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The Rational Method, Regional Regression Equations, and Site-Specific Flood Frequency Relations

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Texas Department of Transportation

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16. Abstract:

A study of 20 undeveloped watersheds with drainage areas from 1.26mi2 to 124mi2, watershed main channel length from 10,300ft to 175,000ft, and dimensionless main channel slope from 0.003 to 0.02, was undertaken to examine the relation between drainage area and appropriate technology for estimating watershed hydrologic response. The rational method, the existing set of regional regression equations, and two new sets of regional regression equations were applied to estimate 5-, 10-, 25-, 50-, and 100-year discharges. In addition, either the four-parameter kappa distribution or the generalized logistic distribution was fit to the annual series of maximum discharges for each watershed. The fitted distribution served as the basis for comparison of the other methods. Watershed area was not a discriminator for appropriate hydrologic method. The Kerby and Kirpich equations for time of concentration produced reasonable results, consistent with results reported in other work. Estimates of discharge using newly developed PRESS-minimized regression equations were subject to less bias than either the regression equations currently in use or the logarithm-transformed equations developed as part of this study. Additional work is required to extend results from the 20 study watersheds to a larger database, including watersheds subject to urbanization.

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By
David B. Thompson

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1. Background

A significant task faced by Texas Department of Transportation (TxDOT) designers is estimation of design discharges for a wide range of watershed drainage areas. Consistent with current design guidance[1], TxDOT designers use four methods for estimating design discharges for hydraulic designs: (1) the rational method, (2) the unit hydrograph method, (3) regional regression equations, and (4) if data are available, the log-Pearson Type III probability distribution (LPIII) fit to the series of annual maxima. One of the criteria for method selection is watershed drainage area, although no strict boundaries exist. As general guidance, the rational method is used for very small watersheds (less than about 200 acres), the unit hydrograph procedure for small watersheds (less than about 20 mi^2, including those less than 200 acres), and statistical approaches (regional regression equations, statistical analysis of the series of annual maximum stream discharge, or similar) for watersheds with drainage areas exceeding about 20 mi^2. Of course, these broad guidelines for watershed drainage area can be ignored based on the designer's judgment.

Based on conversations with TxDOT personnel, there is little institutional memory as to why these particular drainage area guidelines exist.[2] Furthermore, there are discrepancies between design discharges computed using the hydrograph method,[3] the rational method, and the regional regression equations of Asquith and Slade (1997). Attempts to reconcile results from these procedures have been less than satisfactory.[4]

[1] See the 2002 edition of the hydraulic design manual (located electronically at `http://manuals.dot.state.tx.us/dynaweb/colbridg/hyd` at the time of this writing) for details on current TxDOT design practices.

[2] For a lower limit of application of their regional regression equations, Asquith and Slade (1997) suggest 10 mi^2, but Asquith (personal communication, 2005) prefers 10–20 mi^2.

[3] Although new research results have been published, they have not yet been widely deployed by TxDOT. As of this writing, the Natural Resources Conservation Service (NRCS) dimensionless unit hydrograph is used to represent watershed response, the NRCS runoff curve number method is used for estimating effective precipitation (runoff) from rainfall, and the NRCS rainfall distributions (U.S. Department of Agriculture, Natural Resources Conservation Service, 1997) are used for the storm temporal distribution.

[4] These observations came from a series of discussions between project researchers and TxDOT personnel that span a number of years.

Although systematic testing and comparison of the procedures has not been undertaken, it is clear that substantial uncertainty exists in the minds of TxDOT designers. This uncertainty might be partially addressed by research currently underway. TxDOT is addressing portions of the hydrologic process, including runoff generation from rainfall (NRCS curve number technique), unit hydrographs for Texas watersheds, estimation of the time-response of Texas watersheds, and hyetographs for Texas storms.

Consequently, it is desirable to examine the relation between watershed scale and hydrologic variables for a wide range of drainage areas and geographic locations. The primary objective of this project is to explore issues related to watershed scale, particularly scale as represented by watershed drainage area. That is, to examine the relation between hydrologic processes, such as watershed response to rainfall, and watershed drainage area. Ancillary objectives are to determine, if possible, the relation between watershed scale and watershed response processes, to determine how to apply analytic technology to model the relevant processes.

The previous work of Dunne and Black (1970) on how partial source areas and variable source areas generate runoff excess is of interest to Texas hydrologists. The idea that Horton's infiltration equation could represent not only infiltration capacity, but the fraction of the watershed generating surface runoff (Betson, 1964) is an interesting idea that bears exploration. Also, work in other arid and semi-arid regions of Israel (Zaslavsky and Sinai, 1981a,b,c,d,e) is important because of the similarity with the western regions of Texas.

The work of Mandelbrot (1983) on chaotic-dynamics (or chaos theory) received significant attention during the mid and late 1980's. In response, a number of hydrologists examined the relation between watershed scale and watershed response. Some of the scale research mimiced that of Mandlebrot (and other mathematical researchers) in attempts to apply a chaotic-dynamic systems approach to hydrologic responses of watersheds. Unfortunately, chaotic-dynamic systems are usually based on linear equations or linear systems of equations. Because hydrologic systems are generally non-linear (for turbulent flow, velocity is proportional to the square-root of topographic slope), the early adopters of chaos theory did not find the approach suggested by Mandlebrot and other mathematical chaoticists useful. However, the research reinforced the importance of watershed scale issues in hydrologic analysis. Researchers became increasingly aware that watersheds may exhibit disparity in the relative importance of specific hydrologic processes, depending on watershed scale, as represented by drainage area or other scale measures. For example, for a regional riverine system, channel storage terms might be relatively more important than soil infiltration terms; whereas for much smaller watersheds, infiltration terms might predominate and channel storage terms might not exert substantial impact on watershed response as measured at the watershed outlet.

The importance of watershed scale is reflected in recent hydrologic research in Texas. Roussel et al. (2006) observed that watershed time of concentration can be estimated considering two components—overland flow and channel flow. Roussel et al. (2006) report that an estimate of 30 minutes for overland flow time of concentration is reasonable for selected Texas watersheds that formed the basis for that study.[5] As a result, as watershed scale increases, the portion of watershed time of concentration attributable to overland flow becomes an increasingly small component, until at a watershed time of concentration of 300 minutes it constitutes only about 10 percent of watershed time of concentration. Clearly, time of concentration is sensitive to watershed scale. In this example, watershed scale would be measured by the length and slope of the main channel and the length and slope of overland flow, both ostensibly related to watershed drainage area.

According to the research problem statement, the objective of this research project is *to examine the nature of input-response relation for Texas, to assess the viability of various approaches, and to develop or recommend methodology for use in hydrologic modeling in these areas. The literature review will include scale issues for watersheds in general.* Clearly, understanding the size boundaries for various hydrologic techniques would be of interest to TxDOT because of the apparently arbitrary size limitations established in current TxDOT practice.

The objective of this report is to present results from the research, interpretation of those results, and recommendations for further work. The principal researchers were Dr. David B. Thompson[6], Texas Tech University (TTU), and Dr. William H. Asquith, U.S. Geological Survey (USGS).

[5] Roussel et al. (2006) assert the works of Kerby (1959) and Kirpich (1940) produce reasonable estimates of watershed time of concentration, and the works of Kerby and Kirpich are preferred by the author.

[6] In addition, several graduate and undergraduate students played an important role in the development of this research. They are Kirt Harle, Ashish Waghray, Cindy Jones, and George "Rudy" Herrmann, all of whom completed graduate studies under Dr. Thompson's direction.

2. Procedure

In the problem statement and proposal for this project, the following tasks are listed:

1. Literature review
2. Other departments of transportation
3. Regionalization of Texas
4. Identification of applicable modeling techniques
5. Documentation

2.1. Literature Review

A literature review (Thompson, 2004) was prepared and submitted to TxDOT. Thompson (2004) contains references to and analysis of research on hydrologic scale conducted over the last 50 years. The reader is directed to Thompson (2004) for that information. References important to the current (2006) report will be cited as needed.

2.2. Other Departments of Transportation

As part of this study, personnel in departments of transportation in adjacent and special-interest states were queried for hydrologic practices. The basic questions put to personnel of those agencies was "*What hydrologic technologies are used and how are decisions made by designers on selection of appropriate hydrologic technology for a given design.*"

Four states border Texas: New Mexico, Louisiana, Arkansas, and Oklahoma. In addition, California was included in the list of states to be polled. These states were divided up among the researchers for contact. David Stolpa (TxDOT) was instrumental in providing contact information for the state hydraulic engineers.

2.3. Regionalization of Texas

Based on results of the literature review and on the collective experience of the research team, it was decided to *not* subdivide Texas into regions for further study. In an earlier study, Asquith and Slade (1997) developed regional regression equations based on subdivision of Texas into numerous regions. The Asquith and Slade (1997) equations are part of the current (2006) TxDOT hydraulic design guidelines.[1] As a part of the research reported herein, Asquith and Thompson (2005) took the opposite approach and consolidated the dataset used by Asquith and Slade (1997). Asquith and Thompson (2005) generated four sets of regional regression equations, each set comprising six equations, and each equation to estimate an n-year return-interval event applicable to the entire state of Texas.[2]

2.4. Applicable Modeling Techniques

2.4.1. Database

As part of other TxDOT research projects, researchers from Texas Tech University and USGS were joined by researchers from Lamar University and the University of Houston on a pair of research projects to develop a unit hydrograph (TxDOT project 0–4193) and a rainfall hyetograph (TxDOT project 0–4194) for use in executing TxDOT designs. These agencies pooled personnel resources to enter data representing 1,659 storms and runoff hydrographs for 91 watersheds. These data were extracted from USGS small-watershed studies (in excess of 220 paper reports) stored in USGS archives (Asquith et al., 2004).

The resulting database comprised 91 watersheds and about 1,600 hydrologic events. The database is housed on a workstation located at Texas Tech University with regular backups to Austin-based USGS computers. The majority of the watersheds represented in Asquith et al. (2004) are located in west central Texas near the I-35 corridor; a few are located in the eastern and western regions of the state and along the Gulf coast.

The original database of 91 watersheds was reviewed to extract watersheds with at least ten years of record of annual maxima and no significant urbanization or development.

[1]See the 2002 edition of the hydraulic design manual (located electronically at `http://manuals.dot.state.tx.us/dynaweb/colbridg/hyd` at the time of this writing), for details on current TxDOT design practices.

[2]The Asquith and Thompson (2005) report is a significant product of TxDOT project 0–4405. Results from Asquith and Thompson (2005) are included herein, as appropriate.

The purpose of this filtering was to ensure data were available to support development of site-specific flood-frequency curves and to ensure applicability of Asquith and Slade (1997). After reviewing station records, 20 watersheds were selected for further study. The locations of study watersheds are shown in figure 2.1. Basic watershed characteristics are shown in table 2.1.

The minimum drainage area of watersheds in the study database is 0.77 square miles, about 500 acres. This exceeds the TxDOT guideline of 200 acres for application of the rational method. In the literature review, Thompson (2004) reports that other researchers applied the rational method to watersheds with drainage areas exceeding 200 acres. The source of the 200-acre limit is unknown and TxDOT institutional memory is vague on its genesis. There remains no technical reason for limiting application of the rational method at this juncture, certainly not for the purposes of a research project. Therefore the 200-acre limit on drainage area for the rational method was not used for the research reported herein.

Table 2.1: Watershed characteristics

TTU Watershed ID	USGS Gage Station ID	USGS Quadrangle Location	Drainage Area (mi^2)	Channel Length (ft)	Channel Slope (ft/ft)
1003	08088100	True	11.8	24,874	0.003
1004	08093400	Abbot	12.4	52,689	0.004
1007	08160800	Frelsburg	17.3	42,407	0.005
1008	08167600	Fischer	10.9	29,393	0.017
1108	08156800	Austin	12.3	55,144	0.008
1117	08158700	Austin	124	175,221	0.004
1122	08158840	Austin	8.24	25,690	0.010
1407	08178640	San Antonio	2.45	15,786	0.020
1412	08181400	San Antonio	15.0	51,257	0.011
1603	08098300	Cameron	23.0	72,642	0.003
2008	08096800	Moody	5.47	34,715	0.006
2302	08137000	Bangs	4.02	22,238	0.004
2501	08182400	Martinez	7.01	25,586	0.006
2601	08187000	Lenz	3.29	14,477	0.010
2612	08187900	Lenz	8.43	25,011	0.005
2701	08050200	Muenster	0.77	13,497	0.011
2802	08058000	Weston	1.26	10,283	0.009
2903	08052700	Marilee	75.5	122,054	0.002
3002	08042700	Senate	21.6	60,538	0.006
3101	08063200	Coolidge	17.6	45,847	0.004

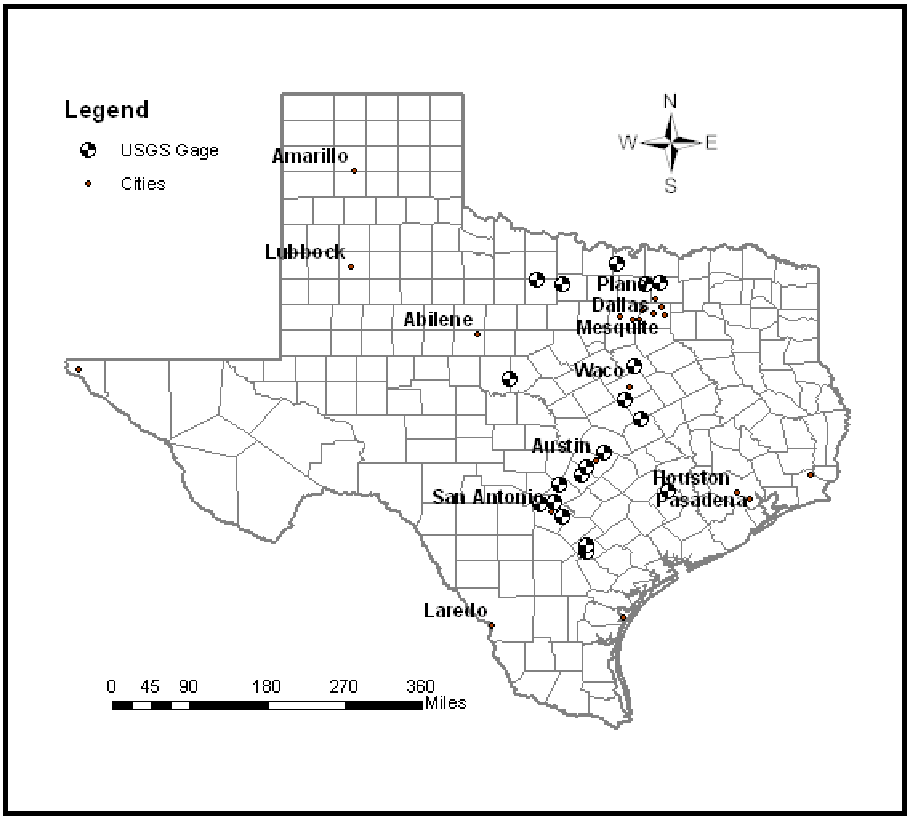

Figure 2.1: Location of selected watersheds

2.4.2. Basis for Comparison

When evaluating results of hydrologic methods, some basis for comparison must be selected. Whereas it would be preferable to compare to some absolute quantity (truth), what we have is a series of measurements, the sequence of annual maxima, from the study watersheds. Because the period of record was chosen to be at least ten years, an estimate of the frequency-based discharge from the study watersheds can be constructed. This was accomplished by fitting a frequency distribution, either a kappa or a generalized logistic distribution (Hosking and Wallis, 1997), to the series of annual maxima. The L-moment procedure was used to develop distribution parameters from observed annual maxima. Details of the procedure are presented in Appendix B.

2.4.3. Rational Method

The rational method is over 100-years old and was first presented in a paper by Kuichling (1889). The rational method commonly is applied to very small watersheds to generate discharges for design of small drainage structures. In TxDOT practice, the rational method often is applied to watershed drainage areas less than about 200 acres. However, as discussed in Section 2.4.1 (Database), for the purposes of the research the rational method was applied to estimate peak discharges from study watersheds regardless of drainage area. It is understood that this is not standard practice, but for the purposes of comparison with results from other methods, it was used. There are precedents for this decision (Bengtsson and Niemczynowicz, 1998; Chui et al., 1994; Madramootoo, 1989; Pilgrim et al., 1989).[3]

Application of the rational method is based on a simple formula that relates runoff-producing potential of the watershed, average intensity of rainfall for a particular length of time (the time of concentration), and watershed drainage area. The formula is

$$Q = C_u C i A, \tag{2.1}$$

where:

$$\begin{aligned} Q &= \text{design discharge } (\mathrm{L}^3/\mathrm{T}), \\ C_u &= \text{units conversion coefficient}, \\ C &= \text{runoff coefficient (dimensionless)}, \\ i &= \text{design rainfall intensity } (\mathrm{L}/\mathrm{T}), \text{ and} \\ A &= \text{watershed drainage area } (\mathrm{L}^2). \end{aligned}$$

The units conversion coefficient, C_u, is necessary because the iA product, although having units of L^3/T, is not a standard unit in the traditional units system.[4]

The runoff coefficient, C, is a dimensionless ratio intended to indicate the amount of runoff generated by a watershed given a average intensity of precipitation for a storm. Whereas it is implied by the rational method that intensity of runoff is proportional to intensity of rainfall, calibration or computation of the runoff coefficient has almost

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[3] The decision of the researchers to apply the rational method to watersheds regardless of drainage area is not intended to suggest that the rational method is *appropriate* for all watersheds. Substantial experience and care is required of the designer when applying *any hydrologic method* for determination of a design discharge.

[4] The product of the dimensions of i, and A, is acre-inches per hour in traditional units. Dimensional analysis of acre-inches per hour shows that it is equivalent to 1.00833 cubic feet per second (cfs). This is close enough to unity to be used as an equivalence for most cases.

always depended on comparing the total depth of runoff to the total depth of precipitation,

$$C = \frac{R}{P}, \tag{2.2}$$

where:

$R =$ Total depth of runoff (L), and
$P =$ Total depth of precipitation (L).

The runoff coefficient represents the fraction of rainfall converted to runoff.

Estimation of the runoff coefficient requires further discussion. Typically the runoff coefficient is estimated from tables based on watershed characteristics, particularly soil type and land cover. For this study, a slight variation on the method of Schaake et al. (1967) was applied to estimate the actual runoff coefficient from site-specific measurements of rainfall and runoff.[5] The procedure for estimating the runoff coefficient is similar to that used by Hjelmfelt (1980) and Hawkins (1993) for estimating the NRCS runoff curve number from rainfall-runoff events. In addition, values for the runoff coefficient were estimated from standard tables for comparison.

Basically, the procedure is to rank-order rainfall depth and runoff depth from largest to smallest then compute the runoff coefficient using equation 2.2 applied to the rank-ordered pairs. A plot of runoff versus runoff coefficient is then prepared and the value of the runoff coefficient for relatively large values of runoff is selected.[6] The resulting estimate of the runoff coefficient is termed the *observed runoff coefficient* because it stems from observations of events with substantial rainfall and runoff. *The observed runoff coefficient is the most appropriate estimate that can be made for a particular watershed because it is based on observations of rainfall and runoff.*

The rational method was then applied to each study watershed. The runoff coefficient used was the observed runoff coefficient. Time of concentration was estimated using the combination of Kerby (1959) and Kirpich (1940).[7] Storm intensity-duration-frequency (IDF) parameters (for equation 2.5, presented later in the text) were taken from the

[5] This is a reason for selection of watersheds with at least ten years of record.

[6] For some study watersheds, one or more events were present in the database with very large values of rainfall and runoff. As a result, the observed runoff coefficient for these events was thought inappropriate for estimation of discharge from relatively rare, but not such extreme events. This issue is discussed further in the Results section of this report.

[7] TxDOT project 0–4696 was recently completed (August 2005) and the results documented in Roussel et al. (2006). Roussel et al. (2006) concluded that the sum of Kerby (1959) and Kirpich (1940) is a reasonable estimate for time of concentration for Texas watersheds.

TxDOT hydraulic design guidelines.[8][9] Discharge estimates were computed for the 2-, 5-, 10-, 25-, 50-, and 100-year events.

The time of concentration for channel flow is computed using Kirpich (1940),

$$t_c = 0.0078(L^3/h)^{0.385}, \tag{2.3}$$

where:

$$\begin{aligned} t_c &= \text{time of concentration for channel flow (minutes)}, \\ L &= \text{length of main channel (feet), and} \\ h &= \text{relief (change in elevation) from outlet to distal end of watershed (feet).} \end{aligned}$$

The overland flow component of time of concentration for undeveloped watersheds is computed using Kerby (1959),

$$t_o = \left(\frac{0.67NL}{S^{0.5}}\right)^{0.467}, \tag{2.4}$$

where:

$$\begin{aligned} t_o &= \text{overland flow time of concentration (minutes)}, \\ L &= \text{length of overland flow (feet)}, \\ N &= \text{Kerby's retardance coefficient, and} \\ S &= \text{overland flow slope.} \end{aligned}$$

Values for Kerby's retardance parameter are presented in table 2.2. The watershed, or total, time of concentration is the sum of t_o and t_c.

As alluded to above, IDF curves are used to estimate average rainfall intensity, i, for the watershed time of concentration, t_c. TxDOT uses equation 2.5 for approximating the IDF curve,[10]

$$i = \frac{b}{(t_c + d)^e}, \tag{2.5}$$

where:

$$\begin{aligned} i &= \text{design rainfall intensity (in/hr)}, \\ t_c &= \text{time of concentration (min), and} \\ b, d, e &= \text{parameters.} \end{aligned}$$

[8] See the 2002 version of the TxDOT Hydraulic Design Guidelines, (`http://manuals.dot.state.tx.us/dynaweb/colbridg/hyd` at the time of this writing).

[9] Curiously, Asquith and Roussel (2004) are not present.

[10] See the 2002 version of the TxDOT Hydraulic Design Guidelines, (`http://manuals.dot.state.tx.us/dynaweb/colbridg/hyd` at the time of this writing).

Table 2.2: Kerby's retardance parameter

Description	N
Pavement	0.02
Smooth, bare packed soil	0.10
Poor grass, cultivated row crops or moderately rough bare surfaces	0.20
Pasture, average grass	0.40
Deciduous forest	0.60
Dense grass, coniferous forest, or deciduous forest with deep litter	0.80

Whereas the exact genesis of the parameters b, d, and e is not published, the values were extracted from analysis of depth-duration-frequency relations from TP–40 (U.S. Weather Bureau, 1963) and HYDRO–35 (National Oceanic and Atmospheric Administration, 1977) according to Smith (1997).

2.4.4. Regional Regression Equations

In general, regional regression equations are developed for state departments of transportation by the USGS. This is certainly true in the case of Texas. Asquith and Slade (1997) presented results of an analysis of data from 664 streamgages on undeveloped watersheds in which 96 regression equations were produced. The number of regions defined in Asquith and Slade (1997) and the protocol presented for some regions for handling watersheds with drainage areas less than 32 mi^2, indicates that the hydrology of Texas watersheds is complex.

Although the regression equations presented in Asquith and Slade (1997) are in general use in Texas, some problems have been experienced.[11] As a result, the approach was revisited as part of this research project. Asquith and Thompson (2005) reported results from revisiting the dataset collected and reported in Asquith and Slade (1997).

Four sets of regional regression equations are presented in Asquith and Thompson (2005). Two sets are based on a logarithmic transformation of regressor variables and

[11] In conversations with TxDOT personnel and with Asquith, the regional regression equations of Asquith and Slade (1997) have questionable performance for watersheds less than 20–30 mi^2.

two sets are based on a power-transformation of the watershed drainage area, A^{λ}. All regression equations are based on least-squares regression to extract the regression coefficients. However, for the power-transformation of watershed drainage area, minimization of the PRESS statistic is used to determine the most appropriate value of λ.

The set of equations based on a log-transform of regressor variables used in this research are

$$Q_2 = 10^{2.339} A^{0.5158}, \tag{2.6}$$
$$Q_5 = 10^{2.706} A^{0.5111}, \tag{2.7}$$
$$Q_{10} = 10^{2.892} A^{0.5100}, \tag{2.8}$$
$$Q_{25} = 10^{3.086} A^{0.5093}, \tag{2.9}$$
$$Q_{50} = 10^{3.209} A^{0.5092}, \text{ and} \tag{2.10}$$
$$Q_{100} = 10^{3.318} A^{0.5094}, \tag{2.11}$$

where:

$$Q_n = n\text{-year peak discharge (cfs), and}$$
$$A = \text{watershed drainage area (square miles).}$$

An alternative set of regression equations, also based on log-transformation of regressor variables, is presented in Asquith and Thompson (2005) which contain additional regressor variables. However, the alternate set of equations was not used in this research.

Two additional sets of regression equations were developed by Asquith and Thompson (2005). Those used in this research are given by

$$Q_2 = 10^{8.280-6.031A^{-0.0465}}, \tag{2.12}$$
$$Q_5 = 10^{7.194-4.614A^{-0.0658}}, \tag{2.13}$$
$$Q_{10} = 10^{6.961-4.212A^{-0.0749}}, \tag{2.14}$$
$$Q_{25} = 10^{6.840-3.914A^{-0.0837}}, \tag{2.15}$$
$$Q_{50} = 10^{6,806-3.766A^{-0.0890}}, \text{ and} \tag{2.16}$$
$$Q_{100} = 10^{6.800-3.659A^{-0.0934}}. \tag{2.17}$$

As previously alluded to, equations 2.12–2.17 were based on a method for minimizing the PRESS statistic, equation B.14, developed by Asquith and Thompson (2005). Minimization of the PRESS statistic is an extension to the weighted least squares approach and has the potential for removing bias from the fitted equations.[12] The approach is to

[12] The ability to fit equations of forms different than $y = aX_1^b X_2^c \ldots$ is addressed in Appendix B.

find the best estimate for λ in A^λ by searching for values of λ that minimize the PRESS statistic. Details are presented in Appendix B.

A second set of PRESS-minimized regression equations were also developed by Asquith and Thompson (2005). The latter included additional regressor variables. However, the alternate set of equations were not used in this research and so are not presented here.

3. Results

3.1. Literature Review

The literature review was written and published (Thompson, 2004). Although a substantial amount of work on scale issues in hydrology exists, the bulk of that work focused on issues other than the relation between watershed scale and appropriate computational technology.

In the conclusions of the literature review, Thompson (2004) writes:

> The results from much research, analysis, and reviews support the fact that spatial variation in rainfall, soil properties, and topography has a direct affect on the accuracy of watershed runoff simulations for drainage areas ranging in size from 2 ha to 520 km^2 (5 ac to 200 mi^2). The presence of spatial variation and heterogeneity in rainfall fields certainly affects discharge from the watershed. Soils and topography have a direct affect on the source areas of watershed runoff. Therefore, these findings demonstrate the importance of identifying size limitations for use of the various hydrologic modeling technologies.
>
> Significant efforts to address the issue of spatial variability associated with scale have been undertaken by researchers over the last fifty years. The concept of an representative elemental area (REA) could become an integral component to the methodologies surrounding hydrologic modeling, if the concept can be sufficiently refined to render it valid and useful. With future research, one must determine exactly what an REA represents and whether not it can be universally applied.
>
> River basins do, in fact, display fractal geometric properties. Knowing that river basins are fractal and self-similar in nature, it would seem possible to use power law distributions to relate watershed characteristics at different scales. Finally, the concept of multiscaling provides a means to further investigate the relation of invariance and watershed scale.

Clearly, substantial issues associated with the relation between watershed scale and hydrologic response remain unresolved. The field remains fertile for subsequent research, provided appropriate data are available.

3.2. Other Departments of Transportation

Hydraulic engineers in other departments of transportation were contacted. Based on these interactions, it became clear that TxDOT is a leader in research on hydrologic and hydraulic technology and the development of such technology for application to highway drainage design. Other agencies use the standard methods much as presented in AASHTO (1999) and hydraulic engineers in other states frequently cited use of the rational method (with a variety of limits on watershed drainage area), regional regression equations developed specifically for their state, and TR–55 (U.S. Department of Agriculture, Natural Resources Conservation Service, 1986). Therefore, inquiries of other state departments of transportation did not yield substantial new information.

3.3. Applicable Modeling Techniques

A number of different hydrologic technologies, which used a variety of parameter estimates, were applied to the study watersheds as part of this project. After substantial reflection, it seemed logical to present results of a subset of those simulations using best estimates of model parameters. In the case of the rational method, this meant using the observed runoff coefficient. For the time of concentration, the combination of Kerby (1959) and Kirpich (1940) was used.[1] Storm events were represented by the IDF curves defined by equation 2.5 and the related parameter set in the TxDOT hydraulic design guidelines. The set of regional regressions equations currently in use by TxDOT (Asquith and Slade, 1997) were applied to study watersheds. The regression equations presented in Asquith and Thompson (2005) were applied to study watersheds. The method of L-moments was applied to the series of annual maxima for each study watershed to determine estimates of the parameters for the site-specific flood frequency distribution.

The approach outlined in the previous paragraph represents only a portion of the work done on this project. Other parameter values were applied during the course of evaluating the relation between hydrologic scale and appropriate technology. Yet, in the

[1]The appropriate use of the combined Kerby and Kirpich approach was a significant conclusion of Roussel et al. (2006).

end, it was thought that too many variations on a theme would not yield substantial insight into the choice of appropriate technology for estimating a design discharge. So, the final choices for results presented in this report are:

- Site-specific frequency curves were developed by applying the method of L-moments using either the four-parameter kappa or the three-parameter generalized logistic distribution to compute the site-specific flood frequency curve.
- Time of concentration was estimated using combination Kerby (1959) for overland flow and Kirpich (1940) for channel flow.
- IDF curves were represented by equation 2.5 with appropriate parameter values.
- The rational method was applied using observed runoff coefficients.
- The logarithm-based regression equations (2.6–2.11) and the PRESS-minimized equations (2.12–2.17) from Asquith and Thompson (2005) were applied to study watersheds.
- The regional regression equations from Asquith and Slade (1997) were applied to study watersheds.

3.3.1. Site-Specific Frequency Distribution

For the discharge record from USGS streamgages on study watersheds, L-moments were computed and used to estimate distribution parameters for the either the four-parameter kappa distribution or the three-parameter generalized logistic distribution. The intent was to use these values to represent the n-year discharges for comparison with discharges computed using other methods. The kappa distribution was used preferentially, unless the L-kurtosis was out of range for the kappa distribution. In that case, the generalized logistic distribution was used.

Asquith and Slade (1997) computed Pearson Type III distributions fit to the logarithms of annual maximum discharges. From the fitted LPIII distributions, they estimated discharges for the 2-, 5-, 10-, 25-, 50-, and 100-year events. The flood-frequency distributions of Asquith and Slade (1997) were based on the procedures presented in the Water Resources Council Bulletin 17B report (U.S. Interagency Advisory Committee on Water Data, 1982), which is another standard method referred to in the TxDOT hydraulic design guidelines. The results of Asquith and Slade (1997) are important because they represent the basis for their regional regression equations. Therefore, for watersheds common to both the database of Asquith and Slade (1997) and the study database, the flood frequency distributions of Asquith and Slade (1997) were plotted on the same coordinate axes. Such a plot is presented in figure 3.1.

The results displayed in figure 3.1 are typical. Data points representing the series of annual maxima were plotted using the Weibull plotting-position formula.

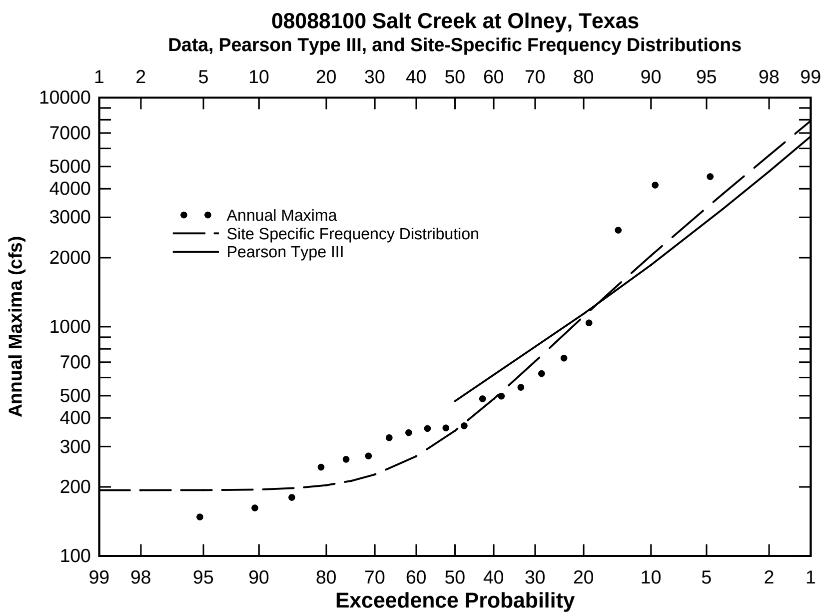

Figure 3.1: Flood frequency curve for USGS Station 08088100

For the research reported herein, the basis for comparison of results from a variety of hydrologic methods and parameter estimates is the n-year discharge from the site-specific flood-frequency distribution (either a kappa distribution or a generalized logistic distribution fitted using L-moments). Further results are based on accepting estimates of the n-year discharge from the site-specific flood-frequency distribution.

3.3.2. Rational Method

Estimates of the runoff coefficient were extracted from observations of rainfall and runoff. The method for extracting the observed runoff coefficient, as defined in the chapter "Procedures," was applied to site-specific observations of storm precipitation and watershed runoff. Illustrative results for the watershed defined by USGS streamgage 08160800 are presented in figure 3.2. For runoff depths exceeding about one watershed inch, values of the runoff coefficient between 0.20 and 0.32 are appropriate.

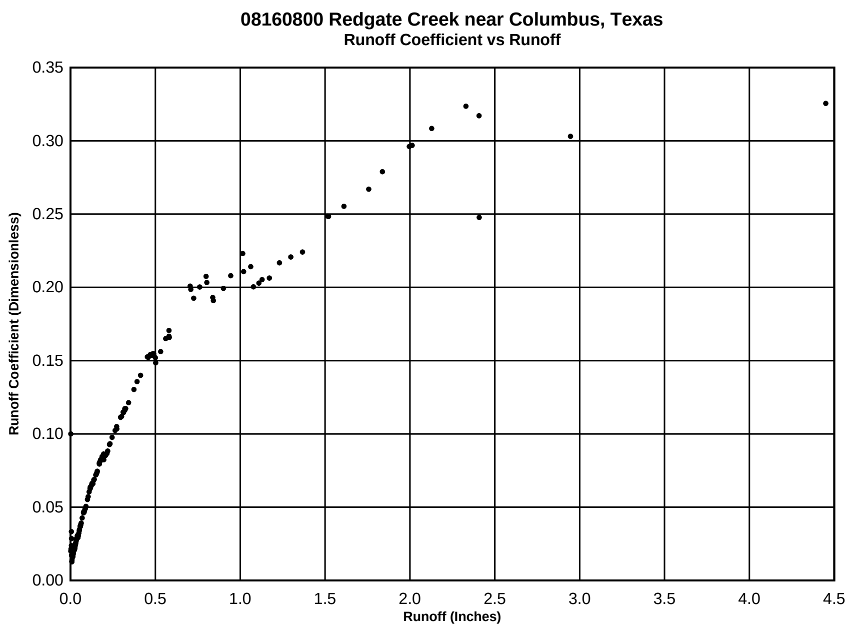

Figure 3.2: Observed runoff coefficient for USGS Station 08160800

An estimate of the observed runoff coefficient for each watershed was extracted from the data set. For some watersheds, the relation between runoff coefficient and runoff was not well defined. For some watersheds, events of large magnitude were present in the rainfall-runoff dataset. In such cases, judgment was required to arrive at an estimate of the observed runoff coefficient. Because watershed runoff of approximately 2–3 watershed inches commonly is encountered when constructing hydrologic designs, when ambiguous results were evident in the observed runoff plots, a value for the observed runoff coefficient for 2–3 watershed inches of runoff was chosen as the representative value. Plots of runoff coefficient versus runoff depth are for all study watersheds are presented in Appendix D. Estimates of observed runoff coefficient are presented in table A.1.

Values of the runoff coefficient from table A.1, average rainfall intensity for the time of concentration from equation 2.5, and watershed drainage area were used to compute n-year discharges for each watershed. Results are presented in table A.3.

The results presented in table A.3 can be analyzed in a number of ways. Figure 3.3 is a boxplot[2] of the ratio of rational method results to discharges from site-specific flood frequency distributions. A value of one represents a match between the two methods (the rational method and the site-specific frequency distribution). The median value of the ratio is about 1 for all return intervals except the 2-year, which is about 1.7. This means that the expected value from application of the rational method to the study watersheds is approximately the same as from application of a frequency analysis to site-specific discharges, with the exception of the 2-year event.

From figure 3.3, with the exception of the 2-year event and perhaps the 5-year event, the rational method produces results that are, on the average, reasonable representations of the site-specific flood-frequency relation for study watersheds.

Results from the rational method were arrayed into ranges of watershed drainage area. Three ranges were used: Drainage areas from 0–5 mi^2, 5–15 mi^2, and 15 mi^2 or more. The ratio of rational method discharge to site-specific frequency distribution discharge sorted by watershed area is presented in figure 3.4. The inter-quartile range (IQR),[3] of observed ratios is similar for each range of drainage area. There is no evidence of dependency of results on watershed drainage area.

Estimates based on the rational method could be modified based on the observations presented in figure 3.2. Because the rational method is a simple relation, the runoff coefficient could be "calibrated" to produce more reliable results, based on knowledge of the outcome such as that presented in figure 3.2. That is, given an initial estimate of the runoff depth from application of the rational method, the runoff coefficient could be modified using plots such as figure 3.2 to refine the estimate of watershed discharge though an iterative process. Given an initial estimate of runoff depth, the runoff coefficient would be selected from the plot. Then a new estimate of runoff depth would

[2]The information presented on a boxplot and interpretation of that information is discussed in Appendix B.

[3]The IQR is defined and explained in Appendix B.

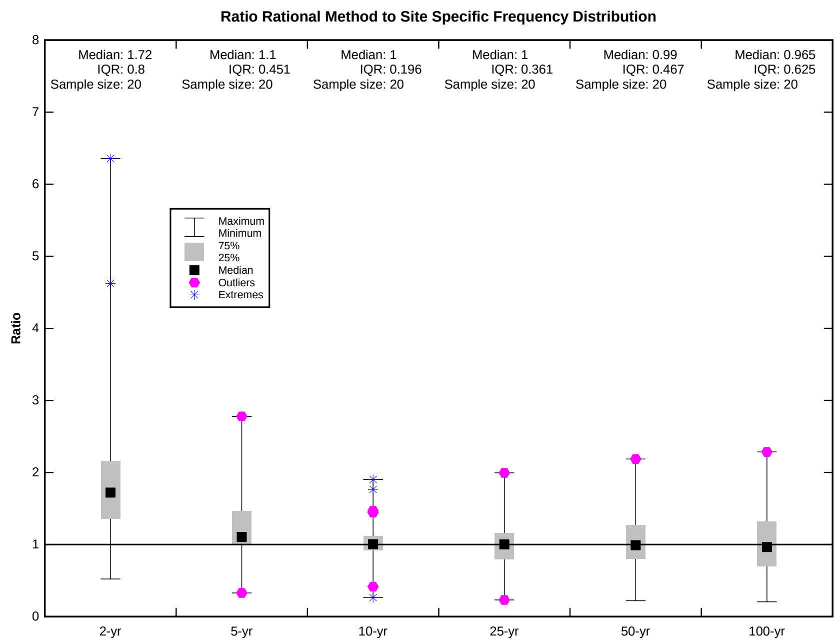

Figure 3.3: Boxplot of the ratio of rational method predicted n-year discharges to site-specific frequency distribution n-year discharges

be computed and the process repeated. Little change in the runoff coefficient would occur after a few iterations. However, such calibration would not reflect current design practice and would give a false impression of the capabilities of the model. Therefore, no adjustments beyond selecting an observed runoff coefficient from the plots of runoff coefficient versus runoff depth were made.

It is possible that use of a single estimate of the observed runoff coefficient for each watershed and for all return intervals explains elevated estimates of the 2- and 5-year discharges. That is, the observed runoff coefficient was selected based on examination of the figures presented in Appendix D for a characteristic value of runoff depth. However, the relation between runoff coefficient and runoff depth is curvilinear and use of a single value of runoff coefficient for each watershed results in estimates of runoff depth that are greater than observed for small runoff depths and less than observed for large runoff depths. Therefore, it is tempting to apply a set of adjustment factors to account for differences in the runoff depth associated with different return intervals, as is presented in the 2002 version of the hydraulic design guidelines. However, the average rainfall intensity (and hence the runoff intensity) varies not only with return interval, but also with time of concentration. Therefore, no general adjustment factor or factors may

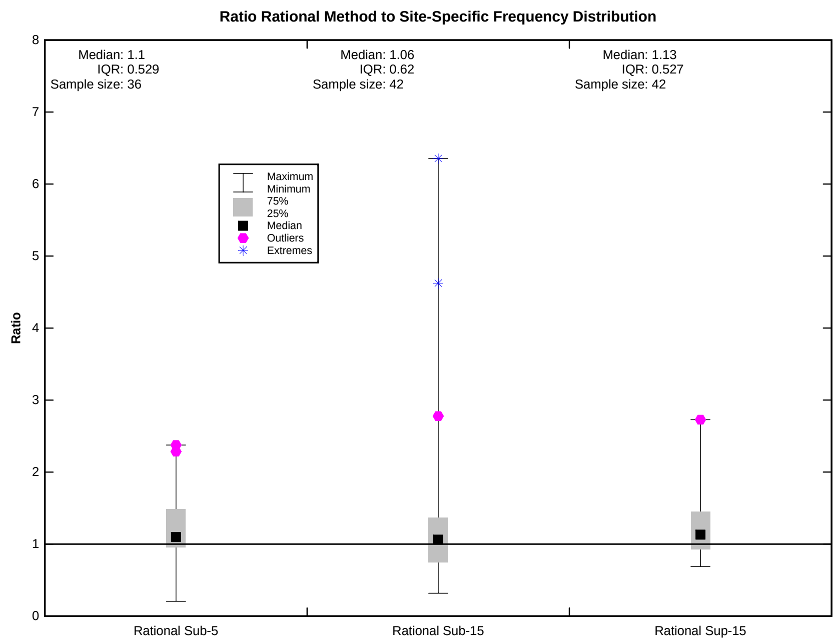

Figure 3.4: Boxplot of the ratio of rational method predicted discharges to site-specific frequency distribution discharges as a function of watershed drainage area

exist. This line of inquiry was not examined as part of the research reported herein and remains an open line of research.

3.3.3. Asquith and Slade (1997)

The regional regression equations of Asquith and Slade (1997) were applied to study watersheds. Results are presented in table A.4 in Appendix A. The ratio of regional regression equation predicted n-year discharge compared to site-specific frequency distribution discharge for study watersheds was computed. A value of one for the ratio represents equality. A boxplot of the ratios is displayed in figure 3.5.

The IQR for all return intervals intersects the reference line. Therefore, a conclusion is that the Asquith and Slade (1997) equations reasonable results. However, a clear trend is visible in the median values. There is a tendency for the equations to underestimate the 2-year peak discharge and an increasing tendency to overestimate the 25-, 50- and

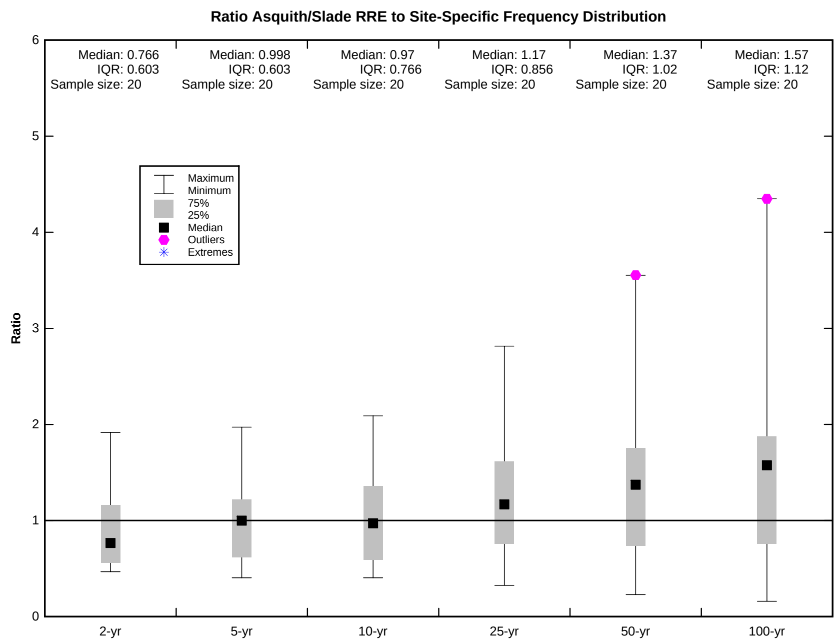

Figure 3.5: Boxplot of the ratio of regional regression equation (Asquith and Slade, 1997) predicted n-year discharges to site-specific flood-frequency distribution n-year discharges

100-year peak discharges.[4]

When examined from the perspective of watershed drainage area, however, a different story emerges. Results of comparison of site-specific flood-frequency distribution to the results from the Asquith and Slade (1997) equations were sorted by drainage area. Because of the relatively small sample size, all return intervals were lumped into each drainage-area range. Figure 3.6 is a plot of discharge ratio in relation to drainage area. The median ratio for the sub-5 mi^2 range is 1.32 and is greater than median values for larger watersheds. Furthermore, the range and IQR of values is greater for the sub-5 mi^2 range than for the others. Consistent with what was reported by Asquith and Thompson (2005), there is bias in theAsquith and Slade (1997) equations when considering very small watershed drainage areas. Specifically, the Asquith and Slade (1997) equations overestimate n-year discharge for very small watersheds.

[4]The trend observed in the results of figure 3.5 are consistent with anecdotal reports from TxDOT hydraulic engineers and with the conclusions of Asquith and Thompson (2005).

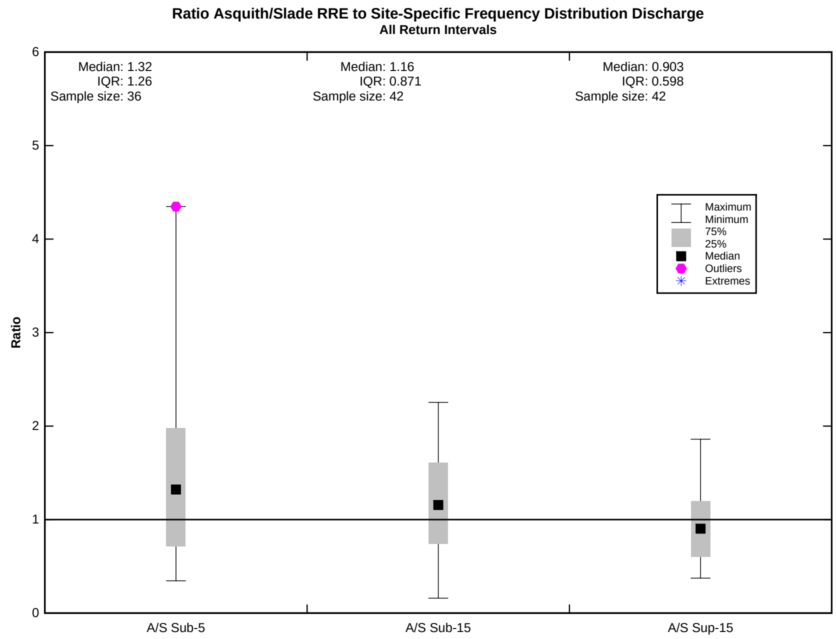

Figure 3.6: Boxplot of the ratio of regional regression equation (Asquith and Slade, 1997) predicted discharges to site-specific frequency distribution discharges as a function of drainage area

3.3.4. Asquith and Thompson (2005) Log-Transformed Equations

As a component of this study, four sets of regression equations were developed (Asquith and Thompson, 2005). The two sets of equations from Asquith and Thompson (2005) used in this research are presented as equations 2.6–2.11 and equations 2.12–2.17. These equations were applied to the study watersheds. Results are presented in table A.5 and table A.6, respectively.

Results from the log-based regression equations (2.6–2.11) were compared with those from site-specific frequency distributions and plots were prepared. Figure 3.7 is a display of the comparisons.

Results from application of log-based regression generally are acceptable. With the exception of the 2-year discharges, the reference line (representing agreement between the site-specific frequency distribution and the regression equations) intersects the IQR. Median values are near unity for the 25- and 50-year return intervals.

However, there is a trend visible in the median values of the ratio of log-based regression discharges to site-specific frequency estimates. Values from the log-based regression equations, as represented by the median of the ratio, tend to be less than those from the frequency analyses. In contrast, the tendency is reversed for larger return intervals. It appears that results from log-transformed regressor variables do not represent watershed behavior as well as other approaches might.

Because the LPIII-based n-year discharges for common streamflow-gaging stations were used by Asquith and Slade (1997) and Asquith and Thompson (2005) and the regression analysis is inherently similar (log-based), the upward trends seen in figures 3.5 and 3.7 reinforce each other.

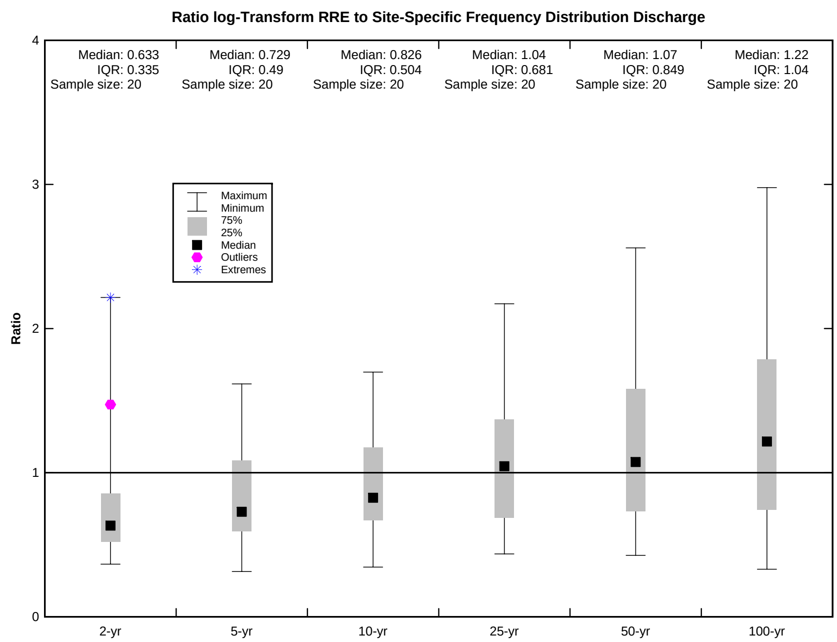

Figure 3.7: Boxplot of the ratio of log-transform regional regression equation predicted n-year discharges to site-specific frequency distribution n-year discharges

When results from the log-based regression equations are organized by drainage area into bins, figure 3.8 results. The IQR intersects the unity reference line, which indicates that the results from the log-based regression equations reasonably represent results from site-specific frequency analysis. But, the median value for small watersheds is about 0.7 and the median value for larger watersheds is about 1.3. Estimates from the log-based regression equations tend to be less than those from site-specific frequency analysis. For larger watersheds, the opposite is true.

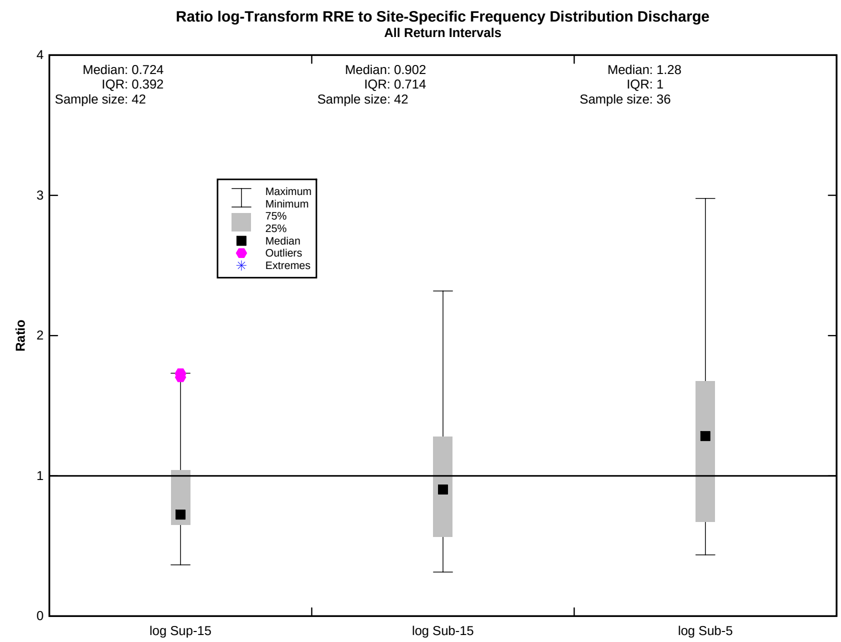

Figure 3.8: Boxplot of the ratio of log-transform regional regression equation predicted discharges to site-specific frequency distribution discharges as a function of drainage area

These results suggest that log-transformation of regressor variables may not be appropriate, at least in the context of the 20 watersheds studied as part of this research.

3.3.5. Asquith and Thompson (2005) PRESS-Minimized Equations

Results from application of the regression equations developed using PRESS minimization (equations 2.12–2.17) were compared with site-specific frequency analysis. The comparison for return intervals is presented in figure 3.9.

As with the other approaches, the 2-year events appear to be different than the other return intervals. For the 2-year events, estimates from the PRESS-minimized regression equations tend to be less than values from the site-specific frequency distributions. Although the IQR for the 5-year return interval intersects the unity reference line, the median value for 5-year events is about 0.75, indicating the predicted 5-year events tend to be less than those from site-specific frequency analysis. Estimates for the remaining return intervals are reasonable.

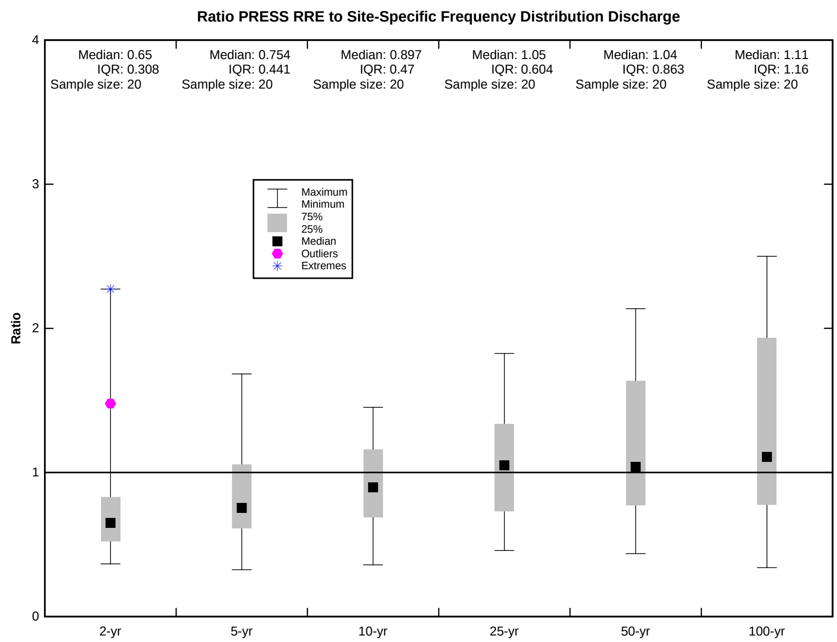

Figure 3.9: Boxplot of the ratio of PRESS-minimized regression equation predicted n-year discharges to site-specific frequency distribution n-year discharges

When comparisons of estimates from the PRESS-minimized regression equations to estimates from site-specific frequency analysis are sorted based on watershed drainage area, the graphic presented in figure 3.10 results. If dependence on watershed drainage area exists, it is not strong.

Apparently, the PRESS-minimization procedure produces regional regression equations that mimic the behavior of site-specific frequency analysis more closely than estimates from log-transformation-based regression equations of Asquith and Slade (1997), at least in the context of watershed drainage area. This is an important observation.

3.3.6. Comparison of Results

When the ratio of method-based estimates of watershed discharge to estimates from site-specific frequency distributions are lumped by method, the graphic of figure 3.11 is produced. Differences exist between the various computational methods. The range of values and the IQR is different, depending on the method. In particular, the range of results from the rational method is the greatest. Clearly, data-based methods are supe-

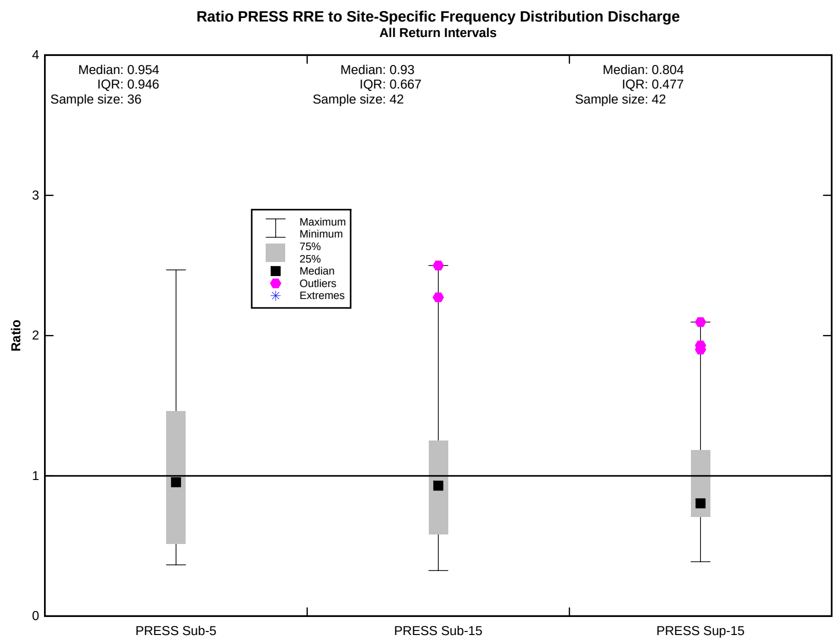

Figure 3.10: Boxplot of the ratio of PRESS-minimized regression equation predicted discharges to site-specific kappa distribution discharges as a function of drainage area

rior, in particular, the PRESS-minimized equations of Asquith and Thompson (2005) have the least range in results. However, all methods applied to the study watersheds produce consistent estimates of watershed peak discharge.

For watersheds with drainage areas less than 5 mi^2, differences between the methods applied to generate estimates of watershed discharge are small. Comparisons are presented in figure 3.12. The regression equations of Asquith and Slade (1997) and log-based regression equations Asquith and Thompson (2005) tend to produce results slightly greater than did either the rational method or the PRESS-minimized regression equations. Results from regression equations tend to be more varied that those from the rational method; no explanation is immediately apparent.

Results of comparisons of methods for watersheds with 5–15 mi^2 drainage areas between are presented in figure 3.13. The range for the rational method is substantially greater than the ranges for the other methods. This is attributable to outliers. Otherwise the results are similar for the watersheds represented in the study database.

Results from all methods for watersheds with drainage areas of 15 mi^2 and larger are

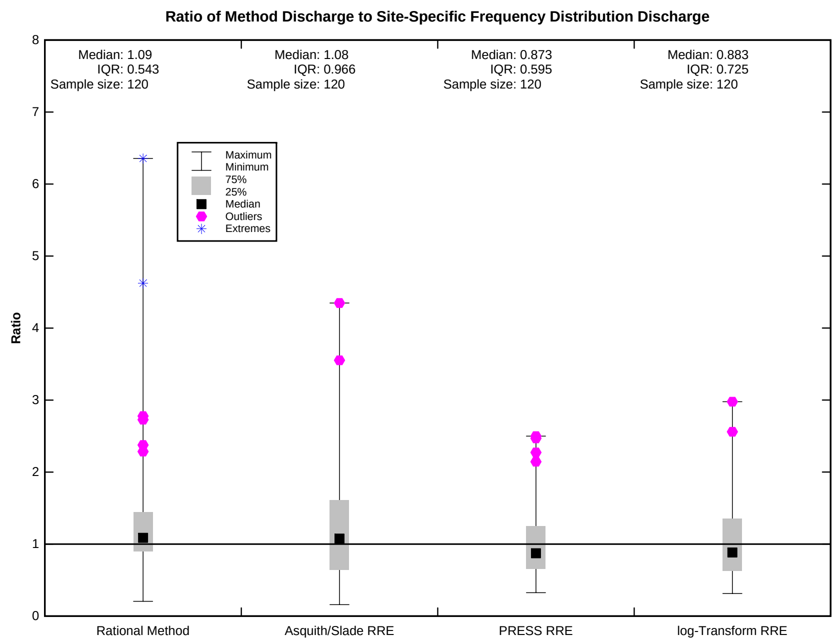

Figure 3.11: Boxplot of the ratio of discharges computed by all methods to discharges from site-specific frequency distribution

presented in figure 3.14. The IQR for all methods intersects the unity reference line. Estimates from the rational method tend to exceed those from site-specific frequency analysis for this group of watersheds, as indicated by the median value of the ratio of rational method estimate to site-specific frequency discharge estimate. Similarly, median values for both the PRESS-minimization and log-based regression equations tend to be somewhat less than those from site-specific frequency analysis.

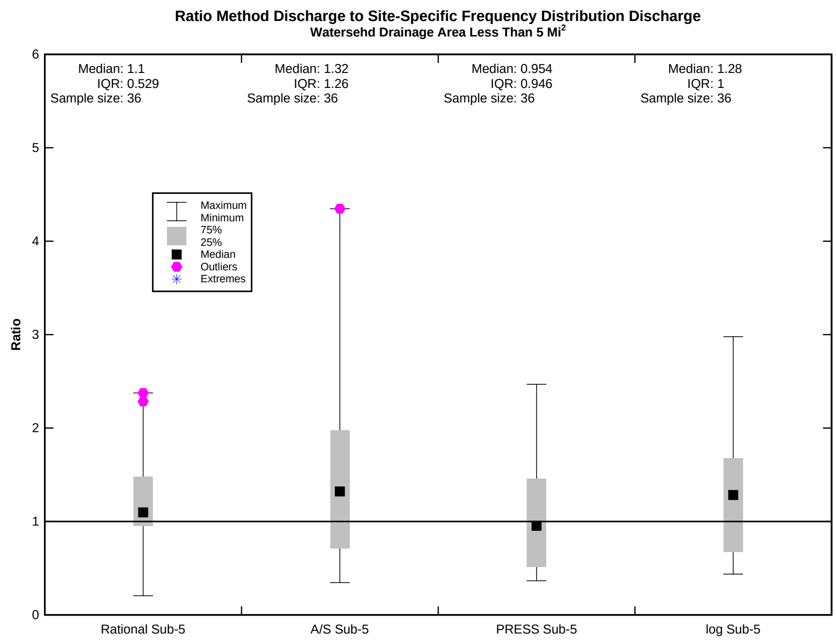

Figure 3.12: Boxplot of the ratio of discharges computed using all methods to discharges from site-specific frequency distribution for watersheds with drainage areas less than five square miles

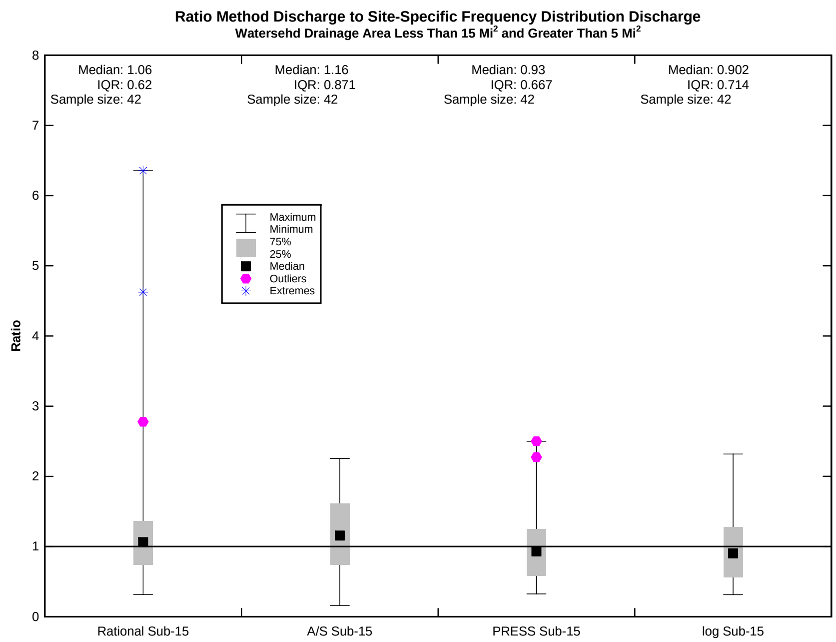

Figure 3.13: Boxplot of the ratio of discharges computed using all methods to discharges from site-specific frequency distribution for watersheds with drainage areas 5–15 square miles

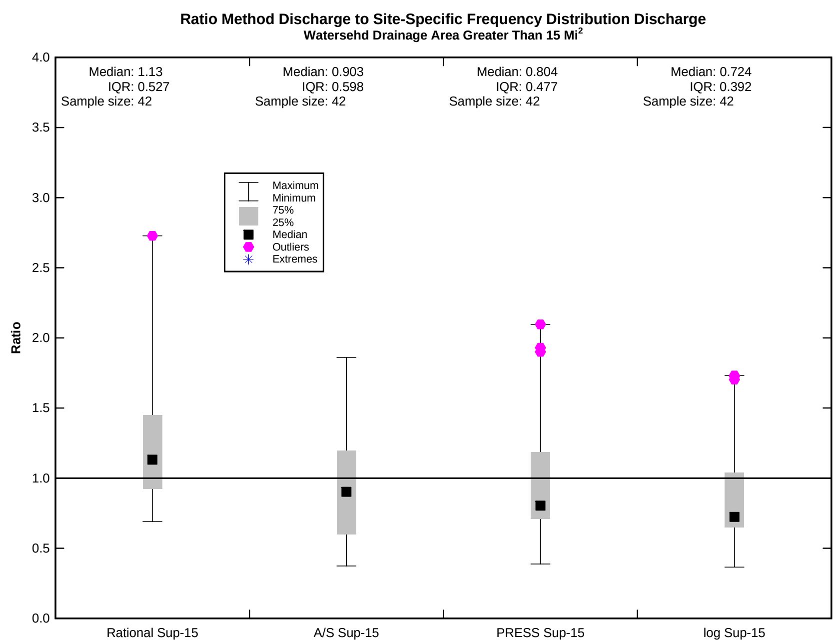

Figure 3.14: Boxplot of the ratio of discharges computed using all methods to discharges from site-specific frequency distribution for watersheds with drainage areas exceeding 15 square miles

4. Summary and Conclusions

The objective of the research reported herein was *...to examine the nature of input-response relation for Texas, to assess the viability of various approaches, and to develop or recommend methodology for use in hydrologic modeling in these areas.* A total of 20 watersheds were studied in the course of the research project. Watershed drainage areas ranged from 1.26 mi^2 to 124 mi^2. Main channel length of study watersheds ranged from 10,300 ft to 175,000 ft. Dimensionless main channel slope of study watersheds ranged from 0.003 to 0.02.

4.1. Summary

Results presented in this report include:

- Time of concentration estimated using the sum of Kerby (1959) for overland flow and (Kirpich, 1940) for channel flow
- IDF curves represented by equation 2.5 with parameters from the TxDOT hydraulic design guidelines
- The rational method applied using observed runoff coefficients
- The L-moment approach used with the four-parameter kappa and generalized logistic distributions to compute the site-specific flood frequency curve
- The PRESS-minimized equations (2.12–2.17) and the logarithm-based regression equations (2.6–2.11) from Asquith and Thompson (2005) applied to study watersheds
- The regional regression equations from Asquith and Slade (1997) applied to study watersheds

When all computational procedures are completed and the workday is done, it is the watershed's response to a stressing event that is important. Watershed response will determine the success or failure of every hydraulic design. Therefore, observations of peak stream discharge are the only basis for assessing the merit of computational procedures. Unfortunately, series of such observations are rare, especially for watersheds with very small to small drainage areas.

Watersheds were selected for study from a larger database assembled as part of a wider research program. (TxDOT projects 0–2104, 0–4193, and 0–4194 used the larger database.) One of the criterion for selection of a watershed from the larger database was a sufficient number of observations to define the flood-frequency relation. Because of the difficulty of assigning an exceedence probability to a particular event, a frequency distribution fitted to the sequence of annual maxima was chosen as the basis for further comparison.

Therefore, discharge estimates computed in the course of this research project were compared based on site-specific flood-frequency analyses. The L-moment procedure was applied to fit four-parameter kappa or three-parameter generalized logistic distributions to annual peak maxima from study watersheds, depending on whether the kappa could achieve the L-kurtosis present in the data. Estimates of n-year discharge were extracted from these distributions to serve as best estimates.

Of course, use of n-year estimates taken from site-specific flood frequency curves necessitates acceptance of this fundamental assumption of "truth." It is the author's assertion that this assumption is better than other possible assumptions because it remains tied to basic observations of watershed response.

4.2. Conclusions

The conclusions of this research report are:

1. Use of Kerby (1959) and Kirpich (1940) for estimating watershed time of concentration yields values that produce reasonable estimates of n-year discharge, at least for the rational method. The preference for Kerby (1959) and Kirpich (1940) is consistent with the conclusions drawn by Roussel et al. (2006).[1]
2. Estimates of n-year discharge using the rational method are based on best estimates for the runoff coefficient taken from a comparison of measured rainfall and runoff events. Selection of proper estimates of the runoff coefficient is critical for correct estimation of n-year discharges. Refinement of this process beyond simple selection of values from a table is appropriate.

[1] Roussel et al. (2006) did not explicitly discuss the rational method.

This observation does not constitute a recommendation for a probabilistic adjustment of runoff coefficient (as recommended in current hydraulic design guidelines). Rather, the observation that runoff coefficient varies with runoff depth (as demonstrated by the figures presented in this report) is important and suggests that a single value for runoff coefficient may not be appropriate. Additional work is necessary to further investigate runoff coefficients for use with the rational method.

3. For simple watersheds, the rational method may be applied when only an estimate of the peak discharge from the runoff hydrograph is required. Watershed drainage area does not seem to be an important consideration. However, this observation must be tempered with a caveat that only 20 watersheds were examined. Furthermore, watershed complexity was not examined as part of this research. Because the rational method is a simple procedure, application of the method to a complex watershed would be an error of judgment and may result in substantial errors in estimated design discharge. Further study through expansion of the study database also is in order.

4. Results from application of Asquith and Slade (1997) regional regression equations are mixed. For study watersheds, estimates of the 2-year discharge tended to be less than those from the site-specific frequency curve. In contrast, estimates for the 25-, 50-, and 100-year discharges tended to exceed those from the site-specific frequency curve. Furthermore, there appears to be a relation between watershed drainage area and median values from the Asquith and Slade (1997) regression equations, especially for the smallest watersheds. Differences in n-year discharge estimates could be a reflection of the possible bias with respect to drainage area present in the Asquith and Slade (1997) equations.

5. Results from Asquith and Thompson (2005) also are mixed. Similar to Asquith and Slade (1997), there appears to be some bias in the log-based regression equations of Asquith and Thompson (2005). Furthermore, median values of the ratio of log-based regression estimates to site-specific frequency estimates tended to increase with increasing return interval. The latter was less in evidence for the PRESS-minimized regression equations, but median values for the more frequent events tended to be less than those from the site-specific frequency distributions. No bias with respect to drainage area appears to be present in the PRESS-minimized regression equations. The PRESS-minimized equations produce more reliable estimates than the log-transformed equations.

6. In general, the methods used in this study did not compare well with the site-specific frequency distributions for the 2-year events, in particular, and the 5-year events to a lesser extent. It is not clear why this should occur. It is possible that

the observed runoff coefficient for the rational method was larger than it should be for the most frequent events. That would explain why the 2-year discharges from the rational method generally exceeded those from site-specific frequency curves. However, additional analysis is necessary to determine the exact cause of this observation. A second possibility is that the processes which produce more frequent flood events differ from those of rare occurrence. For example, watershed characteristics might more strongly drive discharge for larger n-year events and are not as important for discharge from smaller n-year events. This is an item for further research.

7. Whereas watershed drainage area is an important factor in executing hydrologic computations, the work reported in this report suggests that drainage area is not useful for selecting a method for estimating a design discharge. Specifically, watershed drainage area alone is insufficient for accepting or rejecting application of the rational method for producing design estimates. Additional work is required to determine a mechanic for discriminating between methods used to produce design discharge estimates.

4.3. Further Work

During the course of the research, additional lines of inquiry were revealed. In particular, the following lines of investigation are recommended:

1. Very small watersheds (sub-square mile drainage area) should be studied. Watersheds with a drainage area less than about one square mile were not represented in the study database. Data for small watersheds are relatively unavailable in Texas (or at least, could not be identified). Availability of data remains a substantial detriment to detailed study of hydrologic processes for Texas watersheds.

2. Use of Asquith and Roussel (2004) for estimation of IDF relations and storm depths should be investigated. The work of Asquith and Roussel (2004) was under development during the course of this research project so it was not used. The results of Asquith and Roussel (2004) represent a substantial extension of the databases available for the TP-40 (U.S. Weather Bureau, 1963) and HYDRO-35 (National Oceanic and Atmospheric Administration, 1977) analyses.

3. It is critical to perform a complete evaluation of recently developed hydrologic methods and how the components developed by various researchers interact in the production of design discharge estimates.

4. Given that the degree of conservatism of the hydrologic methods presented in recent research is less than that of the technology in current use, parameter selection becomes more important if errors in design estimates are to be avoided.[2] Furthermore, training and evaluation of the role of hydrologic safety factors becomes more important as the ability to refine estimates of n-year discharges are improved. It is critical that this point be investigated and publicized in the public and private sectors.

5. Because watershed drainage area does not appear to be an applicable factor for discriminating between appropriate hydrologic technology, other methods for discrimination between procedures for making design-discharge estimates should be investigated. An alternative might be the geomorphologic watershed order scheme developed by Horton (1945) and Strahler (1952). Additional work on the concepts of representative elemental area and multiscaling should be undertaken.

6. Substantial work remains to develop means for refining estimates of loss-rate model parameters, in particular the rational method runoff coefficient, that are better than simple table estimates. Selection of proper estimates of the runoff coefficient is critical for correct estimation of n-year discharges.

7. Evaluation of the hydrograph method and associated process models should be undertaken.

8. Urban hydrology has not been examined. Yet, a substantial number of highway projects are undertaken in urban or urbanizing regions. At some point in time, effort should be directed at understanding the impact of urbanization on the estimation of design events for highway drainage works.

9. Model-predicted 2-year events, and to a lesser extent, the 5- and 10-year events, regardless of the hydrologic method used, exhibited behavior that differed from the underlying data as represented by the site-specific frequency distribution. Further study of the more frequent events is needed to determine if this observation is real or an artifact of the computational methods used in this research. This is particularly important because minor structures are often designed for the 2-, 5-, or 10-year return interval.

[2] Errors can be over-estimates as well as under-estimates.

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A. Tables of Computational Results

Table A.1: Observed runoff coefficient for selected Texas watersheds

TTU Watershed ID	USGS Gage ID	Observed C	Table C
1003	08088100	0.45	0.40
1004	08093400	0.44	0.37
1007	08160800	0.27	0.41
1008	08167600	0.26	0.68
1108	08156800	0.70	0.68
1117	08158700	0.30	0.40
1122	08158840	0.40	0.46
1407	08178640	0.35	0.46
1412	08181400	0.38	0.48
1603	08098300	0.67	0.38
2008	08096800	0.34	0.36
2302	08137000	0.35	0.36
2501	08182400	0.28	0.34
2601	08187000	0.16	0.38
2612	08187900	0.41	0.39
2701	08050200	0.58	0.41
2802	08058000	0.65	0.36
2903	08052700	0.54	0.41
3002	08042700	0.32	0.44
3101	08063200	0.53	0.36

Table A.2: Results of site-specific frequency analysis for study watersheds

TTU Watershed ID	USGS Gage ID	Distribution	Discharge (cfs)					
			2-year	5-year	10-year	25-year	50-year	100-year
1003	08088100	Kappa	352	1110	2043	3792	5594	7921
1004	08093400	Kappa	1291	2374	2808	3088	3185	3235
1007	08160800	Kappa	1820	3195	3924	4600	4956	5217
1008	08167600	Kappa	1637	5484	7651	9426	10202	10677
1108	08156800	Kapp	3031	5327	7306	10475	13425	16976
1117	08158700	Kappa	3435	8966	11414	13074	13676	13994
1122	08158840	Kappa	1232	3247	4503	5742	6418	6923
1407	08178640	Kappa	379	591	725	886	998	1102
1412	08181400	Kappa	1717	3321	4711	6951	9048	11587
1603	08098300	Kappa	3010	5369	6734	8200	9115	9895
2008	08096800	Kappa	722	1145	1411	1741	1983	2222
2302	08137000	Kappa	374	794	1111	1531	1849	2166
2501	08182400	GLO	550	1178	1865	3275	4947	7441
2601	08187000	Kappa	918	1902	2738	4031	5189	6533
2612	08187900	Kappa	445	1593	3108	6629	11236	18664
2701	08050200	Kappa	295	480	610	790	936	1092
2802	08058000	Kappa	563	1042	1291	1523	1646	1738
2903	08052700	GLO	3445	7067	10487	16669	23207	32039
3002	08042700	Kappa	1722	3105	4222	5917	7420	9157
3101	08063200	Kappa	1479	3353	4200	4813	5055	5192

Table A.3: Results from application of the rational method to study watersheds

TTU Watershed ID	USGS Gage ID	Discharge (cfs)					
		2-year	5-year	10-year	25-year	50-year	100-year
1003	08088100	2236	3085	3603	4677	5637	6754
1004	08093400	2019	2682	3131	4111	5087	5851
1007	08160800	2320	3012	3631	4700	5790	6618
1008	08167600	2059	2727	3174	4070	4950	5812
1108	08156800	3867	5195	5959	7782	9644	11326
1117	08158700	7091	9513	11261	14853	18869	21557
1122	08158840	2246	2989	3419	4444	5465	6428
1407	08178640	899	1196	1380	1767	2181	2519
1412	08181400	2779	3745	4342	5615	7018	8057
1603	08098300	4373	5748	6914	8898	11228	12858
2008	08096800	953	1238	1472	1897	2355	2695
2302	08137000	688	946	1115	1503	1762	2084
2501	08182400	1207	1620	1876	2418	3011	3463
2601	08187000	479	624	720	931	1143	1342
2612	08187900	2058	2695	3151	4075	5045	5927
2701	08050200	362	492	580	732	909	1057
2802	08058000	799	1028	1228	1566	1920	2205
2903	08052700	7388	10184	11906	15108	18802	22100
3002	08042700	2358	3276	3748	4948	6012	7249
3101	08063200	4034	5168	6173	7907	9702	11419

Table A.4: Results from application of the Asquith and Slade (1997) regional equations of to study watersheds

TTU Watershed	USGS Gage	Discharge (cfs)					
ID	ID	2-year	5-year	10-year	25-year	50-year	100-year
1003	08088100	508	1168	1843	3018	4185	5616
1004	08093400	1464	2800	3587	4962	6085	7292
1007	08160800	1245	1583	2110	2746	3204	3642
1008	08167600	823	2517	4405	7805	13856	19571
1108	08156800	3330	5830	7830	10800	13200	15900
1117	08158700	4158	8594	12051	16717	20155	23639
1122	08158840	667	2022	3518	6193	9607	13279
1407	08178640	292	792	1301	2166	3545	4793
1412	08181400	1082	3160	5447	9546	15440	21549
1603	08098300	1406	2799	3887	5352	6493	7675
2008	08096800	825	1647	2272	3183	3938	4754
2302	08137000	232	1324	2321	4310	4325	5959
2501	08182400	598	1784	3081	5384	6514	8695
2601	08187000	520	768	1106	1558	1906	2260
2612	08187900	853	1172	1605	2155	2567	2975
2701	08050200	224	498	712	1029	1298	1592
2802	08058000	305	672	953	1366	1715	2096
2903	08052700	3957	7597	6295	8522	10221	11975
3002	08042700	941	1858	3007	5050	7105	9669
3101	08063200	1845	3618	4362	6045	7414	8883

Table A.5: Results from application of the Asquith and Thompson (2005) logarithm-based equations to study watersheds

TTU Watershed ID	USGS Gage ID	Discharge (cfs) 2-year	5-year	10-year	25-year	50-year	100-year
1003	08088100	780	1794	2746	4285	5686	7312
1004	08093400	800	1840	2816	4394	5831	7499
1007	08160800	950	2182	3337	5206	6909	8885
1008	08167600	748	1723	2637	4115	5461	7022
1108	08156800	796	1833	2804	4376	5807	7468
1117	08158700	2623	5970	9113	14196	18835	24232
1122	08158840	648	1493	2286	3568	4736	6089
1407	08178640	347	803	1232	1924	2554	3283
1409	08178690	109	255	392	614	815	1047
1603	08098300	1100	2523	3859	6019	7987	10272
2008	08096800	524	1211	1855	2896	3844	4942
2302	08137000	447	1035	1585	2476	3286	4225
2501	08182400	596	1375	2105	3286	4362	5608
2601	08187000	403	934	1431	2236	2967	3815
2612	08187900	655	1511	2313	3610	4791	6161
2701	08050200	191	445	683	1067	1416	1820
2802	08058000	246	572	877	1371	1820	2340
2903	08052700	2031	4633	7075	11027	14630	18820
3002	08042700	1065	2444	3737	5830	7736	9949
3101	08063200	958	2201	3367	5252	6970	8963

Table A.6: Results from application of the Asquith and Thompson (2005) PRESS-minimized equations to study watersheds

TTU Watershed	USGS Gage	Discharge (cfs)					
ID	ID	2-year	5-year	10-year	25-year	50-year	100-year
1003	08088100	800	1869	2883	4534	6063	7840
1004	08093400	823	1925	2971	4674	6252	8086
1007	08160800	995	2340	3621	5713	7654	9916
1008	08167600	764	1783	2748	4318	5771	7459
1108	08156800	819	s1916	2956	4651	6221	8046
1117	08158700	2884	6824	10603	16804	22581	29337
1122	08158840	649	1506	2314	3625	4836	6240
1407	08178640	313	699	1053	1617	2131	2721
1412	08181400	917	2153	3329	5246	7024	9093
1603	08098300	1167	2755	4272	6752	9058	11747
2008	08096800	510	1170	1788	2786	3703	4764
2302	08137000	423	963	1465	2271	3010	3863
2501	08182400	590	1364	2092	3271	4358	5617
2601	08187000	375	847	1284	1983	2623	3359
2612	08187900	658	1527	2347	3678	4907	6333
2701	08050200	150	316	463	691	894	1124
2802	08058000	206	446	663	1002	1308	1656
2903	08052700	2226	5279	8209	13018	17500	22740
3002	08042700	1127	2658	4120	6510	8731	11320
3101	08063200	1005	2363	3658	5771	7733	10019

B. Statistical Methods

B.1. L-Moments

If $X_1, X_2, \ldots, X_n$ are n independent and identically distributed continuous random variables having a cumulative distribution function $F(x)$, then the ordered values $X_{1:n} \leq X_{2:n} \leq \ldots \leq X_{n:n}$ represent the order statistics of the random sample $X_1, X_2, \ldots, X_n$. In the sequence, the element $X_{j:n}$ is an observation from the population, n represents the sample size, and j represents the order number when the sample is sorted in ascending order.

Let $x(F)$ be the inverse cumulative distribution function of X. Then the expected value of the jth order statistic can be computed by

$$E[X_{j:n}] = \frac{n!}{(j-1)!(n-j)!} \int_0^1 x(F)^{j-1} (1-F)^{n-j} \, dF, \tag{B.1}$$

The L-moments are linear combinations of the expectations of order statistics,

$$\lambda_{r+1} = \frac{1}{r+1} \sum_{i=0}^{r} (-1)^k \left(\begin{array}{c} r \\ k \end{array} \right) E\left[X_{(r-k+1):(r+1)}\right]. \tag{B.2}$$

From equation B.2, the first four L-moments are:

$$\lambda_1 = E[X_{1:1}], \tag{B.3}$$

$$\lambda_2 = \frac{1}{2}\{E[X_{2:2}] - E[X_{1:2}]\}, \tag{B.4}$$

$$\lambda_3 = \frac{1}{3}\{E[X_{3:3}] - 2E[X_{2:3}] + E[X_{3:3}]\}, \text{and} \tag{B.5}$$

$$\lambda_4 = \frac{1}{4}\{E[X_{4:4}] - 3E[X_{3:4}] + 3E[X_{2:4}] - E[X_{1:4}]\}. \tag{B.6}$$

The L-moments are used separately or in combination to produce parameters for use in probability distributions. The L-moments have names, $\lambda_1 \equiv$ mean, $\lambda_2 \equiv$ L-scale,

$$\tau = \frac{\lambda_2}{\lambda_1}, \tag{B.7}$$

$$\tau_3 = \frac{\lambda_3}{\lambda_2}, \text{ and} \tag{B.8}$$

$$\tau_4 = \frac{\lambda_4}{\lambda_2}. \tag{B.9}$$

In equations B.7–B.9, $\tau_2 \equiv$ coefficient of L-variation (L-CV), $\tau_3 \equiv$ L-skew, and $\tau_4 \equiv$ L-kurtosis.

If $x(F)$ exists, then the theoretical L-moments of a distribution can be computed,

$$\lambda_{r+1} = \int_0^1 x(F)\{\sum_{k=0}^{r}(-1)^{r-k} \begin{pmatrix} r \\ k \end{pmatrix} \begin{pmatrix} r+k \\ k \end{pmatrix} F^k\}dF. \tag{B.10}$$

To fit a distribution to a sample, the sample L-moments are equated to the theoretical L-moments from equation B.10. Values from the distribution can then be computed.

In addition, the L-moment ratios, τ_2, τ_3, and τ_4, can be used to discriminate or select appropriate distributions for application to a particular data set. In particular, the GLO will fit a data set with a wider range of L-moment ratios than the Kappa. These ideas are expanded in Hosking and Wallis (1997). In the case of the research reported herein, if the Kappa distribution was not appropriate (based on the L-moment ratios), then the GLO was fit to the site-specific data set.

B.2. Generalized Logistic Distribution

The generalized logistic distribution is a three-parameter distribution, given by

$$Q(T) = \begin{cases} \xi + \frac{\alpha}{k}\left[1 - \left(\frac{1-F}{F}\right)^k\right], & k \neq 0 \\ \xi - \alpha \log(\frac{1-F}{F}), & k = 0 \end{cases} \tag{B.11}$$

where:

$Q(T)$ = T-year discharge associated the non-exceedence probability F,

$F = 1 - \dfrac{1}{T}$, and

ξ, α, κ = location, scale, and shape parameters for the generalized logistic distribution.

B.3. Kappa Distribution

The four-parameter kappa distribution offers an additional parameter over the Pearson Type III distribution and other lesser known three-parameter distributions, although at the expense of requiring an additional L-moment to be computed. However, the kappa distribution has been suggested for flood-flow frequency analysis in Texas (Asquith, 2001).

The four-parameter kappa distribution is given by

$$Q(T) = \xi + \frac{\alpha}{\kappa}\left\{1 - \left[\frac{1 - F^h}{h}\right]^{\kappa}\right\}, \tag{B.12}$$

where:

$Q(T)$ = T-year discharge associated the non-exceedence probability F,

$F = 1 - \frac{1}{T}$, and

ξ, α, κ, h = location, scale, shape 1, and shape 2 parameters for the kappa distribution.

B.4. Pearson Type III Distribution

U.S. Interagency Advisory Committee on Water Data (1982) presents a particular approach to fitting the Pearson Type III distribution to the logarithms of the annual maxima runoff series. The result is termed the Log-Pearson Type III (LPIII) distribution. The procedure relies on the method of moments to generate parameter estimates for the LPIII. As a result, the parameters are sensitive to extreme events and mechanics are required to mitigate the effect of outliers on the fitted distribution. The Bulletin 17B approach (U.S. Interagency Advisory Committee on Water Data, 1982) is constrained to a single distribution, the LPIII, unlike the more general L-moment method.

B.5. PRESS Statistic

The PRESS residual is defined as

$$e_{(i)} = y_i - y_i', \tag{B.13}$$

where:

$$
\begin{aligned}
e_{(i)} &= \text{PRESS residual,} \\
y_i &= i\text{th observed streamflow value, and} \\
y_i' &= \text{predicted value based on remaining } n-1 \text{ sample points.}
\end{aligned}
$$

From the PRESS residual, the PRESS statistic, then, is defined by

$$PRESS = \sum_{i=1}^{n} w_i e_{(i)}^2, \tag{B.14}$$

where w_i is the ith regression weight factor.

The PRESS statistic can be used as an objective function to fit equations of a variety of forms. One of those forms is $y = 10^{a-\lambda A^b}$. This is in contrast to linear regression, which is applied to equations of the form $y = aX_1^b X_2^c \ldots$ and cannot be used with the previous form. Therefore the PRESS statistic (and other objective functions) provides access to a wider range of regression equations than linear regression. This is a substantial gain for development of regression models for hydrologic approximation.

B.6. Reading a Boxplot

The median and inter-quartile range (IQR) represent location (central tendency) and scale parameters resistant to unusual values. The median is determined by examination of the sorted dataset and is the 50th percentile value. The IQR also is determined from the sorted dataset, and represents the range of values from the lower 25th percentile to the upper 25th percentile, or the range from the 25th percentile to the 75th percentile.

Because the values of the median and IQR are set by counting or position within the dataset, their values are not heavily influenced by the presence of very large or very small values in the dataset. This is not the case for the arithmetic mean, the variance, or higher moments. Therefore, the median and IQR are often considered as useful measures of central tendency and scale than the mean and variance (or standard deviation).

A boxplot is a graphical aid used to display information about the distribution of a dataset. Four components of a distribution are displayed:

1. The center of the data as represented by the median,
2. The variation of the data as represented by the inter-quartile range (IQR). The lower quartile is the lower one-fourth of the data; the upper quartile is the upper

one-fourth of the data. So, the IQR is the data between the lower 25-percent quartile and the upper 25-percent quartile,

3. The skewness is represented by the asymmetry of the IQR, and

4. The presence of unusual values, outliers and extreme values.

C. Flood-Flow Frequency Curves

As described in the body of this report, the L-moment method was applied to observed annual maximum discharges from study watersheds. The resulting L-moments were used to approximate distribution parameters. The fitted distributions were used to estimate n-year flood discharges. Those values, along with the observed annual maxima are displayed on figures C.1–C.20.

Some of watersheds used in this study were also part of the data set presented in Asquith and Slade (1997). Asquith and Slade (1997) used the Bulletin 17B (U.S. Interagency Advisory Committee on Water Data, 1982) method to fit a Pearson Type III distribution to the logarithms of annual maxima (LPIII). When LPIII results from Asquith and Slade (1997) were available, they were superimposed on the figures presented in this appendix.

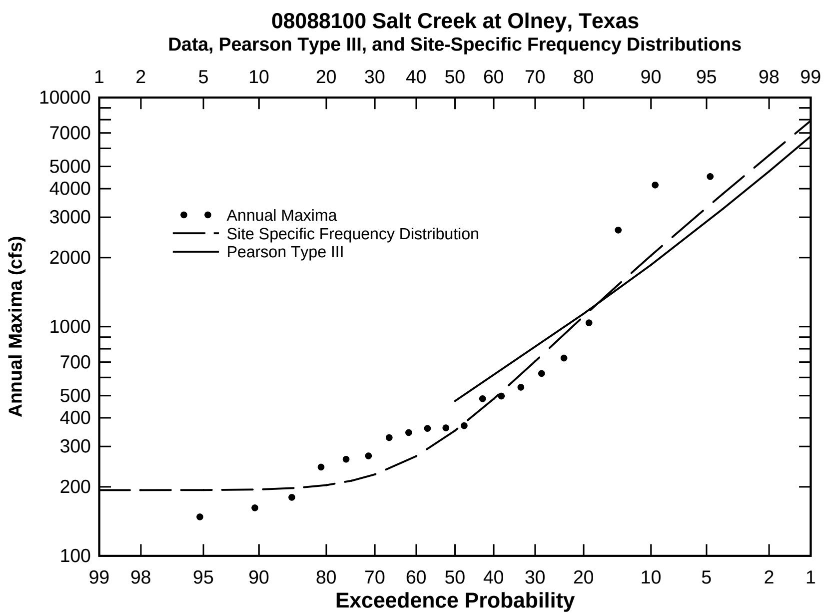

Figure C.1: Flood frequency curve for USGS station 08088100

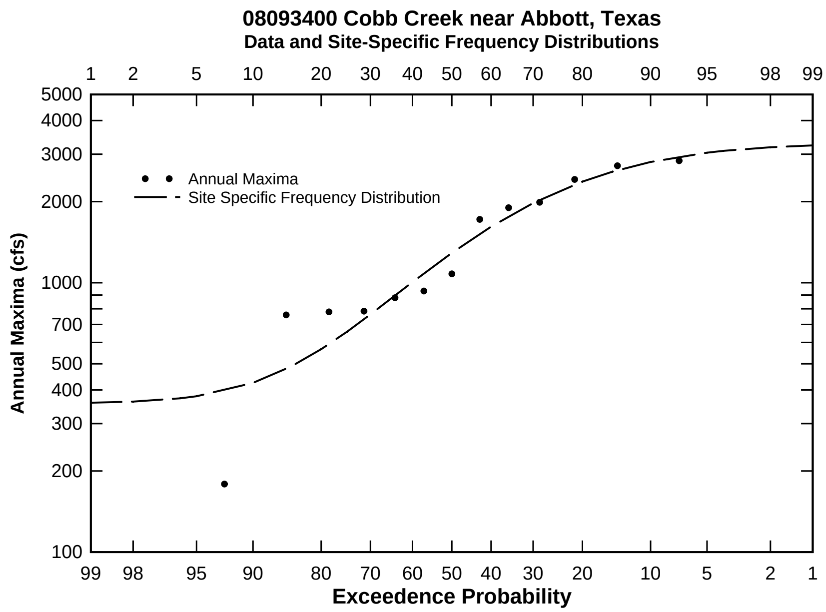

Figure C.2: Flood frequency curve for USGS station 08093400

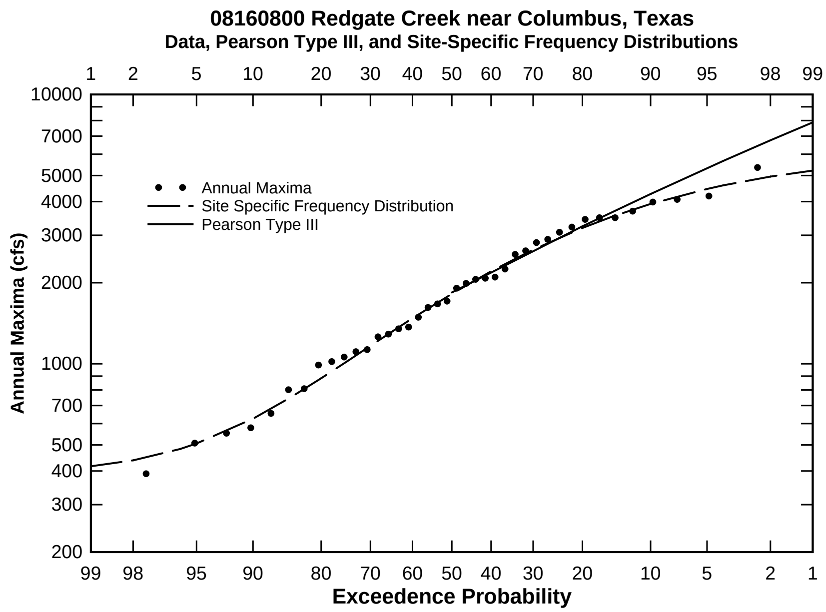

Figure C.3: Flood frequency curve for USGS station 08093400

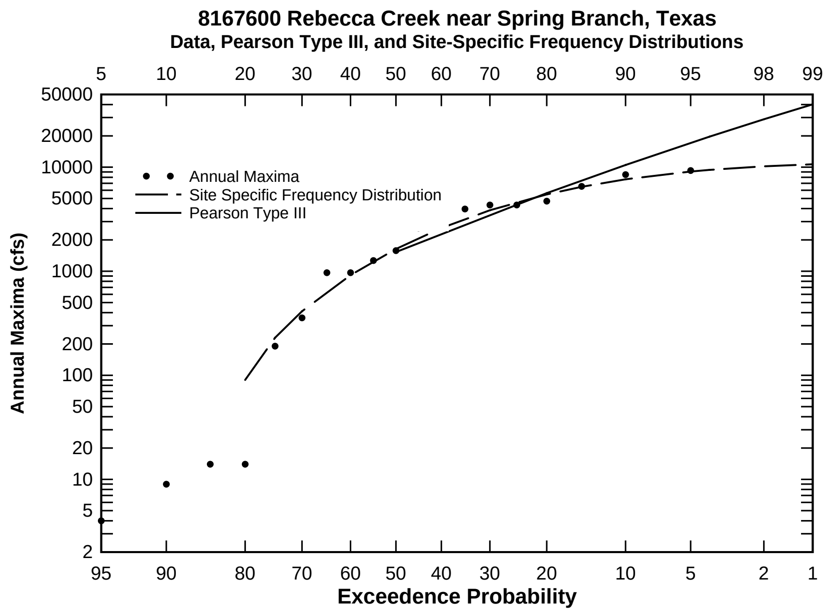

Figure C.4: Flood frequency curve for USGS station 08167600

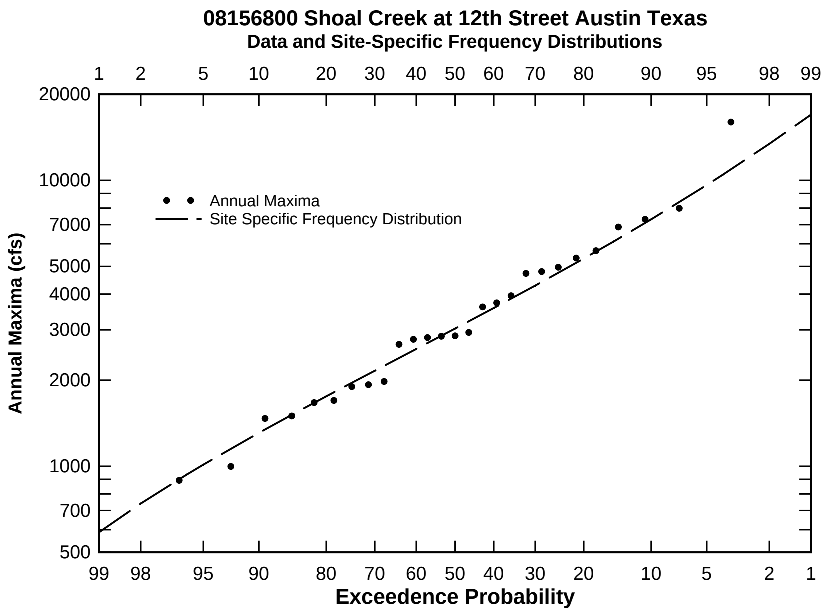

Figure C.5: Flood frequency curve for USGS station 08156800

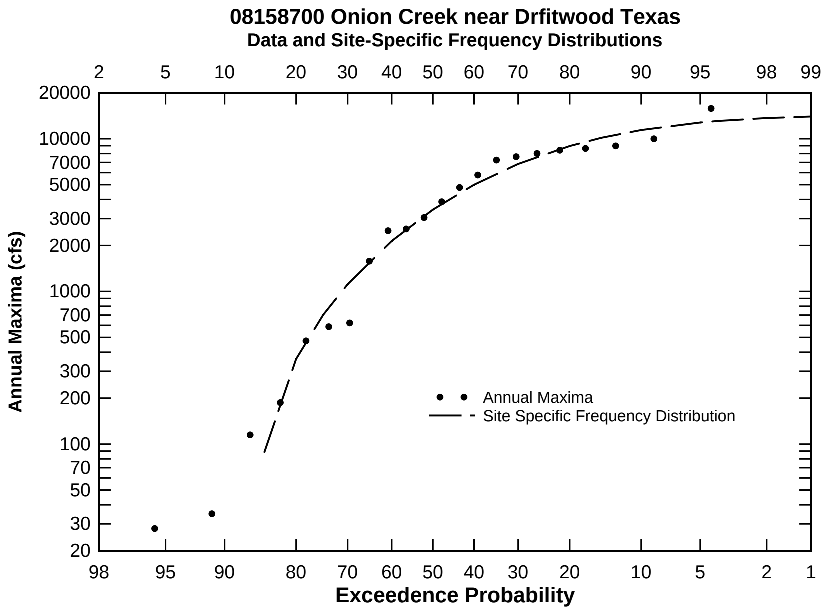

Figure C.6: Flood frequency curve for USGS station 08158700

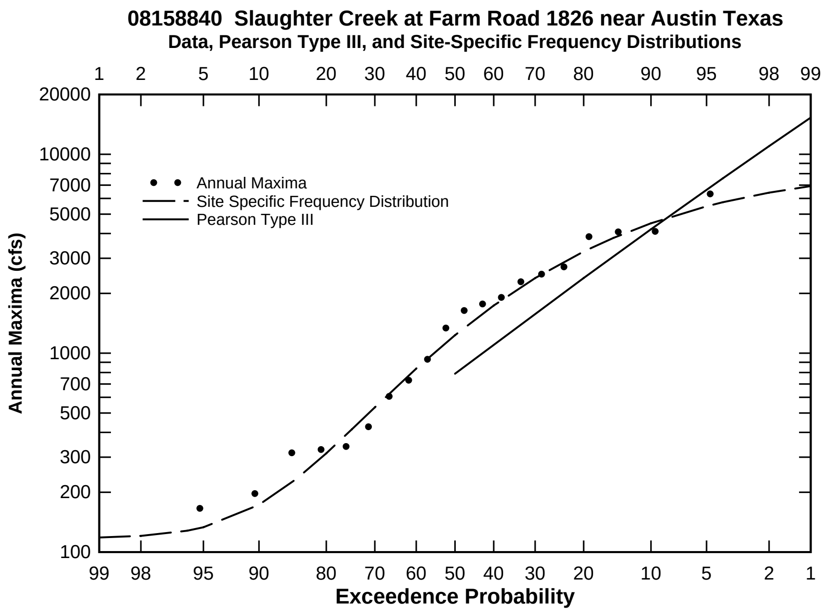

Figure C.7: Flood frequency curve for USGS station 08158840

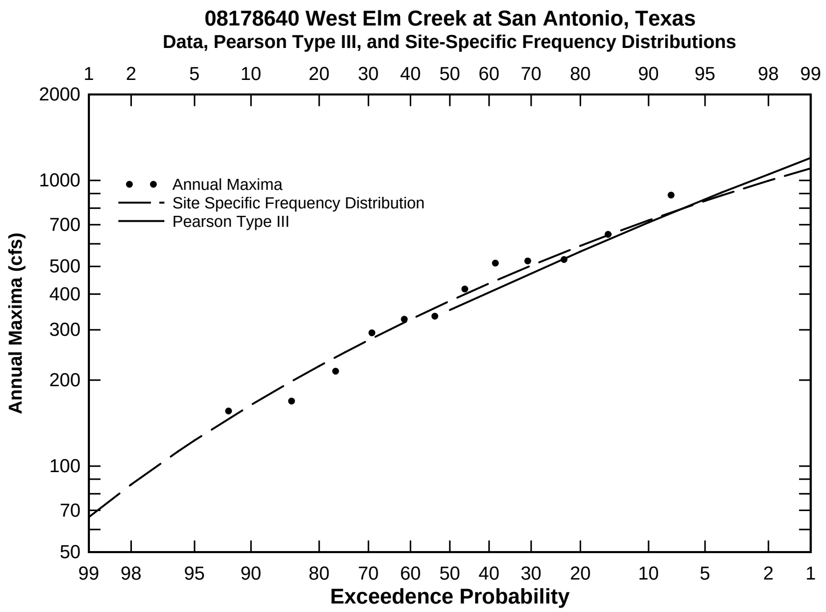

Figure C.8: Flood frequency curve for USGS station 08178640

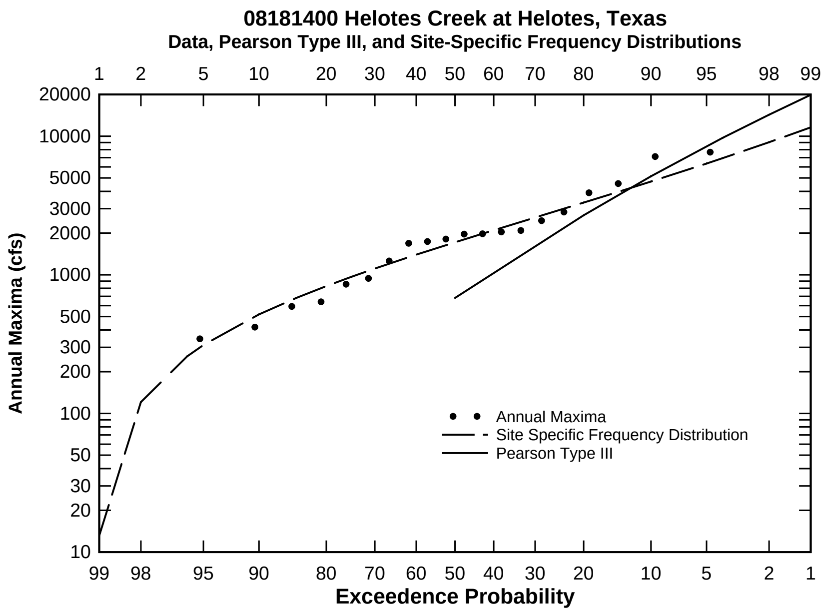

Figure C.9: Flood frequency curve for USGS station 08181400

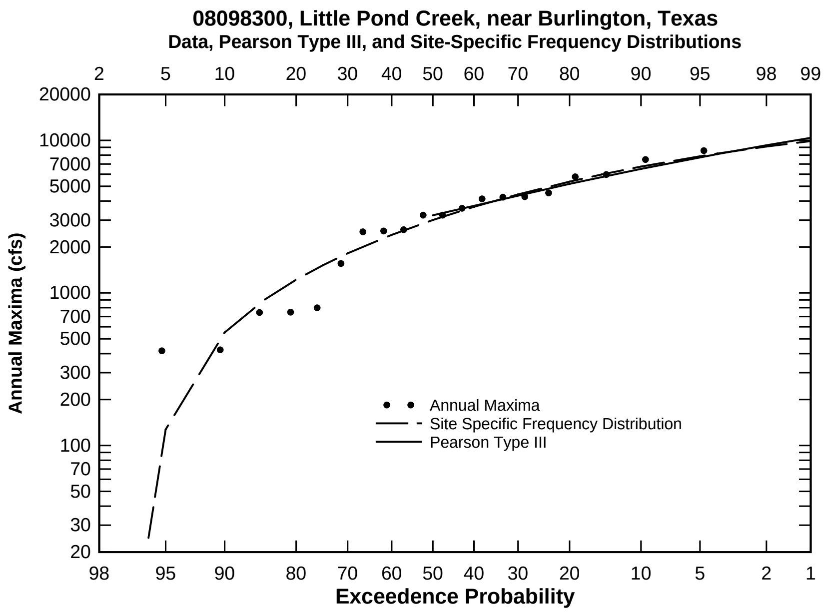

Figure C.10: Flood frequency curve for USGS station 08098300

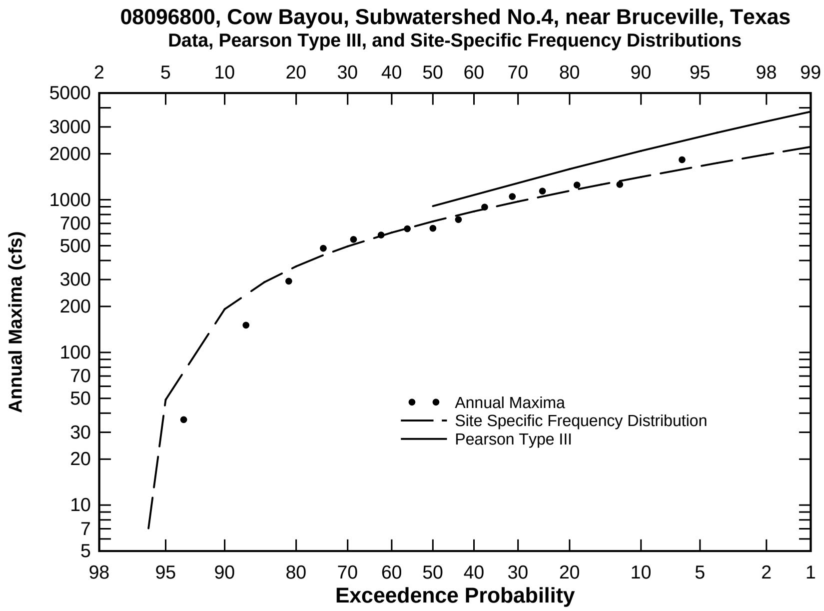

Figure C.11: Flood frequency curve for USGS station 08096800

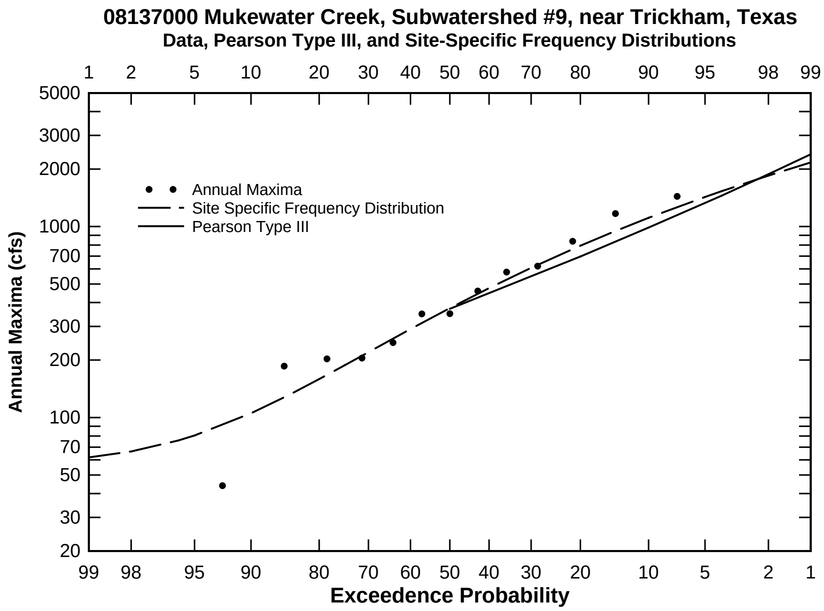

Figure C.12: Flood frequency curve for USGS station 08137000

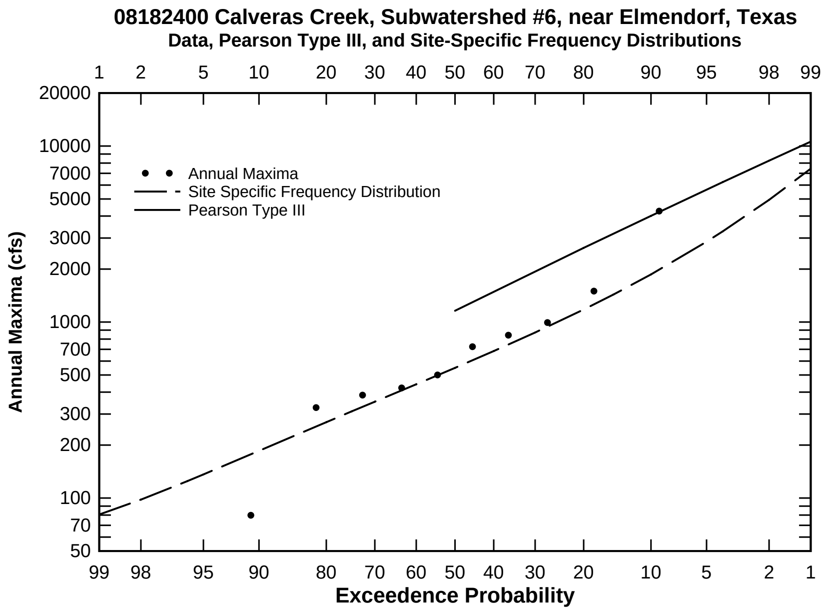

Figure C.13: Flood frequency curve for USGS station 08182400

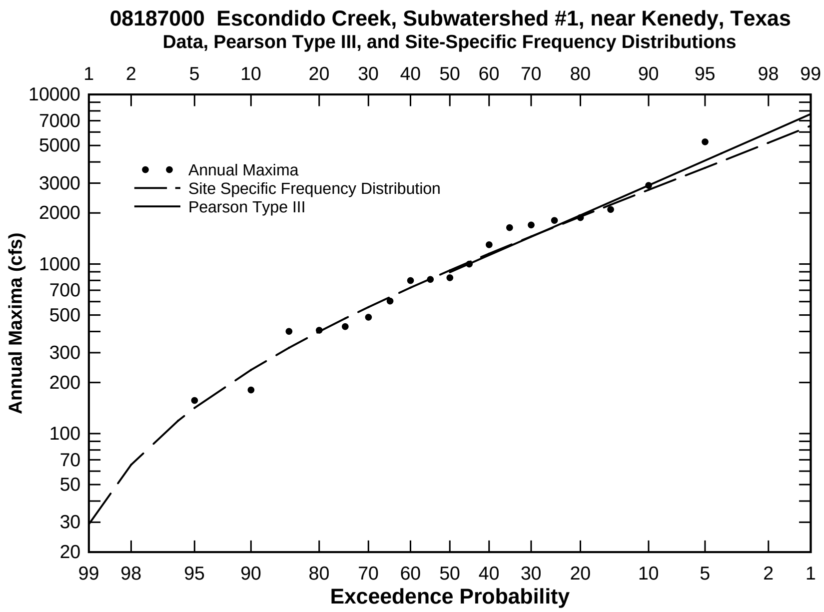

Figure C.14: Flood frequency curve for USGS station 08187000

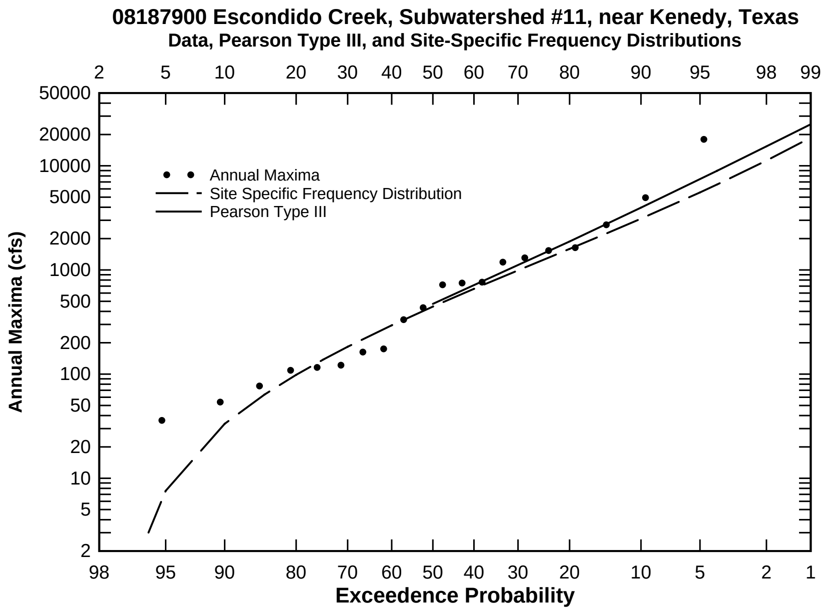

Figure C.15: Flood frequency curve for USGS station 08187900

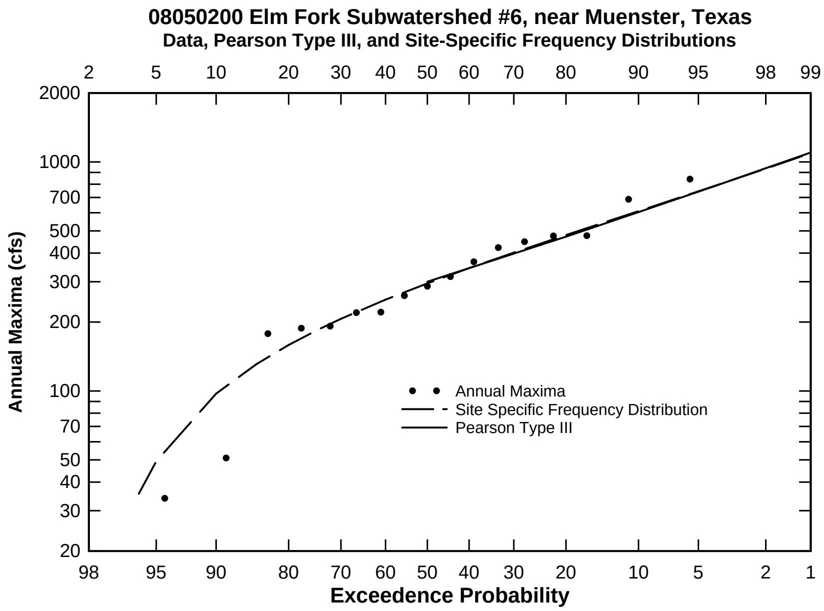

Figure C.16: Flood frequency curve for USGS station 08050200

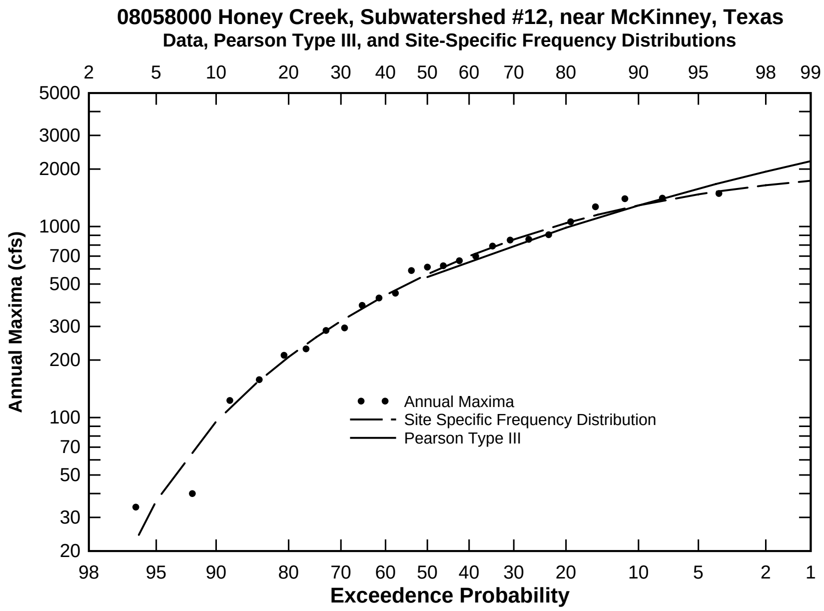

Figure C.17: Flood frequency curve for USGS station 08058000

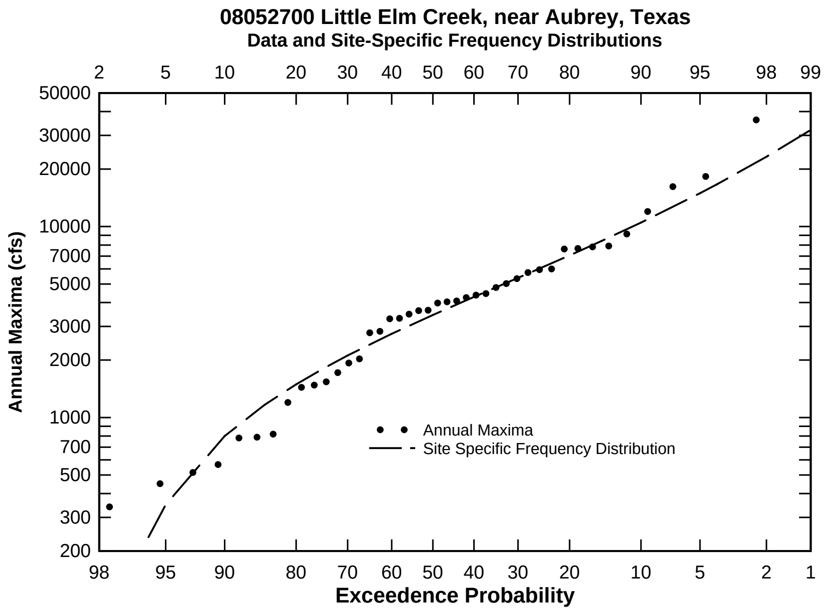

Figure C.18: Flood frequency curve for USGS station 08052700

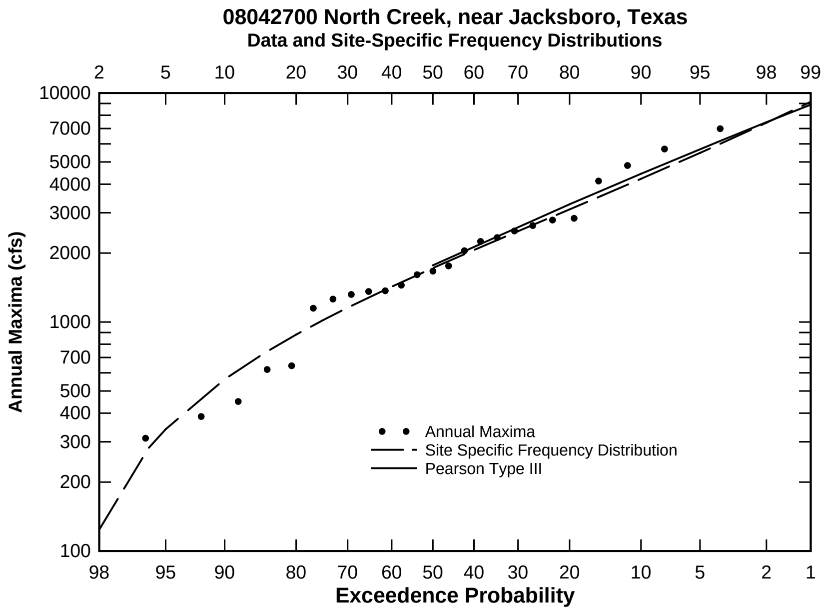

Figure C.19: Flood frequency curve for USGS station 08042700

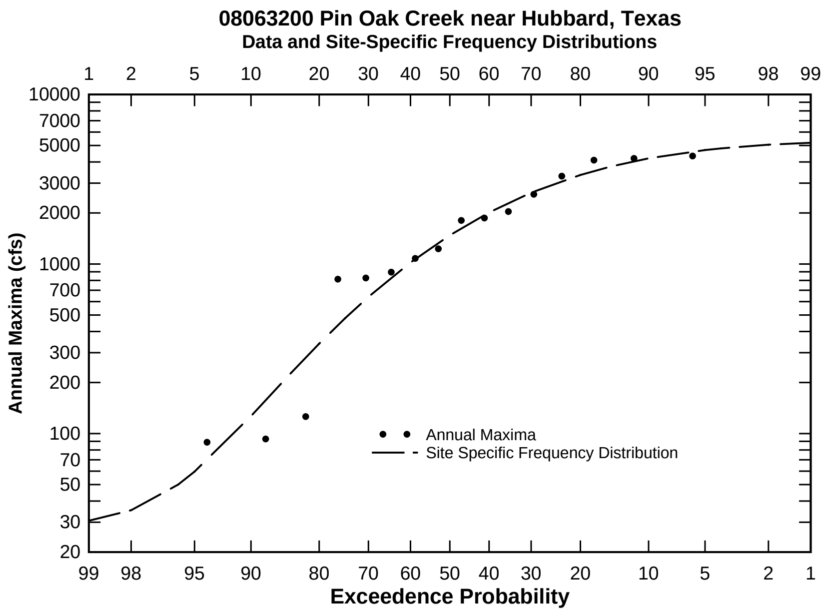

Figure C.20: Flood frequency curve for USGS station 08063200

D. Observed Runoff Coefficients

As described in the body of this report, the method of Schaake et al. (1967) was applied to measurements of rainfall and runoff for each of the study watersheds. Prior to analysis, values of rainfall and runoff were sorted (independently) from largest to smallest. The resulting rank-ordered pairs were used for analysis. Runoff coefficient was computed from rank-ordered rainfall and runoff using equation 2.2. Plots of the resulting runoff coefficient versus runoff depth are displayed on figures D.1–D.20. Values for the observed runoff coefficient (and the table runoff coefficient) are presented in Appendix A as table A.1.

While the sample for these analyses is relatively small, constituting only 20 watersheds, examination of figures D.1–D.20 suggests an adjustment of runoff coefficient for total depth of runoff may be justified. That is, as risk of exceedence diminishes (return interval increases), depth of runoff increases and so does the runoff coefficient. This observation is discussed in the body of the report.

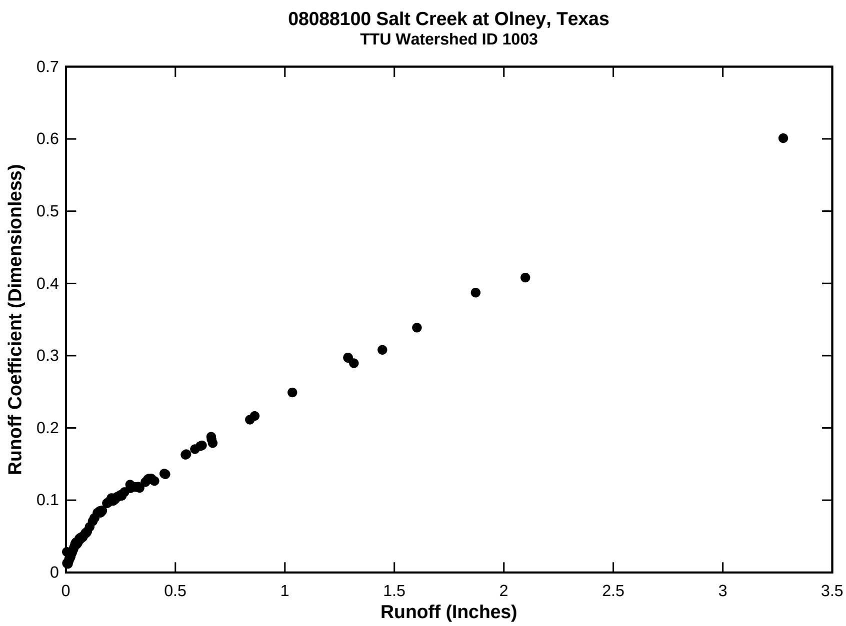

Figure D.1: Plot of runoff coefficient versus runoff depth for watershed 08088100

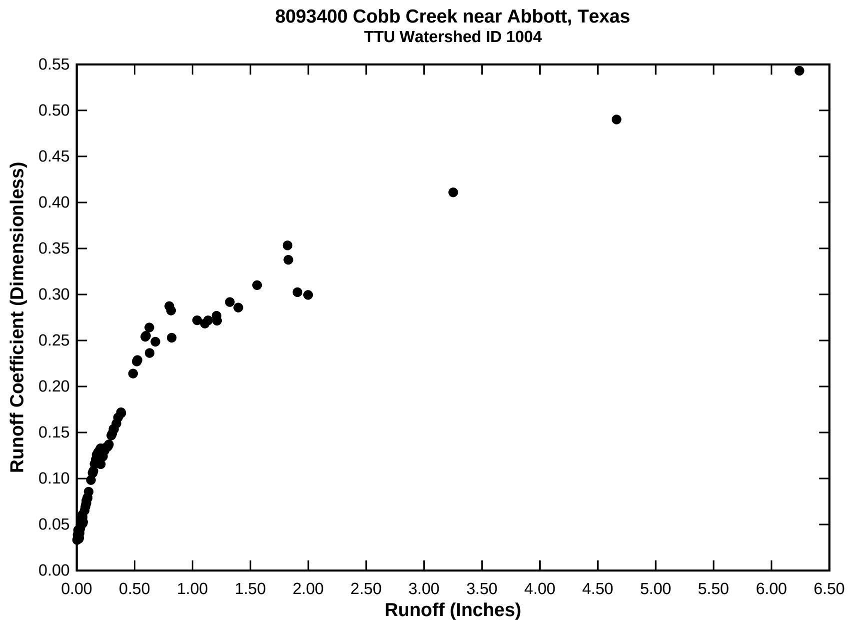

Figure D.2: Plot of runoff coefficient versus runoff depth for watershed 08093400

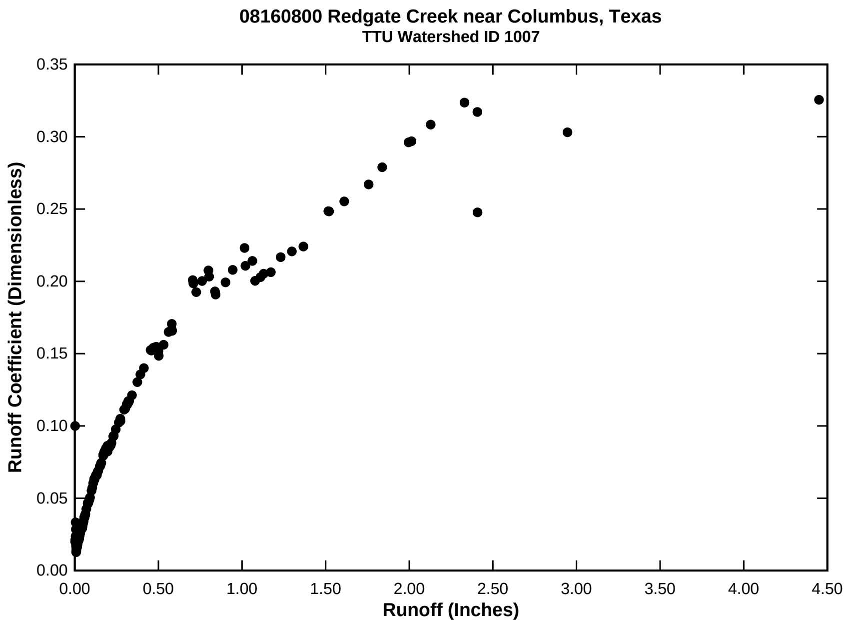

Figure D.3: Plot of runoff coefficient versus runoff depth for watershed 08160800

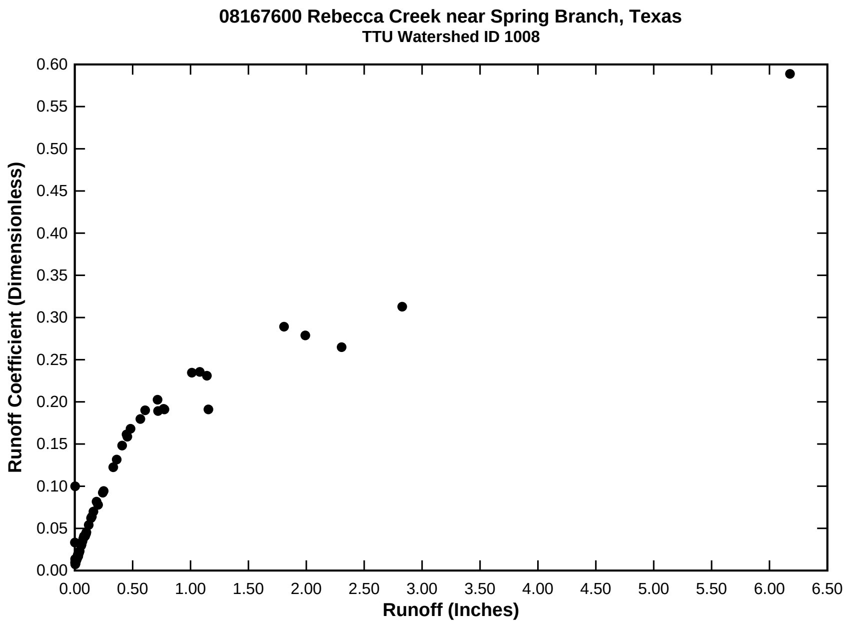

Figure D.4: Plot of runoff coefficient versus runoff depth for watershed 08167600

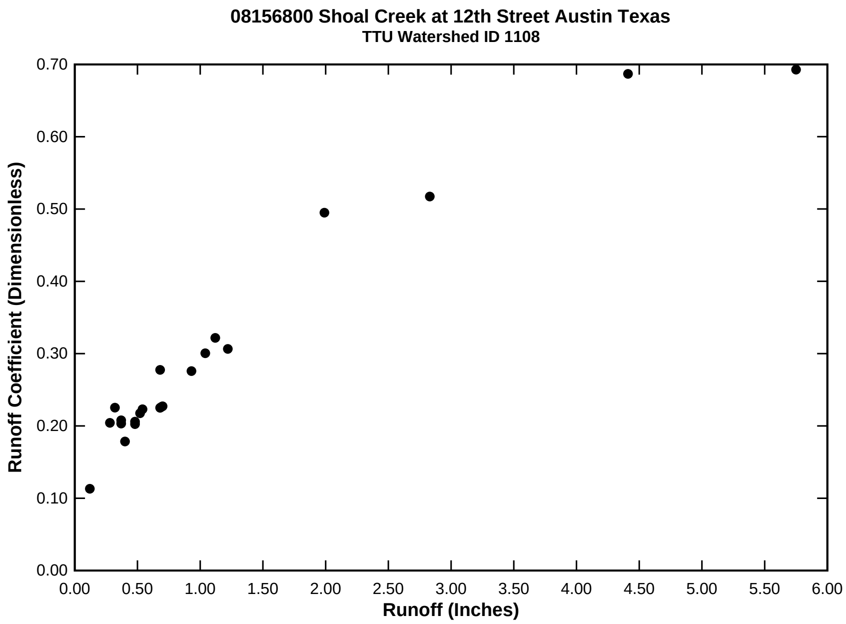

Figure D.5: Plot of runoff coefficient versus runoff depth for watershed 08156800

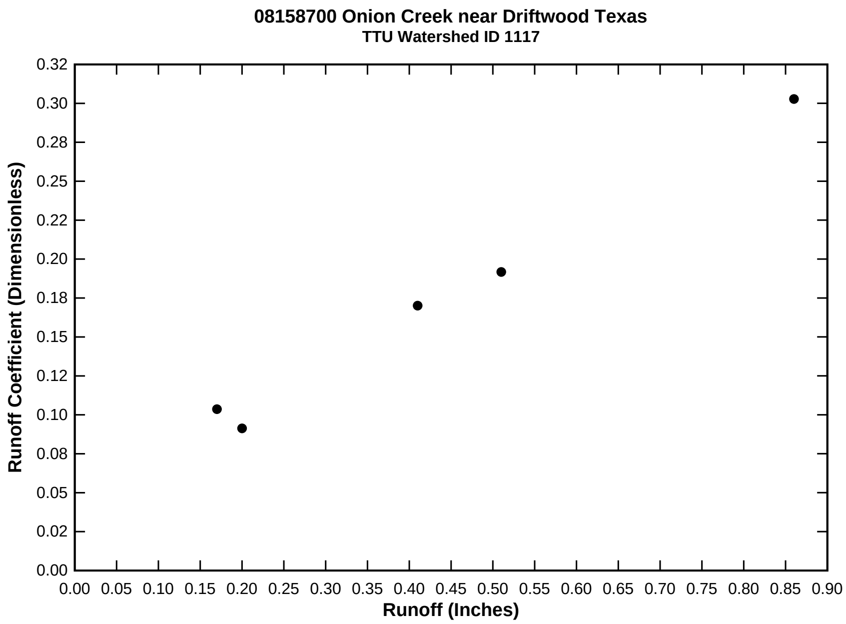

Figure D.6: Plot of runoff coefficient versus runoff depth for watershed 08158700

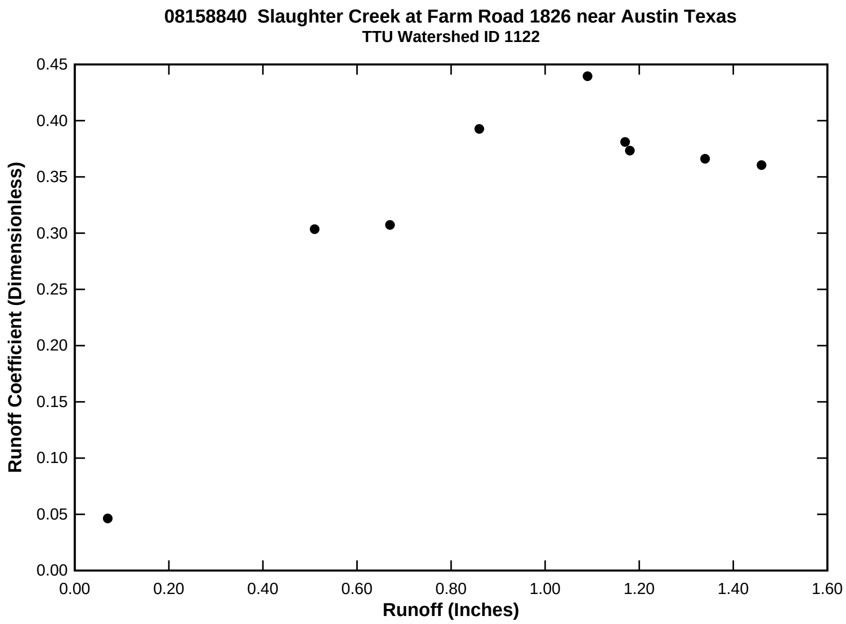

Figure D.7: Plot of runoff coefficient versus runoff depth for watershed 08158840

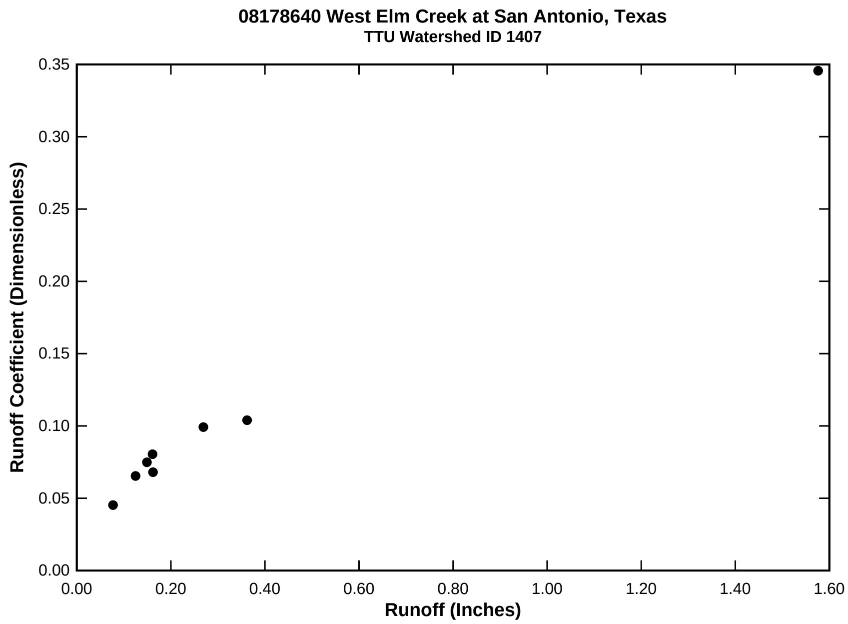

Figure D.8: Plot of runoff coefficient versus runoff depth for watershed 08178640

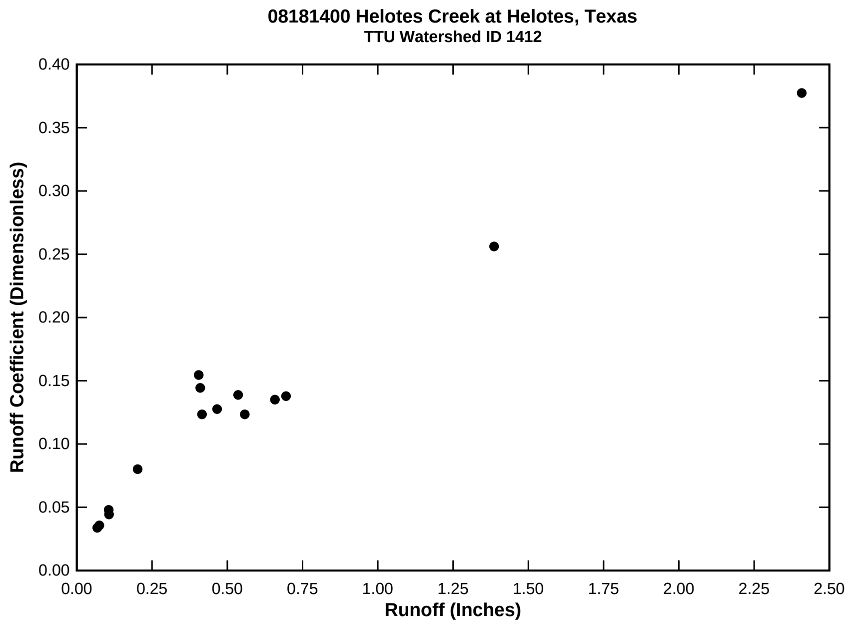

Figure D.9: Plot of runoff coefficient versus runoff depth for watershed 08181400.

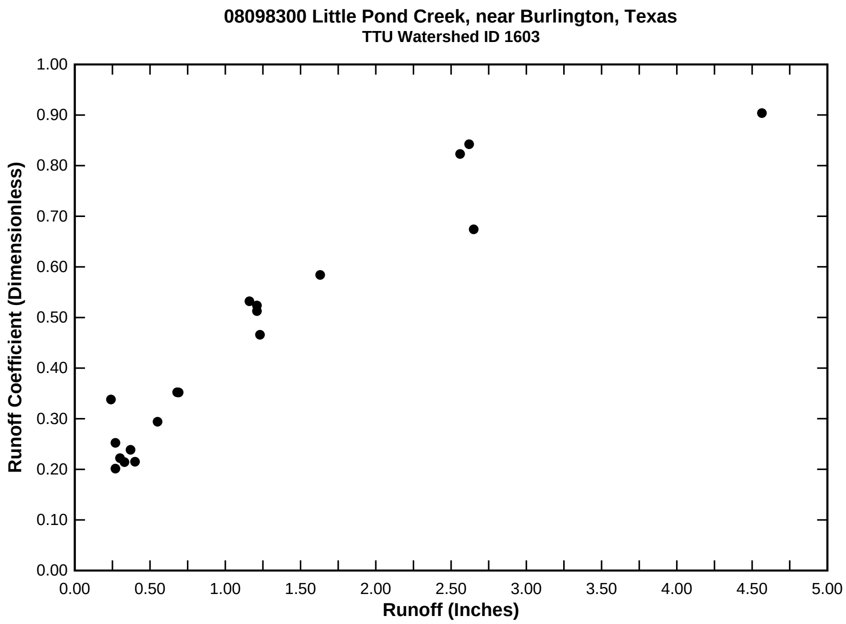

Figure D.10: Plot of runoff coefficient versus runoff depth for watershed 08098300

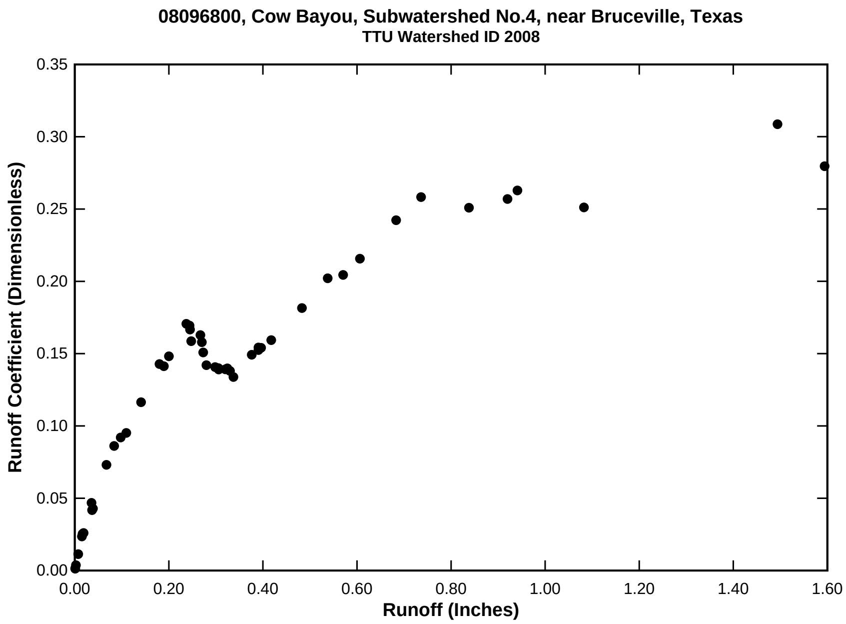

Figure D.11: Plot of runoff coefficient versus runoff depth for watershed 08096800

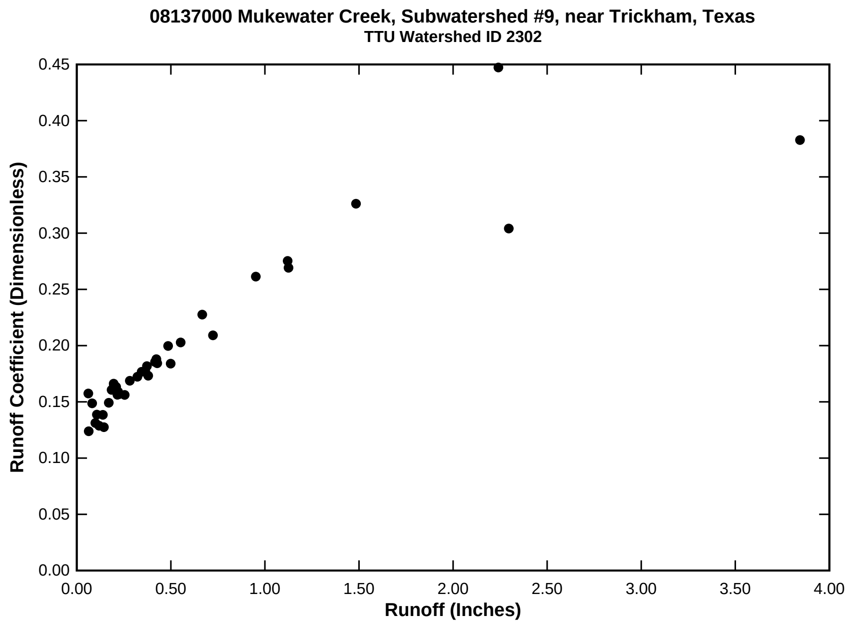

Figure D.12: Plot of runoff coefficient versus runoff depth for watershed 08137000

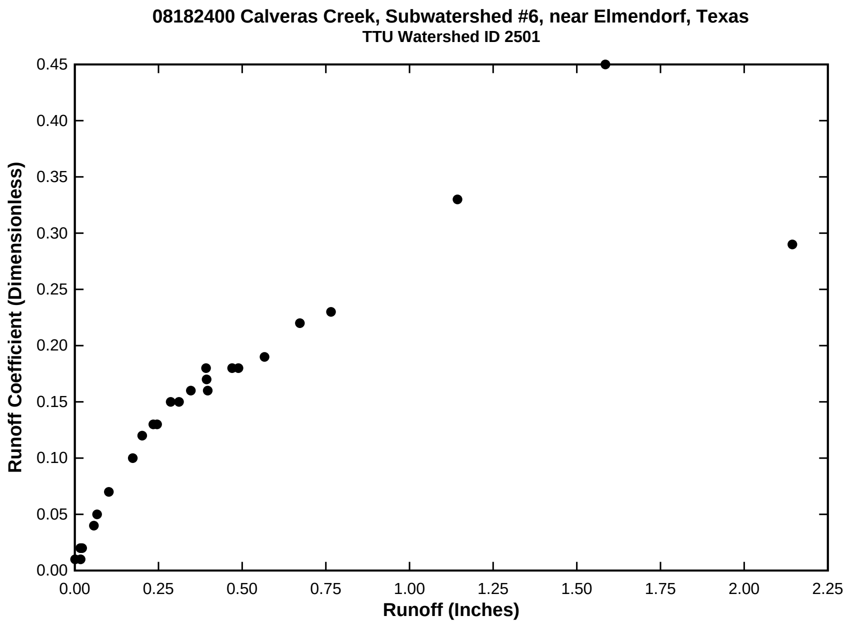

Figure D.13: Plot of runoff coefficient versus runoff depth for watershed 08182400

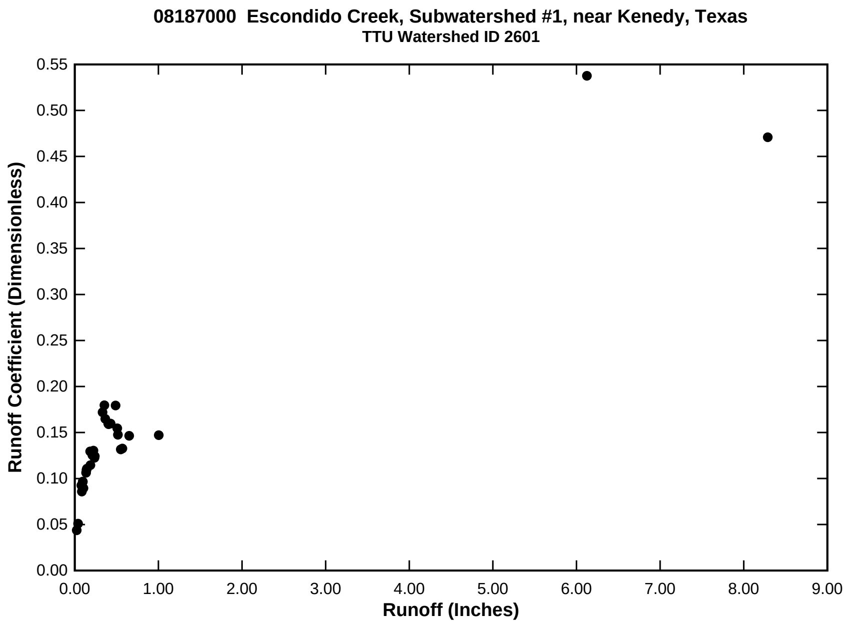

Figure D.14: Plot of runoff coefficient versus runoff depth for watershed 08187000

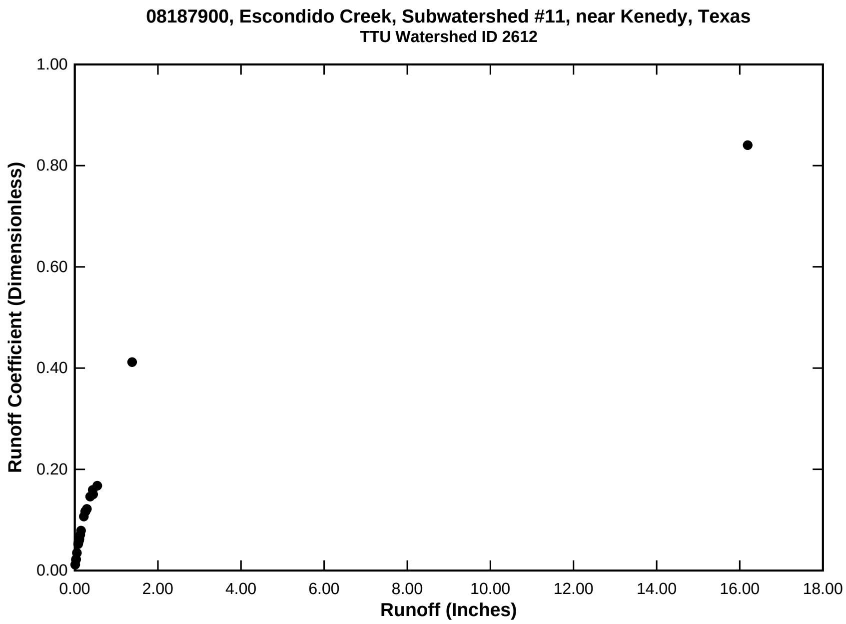

Figure D.15: Plot of runoff coefficient versus runoff depth for watershed 08187900

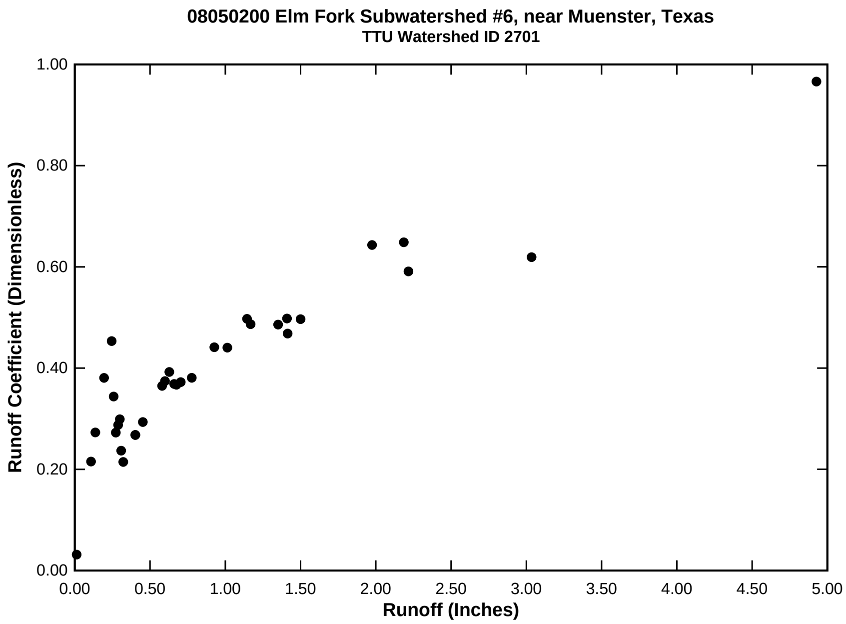

Figure D.16: Plot of runoff coefficient versus runoff depth for watershed 08050200

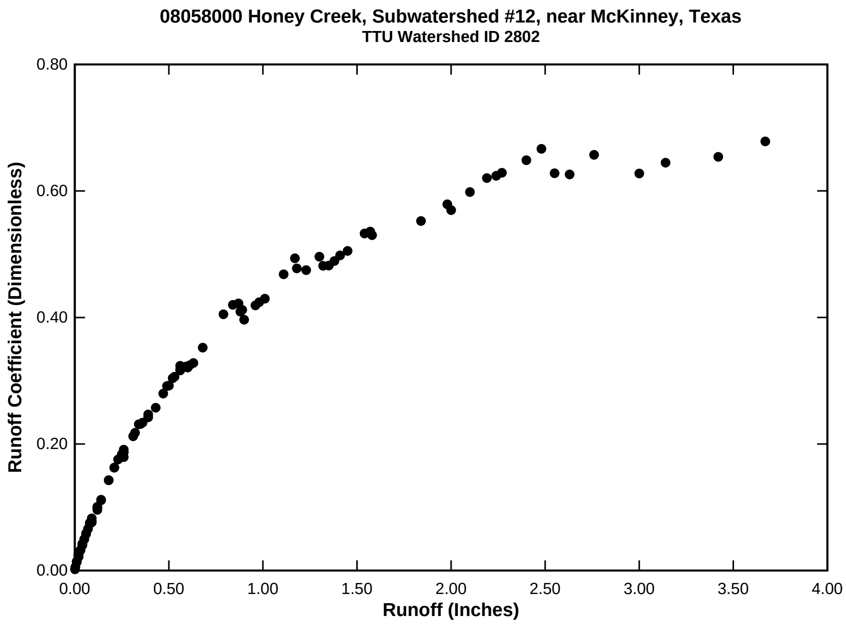

Figure D.17: Plot of runoff coefficient versus runoff depth for watershed 08058000

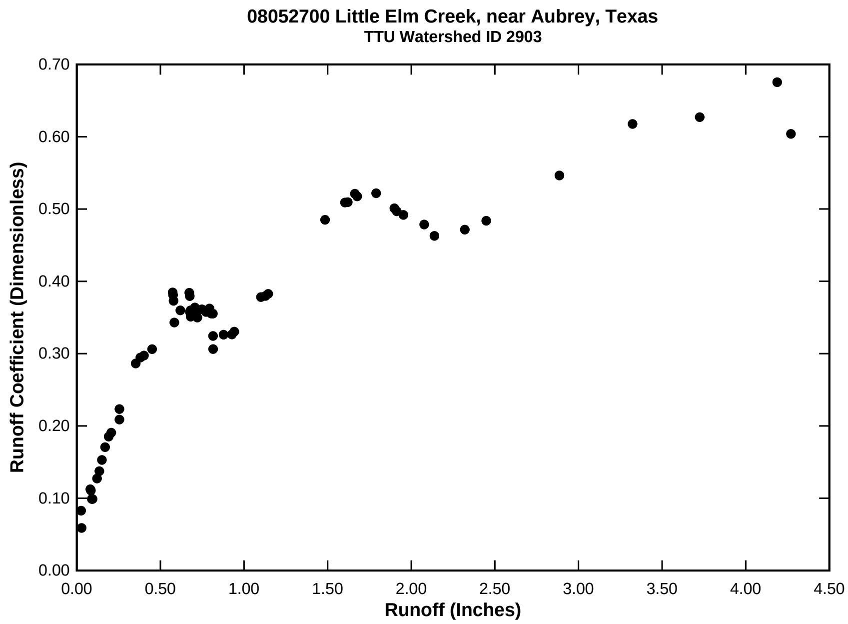

Figure D.18: Plot of runoff coefficient versus runoff depth for watershed 08052700

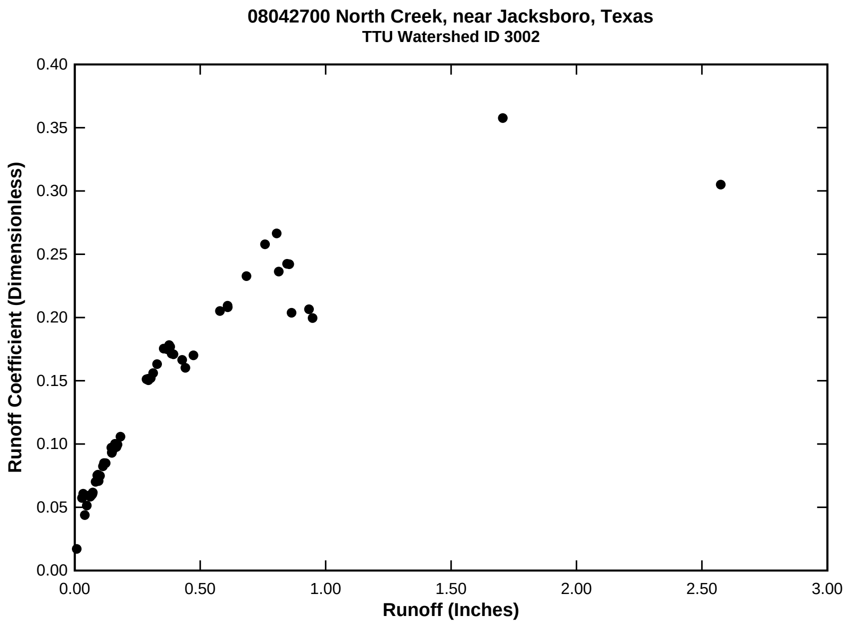

Figure D.19: Plot of runoff coefficient versus runoff depth for watershed 08042700

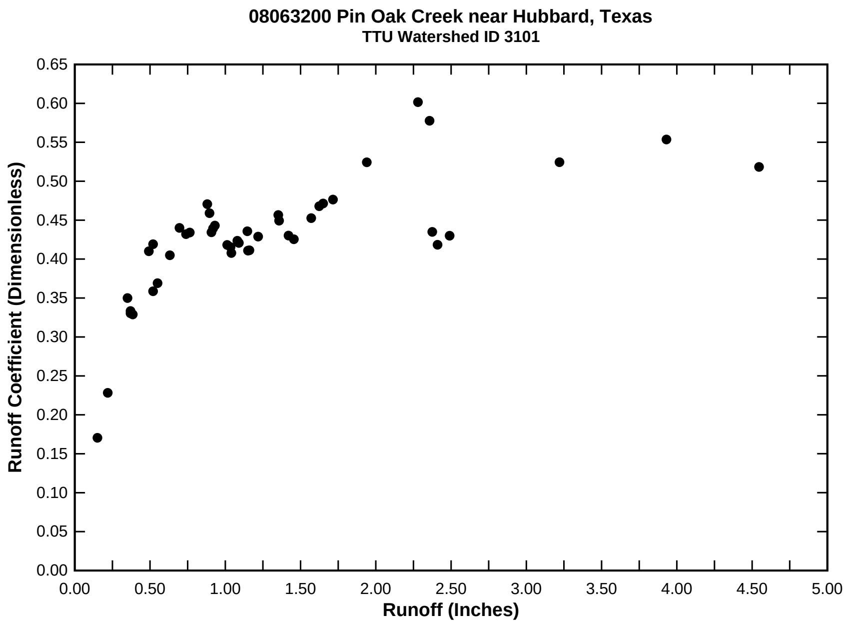

Figure D.20: Plot of runoff coefficient versus runoff depth for watershed 08063200