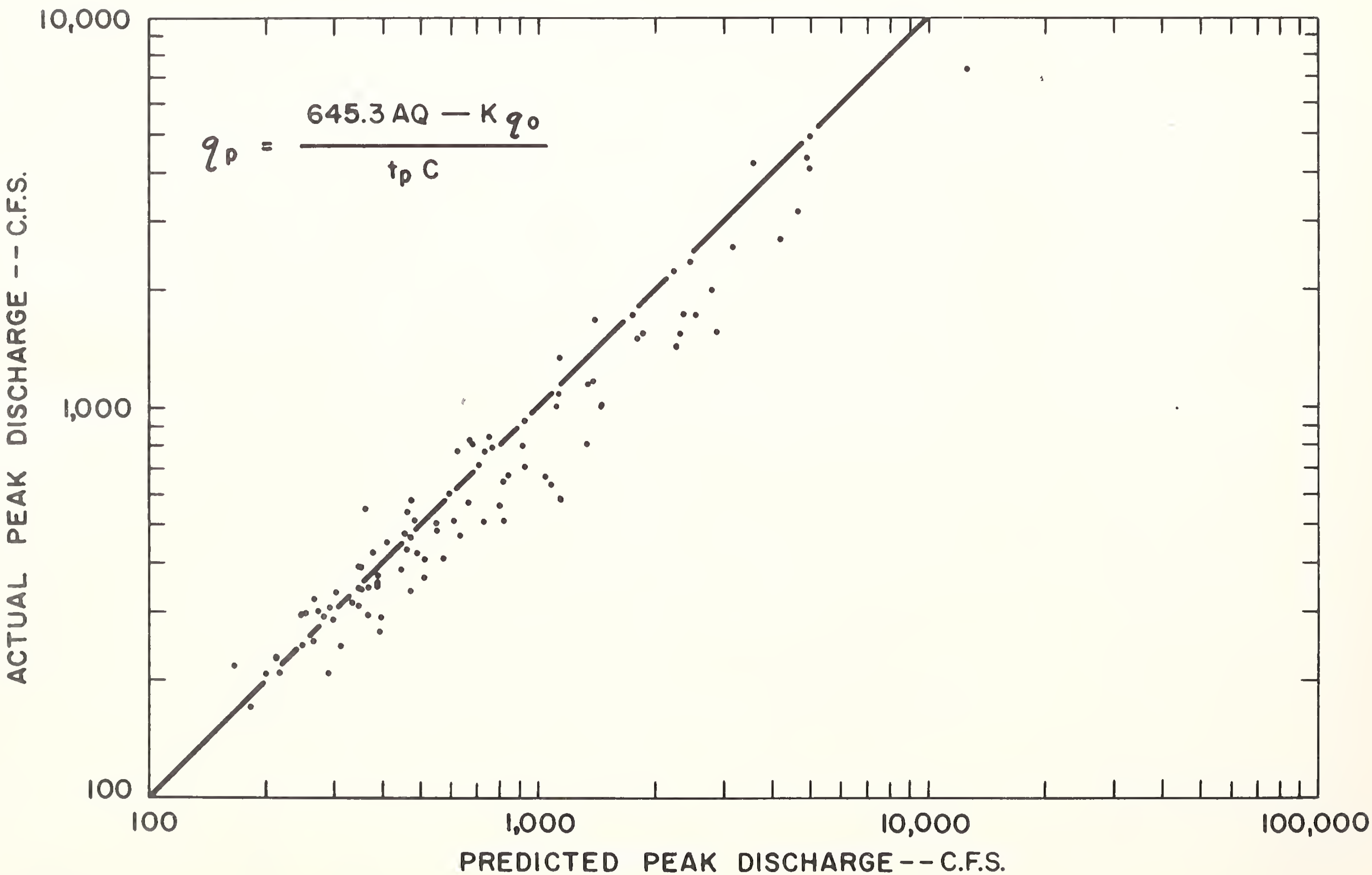


Figure 11.--Measured peak discharge versus peak discharge computed with equation (21) for nine small Blackland watersheds.

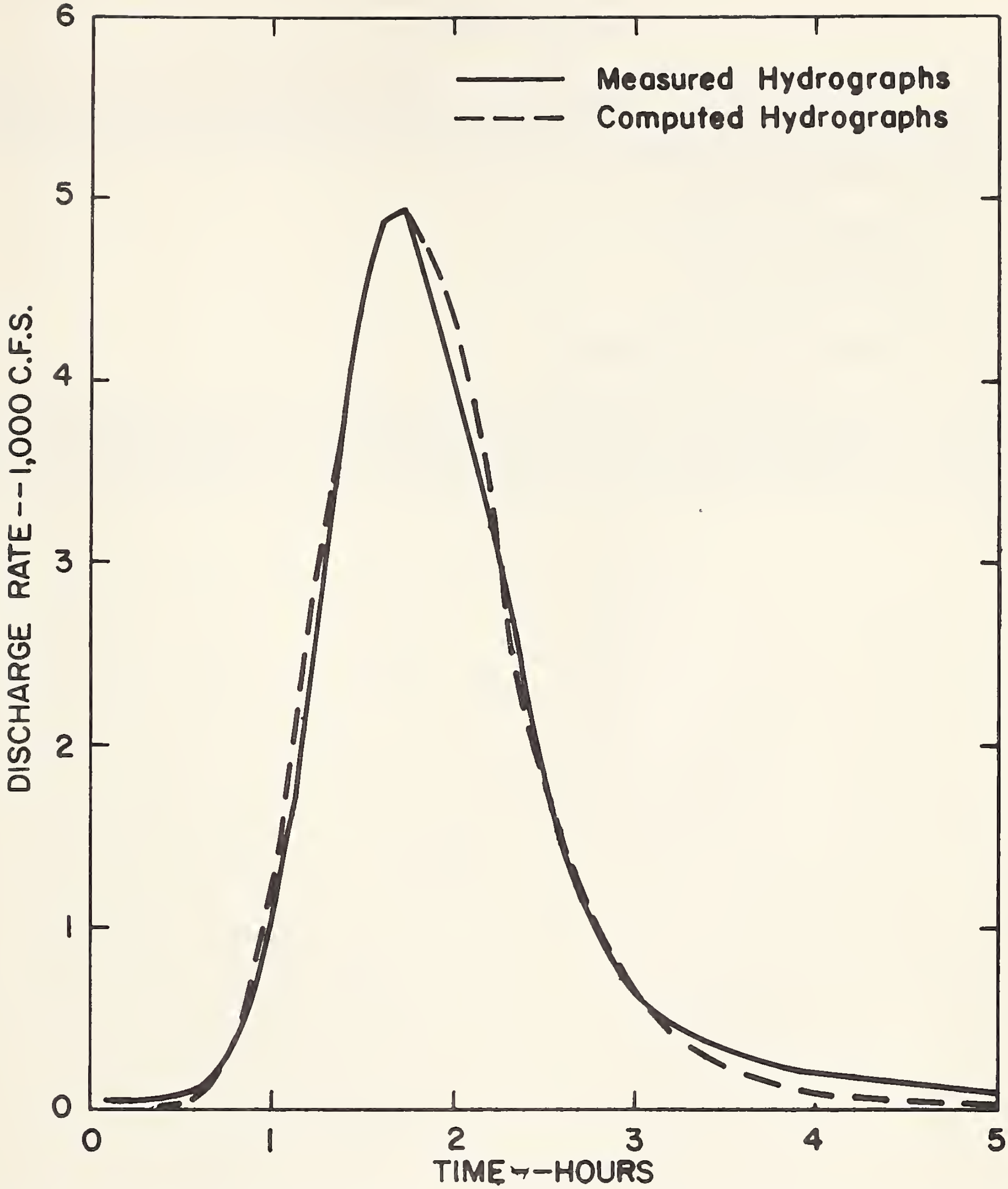


Figure 12.--Measured and computed design hydrographs for storm of April 24, 1957, Cow Bayou Watershed.

## CONCLUSIONS

Runoff hydrographs for ungaged watersheds, ranging in size from 0.275 to 17.6 square miles, in the Blacklands of Texas can be estimated with considerable accuracy by employing certain empirical relationships. These relationships, based on a study of gaged watersheds in the Blacklands, provide estimates of:

(1) The peak rate of runoff,

(2) the time to peak, and

(3) the ordinates of the hydrographs.

These relationships require an independent estimate of the volume of runoff; the rainfall intensity for a time period equal to the recession constant; and measurements of certain dimensions of the watershed including area, length of the main stem, slope of the main stem and the elongation ratio.

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