Prepared in cooperation with the Harris County Flood Control District and the Texas Department of Transportation

A Method for Estimating Peak and Time of Peak Streamflow from Excess Rainfall for 10- to 640-Acre Watersheds in the Houston, Texas, Metropolitan Area

Scientific Investigations Report 2011–5104

U.S. Department of the Interior
U.S. Geological Survey

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Conversion Factors

Multiply	By	To obtain
	Length	
inch (in.)	25.4	millimeter (mm)
foot (ft)	.3048	meter (m)
mile (mi)	1.609	kilometer (km)
	Area	
square mile (mi^2)	2.590	square kilometer (km^2)
acre (acre)	.001563	square mile (mi^2)
acre (acre)	.004047	square kilometer (km^2)
	Flow	
cubic foot per second (ft^3/s)	.02832	cubic meter per second (m^3/s)

Datum

Horizontal coordinate information is referenced to the North American Datum of 1983 (NAD 83).
Vertical coordinate information is referenced to the North American Vertical Datum of 1988 (NAVD 88).

A Method for Estimating Peak and Time of Peak Streamflow from Excess Rainfall for 10- to 640-Acre Watersheds in the Houston, Texas, Metropolitan Area

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Abstract

Estimates of peak and time of peak streamflow for small watersheds (less than about 640 acres) in a suburban to urban, low-slope setting are needed for drainage design that is cost-effective and risk-mitigated. During 2007–10, the U.S. Geological Survey (USGS), in cooperation with the Harris County Flood Control District and the Texas Department of Transportation, developed a method to estimate peak and time of peak streamflow from excess rainfall for 10- to 640-acre watersheds in the Houston, Texas, metropolitan area. To develop the method, 24 watersheds in the study area with drainage areas less than about 3.5 square miles (2,240 acres) and with concomitant rainfall and runoff data were selected. The method is based on conjunctive analysis of rainfall and runoff data in the context of the unit hydrograph method and the rational method. For the unit hydrograph analysis, a gamma distribution model of unit hydrograph shape (a gamma unit hydrograph) was chosen and parameters estimated through matching of modeled peak and time of peak streamflow to observed values on a storm-by-storm basis. Watershed mean or watershed-specific values of peak and time to peak ("time to peak" is a parameter of the gamma unit hydrograph and is distinct from "time of peak") of the gamma unit hydrograph were computed. Two regression equations to estimate peak and time to peak of the gamma unit hydrograph that are based on watershed characteristics of drainage area and basin-development factor (BDF) were developed. For the rational method analysis, a lag time (time-R), volumetric runoff coefficient, and runoff coefficient were computed on a storm-by-storm basis. Watershed-specific values of these three metrics were computed. A regression equation to estimate time-R based on drainage area and BDF was developed. Overall arithmetic means of volumetric runoff coefficient (0.41 dimensionless) and runoff coefficient (0.25 dimensionless) for the 24 watersheds were used to express the rational method in terms of excess rainfall (the excess rational method). Both the unit hydrograph method and excess rational method are shown to provide similar estimates of peak and time of peak streamflow. The results from the two methods can be combined by using arithmetic means. A nomograph is provided that shows the respective relations between the arithmetic-mean peak and time of peak streamflow to drainage areas ranging from 10 to 640 acres. The nomograph also shows the respective relations for selected BDF ranging from undeveloped to fully developed conditions. The nomograph represents the peak streamflow for 1 inch of excess rainfall based on drainage area and BDF; the peak streamflow for design storms from the nomograph can be multiplied by the excess rainfall to estimate peak streamflow. Time of peak streamflow is readily obtained from the nomograph. Therefore, given excess rainfall values derived from watershed-loss models, which are beyond the scope of this report, the nomograph represents a method for estimating peak and time of peak streamflow for applicable watersheds in the Houston metropolitan area. Lastly, analysis of the relative influence of BDF on peak streamflow is provided, and the results indicate a $0.04\log_{10}$ cubic feet per second change of peak streamflow per positive unit of change in BDF. This relative change can be used to adjust peak streamflow from the method or other hydrologic methods for a given BDF to other BDF values; example computations are provided.

Introduction

Estimation of peak and time of peak streamflow from design storms provides for cost-effective, risk-mitigated design of drainage structures such as bridges, culverts, roadways, and other infrastructure. Relevant guidelines or manuals, which provide further context related to infrastructure design, can be found in Texas Department of Transportation (2002) or Harris County Flood Control District (2004).

Accordingly, during 2007–10, the U.S. Geological Survey (USGS), in cooperation with the Harris County Flood Control District (HCFCD) and the Texas Department of Transportation (TxDOT), developed a method for estimating peak and time of peak streamflow from excess rainfall for 10- to 640-acre watersheds in the Houston, Texas, metropolitan area.

Purpose and Scope

The primary purpose of this report is to present a method for estimating peak and time of peak streamflow from excess rainfall for 10- to 640-acre watersheds in the Houston, Texas, metropolitan area that is based on watershed characteristics of drainage area and basin-development factor. A secondary purpose is to report on the conjunctive analysis of rainfall and runoff data in the context of the unit hydrograph and the rational method for 24 watersheds in the Houston metropolitan area. There are three major components of this report:

1. A comprehensive summary of the unit hydrograph method and statistical results from analysis for the 24 watersheds is provided in the section titled "Analysis of Gamma Unit Hydrographs for the 24 Watersheds." The section documents two equations that parameterize a gamma unit hydrograph, and each is used in the development of the method described in component 3.

2. A comprehensive summary of the rational method and statistical results from analysis for the 24 watersheds is provided in the section titled "Analysis of Rational Method for the 24 Watersheds." The section documents an equation and appropriate runoff coefficient values, which are used in the development of the method described in component 3.

3. The method is based on conjunctive analysis of rainfall and runoff data in the context of the unit hydrograph and the rational method for the watersheds and is presented in section titled "A Method for Estimating Peak and Time of Peak Streamflow from Excess Rainfall for 10- to 640-Acre Watersheds in the Houston, Texas, Metropolitan Area." The primary result of the conjunctive analysis is a nomograph. Example computations involving the nomograph also are provided.

Study Watersheds

For this investigation, 21 distinct watersheds (based on latitude and longitude) were identified as pertinent for the Houston metropolitan area. Pertinent watersheds were selected on the basis of drainage area as a measure of watershed size. These watersheds represent generally the smallest watersheds for which there exist paired rainfall and runoff data suitable for conjunctive analysis of the unit hydrograph method and the rational method. Although the focus of the investigation is the unit hydrograph method and the rational method in the context of small watersheds (less than about 640 acres), to support statistical development, watersheds with drainage areas less than about 3.5 square miles (2,240 acres) were selected.

Two of the watersheds were identified as having considerable changes in extent of land development as expressed by a basin-development factor. The period of record for the two watersheds was thus segregated (three total divisions), resulting in the 24 watersheds that are used in this report. The 24 watersheds and ancillary characteristics are listed in table 1 and shown in figure 1. Collectively, these 24 watersheds are assumed to represent the generalized hydrologic and hydraulic conditions of many small, low-slope watersheds in the Houston metropolitan area.

Database of Rainfall and Runoff

A database of rainfall and runoff data from about 1965 through about 2006 for the 24 watersheds was compiled and converted to digital format as needed. The data for the 24 watersheds were obtained from various sources including Liscum and others (1996) and Liscum (2001) and from selected non-USGS rainfall-runoff data (Fred Liscum, PBS&J Inc. [now (2011) with HCFCD], written commun., 2008; and Steve Johnson, LJA Engineering Inc.; written commun., 2008). These collective data used here are stored in files available from the Texas Water Science Center upon request.

An example of headers (definitions) and selected data parts of two data files for a selected storm for one of the study watersheds is shown in figure 2. The definitions in the figure are identical to those described in Asquith, Thompson, and others (2004, p. 11 and 17). For the rainfall data, the time stamps, which are evenly spaced in the example (but not universally in the data), are shown under the DATE_TIME field. The cumulative depth of weighted (WTD) rainfall among all available rain gages is available under the ACCUM_WTD_PRECIP field in units of inches. For the particular example, a single rain gage (PRECIP1) was operated.

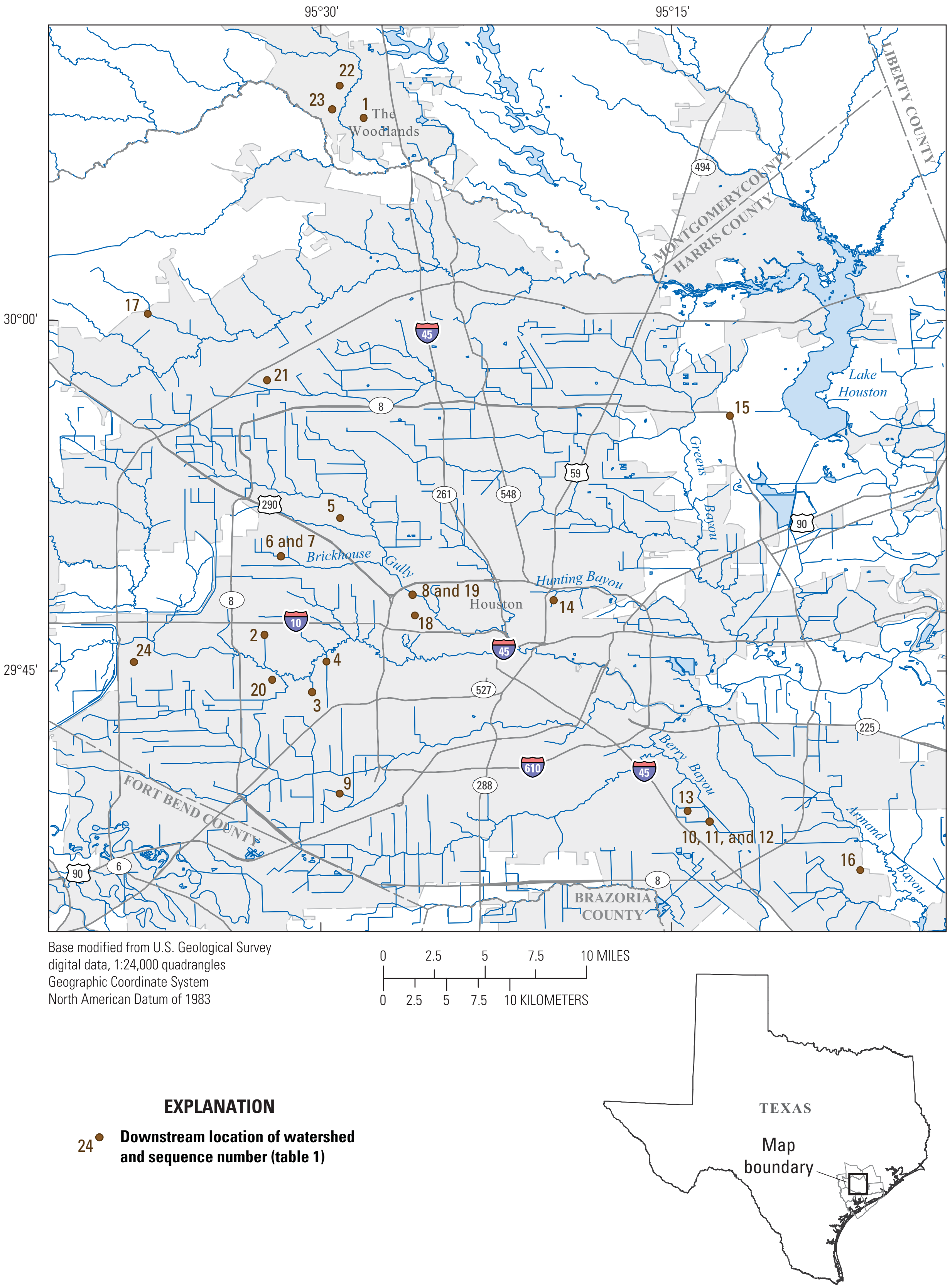


Figure 1. Map showing downstream locations of the 24 selected watersheds in the Houston, Texas, metropolitan area used for gamma unit hydrograph and rational method analysis.

Table 1. Summary of 24 selected watersheds in the Houston, Texas, metropolitan area used for gamma unit hydrograph and rational method analysis.

[no., number; fig., figure; *A*, drainage area (contributing), *L*, main-channel length; *S*, dimensionless main-channel slope; *BDF*, basin-development factor; --, not available; St., Street; Dr., Drive; trib., tributary; BMP, Best-management practice; Pkwy., Parkway; Rd., Road; NPDES, National Pollutant Discharge Elimination System]

Site no. used in fig. 1	Station no. and calendar year range of storms	Station name	Latitude	Longitude	*A* (square miles)	*L* (miles)	*S* (dimensionless)	*BDF* (dimensionless)
1	08068438: 1975–87	Swale no. 8 at Woodlands, Tex.	30°08'38"	95°28'09"	0.55	0.74	0.0077	8
2	08073630: 1979–85	Bettina St. Ditch, Houston, Tex.	29°46'32"	95°32'23"	1.37	.73	.0041	11
3	08073750: 1967–72	Stoney Brook St. Ditch, Houston, Tex.	29°44'05"	95°30'22"	.50	--	--	10
4	08073800: 1964–72	Bering Ditch at Woodway Dr., Houston, Tex.	29°45'22"	95°29'44"	2.77	1.35	.0019	9
5	08074145: 1980–82	Bingle Rd. storm sewer, Houston, Tex.	29°51'31"	95°29'09"	.21	.70	.0011	9
6	08074200: 1965–75	Brickhouse Gully at Clarblak St., Houston, Tex.	29°49'53"	95°31'42"	2.56	.85	.0030	3
7	08074200: 1978–84	Brickhouse Gully at Clarblak St., Houston, Tex.	29°49'53"	95°31'42"	2.56	.85	.0030	6
8	08074400: 1980–82	Lazybrook St. storm sewer, Houston, Tex.	29°48'15"	95°26'04"	.13	.69	.0047	12
9	08074910: 1979–83	Hummingbird St. Ditch, Houston, Tex.	29°39'44"	95°29'11"	.32	1.45	.0008	9
10	08075550: 1966–72	Berry Bayou at Gilpin St., Houston, Tex.	29°38'32"	95°13'22"	3.26	3.34	.0018	5
11	08075550: 1974–78	Berry Bayou at Gilpin St., Houston, Tex.	29°38'32"	95°13'22"	2.87	3.34	.0018	7
12	08075550: 1979–84	Berry Bayou at Gilpin St., Houston, Tex.	29°38'32"	95°13'22"	2.56	3.34	.0018	7
13	08075600: 1965–72	Berry Bayou trib. at Globe St., Houston, Tex.	29°39'00"	95°14'18"	1.58	2.27	.0017	5
14	08075750: 1965–72	Hunting Bayou trib. at Cavalcade St., Houston, Tex.	29°48'00"	95°20'02"	1.20	1.78	.0013	4
15	00000BW8: 2004–06	BMP Beltway 8 mitigation bank near Greens Bayou, Houston, Tex.	29°55'54"	95°12'29"	.14	--	--	11
16	0000B504: 2005–06	BMP Basin 504 on Armand Bayou, Houston, Tex.	29°36'28"	95°06'55"	.19	1.03	.0010	9
17	0000K542: 2005–06	BMP K542 Eldridge Pkwy. at Louetta Rd., Houston, Tex.	30°00'15"	95°37'23"	.18	.81	.0010	12
18	011NS004: 2004–06	NPDES 011NS004 West 11th St., Houston, Tex.	29°47'21"	95°25'58"	.36	--	--	9
19	0LBNS001: 2000–02	NPDES 0LBNS001 Lazybrook storm sewer at White Oak Bayou, Houston, Tex.	29°48'15"	95°26'04"	.10	--	--	12
20	0TGNS002: 2000–02	NPDES 0TGNS002 Tanglewilde at Houston, Tex.	29°44'36"	95°32'04"	.06	--	--	12
21	0WBNS003: 2004–05	NPDES 0WBNS003 Willowbrook Mall near Greens Rd., Houston, Tex.	29°57'25"	95°32'17"	.13	--	--	12
22	X8068420: 1985–1986	Ditch C at Wedgewood Lake, Woodlands, Tex.	30°10'00"	95°29'10"	.33	.93	.0041	0
23	X8068426: 1985–1986	Ditch A at Woodlands Parkway, Woodlands, Tex.	30°09'00"	95°29'30"	.41	1.6	.0030	7
24	0BFDN000: 1992	Briar Forest Dr. storm sewer at Houston, Tex.	29°45'21"	95°37'59"	.07	--	--	4

For other storms or watersheds, two or more rain gages might have been operated, and these data also are available within the database. The rainfall used for data processing herein is always provided in the `ACCUM_WTD_PRECIP` field.

For the runoff data, the time stamps, which also are evenly spaced in the example but not universally in the database, are shown under the `DATE_TIME` field. The time stamps within the separate rainfall and runoff data files for the same storm might not have a one-to-one correspondence. The streamflow in cubic feet per second is listed under the `RUNOFF` field. The cumulative volume of runoff in inches is provided in the `ACCUM_RUNOFF` field.

Selected Watershed Characteristics

Selected characteristics for the 24 watersheds were obtained from various sources including Liscum and others (1996) and from colleagues (Duane Barrett, R.G. Miller Engineers Inc.; Fred Liscum, PBS&J Inc. [now (2011) with HCFCD], and Steve Johnson, LJA Engineering Inc.; written commun., 2008). The characteristics include drainage area (contributing), main-channel length, dimensionless main-channel slope, and basin-development factor:

1. Values for drainage area (contributing) A for each watershed were obtained and are listed in table 1.

2. Values for main-channel length L were obtained and are listed in table 1. The L is defined as the length in stream-course miles of the longest defined channel from the approximate watershed headwaters to the outlet. There is considerable ambiguity in estimation of L in the Houston metropolitan area where small watersheds can contain interconnected channels, ditches, streets, and storm sewers. As a result, some lengths could not be precisely quantified and are not listed in table 1.

3. Values for dimensionless main-channel slope S (feet per feet) were obtained and are listed in table 1. The S is defined as the change in elevation ΔE in feet between the two end points of L divided by L in feet: $S = \Delta E/(5{,}280 \times L)$. Because of the ambiguity in estimation of L, some slopes could not be precisely quantified and are not listed in table 1.

4. Values for the dimensionless basin-development factor BDF were obtained and are listed in table 1. BDFs are integers from 0 to 12. There are 13 BDF categories resulting in 12 discrete changes in BDF, from $0 \rightarrow 1$, $1 \rightarrow 2$, and so forth through $11 \rightarrow 12$. BDF as a concept and definition for this report is described in appendix 1.

Analysis of Gamma Unit Hydrographs for the 24 Watersheds

Background

The unit hydrograph method (Dingman, 2002) estimates the runoff hydrograph given an excess rainfall hyetograph. Excess rainfall is a volume of rainfall per unit area (depth) after watershed losses such as evaporation, infiltration, and depression storage are subtracted (Chow and others, 1988, p. 135). A unit hydrograph is defined as the runoff hydrograph that results from a unit pulse of excess rainfall uniformly distributed over a watershed at a constant rate for a specific duration (Chow and others, 1988, p. 213).

Extensive investigations of the unit hydrograph method with Texas data and gamma distributions as the form of the unit hydrograph (Haan and others, 1994, p. 79) are available in Asquith and others (2005) and Asquith and Roussel (2007). In an associated study, Cleveland and others (2006) documented an independent analysis of unit hydrographs for Texas. A gamma distribution form of the unit hydrograph is referred to as a gamma unit hydrograph. For this report, an analysis of gamma unit hydrographs for the 24 watersheds was made. Results of the gamma unit hydrograph analysis are listed in table 2 (columns 2–5).

Analysis

In total, 317 data files were considered for the 24 watersheds. The number of storms (discrete peaks) analyzed per watershed and per methods of this report is listed in table 2. These numbers were not used as weights in statistical computations, such as weight factors in weighted-least squares regression, because of the absence of numerical similarity in the “No. of storms . . . ” (three separate columns) between the USGS monitored watersheds (the first 14 entries in the table) and the remaining watersheds.

For each watershed, unit hydrographs were generated from the rainfall and runoff data by using a custom modeling technique developed for Asquith and others (2005); Asquith and Roussel (2007). The technique involved an analyst-directed approach for 5-minute gamma unit hydrograph estimation. The output consisted of graphics (not

Table 2. Summary of storm counts and watershed-mean parameters of gamma unit hydrograph analysis and watershed-mean parameters for rational method analysis of 24 selected watersheds in the Houston, Texas, metropolitan area.

[no., number; GUH, gamma unit hydrograph analysis; q_p, peak streamflow; T_p, time to peak; K, gamma hydrograph shape; C_v, volumetric runoff coefficient; C_r, runoff coefficient; T_c, time of concentration]

Station no. and calendar year range of storms	No. of storms for GUH	q_p (inches per hour)	T_p (hours)	K (dimensionless)	No. of storms for C_v	No. of storms for C_r	T_c (hours)	C_v (dimensionless)	C_r (dimensionless)
08068438: 1975–87	11	0.4602	0.9091	1.2529	15	15	1.2625	0.5862	0.3521
08073630: 1979–85	21	.4935	.8333	1.2157	25	25	1.2952	.5494	.3650
08073750: 1967–72	12	.5242	.2847	.2439	14	14	.6095	.4866	.3789
08073800: 1964–72	26	.3450	.5641	.3580	18	15	2.1211	.6398	.4225
08074145: 1980–82	20	1.6242	.5083	4.4461	35	35	.4809	.5250	.4944
08074200: 1965–75	18	.1537	2.3610	.9768	19	17	4.4618	.3531	.0968
08074200: 1978–84	21	.1991	.9048	.3193	32	30	1.8834	.3446	.1761
08074400: 1980–82	22	2.3764	.1288	.7315	29	29	.2550	.4544	.4757
08074910: 1979–83	14	.5224	.4702	.5120	23	23	1.1071	.4832	.2647
08075550: 1966–72	21	.1651	1.8992	.7614	25	24	3.2568	.6089	.2273
08075550: 1974–78	3	.1282	1.1167	.2301	3	3	2.2776	.5406	.1369
08075550: 1979–84	12	.1177	1.6597	.3600	15	15	3.2776	.6300	.2079
08075600: 1965–72	20	.1935	1.2083	.4738	22	20	2.4993	.4783	.1687
08075750: 1965–72	21	.1856	1.1968	.4375	21	21	1.8116	.4658	.1266
00000BW8: 2004–06	8	1.0137	.4792	1.6392	10	10	.8649	.2199	.1244
0000B504: 2005–06	6	1.2564	.4861	2.5042	8	8	.7877	.4485	.4153
0000K542: 2005–06	6	1.4917	.2778	1.2319	7	7	.5699	.3992	.3273
011NS004: 2004–06	7	.7308	.3691	.5945	14	14	.7477	.2735	.2206
0LBNS001: 2000–02	7	1.9788	.3333	2.8948	9	9	.4977	.0334	.0342
0TGNS002: 2000–02	7	1.2183	.2500	.7256	8	8	.4392	.1880	.1436
0WBNS003: 2004–05	8	1.1708	.3125	.9906	12	9	.7156	.2204	.2152
X8068420: 1985–1986	7	.5136	1.7143	5.0355	8	8	1.9466	.2829	.0764
X8068426: 1985–1986	13	.8785	.8910	4.0126	15	14	1.2142	.5844	.4376
0BFDN000: 1992	6	1.2202	.1250	.2515	7	7	.4787	.1060	.0689

```
# HYETOGRAPH FILE
# INPUT Filename = B504_6.txt; Site=B504_6.
# DATE_TIME=date and time in MM/DD/YYYY@HH:MM:SS
# PRECIP1=preciptation in inches for station 1
# ACCUM_WTD_PRECIP=accumulated weighted precip. in
     inches
DATE_TIME        HOURS_PASSED  PRECIP1  ACCUM_WTD_PRECIP
01/22/2006@14:15:00    0.0000    0.000        0.000
01/22/2006@14:30:00    0.2500    0.010        0.010
01/22/2006@14:45:00    0.5000    0.010        0.020
01/22/2006@15:00:00    0.7500    0.100        0.120
01/22/2006@15:15:00    1.0000    0.030        0.150
01/22/2006@15:30:00    1.2500    0.020        0.170
01/22/2006@15:45:00    1.5000    0.070        0.240
01/22/2006@16:00:00    1.7500    0.130        0.370
01/22/2006@16:15:00    2.0000    0.070        0.440
01/22/2006@16:30:00    2.2500    0.130        0.570
01/22/2006@16:45:00    2.5000    0.060        0.630
01/22/2006@17:00:00    2.7500    0.050        0.680
01/22/2006@17:15:00    3.0000    0.050        0.730
01/22/2006@17:30:00    3.2500    0.010        0.740
01/22/2006@17:45:00    3.5000    0.000        0.740
01/22/2006@18:00:00    3.7500    0.000        0.740
01/22/2006@18:15:00    4.0000    0.000        0.740
01/22/2006@18:30:00    4.2500    0.000        0.740
```

```
# HYDROGRAPH FILE
# INPUT Filename = B504_6.txt; Site=B504_6.
# DATE_TIME=date and time in MM/DD/YYYY@HH:MM:SS
# RUNOFF=runoff in cubic feet per second
# ACCUM_RUNOFF=accumulated runoff in inches
DATE_TIME              HOURS_PASSED     RUNOFF   ACCUM_RUNOFF
01/22/2006@14:15:00          0.0000       0.00       0.000000
01/22/2006@14:30:00          0.2500       0.00       0.000000
01/22/2006@14:45:00          0.5000       0.00       0.000000
01/22/2006@15:00:00          0.7500       0.00       0.000000
01/22/2006@15:15:00          1.0000       1.36       0.002786
01/22/2006@15:30:00          1.2500       2.98       0.008883
01/22/2006@15:45:00          1.5000       2.42       0.013844
01/22/2006@16:00:00          1.7500       8.16       0.030560
01/22/2006@16:15:00          2.0000      20.34       0.072231
01/22/2006@16:30:00          2.2500      17.65       0.108399
01/22/2006@16:45:00          2.5000      18.94       0.147199
01/22/2006@17:00:00          2.7500      26.53       0.201552
01/22/2006@17:15:00          3.0000      23.06       0.248801
01/22/2006@17:30:00          3.2500      19.98       0.289730
01/22/2006@17:45:00          3.5000      14.69       0.319839
01/22/2006@18:00:00          3.7500       9.73       0.339783
01/22/2006@18:15:00          4.0000       6.39       0.352868
01/22/2006@18:30:00          4.2500       4.81       0.362714
01/22/2006@18:45:00          4.5000       4.05       0.371009
```

Figure 2. Example of headers (definitions) and selected data parts of rainfall (left) and runoff (right) data files for storm on January 22, 2006, for BMP Basin 504 on Armand Bayou, Houston, Texas.

shown here) and numerical parameters (statistics of which are used here). The numerical parameters are described in this section.

The equation (Asquith and others, 2005; Asquith and Roussel, 2007) defining a gamma hydrograph (unit or otherwise) is

$$\frac{q(t)}{q_p} = \left[\frac{t}{T_p}\mathrm{e}^{1-(t/T_p)}\right]^K, \qquad (1)$$

where q_p is peak streamflow in inches per hour from the watershed, T_p is the time to peak of the gamma distribution in hours, K is a shape parameter that is dependent on q_p and T_p, and $q(t)$ is streamflow in inches per hour at time t in hours. This equation produces the $q(t)$ ordinates of a gamma hydrograph.

The gamma unit hydrograph can attain shapes that mimic the general shape of many observed runoff hydrographs (unit or otherwise). Expression and analysis of unit hydrographs in terms of q_p and T_p are important because the magnitude and timing of peak streamflow Q_p in cubic feet per second are critical for many designs. Two notes concerning nomenclature are needed. First, synonymous use of lower-case q and upper-case Q for "streamflow" with attendant modifiers is made throughout this report; the two differ only in context-dependent units. Second, T_p (time to peak) exists here as a distinct parameter for the gamma unit hydrograph. There also exists a time of peak that is denoted as T^{Q_p}, which represents the real time that Q_p occurs.

Because of the importance of Q_p estimation in hydrologic engineering practice, the optimal q_p and T_p values were computed by using techniques described in Asquith and others (2005) and Asquith and Roussel (2007) for each storm in the database by precisely matching the modeled Q_p and modeled T^{Q_p} to observed values for 291 individual storm runoff peaks. The count of 291 peaks for the unit hydrograph analysis does not match the 317 data files considered because mathematical solutions for the Asquith and others (2005) and Asquith and Roussel (2007) technique were not attainable for 26 storm peaks. The modeling technique also permitted individual (discrete) analysis of multiple Q_p values within a pair (rainfall and runoff) of data files if the storm had peaks in streamflow that were substantially separate in time as judged by the analyst.

The relation between streamflow [Q, $Q(t)$, or Q_p (a peak), in cubic feet per second] and depth runoff [q, $q(t)$, or q_p (a peak) in inches per hour] is

$$Q = 645.33\,qA, \qquad (2)$$

where Q is streamflow in cubic feet per second, q is streamflow in watershed inches per hour, A is drainage area in square miles, and 645.33 is a unit-conversion factor. A review of unit conversions for equation 2 is provided in appendix 2.

Although the three parameters q_p, T_p, and K are shown in equation 1, in practice, K is a function of q_p and T_p, and total runoff volume V. Consequently, any two parameters will yield the third because $V = 1$ (unit volume) for a unit hydrograph. The total runoff volume V of a gamma hydrograph (Haan and others, 1994, p. 79) is

$$V = q_p T_p \, \Gamma(K) \, \left(\frac{\mathrm{e}^{(1)}}{K}\right)^K, \tag{3}$$

where $\Gamma(K)$ is the complete gamma function for K. The time scale of the unit hydrograph is represented by T_p, but T_p does not represent the time base T_b or overall width in time of a runoff hydrograph. The complete gamma function is expressed as the infinite integral:

$$\Gamma(u) = \int_0^\infty x^{u-1} \, \mathrm{e}^{-x} \, \mathrm{d}x \tag{4}$$

For each watershed, mean values of q_p and T_p were computed and are referred to as "watershed specific." These watershed-specific values were used to compute, through numerical root-solving of equation 3, watershed-specific values of K.

Summary statistics for each of the three parameters for the 24 watersheds were computed and are listed in figures 3, 4, and 5 under the SUMMARY STATISTICS heading. Lastly, regression equations were developed by using the R software (R Development Core Team, 2007) to estimate each parameter from the watershed characteristics of drainage area A and basin-development factor BDF.

The regression equation and ancillary details of the analysis for q_p are listed in figure 3. The equation is

$$q_p = 10^{0.02682 \times BDF - 0.5789 \log_{10}(A) - 0.6575}, \tag{5}$$

in which the residual standard error is 0.152 $\log_{10}$(inches per hour) with an adjusted R-squared of about 0.861. The relation between the watershed-specific peak streamflow and fitted values from the regression equation is shown in figure 6. Plotted next to the data points are corresponding values for BDF.

The regression equation and ancillary details of the analysis for T_p are listed in figure 4. The equation is

$$T_p = 10^{-0.03421 \times BDF + 0.3936 \log_{10}(A) + 0.1745}, \tag{6}$$

in which the residual standard error is 0.204 $\log_{10}$(hours) with an adjusted R-squared of about 0.668. The relation between watershed-specific, time to peak and fitted values of time to peak from the regression equation is made in figure 7. Next to the data points are corresponding values for BDF.

The regression equation and ancillary details of the analysis for K are listed in figure 5. The equation is considered statistically weak (p-value $\approx$ 0.08), is not separately typeset, and thus is not suggested for general application. Because the K equation is weak, it is judged that the mean ($K = 1.3$) and median ($K = 0.75$) values of K for the 24 watersheds represent generalized, but acceptable, measures of gamma hydrograph shape for the study watersheds. Dimensionless gamma hydrographs for the mean value ($K = 1.3$) and median value ($K = 0.75$) shape parameter and the shape parameters for developed ($K = 5.2$) and undeveloped ($K = 2.9$) watersheds in Texas (Asquith and Roussel, 2007) are provided in figure 8.

The dimensionless hydrographs considered by Asquith and Roussel (2007) were derived from watersheds mostly in central and north central Texas where watershed slopes are larger than those in the Houston metropolitan area. As a result, these Asquith and Roussel dimensionless hydrographs (dashed lines in fig. 8) represent more "conventional" hydrograph shape than the relatively longer tailed (longer recession limb) hydrographs derived from the 24 watersheds in the Houston metropolitan area. The difference in dimensionless hydrograph shape for the Houston metropolitan area likely means that the watersheds have greater storage and depth-driven streamflow as opposed to the greater gravitationally driven streamflow of larger sloped watersheds west and northwest of the Houston metropolitan area.

For illustration of the interaction between q_p and T_p and using $K = 1$, the relation between q_p and T_p by equation 3 is straightforward: $1 = q_p T_p \, \Gamma(1) \, (\mathrm{e}^{(1)}/1)^1$ becomes $T_p = 0.3679/q_p$.

Comparison of the p-values and adjusted R-squared values in conjunction with visual comparison of the plots in figures 6 and 7 indicates that the q_p and T_p equations are reliable for the types of watersheds included in the analysis. In practice, K would be determined by iterative solution of equation 3 by using q_p and T_p estimates from equations 5 and 6 to maintain unit volume of the gamma hydrograph.

The authors recognize that the potential q_p and T_p parameter space is not as well populated as statistically desired and that the degrees of freedom are barely sufficient to justify two explanatory variables in the regression equations. For example, if figure 7 is used as a guide, without a loss of generality the largest watersheds (large circles on right of the graph) also tend to have the small-

```
SUMMARY STATISTICS FOR qp, IN INCHES PER HOUR
   Min.  1st Qu.  Median    Mean  3rd Qu.    Max.
 0.1177   0.1977  0.5233  0.7901   1.2190  2.3760

REGRESSION EQUATION
Call:
lm(formula = log10(qp) ~ log10(A) + BDF)

Residuals:
   Min.    1st Qu.   Median   3rd Qu.     Max.
-0.28590  -0.11306  0.01154   0.09148  0.23684

Coefficients:
            Estimate  Std.Error t-value Pr(>|t|)
(Intercept) -0.65746    0.08854  -7.425 2.66e-07
log10(A)    -0.57888    0.06339  -9.132 9.27e-09
BDF          0.02682    0.01134   2.364   0.0278
---

Residual standard error: 0.152 on 21 degrees of freedom
Multiple R-Squared: 0.873, Adjusted R-squared: 0.861
F-statistic: 72.22 on 2 and 21 DF,  p-value: 3.869e-10
```

Figure 3. Statistical summary of watershed-specific, peak streamflow of gamma unit hydrograph and regression equation for estimation of peak streamflow for applicable watersheds in the Houston, Texas, metropolitan area.

```
SUMMARY STATISTICS FOR Tp, IN HOURS
   Min.  1st Qu.  Median    Mean  3rd Qu.    Max.
 0.1250   0.3281  0.5362  0.8035   1.1370  2.3610

REGRESSION EQUATION
Call:
lm(formula = log10(Tp) ~ log10(A) + BDF)

Residuals:
   Min.    1st Qu.   Median   3rd Qu.     Max.
-0.48622  -0.08179  0.07531   0.12831  0.24908

Coefficients:
            Estimate  Std.Error t-value Pr(>|t|)
(Intercept)  0.17454    0.11866   1.471 0.156140
log10(A)     0.39361    0.08495   4.633 0.000143
BDF         -0.03421    0.01520  -2.250 0.035263
---

Residual standard error: 0.204 on 21 degrees of freedom
Multiple R-Squared: 0.696, Adjusted R-squared: 0.668
F-statistic: 24.09 on 2 and 21 DF,  p-value: 3.661e-06
```

Figure 4. Statistical summary of watershed-specific, time to peak of gamma unit hydrograph and regression equation for estimation of time to peak for applicable watersheds in the Houston, Texas, metropolitan area.

```
SUMMARY STATISTICS FOR K, DIMENSIONLESS
   Min.  1st Qu.  Median    Mean  3rd Qu.    Max.
 0.2301   0.4181  0.7464  1.3420   1.3490  5.0350

REGRESSION EQUATION
Call:
lm(formula = K ~ log10(A) + BDF)

Residuals:
   Min.  1st Qu.   Median  3rd Qu.    Max.
-2.7905  -0.7324  -0.2728   0.2910  2.8134

Coefficients:
            Estimate  Std.Error t-value Pr(>|t|)
(Intercept)  2.26514    0.75702   2.992  0.00694
log10(A)    -1.23924    0.54196  -2.287  0.03271
BDF         -0.16360    0.09697  -1.687  0.10638
---

Residual standard error: 1.3 on 21 degrees of freedom
Multiple R-Squared: 0.212, Adjusted R-squared: 0.137
F-statistic: 2.829 on 2 and 21 DF,  p-value: 0.08171
```

Figure 5. Statistical summary of watershed-specific, gamma unit hydrograph shape parameter and regression equation for estimation of shape parameter for applicable watersheds in the Houston, Texas, metropolitan area.

```
ABBREVIATIONS SHOWN ON THESE FIGURES

qp          Depth peak streamflow,
               in inches per hour
Tp          Time to peak streamflow, in hours
K           Shape factor, dimensionless
A           Drainage area, in square miles
BDF         Basin-development factor

Min.        Minimum
1st Qu.     First quartile
3rd Qu.     Third quartile
Max.        Maximum
lm()        Linear modeling function
Std.Error   Standard error
t-value     T-statistic
Pr(>|t|)    Probability of absolute value
                of t-value
DF          Degrees of freedom
```

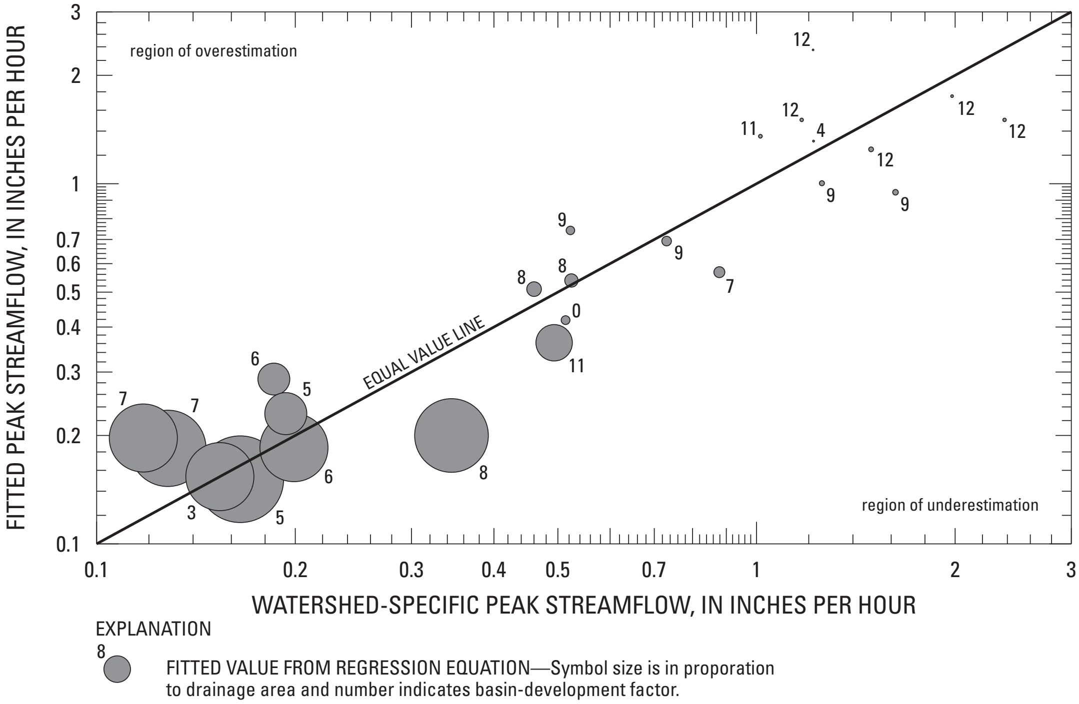


Figure 6. Relation between watershed-specific, peak streamflow and fitted values of peak streamflow by regression shown in equation 5 for a gamma unit hydrograph developed for 24 watersheds in the Houston, Texas, metropolitan area.

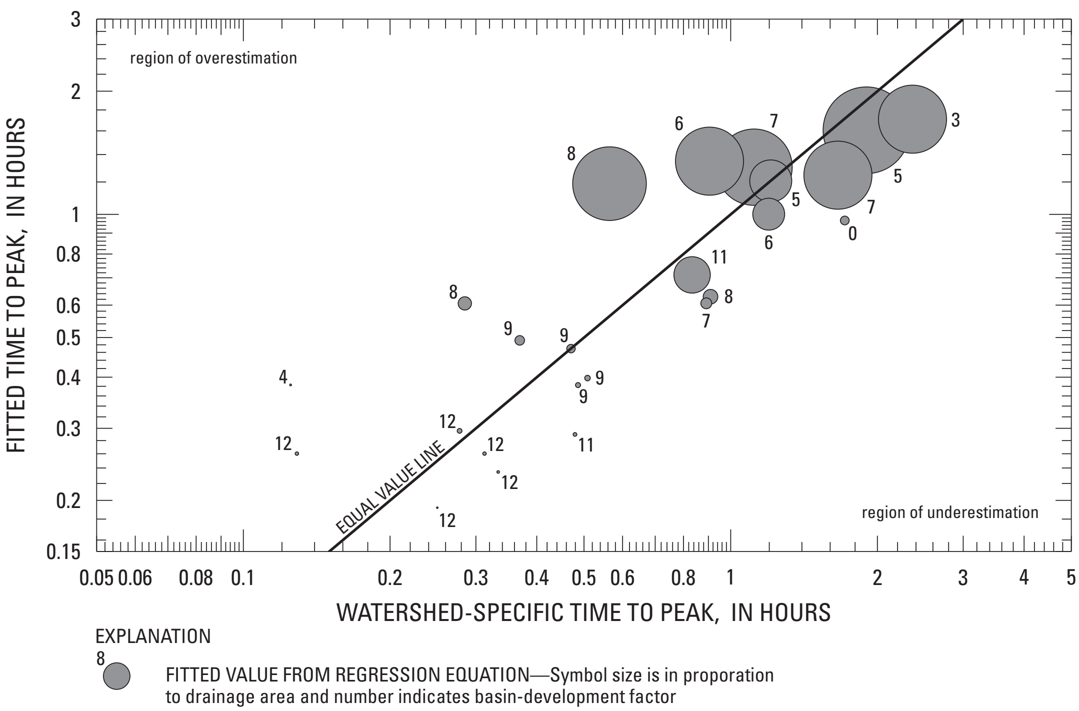


Figure 7. Relation between watershed-specific, time to peak and fitted values of time to peak by regression shown in equation 6 for a gamma unit hydrograph developed for 24 watersheds in the Houston, Texas, metropolitan area.

est BDF values and visa versa. Whether the equations for q_p and T_p fundamentally lack estimation power for largely undeveloped $BDF \leq 3$ or moderately developed $3 \leq BDF \leq 6$ watersheds with $A \leq 1$ square mile cannot be determined because of the general absence of such watersheds among the 24 used for this report.

Computing Gamma Unit Hydrographs

Computations of the 5-minute gamma unit hydrograph are demonstrated in this section. Suppose that 1 inch of excess rainfall is uniformly distributed in a 5-minute interval and across a 0.5-square mile watershed with a $BDF = 9$. What are the estimates of Q_p and T^{Q_p} for this watershed?

An estimate of q_p provides a starting point. The q_p for the watershed from equation 5 by substitution is

$$q_p = 10^{0.02682\times 9 - 0.5789\log_{10}(0.5) - 0.6575}, \tag{7}$$

or $q_p = 0.573$ inches per hour of runoff from the watershed. Using a $K = 1$ (only for these immediate computations), the T_p is $T_p = 0.3679/0.573$ or $T_p = 0.642$ hours (about 40 minutes). Solving for the Q_p by using equation 2, the gamma unit hydrograph provides ${}_{\mathrm{UH}}Q_p = 645.33 \times 0.573 \times 0.5$ or ${}_{\mathrm{UH}}Q_p = 185$ cubic feet per second. The gamma unit hydrograph for this watershed is shown in figure 9.

Extending the example, if $K = 1$ is not assumed, the computation of T_p for the watershed is needed, and K will have to be computed by numerical methods. The T_p from equation 6 by substitution is

$$T_p = 10^{-0.03421\times 9 + 0.3936\log_{10}(0.5) + 0.1745}, \tag{8}$$

or $T_p = 0.560$ hours. The equivalent $K^{q_p=0.573}_{T_p=0.560}$ value is $K = 0.79$ by iterative-solution to equation 3 for $V = 1$. The value $K = 0.79$ is less than 1, so the resulting gamma unit hydrograph (not shown here) would have a heavier tail (longer recession) than the $K = 1$ hydrograph shown in figure 9.

Lastly, the T^{Q_p} of the gamma unit hydrograph for the example is simply ${}_{\mathrm{UH}}T^{Q_p} = T_p$ because the rainfall has a duration of 5 minutes and is equal to that of the gamma unit hydrograph; no convolution is required. In general, however, ${}_{\mathrm{UH}}T^{Q_p}$ requires estimation from convolution of the gamma unit hydrograph with the excess rainfall time series. Similar time computations to these are described in the section titled "Comparison of Unit Hydrograph and Rational Method Analysis."

Analysis of Rational Method for the 24 Watersheds

Background and Mathematical Analysis

The rational method (Pilgrim and Cordery, 1993; Dingman, 2002) is a less complex technique compared to the unit hydrograph method to estimate Q_p. The method originating from Kuichling (1889) is widely used for drainage design in small urban watersheds. The estimation of Q_p by the rational method (${}_{\mathrm{RM}}Q_p$) is obtained by

$${}_{\mathrm{RM}}Q_p = 1.008\,CIA, \tag{9}$$

where 1.008 is a unit-conversion factor for indicated units of other variables, C is a dimensionless runoff coefficient, I, is rainfall intensity in inches per hour, and A is drainage area in acres. A review of the unit-conversion factor is provided in appendix 2.

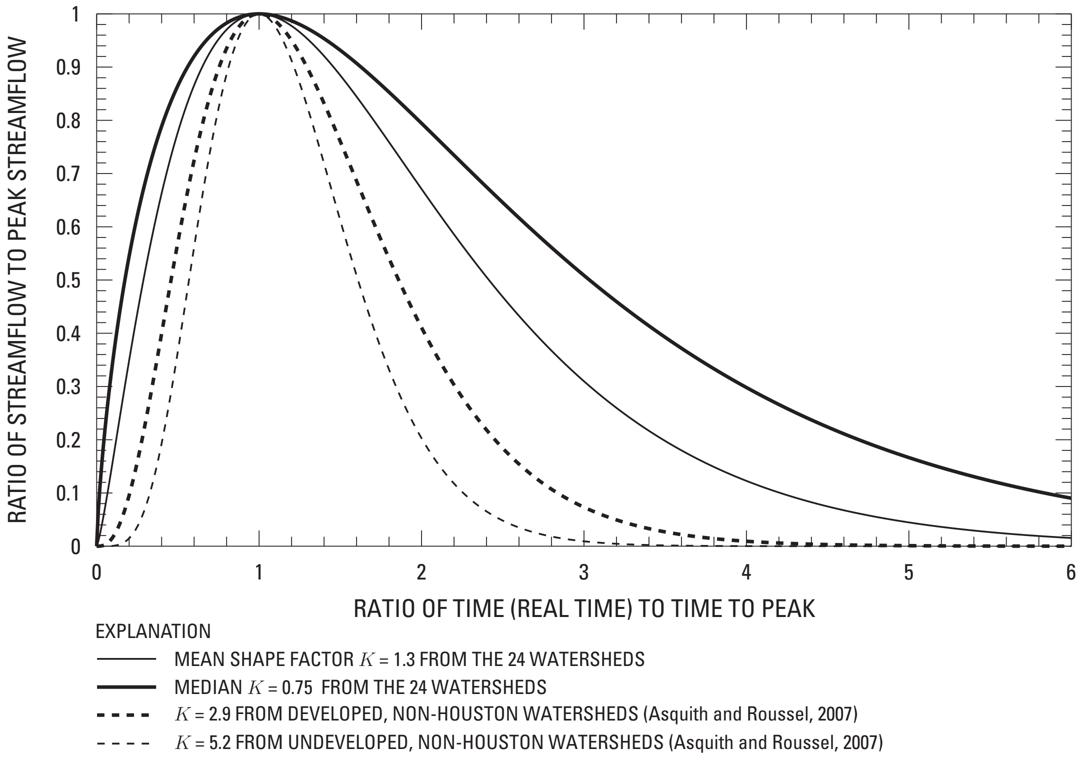
By inspection of equation 9, estimates of ${}_{\mathrm{RM}}Q_p$ are sensitive to values of C. Tables of C for various land use, watershed slope, types of surfaces, and rainfall recurrence intervals or intensities by various authors for different historical contexts are provided in text books (Chow and others, 1988, table 15.1.1 on p. 498), intra-agency procedures (Texas Department of Transportation, 2002), and jurisdictionally specific guidelines (Freeze and Nichols, 2010).

Similar to the gamma unit hydrograph analysis previously described, the rational method can be adapted or rephrased for the generalized hydrologic and hydraulic conditions represented by the 24 watersheds. To this end, the method can be rewritten as

$${}_{\mathrm{RM}}Q_p = 1.008\,C\,\frac{P(T,F)}{T}\,A, \tag{10}$$

where the depth of rainfall $P(T,F)$ for a given duration T and nonexceedance probability F (commonly an expression of recurrence interval) divided by T is equal to I of equation 9. Depth-duration relations of rainfall with recurrence interval could be derived from Asquith and Roussel (2004) or other sources. Given that $P(T,F)$ and A are known quantities in a design context, the rational method is not only a function of C but also of T.

Values for T often are taken to be the "time of concentration," denoted as T_c, for the watershed. Alternatively, values for T are taken to be a time (perhaps not numerically equal to T_c) that is representative of "full watershed contribution." The authors prefer the concept of "critical storm duration" over either of the other two. Methods to estimate T_c based on empirical equations also



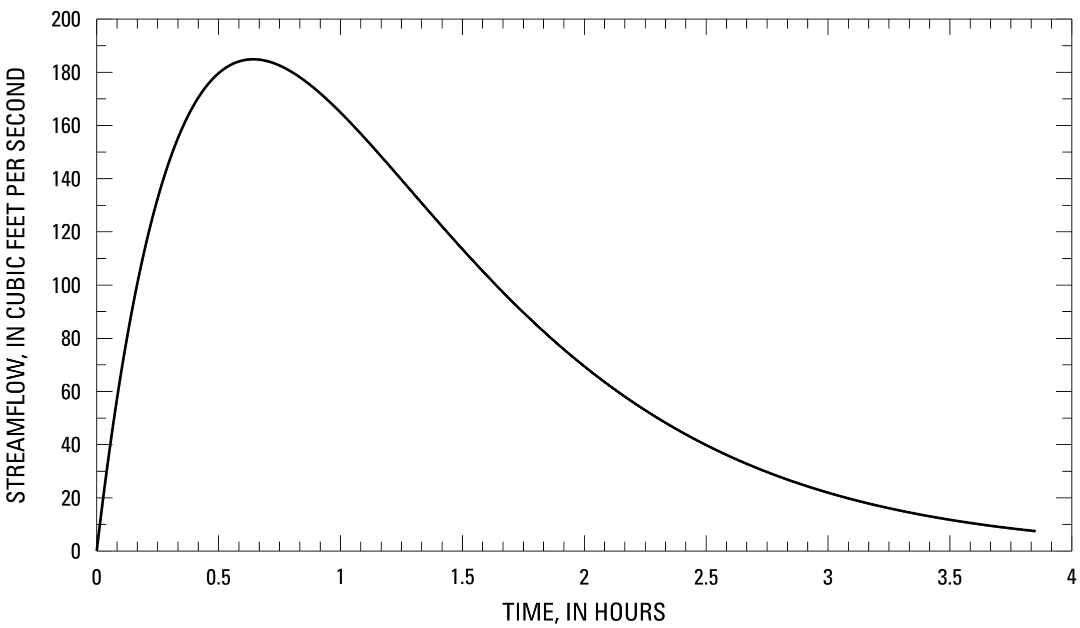


Figure 9. Example of a gamma unit hydrograph with a shape factor of 1 from a 0.5-square mile watershed with a basin-development factor of 9.

are available with various lineages or historical contexts (Dingman, 2002, table 9–9, and references therein). Roussel and others (2005) provided a review of T_c estimation methods pertinent to the needs of TxDOT. Because an adaptation of the rational method specific to small low-slope watersheds in the Houston metropolitan area is needed, existing equations or techniques for T_c estimation were not used in this report.

If equation 10 reasonably expresses the coupled relation between rainfall intensity and peak streamflow, then to maintain proportionality, C cannot be decoupled conceptually from T because the two parameters are interrelated. In a statistical sense, as T becomes too large (is overestimated), then C must increase, and the opposite is true if T is underestimated. Values for T, therefore, are of equal importance to the rational method as values for C. The authors note that the method results in one equation and two unknowns (C and T).

A starting point for the analysis of the rational method is needed. Schaake and others (1967) observed that

> An assumption of the time required for runoff [a time of concentration] to flow from the farthest point in the drainage area is presently used in design practice. However, there is no known way to determine this time of flow, either from measurements in the field during storms or from records of rainfall and runoff.
>
> Except for steady state conditions, which rarely, if ever are reached during a thunderstorm, there is no good reason to believe that the time of flow from the farthest point in a drainage area should necessarily be the best rainfall averaging time to use in the [rational method.] A study, therefore, was made to find for each area the averaging time giving the best correlation between average rainfall intensity and peak runoff rates.

Schaake and others (1967) eventually settled on the lag time from rainfall centroid to runoff centroid as the "best" time. (As discussed in following paragraphs, it is effectively this time definition that the authors of this report computed and recorded as T_r.)

Because of the need to lock either C or T down, the authors of this report considered

$$T_c' = \sqrt{A[\text{square miles}]} = \sqrt{A[\text{acres}]/640}, \tag{11}$$

as a first-order approximation to T_c or T by association, where T_c' is in hours and A is in units as shown. Equation 11 can be physically interpreted as follows: if a watershed is square, and thus has sides of equal length, then the hypotenuse of a unit area is $\sqrt{2} \approx 1.4$, so the approximation assumes that aggregate travel speed (combination of overland flow, storage residence times, and channel flow) is about 1.4 miles per hour or about 2 feet per second. Although perhaps a fast aggregate velocity for overland-flow conditions for undeveloped, low-slope watersheds in the study area, the storm sewers in the Houston metropolitan area generally are engineered for velocities of 3 feet per second (Duane Barrett, R.G. Miller Engineers Inc., written commun., 2008). Hence, 2 feet per second might be considered an appropriate (albeit rough) generalization for the method described here.

Building on equation 11, the rational method for a observed storm event that is denoted by ${}^{\lightning}$ ("storm specific") becomes

$$_{\mathrm{RM}}{}^{\lightning}Q_p = 1.008\,{}^{\lightning}C_r \times \frac{{}^{\lightning}P\}^{\max}_{T_c'} \times A[\text{acres}]}{\sqrt{A[\text{acres}]/640}}, \tag{12}$$

where $_{\mathrm{RM}}{}^{\lightning}Q_p$ is the peak streamflow for the storm in cubic feet per second, and ${}^{\lightning}C_r$ is a storm-specific runoff coefficient (dimensionless) with subscript r ("root") as a reminder that $\sqrt{A[\text{acres}]/640}$ is used to approximate T_c. The term ${}^{\lightning}P\}^{\max}_{T_c'}$ represents the maximum depth of rainfall in inches for a time window (duration) having a width of T_c' by equation 11 that is moved or swept through the observed rainfall time series. The ${}^{\lightning}P\}^{\max}_{T_c'}$ effectively determines the maximum rainfall intensity. The units of A are explicitly shown as a reminder that a mixed unit system for A is involved in mathematical operations in this report.

Equation 12 can be solved ("inverted") for ${}^{\lightning}C_r$

$${}^{\lightning}C_r = {}_{\mathrm{RM}}{}^{\lightning}Q_p \times \frac{\sqrt{A[\text{acres}]/640}}{{}^{\lightning}P\}^{\max}_{T_c'} \times A[\text{acres}]}. \tag{13}$$

Equation 13 is useful because a single runoff coefficient can be extracted. Statistical summaries or methods to estimate C_r from processing of multiple storms for a watershed then would provide estimates of C that are congruent with the assumption of watershed time given by equation 11.

A problem still remains. For computation of ${}^{\lightning}C_r$, each observed time series of rainfall and runoff requires parsing into discrete epochs for computation of ${}^{\lightning}P\}^{\max}_{T_c'}$ and computation of $_{\mathrm{RM}}{}^{\lightning}Q_p$, respectively. This parsing requires analyst input for meaningful interpretation.

To this end, highly specialized, single-purpose software was developed for this report to compute ${}^{\lightning}C_r$ and other features of the rational method. A screenshot is shown in figure 10, which shows selection of a bounding box (the dashed box was manually superimposed on the figure for clarity) on the rainfall time series (top box, the "rainfall epoch") likely producing the observed runoff hydrograph. The runoff time series also requires parsing into the associated runoff (streamflow) hydrograph (bottom box, the "runoff epoch"). Extraction of ${}_{\mathrm{RM}}{}^{\lightning}Q_p$ from the runoff epoch is straightforward. Computation of ${}^{\lightning}P\}^{\max}_{T_c'}$ is algorithmically more complex but readily performed by the computer.

The parsing of the rainfall and runoff in figure 10 is completed by the analyst pressing the "Calculate C" button (

). The results are shown in figure 11. In figure 10, there are 24 rainfall pulses of 5-minute width within the parsed rainfall epoch. Following computational migration to half the rainfall width, 23 unique durations of rainfall exist for the example (see the 23 plotted points in fig. 11). A maximum rainfall intensity and corresponding C value for each unique duration (5-minute increments for the example and this study) can be computed by a movable time window. These are referred to as C_i values and are shown as open circles in figure 11. The C_w value represents the C for the full width of the rainfall epoch, the C associated with the average intensity of the "entire storm" as parsed by the analyst. The primary conclusion deriving from figure 11 is that no unique runoff coefficient exists without an estimate of watershed time (critical storm duration).

Because no single C exists without watershed time and because of the need to simplify forthcoming analysis by having a single representative C by storm, equation 11 was used to provide a first approximation. The filled triangle in figure 11 represents ${}^{\lightning}C_r$ based on ${}^{\lightning}P\}^{\max}_{T_c'}$ that has been "snapped" to the nearest integer multiple of 5 minutes (the time increment of the specific analysis as shown in fig. 10).

For a given storm, the volumetric runoff coefficient ${}^{\lightning}C_v$ can be computed by

$$ {}^{\lightning}C_v = \frac{{}^{\lightning}R}{{}^{\lightning}P}, \qquad (14)$$

where ${}^{\lightning}R$ is the total runoff in inches for the runoff epoch and ${}^{\lightning}P$ is the total rainfall in inches for the rainfall epoch. The ${}^{\lightning}C_v$ is represented in figure 11 by the horizontal dashed line. In general ${}^{\lightning}C_r \neq {}^{\lightning}C_v$.

A useful expression of the rational method for this investigation is as the "excess rational method," which is a solution of the rational method for a unit depth of excess rainfall. The equation for estimation of Q_p by the excess rational method (${}_{\mathrm{ERM}}Q_p$) is

$$ {}_{\mathrm{ERM}}Q_p = 1.008\, C_r \,\frac{(E/C_v) \times A[\mathrm{acres}]}{T_c}, \qquad (15)$$

where ${}_{\mathrm{ERM}}Q_p$ is the Q_p from the excess rational method in cubic feet per second, the term E/C_v is the depth of rainfall in inches, E is excess rainfall in inches, and T_c in hours is explicitly chosen as the critical storm duration. For a unit depth of excess rainfall (rainfall remaining after watershed losses) $E = 1$ inch. The excess rational method can be expressed as

$$ {}_{\mathrm{ERM}}Q_p = 1.008\, \frac{C_r}{C_v}\, \frac{A[\mathrm{acres}]}{T_c}. \qquad (16)$$

To complete the mathematical analysis of the rational method and partly inspired by Schaake and others (1967), a better estimate of time than provided by equation 11 can be computed after parsing of the rainfall and runoff into epochs. The T_r can be defined as follows:

T_r —The center of the window of ${}^{\lightning}P\}^{\max}_{T_c'}$ can be conceptualized as the centroid of rainfall. Time separation between this centroid and the time of peak streamflow ${}^{\lightning}T^{Q_p}$ for the storm is defined as T_r. This time is known as a lag time (Dingman, 2002, table 9–1, term T_{LPC}). For this report, T_r also is referred to as time-R.

By further interpretation of the rational method, both times of rise and recession of a hydrograph are equal to T_c (Pilgrim and Cordery, 1993, p. 9.15–9.16). The time of peak ${}_{\mathrm{ERM}}T^{Q_p}$ (also a "time of rise") of the excess rational method is equivalent to that for rational method and therefore is

$$ {}_{\mathrm{ERM}}T^{Q_p} = {}_{\mathrm{RM}}T^{Q_p} = T_c. \qquad (17)$$

It follows from the definition of T_r and because the width of rainfall was defined by T_c' that the best available estimate of critical storm duration T_c is given by

$$ T_c = T_r + \frac{T_c'}{2} = T_r + \frac{\sqrt{A[\mathrm{acres}]/640}}{2} = {}_{\mathrm{ERM}}T^{Q_p}. \qquad (18)$$

To reiterate, ${}_{\mathrm{ERM}}T^{Q_p}$ is the duration from the beginning of rainfall to the time that Q_p occurs. (Technically, this time represents the duration from the beginning of excess rainfall, but no initial abstraction is formulated in the rational method.) Therefore, ${}_{\mathrm{ERM}}T^{Q_p}$ can be directly compared to ${}_{\mathrm{UH}}T^{Q_p}$. Equation 18 provides a more reasonable estimate of critical storm duration compared to equation 11.

Computational and Statistical Analysis

Watershed-specific values for C_v and C_r were computed for each watershed as the arithmetic mean of ${}^{\lightning}C_v$ and ${}^{\lightning}C_r$ (table 2). Statistical summaries of these C_v and C_r

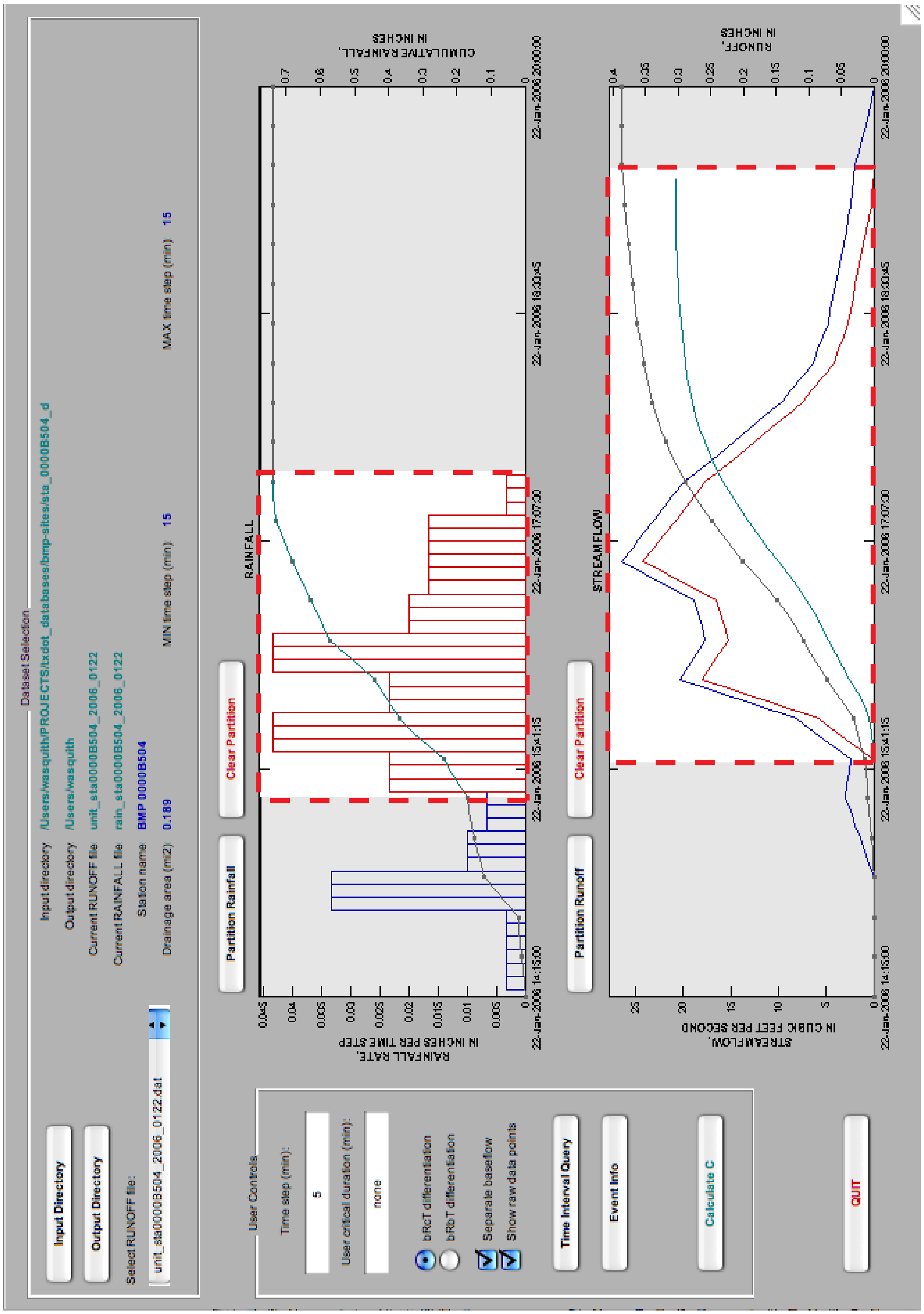


Figure 10. Screenshot of highly specialized, single-purpose software for inversion of rational method for storm on January 22, 2006, for BMP Basin 504 on Armand Bayou, Houston, Texas.

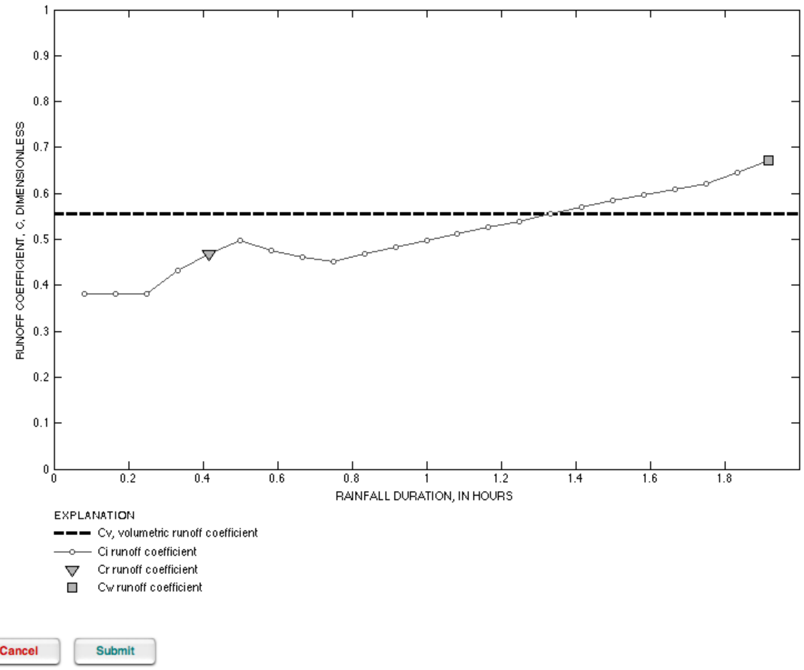


Figure 11. Screenshot of output from highly specialized, single-purpose software showing computational results for storm on January 22, 2006, for BMP Basin 504 on Armand Bayou, Houston, Texas, as shown in figure 10.

values and the ratio C_r/C_v are provided in figure 12. The number of storms varies slightly for the C_v and C_r computations. Because suitable storms had to have less runoff than rainfall, values of ${}^{\lightning}C_v$ were available for each storm. For a few storms, the width of analyst-selected rainfall (the rainfall epoch) was smaller than the $T_c = T_c'$; therefore, an estimate for C_r (and T_r) was not available because the runoff-producing storm pulse was shorter than T_c' in time. In total, 394 storms were analyzed and the ${}^{\lightning}C_v$ computed; of these, 380 storms provided estimates of ${}^{\lightning}C_r$.

An analysis of potential relations between both C_v and C_r (the watershed-specific values listed in table 2) and A and BDF was performed. Consistent with the observation by Chow and others (1988, p. 497) that "C [values are] the least precise variable of the rational method," it is concluded from the analysis that reliable estimation of these C values by A and BDF is difficult to achieve. There are only weak associations of C values with either A or BDF. The weak associations are likely related to many storm-specific factors including antecedent moisture conditions and deviations from uniform spatial distribution of rainfall.

A useful approximation of C_v and C_r for applicable watersheds in the Houston metropolitan area are the mean values of the ${}^{\lightning}C_v$ and ${}^{\lightning}C_r$. These means are reported in the statistical summaries shown in figure 12 for the 24 watersheds: $C_v = 0.41$ and $C_r = 0.25$. The first and third quartiles of the C_r/C_v ratios shown in figure 12 are about 0.37 and 0.81, respectively. These ratios can be used to estimate some of the uncertainty associated with the method described in this report. The authors explicitly use the ratio of respective mean values for C_r and C_v and not the mean value for the C_r/C_v ratios in later computations.

In total, 380 storms were used to estimate ${}^{\lightning}T_r$. Watershed-specific values for T_r were computed as the mean by watershed of ${}^{\lightning}T_r$. The term $\sqrt{A}/2$ subsequently was added to yield the watershed-specific T_c estimates listed in table 2.

A regression equation was developed to estimate T_r from watershed characteristics of drainage area A and basin-development factor BDF. Because A was used in the regression, regression analysis of the T_c values listed in table 2 would be inappropriate because of the explicit inclusion of $\sqrt{A}/2$ in these values.

The regression equation and ancillary details of the analysis for T_r are listed in figure 13. The equation is

$$T_r = 10^{-0.05228 \times BDF + 0.4028 \log_{10}(A) + 0.3926}, \qquad (19)$$

in which the residual standard error is $0.209 \log_{10}(\text{hours})$

with an adjusted R-squared of about 0.718. A comparison of the watershed-specific, T_r (time-R) and fitted values from the regression equation is made in figure 14. Next to the data points are the corresponding values for BDF.

A Method for Estimating Peak and Time of Peak Streamflow from Excess Rainfall for 10- to 640-Acre Watersheds in the Houston, Texas, Metropolitan Area

A method for estimating peak and time of peak streamflow from excess rainfall for 10- to 640-acre watersheds in the Houston, Texas, metropolitan area is presented in this section.

Comparison of Results from Unit Hydrograph and Rational Method Analysis

A comparison of the results from the rainfall and runoff analysis of the 24 watersheds is informative. Many of the computations presented in the comparison will be used for formal definition of the method.

The comparison between (1) the unit hydrograph analysis using a 5-minute gamma unit hydrograph and (2) the rational method analysis is made through graphical depiction of equations 5 (q_p), 6 (T_p), 11 (T_c'), and 19 (T_r) and is shown in figure 15. The figure shows the "timing" and the "peaking" response of the watershed. The figure helps users visualize the regression equations and facilitates estimation of q_p and T_p for applicable watersheds in the Houston metropolitan area. Independent estimates of Q_p and T^{Q_p} from the unit hydrograph and rational methods can be obtained with knowledge of

1. The watershed characteristics A and BDF,
2. Representative runoff coefficients (C_r and C_v) for the excess rational method, and
3. A watershed-loss model that is a conceptual or numerical model of watershed losses (Chow and others, 1988, p. 135), which is beyond the scope of this report.

Hypothetical Watershed—Suppose a watershed has a drainage area of $A = 300$ acres ($A = 0.469$ square miles). By using undeveloped conditions (as represented by $BDF = 0$) and fully developed conditions (as represented by $BDF = 12$), the T^{Q_p} (not T_p or T_c) and Q_p are computed from both the unit hydrograph and excess rational methods.

The mean C_v of the 24 watersheds is about $C_v = 0.41$ (fig. 12), and the mean C_r is about $C_r = 0.25$ (fig. 12). Combining these runoff coefficients into the excess rational method for 1 inch of excess rainfall for applicable watersheds in the Houston metropolitan area, the authors obtain from equation 16 by substitution

$$_{\mathrm{ERM}}Q_p = 1.008 \times \frac{0.25}{0.41}\frac{A}{T_c}, \tag{20}$$

or approximately

$$_{\mathrm{ERM}}Q_p = \frac{0.61A}{T_c}, \tag{21}$$

where $_{\mathrm{ERM}}Q_p$ is the Q_p in cubic feet per second per 1 inch of excess rainfall, A is drainage area in acres, and T_c is estimated by equation 18.

Undeveloped Conditions (*BDF* $= 0$)—The T_p of the gamma unit hydrograph for $A = 300$ acres is about $T_p = 1.09$ hours or about 65 minutes (rounded to nearest 5 minutes) (fig. 15). The q_p for the gamma unit hydrograph is about $q_p = 0.34$ inches per hour. The K is computed by inversion of equation 3 for these T_p and q_p values and is about $K = 1.00$. The T_r for the watershed is about $T_r = 1.82$ hours or about 110 minutes (rounded to nearest 5 minutes) for which $(\sqrt{A[\text{acres}]/640}\,)/2 =$ 0.342 hours or about 20 minutes is added to T_r as per equation 18 to acquire a T_c for the watershed (fig. 15). Therefore, a $T_c = 130$ minutes (about 2.17 hours) is used.

The excess rational method for the undeveloped watershed thus is solved from equation 20:

$$\begin{aligned} {}_{\mathrm{ERM}}Q_p \mid^{BDF=0} &= 1.008\,\frac{0.25}{0.41}\,\frac{A}{T_c} \\ &= 1.008\,\frac{0.25}{0.41}\,\frac{300}{130/60} \\ &= \boxed{84.5 \text{ cubic feet per second}}. \end{aligned} \tag{22}$$

The division by 60 minutes is made to acquire units of hours. Because the T^{Q_p} for the excess rational method is T_c, the authors estimate that $_{\mathrm{ERM}}T^{Q_p} \mid^{BDF=0}$ or "$_{\mathrm{ERM}}T^{Q_p}$ given a $BDF = 0$" is about $\boxed{130 \text{ minutes}}$.

The unit hydrograph method requires convolution of the 5-minute gamma unit hydrograph GUH(q_p=0.34, $T_p = 1.083$, $K = 1.00$) defined by equation 1 with a unit of excess rainfall uniformly distributed across

```
ABBREVIATIONS SHOWN ON THIS FIGURE
Min.        Minimum
1st Qu.     First quartile
3rd Qu.     Third quartile
Max.        Maximum
```

```
# Volumetric runoff coefficient
summary(meansCv) # watershed-specific Cv values
   Min.  1st Qu.  Median     Mean  3rd Qu.    Max.
0.03339  0.28060 0.46010  0.41260  0.54280 0.63980
```

```
# Runoff coefficient for Rational Method
summary(meansCr) # watershed-specific Cr values
   Min.  1st Qu.  Median    Mean  3rd Qu.    Max.
0.03419  0.13430 0.21790 0.24820  0.36850 0.49440
```

```
# Ratio of runoff coefficients for Rational Method
summary(meansCr/meansCv)
   Min.  1st Qu.  Median    Mean  3rd Qu.    Max.
 0.2531   0.3682  0.6549  0.6316   0.8099  1.0470
```

Figure 12. Statistical summary of watershed-specific, runoff and volumetric runoff coefficients, and runoff coefficient ratios for 24 watersheds in the Houston, Texas, metropolitan area.

```
ABBREVIATIONS SHOWN ON THIS FIGURE

qp          Depth peak streamflow,
               in inches per hour
Tr          Time-R, in hours
K           Shape factor, dimensionless
A           Drainage area, in square miles
BDF         Basin-development factor

Min.        Minimum
1st Qu.     First quartile
3rd Qu.     Third quartile
Max.        Maximum
lm()        Linear modeling function
Std.Error   Standard error
t-value     T-statistic
Pr(>|t|)    Probability of absolute value
                of t-value
DF          Degrees of freedom
```

```
SUMMARY STATISTICS FOR Tr, DIMENSIONLESS
   Min. 1st Qu.  Median    Mean 3rd Qu.    Max.
0.07471 0.35480 0.76710 1.01900 1.32400 3.66200

REGRESSION EQUATION
Call:
lm(formula = log10(Tr) ~ log10(A) + BDF)

Residuals:
    Min.  1st Qu.   Median  3rd Qu.     Max.
-0.53502 -0.09974  0.04105  0.13479  0.36084

Coefficients:
            Estimate  Std.Error t-value Pr(>|t|)
(Intercept)  0.39259    0.11653   3.369 0.002902
log10(A)     0.40275    0.08599   4.684 0.000127
BDF         -0.05228    0.01453  -3.597 0.001694
---

Residual standard error: 0.209 on 21 degrees of freedom
Multiple R-Squared: 0.7428, Adjusted R-squared: 0.718
F-statistic: 30.33 on 2 and 21 DF,  p-value: 6.42e-07
```

Figure 13. Statistical summary of watershed-specific, time-R and regression equation for time-R estimation for applicable watersheds in the Houston, Texas, metropolitan area.

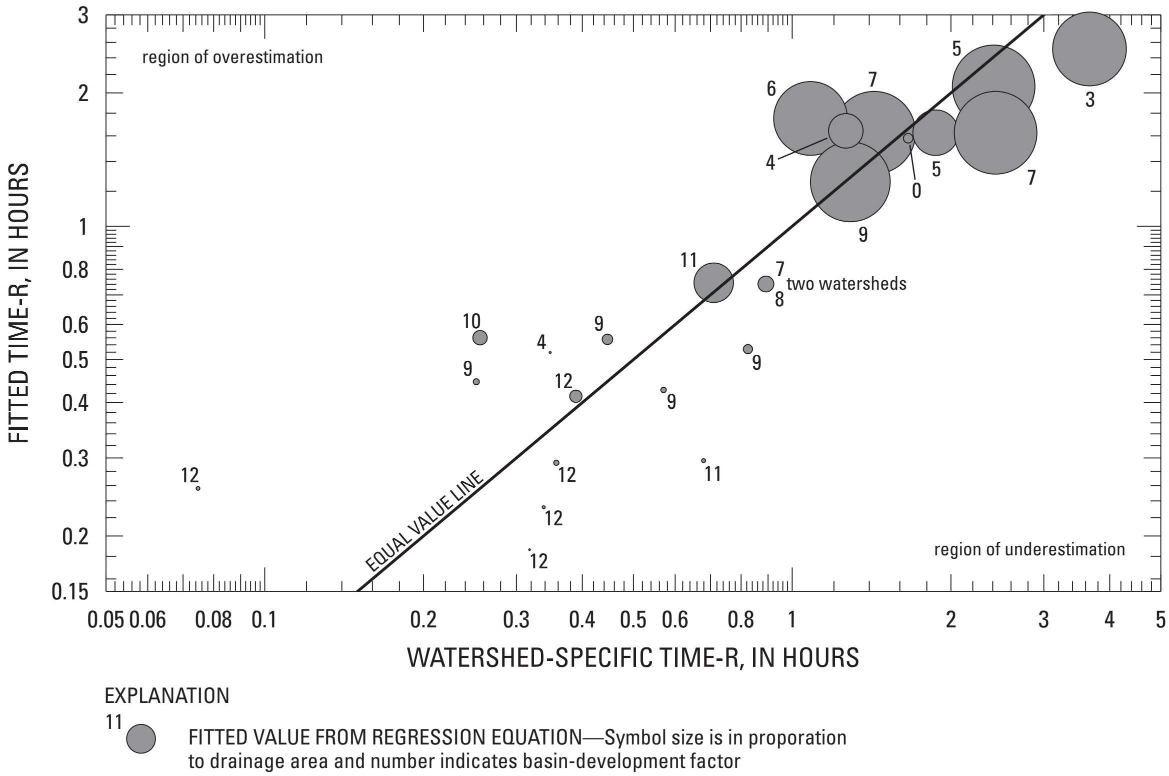


Figure 14. Relation between watershed-specific, time-R and fitted values of time-R by regression shown in equation 19 developed for 24 watersheds in the Houston, Texas, metropolitan area.

$T_c \mid^{BDF=0} = 130$ minutes. The convolutions for these examples were performed in a spreadsheet. Also, because a 5-minute unit hydrograph is used, for convenience and the fact that the watershed is small, 5-minute computational steps were used for the convolution process. The results of the convolution provide a ${}_{\mathrm{UH}}Q_p \mid^{BDF=0}$ of about $\boxed{\text{88.3 cubic feet per second}}$, and a ${}_{\mathrm{UH}}T^{Q_p} \mid^{BDF=0}$ of about $\boxed{\text{145 minutes}}$. This timing is on the order of the $T_c \mid^{BDF=0}$ estimated for the watershed, but logically is slightly longer because of the nature of the resultant hydrograph from the convolution—an *offset*, *multiply*, and *summation* process.

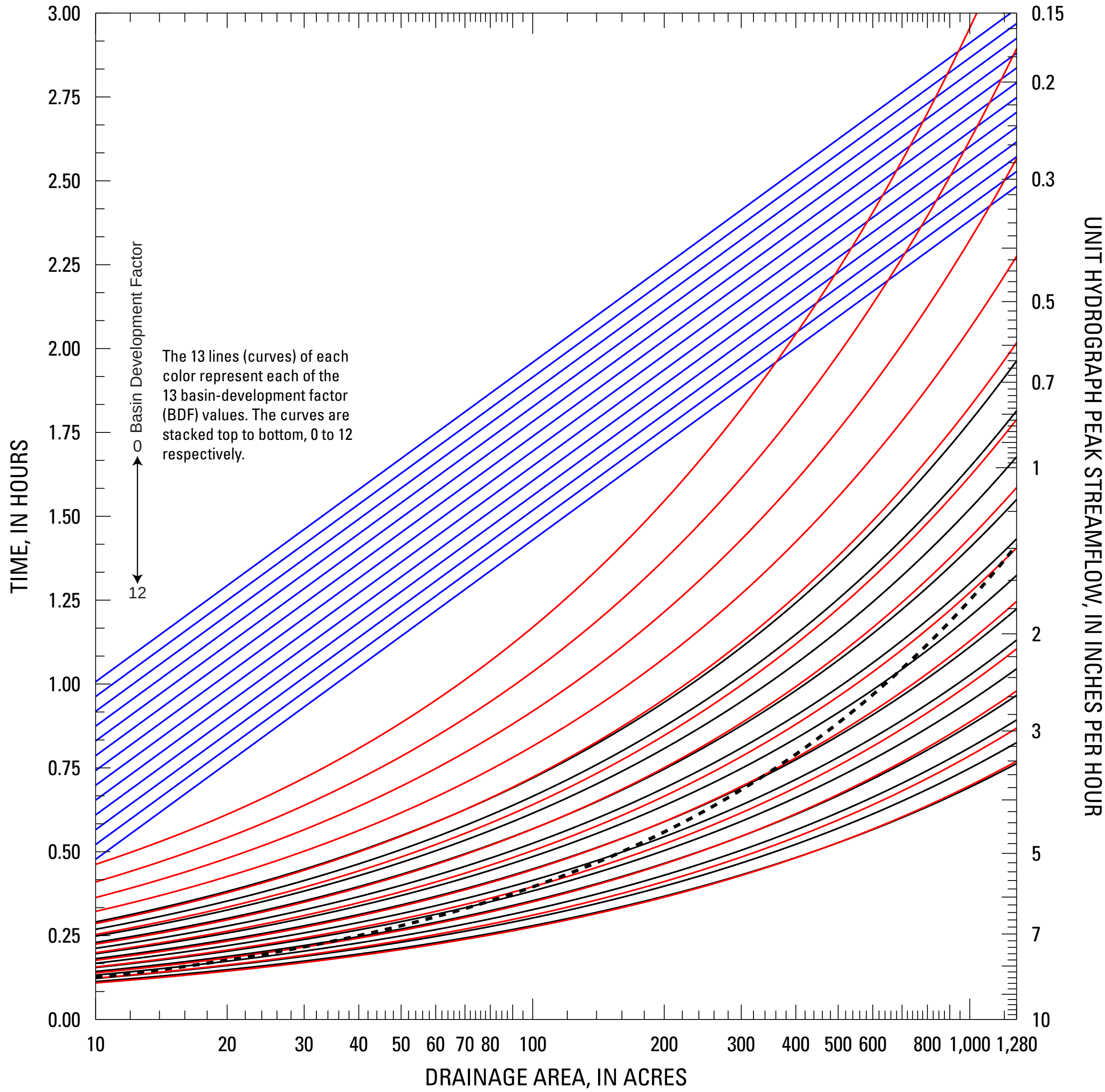
Developed Conditions (*BDF* = 12)—The T_p of the gamma unit hydrograph for $A = 300$ acres is about $T_p = 0.42$ hours or about 25 minutes (rounded to nearest 5 minutes) (fig. 15). Also from the figure, the q_p for the gamma unit hydrograph is about $q_p = 0.73$ inches per hour. The K is computed by inversion of equation 3 for these T_p and q_p values and is about $K = 0.725$. The T_r for the watershed is about $T_r = 0.43$ hours or about 25 minutes (rounded to nearest 5 minutes) for which $(\sqrt{A[\text{acres}]/640}\,)/2 = 0.342$ hours or about 20 minutes is added to T_r as per equation 18 to acquire a T_c for the watershed (fig. 15). Therefore, a $T_c = 45$ minutes (0.75 hours) is used.

The excess rational method (equation 16) for the developed watershed thus is solved from equation 20:

$$\begin{aligned} {}_{\mathrm{ERM}}Q_p \mid^{BDF=12} &= 1.008\,\frac{C_r}{C_v}\,\frac{A}{T_c} \\ &= 1.008\,\frac{0.25}{0.41}\,\frac{300}{45/60} \\ &= \boxed{244\ \text{cubic feet per second}}. \end{aligned} \tag{23}$$

Again, because the T^{Q_p} for the rational method is T_c, the authors estimate that ${}_{\mathrm{ERM}}T^{Q_p} \mid^{BDF=12}$ is about $\boxed{\text{45 minutes}}$.

The unit hydrograph method requires convolution of the gamma unit hydrograph GUH($q_p = 0.73$, $T_p = 0.417$, $K = 0.725$) defined in equation 1 with a unit of excess rainfall uniformly distributed across $T_c \mid^{BDF=12} =$ 45 minutes. The results of the convolution provide ${}_{\mathrm{UH}}Q_p \mid^{BDF=12}$ of about $\boxed{\text{202 cubic feet per second}}$, and a ${}_{\mathrm{UH}}T^{Q_p} \mid^{BDF=12}$ of about $\boxed{\text{50 minutes}}$. This timing is on the order of the $T_c \mid^{BDF=12}$ estimated from figure 15 for the watershed but logically is slightly longer because of



EXPLANATION

- TIME TO PEAK OF 5-MINUTE GAMMA UNIT HYDROGRAPH (T_p, equation 6)
- DEPTH OF PEAK STREAMFLOW OF 5-MINUTE GAMMA UNIT HYDROGRAPH (q_p, equation 6)
- TIME-R FROM ANALYSIS OF RATIONAL METHOD (T_r, equation 19)
- SQUARE ROOT OF DRAINAGE AREA IN SQUARE MILES (not acres) (T_C', equation 11)

Figure 15. Relation between time to peak or depth of peak streamflow of the gamma unit hydrograph to drainage area and basin development factor.

the nature of the resultant hydrograph from the convolution process.

The results for developed and undeveloped watersheds are listed in table 3. The results demonstrate remarkable consistency between the unit hydrograph and rational methods for the hypothetical watershed and by association the methods are expected to agree for similar watersheds less than about 640 acres in the Houston metropolitan area. As a final step, the arithmetic mean of the two T^{Q_p} and Q_p estimates can be used to establish the best estimate from the analysis; these values are listed in table 3 (columns 6 and 7). This suggestion is the basis for the nomograph described in the next section.

Nomograph and Example Computations for the Method

The method is a technique to compute Q_p and T^{Q_p} given excess rainfall for design storms for applicable watersheds in the Houston metropolitan area. The method requires graphical lookup from a nomograph of peak streamflow (cubic feet per second per 1 inch of excess rainfall) and T^{Q_p} (minutes) for the watershed. The Q_p is acquired by multiplying the excess rainfall (inches) for the design storm with the peak streamflow from the nomograph. Given an analyst-selected, watershed-loss model that is beyond the scope of this report, the excess rainfall for the design storm for the duration T_c will require an estimate of T_c.

Nomograph for the Method

To facilitate the method through graphical lookup to avoid multistep mathematics by the user, a nomograph is needed. For A values of 10, 20, 40, 80, 160, 320, and 640 acres and BDF values of 0, 3, 6, 9, and 12, the arithmetic means of Q_p and T^{Q_p} from the gamma unit hydrograph and excess rational methods for excess rainfall of 1 inch were estimated by following the computations described in the previous section. The resultant values were used as ordinates in the nomograph shown in figure 16. Subsequently, minor smoothing of the lines was performed.

Example Computations

Example computations that follow show that the nomograph can be readily used. In this example, the Q_p for a design storm on a 160-acre watershed (0.25 square miles) that has a $BDF = 6$ is needed for a design storm having a recurrence interval of 10 years in the approximate center of the Houston metropolitan area.

First, the nomograph, by graphical lookup, shows that $Q_p \approx 104$ cubic feet per second for 1 inch of excess rainfall and that $T^{Q_p} \approx 60$ minutes.

Second, an estimate of the excess rainfall is needed. The T_c for the watershed is computed by equation 19 with the addition of $(\sqrt{A[\text{acres}]/640})/2$ per equation 18. The estimate of T_r is

$$T_r = 10^{-0.05228\times(6)+0.4028\log_{10}(0.25)+0.3926}$$
$$= 0.69 \text{ hour}, \qquad (24)$$

and the T_c is

$$T_c = 0.69 + \frac{\sqrt{160/640}}{2} = \boxed{0.94 \text{ hour}}. \qquad (25)$$

To keep this example brief as possible, 0.94 hour is rounded to 1 hour, and Asquith and Roussel (2004, fig. 30, p. 37) is used to estimate the 10-year, 1-hour design storm for the approximate center of Harris County, Texas. This depth is about 2.9 inches. This storm has an average intensity of about $2.9/1 = 2.9$ inches per hour. At this average intensity, a 0.94-hour storm has a depth of about 2.7 inches (0.94×2.9).

Continuing with the example, suppose that a watershed-loss model for the watershed, which is beyond the scope of this report, yields an effective rainfall depth of about 1.4 inches for the 2.7-inch design storm (about a 50-percent loss for the example). The Q_p for the watershed thus is $1.4 \times 104 \approx \boxed{146 \text{ cubic feet per second}}$.

As a means for assessment of uncertainty in Q_p estimated from the method, the first and third quartiles of the C_r and C_v values, which are shown in figure 12, are useful. These quartiles can be used to compute lower $Q_p^{\downarrow}$ and upper $Q_p^{\uparrow}$ estimates, which might be interpreted as quartiles of Q_p. The computations of lower $Q_p^{\downarrow}$ are

$$Q_p^{\downarrow} = \frac{0.13}{0.28}\,\frac{146}{0.61} \quad \text{or}$$
$$Q_p^{\downarrow} = 0.76 \times 146 \approx 111 \text{ cubic feet per second}, \qquad (26)$$

and the computations of upper $Q_p^{\uparrow}$ are

$$Q_p^{\uparrow} = \frac{0.37}{0.54}\,\frac{146}{0.61} \quad \text{or}$$
$$Q_p^{\uparrow} = 1.12 \times 146 \approx 164 \text{ cubic feet per second}, \qquad (27)$$

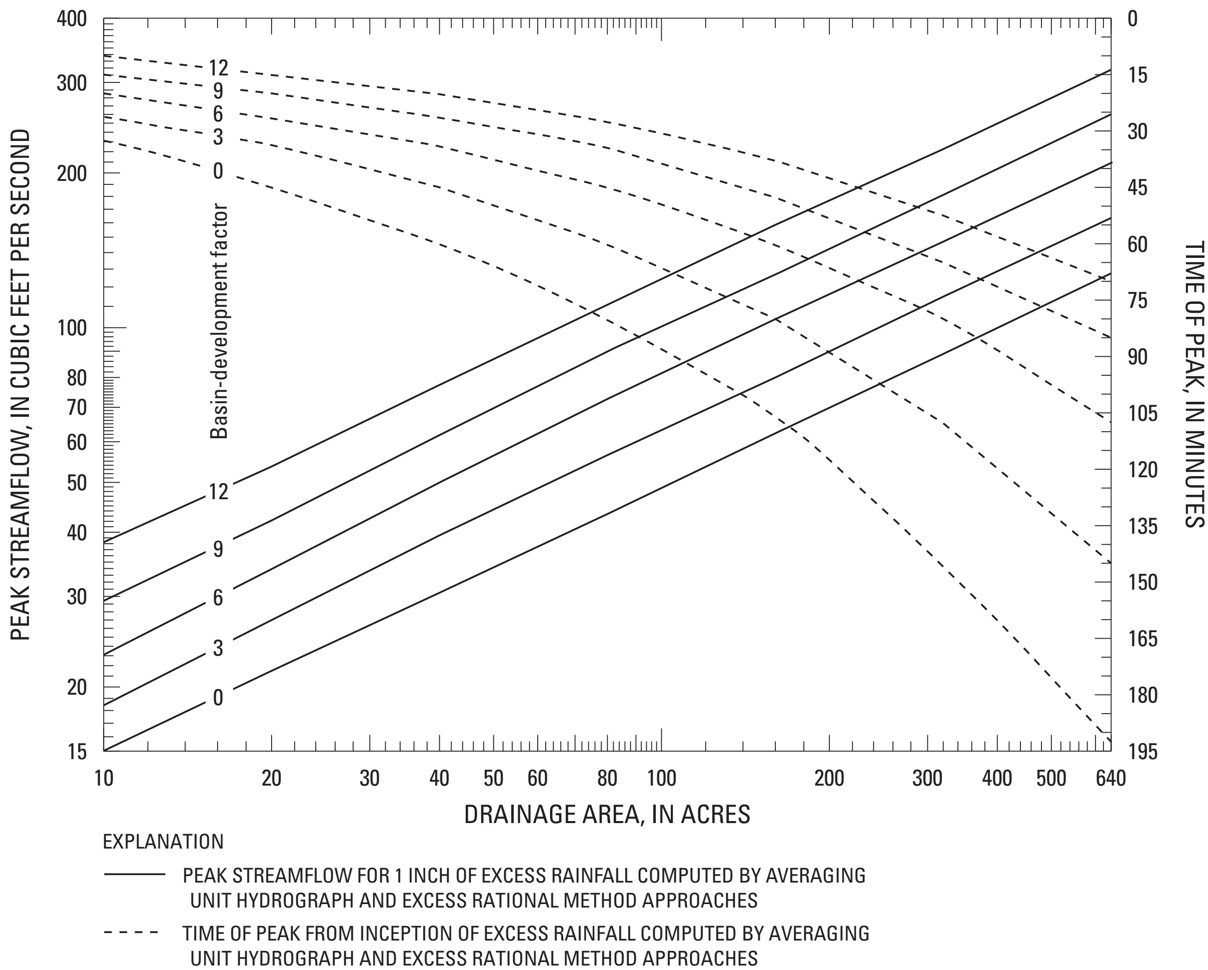


Figure 16. Nomograph of the relation between arithmetic means of peak and time of peak streamflow by gamma unit hydrograph and excess rational method computations for 1 inch of excess rainfall by selected basin-development factor and drainage area for 24 watersheds in the Houston, Texas, metropolitan area.

Table 3. Comparison of values of peak and time of peak streamflow by gamma unit hydrograph and excess rational method computations for 1 inch of excess rainfall for limiting developed and undeveloped conditions for a 300-acre watershed in the Houston, Texas, metropolitan area.

[BDF, basin-development factor; T^{Q_p}, time of peak streamflow; Q_p, peak streamflow; ERM, excess rational method; UH, unit hydrograph method; ft^3/s, cubic feet per second; $\longleftrightarrow$, direct (left to right) comparison can be made. The numerical values listed in this table are found in the text starting on page 17 and are identifiable by frames or boxes around the value and corresponding units.]

BDF (dimensionless)	${}_{\mathrm{ERM}}T^{Q_p}$ (minutes)		${}_{\mathrm{UH}}T^{Q_p}$ (minutes)	${}_{\mathrm{ERM}}Q_p$ (ft^3/s)		${}_{\mathrm{UH}}Q_p$ (ft^3/s)	T^{Q_p} (minutes)	Q_p (ft^3/s)
Developed, $BDF = 12$	45	$\longleftrightarrow$	50	244	$\longleftrightarrow$	202	48	223
Undeveloped, $BDF = 0$	130	$\longleftrightarrow$	145	84.5	$\longleftrightarrow$	88.3	138	86.4

where the value 0.61 (ratio mean C_r to mean C_v) used in the denominator is shown in equation 21. The Q_p for the 160-acre watershed that has a $BDF = 6$ estimated from method could be written as

$$\boxed{Q_p = 146 \pm 18 \mid 35 \text{ cubic feet per second}},$$

where the values 18 and 35 are computed by $18 = 164 - 146$ and $35 = 146 - 111$. Lastly, the steps of the method are shown in figure 17.

Potential Bias in the Excess Rational Method

The method was developed through conjunctive analysis of the unit hydrograph method and the excess rational method. For the excess rational method, as expressed by equation 16, a major generalization is made that the ratio of C_r to C_v is approximately 0.61 (see equation 20). The reliability of the generalization that $C_r/C_v \approx 0.61$ is important because the resultant ${}_{\mathrm{ERM}}Q_p$ is obviously in direct proportion with the ratio.

To explore the reliability, the ratios of the watershed-specific values for C_r and C_v were computed. The relation between these ratios and drainage area is shown in figure 18.

From the figure, it can be concluded that the ratio of C_r/C_v tends to be larger than about 0.61 for the larger BDF values and smaller for the smaller BDF values. But exactly how the ratio changes with BDF and the additional interaction with drainage area is uncertain because of the disparity in the distribution of low BDF values for very small watersheds. If for high BDF ($BDF \approx$ 10–12) a more appropriate generalization of the C_r/C_v ratio is about 0.9, then the ${}_{\mathrm{ERM}}Q_p$ used for the development of the method are a factor of about 0.67 too low (0.61/0.9). If for low BDF ($BDF \approx$ 0–2) a more appropriate generalization of C_r/C_v is about 0.3, then ${}_{\mathrm{ERM}}Q_p$ used in the creation of the method are a factor of about 2.0 too high (0.61/0.3).

This discussion can be generalized as meaning that the excess rational method in equation 20 has the potential for considerable overestimation of Q_p for undeveloped or lightly developed watersheds as expressed by BDF. As a result, concerns over the accuracy of the Q_p from the method naturally arise. A discussion of the relative influence of BDF on Q_p therefore is needed and provided in the next section.

On the Relative Influence of Basin-Development Factor on Peak Streamflow

Influence of Basin-Development Factor

There are many hydrologic methods to estimate Q_p for design storms including empirical equations and coupled hydrologic and hydraulic models. The method described here is but one that has specific focus on excess rainfall and thus is conceptually independent from watershed-loss models. A discussion of the relative influence of BDF on Q_p as demonstrated in this report is useful because BDF can be used as an expression of the transition of a watershed from undeveloped to developed conditions or from a given state of development back to a lesser state of development.

The influence of BDF on Q_p can be generalized in terms of $\log_{10}$-cycle change per unit change of BDF. BDFs are from 0 to 12 (13 categories) and are scored representations of the presence of various development

Implementation of the Method

1. Compute T_r, the center of the window for the maximum rainfall during the rainfall epoch, with equation 19 by using area A in square miles and basin-development factor BDF.
2. Compute T_c with equation 18 by using T_r and A in square miles.
3. Determine design storm depth (the total design rainfall, referred to as P^{storm}) for the duration T_c as well as for an appropriate annual recurrence interval.
4. Determine excess rainfall E^{storm} of the design storm depth by using a watershed-loss model; it necessarily follows that $E^{\text{storm}} < P^{\text{storm}}$.
5. Estimate the peak streamflow (identify as $Q_p^{\text{fig. 16}}$) for 1 inch of excess rainfall from figure 16 by using A in acres and BDF; linearly interpolate to BDF values not shown in the figure.
6. Compute Q_p in cubic feet per second for the design storm by $Q_p^{\text{storm}} = Q_p^{\text{fig. 16}} \times E^{\text{storm}}$.
7. Estimate the time of peak $T^{Q_p^{\text{storm}}}$ in minutes for the design storm from figure 16 by using A in acres and BDF. Linearly interpolate to BDF values not shown in the figure.

Figure 17. Steps for implementing the method for 10- to 640-acre watersheds in the Houston, Texas, metropolitan area.

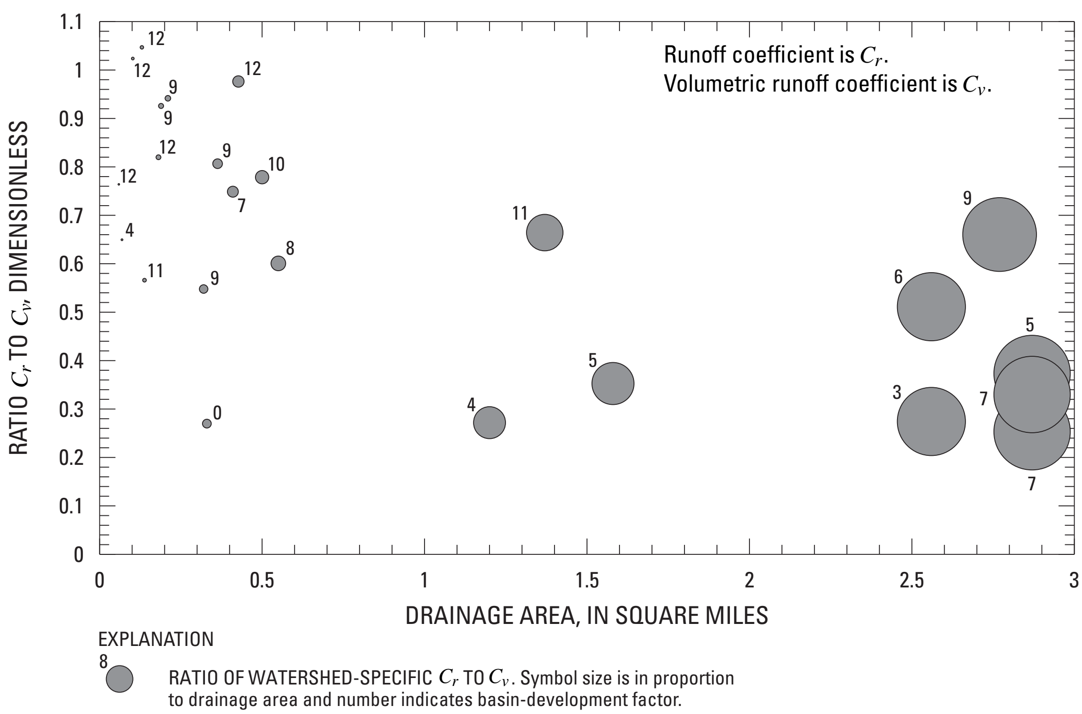


Figure 18. Graph showing relation between ratio of watershed-specific (mean) values of runoff coefficient and volumetric runoff coefficient to drainage area for 24 watersheds in the Houston, Texas, metropolitan area.

conditions on each third of a watershed. Inspection of the Q_p lines (solid) in figure 16 on the left axis shows a maximum of about 38 and a minimum of about 15, which implies a change of $0.031 \log_{10}$ cubic feet per second of Q_p per unit change in BDF or $\log_{10}(38) - \log_{10}(15) = 0.404$ and $0.404/12 = 0.034$ for the 12 discrete changes in BDF : 0–12.

In reference to the unit hydrograph method, the 0.034 value compares favorably with the coefficient of about 0.027 on BDF in equation 5 and by association the absolute value of the coefficient of about 0.034 on BDF in equation 6. (The Q_p is inversely dependent on value for T_p.)

In reference to the rational method, the 0.034 value also compares favorably with the absolute value of the coefficient of about 0.052 on BDF in equation 19.

After reviewing these various coefficients (about 0.034, 0.027, 0.034, and 0.052) and with the purpose of generalizing the influence of BDF on Q_p in terms of a single number that is appropriate for 10- to 640-acre watersheds in the Houston metropolitan area, an ad hoc value of $0.04 \log_{10}$ cubic feet per second change of Q_p per positive unit of change in BDF is deemed to be appropriate.

Example Computations

Example computations, which demonstrate the generalized influence of BDF on Q_p, are informative. For example, assume that a numerical hydrologic and hydraulic model of a 200-acre watershed with a $BDF = 12$ results in $Q_p\ |^{BDF=12} = 600$ cubic feet per second. Also for this hypothetical watershed, rainfall and runoff data only exist for $BDF = 12$ conditions. It is further assumed that the hydrologic and hydraulic model model has been reasonably calibrated with these data.

Can the estimate of $Q_p\ |^{BDF=0}$ for this watershed be obtained by using the generalized influence of BDF on Q_p? By using the suggested value of $0.04 \log_{10}$ cubic feet per second change of Q_p per positive unit of change in BDF, the computation is made for the base-10 log-transformed estimate of Q_p:

$$\Psi = \log_{10}(Q_p) + 0.04 \times (\Delta BDF) \tag{28}$$

where Q_p is the peak streamflow in cubic feet per second for the watershed and the respective BDF value and ΔBDF is the change in BDF. For a $BDF = 12 \rightarrow 0$ the $\Delta BDF = -12$ (increments), which yields, in equation 28 $(\log_{10}(600) + 0.04 \times (-12))$, and $\Psi = 2.30$. When Ψ is retransformed by 10^{Ψ}, the estimate is $Q_p\ |^{BDF=0} \approx$ $\boxed{\text{200 cubic feet per second}}$.

For a second example computation, a watershed has $BDF = 9$ and an estimated $Q_p\ |^{BDF=9} = 600$ cubic feet per second. An estimate of $Q_p\ |^{BDF=12}$ for this watershed using the generalized influence of BDF on Q_p can be computed. For a $BDF = 9 \rightarrow 12$ the $\Delta BDF = 3$, which yields, in equation 28 $(\log_{10}(600) + 0.04 \times (3))$, an $\Psi = 2.90$. When Ψ is retransformed by 10^{Ψ}, the estimate is $Q_p\ |^{BDF=12} \approx$ $\boxed{\text{791 cubic feet per second}}$.

Summary

Estimation of peak and time of peak streamflow from design storms provides for cost-effective, risk-mitigated design of drainage structures such as bridges, culverts, roadways, and other infrastructure. During 2007–10, the U.S. Geological Survey (USGS), in cooperation with the Harris County Flood Control District and the Texas Department of Transportation, developed a method to estimate peak and time of peak streamflow from excess rainfall for 10- to 640-acre low-slope watersheds in the Houston, Texas, metropolitan area. The method is based on conjunctive analysis of rainfall and runoff data in the context of the unit hydrograph method and the rational method.

For this investigation, 21 distinct watersheds (based on latitude and longitude) were identified as pertinent for the Houston metropolitan area. Pertinent watersheds were selected on the basis of drainage area as a measure of watershed size. These watersheds represent generally the smallest watersheds for which there exist paired rainfall and runoff data suitable for conjunctive analysis of the unit hydrograph method and the rational method.

The focus of the investigation is the conjunctive analysis of the unit hydrograph method and various applications of the rational method in the context of small watersheds (less than about 640 acres). To support statistical development, watersheds with drainage areas less than about 3.5 square miles (2,240 acres) were selected. Two of the watersheds were identified as having considerable changes in land development as expressed by the basin-development factor. The period of record for these two watersheds was thus segregated (three total divisions), resulting in the 24 watersheds that are used in this report. Collectively, these 21 watersheds are assumed to represent the generalized hydrologic and hydraulic conditions of many small, low-slope watersheds in the Houston metropolitan area.

For the 24 watersheds, a database of rainfall and runoff data for the selected watersheds in the Houston metropolitan area was compiled and converted to digital format as needed. The data for the 24 watersheds were obtained from various sources. Also, selected watershed characteristics of drainage area, basin-development factor, and others for the 24 watersheds were obtained from various sources. The basin-development factor *BDF* is conceptualized as a measure of runoff transport efficiency of a watershed containing interconnections of various drainage systems. *BDF*s are from 0 to 12 point (13 categories) and are scored representations of the presence of various development conditions on each third of a watershed.

For the unit hydrograph analysis, background of the unit hydrograph method and discussion of a gamma unit hydrograph are provided. A gamma distribution model of unit hydrograph shape (a gamma unit hydrograph) was chosen and parameters estimated for the storms for the 24 watersheds through matching of modeled peak and time of peak streamflow to observed values on a storm-by-storm basis. Watershed mean or watershed-specific values of peak and time to peak ("time to peak" is a parameter of the gamma unit hydrograph and is distinct from "time of peak") of the gamma unit hydrograph were recorded. Two regression equations to estimate peak and time to peak of the gamma unit hydrograph based on watershed characteristics of drainage area and basin-development factor were developed.

For the rational method analysis, background and specific mathematical analysis were provided. From analysis of the rational method for the 24 watersheds, a lag time (time-R), volumetric runoff coefficient, and runoff coefficient were computed on a storm-by-storm basis. Watershed-specific values of these three metrics were computed. A regression equation to estimate time-R based on drainage area and basin-development factor was developed. Overall arithmetic means of volumetric runoff coefficient (0.41 dimensionless) and runoff coefficient (0.25 dimensionless) for the 24 watersheds were used to express the rational method in terms of excess rainfall.

The peak and time of peak streamflow estimates from both the unit hydrograph and rational methods were combined by using the arithmetic mean of the values from each method (unit hydrograph and excess rational). Example computations are shown. These computations were used to develop the method.

The method is a technique to compute peak and time of peak streamflow given excess rainfall for design storms for applicable watersheds in the Houston metropolitan area. The method requires graphical lookup from a nomograph of peak streamflow (cubic feet per second per 1 inch of excess rainfall) and time of peak streamflow (minutes) for the watershed. The peak streamflow is acquired by multiplying the excess rainfall (inches) for the design storm with the peak streamflow from the nomograph. Given an analyst-selected, watershed-loss model, which is beyond the scope of this report, the excess rainfall for the design storm will require an estimate of time of concentration.

The nomograph shows the respective relations between peak and time of peak streamflow to drainage areas ranging from 10- to 640-acres. The nomograph also shows the respective relations for selected basin-development factors, which range from undeveloped to fully developed conditions. The nomograph represents the peak streamflow for 1 inch of excess rainfall based on drainage area and basin-development factor; the peak streamflow for design storms from the nomograph can be multiplied by the excess rainfall (inches) to estimate peak streamflow. Time of peak streamflow is readily obtained from the nomograph. Therefore, given excess rainfall values derived from watershed-loss models, which are beyond the scope of this report, the nomograph represents a method for estimating peak and time of peak streamflow for applicable watersheds in the Houston metropolitan area. Example computations of the method are provided for a hypothetical watershed along with an estimate of peak streamflow uncertainty associated with the method.

Lastly, analysis of the relative influence of basin-development factor on peak streamflow is provided and the results suggest that a $0.04 \log_{10}$ cubic feet per second change of peak streamflow per positive unit of change in basin-development factor is appropriate. This relative change can be used to adjust peak streamflow from alternative hydrologic methods for a given *BDF* to other basin-development factor values; example computations were provided.

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Glossary

Acronyms

HCFCD — Harris County Flood Control District (first use on page 2).

TxDOT — Texas Department of Transportation (first use on page 2).

ERM — Excess rational method defined on page 14 and by equation 15.

RM — Rational method defined on page 11 and by equation 9.

GUH — Gamma unit hydrograph and used in context of parameterization of such a unit hydrograph (first use on page 17).

UH — Unit hydrograph and used in context as a modifier of other variables defined in this glossary (first use on page 11).

Selected Definitions

Design Storm — A design storm is a conceptual model that can be either a hypothetical or observed storm (or combination of both) that is characterized by depth, areal extent, duration, and temporal intensities. The design storm is used as an input into one or more also conceptual or numerical models of a watershed. The design storm is often chosen to represent some type of "hazard" used to illicit hydrologic and hydraulic response of similar hazard within the watershed's interconnected water courses and channels.

Symbols

$\lightning$ — Represents "storm specific" (see page 13 and other entries in this glossary).

$|$ — Represents the two concepts of "given" (first use on page 17) and "or" (first use on page 23) as mathematical context dictates. For example, $Q_p \mid^{BDF=0}$ is to read "peak streamflow in cubic feet per second *given* a basin-development factor of zero." This is the by far the most common context for $|$ used in this report. For the other example, $Q_p = 146 \pm 18 \mid 35$ is to read "peak streamflow of 146 cubic feet per second plus 18 cubic feet per second *or* minus 35 cubic feet per second" (see page 23).

$\pm$ — Represents "plus or minus" (see page 23).

$\neq$ — Represents "not equal" as a numerical concept (see page 14).

$\approx$ — Represents "approximate" or "approximately" as a numerical concept (see page 8)

Greek Alphabet

ΔE — Change in elevation in feet (ΔE) between the two end points of the main-channel of a watershed (see page 5).

ΔBDF — Change in BDF (see page 25). For example, suppose that a major, albeit unusual, low-impact redevelopment of a watershed causes BDF to go from 10 to 8 $(10 \rightarrow 8)$. Such a watershed has $\Delta BDF = -2$, which by the methods described in this report would result in a reduction in Q_p and lengthening (dilation) of T^{Q_p}.

$\Gamma(u)$ — The complete gamma function is expressed as an infinite integral defined in equation 4 and used in equation 3.

Ψ — A quantity defined in equation 28 that represents the base-10 transformed estimate of Q_p based on ΔBDF.

Latin Alphabet

A — Drainage area (contributing) in square miles (or acres as indicated in text). A mixed unit system for A in this report was required based on the recognized traditions of the hydrologic engineering community. Values for the 24 watersheds are listed in table 1 (see page 5).

BDF — Basin-development factor thoroughly described in appendix 1. Values for the 24 watersheds are listed in table 1 (see page 5).

C — Runoff coefficient of rational method (see equation 9).

C_i — Runoff coefficient that corresponds to maximum rainfall intensity that has been determined for each unique duration (5-minute increments in this study) from the rainfall epoch (see page 14 and fig. 11).

C_r — Runoff coefficient from computations involving rational method from observed rainfall and runoff for storm duration T_c' (see page 14).

${}^{\lightning}C_r$ — The C_r value for a particular storm (see equations 12 and 13).

C_v — Volumetric runoff coefficient (total rainfall divided by total runoff, dimensionless) (see page 14).

${}^{\lightning}C_v$ — Volumetric runoff coefficient for a given storm (dimensionless, see equation 14).

C_w — Runoff coefficient for the full width of the rainfall epoch (see page 14 and fig. 11).

E — Excess rainfall in inches (see equation 15).

I — Rainfall intensity in inches per hour (see equation 9).

K — The shape parameter of gamma hydrograph (see equation 1).

L — Main-channel length: The L is defined as the length in stream-course miles of the longest defined channel from the approximate watershed headwaters to the outlet. Values for the 24 watersheds are listed in table 1 (see page 5).

P — Depth of rainfall in inches (see equation 10).

${}^{\lightning}P$ — Depth of rainfall for a particular storm in inches (see equation 14).

$P(T,F)$ — Depth of rainfall in inches as function of some storm duration T and nonexceedance probability F (see equation 10).

${}^{\lightning}P\}^{\max}_{T_c'}$ — Maximum depth of rainfall in inches for a time window (duration) having width of T_c' (see equations 12 and 13).

q — Depth of streamflow in watershed inches per hour. The relation between Q and q is seen in equation 2.

$q(t)$ — Depth of streamflow in watershed inches per hour at time t of the gamma unit hydrograph of equation 1. The relation between Q and q is seen in equation 2.

q_p — Depth of peak streamflow in watershed inches per hour of the gamma unit hydrograph of equation 1. The estimate of q_p for the gamma unit hydrograph is provided by equation 5. The relation between Q and q is seen in equation 2.

Q — Streamflow in cubic feet per second. The relation between Q and q is seen in equation 2.

Q_p — The peak or maximum instantaneous streamflow in cubic feet per second (see page 7).

$Q_p^{\downarrow}$ — The lower estimate of peak streamflow Q_p from the method in cubic feet per second (see page 21).

$Q_p^{\uparrow}$ — The upper estimate of peak streamflow Q_p from the method in cubic feet per second (see page 21).

${}_{\mathrm{ERM}}Q_p$ — Solution to Q_p from the excess rational method in cubic feet per second (see equations 15 and 16).

${}_{\mathrm{RM}}Q_p$ — Solution to Q_p from the rational method in cubic feet per second (see equation 10).

${}_{\mathrm{RM}}{}^{\lightning}Q_p$ — Peak streamflow Q_p for a particular storm from the rational method in cubic feet per second (see equations 12 and 13).

${}_{\mathrm{UH}}Q_p$ — Solution to Q_p from the unit hydrograph method in cubic feet per second (see page 11).

R — Depth of runoff in inches, see ${}^{\lightning}R$ entry in this glossary.

${}^{\lightning}R$ — Depth of runoff for a particular storm in inches (see equation 14).

S — Dimensionless main-channel slope: The S is defined as the change in elevation ΔE in feet between the two end points of L divided by L in feet: $S = \Delta E/(5{,}280 \times L)$. Values for the 24 watersheds are listed in table 1 (see page 5).

t — Time metric in hours of equation 1.

T — Rainfall duration (critical storm duration) in hours (see equation 10).

T_{LPC} — Lag-time of a watershed (see page 14).

T_b — Time base of a streamflow hydrograph (see page 8).

T_c — Time of concentration in hours (see equation 18 for the primary definition of this report).

T'_c — First-order approximation to T_c as a critical storm duration in hours that is equal to $\sqrt{A[\text{square miles}]}$ or $\sqrt{A[\text{acres}]/640}$ (see equation 11).

T_p — The time to q_p of a gamma unit hydrograph in hours of equation 1. Or in other words, the time to peak of the gamma distribution used to model the unit hydrograph. The estimate of T_p for the gamma unit hydrograph is provided by equation 6.

T^{Q_p} — The time of Q_p occurrence in hours (see page 7).

${}_{\mathrm{ERM}}T^{Q_p}$ — Solution to T^{Q_p} from the excess rational method in hours (see equation 17).

${}_{\mathrm{RM}}T^{Q_p}$ — Solution to T^{Q_p} from the rational method in hours, which is same as ${}_{\mathrm{ERM}}T^{Q_p}$ (see equation 17).

${}_{\mathrm{UH}}T^{Q_p}$ — Solution to T^{Q_p} from the unit hyrograph method in hours (see page 11).

T_r — The quantity "time-R" defined on page 14, which represents a time metric between rainfall and resulting time of peak. Equation 19 provides an estimate from analysis of rainfall and runoff described in this report.

V — The volume in depth of a runoff hydrograph and for a unit hydrograph $V = 1$ inch (see equation 3).

Appendix 1—Basin-Development Factor

Basin-Development Factor

The basin-development factor *BDF* is conceptualized as a measure of runoff transport efficiency of a watershed containing interconnections of various drainage systems (constructed or otherwise). The *BDF* is a 0–12 point, categorical variable that is a scored representation of the presence of various development conditions (scored 1 through 3) on each third of a watershed. Although a categorical variable, the *BDF* is treated as a continuous variable in a multilinear regression context herein. Values for *BDF* are defined by dividing the watershed into thirds (see fig. 1.1) and evaluating each third with respect to four indices of urbanization.

The *BDF* is considered by Sauer and others (1983) in the context of estimation of urban flooding potential and is shown to be a useful predictive variable for that purpose. The *BDF* description that follows is a near verbatim quote by Sauer and others (1983, p. 8), who state:

> The most significant index of urbanization that resulted from [the 1983] study is a basin-development factor (*BDF*), which provides a measure of the efficiency of the drainage system. This parameter, which proved to be highly significant in the regression equations [of the 1983 report], can be easily determined from drainage maps and field inspections of the drainage basin. The basin is first divided into thirds as [shown in figure 1.1]. Then, within each third, four aspects of the drainage system are evaluated and each assigned a code as follows:
>
> 1. Channel improvements.—If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), then a code of 1 is assigned. Any or all of these improvements would qualify for a code of 1. To be considered prevalent, at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of zero is assigned.
> 2. Channel linings.—If more than 50 percent of the length of the main drainage channels and principal tributaries has been lined with an impervious material, such as concrete, then a code of 1 is assigned to this aspect. If less than 50 percent of these channels is lined, then a code of zero is assigned. The presence of channel linings would obviously indicate the presence of channel improvements as well. Therefore, this is an added factor and indicates a more highly developed drainage system.
> 3. Storm drains, or storm sewers.—Storm drains are defined as enclosed drainage structures (usually pipes), frequently used on the secondary tributaries where the drainage is received directly from streets or parking lots. Many of these drains empty into open channels; however, in some basins they empty into channels enclosed as box or pipe culverts. When more than 50 percent of the secondary tributaries within a subarea (third) consists of storm drains, then a code of 1 is assigned to this aspect; if less than 50 percent of the secondary tributaries consists of storm drains, then a code of zero is assigned. It should be noted that if 50 percent or more of the main drainage channels and principal tributaries are enclosed, then the aspects of channel improvements and channel linings would also be assigned a code of 1.
> 4. Curb-and-gutter streets.—If more than 50 percent of a subarea (third) is urbanized (covered by residential, commercial, and/or industrial development), and if more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters, then a code of 1 would be assigned to this aspect. Otherwise, it would receive a code of zero. Drainage from curb-and-gutter streets frequently empties into storm drains.
>
> The above guidelines for determining the various drainage-system codes are not intended to be precise measurements. A certain amount of subjectivity will necessarily be involved. Field checking should be performed to obtain the best estimate. The basin-development factor (*BDF*) is the sum of the assigned codes; therefore, with three subareas (thirds) per basin, and four drainage aspects to which codes are assigned in each subarea, the maximum value for a fully developed drainage system would be [$BDF = 12$]. Conversely, if the drainage system were totally undeveloped, then a *BDF* of zero would result. Such a condition does not necessarily mean that the basin is unaffected by urbanization. In fact, a basin could be partially urbanized, have some impervious area, have some improvement of secondary tributaries, and still have an assigned *BDF* of zero. As is discussed later in [the 1983 report], such a condition still frequently causes peak discharges to increase.

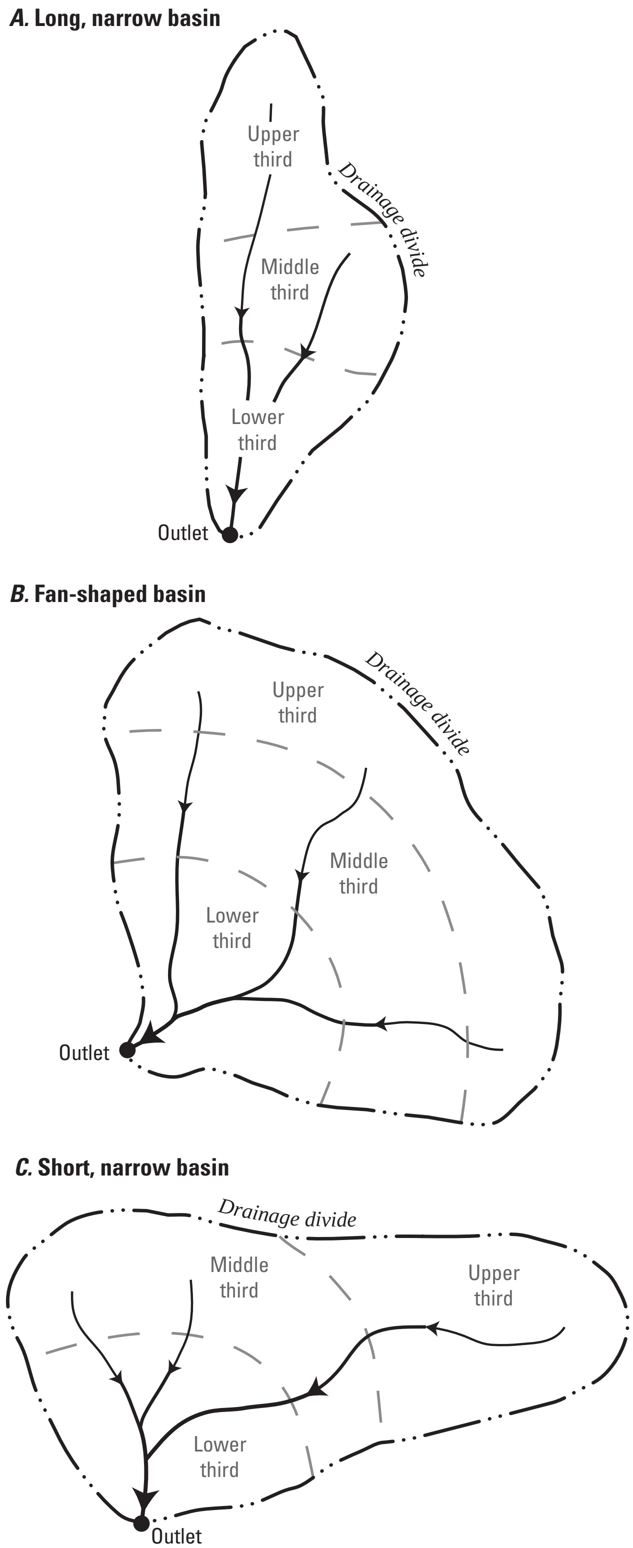

Figure 1.1. Diagram of typical drainage basin shapes and subdivision into basin thirds. Note that the stream-channel distances within any given third of a basin in the examples are approximately equal, but between basin thirds the distances are not equal, to compensate for relative basin width of the thirds.

> The *BDF* is a fairly [straightforward] index to estimate for an existing urban basin. The 50-percent guideline will usually not be difficult to evaluate because many urban areas tend to use the same design criteria, and therefore have similar drainage aspects, throughout. Also, the *BDF* is convenient for projecting future development. Obviously, full development and maximum urban effects on peaks would occur when $BDF = 12$. Projections of full development or intermediate stages of development can usually be obtained from city engineers.
>
> A basin-development factor was evaluated for each of the 269 sites used in [the 1983] study. Approximately 30 people were involved in making these evaluations, using guidelines similar to the ones described in the preceding paragraphs but somewhat less explicit. Tests have not been made to see how consistently two or more people can estimate the *BDF* for a basin. However, this study indicates that fairly consistent estimates can be made by different people. A relatively large group of individuals made the estimates for this study and the parameter was statistically very significant in the regression equations. If the results obtained by various individuals had not been consistent, it is doubtful that the statistical results [reported in the 1983 study] would be so significant.

Values of the *BDF* change with urbanization; therefore, *BDF* values need to be redefined whenever significant changes take place within a watershed. Further, because the current (2010) investigation is focused on watersheds in the Houston, Texas, metropolitan area, select modifications to the method of *BDF* computation by Sauer and others (1983) were needed (Fred Liscum, PBS&J Inc. [now (2011) with HCFCD], written commun., 2008). Modifications to the method were deemed important in order to account for conditions commonly encountered in the Houston metropolitan area. These modifications consider (1) the presence of enclosed channels and (2) the prevalence of roadside ditch drainage in a particular watershed.

The following discussion presents suggested modifications to the *BDF* method described by Sauer and others (1983). These modifications were used for the 24 watersheds considered in the current (2010) investigation. The four “aspects” of Sauer and others (1983) are summarized and the modification shown in italic type.

1. **Channel Improvements**—If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries, then a code of 1 is assigned. Otherwise, a code of 0 is assigned *unless one of the following applies:*
    - *If 50 percent or more of the main drainage channel and/or principal tributaries are enclosed (for example by storm sewers), then a code of 1 is assigned.*
    - *For areas with roadside ditch drainage that satisfy the 50 percent or more criteria, a code of 0.5 is assigned in lieu of the normal value of 1.*
2. **Channel Linings**—If more than 50 percent of the length of the main drainage channel and principal tributaries has been lined with an impervious material such as concrete, a code of 1 is assigned. Otherwise, a code of 0 is assigned *unless one of the following applies:*
    - *If 50 percent or more of the main drainage channel and/or principal tributaries are enclosed (for example by storm sewers), then a code of 1 is assigned.*
    - *For areas with roadside ditch drainage that satisfy the 50 percent or more criteria, a code of 0.5 is assigned in lieu of the normal value of 1.*
3. **Storm Sewers**—If more than 50 percent of the main channel and secondary tributaries are enclosed as storm sewers, a code of 1 is assigned. Otherwise, a code of 0 is assigned.
    - *In the absence of channels (conventional open-channels), the main trunk of the storm sewer system is treated as a “lined” channel for the purposes of scoring the* ***Channel Linings*** *aspect.*
    - *In the absence of channels (conventional open-channels), the main trunk of the storm sewer system is scored as* ***Channel Improvements****.*
4. **Curb-and-Gutter Streets**—If more than 50 percent of a third is urbanized, and if more than 50 percent of the streets and roads within the area are constructed with curbs and gutters, a code of 1 is assigned. Otherwise, a code of 0 is assigned.

Reference

Sauer, V.B., Thomas, W.O., Stricker, V.A., Wilson, K.V., 1983, Flood characteristics of urban watersheds in the United States: U.S. Geological Survey Water-Supply Paper 2207, 63 p. (Also available at *http://pubs.er.usgs.gov/djvu/WSP/wsp_2207.djvu*.)

Appendix 2—Selected Unit Conversions

The relation between q in inches per hour and Q in cubic feet per second needed for equation 2 is readily obtained by dimensional analysis as follows:

$$Q\ \frac{\text{feet}^3}{\text{second}} = q\ \frac{\text{inches}}{\text{hour}} \times A\ \frac{\text{miles}^2}{1} \times \underbrace{\frac{1\ \text{hour}}{3{,}600\ \text{seconds}} \times \frac{1\ \text{foot}}{12\ \text{inches}} \times \frac{5{,}280^2\ \text{feet}^2}{1\ \text{mile}^2}}_{645.33},$$

which upon simplification and dropping of units becomes $Q = 645.33 \times q \times A$.

The unit-conversion factor 1.008 for the rational method in equation 9 is readily obtained by dimensional analysis as follows:

$$Q\ \frac{\text{feet}^3}{\text{second}} = \kappa\ \text{dimensionless} \times I\ \frac{\text{inches}}{\text{hour}} \times A\ \frac{\text{acre}}{1} \times \underbrace{\frac{1\ \text{hour}}{3{,}600\ \text{seconds}} \times \frac{1\ \text{foot}}{12\ \text{inches}} \times \frac{208.7^2\ \text{feet}^2}{\text{acre}}}_{1.008},$$

which upon simplification becomes $\kappa = 1.008$.

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