Development of Guidance for Runoff Coefficient Selection and Modified Rational Unit Hydrograph Method for Hydrologic Design

by

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Abstract

The rational method is the most widely used method by hydraulic and drainage engineers to estimate design discharges. The runoff coefficient (C) is a key parameter for the rational method. Literature-based C values (C_{lit}) are listed for different land-use/land-cover conditions in various design manuals and textbooks, but C_{lit} appear not to be derived from any observed data. In this study, C_{lit} values were derived for 90 watersheds in Texas from two sets of land-cover data for 1992 and 2001. C values were also estimated using observed rainfall and runoff data for more than 1,600 events in the study watersheds using two different approaches (1) the volumetric approach (C_v) (2) the rate-based approach (C_{rate}).When compared with the C_v values, about 80 percent of C_{lit} values were greater than C_v values. This result might indicate that literature-based C overestimate peak discharge for drainage design when used with the rational method. Similarly, when compared with the C_{rate} values, about 75 percent of C_{lit} values were greater than C_{rate} values, however, for developed watersheds with more impervious cover, C_{lit} values were greater than C_{rate} values. Rate-based C were also developed as function of return period for 36 undeveloped watersheds in Texas using peak discharge frequency from previously published regional regression equations and rainfall intensity frequency for return periods of 2, 5, 10, 25, 50, and 100 years. The C values of this study increased with return period more rapidly than the increase suggested in prior literature.

To use the rational method for hydraulic structures involving storage, the modified rational method (MRM) was developed. The hydrograph developed using the MRM can be considered application of a special unit hydrograph (UH) that is termed the modified rational unit hydrograph (MRUH) in this study. Being a UH, the MRUH can be applied to nonuniform rainfall distributions and for watersheds with drainage areas greater than typically used for the rational method (a few hundred acres). The MRUH was applied to 90 watersheds in Texas using 1,600 rainfall-runoff events. The MRUH performed as well as other three UH methods (Gamma, Clark-HEC-1, and NRCS) when the same rainfall loss model was used.

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List of Abbreviations

ASCE	American Society of Civil Engineers
DEM	Digital Elevation Model
GIS	Geographic Information System
IDF	Intensity-Duration-Frequency
IUH	Instantaneous Unit Hydrograph
LULC	Land-use/land-cover
MRM	Modified Rational Method
MRUH	Modified Rational Unit Hydrograph Method
NLCD	National Land Cover Dataset
NRCS	Natural Resources Conservation Service
NURP	National Urban Runoff Program
RHM	Rational Hydrograph Method
SBUH	Santa Barbara Unit Hydrograph
TxDOT	Texas Department of Transportation
USEPA	United States Environmental Protection Agency
USGS	U.S. Geological Survey
WPCF	Water Pollution Control Federation

Chapter1. Introduction

1.1 Background

Rational method and Runoff coefficient

Early storm water or catchment runoff estimation throughout the world was based on designer's experience and judgment. Current practice is that the watershed that is to be drained by a proposed storm sewer system will be generally divided into one or more sub-catchments or sub-watersheds that are of reasonable size and are approximately homogeneous in nature. These urban watersheds may include residential, commercial or industrial areas, but usually have larger proportions of pavement and the streets and roads which are the principal surface drainage conveyance, have short time of concentration, and have well-defined flow paths, typically through gutters, ditches and medians of streets and roads. Each year, billions of dollars are spent on new construction of drainage structures. For the safety, design of hydraulic structures is done based on the peak discharge (Q_p) as the design flow. Therefore, Q_p is the major hydrological parameter required for the hydraulic design purpose. Additional parameters such as volume of runoff and time of peak flow are required in some cases such as for the design of facilities that use storage such as detention and retention basins.

The rational method is the most widely used method by hydraulic and drainage engineers to estimate peak design discharges, which are used to size a variety of drainage structures for small urban (developed) and rural (undeveloped) watersheds (Viessman and Lewis 2003). The rational method was developed in the United States by Emil Kuichling (1889) and introduced to Great Britain by Lloyd-Davies (1906). The peak discharge (Q_p in m^3/s in SI units or ft^3/s in English units) for the method is computed using:

$$Q_p = m_0 CIA \tag{1.1}$$

where C is the runoff coefficient (dimensionless), I is the average rainfall intensity (mm/hr or in./hr) for a storm with a duration equal to a critical period of time (typically assumed to be the time of concentration), A is the drainage area (hectares or acres), and m_o is a dimensional correction factor (1/360 = 0.00278 in SI units, 1.008 in English units).

C is the variable of the rational method least amenable to precise determination, and estimation of C calls for judgment on the part of the engineer (ASCE and WPCF 1960; TxDOT 2002). C can vary substantially depending on watershed conditions. Therefore, research to document appropriate values of C is needed. Typical C values, representing the integrated effects of many watershed conditions (C_{lit}), are listed for different land-use/land-cover (LULC) conditions in various design manuals and textbooks (Chow et al. 1988; Viessman and Lewis 2003). Values of C_{lit} published by the joint committee were obtained from a survey, which received “71 returns of an extensive questionnaire submitted to 380 public and private organizations throughout the United States.” The results represented decades of professional practice experience using the rational method to determine runoff volumes in storm-sewer design applications (ASCE and WPCF 1960). No justification based on observed rainfall and runoff data for the selected C_{lit} values was provided in the ASCE and WPCF (1960) manual. In short, we conclude that C_{lit} values appeared heuristically determined, and therefore comparison of C values derived from observed rainfall and runoff data to the C_{lit} values is important.

In this study, we focus on applying different methods to estimate C for 90 watersheds in Texas using observed rainfall and runoff data for 1,600 events. C_{lit} values were derived from two sets of LULC data for 1992 and 2001. Volumetric runoff coefficients (C_v) were estimated by event totals of observed rainfall and runoff depths from more than 1,600 events observed in the watersheds. C_v values were also estimated using rank-ordered pairs of rainfall and runoff depths (frequency matching).

It is important to stress that rational method is the rate-based method (eq. 1.1). Current runoff coefficients given in textbooks and design manuals are neither volumetric nor rate-based because they were not derived from observed data but are used for the rate-based rational method. In this study, the rate-based runoff coefficients, C_{rate}, were estimated for each of 1,600 rainfall-runoff events and the time window used to determine average rainfall intensity was time of concentration computed using the Kirpich method. Subsequently, the frequency-matching approach was used to extract a representative runoff coefficient, C_r, for each watershed. C values developed from both the volumetric and rate-based approaches are compared independently with C_{lit} values.

A substantial criticism of the rational method arises because observed C values vary from storm to storm (Schaake et al. 1967; Pilgrim and Cordery 1993). The C has been considered a function of return period by various researchers (Jens 1979; Pilgrim and Cordery 1993; Hotchkiss and Provaznik 1995; Titmarsh et al. 1995; Young et al. 2009). Using watershed parameters, such as drainage area, slope, and channel length and regression equations of discharge and rainfall intensity at different return periods *T* of 2-, 5-, 10-, 25-, 50-, and 100-years, the *C(T)* were estimated in 36 undeveloped watersheds in Texas. Subsequently, frequency factors $C_f(T) = C(T)/C(10)$ were computed. Results of *C(T)* and frequency factors $C_f(T)$ were analyzed and compared with previous studies.

Modified Rational Method **(MRM)**

Incorporation of detention basins to mitigate effects of urbanization on peak flows required design methodologies to include the volume of runoff as well as the peak discharge (Rossmiller 1980). To use the rational method for hydraulic structures involving storage, the modified rational method (MRM) was developed (Poertner 1974). While the original rational method is meant to produce only the peak design discharge, the MRM produces a runoff hydrograph and the runoff volume of the entire watershed. The MRM, which has found widespread use in the engineering practice since 1970s, is used to size detention/retention facilities for a specified recurrence interval and concurrent release rate. The MRM is based on the same assumptions as the conventional rational method, that is, the rainfall is uniform in space over the drainage area being considered and the rainfall intensity is uniform throughout the duration of the storm (Rossmiller 1980).

The MRM was revisited and reevaluated in this study. The hydrograph developed from application of the MRM is a special case of the unit hydrograph method and will be termed the modified rational unit hydrograph (MRUH) in this study. Being a unit hydrograph, the MRUH can be applied to nonuniform rainfall distributions. Furthermore, the MRUH can be used on watersheds with drainage areas in excess of the typical limit for application of the rational method or the modified rational method (a few hundred acres). The MRUH was applied to 90 watersheds in Texas using 1,600 rainfall-runoff events. The Gamma UH, Clark-HEC-1 UH, and NRCS dimensionless UH were also used to predict peak discharges of all events in the database.

1.2 Research Objectives

This research work is a part of TxDOT Project 0-6070 "Use of the Rational and the Modified Rational Methods for TxDOT Hydraulic Design". The principal objective of the project is to evaluate appropriate conditions for the use of the rational method and modified rational methods for designs on small watersheds, evaluate and refine, if necessary, current tabulated values of the runoff coefficient and construct guidelines for TxDOT analysts for the selection of appropriate parameter values for Texas conditions. The specific objectives are:

1. Estimation of areally-weighted literature-based runoff coefficients (C_{lit}) for the study watersheds using land-use data.

2. Estimation of volumetric runoff coefficients (C_v) for the study watersheds using rainfall-runoff data.

3. Estimation of the rate based runoff coefficients (C_{rate}) for the study watersheds using rainfall-runoff data and comparison of C_{lit} with both the C_v and C_{rate}.

4. Estimation of the runoff coefficients for different return periods and compare the current frequency multiplier $C(T)/C(10)$ in the literature with our results.

5. Evaluate the applicability of the modified rational unit hydrograph method (MRUH) if blindly applied to watersheds of sizes greater than originally intended with either the rational method or the modified rational method (that is, a few hundred acres).
6. Study the effects of runoff coefficient and the timing parameters on predictions of runoff hydrographs using MRUH.

1.3 Study Area and Rainfall-Runoff Database

Watershed data from a larger dataset accumulated by researchers from the U.S. Geological Survey (USGS) Texas Water Science Center, Texas Tech University, University of Houston, and Lamar University (Asquith et al. 2004) and previously used in a series of research projects funded by the Texas Department of Transportation (TxDOT) were used for this study. The data were collected as a part of USGS small-watershed projects and urban watershed studies during 1959–1986 (Asquith et al. 2004). The original data, available in the form of 220 printed USGS data reports, were transcribed to digital format manually (Asquith et al. 2004). Incidentally, these data also are used by Cleveland et al. (2006), Asquith and Roussel (2007), Fang et al. (2007, 2008), and Dhakal et al. (2012).

The dataset comprises 90 USGS streamflow-gaging stations in Texas, each representing a different watershed (Fang et al. 2007, 2008). There are 29, 21, 7, 13 watersheds in Austin, Dallas, Fort Worth, and San Antonio areas, respectively, and remaining 20 watersheds are small rural watersheds in Texas. The drainage area of study watersheds ranged from approximately 0.8–440.3 km^2 (0.3–170 mi^2), with median and mean values of 17.0 km^2 (6.6 mi^2) and 41.1 km^2 (15.9 mi^2), respectively. There are 33, 57, and 80 study watersheds with drainage areas less than 13 km^2 (5 mi^2), 26 km^2 (10 mi^2), and 65 km^2 (25 mi^2), respectively. The stream slope of study watersheds ranged from approximately 0.0022–0.0196, with median and mean values of 0.0075 and 0.081, respectively. The percentage of impervious area (IMP) of study watersheds ranged from approximately 0.0–74.0, with median and mean values of 18.0 and 28.4, respectively.

The rainfall-runoff dataset comprised about 1,600 rainfall-runoff events. The number of events available for each watershed varied; for some watersheds only a few events were available whereas for some others as many as 50 events were available (Cleveland et al. 2006). Values of rainfall depths for 1,600 events ranged from 3.56 mm (0.14 in.) to 489.20 mm (19.26 in.), with median and mean values of 57.15 mm (2.25 in.) and 66.29 mm (2.61 in.), respectively. Values of maximum rainfall intensities calculated using time of concentration for 1,600 events ranged from 0.01 mm/min (0.03 in./hr) to 2.54 mm/min (6.01 in./hr), with median and mean values of 0.25 mm/min (0.58 in./hr) and 0.30 mm/min (0.72 in./hr), respectively.

1.4 Organization of Dissertation

This dissertation is organized into six chapters. Chapter two to five are organized in journal paper format prepared for ASCE journal publication. Parts of results of the study were presented in two conference papers:

1. Dhakal, N., Fang, X., Cleveland, T. G., Thompson, D. B., and Marzen, L. J. (2010). "Estimation of rational runoff coefficients for Texas watersheds." *Proceeding (CD-ROM) for 2010 World Environmental and Water Resources Congress*, Providence, Rhode Island.

2. Nirajan Dhakal, Xing Fang, Theodore G. Cleveland, and David B. Thompson, 2011. "Revisiting Modified Rational Method". *Proceeding (CD-ROM) for 2011 World Environmental and Water Resources Congress*, Palm Springs, CA, May 22-26, 2011.

Chapter two deals with the estimation of the volumetric runoff coefficients (C_v) for the study watersheds using both land-use and rainfall-runoff data. Two regression equations of C_v versus percent impervious area were developed and combined into a single equation which can be used to rapidly estimate C_v values for similar Texas watersheds. The work of this chapter has been published in the ASCE Journal of Irrigation and Drainage Engineering (Nirajan Dhakal, Xing Fang, Theodore G. Cleveland, David B. Thompson, William H. Asquith, and Luke J. Marzen, 2012 (January). "Estimation of Volumetric Runoff Coefficients for Texas Watersheds Using Land-Use and Rainfall-Runoff Data." *ASCE Journal of Irrigation and Drainage Engineering, 138(1):43-54, DOI=10.1061/(ASCE)IR.1943-4774.0000368*).

Chapter three deals with the estimation of the rate based runoff coefficients for the study watersheds from the rainfall-runoff data. An equation applicable to many Texas watersheds is proposed to estimate C as a function of impervious area. The work of this chapter has been revised and resubmitted for publication in the ASCE Journal of Hydrologic Engineering (Nirajan Dhakal, Xing Fang, William H. Asquith, Theodore G. Cleveland, and David B. Thompson. "Rate-based Estimation of the Runoff Coefficients for Selected Watersheds in Texas". *ASCE Journal of Hydrologic Engineering*).

Chapter four deals with the estimation of the runoff coefficients based on the return period. The work of this chapter has been submitted for review and publication in the ASCE Journal of Irrigation and Drainage Engineering (Nirajan Dhakal, Xing Fang, William H. Asquith, Theodore G. Cleveland, and David B. Thompson. “Return Period Adjustments for Runoff Coefficients Based on Analysis in Texas Watersheds”. *ASCE Journal of Irrigation and Drainage Engineering*. *In Review*).

Chapter five deals with the development and application of the Modified Rational Unit Hydrograph Method (MRUH) for the study watersheds. The Gamma UH, Clark-HEC-1 UH, and NRCS dimensionless UH were also used to predict peak discharges of all events in the database. The work of this chapter has been submitted for review and publication in the ASCE Journal of Hydrologic Engineering (Nirajan Dhakal, Xing Fang, David B. Thompson, and Theodore G. Cleveland. “Modified Rational Unit Hydrograph Method and Applications in Texas Watersheds”. *ASCE Journal of Hydrologic Engineering*. *In Review*).

Chapter six summarizes the conclusion of the study and provides some recommendations for the future study in this area.

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Chapter 2. Estimation of Volumetric Runoff Coefficients for Texas Watersheds Using Land-Use and Rainfall-Runoff Data

2.1 Abstract

The rational method for peak discharge (Q_p) estimation was introduced in the 1880s. Although the rational method is considered simplistic, it remains an effective method for estimating peak discharge for small watersheds. The runoff coefficient (C) is a key parameter for the rational method and there are various ways to estimate C. Literature-based C values (C_{lit}) are listed for different land-use/land-cover (LULC) conditions in various design manuals and textbooks. However, these C_{lit} values were developed without much basis on observed rainfall and runoff data. C_{lit} values were derived for 90 watersheds in Texas from two sets of LULC data for 1992 and 2001; C_{lit} values derived from the 1992 and 2001 LULC datasets were essentially the same. Also for this study, volumetric runoff coefficients (C_v) were estimated by event totals of observed rainfall and runoff depths from more than 1,600 events observed in the watersheds. Watershed-median and watershed-average C_v values were computed and both are consistent with the data from the National Urban Runoff Program. C_v values also were estimated using rank-ordered pairs of rainfall and runoff depths (frequency matching). As anticipated, C values derived by all three methods (literature-based, event totals, and frequency matching) consistently have larger values for developed watersheds than undeveloped watersheds. Two regression equations of C_v versus percent impervious

area were developed and combined into a single equation which can be used to rapidly estimate C_v values for similar Texas watersheds.

2.2 Introduction

Estimation of peak discharge and runoff values for use in designing certain hydraulic structures (e.g., crossroad culverts, drainage ditches, urban storm drainage systems, and highway bridge crossings) are important and challenging aspects of engineering hydrology (Viessman and Lewis 2003). Various methods are available to estimate peak discharges and runoff volumes from urban watersheds (Chow et al. 1988). The rational method is the most widely used method by hydraulic and drainage engineers to estimate design discharges, which are used to size a variety of drainage structures for small urban (developed) and rural (undeveloped) watersheds (Viessman and Lewis 2003). The rational method was developed in the United States by Emil Kuichling (1889) and introduced to Great Britain by Lloyd-Davies (1906). The peak discharge (Q_p in m^3/s in SI units or ft^3/s in English units) for the method is computed using:

$$Q_p = m_o C I A, \tag{2.1}$$

where C is the runoff coefficient (dimensionless), I is the average rainfall intensity (mm/hr or in./hr) for a storm with a duration equal to a critical period of time (typically assumed to be the time of concentration), A is the drainage area (hectares or acres), and m_o is a dimensional correction factor (1/360 = 0.00278 in SI units, 1.008 in English units).

The precise definition and subsequent interpretation of C varies. The C of a watershed can be defined either as the ratio of total depth of runoff to total depth of rainfall or as the ratio of peak rate of runoff to rainfall intensity for the time of concentration (Wanielista and Yousef 1993). Kuichling (1889) analyzed observed rainfall and discharge data for developed urban watersheds in Rochester, NY, and computed the percentage of the rainfall discharged during the period of greatest flow as $Q_p/(IA)$, which is equal to runoff coefficient C from equation (2.1). Kuichling concluded that the percentage of the rainfall discharged for any given watershed studied is nearly equal to the percentage of impervious surface within the watershed, and this is the original meaning of C introduced by Kuichling (1889). Using Kuichling's definition, $C = 0$ for a strictly pervious surface and $C = 1$ for a strictly impervious surface.

C is the variable of the rational method least amenable to precise determination, and estimation of C calls for judgment on the part of the engineer (ASCE and WPCF 1960; TxDOT 2002). Typical C values, representing the integrated effects of many watershed conditions, are listed for different land-use/land-cover (LULC) conditions in various design manuals and textbooks (Chow et al. 1988; Viessman and Lewis 2003). The source of these published C values (literature-based C, called C_{lit} in this paper) derives from the 1960 sanitary and storm sewer design manual produced by a joint committee of the American Society of Civil Engineers (ASCE) and the Water Pollution Control Federation (WPCF). Values of C_{lit} published by the joint committee were obtained from a survey, which received "71 returns of an extensive questionnaire submitted to 380 public and private organizations throughout the United States." The results represented decades of professional practice experience using the rational method to determine runoff volumes in storm-sewer design applications (ASCE and WPCF 1960). No justification based on observed rainfall and runoff data for the selected C_{lit}

values was provided in the ASCE and WPCF (1960) manual. However, analysis of observed rainfall and runoff data was presented by Kuichling (1889).

In this paper, three methods were implemented to estimate *C* for 90 watersheds in Texas (Figure 2.1). The first method used LULC information for a watershed and published C_{lit} values for various land uses to derive a watershed-composite C_{lit}. The second method estimated the volumetric runoff coefficient (C_v) values by the ratio of total runoff depth to total rainfall depth for individual storm events. About 1,600 rainfall-runoff events measured from 90 Texas watersheds were analyzed to determine event, watershed-median, and watershed-average C_v values. C_v determined from storm events is called the back-computed volumetric runoff coefficient (C_{vbc}) in this paper. The third method computed probabilistic C_v values from the rank-ordered pairs of observed rainfall and runoff depths of a watershed and extracted a representative C_v for the watershed from the plot of C_v versus rainfall depths. C_v determined from the rank-ordered data is called the rank-ordered volumetric runoff coefficient (C_{vr}) in this paper. The third method is similar to the procedure used by Schaake et al. (1967). *C* values estimated by the three different methods were analyzed and compared. Regression equations of C_{vbc} and C_{vr} versus percent impervious area are presented.

2.3 Watersheds Studied and Rainfall-Runoff Database

Watershed data taken from a larger dataset (Asquith et al. 2004) accumulated by researchers from the U.S. Geological Survey (USGS) Texas Water Science Center, Texas Tech University, University of Houston, and Lamar University, and previously used in a series of research projects funded by the Texas Department of Transportation (TxDOT) were used for this study. The dataset comprises 90 USGS streamflow-gaging stations in Texas, each representing a different watershed (Fang et al. 2007, 2008). Location and distribution of the stations in Texas are shown in Figure 2.1. There are 29, 21, 7, 13 watersheds in Austin, Dallas, Fort Worth, and San Antonio areas, respectively, and remaining 20 watersheds are small rural watersheds in Texas (Figure 2.1). The drainage area of study watersheds ranged from approximately 0.8–440.3 km^2 (0.3–170 mi^2), with median and mean values of 17.0 km^2 (6.6 mi^2) and 41.1 km^2 (15.9 mi^2), respectively. There are 33, 57, and 80 study watersheds with drainage areas less than 13 km^2 (5 mi^2), 26 km^2 (10 mi^2), and 65 km^2 (25 mi^2), respectively. The stream slope of study watersheds ranged from approximately 0.0022–0.0196, with median and mean values of 0.0075 and 0.081, respectively. The percentage of impervious area (IMP) of study watersheds ranged from approximately 0.0–74.0, with median and mean values of 18.0 and 28.4, respectively.

Many would argue that the application of the rational method is not appropriate for the range of watershed areas presented in this study. For example, watershed drainage area is a criteria used to select a hydrologic method (Chow et al. 1988) to compute peak discharge according to TxDOT guidelines for drainage design. The TxDOT guidelines recommend the use of the rational method for watersheds with drainage areas less than 0.8 km^2 (200 acres) (TxDOT 2002). However, French et al. (1974) estimated values of the runoff coefficient in New South Wales, Australia, for 37 rural watersheds, ranging in size up to 250 km^2 (96 mi^2). Young et al. (2009) determined runoff coefficients for 72 rural watersheds in Kansas with drainage areas up to 78 km^2

(30 mi^2). ASCE and WPCF (1960) made the following statements when the rational method was introduced for design and construction of sanitary and storm sewers:

"Although the basic principles of the rational method are applicable to large drainage areas, reported practice generally limits its use to urban areas of less than 5 sq miles. Development of data for application of hydrograph methods is usually warranted on larger areas" (ASCE and WPCF 1960, p. 32).

Chow et al. (1988) and Viessman and Lewis (2003) do not specify an area limit for application of the rational method. Pilgrim and Cordery (1993) stated that the rational method is one of the three methods widely used to estimate peak flows for small to medium sized basins. "It is not possible to define precisely what is meant by "small" and "medium" sized, but upper limits of 25 km^2 (10 mi^2) and 550 km^2 (200 mi^2), respectively, can be considered as general guides" (Pilgrim and Cordery 1993, p. 9.14). Results of this study will further indicate that there is no demonstrable trend in runoff coefficient with drainage area.

The rainfall-runoff dataset comprised about 1,600 rainfall-runoff events recorded during 1959–1986. The number of events available for each watershed varied; for some watersheds only a few events were available whereas for some others as many as 50 events were available (Cleveland et al. 2006). Values of rainfall depths for 1,600 events ranged from 3.56 mm (0.14 in.) to 489.20 mm (19.26 in.), with median and mean values of 57.15 mm (2.25 in.) and 66.29 mm (2.61 in.), respectively. Values of maximum rainfall intensities calculated using time of concentration for 1,600 events ranged from 0.01 mm/min (0.03 in./hr) to 2.54 mm/min (6.01 in./hr), with median and mean values of 0.25 mm/min (0.58 in./hr) and 0.30 mm/min (0.72 in./hr), respectively.

A geospatial database was developed from another TxDOT project (Roussel et al. 2005), containing watershed boundaries for the 90 watersheds, delineated using a 30-meter digital elevation model (DEM). The geospatial database contains watershed drainage area, longitude and latitude of the USGS streamflow-gaging station, which was treated as the outlet of the watershed, and 42 watershed characteristics (e.g., main channel length, channel slope, basin width, etc.) of each individual watershed (Roussel et al. 2005). Each of the 90 watersheds was classified as either developed (urbanized) or undeveloped (Roussel et al. 2005, Cleveland et al. 2008). Forty-four developed watersheds are located in four metropolitan areas in Texas (Austin, Dallas, Fort Worth, and St. Antonio) and used for USGS urban studies from 1959 to 1986. Thirty-six undeveloped watersheds include 20 small rural watersheds and 16 watersheds in suburban of the four metropolitan areas. The classification scheme of developed and undeveloped watersheds parallels and accommodates the disparate discussion and conceptualization in more than 220 USGS reports that provided the original data for the rainfall and runoff database (Asquith et al. 2004). Although this binary classification seems arbitrary, it was purposeful and reflected the uncertainty in watershed development condition at the time the rainfall-runoff data were collected (Asquith and Russell 2007). This binary classification was successfully used to develop regression equations to estimate the shape parameter and the time to peak for regional Gamma unit hydrographs for Texas watersheds (Asquith et al. 2006).

2.4 Estimation of Runoff Coefficients Using LULC Data

C is strongly dependent on land use, and to a lesser extent, on watershed slope (Schaake et al. 1967; ASCE 1992). For watersheds with multiple land-use classes, a composite (area-weighted average) runoff coefficient, C_{lit}, can be estimated using:

$$C_{lit} = \frac{\sum_{i=1}^{n} C_i A_i}{\sum_{i=1}^{n} A_i}, \tag{2.2}$$

where, $i = i^{th}$ sub-area with particular land-use type, n = total number of land-use classes in the watershed, C_i = literature-based runoff coefficient for i^{th} land-use class, and A_i = sub-area size for i^{th} land-use class in the watershed (TxDOT 2002). In this study, watershed-composite C_{lit} values were derived for the 90 watersheds in Texas using LULC information and published C_{lit} values from various literatures. A geographic information system (GIS) was used for sub-areal extraction of different LULC classes within a particular watershed (ESRI 2004). The 1992 and 2001 National Land Cover Data (NLCD) for Texas were obtained from the USGS website http://seamless.usgs.gov/ (accessed on May 30, 2008).

Each watershed has different LULC classes distributed within its boundary. Of 16-LULC classes from the NLCD 2001 data, 15 were used for the 90 watersheds studied; definitions of NLCD LULC classes are available at http://www.epa.gov/mrlc/definitions.html (accessed on May 30, 2008). Runoff coefficients were assigned for the 12-NLCD, 2001-LULC classes or mixed classes as listed in Table 2.1, which are based on 15-NLCD 2001-LULC classes; the table includes sources and references for the selected C values. From all sources considered, C values typically were not available for most of 15-NLCD LULC classes, but similar land-use types from literature were identified to match NLCD LULC (Table 2.1). A C value of 1 was assigned to open water, woody wetlands and emergent herbaceous wetlands and is not shown in Table 2.1. For the other LULC classes a range of C values were available from the mentioned sources under similar LULC types, and the average values (listed in column 3 of Table 2.1) were taken as literature C values for the study before a sensitivity analysis of C_{lit} on selected C values for different LULC classes from literature was conducted.

Using NLCD 2001 data and standard published mean C values (Table 2.1), composite runoff coefficients, C_{lit}, for the 90 Texas watersheds were developed using equation (2.2) (Table 2.2). Values of C_{lit} ranged from 0.29 to 0.63, with median and mean values of 0.50 and 0.47, respectively (Table 2.2). Estimates of C_{lit} for a given watershed may differ, depending on the experience and judgment used in assigning them to LULC classes and estimating areas for land-use classes. For example, Harle (2002) determined C_{lit} for a subset of 36 watersheds from the 90 Texas watersheds using standard C_{lit} tables published by TxDOT (2002). The average absolute difference between Harle's estimate of C_{lit} and that presented in this study was 0.06 and the maximum absolute difference was 0.13.

The NLCD 1992 data were used to examine the potential for temporal differences in composite C_{lit} estimates. When NLCD 1992 data were used, 18 out of 21 LULC classes for the 90 watersheds were used. This difference (from the 15-LULC classes determined using NLCD 2001) occurred because there were more land use (LU) codes for some land-cover classes in NLCD 1992. Summary statistics of the composite runoff coefficients, C_{lit}, obtained using NLCD 1992 and 2001 data are listed in Table 2.2. The average values of the runoff coefficients for 90 watersheds derived from two LULC datasets are the same (0.47, see Table 2.2). The median absolute difference of C_{lit} derived from the two LULC datasets is 0.03 with a minimum difference of 0.00 and a maximum difference of 0.14. The differences between C_{lit} obtained using two LULC datasets are plotted in Figure 2.2 (top). The authors conclude that there is no substantial difference between C_{lit} values derived from the 1992 and 2001 LULC datasets because the paired t-test gives p-value (Ayyub and McCuen 2003) of 0.88, much larger than the level of significance 0.05 for the level of confidence 95%.

A sensitivity analysis was conducted to examine effect of selected C values for different LULC classes from literature on watershed composite runoff coefficient C_{lit}. The minimum and maximum C values for each LULC class from literature (Table 2.1) were used to derive watershed-minimum and watershed-maximum C_{lit} values for each watershed, respectively. The NLCD 2001 data was used for the sensitivity analysis. The cumulative distributions of watershed-minimum, -average, and -maximum C_{lit} values obtained using NLCD 2001 are shown in Figure 2.2 (top) and summary statistics of these C_{lit} values are listed in Table 2.2. Values of watershed-minimum C_{lit} ranged from 0.13 to 0.60, with median and mean values of 0.41 and 0.38, respectively (Table 2.2). Values of watershed-maximum C_{lit} ranged from 0.38 to 0.68, with median and mean values of 0.58 and 0.55, respectively (Table 2.2). The differences of watershed-maximum and watershed-minimum C_{lit} values for 90 Texas watersheds ranged from 0.04 to 0.34, with median and mean differences of 0.14 and 0.17, respectively. ASCE and WPCF (1960) and design manuals (e.g., TxDOT 2002) and textbooks (e.g., Viessman and Lewis 2003) give a range of C values (not a single value) for different land use types, and the range of published C values for the same land use is from 0.04 to 0.3, same variations of C_{lit} for 90

Texas watersheds. This indicates uncertainty and variation of peak discharge estimation using the rational method.

The amount of developed land in a watershed is a key factor governing the runoff from the watershed. To study the relation of the composite runoff coefficients to the development factor of the watersheds, statistical summaries of C_{lit} (Table 2.3) from NLCD 2001 data were obtained separately for the 90 watersheds, which were classified as developed or undeveloped (Roussel et al. 2005). The corresponding cumulative frequency distributions are shown in Figure 2.2 (bottom). The median value of C_{lit} (watershed-average) for undeveloped watersheds is 0.37 and for the developed watersheds is 0.54. The average values of C_{lit} for undeveloped and developed watersheds are 0.39 and 0.54, respectively (Table 2.3). C_{lit} values of the developed watersheds are distinctly greater than those from undeveloped watersheds (p-value < 0.0001 from the pooled t-test), as shown in Figure 2.2 (bottom); the combination of LULC data and published C_{lit} values provides representative estimates of C_{lit} to reflect land-use development in a watershed.

2.5 Estimation of the Back-Computed Volumetric Runoff Coefficients (C_{vbc}) Using Observed Rainfall-Runoff Data

The concept of a rainfall-runoff event volumetric runoff coefficient (C_v) in hydrology dates to the beginning of the 20th century. An example is Sherman (1932), who used the percent of rainfall when he introduced the unit-hydrograph method. C_v is defined as the portion of rainfall that becomes runoff during an event (Merz et al. 2006). Estimates of C_v from an individual event are usually determined by three steps: (1)

separation into single events, (2) separation of observed streamflow into base flow and direct runoff, and (3) estimation of event C_v as the ratio of direct flow or runoff volume to event rainfall volume (Merz et al. 2006). C_v is based on the integrated response of the watershed, that is, the transformation of rainfall volume to runoff volume.

French et al. (1974) evaluated C_v for several rural catchments in New South Wales; Calomino et al. (1997) computed C_v for 66 events for a urban watershed (91.5% impervious area and total drainage area of 0.019 km^2 (1.89 ha)). For the urbanized watershed studied by Calomino et al. (1997), event C_v ranged from 0.31 to 0.88, and C_v was strongly correlated to the total rainfall depth (P): $C_v = 0.57\ P^{0.042}$ ($R^2 = 0.96$) (Calomino et al. 1997). The Water Planning Division of the United States Environmental Protection Agency (USEPA) operated the National Urban Runoff Program (NURP). This program had 20 projects throughout the United States to study pollutants from 76 urban watersheds, with drainage area ranging from 0.004 to 115 km^2 (USEPA 1983). NURP researchers collected rainfall and runoff data from these watersheds, with the number of events ranging from 5 to 121. A runoff coefficient, R_v (USEPA 1983), defined as the ratio of runoff volume to rainfall volume, was determined for each of the NURP-monitored storm events. The median value of the runoff coefficients, the coefficient of variation, and the percent impervious area were reported for all watersheds used in the study (USEPA 1983).

In this study, estimates of the volumetric runoff coefficient are called the back-computed volumetric runoff coefficient (C_{vbc}), and an individual-event C_{vbc} was obtained for *kth* storm event by the ratio of the total runoff depth, R_k (mm or in.), to the total rainfall depth, P_k (mm or in.), by:

$$C_{vbc}^{k} = \frac{total\ event\ runoff\ ,\ R_k}{total\ rainall\ for\ the\ event, P_k} \tag{2.3}$$

The study database comprised 1,600 rainfall-runoff events with observed rainfall and runoff data collected from 90 watersheds in Texas (Fang et al. 2007). Therefore, 1,600 event runoff coefficients C_{vbc} were obtained using equation (2.3). Event C_{vbc} ranged from near 0.0 to 1.0, covering the range of possible values. The cumulative distributions of C_{vbc} are presented in Figure 2.3 and summary statistics are listed in Table 2.4. For the 90 study watersheds in Texas, no substantial relation between rainfall depth and C_{vbc} was detected (Pearson's correlation coefficient $r = 0.2$ at the 0.1 percent level of significance because of p-value less than 0.0001). For example, for 19 events with total rainfall depth less than 12.7 mm (0.5 in.), computed C_{vbc} ranged from 0.050 to 0.844. For 253 events with total rainfall depth between 76.2 mm (3 in.) and 101.6 mm (4 in.), the computed C_{vbc} ranged from 0.006 to 0.982. Based on review of Figure 2.3, C_{vbc} is less than 0.1 for 13 percent of events. Furthermore, C_{vbc} exceeds 0.9 for one percent of events. The regression relation between the runoff coefficient C_{vbc} and the total runoff depth (R) was $C_{vbc} = 0.374\ R^{0.699}$. The regression explained about 76 percent of the variance between runoff depth and runoff coefficient and the regression coefficients were statistically significant at the 0.1 percent level of significance (p-value less than 0.0001).

C_{vbc} values calculated for all events in the same watershed varied from one event to another, e.g., depending on antecedent moisture condition before a rainfall event. Statistical parameters of the range of C_{vbc} values as the difference between maximum and minimum C_{vbc} values calculated for all events in the same watershed are given in Table 2.4. The maximum and average values of the range of event C_{vbc} in the same watershed is 0.97 and 0.52 (Table 2.4) for 1,600 rainfall-runoff events in 90 Texas watersheds, respectively. This finding is supported by previous studies by French et al. (1974) and USEPA (1983). Variations of event C_{vbc} in the same watershed determined from observed rainfall and runoff data are much larger than ranges of published C values for the same land use type.

Watershed-average and median values of C_{vbc} were calculated from C_{vbc} values for all rainfall-runoff events observed in the same watershed and developed for 83 of the 90-watershed dataset in Texas. Of the 90 watersheds in Texas, 7 were excluded; less than 4 rainfall-runoff events were available for analysis in each of these 7 watersheds. Computed watershed-average C_{vbc} ranged from about 0.1 to 0.67 and from about 0.06 to 0.76 for the watershed-median C_{vbc} (Table 2.4). These values are similar to values of C_{lit} estimated from LULC data, which ranged from 0.29 to 0.68 (Figure 2.2 and Table 2.2). About 80% of the C_{vbc}-median and -average values were less than 0.5. Watershed-median R_v ranged from 0.02 to 0.93 for 76 watersheds studied in the National Urban Runoff Program (USEPA 1983). The average values of the watershed-average and the watershed-median C_{vbc} are approximately the same, 0.33 and 0.31, respectively (Table 2.4). As shown in Figure 2.3, the cumulative frequency distributions of the watershed-average and watershed-median of C_{vbc} values are similar and the maximum absolute difference of watershed-average and median C_{vbc} is less than 0.10.

The developed and undeveloped watershed classifications (Roussel et al. 2005) were used to sort the watershed-average C_{vbc} values for additional statistical analysis. The results are listed in Table 2.5 and cumulative distributions of C_{vbc} are shown in Figure 2.3. The cumulative distributions are distinctly different: developed watersheds have greater C_{vbc} (watershed-average) in comparison to undeveloped watersheds (p-value <

0.0001 from the pooled *t*-test). The median values of the watershed-average C_{vbc} for undeveloped and developed watersheds are 0.19 and 0.37, respectively (Table 2.5).

For this study, the percentage of impervious area (*IMP*) was computed using 1992 NLCD. Of the 90 study watersheds, 45 have percent impervious area greater than 15 percent. The watershed-median runoff coefficients C_{vbc} and R_v versus percent impervious area for the 45 developed watersheds in Texas and the 60 watersheds from NURP are shown in Figure 2.4 (top). For 76 watersheds among those studied in NURP (USEPA 1983), two separate graphs of watershed-median runoff coefficient versus percent impervious area were developed and reported by USEPA (1983): one graph is for the 60 watersheds and another is for 16 watersheds.

"The separate grouping is based on the fact that the relationship for these sites (16 watersheds) is internally consistent and significantly different (much lower) than the bulk of the project results" (USEPA 1983 p. 6-60).

Polynomial regression lines were fit to the 60 NURP watershed data and to the combined watershed data for the combined group of 60 NURP and 45 Texas watersheds (watershed-median C_{vbc}). The regression lines are displayed in Figure 2.4 (top). Coefficients of determination R^2 for the two datasets are 0.79 and 0.57, respectively.

The regression equation obtained from the combined 60 NURP watershed data and the 45 Texas watershed data (watershed-median C_{vbc}) is

$$C_v = 1.843\,IMP^3 - 2.275\,IMP^2 + 1.289\,IMP + 0.036, \tag{2.4}$$

where, C_v = volumetric runoff coefficient and *IMP* = percent impervious area expressed as a fraction (50% = 0.5) of the watershed area. Urbanization alters the land surface and increases *IMP*. Although other watershed parameters, e.g., basin development factor

(Sauer et al 1983), can be used to quantify the degree of urbanization, *IMP* was used in this study to correlate it to C_v because Kuichling (1889) concluded that runoff coefficient for any given watershed he studied is nearly equal to the percentage of impervious surface within the watershed.

For comparison, Urbonas et al. (1989) used watershed-median runoff coefficients from the group of 60 NURP watersheds and several runoff coefficients developed for watersheds in the Denver area to develop a polynomial regression equation between runoff coefficient and percent impervious area. The Urbonas et al. equation (not repeated here) currently (2010) is used by the Denver Urban Drainage and Flood Control District in its Drainage Criteria Manual (from http://www.udfcd.org/downloads/down_critmanual.htm, accessed on January 10, 2010) to determine C for hydrologic soil group (HSG) Types C and D. The curve for the Urbonas et al. equation also is shown in Figure 2.4 (top). Although the regression parameters differ between the three regressions shown, the three regression curves are similar and have a maximum absolute difference of C_v less than 0.1.

Values of watershed-median C_{vbc} for the 45 Texas watersheds are generally consistent with those from the 60 NURP watershed data (Figure 2.4). Standard deviations from the watershed-average C_{vbc} were calculated for Texas watersheds and shown in Figure 2.4 (bottom) as solid circles with thick error bars. Standard deviations and coefficients of variation from watershed-averages C_{vbc} ranged from 0.04 to 0.30 (Table 2.4) and 0.15 to 1.34 for 83 Texas watersheds, respectively.

For the NURP data, watershed-median values and the coefficients of variation were reported (USEPA 1983), but the watershed-average runoff coefficients (R_v) were not reported. In order to examine the variation of runoff coefficient for NURP data, reported watershed-median values were used as watershed-averages to estimate the standard deviations from reported coefficient of variations, and the statistical distribution parameters of estimated standard deviations for NURP watersheds is listed in Table 2.4. Estimated maximum standard deviation from NURP watershed data is greater than 1.0, which is impossible if R_v ranged from 0.0 to 1.0, possibly because watershed-median R_v were used. That NURP watershed has watershed-median R_v of 0.17 and the coefficient of variation of 6.64, which is much larger than 1.34, the maximum coefficient of variation for 83 Texas watersheds. There are 15 NURP watersheds having estimated standard deviations greater than 0.3, the maximum standard deviation for 83 Texas watersheds (Table 2.4). The median standard deviations are approximately equal for both datasets (Table 2.4). The NURP data (median R_v for the 60 watersheds) plus and minus estimated one standard deviation are shown in Figure 2.4 (bottom) as open squares with wide error bars. Standard deviations from watershed-average C_{vbc} for the 45 Texas watersheds are consistently less than those from the 60 NURP watersheds (Figure 2.4). The NURP data cover a greater range of percent impervious area for watersheds (Figure 2.4) that are useful to develop the regression equation (2.4), which make the regression applicable to a wider range of watersheds.

2.6 Estimation of Volumetric Runoff Coefficients from the Rank-Ordered Pairs of Observed Rainfall and Runoff Depths

Schaake et al. (1967) examined the rational method using observed rainfall and runoff data collected from 20 gaged urban watersheds in Baltimore, MD. The size of watershed drainage area used by Schaake et al. was 0.6 km^2 (150 acres) or smaller. Schaake et al. (1967) used a frequency-matching approach to prepare their data for analysis. The frequency-matching approach was independently sorting observed rainfall intensity (average intensity over the watershed lag time) and peak runoff rate before computing the runoff coefficient using the rational method. That is, the rainfall intensity and peak runoff rate are paired on the rank order and not the event order.

Schaake et al. (1967) concluded that the frequency of occurrence of the computed design peak runoff rate is the same as the frequency of the rainfall intensity selected by the designer. Schaake et al. (1967) developed a regression equation to relate rate-based C (determine from peak discharge and rainfall intensity) to the imperviousness of the watershed and the main channel slope. Hjelmfelt (1980) and Hawkins (1993) used a similar frequency matching procedure as Schaake et al. (1967), except they used rank-ordered rainfall and runoff depths for computing actual curve numbers from historical rainfall-runoff events.

For each of the 90 Texas watersheds in this study, the total rainfall depth and the total runoff depth were ranked independently from greatest to least. As an example, the rank-ordered pairs of total rainfall depth (mm) and total runoff (mm) for 13 events at USGS gage station 08042650 (North Trinity Basin, TX) are presented in Figure 2.5 (top panel). The volumetric runoff coefficient C_v was computed from the rank-ordered pairs of total runoff and rainfall depths using:

$$C_{vrj} = \frac{R_j}{P_j}, \tag{2.5}$$

where, C_{vrj} is the C_v corresponding to the total runoff depth R_j and the total rainfall depth P_j of the j^{th} order of rainfall-runoff pairs (subscript "r" for C_{vrj} stands for rank-ordered).

A plot of runoff coefficient (C_{vrj}) versus the total rainfall depth was prepared for each watershed. For example, the plot for USGS gage station 08042650 is presented in Figure 2.5 (bottom panel). For most of the study watersheds, C_{vrj} increases until acquiring an approximate constant value. This constant value was considered as representative C_{vr} for the watershed; for example, watershed representative runoff coefficient C_{vr} = 0.17 for North Trinity Basin watershed associated with USGS gage station 08042650 (Figure 2.5). In addition to applying Hawkins's procedure (Hawkins 1993), i.e., asymptotic determination of C_{vr} from C_{vrj} versus rainfall depth, watershed C_{vr} can be estimated from the slope of the regression line obtained from the plots of the rank-ordered total runoff depth versus the rank-ordered total rainfall depth as shown in the top panel of Figure 2.5. For example, for USGS station 08042650, the regression equation developed from rank-ordered runoff and rainfall data is "Total-runoff (mm) = 0.167 × Total-rainfall-depth (mm)". Therefore, the watershed representative C_{vr} is 0.167 and equal to the slope of the regression equation. For most of the study watersheds, C_{vr} obtained from both procedures have approximately the same value. Statistical distribution parameters of C_{vr} for 83 Texas watersheds (7 out of 90 watersheds were excluded because of the too few rainfall-event data available) are listed in Table 2.6. The mean and median values of C_{vr} are the same and equal to 0.40. The values of C_{vr} range from 0.10 to 0.78. The cumulative distributions of the runoff coefficients C_{vr} and C_{lit} are shown in Figure 2.6 (top). The median value of the absolute differences $|C_{vr} - C_{lit}|$ is 0.14 (Table 2.6).

Examining the cumulative frequency distributions of C_{lit} and C_{vr} for the 90 study watersheds (Figure 2.6 top), about 80 percent of the C_{lit} values exceed the C_{vr} values.

Watershed-representative runoff coefficients C_{vr} were grouped into two categories: those from developed and those from undeveloped watersheds (Roussel et al. 2005). The statistical summary of C_{vr} for the two groups is listed in Table 2.7 and the corresponding cumulative frequency distributions are presented in Figure 2.6 (bottom). The median runoff coefficient C_{vr} from undeveloped watersheds is 0.24, and the median value from developed watersheds is 0.48. Based on this observation, C_{vr} derived from rank-ordered rainfall-runoff data reflects effects of watershed development, specifically the increase of percent impervious area (Figure 2.7). A statistical summary of the absolute differences $|C_{vr} - C_{vbc}|$ and $|C_{vr} - C_{lit}|$ for the 45 developed watersheds in Texas is listed in Table 2.7. Small average and median values of absolute differences $|C_{vr} - C_{vbc}|$ indicate that C_{vr} is similar to C_{vbc} because both were derived from observed rainfall-runoff data. Average and median values of absolute differences $|C_{vr} - C_{lit}|$ are greater than those of $|C_{vr} - C_{vbc}|$ (Table 2.7), and indicate that C_{vr} derived from rainfall-runoff data differs from C_{lit} derived from land-use data and published runoff coefficients (see Figures 2.2 and 2.6).

The watershed representative C_{vr} and watershed-median C_{vbc} from 45 developed Texas watersheds and runoff coefficient R_v from 60 NURP watersheds were plotted against percent impervious area (Figure 2.7). Results from these three datasets are consistent – overall increasing volumetric runoff coefficient with the increase of percent impervious area or degree of development. A polynomial regression line was fitted to combined data from 60 NURP watersheds and watershed representative C_{vr} for the 45

Texas developed watersheds (Figure 2.7). R^2 for the regression equation (2.6) = 0.57, which is the same as R^2 for equation (2.4).

$$C_v = 1.469\, IMP^3 - 1.940\, IMP^2 + 1.315\, IMP + 0.043 \tag{2.6}$$

The regression equations (2.4) and (2.6) were combined by averaging their coefficients to get a single equation for general application in Texas watersheds similar to 45 developed Texas watersheds.

$$C_v = 1.66 IMP^3 - 2.11 IMP^2 + 1.30 IMP + 0.04 \tag{2.7}$$

Equation (2.7) can be used to estimate C_v for developed (urban) watersheds based on impervious cover. Equation (2.7) is plotted on Figure 2.7, which also includes a curve for Equation (2.6) and data points of C_{vbc}, C_{vr}, and R_v values versus percent impervious area (*IMP*). Figure 2.7 and Equation (2.7) indicate that C_v is not equal to 1.0 when IMP = 100%. This is because R_v estimated in the NURP study is for watersheds greater than 0.004 km^2 (1 acre) and C_v estimated in this study for watersheds greater than 0.8 km^2 (200 acres); therefore, Equation (2.7) does not apply to very small 100% impervious catchment such as a small parking lot. Further study is needed to correlate runoff coefficients for undeveloped watersheds (Figure 2.6) to soil types and other watersheds characteristics.

2.7 Discussion

Volumetric runoff coefficients, watershed-average C_{vbc} and C_{vr} for 83 Texas watersheds, watershed-average C_{lit} for 90 Texas watersheds, and watershed-median R_v for NURP 60 watersheds (USEPA 1983), were plotted against drainage area A in km^2 (Figure 2.8). Pearson's correlation coefficients between C_{vbc}, C_{vr}, C_{lit}, R_v and A (km^2) are -0.20, -0.12, -0.27, and -0.26 with p-values of 0.060, 0.256, 0.009, and 0.044, respectively. Therefore, at the 90% confidence level, C_{vbc}, C_{lit}, and R_v has no substantial relation with area (Figure 2.8). C_{vr} has no substantial relation with area only at the 70% confidence level. Above statistical analyses between volumetric C values and drainage area indicated that there is no demonstrable relation in volumetric C with drainage area as Young et al. (2009) reported. This finding supports the conclusion by ASCE and WPCF (1960), Pilgrim and Cordery (1993), and Young et al. (2009) that the rational method may be applied to much larger drainage areas than typically assumed in some design manuals, as long as the watershed is unregulated (Young et al. 2009). The authors do not advocate specifying specific limit that should be imposed on drainage area for application of the rational method. It is the duty and responsibility of the end-user of the rational method to apply appropriate engineering judgment and experience in developing designs.

The authors explicitly are not advocating application of the rational method for larger watersheds because the steady-state assumption of the rational method for design purposes is questionable. However, extensive data analysis (Asquith 2010) suggests that inherent relations between runoff coefficient and drainage area are insubstantial if time of concentration of a watershed is reasonably estimated for determining rainfall intensity. The authors support the recommendation of ASCE and WPCF in 1960 “Development of data for application of hydrograph methods is usually warranted on larger areas” (ASCE and WPCF 1960, p. 32).

The authors explicitly recognize that volumetric runoff coefficients might not have direct applicability in use of the rational method for engineering design purposes. Therefore, the authors did not apply the rational method and use volumetric runoff coefficients (C_{vbc} and C_{vr}) to predict peak discharges for 1,600 events and compare predicted and observed peak discharges. To predict peak flows using volumetric runoff coefficients is inconsistent with the assertion that rate-based values for the runoff coefficient be used. In a subsequent paper, the authors determined rate-based rational runoff coefficients for these 90 Texas watersheds, and applied the rational method with rate-based runoff coefficients to predict peak discharges and compared predicted and observed peak discharges.

Volumetric runoff coefficients estimated from observed rainfall and runoff data for 83 Texas watersheds were plotted against literature-based C_{lit} (watershed-average) determined for the same watersheds (Figure 2.9). Regression equations between watershed-average C_{vbc}, C_{vr} and C_{lit} were developed and shown in Figure 2.9, with Pearson's correlation coefficients $r = 0.36$ and 0.26 at the 95% confidence level (p-values of 0.0007 and 0.01), respectively. Therefore, regression analyses indicated volumetric runoff coefficients determined from rainfall and runoff data are weakly correlated to literature-based C_{lit}.

2.8 Summary

Volumetric runoff coefficients were estimated for 90 Texas watersheds using three different methods. The first method is estimation of literature-based runoff coefficients (C_{lit}) using published values and GIS analysis of LULC classes to construct areally weighted values over a watershed. C_{lit} was obtained independently from 1992 and 2001 NLCD and using minimum, average, and maximum published C values for different LULC. No substantial difference in the results of watershed-average C_{lit} was obtained using the 1992 or 2001 versions of the land-use data. For the study watersheds, watershed-average C_{lit} ranged from 0.29 to 0.68 with median and average values about 0.5. The differences of watershed-maximum and watershed-minimum C_{lit} values for 90 Texas watersheds ranged from 0.04 to 0.34, with median and mean differences of 0.14 and 0.17, respectively. When C_{lit} (watershed-average) is grouped into developed and undeveloped watersheds, the range of C_{lit} for developed watersheds was between 0.37 and 0.63, with a median value of 0.54. The median value of C_{lit} for developed watersheds exceeds that for undeveloped watersheds. This result stems from the fact that published runoff coefficients, even though they were not developed from observed rainfall-runoff measurements and instead resulted from a survey on engineering practices in 1950s, reflect the physical meanings of the original runoff coefficients introduced by Kuichling in 1889 — the runoff coefficient is related to the percent impervious area within the watershed. Therefore, published runoff coefficients remain useful for engineering design of drainage systems.

The second method is based on use of back-computed volumetric runoff coefficients (C_{vbc}) from observed rainfall-runoff measurements of more than 1,600 events by the ratio of total runoff depth to total rainfall depth for individual storm event. Event volumetric runoff coefficients cover all possible values from 0.0 to 1.0 with 10% of all values less than 0.08 and 10% of all values greater than 0.63 (Figure 2.3). The maximum and average values of the range of event C_{vbc} in the same watershed is 0.97 and 0.52 (Table 2.4) for 1,600 rainfall-runoff events in 90 Texas watersheds, respectively. Watershed-average and watershed-median values of C_{vbc} and estimates of standard deviations were extracted. The distributions of the watershed-average and watershed-median C_{vbc} are similar. Watershed-median values of C_{vbc} ranged from 0.06 to 0.76 with an average of 0.31. Watershed-median values of C_{vbc} for 45 developed watersheds in Texas with percent imperviousness greater than 15% are consistent with median values of runoff coefficient R_v reported for 60 NURP watersheds by the USEPA.

The third method involved the computation of runoff coefficients by the frequency matching procedures of observed total rainfall-runoff depths from a watershed. A single watershed-specific value of the runoff coefficient (C_{vr}) was developed from the plot of rank-ordered runoff coefficients versus rainfall depths. The values of C_{vr} ranged from 0.10 to 0.78 with the median value 0.40. The C_{vr} values for the developed watersheds are consistently higher than those for the undeveloped watersheds. The distribution of C_{vr} is different from that of C_{lit} with about 80 percent of C_{lit} value greater than C_{vr} value. This result might indicate that literature-based runoff coefficients overestimate peak discharge for drainage design when used with the rational method.

Runoff coefficients derived from observed rainfall and runoff data in 90 Texas watersheds in this study are volumetric based (ratio of total runoff and rainfall depth) and are useful in transforming rainfall depth to runoff depth such as is done in the curve number method (SCS 1963) and for watershed rainfall-runoff modeling, e.g., the fractional loss model (McCuen 1998, p. 493). Current runoff coefficients given in textbooks and design manuals are neither volumetric nor rate-based (determined from peak discharge and rainfall intensity) because they were not derived from observed data but are used for the rate-based rational method.

Two regression equations (2.4) and (2.6) were developed using watershed-median C_{vbc} and watershed-representative C_{vr} data combined with median runoff coefficients R_v from 60 NURP watersheds. Coefficient of determination R^2 for both equations are 0.57

and these equations were combined into a single equation (2.7) which can be used to estimate volumetric runoff coefficients for developed urban watersheds that are similar to the 45 developed watersheds in Texas. The published limits on drainage area for application of the rational method seem to be arbitrary. Results from this study supports the conclusion by ASCE and WPCF (1960), Pilgrim and Cordery (1993), and Young et al. (2009) that the rational method may be applied to much larger drainage areas than typically assumed in some design manuals.

2.9 Acknowledgments

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2.10 Notation

The following symbols are used in this paper:

A = watershed drainage area in hectares or acres;

A_i = sub area for *ith* land cover classes in the watershed;

C = runoff coefficient;

C_i = literature-based runoff coefficient for *ith* land-cover class (Table 2.1);

C_v = volumetric runoff coefficient, portion of rainfall that becomes runoff, determined from regression equations;

C_{vbc} = watershed average or median back-computed volumetric runoff coefficient;

C^k_{vbc} = back-computed volumetric runoff coefficient for the *kth* event;

C_{lit} = literature–based runoff coefficient developed from landuse data;

C_{vrj} = runoff coefficient estimated from the ratio of *jth* rank-ordered runoff and rainfall data pair;

C_{vr} = watershed representative runoff coefficient estimated from the distribution of ratios of rank-ordered runoff and rainfall;

I = average rainfall intensity (mm/hr or in./hr) with the duration equal to time of concentration;

IMP = percent of impervious area expressed as a fraction (50% = 0.5) for a watershed area;

m_o = the dimensional correction factor (1.008 in English units, 1/360 = 0.00278 in SI units);

no = total number of land cover classes in a watershed (Equation 2.2);

P_j = total rainfall depth of the j_{th} order of rank runoff data series;

P_k = total rainfall depth of the k_{th} event;

Q_p = peak discharge or runoff rate in m^3/s or ft^3/s;

R_j = total runoff depth of the j_{th} order of rank rainfall data series;

R_k = total runoff depth of the k_{th} event;

R_v = runoff coefficient as the ratio of runoff volume to rainfall volume determined by USEPA for the NURP data.

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Table 2.1 Runoff Coefficients (C) Selected for Various Land-Cover Classes from NLCD 2001.

NLCD classification	NLCD classification description	C	Land use or description in the source
21	Developed, Open Space	0.4[(1),(2)]	Residential: single family areas (0.3-0.5)
22	Developed , Low Intensity	0.55[(3)]	50 % of area impervious (0.55)
23	Developed, Medium Intensity	0.65[(3)]	70% of area impervious (0.65)
24	Developed , High Intensity	0.83[(2)]	Business: downtown areas (0.7-0.95)
31	Barren Land	0.3[(2),(7)]	Sand or sandy loam soil, 0-5% (0.15-0.25); black or loessial soil, 0-5% (0.18-0.3); heavy clay soils; shallow soils over bedrock: pasture (0.45)
41	Deciduous Forest	0.52[(4)]	Deciduous forest (Tennessee) (0.52)
42	Evergreen Forest	0.48[(5),(6)]	Forest (UK) (0.28-0.68); Forest (Germany) (0.33-0.59)
43	Mixed Forest	0.48[(5),(6)]	Forest (UK) (0.28-0.68); Forest (Germany) (0.33-0.59)
52	Shrub/Scrub	0.3[(7)]	Woodland, sandy and gravel soils (0.1); loam soils (0.3); heavy clay soils (0.4); shallow soil on rock (0.4)
71	Grassland/Herbaceous	0.22[(3)]	Pasture, grazing HSG A (0.1); HSG B (0.2); HSG C (0.25); HSG D (0.3)
81	Pasture/Hay	0.35[(7)]	Pasture, sandy and gravel soils (0.15); loam soils (0.35); heavy clay soils (0.45); shallow soil on rock (0.45)
82	Cultivated Crops	0.4[(7)]	Cultivated, sandy and gravel soils (0.2); loam soils (0.4); heavy clay soils (0.5); shallow soil on rock (0.5)

Sources: - [(1)] ASCE (1992), [(2)] TxDOT (2002), [(3)] Schwab and Frevert (1993), [(4)] Mulholland et al. (1990) , [(5)] Law (1956), [(6)] Hydrology (1976), [(7)] Dunne and Leopold (1978)

Note Numbers in parenthesis are ranges for runoff coefficients given in the source (literature).
HSG = hydrologic soil group

Table 2.2 Statistical Summary of Average C_{lit} Using NLCD 1992 and Watershed-Minimum, -Average and -Maximum C_{lit} Using NLCD 2001.

Statistical distribution parameters	Watershed-average [1] C_{lit} using NLCD 1992 (1)	C_{lit} using NLCD 2001			Absolute difference (1) – (3)
		Watershed-minimum [1] (2)	Watershed-average (3)	Watershed-maximum [1] (4)	
Minimum	0.32	0.13	0.29	0.38	0.00
Maximum	0.68	0.60	0.63	0.68	0.14
25% Quartile	0.40	0.24	0.38	0.48	0.02
Median	0.47	0.41	0.50	0.58	0.03
75% Quartile	0.52	0.50	0.55	0.60	0.05
Average	0.47	0.38	0.47	0.55	0.04
Standard deviation	0.09	0.14	0.10	0.09	0.03

[1] Watershed-average, -minimum, and –maximum C_{lit} values were derived using mean, minimum, and maximum C values for each LULC from literature (Table 2.1), respectively.

Table 2.3 Statistical Summary of Watershed-average C_{lit} Using NLCD 2001 for Developed and Undeveloped Watersheds.

	Undeveloped	Developed
Minimum	0.29	0.37
Maximum	0.59	0.63
25% Quartile	0.33	0.52
Median	0.37	0.54
75% Quartile	0.43	0.58
Average	0.39	0.54
Standard deviation	0.07	0.06

Table 2.4 Statistical Summary of C_{vbc} and R_v from NURP (USEPA 1983).

	C_{vbc} All events	Range of C_{vbc} [1]	Watershed-Median C_{vbc}	Watershed-average C_{vbc}	Standard deviation C_{vbc}	Standard deviation R_v [2]
Minimum	0.00	0.02	0.06	0.10	0.04	0.02
Maximum	0.99	0.97	0.76	0.67	0.30	1.13
25% Quartile	0.17	0.37	0.17	0.20	0.12	0.10
Median	0.29	0.53	0.30	0.31	0.16	0.16
75% Quartile	0.47	0.66	0.42	0.42	0.19	0.28
Average	0.33	0.52	0.31	0.33	0.16	0.21
Standard deviation	0.21	0.22	0.17	0.15	0.05	0.18

[1] Range of C_{vbc} is difference between maximum and minimum C_{vbc} values calculated for all events in the same watershed. [2] Standard deviations for R_v (USEPA 1983) were estimated from median values and coefficients of variation of R_v for 60 NURP watersheds.

Table 2.5 Statistical Summary of Watershed-average C_{vbc} for Developed and Undeveloped Watersheds.

	Undeveloped	Developed
Minimum	0.10	0.17
Maximum	0.56	0.67
25% Quartile	0.15	0.30
Median	0.19	0.37
75% Quartile	0.36	0.48
Average	0.24	0.39
Standard deviation	0.12	0.13

Table 2.6 Statistical Summary of C_{vr} and Absolute Difference (ABS) of C_{vr} with Watershed-average C_{lit} for 83 Texas Watersheds.

	C_{vr}	ABS (C_{vr} - C_{lit})
Minimum	0.10	0.01
Maximum	0.78	0.40
25% Quartile	0.24	0.07
Median	0.40	0.14
75% Quartile	0.52	0.24
Average	0.40	0.16
Standard deviation	0.18	0.11

Table 2.7 Statistical Summary of C_{vr} for Developed and Undeveloped Watersheds and Absolute Difference (ABS) of C_{vr} with Watershed-average C_{vbc} and C_{lit} for Developed Watersheds.

	Undeveloped	Developed	ABS (C_{vr} - C_{vbc})	ABS (C_{vr} - C_{lit})
Minimum	0.10	0.20	0.00	0.01
Maximum	0.70	0.74	0.45	0.38
25% Quartile	0.18	0.34	0.02	0.06
Median	0.24	0.48	0.04	0.12
75% Quartile	0.44	0.60	0.12	0.18
Average	0.31	0.46	0.08	0.14
Standard deviation	0.18	0.15	0.10	0.10

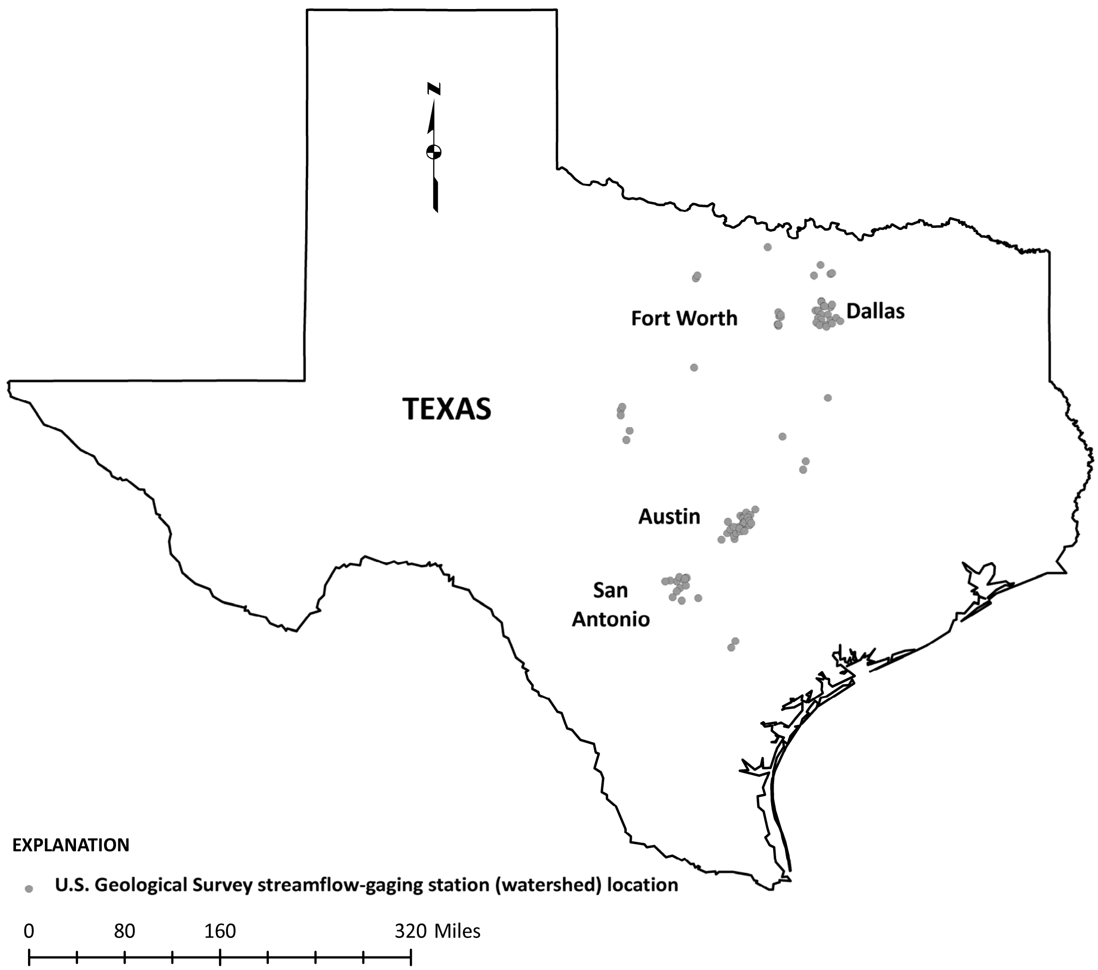

Fig. 2.1 Map showing the U.S. Geological Survey streamflow-gaging stations (dots) associated with the watershed locations in Texas.

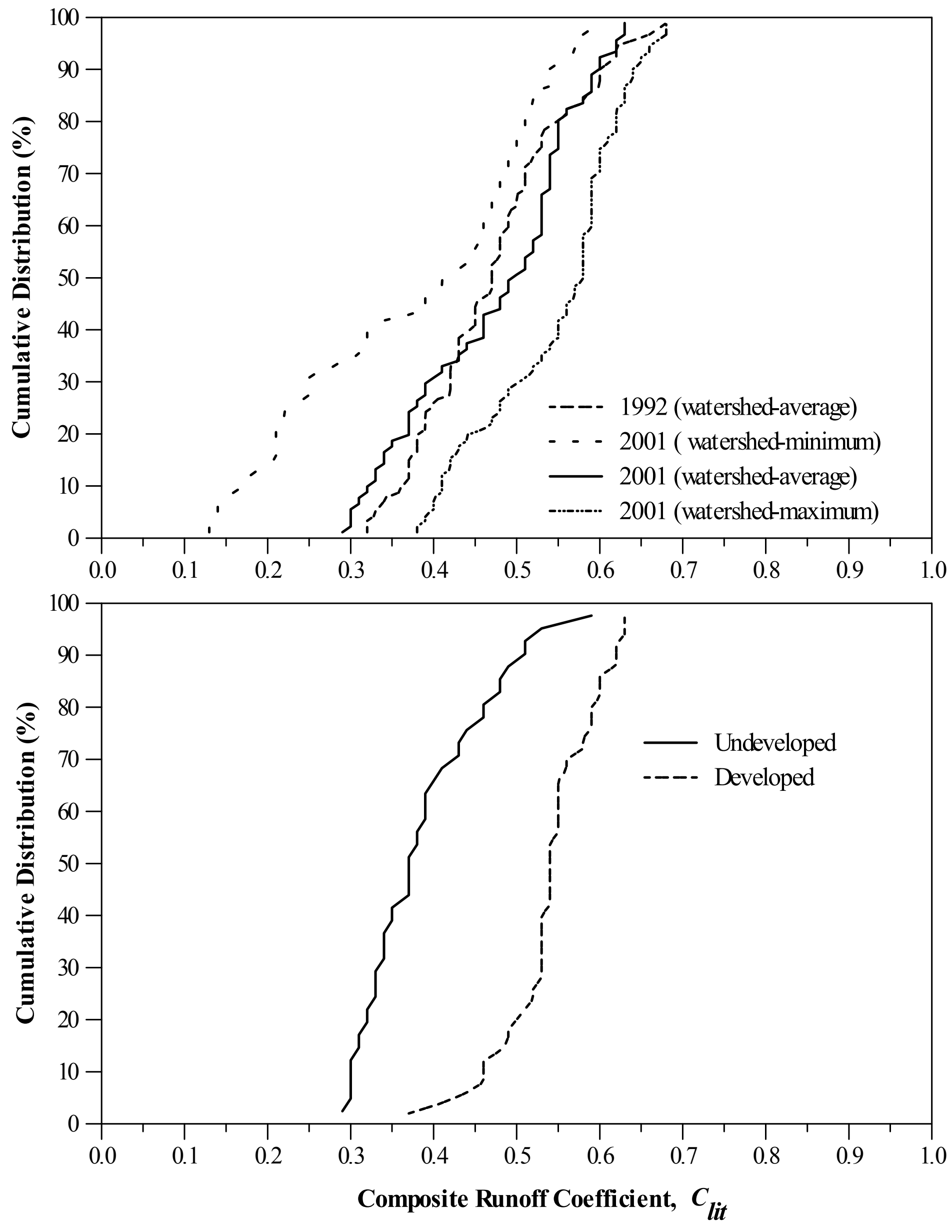

Fig. 2.2 Cumulative distributions of C_{lit} obtained using NLCD 1992 and NLCD 2001 (top) and C_{lit} using NLCD 2001 for developed and undeveloped watersheds (bottom).

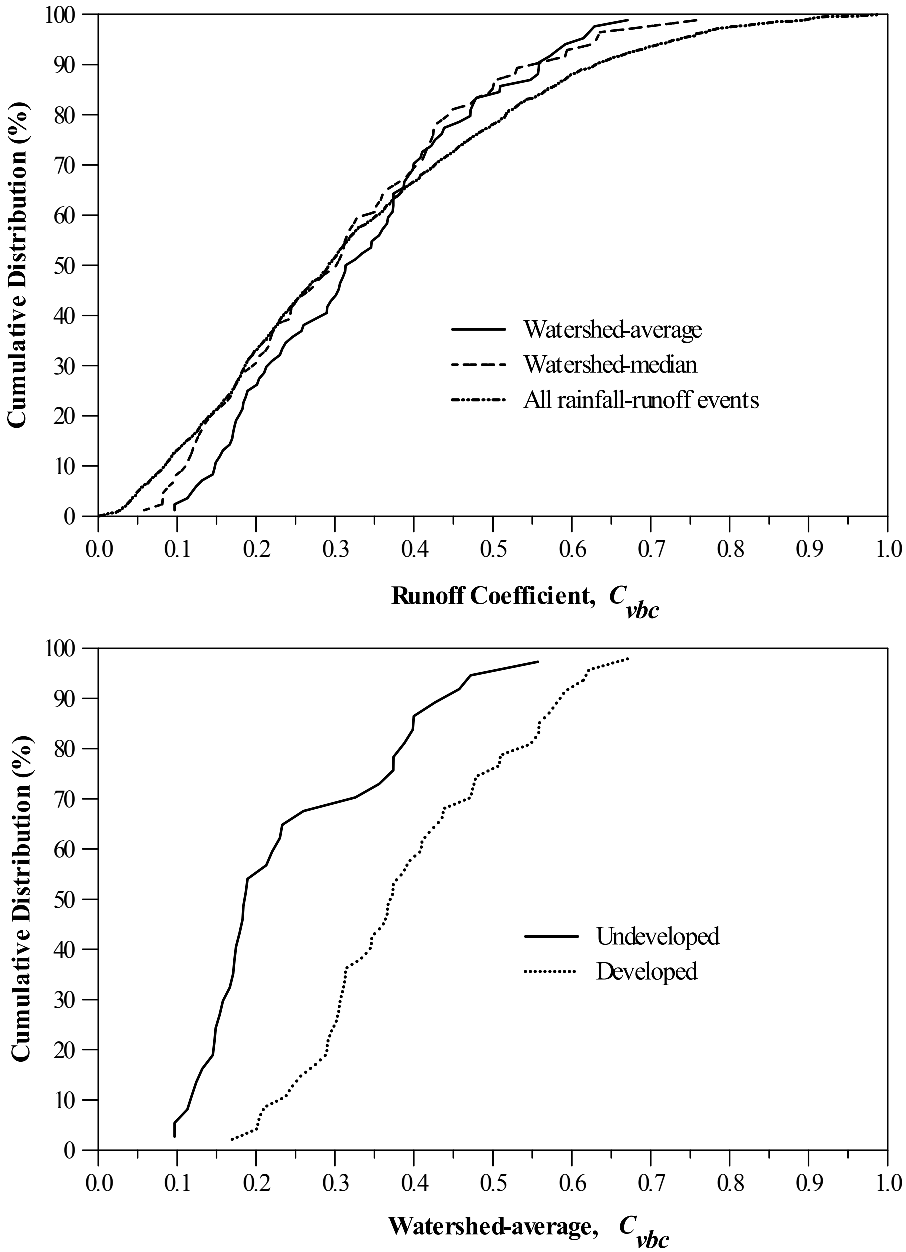

Fig 2.3. Cumulative distributions of C_{vbc} for watershed-average, watershed-median, and all rainfall-runoff events (top) and watershed-average C_{vbc} for developed and undeveloped watersheds (bottom).

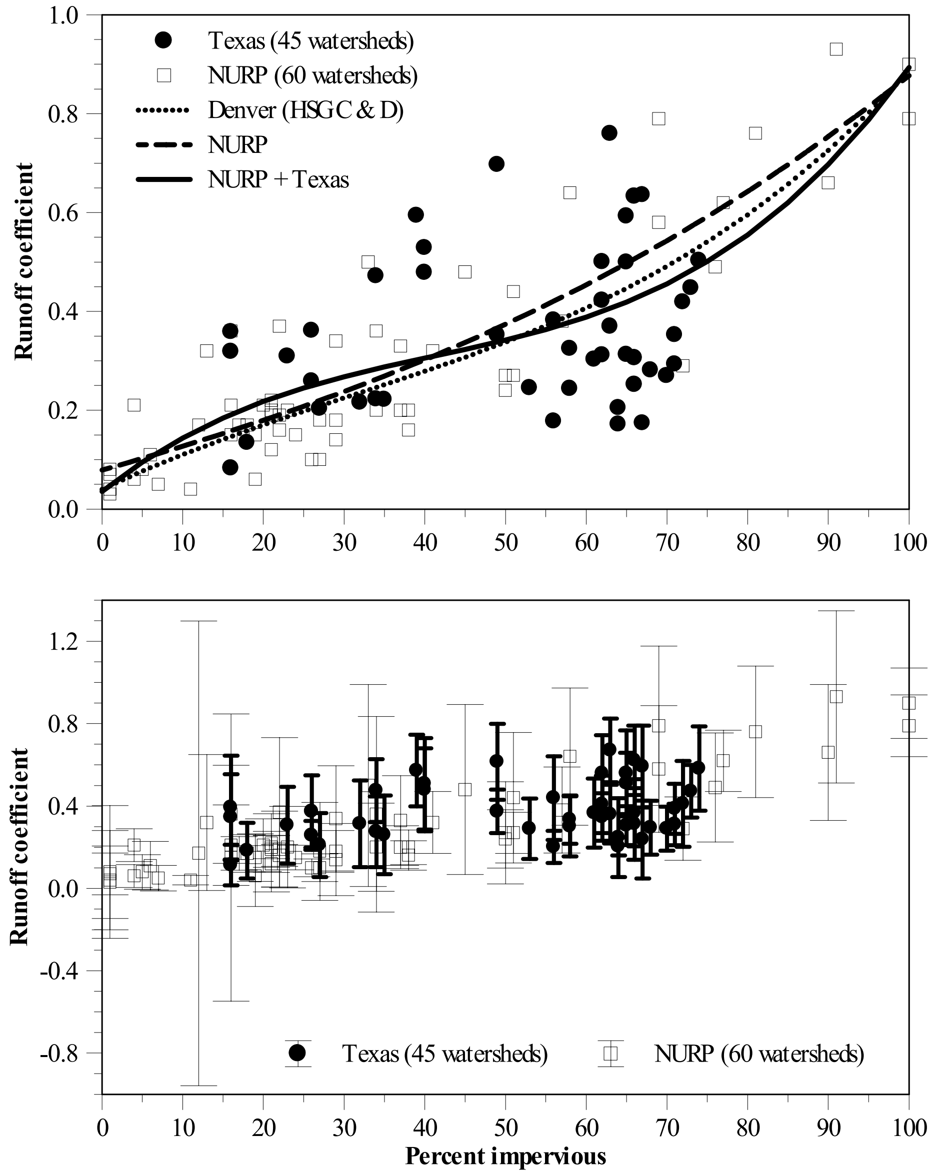

Fig. 2.4 Volumetric runoff coefficients from different studies versus percent impervious area including regression lines (top) and runoff coefficients with one standard deviations for 45 Texas watersheds and 60 NURP watersheds (bottom).

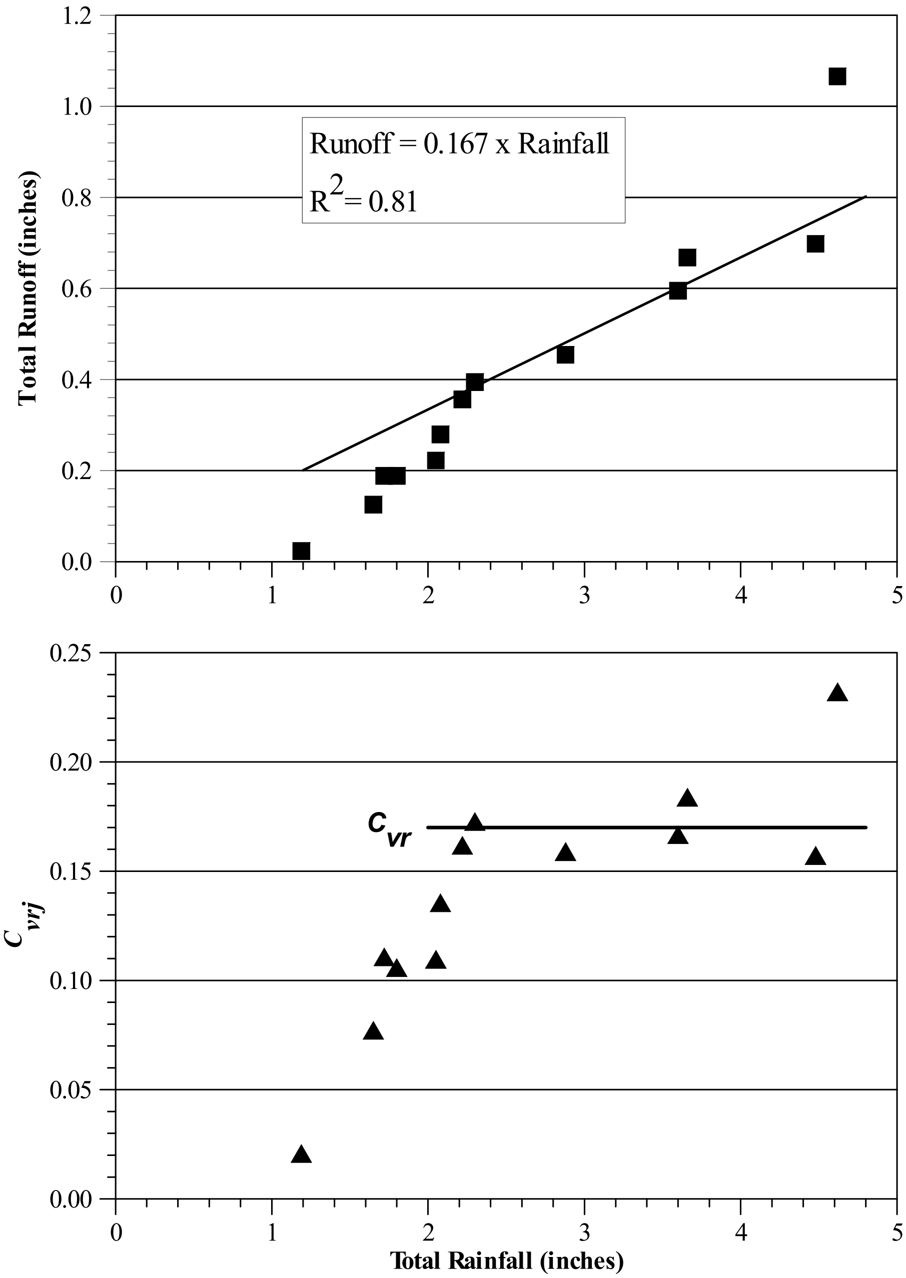

Fig. 2.5 The rank-ordered pairs of observed runoff and rainfall depths (top) and runoff coefficients derived from the rank-ordered pairs of observed runoff and rainfall depths versus total rainfall depths (bottom). All data presented are for the USGS gage station 08042650 (North Trinity Basin in Texas).

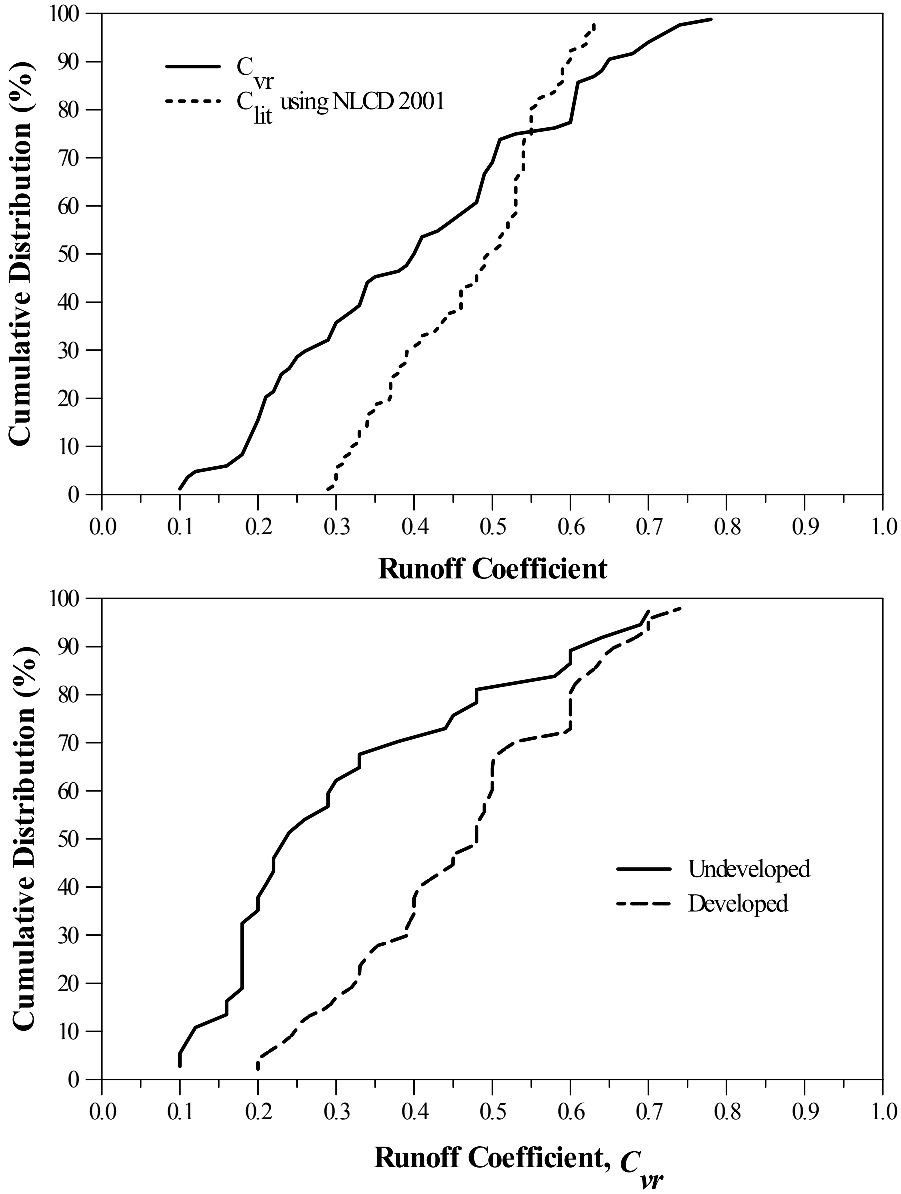

Fig. 2.6 Cumulative distributions of C_{vr} and C_{lit} (top) and C_{vr} for developed and undeveloped watersheds (bottom).

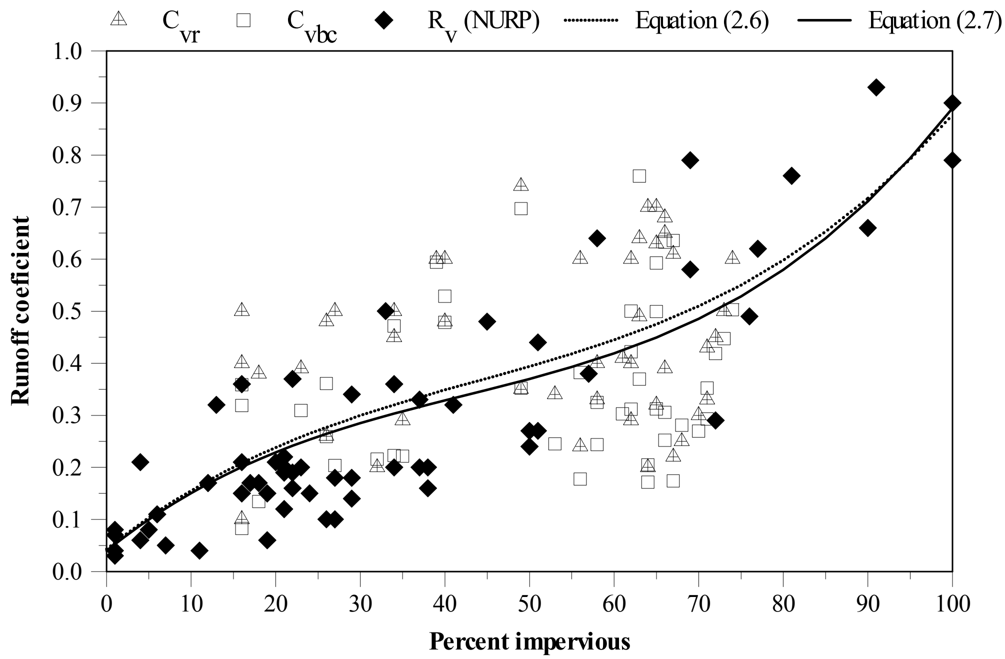

Fig. 2.7 Runoff coefficients C_{vr}, C_{vbc} (watershed-average), and runoff coefficients R_v from 60 NURP watersheds versus percent impervious area including lines for the regression equation (2.6) and the regression equation (2.7).

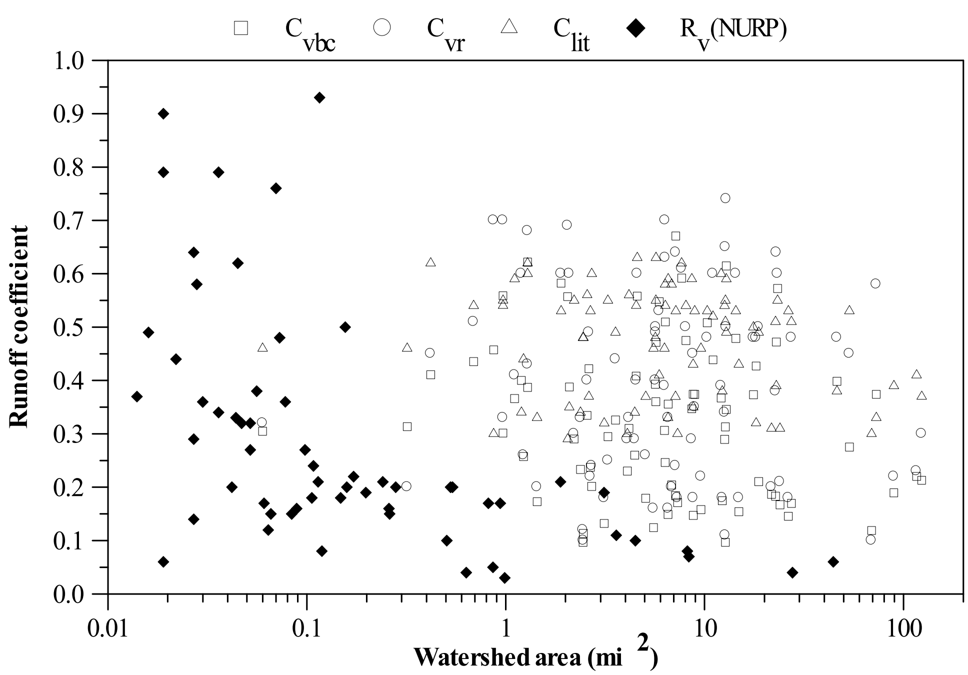

Fig. 2.8 Runoff coefficients C_{lit} (watershed-average), C_{vbc} (watershed-average), C_{vr}, and R_v plotted against watershed area (km^2).

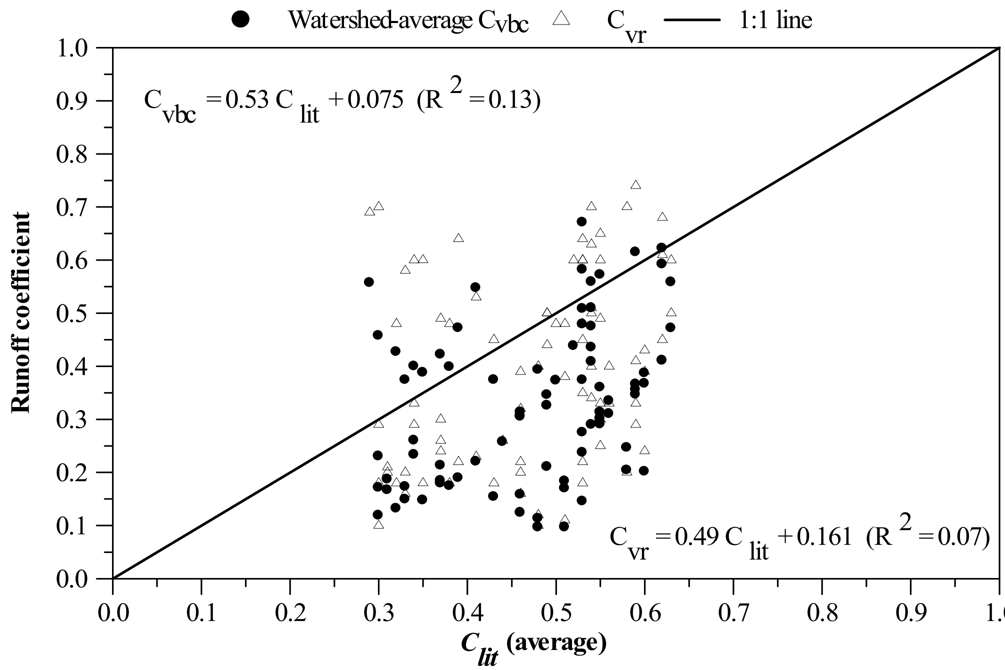

Fig. 2.9 Runoff coefficients C_{vbc} (watershed-average) and C_{vr} plotted against C_{lit} (watershed-average) for 83 Texas watersheds.

Chapter 3. Rate-based Estimation of the Runoff Coefficients for Selected Watersheds in Texas

3.1 Abstract

The runoff coefficient, C, of the rational method is an expression of rate proportionality between rainfall intensity and peak discharge. Values of C were derived for 80 developed and undeveloped watersheds in Texas using two distinct methods. First, the rate-based runoff coefficients, C_{rate}, were estimated for each of 1,500 rainfall-runoff events. Second, the frequency-matching approach was used to extract a runoff coefficient, C_r, for each watershed. Using the 80 Texas watersheds, comparison of the two methods shows that about 75 percent of literature-based runoff coefficients are greater than C_r and the watershed-median C_{rate}, but for developed watersheds with more impervious cover, literature-based runoff coefficients are less than C_r and C_{rate}. An equation applicable to many Texas watersheds is proposed to estimate C as a function of impervious area.

3.2 Introduction

The search for a reliable method for estimation of peak discharges for small and ungaged undeveloped (rural) watersheds has led to various engineering-design methods (French et al. 1974). These various methods often are applicable for developed (suburban to urban) watersheds (Chow et al. 1988). The rational method likely is the most often applied method used by hydraulic and drainage engineers to estimate design discharges for small watersheds. These design discharges are used to size a variety of drainage structures for small undeveloped and developed watersheds throughout the United States (Viessman and Lewis 2003).

The rational method (Kuichling 1889) computes the peak discharge, Q_p, (in m^3/s in SI units or ft^3/s in English units) by:

$$Qp = m_0 CIA \tag{3.1}$$

where C is the runoff coefficient (dimensionless), I is the rainfall intensity (mm/hr or in/hr) over a critical period of storm time (typically taken as the time of concentration, T_c), A is the drainage area (hectares or acres), and m_o is the dimensional correction factor (1/360 = 0.00278 in SI units, 1.008 in English units). Steady-state conditions are needed for the application of the rational method (French et. al 1974). From inspection of the equation, it is evident that C is an expression of rate proportionality between rainfall intensity and peak discharge (flow rate). The theoretical range of values for C is between 0 and 1. The typical "whole watershed" C values, that is, C values representing the integrated effects of various surfaces in the watershed and other watershed properties, are listed for different general land-use conditions in various design manuals and textbooks. Examples of textbooks that include tables of C values are Chow et al. (1988) and Viessman and Lewis (2003). Published C values, C_{lit}, were sourced from American Society of Civil Engineers (ASCE) and the Water Pollution Control Federation (WPCF) in 1960 (ASCE and WPCF 1960). The C_{lit} values were obtained from a response survey, which received "71 returns of an extensive questionnaire submitted to 380 public and private organizations throughout the United States." No justification based on analyses of observed rainfall intensity and peak discharge data for the C_{lit} values is apparent in ASCE and WPCF (1960)—a few analyses of observed relations between rainfall intensity and peak discharge were considered by Kuichling (1889). In short, the authors of this paper conclude that C_{lit} values appear to be heuristically determined, and therefore a comparison of rate-based C values derived from observed rainfall and runoff data to the C_{lit} values is made in this study.

Estimation of reliable values of C presents a substantial difficulty in the rational method and a major source of uncertainty in many small watershed projects (Pilgrim and Cordery 1993). Furthermore, the concept of "runoff coefficient" for a watershed is a term fraught with ambiguities. The volumetric runoff coefficient, C_v, is the ratio of total runoff to rainfall (Merz et al. 2006; Dhakal et al. 2012). Because C as an expression of rate proportionality (equation 3.1), such a coefficient is termed a rate-based runoff coefficient, C_{rate}. It is important to stress that although the C_{lit} found in ASCE and WPCF (1960) are intended for use in the rate-based rational method, C_{lit} appear not to be derived from any observed data.

When observed rainfall and runoff data are available, the C_{rate} is computed through the rational method (Pilgrim and Cordery 1993) as:

$$C_{rate} = \frac{Q_p}{m_0 I(t_{av}) A} \quad , \qquad (3.2)$$

where $I(t_{av})$ is the average rainfall intensity over a t_{av} time period, which is the rainfall intensity averaging time period (a critical time period) rather than the entire rainfall event duration (Kuichling 1889; Schaake et al. 1967). The t_{av} should be a period of time for a storm that contributes runoff that produces the observed Q_p. Kuichling (1889) argues against using the entire rainfall event duration, t_w, to obtain average rainfall intensity, I_w, because I_w is in general not appropriate, which results from rainfall durations of real storms often being greater than the characteristic time of small watersheds considered by Kuichling. Thus, a lasting contribution of Kuichling (1889) was the introduction of the concept of the time of concentration, T_c, of a watershed. This time also is termed a "critical storm duration" because this uses an average rainfall intensity that produces reliable peak discharge estimates. The T_c is influenced in part by drainage area, which is a major criterion used to assess the applicability of the rational method (Chow et al. 1988). In particular, TxDOT (2002) recommends the use of the rational method for watersheds with very small drainage areas of < 0.8 km^2 (200 acres).

Other investigators have reported C values derived from analysis of observed rainfall and runoff data for various watersheds throughout the world. Schaake et al. (1967) examined the rational method using experimental rainfall and runoff data collected from 20 small urban watersheds of < 0.6 km^2 (150 acres) in Baltimore, MD. Those authors used watershed lag time to compute average rainfall intensity and used a frequency-matching approach.

Hotchkiss and Provaznik (1995) estimated C_{rate} for 24 rural watersheds in south-central Nebraska using event-paired and frequency-matched data. Young et al. (2009) estimated C_{rate} for 72 rural watersheds in Kansas with drainage areas of < 78 km^2 (30 mi^2) for different return periods. The peak discharge for each return period was estimated using annual peak frequency analysis of the gaged peak discharges and rainfall intensity obtained from rainfall intensity-duration-frequency tables (Young et al. 2009).

In this study, two methods were used to estimate C_{rate} for 80 selected watersheds in Texas. Both methods rely on analysis of observed rainfall and runoff data. First, C_{rate} was estimated using equation (3.2); the $I(t_{av})$ was computed as the maximum intensity for a moving time window of duration T_c before and up to the time to peak, T_p. The T_c was derived for the study watersheds using the Kerby-Kirpich approach (Roussel et al. 2005; Fang et al. 2008). A total of about 1,500 rainfall-runoff events from 80 Texas watersheds were analyzed to determine event-specific, watershed-median, and watershed-mean C_{rate} values. Second, the frequency-matching approach (Schaake et al., 1967) was used to derive a representative C referred to as C_r for each of the 80 watersheds. The study also compares C values from the two different methods and those published in the literature. Finally, an equation of C as a function of the percentage of impervious area is proposed for the 80 watersheds.

3.3 Study Area and Rainfall-Runoff Database

Watershed data from a larger dataset accumulated by researchers from the U.S. Geological Survey (USGS) Texas Water Science Center, Texas Tech University, University of Houston, and Lamar University (Asquith et al. 2004) are used for this study. Ten watersheds out of about 90 represented by USGS streamflow-gaging stations in the source database (Asquith et al. 2004) are not used in this study because less than four rainfall and runoff events were recorded for each of these 10 watersheds. The locations of 80 USGS streamflow-gaging stations representing 80 watersheds in Texas are shown in Figure 3.1. Incidentally, these data also are used by Asquith and Roussel (2007), Cleveland et al. (2006), Fang et al. (2007, 2008), and Dhakal et al. (2012). The rainfall-runoff dataset consists of about 1,500 rainfall-runoff events which occurred between 1959 and 1986. The number of events available for each watershed varied from 4 to 50 events with median and mean values of 16 and 19 events, respectively. Values of rainfall depths for about 1,500 events ranged from 3.56 mm (0.14 in.) to 489.20 mm (19.26 in.), with median and mean values of 57.66 mm (2.27 in.) and 66.8 mm (2.63 in.), respectively.

The drainage area of study watersheds range from approximately 0.2–320 km^2 (0.1–123.6 mi^2); the median and mean values are 17.0 km^2 (6.6 mi^2) and 37.3 km^2 (14.4 mi^2), respectively. The stream slope of study watersheds range from approximately 0.0022–0.0196 dimensionless; the median and mean values are 0.0076 and 0.0081, respectively. The percentage of impervious area (*IMP*) of study watersheds range from approximately 0 to 73%; the median and mean values are 18.0 and 28.2, respectively.

There has been discussion in the literature concerning the size of watersheds for which the application of the rational method is appropriate. For application of the rational formula, Kuichling (1889, pages 40–41) stated: "For large areas, on the other hand, a more elaborate analysis becomes necessary in order to find under what condition the absolute maximum discharge will occur, although the method of procedure above indicated will remain the same." Kuichling (1889) did not suggest a specific large area limit. ASCE and WPCF (1960, p. 32), made the following statement when the rational method was introduced for design and construction of sanitary and storm sewers: "Although the basic principles of the rational method are applicable to large drainage areas, reported practice generally limits its use to urban areas of less than 5 square miles." Pilgrim and Cordery (1993, p. 9.14) explained that the rational method is one of the three methods widely used to estimate peak flows for small to medium sized basins, and wrote "it is not possible to define precisely what is meant by 'small' and 'medium' sized, but upper limits of 25 km^2 (10 mi^2) and 550 km^2 (200 mi^2), respectively, can be considered as general guides." Young et al. (2009) stated that the rational method might be applied to much larger drainage areas than typically assumed in some design manuals, as long as the watershed is unregulated. Results of this study will further indicate that there is no demonstrable relation between runoff coefficient and drainage area.

For any watershed (regardless of its size), some of the attributes necessary to apply the Kuichling method are the time of concentration (T_c), main channel length (L_c), and channel slope (S_c). For each of the 80 Texas watersheds, a geospatial database was developed by Roussel et al. (2005) containing L_c and S_c for each watershed, along with drainage area, basin width, longitude, latitude, and 39 other watershed characteristics. For this paper, L_c and S_c are used to estimate time of concentration T_c by Kirpich (1940) for channel flow plus travel time for overland flow using Kerby (1959). A combination of the methods of Kirpich (1940) and Kerby (1959) is discussed by Roussel et al. (2005) and Fang et al. (2008). The Kirpich equation (1940) was developed from the Soil Conservation Services (SCS) data for rural watersheds with drainage areas less than 0.45 km^2 and is presented below:

$$T_c = 3.978\, L_c^{0.77} S_c^{-0.385}, \tag{3.3}$$

where, L_c is the channel length in km and S_c is the channel slope in m/m. Fang et al. (2007, 2008) demonstrated that, for watersheds with relatively large drainage areas (more than 50 km^2), the Kirpich equation provides as reliable an estimate of T_c as the other empirical equations developed for large watersheds and the SCS velocity method

(Viessman and Lewis 2003). The T_c estimated using the Kirpich equation reasonably approximate the average T_c estimated from observed rainfall and runoff data (Fang et al. 2007). The T_c for the study watersheds ranged from 1.1 hours to 16.7 hours with median and mean values of 2.8 hours and 3.8 hours respectively.

Each of the 80 Texas watersheds was previously classified as either developed or undeveloped (Roussel et al. 2005, Cleveland et al. 2008). The classification scheme of developed and undeveloped watersheds is consistent with the characterization of watersheds in more than 220 USGS reports of Texas data from which the original data for the rainfall and runoff database were obtained (Asquith et al. 2004). Although this binary classification seems arbitrary, it does take into account the uncertainty in watershed development conditions for the time period of available data (Asquith and Roussel 2007). This binary classification was used by Asquith et al. (2006) in a regionalization study of unit hydrographs for the Texas watersheds (Asquith et al., 2004). Using the binary classification scheme for the 80 Texas watersheds, there are 44 developed watersheds in four metropolitan areas in Texas (Austin, Dallas, Fort Worth, and San Antonio) and 36 undeveloped watersheds. The 36 undeveloped watersheds consist of 16 watersheds near these four cities and 20 rural watersheds.

3. 4 Runoff Coefficients Estimated from Event Rainfall-Runoff Data

Rate-based C Derived for Individual Rainfall-Runoff Events

For this study, the intensity I in equation (3.2) is the maximum rainfall intensity before the time to peak, T_p, of a runoff hydrograph and is calculated as the maximum intensity found by a moving time window of duration t_{av} through the 5-minute interval rainfall hyetograph for the storm event. For data processing, only the largest Q_p for each storm event (in the case of multiple peaks in the overall hydrograph) was used.

The computation of C_{rate} is illustrated by example. In Figure 3.2, the I and C_{rate} values are shown as a function of t_{av} for two storm events gaged by the USGS: one on 09/22/1969 at USGS streamflow-gaging station 08048550 Dry Branch at Blandin Street, Fort Worth, Texas (hereinafter Dry Branch) and the second on 04/25/1970 at 08058000 Honey Creek near McKinney, Texas (hereinafter Honey Creek). As shown in Figure 3.2, as t_{av} increases I decreases and C_{rate} increases. For example, for the storm event at Dry Branch, as t_{av} increases from 5 minutes to 3.5 hours, I decreases from about 119 mm/hr (4.7 in/hr) to about 16.3 mm/hr (0.64 in/hr) and C_{rate} increases from 0.04 to 0.30. For the Dry Branch and Honey Creek watersheds, estimated T_c is 1.8 hours and 1.5 hours, respectively (Figure 3.2). These T_c values are derived from the Kerby-Kirpich method (Roussel et al. 2005; Fang et al. 2008); the corresponding C_{rate} values for the Dry Branch and Honey Creek watersheds are 0.23 and 1.70, respectively (Figure 3.2). Following the analysis leading to Figure 3.2, one C_{rate} value was determined using I corresponding to a moving time window T_c for each of about 1,500 events from the 80 Texas watersheds.

The occurrence of $C_{rate} > 1$ is related to unknown errors in T_c used to calculate I (see Figure 3.2), rainfall characteristics, fundamental measurement errors of rainfall and runoff data, or other unusual hydrologic factors. Several studies (French et. al 1974; Pilgrim and Cordery 1993; Young et al. 2009) have shown that values of C_{rate} greater than 1 are possible when rate-based C was determined from observed peak flow rate and computed rainfall intensity over a critical period of storm time. If the time of concentration is exactly correct for the watershed, and if the rainfall were spatially and temporally homogeneous and isolated (no preceding rainfall), then C_{rate} would have to be less than or equal to 1, but the rainfall normally varies in time and in space. Averaging temporal and spatial variability of rainfall leads to lower rainfall intensities and consequently lower predicted peak discharge values. Thus, when using an average rainfall as a predictor of Q_p, the C value will necessarily be higher than if the rainfall were truly uniform in space and time.

Most of the C_v values derived from about 1,500 rainfall events in Texas watersheds (Dhakal et al. 2012) are between 0 and 1. Rate-based runoff coefficient C_{rate} and volumetric-based runoff coefficient C_v are defined differently and were determined using different approaches from observed rainfall and runoff data. The use of rate-based runoff coefficients is appropriate if one wants to determine peak discharge using the rational method, and volumetric runoff coefficient can be used to estimate fractional rainfall loss using the constant fraction method (McCuen 1998) or for hydrologic modeling and runoff volume design purposes for a stormwater quality control basin (USEPA 1983; Guo and Urbonas 1996; Mays 2004).

Frequency distributions of C_{rate} values computed for about 1,500 events from the 80 Texas watersheds are shown in Figure 3.3, and summary statistics are listed in Table 3.1. Recalling that T_c was computed using the Kerby-Kirpich approach, for events where $T_p < T_c$, the mean value of C_{rate} is 0.31. In contrast, for events where $T_p \geq T_c$, the mean value of C_{rate} is 0.50 (Table 3.1). From inspection of the frequency distributions of C_{rate}

shown in Figure 3.3, estimates of C_{rate} are significantly greater (Welch-Satterthwaite t-test, p-value < 0.0001) for storm events when $T_p \geq T_c$ than those from events when $T_p < T_c$. Therefore, values of C_{rate} are dependent on the duration of rainfall event, which supports the idea proposed by Kuichling (1889). Kuichling's idea is that as T_c is reached, discharge for a watershed becomes a maximum (a peak) because the entire area is contributing runoff to the outlet. For cases considered in this research, the maximum value of *Crate* sometimes exceeded 1 (up to 4.48 when $T_p \geq T_c$). For 124 of about 1,500 events, the calculated C_{rate} is greater than 1.

Watershed Mean and Median Runoff Coefficients

Watershed mean and median values of C_{rate} for the 80 Texas watersheds were calculated for all observed storms in the same watershed (regardless of whether T_p was less than or greater than T_c). Computed watershed means of C_{rate} range from 0.07 to 1.79, and the watershed medians of C_{rate} range from 0.07 to 1.73 (Table 3.2). The average values of the individual watershed mean and median C_{rate} are 0.44 and 0.40, respectively (Table 3.2). Standard deviations from the watershed-means C_{rate} range from 0.03 to 0.87. Frequency distributions of the watershed mean and median of C_{rate} are shown in Figure 3.3; these distributional locations (means) are similar at a significance level of 0.01 (paired t-test, p-value = 0.04).

The amount of developed land in a watershed influences various runoff characteristics of a watershed. To study the relation between C_{rate} and the binary watershed development classification, statistical summaries of watershed median C_{rate} were computed (Table 3.3). The watershed median C_{rate} for developed watersheds range from 0.17 to 1.73, and for undeveloped watersheds, the watershed median C_{rate} ranged from 0.07 to 0.73. Of the developed watersheds, only two had a watershed-median $C_{rate} > 1$. The corresponding frequency distributions of watershed-median C_{rate} for developed and undeveloped watersheds are shown in Figure 3.3B. The watershed-median value of C_{rate} for developed watersheds is 0.40 and for undeveloped watersheds watershed-median value of C_{rate} is 0.20 (Table 3.3). The C_{rate} values of the developed watersheds are significantly larger than those from the undeveloped watersheds (Figure 3.3) as anticipated (Welch-Satterthwaite t-test, p-value <0.0001).

3.5 Runoff Coefficients from the Frequency-Matching Approach

The frequency-matching approach assumes return periods of rainfall and runoff events are the same (Hawkins 1993). Specifically, the *T*-year storm produces the *T*-year peak discharge. An alternative viewpoint is that the frequency-matching approach forces the largest rainfall intensity to produce the largest peak discharge within a given dataset. The authors observed that this assumption is implicit in circumstances of practical application of the rational method. Many design engineers assume that the *T*-year storm produces the *T*-year discharge. Although not a physical requirement, this assumption generally is appropriate in small watersheds.

The maximum rainfall intensities and the observed peak discharges were independently ranked from largest to smallest for each of the 80 Texas watersheds. The frequency-matched *C* was computed from the rank-ordered pairs of the observed peak discharge and the maximum rainfall intensity for each storm event using:

$$C_{rj} = \frac{Q_{pj}}{m_0 I_j A} \quad , \tag{3.4}$$

where C_{rj} is the runoff coefficient corresponding to the maximum rainfall intensity I_j, the observed peak discharge Q_{pj} of the j_{th} rank-order of I_{max}-Q_p data pairs, and drainage area A. A plot of runoff coefficients, C_{rj}, versus the maximum rainfall intensity was prepared for each watershed. For most of the watersheds, the C_{rj} increases until acquiring an approximate constant value as judged by an analyst. This constant value is referred to as C_r. For example, the plot for USGS streamflow-gaging station 08042650 North Creek Surface Water Station 28A near Jermyn, Texas (hereinafter North Creek near Jermyn) is presented in Figure 3.4A; $C_r = 0.20$ for this watershed.

The C_r also can be estimated from the slope of the regression line obtained from the plots of the rank-ordered $Q_{pj}/(0.00278*A)$ or Q_{pj*} values versus the rank-ordered I_j. For example, the regression equation for North Creek near Jermyn is Q_{pj*} (m^3/s/ha) = 0.19 * I_j (mm/hr) [Figure 3.4B]. The slope of this (and other similar equations) also is representative of C_r. Using the slope of the line, $C_r = 0.19$ for the North Creek near Jermyn. For most of the 80 Texas watersheds, C_r values obtained using analyst judgment or the regression slope method have approximately the same value, and C_r values ranged from 0.10 to 1.2 (one outlying C_r value of 1.2 was the only C_r value > 1). The mean and medians for C_r were 0.42 and 0.37, respectively. A statistical summary of C_r is listed in Table 3.4.

Comparison C_r to Watershed Median C_{rate} and Literature-based C_{lit}

For the 80 Texas watersheds, distributions of C_r and watershed median C_{rate} follow the same shape (Figure 3.5A) and are not statistically different at the 0.05 significance level (paired t-test, p-value = 0.27). The difference between C_r and watershed median C_{rate} for each watershed was calculated, and a statistical summary of the differences is listed in Table 3.4. The median value of C_r minus C_{rate} differences is 0.03 (Table 3.4). The minimum and maximum differences are -0.53 and 0.52 (Table 3.4), respectively, and quartiles of the differences between C_r and watershed-median C_{rate} are considered acceptably small (less than 0.06). About 74% of C_r and watershed-median C_{rate} values differ less than ±0.1 (Table 3.4).

The frequency distributions of literature-based C_{lit} from land-use data (Dhakal et al. 2012) for developed and undeveloped watersheds are shown in Figures 3.5A and 3.5B. The differences between C_r and C_{lit} or C_{rate} and C_{lit} are larger than the differences between C_r and C_{rate}. When the runoff coefficient is less than 0.55, C_{lit} is greater than C_r, otherwise C_{lit} is smaller than C_r (Figure 3.5A). About 75% of C_{lit} values are greater than C_r (Figure 3.5A). For typical applications of the rational method in urban (developed) watersheds, using the typically smaller C_{lit} value for the watershed would underestimate Q_p for design purposes. The difference between watershed-median C_r and C_{lit} or C_{rate} and C_{lit} for each watershed was calculated, and statistical summary of the differences is listed in Table 3.4. The median (50th percentile) of C_r minus C_{lit} and C_{rate} minus C_{lit} are -0.11 and -0.14, respectively, compared to the smaller mean differences between C_r and C_{lit} and C_{rate} and C_{lit} (-0.06 or -0.07, respectively) (Table 3.4).

C_r and C_{lit} for Developed and Undeveloped Watersheds

C_r and C_{lit} were grouped into two categories of developed and undeveloped watersheds (Roussel et al. 2005). Statistical summaries of C_r and C_{lit} for developed and undeveloped watersheds are listed in the Table 3.5; C_r and C_{lit} frequency distributions are shown in Figure 3.5B. The median value of C_r for undeveloped watersheds is 0.26 and the median value for the developed watershed is 0.45. These median values are similar to those for watershed-median C_{rate} (Table 3.3). The median and mean values of C_{lit} are larger than those of C_r for both developed and undeveloped watersheds (Table 3.5). About 68 and 78% of C_{lit} are larger than C_r for developed and undeveloped watersheds, respectively (Figure 3.5).

***C_r* in relation to impervious area**

For this study, the percentage of impervious area for each watershed was computed using 1992 National Land Cover Data for Texas (Vogelmann and others, 2001). C_r for 45 Texas watersheds with watershed imperviousness (*IMP*) greater than 10% are plotted in Figure 3.6. Schaake et al. (1967) developed the regression equation $C = 0.14 + 0.65 IMP + 0.05 S$ (referred to herein as the "Schaake et al. equation") for urban drainage areas in Baltimore, MD to relate C_r (for a return period of 5 years) to the relative imperviousness of the drainage area and channel slope of the watershed. C_r was calculated using the Schaake et.al equation for the 45 Texas watersheds with watershed imperviousness greater than 10%. For comparison purposes with C_r values for 45 Texas watersheds with watershed imperviousness (*IMP*) greater than 10%, C_r values calculated using the Schaake et. al equation also are plotted on Figure 3.6, along with *C* values extracted from Jens (1979), and equation 3.5 from Asquith (2011).

The results of these three studies [this study, Schaake et al. (1967), and Jens (1979)] are consistent—the value of C increases with increasing IMP. Asquith (2011) proposes a single equation to estimate C for Texas watersheds as a function of IMP. The equation is

$$C = 0.85 IMP + 0.15 \tag{3.5}$$

The equation was used to estimate the runoff coefficient C^* ("C-star") for the unified rational method (URAT) developed for a TxDOT research project summarized in Cleveland et al. (2011). Equation 3.5 is plotted in Figure 3.6 and is consistent with the general pattern of the data.

Several studies (Jens 1979; Pilgrim and Cordery 1993; Hotchkiss and Provaznik 1995; Titmarsh et al. 1995; Young et al. 2009) have demonstrated that C is highly dependent on the return period T. In this study, rate-based runoff coefficients were not derived for any return period because the observed data do not include all events that would constitute the complete annual series needed for the frequency analysis. Return-period based $C(T)$ values were computed by the authors using regional regression equations for Q_p and I for the 36 undeveloped Texas watersheds in the database and presented as a separate paper (Dhakal et al. 2011).

Correlation between C and Watershed Area

In order to evaluate the C values for the 80 Texas watersheds, C_r and watershed-median C_{rate} were used to estimate the peak discharge rates (Q_p) for each of about 1,500 rainfall-runoff events using the rational equation (3.1). The observed versus the modeled

Q_p are shown in Figure 3.7. The peak relative error (QB) between the observed and the modeled peak discharges was estimated to analyze the model results (Cleveland et al. 2006):

$$QB = \frac{P_i - O_i}{O_i}, \tag{3.6}$$

where, P_i are the modeled peak discharge values, O_i are the observed peak discharge values. Cleveland et al. (2006) suggested the following range of the QB for the acceptance of model performance:

$$-0.25 \le QB \le 0.25 \tag{3.7}$$

Median QB values derived using C_r and watershed-median C_{rate} are 0.11 and 0.00, respectively. Similarly, use of C_r resulted in about 56% and use of watershed-median C_{rate} resulted in about 59% of storms with QB less than ±50%. About 87 percent of the modeled Q_p values from both cases are within about half of a log cycle from the equal value line (Figure 3.7). The differences between the observed and modeled Q_p are generally within about a third of a log cycle, which is an uncertainty similar to that reported for regional regression equations of peak discharge in Texas by Asquith and Roussel (2009).

The observation that about 87% of the modeled Q_p values from both cases are within about half of a log cycle from the equal value line supports the conclusion by ASCE and WPCF (1960), Pilgrim and Cordery (1993), and Young et al. (2009) that the rational method may be applied to much larger drainage areas than typically indicated (assumed) in some design manuals, as long as streamflows in the watershed are unregulated (Young et al. 2009). ASCE and WPCF (1960) state “Although the basic principles of the rational method are applicable to large drainage areas, reported practice generally limits its use to urban areas of less than 5 square miles. Development of data for application of hydrograph methods is usually warranted on larger areas." Kuichling (1889, pages 40–41) made a similar statement for application of the rational formula to large watershed areas.

C_{rate} for all events, watershed-mean C_{rate}, and C_r versus watershed area (km^2) are displayed on Figure 3.8. Of note on Figure 3.8 is that the runoff coefficients are subject to substantial variability. That is, based on visual examination, there appears to be no relation between watershed drainage area and runoff coefficient. To apply a quantitative test, Pearson correlation coefficients between watershed-mean C_{rate} and C_r and watershed area are -0.27 and -0.26 with p-values of 0.012 and 0.018, respectively. Therefore, at the 95% confidence level (n = 80 observations), Pearson correlation coefficients between C_{rate} and C_r and watershed area are statistically significant but correlations are weak (determination coefficient $r^2 \approx 0.07$) and C exhibits high variability (Fig. 3.8). Only about 7% of the variance is described by the correlation. Although statistically significant, the contribution of the correlation to description of the variability of C_{rate} and C_r is not useful in an engineering context.

Of the 80 study watersheds, drainage area exceeds 40 km^2 (15 mile^2) for 17. The choice of 40 km^2 is completely arbitrary for the purposes of examining the runoff coefficient for relatively large watersheds. For this group of largest watersheds, average values of watershed-mean C_{rate} and C_r are 0.27 and 0.29, respectively. Standard deviations of watershed-mean C_{rate} and C_r are 0.11 and 0.12, respectively. In comparison, literature-based C_{lit} from land-use data (Dhakal et al. 2012) for these watersheds ranged from 0.30 to 0.55 with average value of 0.42 and standard deviation of 0.10. By inference, literature-based C_{lit} might be too large (Fig. 3.5), so estimates derived from application of C_{lit} to relatively large watersheds might lead to overly conservative estimates of discharge. Therefore, literature-based C values, e.g., published by ASCE and WPCF (1960) and current textbooks and design manuals, should not be used for watersheds with large drainage area.

Although published values for C are not appropriate for relatively large watersheds, the rational method can be applied if reasonable estimates of runoff coefficient can be derived. One source would be observations of runoff coefficient from hydrologically similar watersheds. Another would be derivation from observations of rainfall and runoff from the watershed of interest. The published limits [5 square miles, ASCE and WPCF (1960); 200 acres, TxDOT (2002)] on the maximum drainage area for application of the rational method seem to be arbitrary.

The authors do not advocate any specific limits that should be imposed on drainage area for application of the rational method. Therefore, it remains the responsibility of the end-user to apply appropriate engineering judgment when applying the rational method and the assumptions associated with the method, such as steady-state conditions.

3.6 Summary

The runoff coefficient, C, of the rational method is an expression of rate proportionality between rainfall intensity and peak discharge. Two methods were used to estimate C. Both methods used about 1,500 observed rainfall and runoff events data from

80 Texas watersheds to derive C. For the first method, the rate-based runoff coefficient, C_{rate}, was estimated for each rainfall-runoff event by the ratio of event peak discharge in a time series to the corresponding largest average rainfall intensity, I, in the same time series, averaged over the time window length. Time of concentration, T_c, was used as the time window length to estimate I. The T_c values estimated using the Kerby-Kirpich method were used for the 80 watersheds studied. The rate-based C is dependent on rainfall intensity averaging time t_{av} used for the study, because based on equation (3.2), estimates of the runoff coefficient based on observed data cannot be decoupled from the selection of the time-response characteristic. Watershed-mean and watershed-median values of C_{rate} were derived. The distributions of the watershed-mean and watershed-median C_{rate} are similar. Lastly, the C_{rate} values for the developed watersheds are consistently higher than those for undeveloped watersheds. For the second method, the frequency-matching approach, similar to the procedure used by Schaake et al. (1967), was used to sort peak discharges and average rainfall intensities independently and then to compute the rate-based C from the rational formula. A constant runoff coefficient C_r for the watershed was derived from the plot of the rate-based C versus I. The C_r values for the developed watersheds are consistently greater than those for the undeveloped watersheds; about 74% of C_r and watershed-median C_{rate} differ less than ±0.1 (Table 3.4). The values of C_r and C_{rate} were compared with the literature based runoff coefficients (C_{lit}) developed from land-use data for these study watersheds (Dhakal et al. 2012). About 75% of C_{lit} values are greater than C_r (Figure 3.5). For typical applications of the rational method in developed (urban) watersheds, watershed C_{lit} is less than C_r (Figure 3.5); using smaller C_{lit} would underestimate Q_p for design. An equation was proposed to

estimate rate-based C as a function of the percentage of impervious area *(IMP)* for Texas watersheds, and prediction from the equation is consistent with the results from Schaake et al. (1967) and Jens (1979).

3.7 Acknowledgments

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3.8 Notation

The following symbols are used in this paper:

A = drainage area in hectares or acres;

C_{lit} = literature–based runoff coefficient developed from landuse data;

C_{rate} = rate-based runoff coefficient;

C_r = runoff coefficients from the frequency matching approach;

C_{rj} = runoff coefficient estimated from the ratio of *jth* rank-ordered peak discharge and the maximum rainfall intensity data pairs;

C_v = volumetric runoff coefficient;

C^* = runoff coefficients as a function of percentage of impervious area from equation (3.5);

I = average rainfall intensity (mm/hr or in. /hr) with the duration equal to time of concentration;

I_j = the maximum rainfall intensity of the j_{th} order;

IMP = percentage of impervious area expressed as a decimal (50% = 0.5) for a watershed area;

I_w = average rainfall intensity from the entire rainfall event duration;

j = j_{th} term in the sequence of ordered peak discharge and the maximum rainfall intensity data pairs;

L_c = channel length in km;

m_o = the dimensional correction factor (1.008 in English units, 1/360 = 0.00278 in SI units);

O_i = observed peak discharge for computing QB;

P_i = modeled peak discharge for computing QB;

QB = peak relative error between the observed and simulated peak discharges;

Q_p = peak discharge in m^3/s or ft^3/s;

Q_{pj} = peak discharge of the j_{th} rank-order of maximum rainfall intensity and peak discharge data pairs in cubic meters per second;

Q_{pj^*} = Q_{pj} divided by 0.0028 times the drainage area in cubic meters per second per hectare;

S = channel slope (Schaake et al. (1967));

S_c = channel slope in m/m;

t_{av} = rainfall intensity averaging time period;

T_c = time of concentration;

T_p = time to peak;

t_w = rainfall event duration;

3.9 References:

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Table 3.1 Statistical Summary of C_{rate} Calculated from Observed Rainfall-Runoff Event Data.

	$T_p \geq T_c$ [1]	$T_p < T_c$ [2]	*All events*
Minimum	0.01	0.01	0.01
Maximum	4.48	2.68	4.48
25% Quartile	0.21	0.13	0.17
Median	0.38	0.24	0.32
75% Quartile	0.68	0.40	0.56
Mean	0.50	0.31	0.43
Standard deviation	0.42	0.27	0.39

Note: [1] for 952 events, and [2] for 548 events.

Table 3.2 Statistical Summary of Watershed-Median, Watershed-Mean and Standard deviation Values of C_{rate} for 80 Texas Watersheds.

	Watershed-Median C_{rate}	Watershed-Mean C_{rate}	Standard deviation
Minimum	0.07	0.07	0.03
Maximum	1.73	1.79	0.87
25 % Quartile	0.20	0.27	0.16
Median	0.31	0.36	0.22
75 % Quartile	0.55	0.56	0.37
Mean	0.40	0.44	0.27
Standard deviation	0.29	0.27	0.15

Table 3.3 Statistical Summary of Watershed-Median C_{rate} for Developed and Undeveloped Watersheds.

	Undeveloped [1]	Developed [2]
Minimum	0.07	0.17
Maximum	0.73	1.73
25%Quartile	0.13	0.30
Median	0.20	0.40
75%Quartile	0.28	0.71
Mean	0.24	0.53
Standard deviation	0.16	0.31

Note: [1] for 36 undeveloped watersheds, and [2] for 44 developed watersheds.

Table 3.4 Statistical Summary of C_r, and Differences among C_r, C_{rate} and C_{lit} for 80 Texas watersheds.

	C_r	$C_r - C_{rate}$ [1]	$C_r - C_{lit}$	C_{rate} [1] $- C_{lit}$
Minimum	0.09	-0.53	-0.44	-0.46
Maximum	1.20	0.52	0.66	1.19
25 % Quartile	0.25	-0.02	-0.17	-0.24
Median	0.37	0.03	-0.11	-0.14
75 % Quartile	0.54	0.06	0.02	-0.01
Mean	0.42	0.02	-0.06	-0.07
Standard deviation	0.23	0.14	0.21	0.27

[1] is watershed-median C_{rate}.

Table 3.5 Statistical Summary of C_r and C_{lit} for Developed and Undeveloped Watersheds.

	C_r Undeveloped watersheds	C_r Developed watersheds	C_{lit} Undeveloped watersheds	C_{lit} Developed watersheds
Minimum	0.09	0.18	0.29	0.37
Maximum	0.69	1.20	0.59	0.63
25%Quartile	0.20	0.34	0.33	0.52
Median	0.26	0.45	0.37	0.54
75%Quartile	0.42	0.62	0.44	0.58
Mean	0.32	0.50	0.39	0.54
Standard deviation	0.17	0.23	0.08	0.06

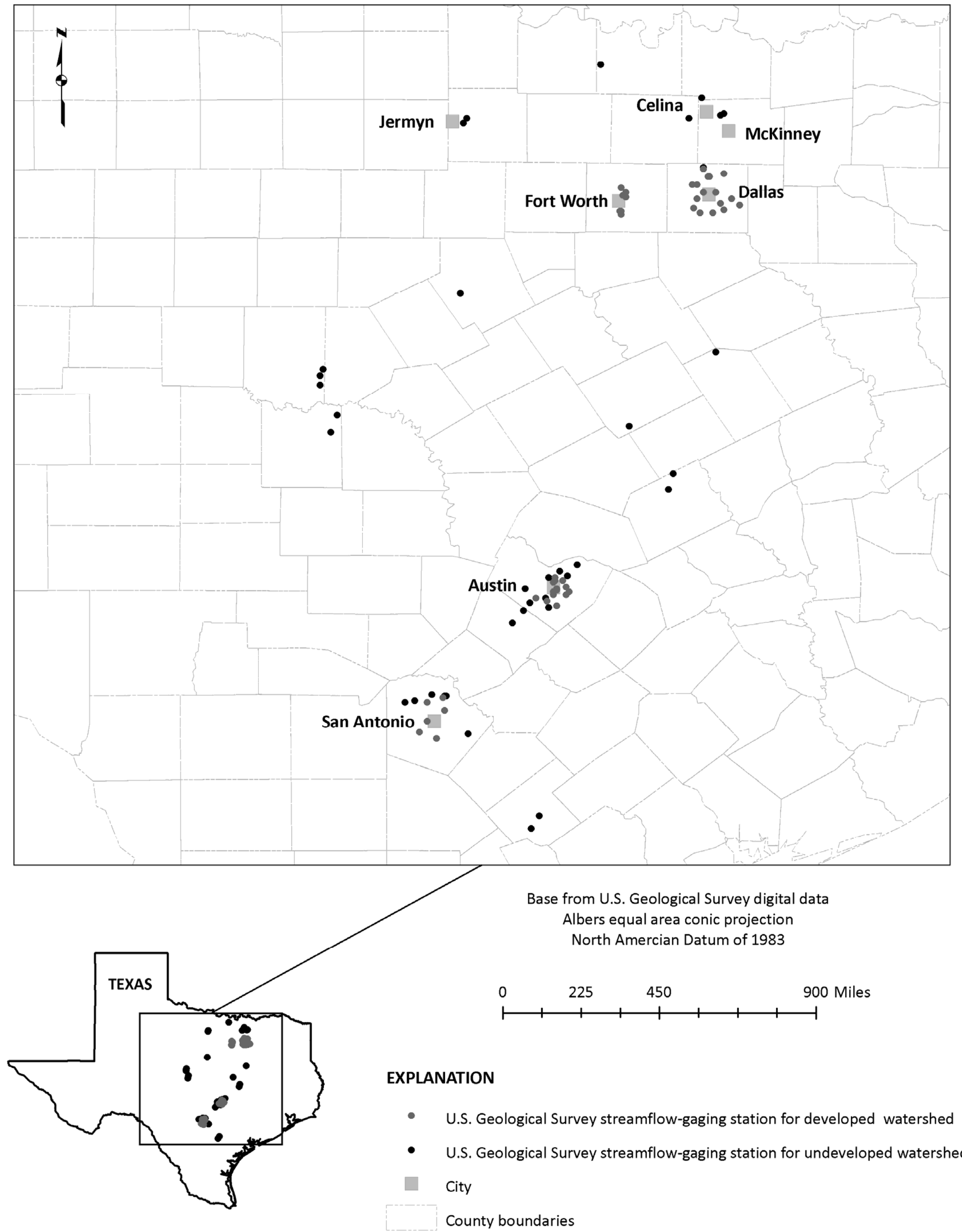

Fig. 3.1 Map showing U.S. Geological Survey streamflow-gaging stations representing 80 developed and undeveloped watersheds in Texas.

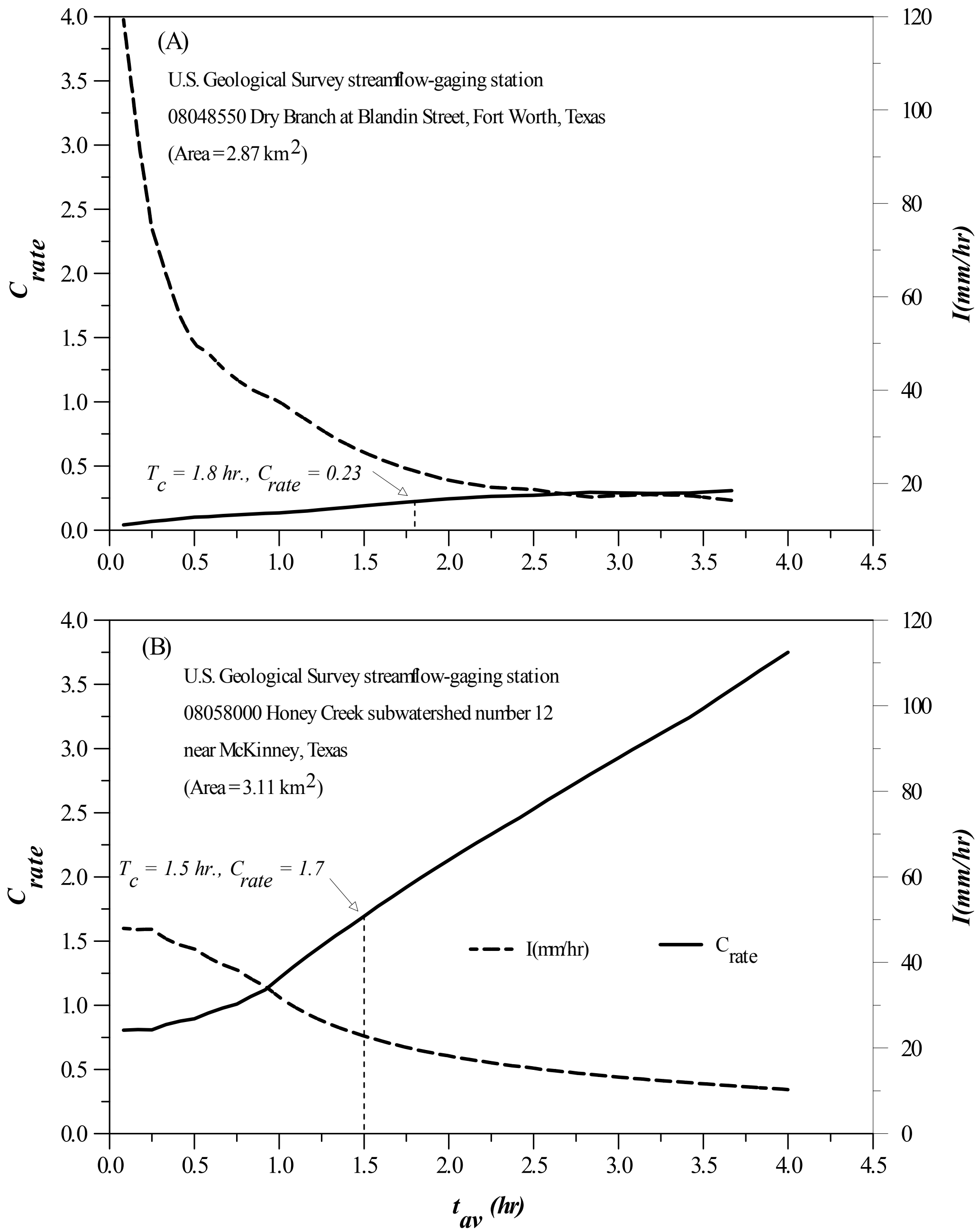

Fig 3.2 Average rainfall intensity *I* and runoff coefficient C_{rate} as a function of t_{av} for two storm events: (A) on 09/22/1969 in Dry Branch in Fort Worth, Texas (U.S. Geological Survey [USGS] streamflow-gaging station 08048550 Dry Branch at Blandin Street, Fort Worth Texas), and (B) on 04/25/1970 in Honey Creek near Dallas, Texas. (USGS streamflow-gaging station 08058000 Honey Creek subwatershed number 12 near McKinney, Texas).

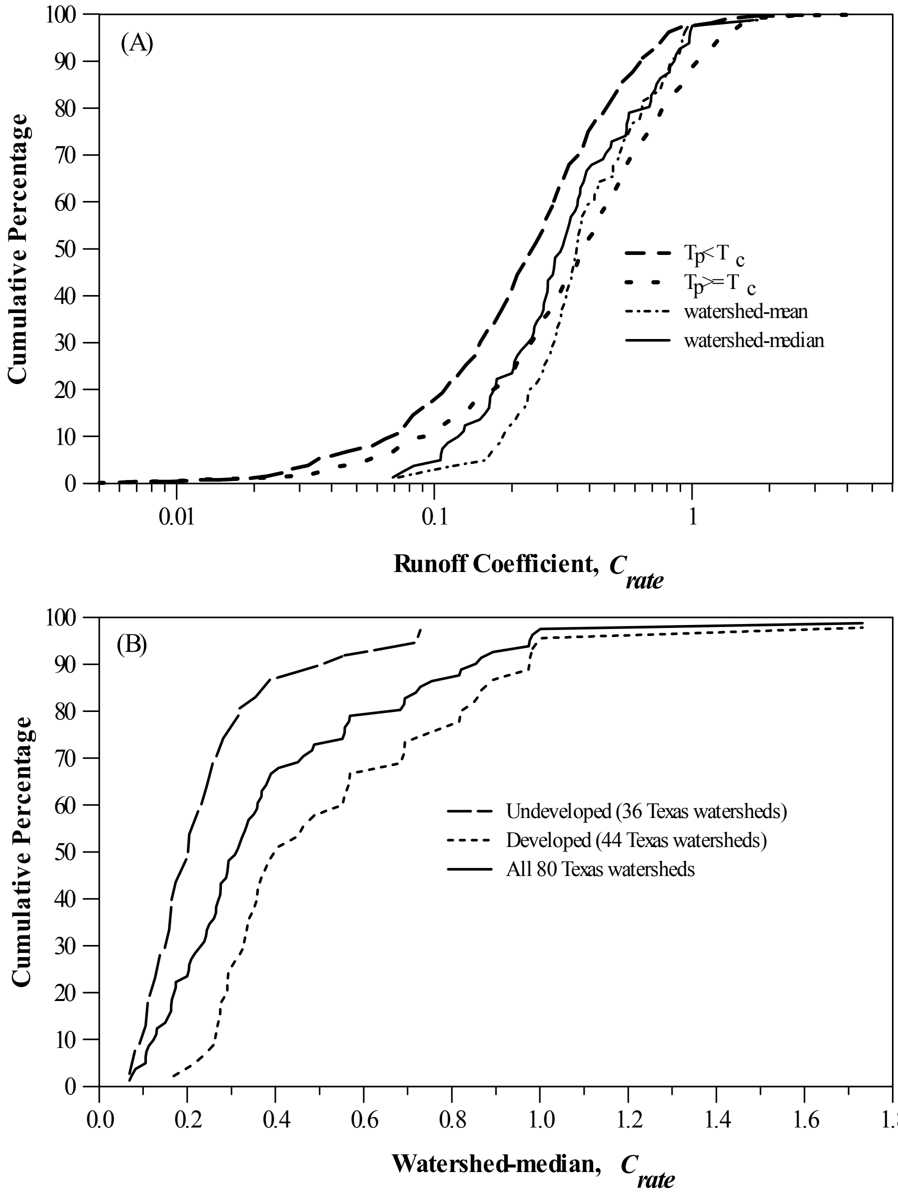

Fig. 3.3 Cumulative distributions of runoff coefficient (C_{rate} values): (A) for rainfall-runoff events when the time to peak (T_p) was less than the time of concentration (T_c); for rainfall-runoff events when the time to peak (T_p) was greater than or equal to the time of concentration (T_c); watershed-average (mean); and watershed-median; and (B) watershed-median C_{rate} for developed and undeveloped Texas watersheds.

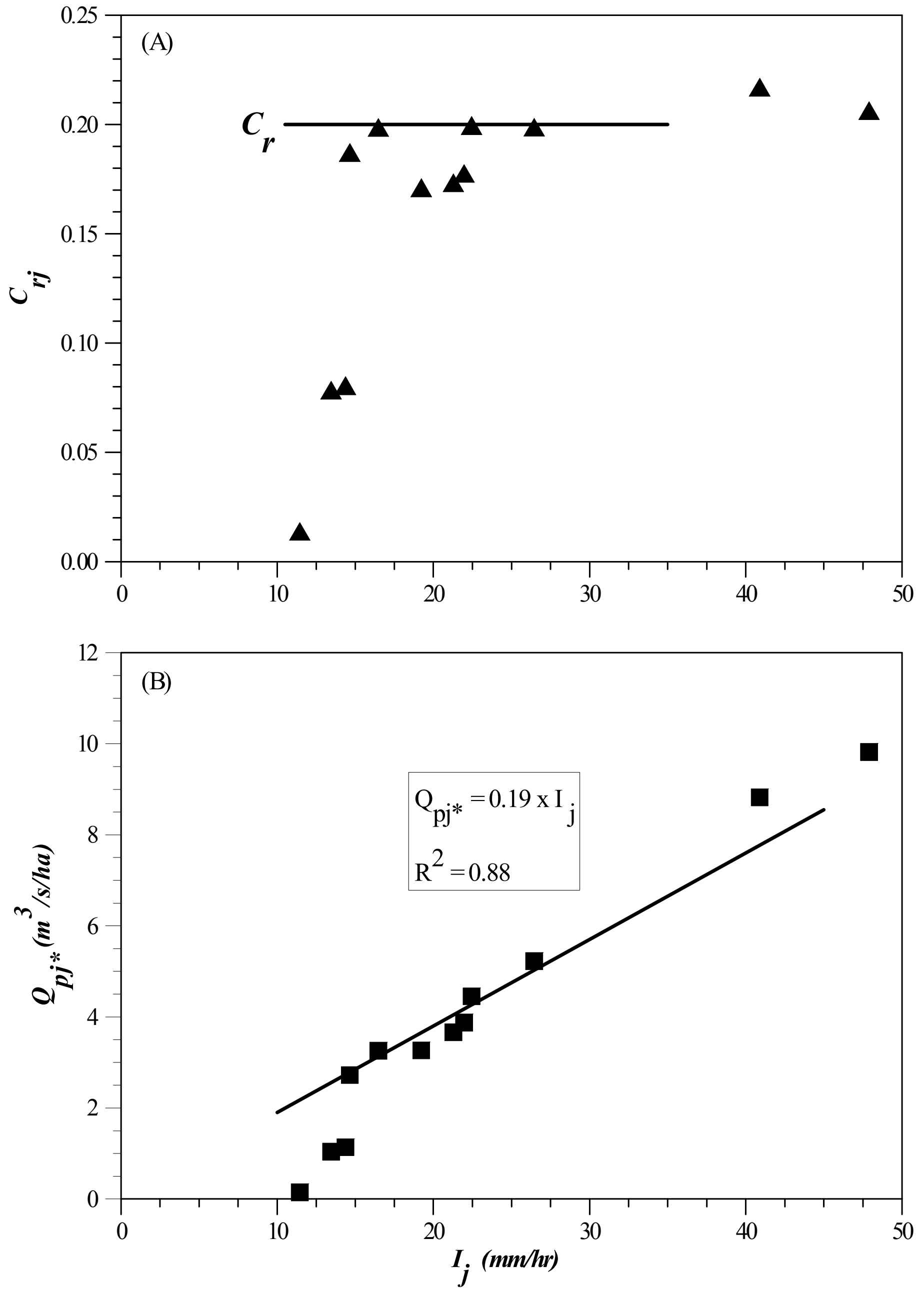

Fig. 3.4 (A) Runoff coefficient (C_r) values derived from the rank-ordered pairs of observed peak discharge and maximum rainfall intensity during each storm event in mm/hr at USGS streamflow-gaging station 08042650 North Creek Surface Water Station 28A near Jermyn, Texas, and (B) the rank-ordered pairs of the observed peak discharge and the average rainfall intensity for the same station.

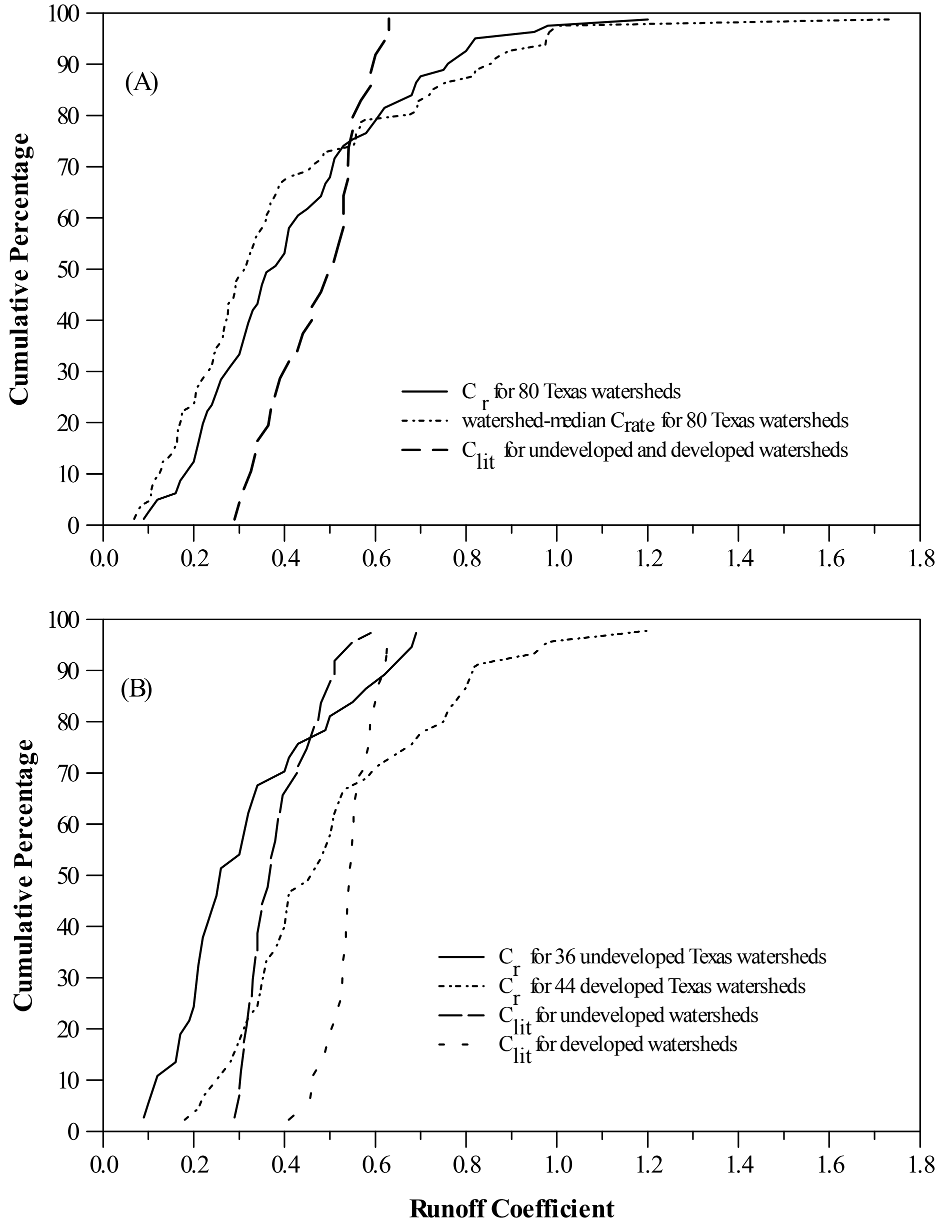

Fig. 3.5 Cumulative distributions of: (A) C_r, watershed-median C_{rate} and C_{lit}, and (B) distributions of C_r and C_{lit} for developed and undeveloped watersheds.

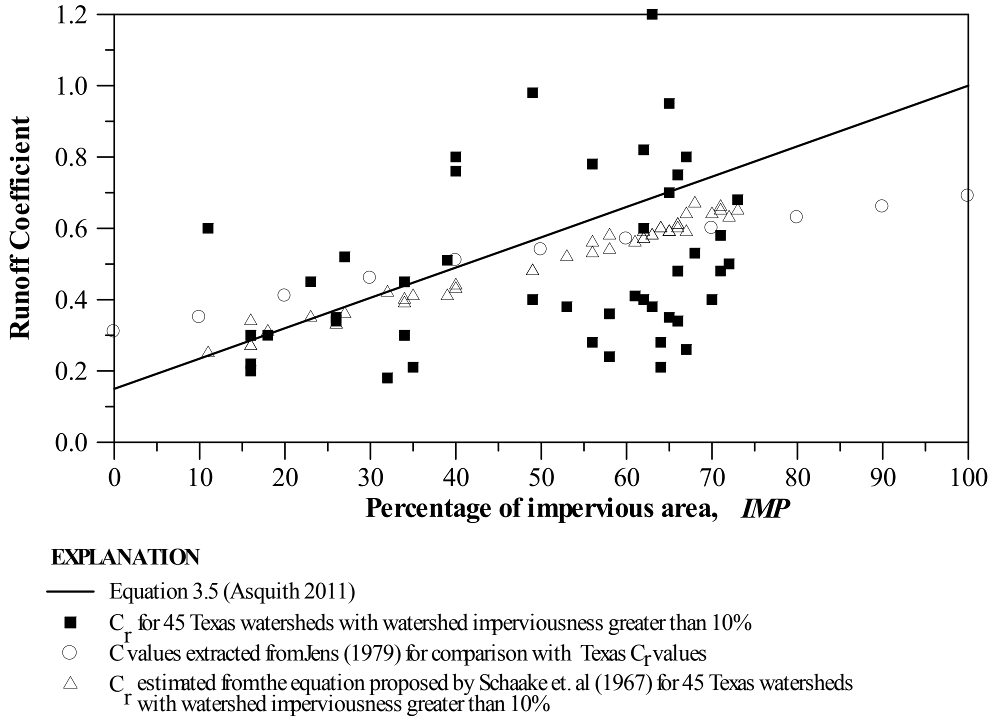

Fig. 3.6 Runoff coefficients versus the percentage of impervious area, IMP.

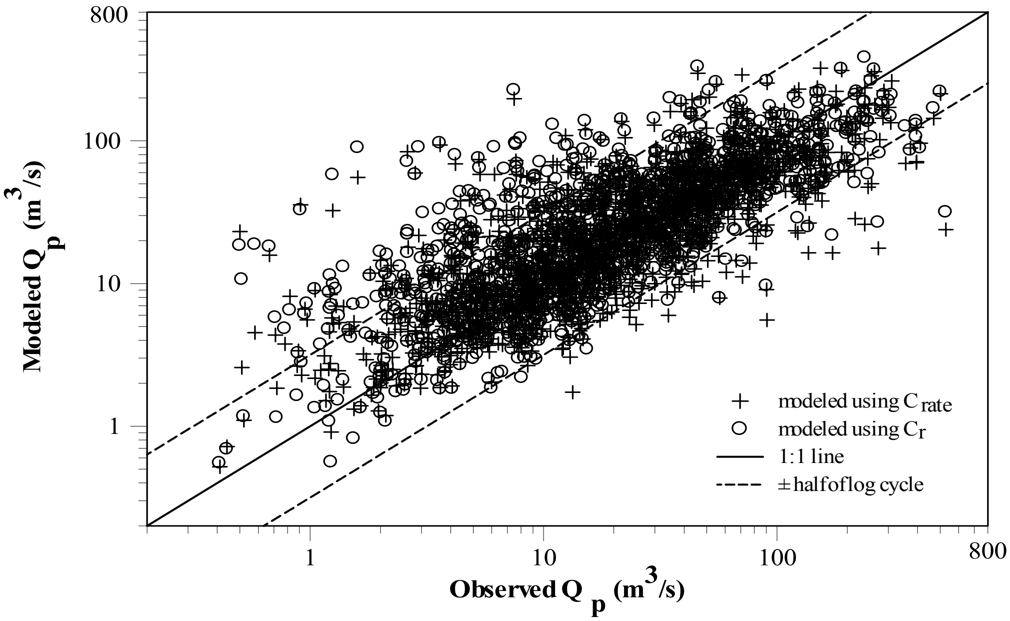

Fig. 3.7 Modeled peak discharges (Q_p) from rational equation (3.1) using C_r and watershed-median C_{rate} for 1,500 rainfall-runoff events in 80 Texas watersheds against observed peak discharges.

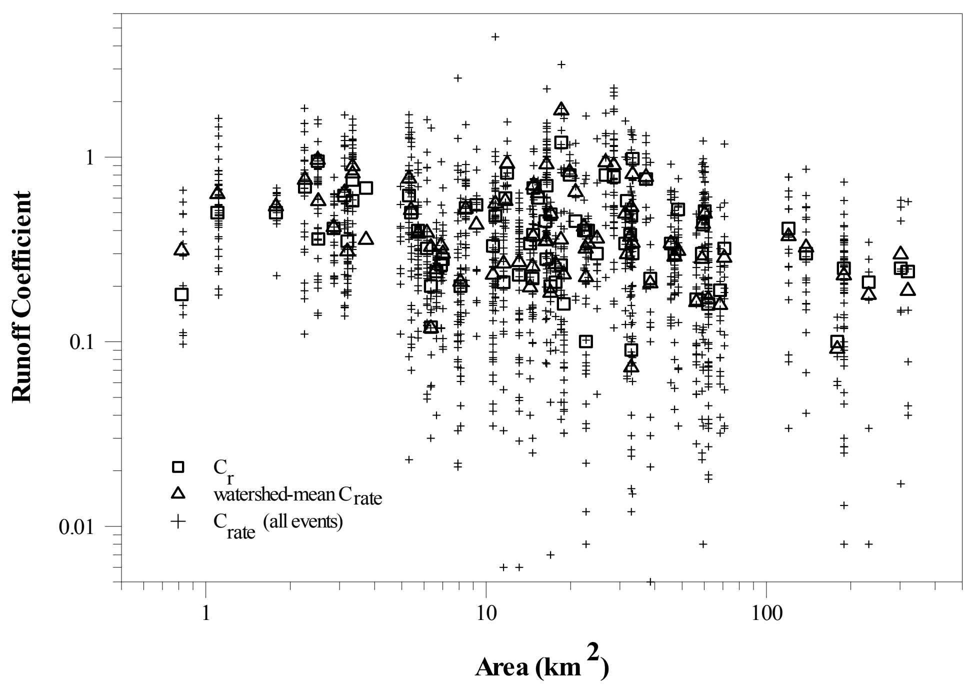

Fig 3.8. Rate-based runoff coefficients C_{rate} for all events, watershed-mean C_{rate}, and C_r versus drainage area (km^2) in 80 Texas watersheds.

Chapter 4. Return Period Adjustments for Runoff Coefficients Based on Analysis in Texas Watersheds

4.1 Abstract

The rational method for peak discharge (Q_p) estimation was introduced in the 1880s. The runoff coefficient (C) is a key parameter for the rational method, and the C has been declared a function of return period by various researchers. Rate-based runoff coefficients as function of return period, $C(T)$, were developed for 36 undeveloped watersheds in Texas using peak discharge frequency from previously published regional regression equations and rainfall intensity frequency for return periods T of 2, 5, 10, 25, 50, and 100 years. The $C(T)$ values developed in this study are most applicable to undeveloped watersheds. The $C(T)$ values of this study increase with T more rapidly than the increase suggested in prior literature. When the larger frequency factors are applied, if any resulting $C(T)$ is greater than unity, $C(T)$ is suggested to be set to 1.

4.2 Introduction

The rational method introduced by Kuichling (1889) is typically used to compute the peak discharge, Q_p (in m^3/s in SI units or ft^3/s in English units) for designing drainage structures:

$$Q_p = m_o CIA, \tag{4.1}$$

where C is the runoff coefficient (dimensionless), I is the rainfall intensity (mm/hr or in/hr) over a critical period of storm time (typically taken as the time of concentration, T_c, of the watershed), A is the drainage area (hectares or acres), and m_o is the dimensional correction factor (1/360 = 0.00278 in SI units, 1.008 in English units). From inspection of the equation, it is evident that C is an expression of rate proportionality between I and Q_p.

Typical "whole watershed" C values, that is, C values representing the integrated effects of various surfaces in the watershed and other watershed properties, are listed for different general land-use conditions in various design manuals and textbooks. Examples of textbooks that include tables of C values are Chow et al. (1988) and Viessman and Lewis (2003). Published C values, C_{lit}, were sourced from American Society of Civil Engineers (ASCE) and the Water Pollution Control Federation (WPCF) in 1960 (ASCE and WPCF 1960). The C_{lit} values were obtained from a response survey, which received "71 returns of an extensive questionnaire submitted to 380 public and private organizations throughout the United States." No justification based on observed rainfall and runoff data for the selected C_{lit} values was provided in the ASCE and WPCF (1960) manual.

A substantial criticism of the rational method arises because observed C values vary from storm to storm (Schaake et al. 1967; Pilgrim and Cordery 1993). The ASCE and WPCF (1960) manual, in describing tabulations of rational method C, state "The coefficients on these two tabulations (of C values) are applicable for storms having 5- to 10-year return periods (0.2 to 0.1 annual exceedance probabilities). Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionately smaller effect on runoff." Schaake et al. (1967) found that the average percentage increase of the coefficient for the 10-year return period $C(10)$ was only 10 percent as compared to the coefficient for the 1-year return period $C(1)$, and proposed adoption of a single value of C for design circumstances. The C has been considered a function of return period by various researchers (Jens 1979; Pilgrim and Cordery 1993; Hotchkiss and Provaznik 1995; Titmarsh et al. 1995; Young et al. 2009). Considering C as function of return period T, the rational formula can be expressed as (Jens 1979; Pilgrim and Cordery 1993):

$$Q(T) = C(T)\, I(T)\, A = C_{lit}\, C_f(T)\, I(T)\, A, \tag{4.2}$$

where $C(T)$ is C as function of the return period T, $I(T)$ is rainfall intensity as function of T, C_{lit} is literature-based C as previously defined herein based on T values ranged from 2- to 10- year recurrence intervals from text books (e.g., Viessman and Lewis 2003) or design manuals (e.g., TxDOT 2002), and $C_f(T)$ is a frequency factor or multiplier (DRCG 1969; Jens 1979). Equation 4.2 implies a conversion of $I(T)$ to $Q_p(T)$ where T denotes the same return period for both I and Q. Equation (4.2) also is the probabilistic interpretation of the rational formula and commonly used in design practices (French et. al 1974; Pilgrim and Cordery 1993).

Relatively, few studies have been performed to determine rational C values using frequency-based analysis of data (Young et al. 2009). An influential paper by Schaake et al. (1967) examined the rational method using experimental rainfall and runoff data collected from 20 small urban watersheds [< 0.6 km^2 (0.23 mi^2)] in Baltimore, MD. Those authors used watershed lag time to compute average rainfall intensity and used a frequency-matching approach to compute rate-based C values. Hotchkiss and Provaznik (1995) estimated C_{rate} for 24 rural watersheds in south-central Nebraska using event-paired and frequency-matched data. Young et al. (2009) estimated C_{rate} for 72 rural watersheds in Kansas with drainage areas less than 78 km^2 (30 mi^2) for different return periods. The peak discharge $Q(T)$ for each T was estimated using annual peak frequency analysis of observed streamflow records and rainfall intensity obtained from rainfall intensity-duration-frequency tables (Young et al. 2009).

For this study, return-period based runoff coefficients $C(T)$ were computed using Q_p and I calculated, not from statistical analysis of observed pairing of rainfall and runoff data, but from coupling of regional regression equations (Asquith and Slade 1997) and rainfall intensity (TxDOT 2002) for 36 undeveloped Texas watersheds. Subsequently, frequency factors $C_f(T) = C(T)/C(10)$ were computed. Using C for the 10-year return period as a base value to compute frequency factors $C_f(T)$ is consistent with literature (French et. al 1974; Pilgrim and Cordery 1993; Young et al. 2009). Results of $C(T)$ and frequency factors $C_f(T)$ were analyzed and compared with previous studies.

4.3 Study Watersheds

The study watersheds comprise 36 undeveloped watersheds in Texas, which have been used previously by the authors and associates (Asquith et al. 2004). The 36 watersheds consist of 20 rural watersheds and 16 suburban watersheds in near four cities: Austin, Dallas, Fort Worth, and San Antonio. Locations and geographic distribution of the streamflow-gaging stations associated with these watersheds are shown in Figure 4.1.

The classification scheme of developed and undeveloped watersheds accommodates the characterization of watersheds in more than 220 USGS reports of Texas data from which the original data for the rainfall and runoff database were obtained (Asquith et al. 2004). Although this binary classification seems arbitrary, it was purposeful and reflects the uncertainty in precise watershed development conditions for the time period of available data (Asquith and Roussel 2007). This same binary classification was successfully used to prepare regression equations to estimate the shape parameter and the time to peak for regional gamma unit hydrographs for Texas watersheds (Asquith et al. 2006).

The drainage area of study watersheds range from approximately 2.3–320 km^2 (0.9–123.6 mi^2); the median and mean values are 20.7 km^2 (8 mi^2) and 56.7 km^2 (21.9 mi^2), respectively. The stream slopes of study watersheds range from approximately 0.0022–0.0196 dimensionless; the median and mean values are both 0.0089, respectively. Many practitioners could argue that the application of the rational method is not appropriate for the range of watershed areas presented in this study. ASCE and WPCF (1960) made the following statements when the rational method was introduced for design and construction of sanitary and storm sewers: "Although the basic principles of the rational method are applicable to large drainage areas, reported practice generally limits its use to urban areas of less than 5 sq miles." (ASCE and WPCF 1960, p. 32). Pilgrim and Cordery (1993) stated that the rational method is one of the three methods widely used to estimate peak flows for small to medium sized basins. "It is not possible to define precisely what is meant by "small" and "medium" sized, but upper limits of 25 km^2 (10 mi^2) and 550 km^2 (200 mi^2), respectively, can be considered as general guides" (Pilgrim and Cordery 1993, p. 9.14). Young et al. (2009) stated that the rational method may be applied to much larger drainage areas than typically assumed in some design manuals provided that the watershed is unregulated. Thompson (2006) stated that watershed drainage area does not appear to be an applicable factor for discriminating between appropriate hydrologic technologies (such as rational method, regional regression equations, and site-specific flood frequency relations), other methods for discrimination between procedures for making design-discharge estimates should be investigated.

A geospatial database of properties for the 36 watersheds was developed by Roussel et al. (2005). For this paper, basin-shape factor, main channel length and channel slope were used to estimate time of concentration T_c by Kirpich (1940) for channel flow plus travel time for overland flow using Kerby (1959). This combination of methods to compute T_c is discussed by Roussel et al. (2005) and Fang et al. (2008). The Kirpich equation (1940) was developed from the Natural Resources Conservation Service (NRCS) data for rural watersheds with drainage areas less than about 0.45 km^2. Fang et al. (2007, 2008) demonstrated that, for watersheds with large drainage areas, the Kirpich equation provides as reliable an estimate of T_c as the other empirical equations developed for large watersheds and as the NRCS velocity method (Viessman and Lewis 2003). The T_c estimated using the Kirpich equation reasonably approximate the average T_c estimated from observed rainfall and runoff data (Fang et al. 2007).

4.4 Runoff Coefficients for Different Return Periods

Rate-based $C(T)$ values for the 36 study watersheds in Texas, and corresponding frequency factors were determined for various return periods using equation (4.3) (Pilgrim and Cordery 1993):

$$C(T) = \frac{Q(T)}{m_0\, I(T_c, T) A}, \tag{4.3}$$

where $Q(T)$, $C(T)$, and $I(T_c, T)$ are peak discharge, runoff coefficient, and rainfall intensity for the recurrence interval T, respectively. In this study, $Q(T)$ for each of the 36 undeveloped Texas watersheds was estimated by regional regression equations for Texas developed by Asquith and Slade (1997) that use contributing drainage area, a basin-shape factor, and main channel slope. The basin-shape factor is defined as the ratio of main channel length squared to contributing drainage area (sq. mi./sq. mi. or km^2/km^2) (TxDOT 2002).

The T_c for each watershed in Texas was developed using the Kerby-Kirpich equation (Roussel et al. 2005; Fang et al. 2008). Considering the county in which each watershed is located, the rainfall intensity, $I(T_c, T)$, for each return period was estimated using rainfall intensity-duration-frequency (IDF) relations (TxDOT 2002) with duration T_c:

$$I(T_c, T) = \frac{e}{(T_c + f)^g}, \tag{4.4}$$

where e, f, and g are coefficients for specific frequencies and Texas counties (TxDOT 2002). With $Q(T)$ from Asquith and Slade (1997) and $I(T_c, T)$ from TxDOT design manual (TxDOT 2002), equation (4.3) was used to compute $C(T)$ for each watershed and for each return period of T = 2, 5, 10, 25, 50 and 100 years.

The $C(T)$ versus T for three undeveloped Texas watersheds are presented as illustrative examples in Figure 4.2. For these three watersheds, $C(T)$ increases with increasing T. The value of $C(100)$ is 0.6 for Deep Creek, 1.05 for East Elm Creek, and 1.3 for Escondido Creek. The occurrence of $C(T) > 1$ could be related to inherent uncertainties of $Q(T)$ and $I(T_c, T)$. Several studies (French et. al 1974; Pilgrim and Cordery 1993; Young et al. 2009) have shown that values of $C(T)$ greater than 1 are possible when rate-based C was determined from observed peak discharge and rainfall intensity. So it is not a stretch to see $C(T) > 1$ for purely statistically independent studies of Q and I such as Asquith and Slade (1997) and TxDOT (2002). Analysis of observed rainfall and runoff data in 90 Texas watersheds has shown that only the volumetric runoff coefficient, C_v, as the ratio of total runoff depth to total rainfall depth, is less than 1 for all storm events (Dhakal et al. 2012).

Statistical summaries of *C(T)* are listed in Table 4.1 and corresponding boxplots of the distribution are shown in Figure 4.3. The median *C* values by *T*, as well as the curves shown in Figure 4.2, show that *C(T)* increases with the increasing recurrence interval for undeveloped watersheds in Texas. Ratios of *C(T)/C(10)* or frequency factors $C_f(T)$ are derived for the Texas watersheds and statistical summaries of the ratios are listed in Table 4.2; the mean and median values are of special importance for representation of frequency.

4.5 Discussion

Comparing C(T) and $C_f(T)$ for Texas Watersheds with Other Studies

Young et al. (2009) estimated median *C(T)* from observed data for 72 rural watersheds in Kansas and these values are shown in Figure 4.3 for comparison. The results of *C(T)* for Texas watersheds reported in Table 4.1 and Figure 4.3 are consistent with results reported by Young et al. (2009). The mean values of *C*(*T*) derived from observed data for 24 rural watersheds in south-central Nebraska (Hotchkiss and Provaznik 1995) are shown in Figure 4.3. Literature-based *C* values for the Nebraska watersheds are 0.35 for $T < 10$ years (Hotchkiss and Provaznik 1995). The mean *C(T)* values reported by Hotchkiss and Provaznik (1995) for the Nebraska watersheds are larger than not only median $C(T)$ determined for the Texas and Kansas watersheds but also C_{lit}.

French et al. (1974) mapped $C(10)$ values in New South Wales, Australia, for 37 rural watersheds, with drainage area up to 250 km^2 (96 mi^2). The relations between frequency factors and return period from French et al. (1974), reported by Young et al. (2009), Jens (1979) and Gupta (1989), and the results for the Texas watersheds are shown in Figure 4.4. The frequency factors $C_f(T)$ determined (1) for the Texas watersheds, (2) by Young et al. (2009), and (3) by French et al. (1974) exceed the textbook values from Gupta (1989) and Viessman and Lewis (2003), and exceed TxDOT (2002) values when $T > 10$ years. The Texas frequency factors $C_f(T)$ are similar to those determined for Kansas watersheds by Young et al. (2009). Lastly, the Texas frequency factors $C_f(T)$ exceed those from watersheds in New South Wales, Australia (French et al. 1974) by about 15 percent when $T > 10$ years, and less when $T < 10$ years.

The frequency factors $C_f(T)$ specified for Denver watersheds (DRCG 1969; Jens 1979) and later published in other textbooks (e.g., Gupta 1989, Viessman and Lewis 2003) and design manuals (e.g., TxDOT 2002) are listed in Table 4.3. Typically, a frequency factor $C_f(T)$ of 1.0 is used when $T < 10$ years (Table 3). The frequency factors $C_f(T)$ extracted from the FHWA curve (Jens 1979) for percent impervious area equal to 0 percent and 65 percent and from the study by French et al. (1974) and Young et al. (2009) are listed in Table 4.3 for comparison. Of special note is the observation that the Texas frequency factors, $C(2)/C(10)$ and $C(5)/C(10)$ as well as those from French et al. (1974) and Young et al. (2009), are not equal to 1.0 as in ASCE and WPCF (1960) [and later by Gupta (1989) and Viessman and Lewis (2003) and in the TxDOT (2002)] but rather are ratios less than 1.0 (Table 4.2 and 4.3). This means that the *C(T)* for $T < 10$ year (more frequent storms) of Texas, Kansas and Australia is less than C_{lit} commonly recommended in the literature.

The frequency factors $C_f(T)$ values were extracted from the FHWA curve (Jens 1979) for percent impervious areas of 65 percent because they are approximately the same as frequency factors $C_f(T)$ values presented in design manuals and textbooks (e.g., TxDOT design manual [2002]; Gupta 1989; Viessman and Lewis 2003). The frequency factors $C_f(T)$ presented in design manuals and textbooks are seemingly more appropriate for urban watersheds with relatively large percentage of impervious area.

The larger frequency factors $C_f(T)$ determined for Texas watersheds and those determined for Kansas watersheds (Young et al. 2009) are for undeveloped watersheds with impervious cover less than a few percent. The larger frequency factors $C_f(T)$ of Texas are similar to frequency factors $C_f(T)$ extracted from the FHWA curve (Jens 1979) for 0 percent impervious areas (Table 4.3). These frequency factors $C_f(T)$ were proposed by Bernard (1938). The frequency factors $C_f(T)$ from the FHWA curve (Jens 1979) for 100 percent impervious area is approximately 1.1 for *T* of 25, 50, and 100 years. If it is assumed that *C* is 1 for 100 percent impervious areas, then frequency factors $C_f(T)$ should be 1.0 for 100 percent impervious area for any *T*. Therefore, variable frequency factors $C_f(T)$ as additional function of percent of impervious area (Jens 1979) is a reasonable conjecture and supported by Young et al. (2009) as well as this study for Texas watersheds. When frequency factors $C_f(T)$ is applied and if resulting *C(T)* is greater than 1, Jens (1979), Gupta (1989), and TxDOT (2002) indicate *C(T)* should be set equal to 1.

***C* for 100-year Return Period**

In an adaption of the rational method, Bernard (1938) proposed that *C* varied in a functional manner with the *T*-year return period when related to the maximum or limiting *C* values (called C_{max}):

$$C = C_{\max}(T/100)^{x}, \quad (4.5)$$

where *x* is the exponent and ranges from 0.15 to 0.23 for undeveloped watersheds (Bernard 1938). Bernard (1938) assumed the C_{max} value corresponds to *C(100)*. In relation to equation (4.5), Jens (1979) proposed C_{max} = *C(100)* = 1.0 for watersheds with any percentage of impervious area for application of the equation (4.5) for the FHWA Manual (Jens 1979). *C(100)* for the 36 Texas watersheds range from 0.34 to 1.44 with mean and median values of 0.86 and 0.94 (Table 4.1). *C(100)* for three Texas watersheds also are presented as illustrative examples shown in Figure 4.2. Stubchaer (1975) applied the calibrated Santa Barbara Unit Hydrograph (SBUH) method on a 388-acre urban watershed and developed *C(T)* using the frequency analysis of rainfall and simulated runoff from the SBUH. The *C(100)* value determined for the watershed is 0.65 (Stubchaer 1975). The *C(100)* values for watersheds with different percentages of impervious cover from the Denver Manual (DRCG 1969) range from 0.20 to 0.96 and from Chow et al. (1988) range from 0.36 to 0.97; *C(100)* are consistently less than 1.

4.6 Summary

The runoff coefficients *C(T)* for different return periods (*T*) were developed for the 36 undeveloped Texas watersheds using previously published regional regression equations of peak discharge and county-based tabulated empirical coefficients for a model of rainfall intensities at different T. $C(T)$ values increase with T and these increases are more than previously thought. The frequency factors $C_f(T) = C(T)/C(10)$ determined in this study exceed those values in textbooks such as Gupta (1989) and Viessman and Lewis (2003) and those from TxDOT (2002) when $T > 10$ years. The frequency factors $C(2)/C(10)$ and $C(5)/C(10)$ for the Texas watersheds (Table 4.2) as well as from French et al. (1974) and Young et al. (2009) are not equal to 1 as assumed in ASCE and WPCF (1960) and published by Gupta (1989) and Viessman and Lewis (2003) and the design manual (TxDOT 2002) (Table 4.3 and Figure 4.4) but less than 1.

The frequency factors determined for the 36 Texas watersheds and the 72 Kansas watersheds (Young et al. 2009), larger than those mostly found in literature, are for undeveloped watersheds with relatively small percent impervious areas. The frequency factors mostly found in the literature, smaller than those determined for the 36 Texas watersheds, are appropriate for urban watersheds with relatively large percentages of impervious area, as supported and presented in literature (e.g., DRCG 1969; Stubchaer 1975; Jens 1979; Gupta 1989; Viessman and Lewis 2003; TxDOT 2002). Such frequency factors are consistent with those proposed by Jens (1979). When the frequency factor is applied, if resulted $C(T)$ is greater than unity, Jens (1979), Gupta (1989) and TxDOT (2002) suggested setting $C(T)$ equal to 1.

4.7 Acknowledgments

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4.8 Notation

The following symbols are used in this paper:

A = watershed area in hectares or acres;

$C_f(T) = C(T)/C(10)$, frequency factor or frequency multiplier;

C_{max} = maximum runoff coefficient for the return period 100 years;

$C(T)$ = rate-based runoff coefficient for return period T;

C_v = volumetric runoff coefficient as the ratio of total runoff depth and total rainfall depth;

I = average rainfall intensity (mm/hr or in. /hr) with the duration equal to time of concentration;

m_o = the dimensional correction factor (1.008 in English units, 1/360 = 0.00278 in SI units);

Q_p = peak runoff rate in m^3/s or ft^3/s;

$Q(T)$ = peak discharge for return period T;

Q_T = regional regression equation for natural basins developed for TxDOT;

T = recurrence interval or return period in years;

T_c = time of concentration;

4.9 References:

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Table 4.1 Statistical Summary of *C(T)* for Select Return Periods *(T)* for Texas Watersheds.

	2 years	5 years	10 years	25 years	50 years	100 years
Minimum	0.08	0.14	0.18	0.24	0.30	0.34
Maximum	0.39	0.64	0.77	0.97	1.14	1.44
25th percentile	0.12	0.29	0.39	0.48	0.50	0.55
Median	0.15	0.32	0.43	0.62	0.77	0.94
75th percentile	0.22	0.37	0.50	0.70	0.91	1.12
Average	0.18	0.33	0.44	0.59	0.73	0.86
Standard deviation	0.075	0.091	0.110	0.163	0.238	0.319

Table 4.2 Statistical Summary of the Frequency Factors *C(T)/C(10)* for Texas Watersheds.

	C(2)/C(10)	*C(5)/C(10)*	*C(25)/C(10)*	*C(50)/C(10)*	*C(100)/C(10)*
Minimum	0.25	0.63	1.10	1.14	1.16
Maximum	0.69	0.89	1.55	2.45	3.14
25th percentile	0.30	0.68	1.23	1.34	1.47
Median	0.35	0.72	1.42	1.66	1.93
75th percentile	0.51	0.85	1.49	2.06	2.54
Average	0.41	0.75	1.36	1.68	1.98
Standard deviation	0.14	0.09	0.14	0.39	0.578

Table 4.3 Frequency Factor or Multiplier for Literature-based Rational Runoff Coefficient *C* from Different Sources.

Return period, *T*, years	Frequency factor, $C_f(T)$, *C(T)/C(10)*				
	Gupta (1989)[1]	0 % *IMP*[2]	65 % *IMP*[2]	Young et al. (2009)	Texas watersheds
2	1.0	0.48	0.69	0.45	0.41
5	1.0	0.77	0.87	0.77	0.75
10	1	1	1	1	1
25	1.1	1.22	1.15	1.30	1.36
50	1.2	1.40	1.22	1.54	1.68
100	1.25	1.60	1.30	1.77	1.98

[1] $C_f(T)$ from the Denver material (DRCG 1969; Jens 1979) and later published in other textbooks (e.g., Gupta 1989, Viessman and Lewis 2003) and design manuals (e.g., TxDOT 2002)

[2] From Jens (1979)

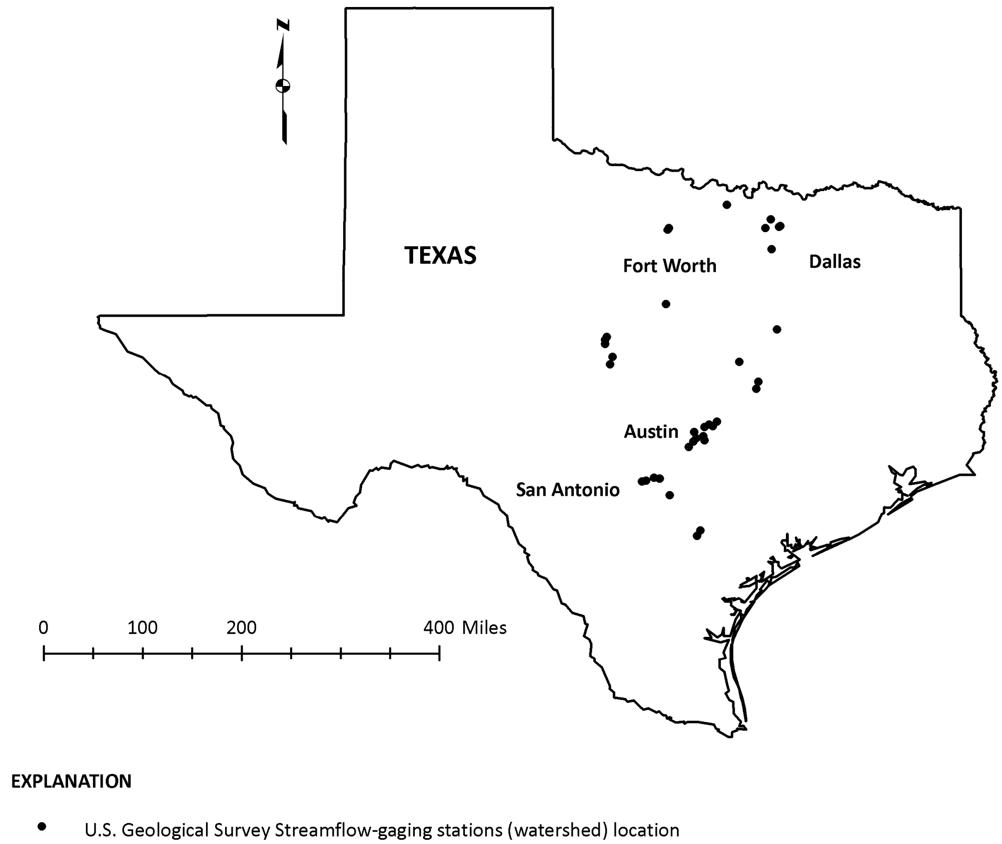

Fig. 4.1 Map showing the locations of U.S. Geological Survey streamflow-gaging stations in Texas associated with the 36 undeveloped watersheds considered for this study (two stations are very close and overlapped each other).

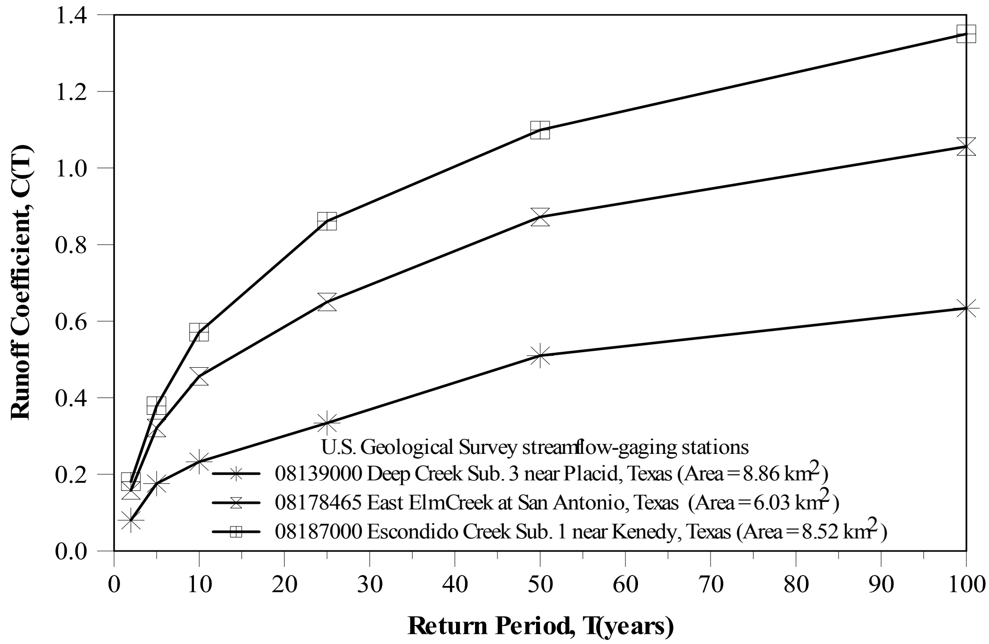

Fig. 4.2 Estimation of *C(T)* versus *T* for three undeveloped Texas watersheds.

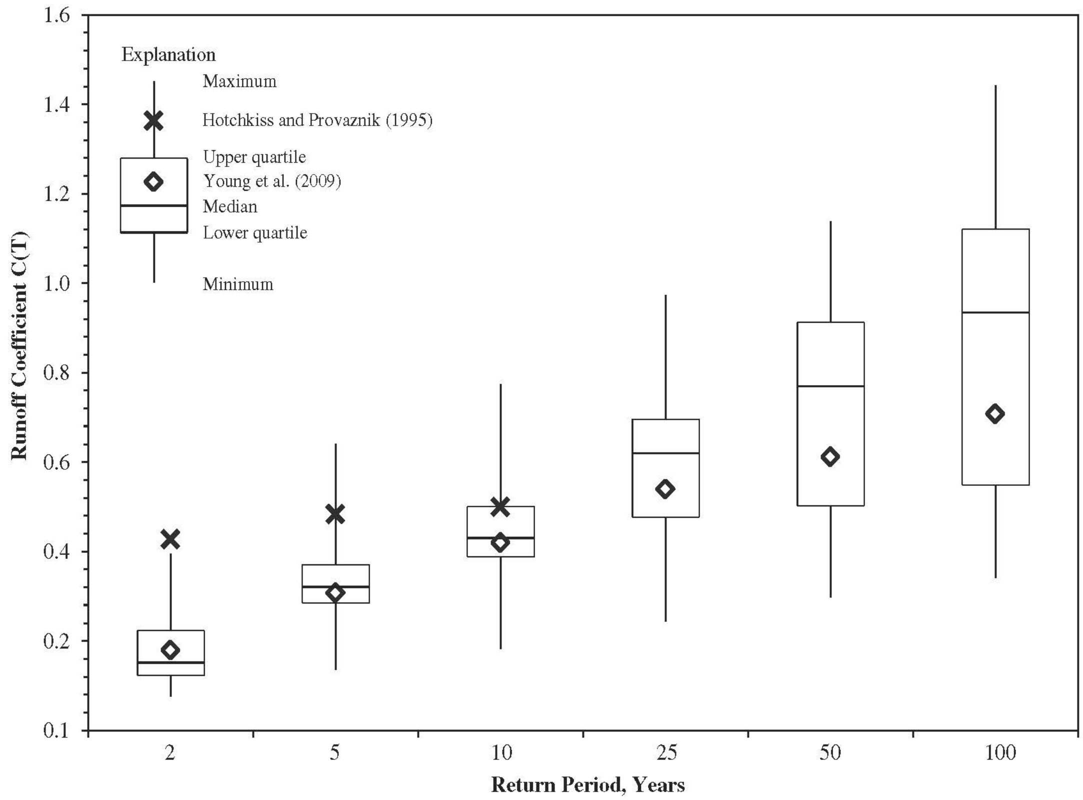

Fig. 4.3 Box plot for the distribution of runoff coefficients for different return periods from Texas watersheds, from Hotchkiss and Provaznik (1995) and from Young et al. (2009).

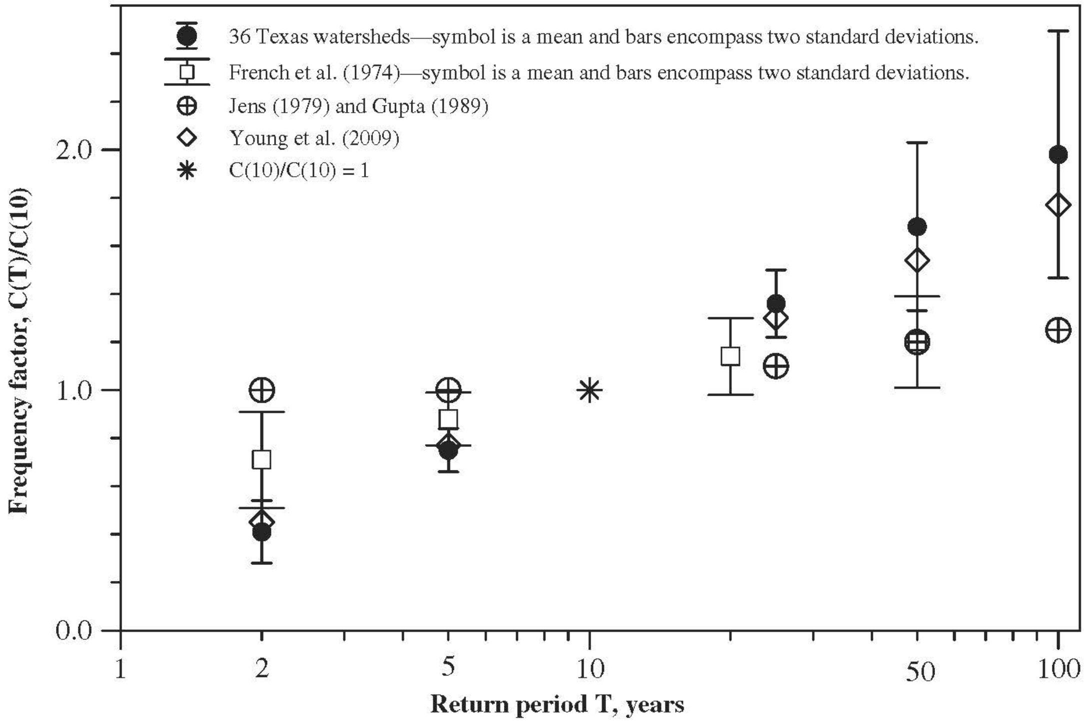

Fig. 4.4 Frequency factors for different return periods from Texas watersheds, Young et al. (2009), French et al. (1974) and FHWA (Jens 1979) and Gupta (1989).

Chapter 5. Modified Rational Unit Hydrograph Method and Applications in Texas Watersheds

5.1 Abstract

The modified rational method (MRM) is an extension of the rational method to develop simple runoff hydrographs. The hydrographs developed using the MRM can be considered application of a special unit hydrograph (UH) that is termed the modified rational unit hydrograph (MRUH) in this paper. Being a UH, the MRUH can be applied to nonuniform rainfall distributions and for watersheds with drainage areas greater than typically used for the rational method (a few hundred acres). The MRUH was applied to 90 watersheds in Texas using 1,600 rainfall-runoff events. Application of the MRUH involved three steps: (1) determination of rainfall excess using the runoff coefficient, (2) determination of the MRUH using drainage area and time of concentration, and (3) applying the unit hydrograph convolution. Times of concentration for the study watersheds were estimated using four empirical equations. Runoff coefficients used were estimated using two methods (literature-based from land-use information and back-computed from observed rainfall and runoff data). The Gamma UH, Clark-HEC-1 UH, and NRCS dimensionless UH were also used to predict peak discharges of all events in the database. The MRUH performed about as well as these UH methods when the same rainfall loss model was used.

5.2 Introduction

The rational method was originally developed for estimating peak discharge, Q_p, for sizing drainage structures, such as storm drains and culverts. The peak discharge, Q_p, (in m^3/s in SI units or ft^3/s in English units) is computed using:

$$Q_p = m_o CIA, \tag{5.1}$$

where C is the runoff coefficient (dimensionless), I is the rainfall intensity (mm/hr or in./hr) over a critical period of storm time (the time of concentration, T_c), A is the drainage area (hectares or acres), and m_o is the dimensional correction factor (1/360 = 0.00278 in SI units, 1.008 in English units). Kuichling (1889) and Llyod–Davies (1906) are credited with independent development of the rational method (Singh and Cruise 1992). Texas Department of Transportation (TxDOT) guidelines for drainage design recommend use of the rational method for watersheds with drainage areas less than 0.8 km^2 or 200 acres (TxDOT 2002).

Incorporation of detention basins to mitigate effects of urbanization on peak flows requires design methods to include the volume of runoff as well as the peak discharge (Rossmiller 1980). To use the rational method for hydraulic structures involving storage, the modified rational method (MRM) was developed (Poertner 1974). The term “modified rational method analysis" refers to “a procedure for manipulating the basic rational method techniques to reflect the fact that storms with durations greater than the normal time of concentration for a basin will result in a larger volume of runoff even though the peak discharge is reduced" [p. 54 Poertner (1974)]. Emil Kuichling (1889) stated: “in drainage areas of moderate size, the heaviest discharge always occurs when the rain lasts long enough at its maximum intensity to enable all portions of the area to contribute to the flow.” The MRM is based on the same assumptions as the conventional rational method and is a conceptual extension of the rational method (Viessman and Lewis 2003).

The MRM was revisited and reevaluated in this study. The hydrographs developed using the MRM represent application of a special unit hydrograph (UH) that will be termed the modified rational unit hydrograph (MRUH) in this study. The MRUH and unit hydrograph convolution were used to compute the direct runoff hydrographs for 1,600 rainfall-runoff events for 90 Texas watersheds. The objectives of the MRUH application were (1) to evaluate the applicability of the method if blindly applied to watersheds of size greater than typically used with either the rational method or the modified rational method (that is, a few hundred acres), and (2) to study the effects of the runoff coefficient and the time of concentration on prediction of runoff hydrographs using MRUH. In addition, three other unit hydrograph models—Clark unit hydrograph developed for HEC–1's generalized basin (Clark 1945; USACE 1981), Gamma unit hydrograph for Texas watersheds (Pradhan 2007) and Natural Resources Conservation Service Dimensionless (NRCS) unit hydrograph (NRCS 1972)—were used to compute the direct runoff hydrograph for each of the 1,600 rainfall-runoff events in 90 Texas watersheds for comparison of the results derived from application of the MRUH with these unit hydrograph methods.

Revisit of MRM

For the MRM, an urban stormwater runoff hydrograph resulting from the design storm is approximated as being either triangular or trapezoidal in shape (Smith and Lee 1984; Walesh 1989; Viessman and Lewis 2003), depending on the relation between the storm duration *(D)* and the time of concentration (T_c). The rising and the falling limbs are linear because the time-area relation for a watershed is assumed to be linear. If the storm duration (*D)* is equal to time of concentration (T_c) of a watershed, the resulting hydrograph is triangular with a peak discharge of $Q_p = CIA$ at time $t = T_c$; that is Case (A) in Fig. 5.1. If D is greater than T_c, the resulting hydrograph is trapezoidal with uniform maximum discharge, $Q_p = CIA$ from time D to T_c; that is Case (B) in Fig. 5.1. The linear rising and falling limbs each has duration of T_c, as shown in Fig. 5.1 (e.g., from Walesh 1989; Viessman and Lewis 2003). If the storm duration D is less than T_c, then the resulting hydrograph is trapezoidal with a maximum uniform discharge of Q_p' (Eq. 5.2) from the end of the storm *(D)* to the time of concentration T_c. The linear rising and falling portions of the hydrograph each has duration of $D < T_c$ as shown in Case (C) in Fig. 5.1. Smith and Lee (1984) and Walesh (1989) reported the modified rational hydrograph for the case when D is less than T_c and stated that Q_p' can be calculated by:

$$Q_p' = CIA\left(D / T_c \right) \tag{5.2}$$

Chien and Saigal (1974) used a linearized subhydrograph approach to derive three runoff hydrographs depending on rainfall duration and time of concentration, although there was an error for the case of $D < T_c$ as reported by Walesh (1975). Wanielista (1990) discussed the rational hydrograph in the context of the contributing area and assumed that the contributing area varies linearly with time. He derived a triangular hydrograph for $D = T_c$ and a trapezoidal hydrograph for $D > T_c$ from the rational method, similar to results by Chien and Saigal (1974) and Walesh (1975).

Smith and Lee (1984) examined the rational method as a unit hydrograph. They noted that if the rate of change of the contributing area is constant so that the accumulated tributary area increases and decreases linearly and symmetrically with the time, then the instantaneous unit hydrograph (IUH) response function, *u(t)*, is of rectangular shape given by:

$$u(t) = \frac{dA}{dt} = \frac{A}{T_c} \tag{5.3}$$

Using the rectangular response function (Eq. 5.3) in conjunction with a uniform rainfall intensity, Smith and Lee (1984) derived the resulting direct runoff hydrographs, *q(t)* (in watershed depth per time), by convolution as:

$$q(t) = \int_0^t i_e(\tau)u(t-\tau)\,d_\tau \tag{5.4}$$

where τ is the time with respect to which the integration is carried out and $i_e(\tau) = Ci$ is the effective rainfall intensity. Two types of outflow hydrographs, triangular and trapezoidal shape (Fig. 5.1), were obtained from Eq. (5.4), depending on the duration of rainfall. Similar to Smith and Lee (1984), Singh and Cruise (1992) assumed the watershed is represented as a linear, time-invariant system whose instantaneous unit hydrograph (IUH) is a uniform rectangular distribution of base time equal to the time of concentration of the watershed. They used convolution to derive the S-hydrograph and *D*-hour unit hydrograph from application of the rational method. The unit hydrograph developed thereby is called the modified rational unit hydrograph (MRUH) in this study. The MRUH is trapezoidal in shape and three examples of the MRUH used in this study are shown in Fig. 5.2. Cases (A), (B), and (C) of the MRM in Fig. 5.1 are runoff hydrographs and none is a unit hydrograph, although Cases (B) and (C) have the same shape as MRUH in Fig. 5.2.

Guo (2000, 2001) developed a rational hydrograph method (RHM) for continuous, nonuniform rainfall events. The RHM was used to extract the runoff coefficient and the time of concentration from observed rainfall and runoff data through optimization. The RHM developed by Guo (2000, 2001) is not unit hydrograph method, but is a practical procedure to compute the moving average rainfall intensity for application of the MRM. Guo (2000, 2001) used a linear approximation from the discharge $Q(T_d)$ at the end of the rainfall event to zero at the time $T_d + T_c$ for RHM. We checked that, for nonuniform rainfall events, this approximation is incorrect because it violates the conservation of mass between the rainfall excess and the runoff hydrograph. Treating the MRM as a unit hydrograph method, such as the MRUH, will always conserve mass.

Bennis and Crobeddu (2007) developed an improved rational hydrograph (IRH) method for small urban catchments using a rectangular impulse response function (Singh and Cruise 1992; Smith and Lee 1984). They considered impervious and pervious areas separately. The IRH method was calibrated and validated using ten rainfall events from two urban catchments. However, the Bennis and Crobeddu (2007) IRH is not a unit hydrograph method.

The unit hydrograph for a watershed can be used to predict the direct runoff hydrograph for any given rainfall excess hyetograph (uniform or non-uniform distribution) using unit hydrograph convolution (Chow et al. 1988; Viessman and Lewis 2003). If the MRM is application of a unit hydrograph method, then the approach establishes a continuity of hydrograph-development methods from very small watersheds to relatively large watersheds. For the MRUH, the assumption and restriction of the MRM to uniform rainfall distributions, as stated by Rossmiller (1980) and others, is not necessary. The MRUH can be applied to nonuniform rainfall hyetographs to obtain direct runoff hydrographs using convolution similar to application of other unit hydrograph methods.

The *D*-hr MRUH results from a uniform excess rainfall intensity of *1/D* in./hr over *D* hrs and has a peak discharge of A/T_c in ft^3/s when drainage area *A* is in acres and T_c is in hours (taking into account that one-acre inch per hour is nearly equal to one cubic foot per second). If SI units are used (drainage area *A* in hectare and rainfall intensity in mm/hr), the peak discharge from the MRUH should be equal to $A/(360T_c)$ in m^3/s. The MRUH has only one control parameter—time of concentration of the watershed. The runoff coefficient, *C*, for the rational method is not a control parameter of the MRUH. This is because the MRUH results from one unit of rainfall excess depth and the runoff coefficient is actually used to determine rainfall excess, not for transformation of effective rainfall to direct runoff hydrograph (DRH) through application of the MRUH.

Application of the MRUH is straightforward and similar to application of other unit hydrograph methods. Convolution of the unit hydrographs with the rainfall excess is applied to obtain the direct runoff hydrograph for each storm event. The excess rainfall or the net rainfall is obtained from the product of the incremental rainfall and *C,* similar to Smith and Lee (1984). The MRUH was first tested using data obtained for concrete surfaces from Yu and McNown (1964). The first dataset was based on a test bed with an area of 152.4 m by 0.3 m (500 ft by 1 ft), surface slope of 0.02, and a uniform rainfall intensity of 189 mm/hr. The second dataset was based on a test bed with an area of 76.8 m by 0.3 m (250 ft by 1 ft), surface slope of 0.005, and a variable rainfall intensity with an initial rate of 43.2 mm/hr, increasing to 96 mm/hr at t = 6 minutes, decreasing to 45 mm/hr at t = 18 minutes, and ending at t = 32 minutes. The T_c of about 5 minutes was computed using the Kirpich method (Kirpich 1940) for both experiments. A trapezoidal 1-minute MRUH was developed for each experiment (Fig. 5.2A). The runoff coefficient was taken to be unity. The time interval used for unit hydrograph convolution was 1 minute. Predicted and observed hydrograph ordinates used the same time interval. For both cases, the modeled results match the observed results well (Fig. 5.3).

The Nash-Sutcliffe efficiency, *EF,* is a parameter to measure goodness-of-fit between modeled and observed data (Legates and McCabe 1999) and is defined by Eq. (A.3) in Appendix A. For hydrograph simulation, a good agreement between the simulated and the measured data is reached when *EF* is higher than 0.7 (Bennis and Crobeddu 2007). For the experiment using the uniform rainfall intensity (Fig. 5.3A), *EF* was 0.93 and for the experiment using the nonuniform rainfall intensity (Fig. 5.3B), *EF* was 0.80, which are greater than 0.7 for both cases indicating a good fit.

5.3 Applications of MRUH in Texas Watersheds

Watersheds Studied and Rainfall-Runoff Database

Watershed data taken from a larger dataset (Asquith et al. 2004) accumulated by researchers from the U.S. Geological Survey (USGS) Texas Water Science Center, Texas Tech University, University of Houston, and Lamar University were used for this study. The dataset comprises 90 USGS streamflow-gaging stations in Texas, each representing a different watershed (Fang et al. 2007, 2008). Location and geographic distribution of the stations are shown in Fig. 5.4. There are 29, 21, 7, 13 watersheds in Austin, Dallas, Fort Worth, and San Antonio areas, respectively. The remaining 20 watersheds are small watersheds located in rural areas of Texas. The drainage areas of study watersheds ranged from approximately 0.8 to 440.3 km^2 (0.3 to 170 mi^2), with median and mean values of 17.0 km^2 (6.6 mi^2) and 41.1 km^2 (15.9 mi^2), respectively. There are 33, 57, and 80 study watersheds with drainage areas less than 13 km^2 (5 mi^2), 26 km^2 (10 mi^2), and 65 km^2 (25 mi^2), respectively. The stream slope of study watersheds ranged from 0.0022 to 0.0196, with median and mean values of 0.0075 and 0.081, respectively. The percentage of impervious area (*IMP*) of study watersheds ranged from approximately 0.0 to 74.0, with median and mean values of 18.0 and 28.4, respectively.

The rainfall-runoff dataset comprised 1,600 rainfall-runoff events recorded during 1959–1986. The number of events available for each watershed varied—for some watersheds less than 4 events were available, whereas for others as many as 50 events were available (Cleveland et al. 2006). Rainfall depths ranged from 3.56 mm (0.14 in.) to 489.20 mm (19.26 in.), with median and mean values of 57.66 mm (2.27 in.) and 66.8 mm (2.63 in.), respectively. Maximum rainfall intensities calculated using the time of concentration ranged from 0.01 mm/min (0.03 in./hr) to 2.54 mm/min (6.01 in./hr), with median and mean values of 0.25 mm/min (0.58 in./hr) and 0.30 mm/min (0.72 in./hr), respectively.

Roussel et al. (2005) developed a geospatial database containing watershed drainage area, longitude and latitude of the USGS streamflow-gaging station, (which was treated as the outlet of the watershed), and 42 watershed characteristics of each individual watershed. These watershed parameters were used to estimate time of concentration for study watersheds using four empirical methods (Roussel et al. 2005; Fang et al. 2008).

Time of Concentration and Runoff Coefficients

Time of concentration, T_c, and the runoff coefficient, C, are the required parameters for application of the MRUH. The T_c values for the study watersheds were estimated by Fang et al. (2008) using four empirical equations: (1) Williams equation (1922) developed from data for watersheds with drainage areas less than 129.5 km^2 (50 mi^2), (2) Kirpich equation (1940) developed from NRCS data for rural watersheds with drainage areas less than 0.45 km^2, (3) Johnstone–Cross equation (1949) developed from data for watersheds with drainage areas between 65 and 4206 km^2 (25–1620 mi^2), and (4) Haktanir–Sezen equation (1990) developed from data for watersheds with drainage areas from 11 to 9867 km^2 (4 to 3811 mi^2). For large watersheds, Fang et al. (2007, 2008) demonstrated that the Kirpich equation provides as reliable an estimate of T_c as other empirical equations and the NRCS velocity method (Viessman and Lewis 2003). T_c estimated using the Kirpich equation reasonably approximates the average T_c estimated from observed rainfall and runoff data (Fang et al. 2007). Application of the four empirical equations requires watershed parameters watershed drainage area, channel length, channel slope, and watershed shape (Fang et al. 2008). For the present study, T_c was estimated using watershed parameters developed by USGS researchers through automated watershed delineation using digital elevation models and geographic information system (GIS) software (Fang et al. 2008). The range of T_c values for the study watersheds, along with median and mean values, estimated using the four empirical equations are presented in Table 5.1.

Rainfall excess was computed using the volumetric interpretation of the rational runoff coefficient. Wanielista et al. (1997) showed that rainfall loss for a uniform rainfall input (intensity of i) is equal to $(1 - C)\ iDA$ and rainfall excess is equal to $CiDA$. The rational rainfall loss method is the constant fractional loss model—in which it is assumed that the watershed immediately converts a constant fraction (proportion) of each rainfall input into an excess rainfall fraction (McCuen 1998).

Two estimates of the runoff coefficient were examined for the application of MRUH. The first is a watershed composite, literature-based coefficient (C_{lit}) derived from land-use information for the watershed and published C_{lit} values for appropriate land-uses (Dhakal et al. 2011). The composite C assigned to a watershed is the area-weighted mean C derived from the land-use classes in the watershed. Values of C_{lit} for the study watersheds ranged from 0.29 to 0.63, with median and mean values of 0.50 and 0.47, respectively.

The second runoff coefficient is a back-computed, volumetric runoff coefficient, C_{vbc}, determined by preserving the runoff volume using observed rainfall and runoff data. C_{vbc} was estimated by the ratio of total runoff depth to total rainfall depth for individual observed storm event. Computed C_{vbc} ranged from 0.001 to 0.99, with median and mean values of 0.29 and 0.33, respectively, for 1,600 rainfall events in the study watersheds. The determination and comparison of C_{lit} and C_{vbc} for the study watersheds was documented by Dhakal et al. (2011).

Estimated Runoff Hydrographs Using the MRUH

For the 90 Texas watersheds, observed rainfall hyetograph and runoff hydrograph data were tabulated using a time interval of five minutes. Therefore, the five-minute MRUH was developed for each of the 90 study watersheds. The five-minute MRUH duration is less than the time of concentration for all study watersheds. The basic time interval used for unit hydrograph convolution and hydrograph ordinates was five minutes. Comparison between observed and simulated peak discharges and time to peak are presented in Figs. 5.6 and 5.7.

The results for the event on 07/08/1973 at the USGS streamflow-gaging station 08157000 Waller Creek, Austin, Texas are presented in Fig. 5.5 as an illustrative example. The watershed drainage area is 5.72 km^2 (2.21 square miles). The back-computed volumetric runoff coefficient, C_{vbc}, is 0.29. The T_c values estimated using Kirpich, Haktanir-Sezen, Johnstone-Cross, and Williams equations are 1.7, 2.2, 1.4, and 3.4 hours, respectively. Peak discharge of the 5-minute trapezoidal unit hydrograph (Fig. 5.2B) is 24 m^3/s (cms). Duration of the rainfall event was 19 hours. Three distinct rainfall episodes resulted in three distinct discharge peaks. These were reasonably represented by results from the MRUH using T_c estimated by Kirpich, Haktanir-Sezen, and Johnson-Cross equations. Results developed from the Williams equation appear to over-estimate time of concentration for the watershed and peak discharges were then underestimated (Fig. 5.5). The Nash-Sutcliffe efficiencies from MRUH model results using T_c values estimated from Kirpich, Haktanir-Sezen, Johnstone-Cross, and Williams equations are 0.83, 0.86, 0.70, and 0.63, respectively. Simulated time to peak agrees reasonably well with observed values (Fig. 5.5) when using T_c estimated by Kirpich, Haktanir-Sezen, and

Johnson-Cross equations. However, using T_c estimated by Williams equation resulted in the computed time to peak exceeding the observed time to peak. Although the drainage area of Waller Creek watershed exceeds that usually accepted for MRM application, results from application of the MRUH reasonably approximate watershed behavior.

Different combinations of T_c and C were used for applications of MRUH to predict the direct runoff hydrographs for 1,600 rainfall-runoff events in 90 Texas watersheds to determine the sensitivity of the peak discharges and time to peak to different T_c and C values. Five combinations of T_c and C were used:

(A) T_c estimated using Haktanir-Sezen equation and C_{vbc},

(B) T_c estimated using Johnstone-Cross equation and C_{vbc},

(C) T_c estimated using Williams equation and C_{vbc},

(D) T_c estimated using Kirpich equation and C_{vbc}, and

(E) T_c estimated using Kirpich equation and C_{lit}.

Figure 5.6 is a plot of the observed and computed peak discharges using C_{vbc} and T_c values calculated using the four different empirical equations (Fang et al. 2008). In comparison to observed peak discharges, modeled peak discharges using T_c estimated from the Haktanir-Sezen, Johnstone-Cross and Kirpich equations not only graphically look alike (Fig. 5.6) but also are similar with respect to four statistical parameters (Table 1): relative root mean square error, *RRMSE* (Eq. A.1 in Appendix A); coefficient of determination, R^2 (Eq.. A.2); Nash-Sutcliffe efficiency, *EF* (Eq. A.3); and peak relative error, *QB* (Eq. A.4). The results for *EF* and *RRMSE* using the Williams equation seem to be inferior to others. The fraction of modeled peak discharges that are within 1/3 of a log-cycle from the 1:1 line are summarized in Table 1 and ranged from 67.7% (Williams equation) to 89.1% (Kirpich equation). Fraction of storms with peak relative error (*QB*) less than ±25% and ±50% is listed in Table 5.1 for applications of MRUH with different combinations of T_c and *C*. Applications of MRUH with T_c estimated from Kirpich equation and back-computed C_{vbc} resulted in 73% of storms with *QB* less than ±50%. Use of C_{vbc} results in preservation of event runoff volume. Ideally, computed and observed peaks should plot precisely along the equal value line (black line in Fig. 5.6). However, the unit hydrograph is a mathematical model that is an incomplete description of the complexity of the combination of the rainfall-runoff process and runoff dynamics. Therefore, the relatively simple approach cannot fully capture the nuances of watershed dynamics and deviations from this ideal (the equal-value line) are expected. For example, Asquith and Roussel (2009) computed mean residual standard error about 1/3 of a log cycle for annual peak discharges at 638 streamflow gauging stations in Texas.

Figure 5.7 is a plot of the observed time to peak (T_p) and computed T_p values predicted using C_{vbc} and T_c values calculated using the four different empirical equations (Fang et al. 2008). For T_p, application of MRUH using T_c estimated from the Haktanir-Sezen, Johnstone-Cross and Kirpich equations produces the similar values of the quantitative measures: median value of T_p relative error (*TB*) and fraction of storms with *TB* less than ±25% and ± 50% (Table 5.2), and *TB* equal to ± 50% is graphically presented in Fig. 5.7 also. The T_p results using the Williams equation seem to be slightly inferior to others with respect to *TB* (Table 5.2). In summary, for predicting peak discharge and time to peak, use of T_c estimated from Williams equation with the MRUH produces less accurate results than those computed using the Kirpich, Haktanir-Sezen and Johnstone-Cross equations.

Simulated peak discharges derived using MRUH with the forward-computed (literature-based) runoff coefficient (C_{lit}) are compared against the results obtained from the back-computed (C_{vbc}) runoff coefficient (Fig. 5.8) when T_c values were estimated using the Kirpich equation. For the peak discharges predicted using C_{lit}, most of the values are above the equal value line (1:1 line). Based on visual inspection, about one-third of the peak discharges from C_{lit} (triangles) are quite far from the peak discharges using C_{vbc} (black circles). Peak discharges computed using C_{vbc} are superior to those using C_{lit} with respect to all statistical measures used to assess goodness of fit (Table 5.3). Therefore, use of literature based values (C_{lit}) will tend to generate estimates of peak discharge that exceed expected values (observations) when the C_{lit} values are interpreted as volumetric coefficients. Furthermore, literature-based estimates (C_{lit}) of the runoff coefficient yield results that do not preserve runoff volume when applied to measured rainfall-runoff events. In contrast, there is no difference in quantitative measures between the observed and predicted time to peak values, regardless of which runoff coefficient is used (Table 5.3). This is because T_p is controlled by time of concentration and rainfall hyetograph and the same T_c values were used with different runoff coefficients (Fig. 5.8) for each of 90 Texas watersheds.

Hence, the simulation results of peak discharge are more sensitive to the choice of the runoff coefficients (C) or rainfall loss model. Furthermore, the time to peak results are not related to C when MRUH was used.

5.4 Estimated Runoff Hydrographs from Different Unit Hydrograph Methods

In addition to application of the MRUH for 90 Texas watersheds, three other unit hydrograph models—the unit hydrograph developed using the Clark method (Clark 1945) with HEC–1's generalized basin shape (USACE 1981), the NRCS unit hydrograph (NRCS 1972), and the Gamma unit hydrograph (GUH) for Texas watersheds (Pradhan 2007)—were used to develop the direct runoff hydrograph for each rainfall-runoff event in the database. The GUH used in this study was that developed by researchers at Lamar University for TxDOT project 0–4193 "Regional Characteristics of Unit Hydrographs." Linear programming was used to develop unit hydrographs from observed rainfall hyetographs and runoff hydrograph, and the GUH was fitted to each derived unit hydrograph. Regression equations were developed for five-minute GUH parameters: peak discharge Q_p (in cfs) and time to peak T_p (in hours) (Pradhan 2007),

$$T_p = 0.55075\, A^{0.26998}\, L^{0.42612}\, S^{-0.06032}, \tag{5.5}$$

$$Q_p = 93.22352\, A^{0.83576}\, L^{-0.326}\, S^{0.5}, \tag{5.6}$$

where A is drainage area in square miles, L is main channel length in miles, and S is main channel slope (ft/mile, elevation difference in feet divided by main channel length in miles). The ordinates of the GUH can be obtained from (Viessman and Lewis 2003):

$$Q = Q_p \left({}^{t}\!/_{T_p} \right)^{\alpha} e^{[1-({}^{t}\!/_{T_p})]^{\alpha}}, \tag{5.7}$$

where Q is the discharge ordinate at time t and α is the shape parameter of GUH. Two of the three GUH parameters (Q_p, T_p, and α) are independent and the shape factor is determined from Q_p and T_p (Aron and White 1982).

Clark's (1945) instantaneous unit hydrograph (IUH) method is based on the time-area curve method (Bedient and Huber 2002). The Clark IUH method is one of the unit hydrographs available in the flood hydrograph package HEC-1 (USACE 1981) and the hydrologic modeling system HEC-HMS (USACE 2000). A synthetic time-area curve derived from a generalized basin shape is used to implement Clark's IUH in HEC-1 and HEC-HMS. The equations for the time-area curve are

$$AI = 1.414\, TI^{1.5}, \qquad 0 \le TI \le 0.5 \tag{5.8}$$

$$1 - AI = 1.414\, (1-TI)^{1.5}, \qquad 0.5 < TI < 1 \tag{5.9}$$

where AI is the cumulative area as a fraction of watershed area and TI is fraction of time of concentration. These equations are applicable to most basins (Bedient and Huber 2002). In the Clark method and HEC-1/HEC-HMS programs, the resulting hydrograph is routed through a linear reservoir at the outlet of a watershed. The linear reservoir routing was not implemented because this study is to only compare MRUH resulted from an equal time-area curve (such as a rectangular watershed) and Clark-HEC-1 UH resulted from a generalized watershed shape (ellipse-like shape having a non-uniform time-area curve). Both MRUH and Clark-HEC-1 UH implemented without routing assume that the outflow hydrograph results from pure translation of direct runoff to the outlet.

The NRCS dimensionless unit hydrograph was developed in the late 1940s (NRCS 1972). NRCS personnel analyzed a large number of unit hydrographs for watersheds of different sizes and in different geographic locations to develop a generalized dimensionless unit hydrograph in terms of t/T_p and q/Q_p. The peak flow, Q_p, for the unit hydrograph is computed by approximating the unit hydrograph with a triangular shape having base time of $8/3T_p$ and unit area (Viessman and Lewis 2003):

$$Q_p = \frac{484A}{T_p}, \tag{5.10}$$

where Q_p is the peak discharge (cfs) and A is the drainage area (mi^2).

Unit hydrographs developed using all four UH models, including the MRUH, for the watershed associated with the USGS streamflow-gaging station 08048520 Sycamore Creek in Fort Worth are shown in Fig. 5.9 (A). The purpose of this example is to illustrate the differences and similarities of the UH models. The shape of the MRUH is trapezoidal. Unit hydrographs from the Clark-HEC-1, the Gamma, and the NRCS methods are curvilinear. The unit hydrograph peak discharge from each model is different (Fig. 5.9A). However, the area under the UH curves is the same. This is because each UH corresponds to 1 inch of a uniform excess rainfall over 5-minute duration (one impulse).

Gamma, Clark-HEC-1, and NRCS unit hydrographs developed for each watershed were applied to the 1,600 rainfall-runoff events in the database to generate direct runoff hydrographs. The constant fraction rainfall loss method (rational method) was used to estimate rainfall excess for each rainfall event. The runoff coefficient C_{vbc} determined for each event was used. T_c determined using the Kirpich method (1940) was used for those methods that require T_c. As an illustrative example, the results for the observed and simulated direct runoff hydrographs for the rainfall event on 07/28/1973 at the USGS streamflow-gaging station 08048520 (Sycamore Creek in Fort Worth, Texas) by the four models (base flow was assumed to be zero) is presented in Fig. 5.9 (B). The watershed area is of 45.66 km^2 (17.63 square miles), the time of concentration is 3.96 hours from the Kirpich method, and the back-computed volumetric runoff coefficient, C_{vbc}, is 0.20.

Simulated peak discharges from the four UH methods are different, but comparable. For the particular example shown in Fig. 5.9 (B), the MRUH and the Clark-HEC-1 model appear to perform better than the other models with regard to prediction of peak discharge. For the time to peak, simulated values using the four methods agree reasonably well with the observed value (Fig. 5.9). Additionally, the area under the four simulated hydrographs matches the observed curve because event C_{vbc} was used. Although the drainage area of Sycamore Creek watershed exceeds that usually accepted for rational method application, results from the MRUH reasonably approximate watershed behavior.

The observed and modeled peak discharges from all four UH models developed using back-computed runoff coefficient, C_{vbc} and time of concentration from the Kirpich method are presented in Fig. 5.10 for 1,600 rainfall and runoff events. Modeled peak discharges from all the four UH models are similar (Fig. 5.10). Four statistical parameters *RRMSE*, R^2, *EF*, and *QB* (Appendix A) between the observed and modeled peak discharges were computed for evaluation of model performance (Loague and Green 1991; Cleveland et al. 2006) and are listed in Table 5.4. Based on the statistical measures, all the four UH models perform similarly. However, the GUH developed for Texas watersheds perform somewhat worse than the other three UH models (Table 5.4). Fractions of storms for each model meeting the acceptance tolerance of *QB* and *TB* are also listed in Table 5.4 (Eqs. A.4 and A.6). Using this acceptance approach, again all the models perform similarly.

5.5 Sensitivity of the MRUH to unit hydrograph duration

A sensitivity analysis was performed for peak discharges derived from application of MRUH using different unit hydrograph durations. The simulated runoff hydrographs were obtained for unit hydrograph durations of 5-, 10-, 20-, 30-, 40- and 50-minutes. To minimize error in developing discrete MRUH and DRH, a time interval of 10 minutes was used for hydrograph convolution when unit hydrograph durations exceeded 10 minutes.

Predicted runoff hydrographs for the rainfall event on 07/28/1973 for the watershed associated with the USGS streamflow-gaging station 08178600 Salado Creek San Antonio are shown in Fig. 5.11 as an illustrative example. No noticeable differences were visible both in terms of the peak and the shape of the hydrographs regardless of the unit hydrograph duration. The Nash-Sutcliffe efficiency and relative change in the peak discharge for the simulated results are presented in Table 5.5. Based on review of statistical measures, results are not sensitive to changes in unit hydrograph duration (Table 5.5, *EF* is derived from runoff hydrograph ordinates).

For the 1,600 rainfall events in the database, the median relative change in the peak discharge (Q_{RE}) for the changes in the UH durations are all 0% (Table 5.6). Fractions of storms with Q_{RE} less than ±5% and ±10% are listed in Table 5.6, and almost all of rainfall events (95 to 99%) has Q_{RE} less than ±10%. The application of MRUH is not sensitive to the selection of the unit hydrograph duration so long as the same time interval is used for hydrograph convolution.

5.6 Discussion and Summary

The modified rational method, MRM, is an extension of the rational method to produce simple runoff hydrographs for applications that do not warrant a more complex modeling approach. In this study, the MRM was revisited. The hydrographs developed using MRM can be considered an application of a special unit hydrograph termed the modified rational unit hydrograph, MRUH. The MRUH method was applied to develop unit hydrographs for 90 watersheds in Texas. Unit hydrograph convolution was used to determine the direct runoff hydrograph for 1,600 rainfall-runoff events associated with the Texas database. The purposes were (1) to evaluate the applicability of the method if blindly applied to watersheds of any size, and (2) to study the effects of runoff coefficient and the time of concentration on prediction of runoff hydrograph using the MRUH.

Runoff coefficients estimated using two approaches by Dhakal et al. (2011) were examined for application with the MRUH. The first was a watershed composite literature-based coefficient (C_{lit}) derived using the land-use information for the watershed and published C_{lit} values for various land-uses. The second was a back-computed volumetric runoff coefficient (C_{vbc}) determined by preserving the runoff volume and using observed rainfall and runoff data. Times of concentration for study watersheds were estimated by Fang et al. (2008) from four empirical equations, which were based on several watershed characteristics. Predicted and observed discharge hydrographs were reported and compared. Simulated peak discharges and times to peak from MRUH agree reasonably well with observed values. The drainage area of the study watersheds (average 440 km^2 or 15.6 mi^2) is greater than that usually accepted for rational method application (0.8 km^2 or 0.3 mi^2), yet results from the MRUH reasonably approximate watershed behavior regardless of watershed size. Simulated peak discharges are more sensitive to the choice of the runoff coefficient than the time of concentration. Simulated times to peak are moderately sensitive to the time of concentration but independent of the runoff coefficient. A sensitivity analysis of the MRUH to the unit hydrograph duration was performed. The MRUH is not sensitive to the selection of the unit hydrograph duration so long as the same time interval is used for hydrograph convolution.

Three other unit hydrograph models, the Clark (using HEC–1's generalized basin equations), the Gamma, and the NRCS unit hydrographs were also used to compute the direct runoff hydrograph for each rainfall-runoff event in the database. Runoff hydrographs simulated using all four methods were similar. Simulated peak discharges for all events in the database were similar regardless of statistical or quantitative measures used for comparison. For time to peak, simulated values using all four models agree reasonably well with observed values. The four UH models produce similar values of statistical and quantitative measures for both peak discharges and time to peak.

Three general conclusions for MRUH are: (1) Being a unit hydrograph, it can be applied to nonuniform rainfall distributions and for watersheds with drainage areas greater than typically used with either the rational method or the modified rational method (that is, a few hundred acres). (2) The MRUH performs about as well as other unit hydrograph methods used in this study for predicting the peak discharge and time to peak of the direct runoff hydrograph, so long as the same rainfall loss model is used. (3) Modeled peak discharges from application of the MRUH are more sensitive to the selection of runoff coefficient, less sensitive to T_c, and not sensitive to the selection of the unit hydrograph duration. In predicting peak discharges and runoff hydrographs for engineering design, rainfall loss estimation results in greater uncertainty and contributes more model errors than variations of UH methods and model parameters for UH.

5.7 Notation

The following symbols are used in this paper:

α = shape parameter of gamma unit hydrograph;

A = drainage area in hectares or acres;

AI = cumulative area as a fraction of watershed area;

C = runoff coefficient;

C_{lit} = composite literature-based runoff coefficient;

C_{vbc} = back-computed volumetric runoff coefficient;

D = storm duration;

EF = Nash-Sutcliffe efficiency;

I = average rainfall intensity (mm/hr or in. /hr) with the duration equal to time of concentration;

L = main channel length in mile;

m_o = the dimensional correction factor (1.008 in English units, 1/360 = 0.00278 in SI units);

Q_p = peak runoff rate in m^3/s or ft^3/s;

Q_p' = peak runoff rate of the modified rational hydrograph for the case when the storm duration is less than the time of concentration of the drainage area;

QB = peak relative error between the observed and simulated peak discharges;

$q(t)$ = direct runoff hydrographs (in watershed depth per time) by convolution;

R^2 = coefficient of determination;

RRMSE = relative root mean square error;

S = main channel slope (ft/mile);

t = shape parameter of gamma unit hydrograph;

TB = relative bias of direct runoff hydrograph time to peak;

TI = fraction of time of concentration;

T_c = time of concentration;

T_p = time to peak;

u(t) = rectangular instantaneous unit hydrograph response function;

5.8 Appendix A: *Statistical Measures to Evaluate Model Performance*

Four statistical measures were used to analyze model results. They are the relative root mean square error (*RRMSE*), the coefficient of determination (R^2), the Nash-Sutcliffe efficiency (*EF*), and the peak relative error (*QB*) between the observed and simulated peak discharges and times to peak (Loague and Green 1991; Feyen et al. 2000; Cleveland et al. 2006). The equations used to compute these measures are:

$$RRMSE = \frac{\left[\sum_{i=1}^{n} (P_i - O_i)^2 \big/ n\right]^{0.5}}{\overline{O}} , \tag{A.1}$$

$$R^2 = \left(\frac{\sum_{i=1}^{n} (O_i - \overline{O})(P_i - \overline{P})}{\sqrt{\sum_{i=1}^{n} (O_i - \overline{O})^2} \sqrt{\sum_{i=1}^{n} (P_i - \overline{P})^2}} \right)^2 , \tag{A.2}$$

$$EF = \left(\sum_{I=1}^{N} (O_i - \overline{O})^2 - \sum_{i=1}^{n} (P_i - O_i)^2 \right) \Big/ \sum_{i=1}^{n} (Oi - \overline{O})^2 \text{, and} \tag{A.3}$$

$$QB = \frac{P_i - O_i}{O_i}, \tag{A.4}$$

where, P_i are the simulated peak discharge values, O_i are the observed peak discharge values, n is the number of observations and $\overline{O}$ is the mean of the observed peak discharge values. *RRMSE* is a measure of overall spread of the residuals with respect to the mean observed value. The target *RRMSE* value is 0 for acceptance of a model (Feyen et al. 2000). The coefficient of determination R^2 is a measure of the proportion of the total variability in observed data that can be explained by the model and ranges from 0 to 1. According to Moriasi et al. (2007) values of R^2 greater than 0.5 are considered acceptable for a model. Although R^2 and *EF* values are used often for the evaluation of a model, Legates and McCabe (1999) suggested that *EF* is a more appropriate measure for goodness-of-fit. For hydrograph simulation, a good agreement between the simulated and the measured data is reached when *EF* is higher than 0.7 (Bennis and Crobeddu 2007). Peak relative error, *QB,* is the difference in magnitude between the modeled and observed peak divided by observed peak discharge. Similar to *RRMSE*, values of *QB* near to 0 indicate correspondence between modeled and observed values. Cleveland et al. (2006) suggested the following range of the *QB* for the acceptance of model performance:

$$-0.25 \le QB \le 0.25 \tag{A.5}$$

For evaluation of the time to peak results, relative bias of direct runoff hydrograph time to peak (*TB*) was estimated (Zhao and Tung 1994) for each storm event using:

$$TB = \frac{t_{pm} - t_{po}}{t_{po}}, \tag{A.6}$$

where, t_{pm} is the modeled time to peak and t_{po} is the observed time to peak. A positive TB indicates that the observed peak occurs sooner (smaller) than the modeled peak (i.e. the model predicts a late peak). Similarly, a negative TB indicates that the observed peak occurs later (larger) than the modeled peak (i.e. the model predicts an early peak) [Cleveland et al. 2006].

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Table 5.1 Quantitative Measures of the Success of the Peak Discharges Modeled Using MRUH with Back-Computed Volumetric Runoff Coefficient (C_{vbc}) and Time of Concentration (T_c) Estimated Using Four Equations.

Statistical Parameters	Haktanir-Sezen equation [1]	Johnstone-Cross equation [2]	Williams equation [3]	Kirpich equation [4]
RRMSE (Eq. A.1) [5]	0.87	0.78	1.03	0.75
R^2 (Eq. A.2)	0.66	0.72	0.70	0.75
EF (Eq. A.3)	0.64	0.71	0.49	0.73
Median value of *QB* (Eq. A.4)	-0.18	0.04	-0.40	-0.09
Fraction of storms with -0.25 ≤ *QB* ≤ 0.25	0.35	0.39	0.25	0.40
Fraction of storms with -0.5 ≤ *QB* ≤ 0.5	0.68	0.70	0.60	0.73
% of events within ± 1/3 of a log cycle	81.5	86.4	67.7	89.1

[1] T_c computed using Haktanir-Sezen equation ranged from 0.8 to 17.6 hours in the study watersheds, with median and mean values of 2.7 hours and 3.7 hours, respectively.

[2] T_c computed using Johnstone-Cross equation ranged from 0.7 to 7.7 hours in the study watersheds, with median and mean values of 1.9 hours and 2.3 hours, respectively

[3] T_c computed using Williams equation ranged from 1.2 to 31.4 hours in the study watersheds, with median and mean values of 4.0 hours and 5.9 hours, respectively

[4] T_c computed using Kirpich equation ranged from 0.6 to 16.2 hours in the study watersheds, with median and mean values of 2.3 hours and 3.2 hours, respectively

[5] Statistical parameters are defined in Appendix A.

Table 5.2 Quantitative Measures of the Success of the Time to Peak Modeled Using MRUH with Back-Computed Volumetric Runoff Coefficient (C_{vbc}) and Time of Concentration Estimated Using Four Equations.

Statistical Parameters	Haktanir-Sezen equation	Johnstone-Cross equation	Williams equation	Kirpich equation
Median value of *TB* (Eq. A.6)	0.00	-0.07	0.09	-0.03
Fraction of storms with -0.25 ≤ *TB* ≤ 0.25	0.52	0.52	0.46	0.53
Fraction of storms with -0.5 ≤ *TB* ≤ 0.5	0.70	0.71	0.64	0.71

Table 5.3 Quantitative Measures of the Success of the Peak Discharges and Time to Peak Modeled Using MRUH with Time of Concentration Estimated Using Kirpich Equation and Runoff Coefficients Estimated Using Two Different Methods (C_{vbc} and C_{lit}).

Statistical Parameters	C_{vbc}	C_{lit}
RRMSE (Eq. A.1)	0.75	1.07
R^2 (Eq. A.2)	0.75	0.48
EF (Eq. A.3)	0.73	0.45
Median value of *QB* (Eq. A.4)	-0.09	0.44
Fraction of storms with -0.25 ≤ *QB* ≤ 0.25	0.40	0.22
Fraction of storms with -0.5 ≤ *QB* ≤ 0.5	0.73	0.46
% of events within ± 1/3 of a log cycle	89.1	63.3
Median value of *TB* (Eq. A.6)	-0.03	-0.03
Fraction of storms with -0.25 ≤ *TB* ≤ 0.25	0.53	0.53
Fraction of storms with -0.5 ≤ *TB* ≤ 0.5	0.71	0.71

Table 5.4 Quantitative Measures of the Success of the Peak Discharges Modeled Using Four Unit Hydrograph Models for 1,600 Rainfall-Runoff Events in 90 Texas Watersheds.

Statistical Parameters	MRUH	Gamma UH	Clark-HEC-1 UH	NRCS UH
RRMSE (Eq. A.1)	0.75	0.87	0.74	0.72
R^2 (Eq. A.2)	0.75	0.80	0.74	0.78
EF (Eq. A.3)	0.73	0.65	0.74	0.76
Median value of *QB* (Eq. A.4)	-0.09	-0.31	0.04	-0.10
Fraction of storms with -0.25 ≤ *QB* ≤ 0.25	0.40	0.32	0.39	0.39
Fraction of storms with -0.5 ≤ *QB* ≤ 0.5	0.73	0.71	0.70	0.76
% of events within ± 1/3 of a log cycle	89.1	81.2	87.0	90.6
Median value of *TB* (Eq. A.6)	-0.03	0.03	-0.04	0.00
Fraction of storms with -0.25 ≤ *TB* ≤ 0.25	0.53	0.53	0.53	0.53
Fraction of storms with -0.5 ≤ *TB* ≤ 0.5	0.71	0.72	0.74	0.73

Table 5.5 Sensitivity of Peak Discharges Modeled Using MRUH on Unit Hydrograph Duration for the Rainfall Event on 07/28/1973 for the Watershed Associated with the USGS Streamflow-gaging Station 08178600 Salado Creek, San Antonio, Texas.

UH Duration	*EF*[1]	Change in UH duration (minutes)	Relative (%) change in Q_p[2]
5-min	0.89		
10-min	0.90	5 to 10	0.00
20-min	0.91	10 to 20	-0.30
30-min	0.92	20 to 30	-2.42
40-min	0.91	30 to 40	1.27
50-min	0.92	40 to 50	-2.07

[1] Nash -Sutcliffe efficiency (Eq. A.3) derived from runoff hydrograph ordinates

[2] Relative change in $Q_p = Q_{RE}$ (%) = $(Q_{p10} - Q_{p5})/Q_{p5} \times 100\%$, where Q_{p10} and Q_{p5} are peak discharges calculated using unit hydrograph durations of 10 and 5 minutes, respectively. Q_{RE} for other UH durations is calculated in a similar way.

Table 5.6 Sensitivity of Peak Discharges Modeled Using MRUH on Unit Hydrograph Duration for 1,600 Rainfall Events in 90 Texas Watersheds.

Change in UH duration (minutes)	Median value of Q_{RE}[1]	Fraction of storms with $-5\% \le Q_{RE} \le 5\%$	Fraction of storms with $-10\% \le Q_{RE} \le 10\%$
5 to 10	0.00	0.93	0.99
10 to 20	0.00	0.98	0.99
20 to 30	0.00	0.96	0.99
30 to 40	0.00	0.90	0.96
40 to 50	0.00	0.87	0.95

[1] Q_{RE} is defined in Table 5.

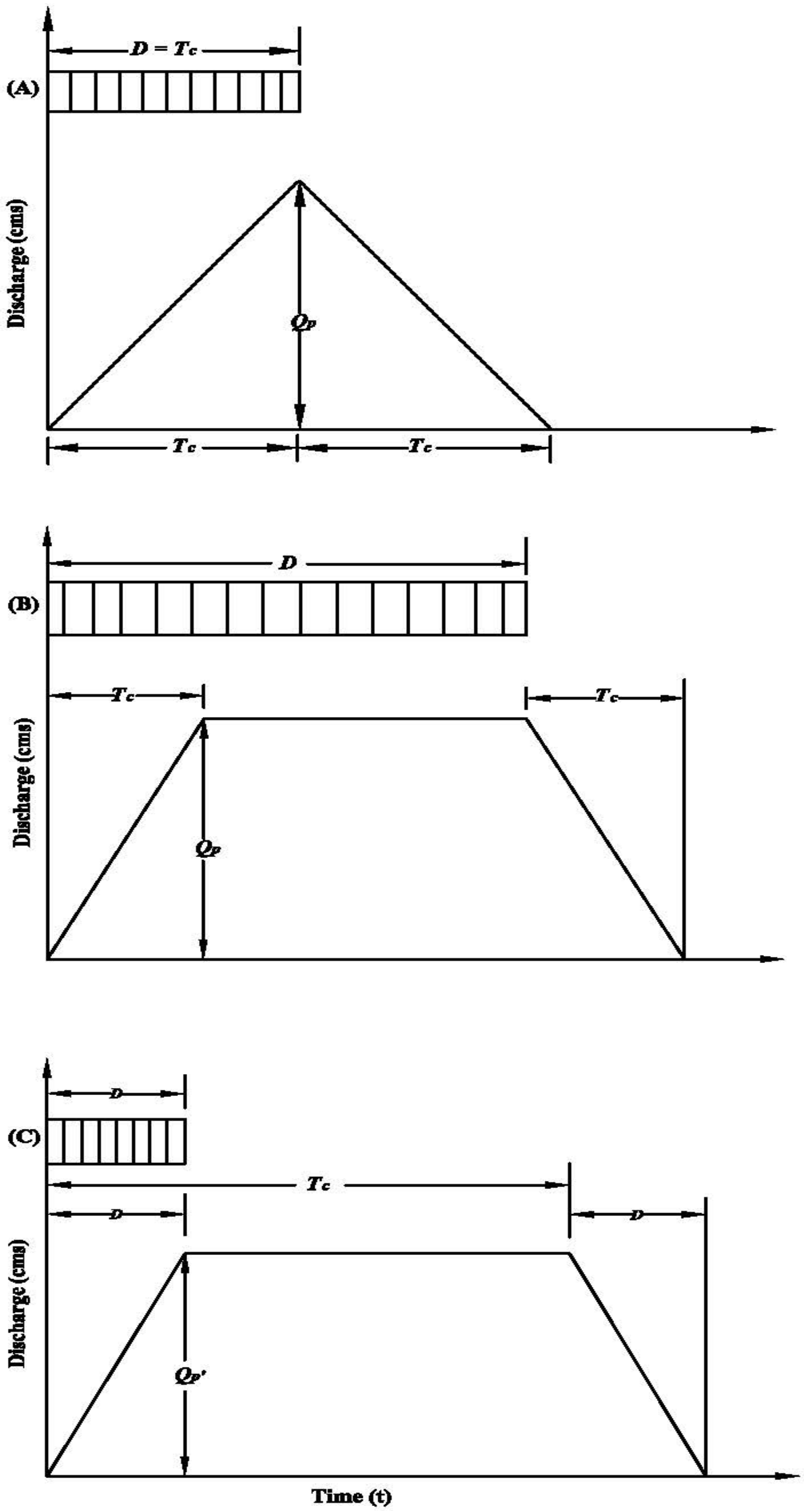

Fig 5.1 The modified rational hydrographs for three different cases: (A) Duration of rainfall (D) is equal to time of concentration (T_c), (B) Duration of rainfall is greater than T_c, and (C) Duration of rainfall is less than T_c.

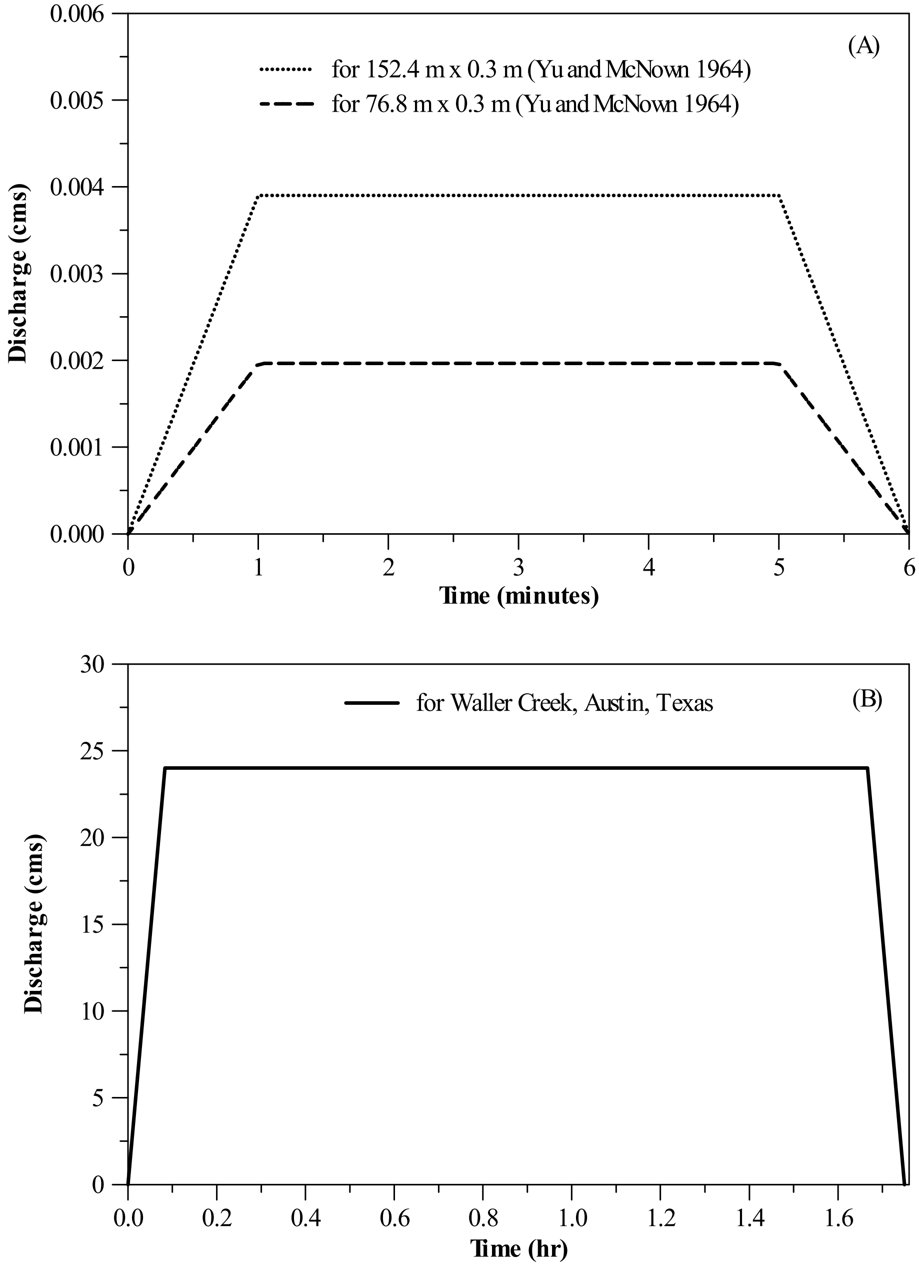

Fig. 5.2 The modified rational unit hydrographs (MRUH) developed for: (A) two lab settings from Yu and McNown (1964) and (B) for the watershed associated with USGS streamflow-gaging station 08157000 Waller Creek, Austin, Texas.

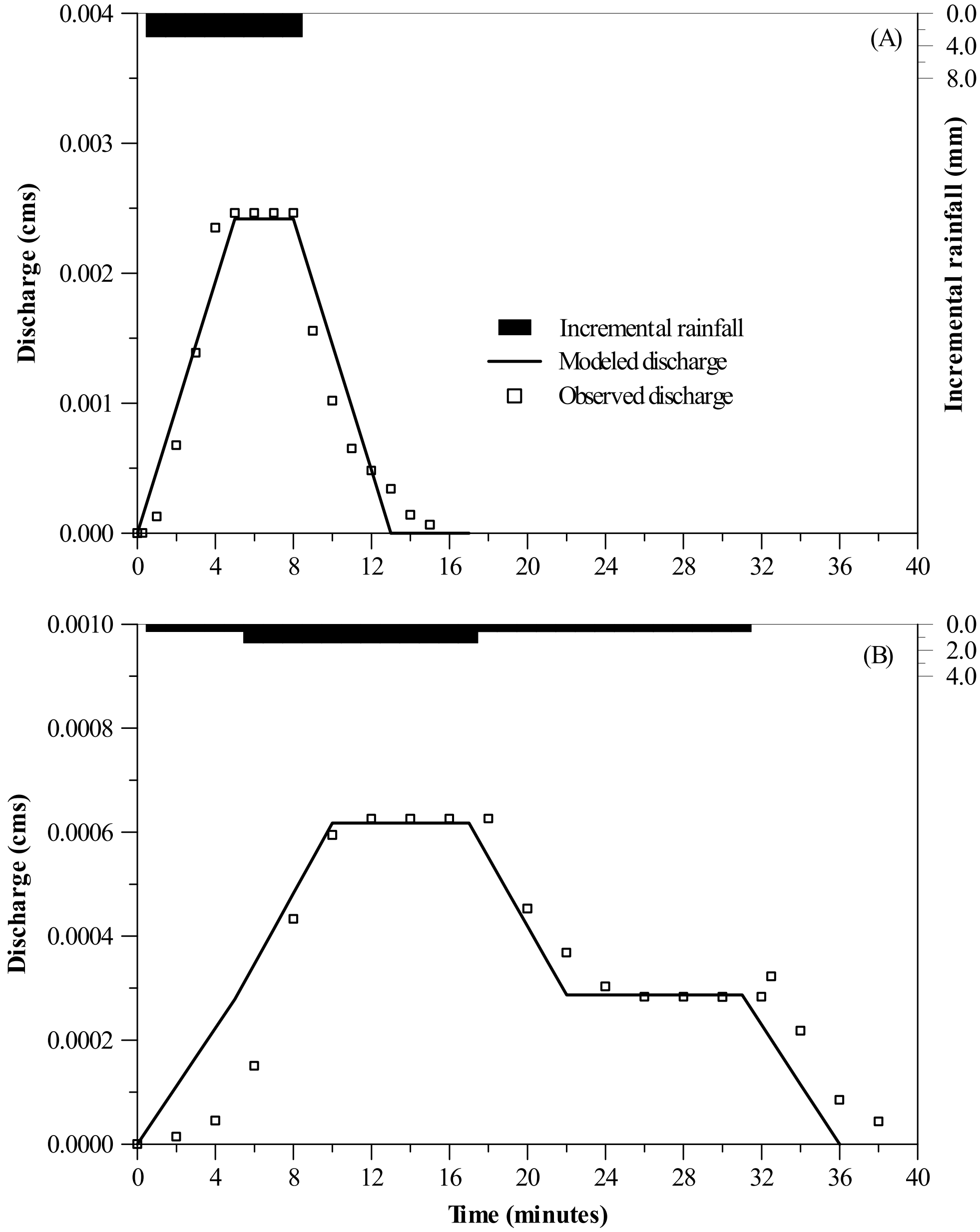

Fig. 5.3 Incremental rainfall hyetograph and observed and modeled runoff hydrographs using the MRUH for the two lab tests on concrete surfaces: (A) 152.4 m × 0.3 m and (B) 76.8 m × 0.3 m reported by Yu and McNown (1964).

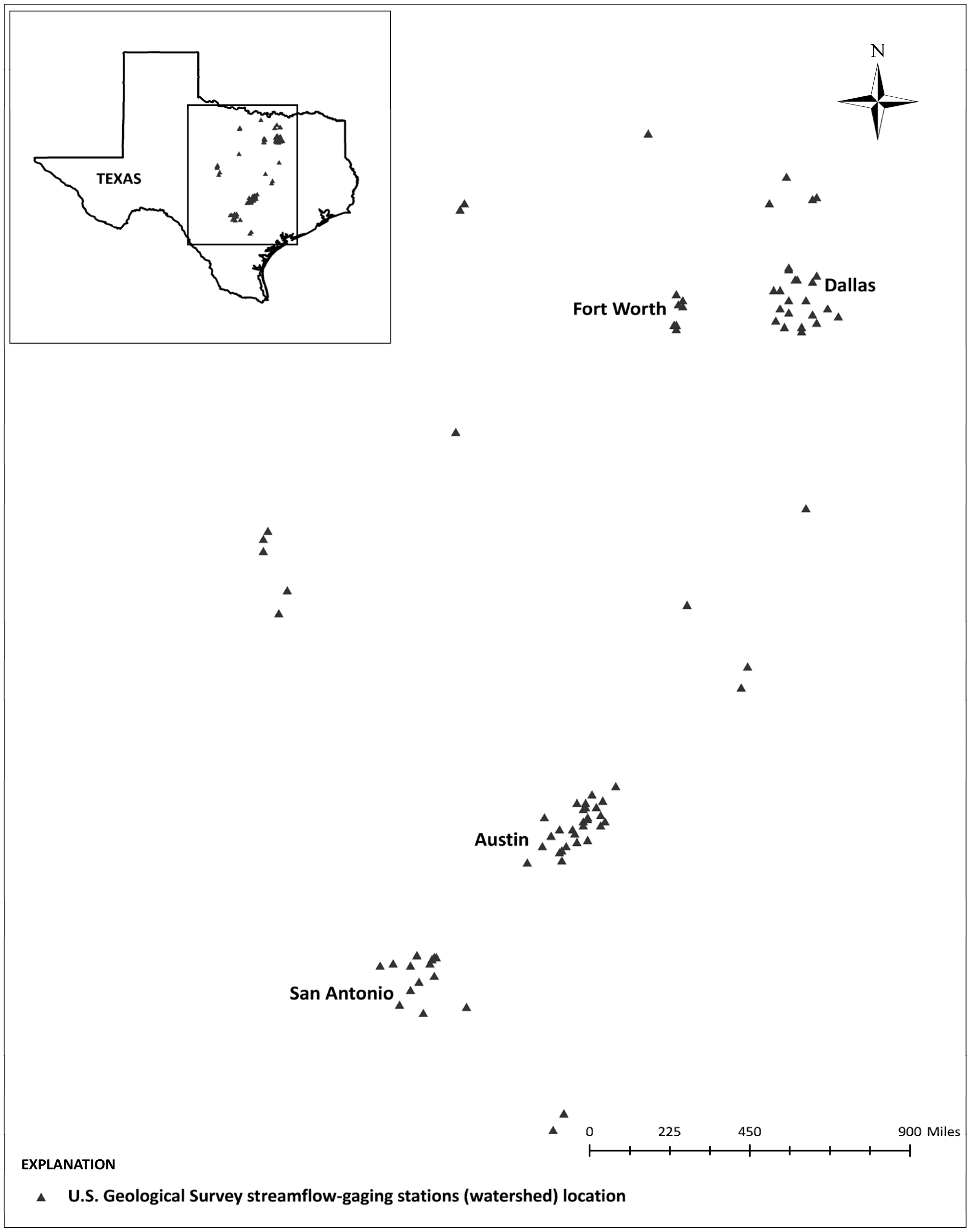

Fig. 5.4 Map showing the U.S. Geological Survey streamflow-gaging stations (dots) associated with the watershed locations in Texas.

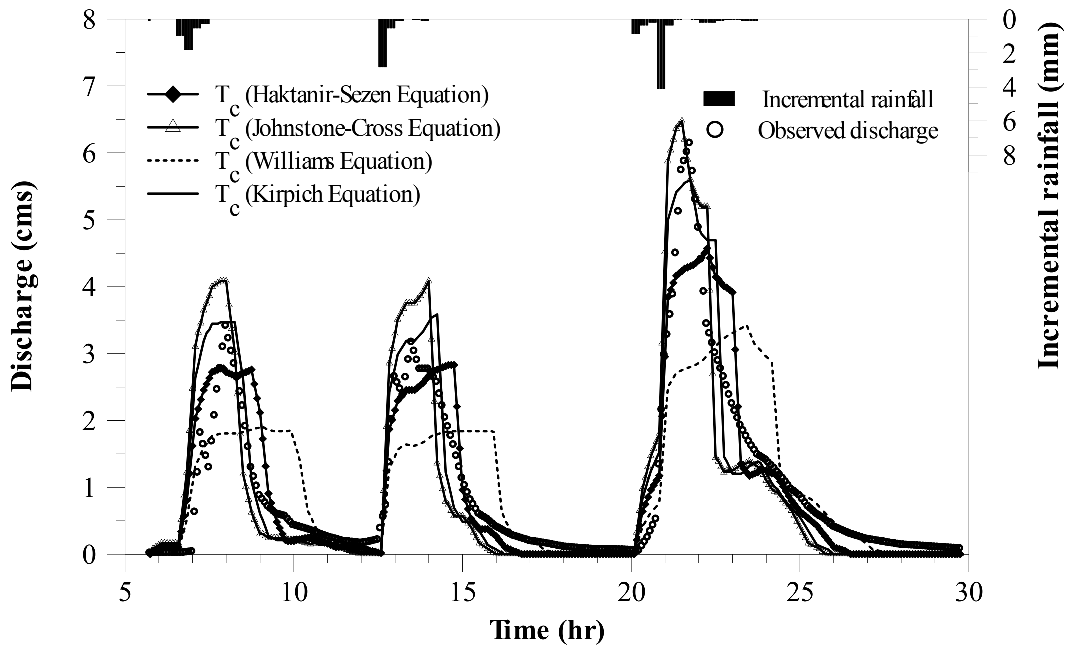

Fig. 5.5 Sensitivity of Peak Discharges Modeled using MRUH on Unit Hydrograph Duration for the Rainfall Event on 07/28/1973 for the Watershed Associated with the USGS Streamflow-gaging Station 08178600 Salado Creek, San Antonio, Texas.

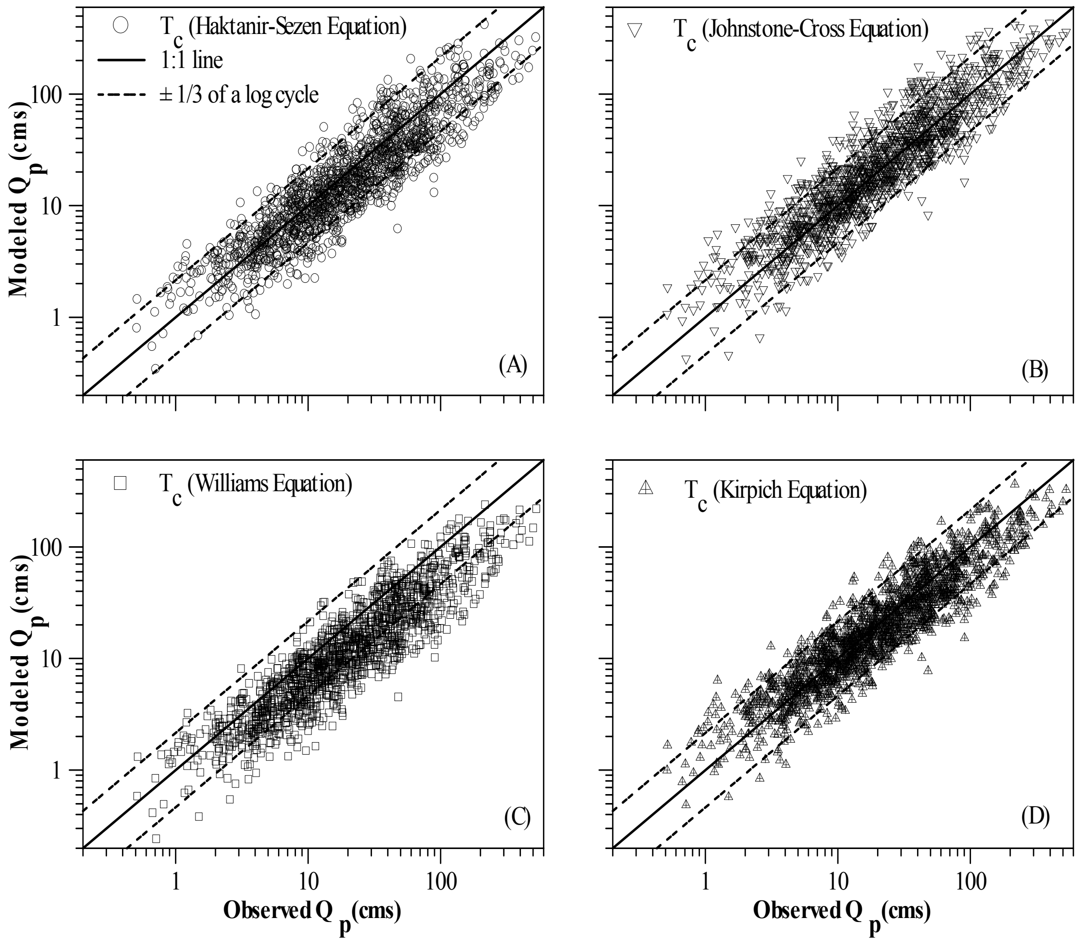

Fig. 5.6 Modeled versus observed peak discharges for 1,600 rainfall-runoff events in 90 Texas watersheds. Modeled results were developed from MRUH using event C_{vbc} and T_c estimated using four different methods: (A) Haktanir-Sezen equation, (B) Johnstone-Cross equation, (C) Williams equation and (D) Kirpich equation.

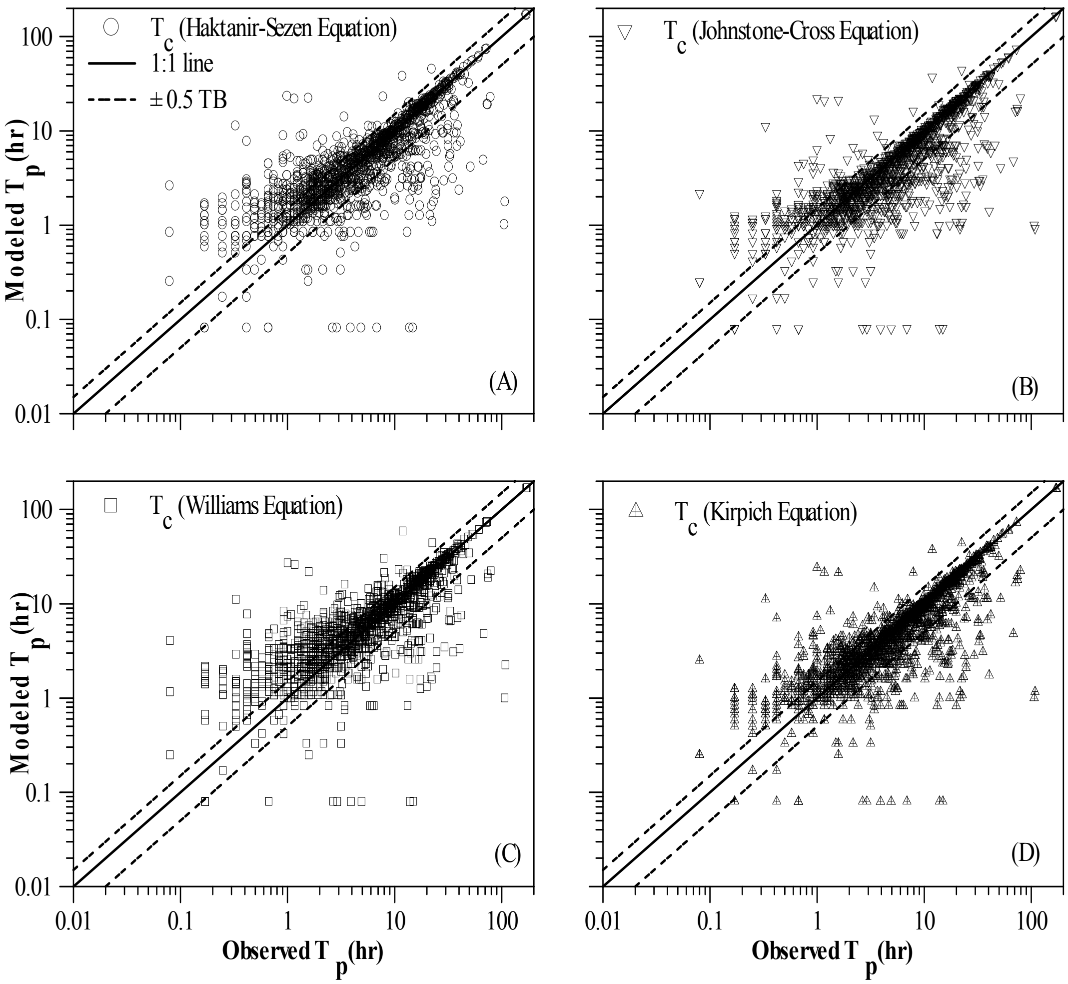

Fig. 5.7 Modeled versus observed time to peak for 1,600 rainfall-runoff events in 90 Texas watersheds. Modeled results were developed from MRUH using event C_{vbc} and T_c estimated using four different methods: (A) Haktanir-Sezen equation, (B) Johnstone-Cross equation, (C) Williams equation and (D) Kirpich equation.

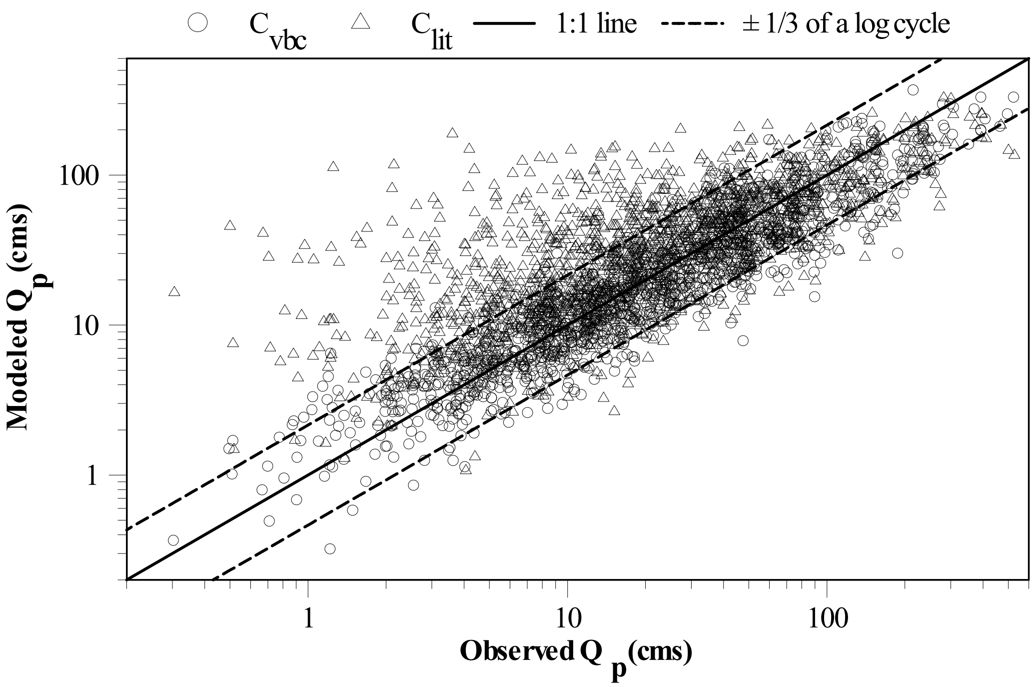

Fig. 5.8 Modeled versus observed peak discharges developed from MRUH using C_{vbc} (triangles) and C_{lit} (circles) with time of concentration estimated using Kirpich equation for 90 Texas watersheds.

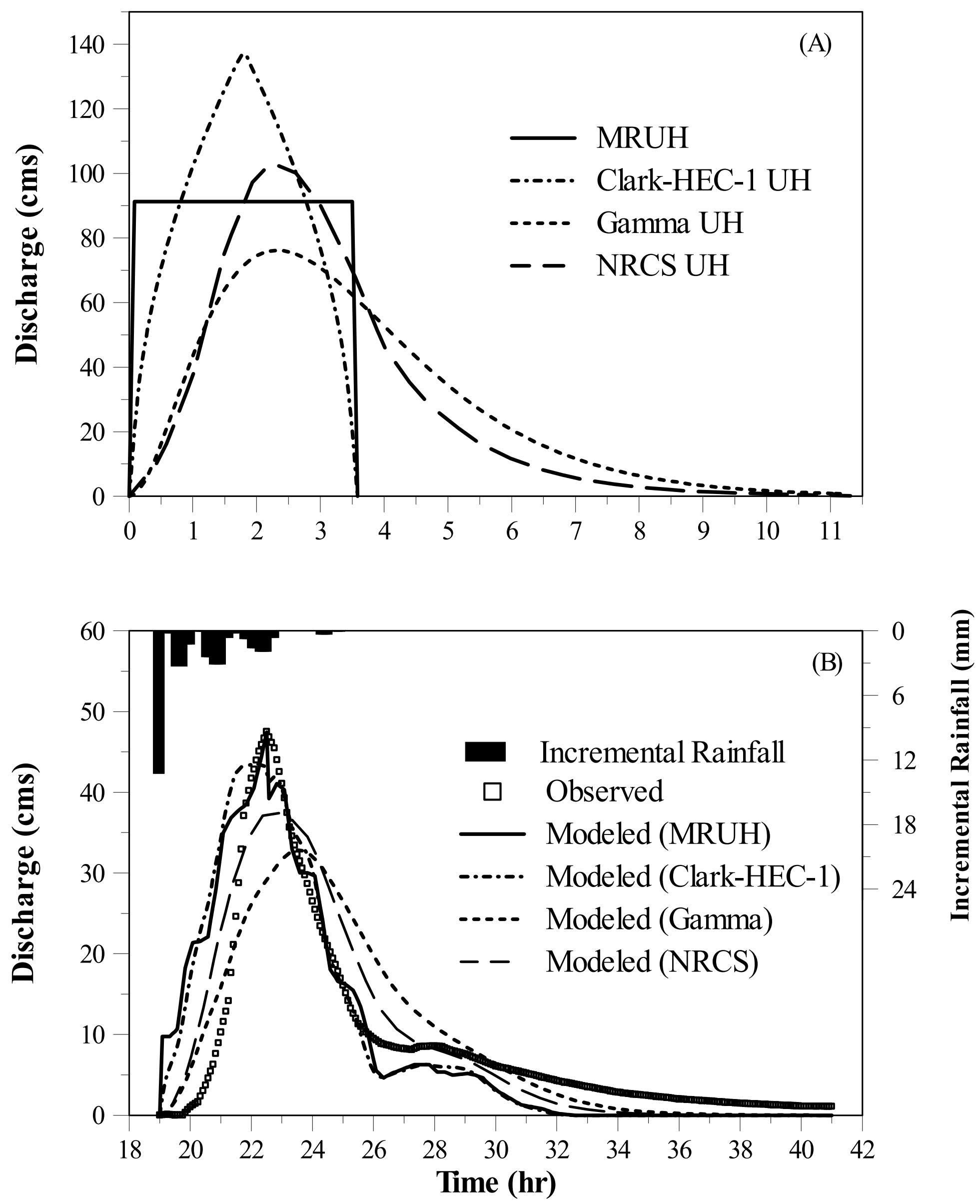

Fig. 5.9 (A) Modified rational, Gamma, Clark-HEC-1, and NRCS unit hydrographs developed for the watershed associated with USGS streamflow-gaging Station 08048520 Sycamore, Fort Worth, Texas; and (B) Rainfall hyetograph, observed and modeled runoff hydrographs using the four different unit hydrographs for the rainfall event on 07/28/1973 for the same watershed.

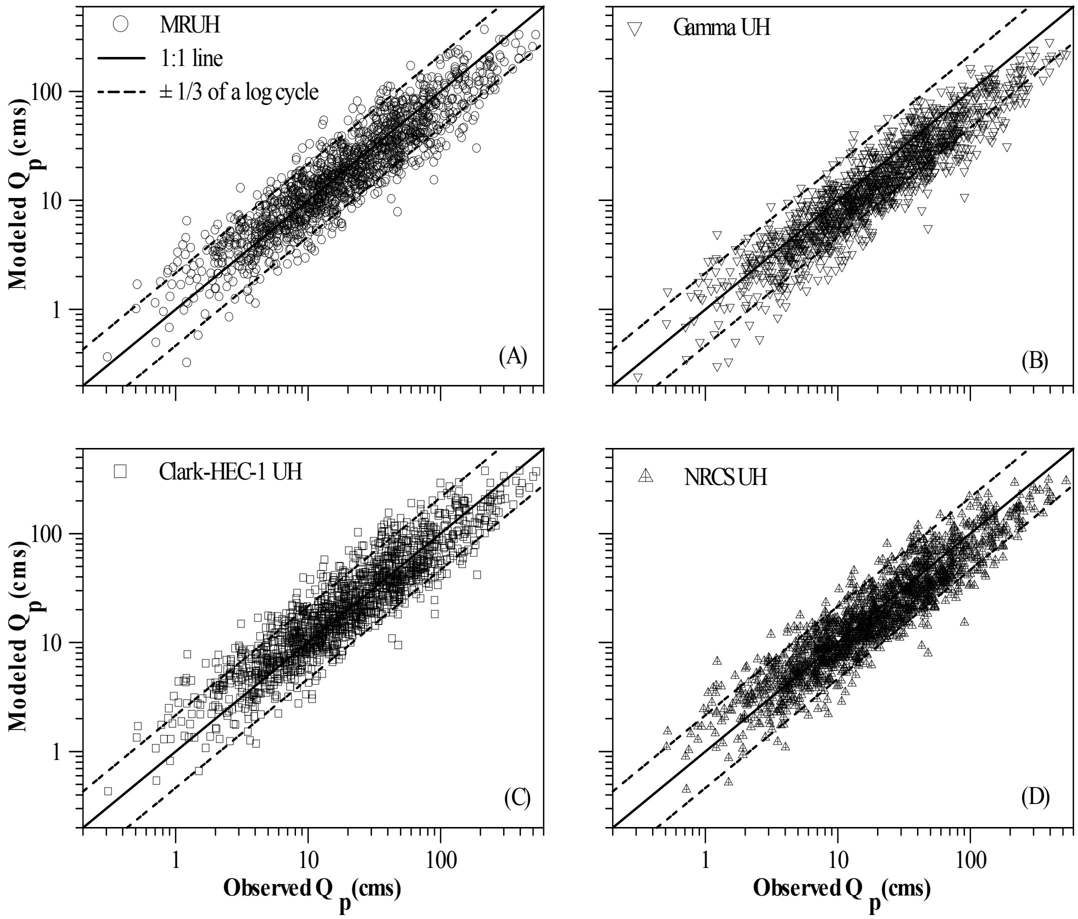

Fig. 5.10 Modeled versus observed peak discharges using: (A) MRUH, (B) Gamma UH, (C) Clark-HEC-1 UH, and (4) NRCS UH for 1,600 rainfall-runoff events in 90 Texas watersheds.

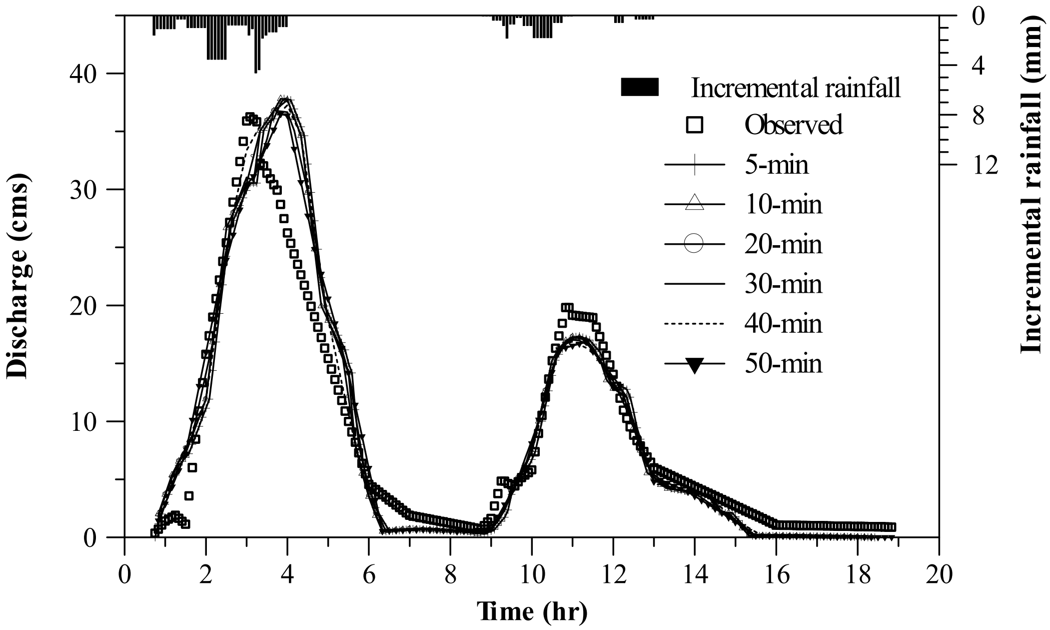

Fig. 5.11 Observed and modeled runoff hydrographs using MRUH with six unit hydrograph durations (5, 10, 20, 30, 40 and 50 minutes) for the rainfall event on 07/28/1973 for the watershed associated with the USGS streamflow-gaging station 08178600 Salado Creek, San Antonio, Texas.

Chapter 6 Conclusions and Recommendations

6.1 Conclusions

This research work is a part of TxDOT Project 0-6070 "Use of the Rational and the Modified Rational Methods for TxDOT Hydraulic Design". The objective of the project is to evaluate appropriate conditions for the use of the rational method and modified rational methods for designs on small watersheds; evaluate and refine, if necessary, current tabulated values of the runoff coefficient and construct guidelines for TxDOT analysts for the selection of appropriate parameter values for Texas conditions. The objective is achieved through four different phases of work.

For the first phase of our study, volumetric runoff coefficients were estimated for 90 Texas watersheds using three different methods–(1) a watershed composite, literature-based coefficient (C_{lit}) was derived from land-use information for the watershed and published C_{lit} values for appropriate land-uses, (2) back-computed volumetric runoff coefficient (C_{vbc}) was estimated by the ratio of total runoff depth to total rainfall depth for individual storm events and, (3) rank-ordered volumetric runoff coefficient (C_{vr}) was determined from the rank-ordered data; similar to the procedure used by Schaake et al. (1967). The median value of C_{lit} for developed watersheds exceeds that for undeveloped watersheds. Watershed-median values of C_{vbc} for 45 developed watersheds in Texas with percent imperviousness greater than 15% are consistent with median values of runoff coefficient R_v reported for 60 NURP watersheds by the USEPA.

The key conclusions of this phase of study are:

- Published runoff coefficients, even though they were not developed from observed rainfall-runoff measurements and instead resulted from a survey on engineering practices in 1950s, reflect the physical meanings of the original runoff coefficients introduced by Kuichling in 1889 — the runoff coefficient is related to the percent impervious area within the watershed. Therefore, published runoff coefficients remain useful for engineering design of drainage systems.
- The distribution of C_{vr} is different from that of C_{lit} with about 80 percent of C_{lit} value greater than C_{vr} value. This result might indicate that literature-based runoff coefficients overestimate peak discharge for drainage design when used with the rational method.
- Volumetric runoff coefficients are useful in transforming rainfall depth to runoff depth such as is done in the curve number method (SCS 1963) and for watershed rainfall-runoff modeling, e.g., the fractional loss model (McCuen 1998, p. 493).

For the second phase of our study, rate-based runoff coefficients, C_{rate}, were estimated using two different methods. First, C_{rate} was estimated using rational equation; the rainfall intensity was computed as the maximum intensity for a moving time window of duration T_c before and up to the time to peak, T_p. Second, the frequency-matching approach (Schaake et al., 1967) was used to extract a representative runoff coefficient (C_r) for each watershed. The key conclusions of this phase of study are:

- The rate-based C is dependent on rainfall intensity averaging time t_{av} used for the study, because estimates of the runoff coefficient based on observed data cannot be decoupled from the selection of the time-response characteristic.
- The distributions of the watershed-average and watershed-median C_{rate} are similar. The C_r values for the developed watersheds are consistently greater than those for the undeveloped watersheds.
- The values of C_r and C_{rate} were compared with the literature based runoff coefficients (C_{lit}) developed from land-use data for these study watersheds. About 75 percent of C_{lit} values are greater than C_r.
- For typical applications of the rational method in urban watersheds, watershed C_{lit} is less than C_r; using smaller C_{lit} would underestimate Q_p for design.

For the third phase of our study, the runoff coefficients $C(T)$ for different return periods (T) were developed for the 36 undeveloped Texas watersheds using previously published regional regression equations of peak discharge and county-based tabulated empirical coefficients for a model of rainfall intensities at different T. The frequency factors $C_f(T) = C(T)/C(10)$ determined in this study exceed those values in textbooks such as Gupta (1989) and Viessman and Lewis (2003) and those from TxDOT (2002) when $T > 10$ years. The key conclusions of this phase are:

- $C(T)$ values increase with T and these increases are more than previously thought.
- The frequency factors determined for the 36 Texas watersheds and the 72 Kansas watersheds (Young et al. 2009), larger than those mostly found in literature, are for undeveloped watersheds with relatively small percent impervious areas.

- The frequency factors mostly found in the literature, smaller than those determined for the 36 Texas watersheds, are appropriate for urban watersheds with relatively large percentages of impervious area, as supported and presented in literature (e.g., DRCG 1969; Stubchaer 1975; Jens 1979; Gupta 1989; Viessman and Lewis 2003; TxDOT 2002).
- When the frequency factor is applied, if resulted $C(T)$ is greater than unity, Jens (1979), Gupta (1989) and TxDOT (2002) suggested setting $C(T)$ equal to 1.

The modified rational method, MRM, is an extension of the rational method to produce simple runoff hydrographs for applications that do not warrant a more complex modeling approach. For the fourth phase of our study, the MRM was revisited. The hydrographs developed using MRM can be considered an application of a special unit hydrograph termed the modified rational unit hydrograph, MRUH. The MRUH method was applied to develop unit hydrographs for 90 watersheds in Texas. Runoff coefficients estimated using two approaches were examined for application with the MRUH. The first was a watershed composite literature-based coefficient (C_{lit}) derived using the land-use information for the watershed and published C_{lit} values for various land-uses. The second was a back-computed volumetric runoff coefficient (C_{vbc}) determined by preserving the runoff volume and using observed rainfall and runoff data. Times of concentration for study watersheds were estimated by Fang et al. (2008) from four empirical equations, which were based on several watershed characteristics. Simulated peak discharges and times to peak from MRUH agree reasonably well with observed values. The drainage area of the study watersheds (average 440 km^2 or 15.6 mi^2) is greater than that usually accepted for rational method application (0.8 km^2 or 0.3 mi^2), yet results from the MRUH reasonably approximate watershed behavior regardless of watershed size. Simulated peak discharges are more sensitive to the choice of the runoff coefficient than the time of concentration. Simulated times to peak are moderately sensitive to the time of concentration but independent of the runoff coefficient. The MRUH is not sensitive to the selection of the unit hydrograph duration so long as the same time interval is used for hydrograph convolution. Three other unit hydrograph models, the Clark (using HEC–1's generalized basin equations), the Gamma, and the NRCS unit hydrographs were also used to compute the direct runoff hydrograph for each rainfall-runoff event in the database. Runoff hydrographs simulated using all four methods were similar. Simulated peak discharges for all events in the database were similar regardless of statistical or quantitative measures used for comparison. For time to peak, simulated values using all four models agree reasonably well with observed values. The four UH models produce similar values of statistical and quantitative measures for both peak discharges and time to peak. Three key conclusions for MRUH are:

- Being a unit hydrograph, it can be applied to nonuniform rainfall distributions and for watersheds with drainage areas greater than typically used with either the rational method or the modified rational method (that is, a few hundred acres).
- The MRUH performs about as well as other unit hydrograph methods used in this study for predicting the peak discharge and time to peak of the direct runoff hydrograph, so long as the same rainfall loss model is used.
- Modeled peak discharges from application of the MRUH are more sensitive to the selection of runoff coefficient, less sensitive to T_c, and not sensitive to the

selection of the unit hydrograph duration. In predicting peak discharges and runoff hydrographs for engineering design, rainfall loss estimation results in greater uncertainty and contributes more model errors than variations of UH methods and model parameters for UH.

6.2 Recommendations

Based on the analysis of the volumetric runoff coefficients for 45 developed watersheds in Texas and 60 NURP watersheds, a polynomial regression equation was recommended which can be used to estimate volumetric runoff coefficients for developed urban watersheds that are similar to the 45 developed watersheds in Texas:

$$C_v = 1.66IMP^3 - 2.11IMP^2 + 1.30IMP + 0.04 \tag{6.1}$$

Above equation is useful mainly for the urban (developed) watersheds. Further study is recommended in future to correlate runoff coefficients for undeveloped watersheds to soil types and other watersheds characteristics like slope, initial soil moisture condition and land-use.

A single equation was recommended to estimate the rate-based runoff coefficient *C** ("*C*-star") for the unified rational method (URAT) developed for TxDOT:

$$C = 0.85IMP + 0.15 \tag{6.2}$$

The above equation is consistent with the Kuichling's original idea of the runoff coefficient as the amount of imperviousness of the drainage area. Kuichling (1889) concluded that the percentage of the rainfall discharged for any given watershed studied is nearly equal to the percentage of impervious surface within the watershed. Several researchers [Longobardi et al. (2003); Merz and Blöschl (2009)] have shown that runoff coefficients are strongly correlated with the initial soil moisture condition. With the increase of the rainfall duration, the degree of land saturated also increases and the runoff coefficient increases (or the imperviousness as proposed by Kuichling increases). So for future study, it is recommended to study the variation of the runoff coefficient with the degree of saturation of the land (temporal variation of *C*).

Many would argue that the application of the rational method is not appropriate for the range of watershed areas presented in this study. The TxDOT guidelines recommend the use of the rational method for watersheds with drainage areas less than 0.8 km^2 (200 acres) (TxDOT 2002). However our study showed that there is no demonstrable trend in runoff coefficient with drainage area. We applied rate-based *C* to estimate the peak discharge for the study watersheds and found out that the differences between the observed and modeled Q_p are generally within the expected errors from typical hydrologic analysis. We do not advocate any specific limits that should be imposed on drainage area for application of the rational method. However further study is recommended in other watersheds and with more extensive database, to determine what is the reasonable size that can be used with the rational method for the hydrologic design.

It is observed that application of the MRUH is simple and straightforward. Like other UH methods, MRUH can be applied to large watersheds with non-uniform rainfall distribution. However, using the runoff coefficient for the rainfall loss estimation doesn't account for the initial moisture condition of the watershed. We concluded that in predicting peak discharges and runoff hydrographs for engineering design, rainfall loss estimation results in greater uncertainty and contributes more model errors than variations of UH methods and model parameters for UH. So for future study it is recommended to incorporate the runoff coefficient with another loss parameter which accounts for the initial moisture condition of the watershed for rainfall loss estimation in application of the MRUH.

6.3 References

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