COMPARISON OF ESTIMATED AND OBSERVED STORMWATER RUNOFF FOR FIFTEEN WATERSHEDS IN WEST-CENTRAL FLORIDA, USING FIVE COMMON DESIGN TECHNIQUES

By J.T. Trommer, J.E. Loper, K.M. Hammett, and Geronia Bowman

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CONTENTS

Figures

Figures--Continued

Figures--Continued

Figures--Continued

Tables

CONVERSION FACTORS, VERTICAL DATUM, AND ACRONYMS

Multiply	By	To obtain
inch (in.)	25.40	millimeter
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
inch per hour (in./hr)	25.40	millimeter per hour
inch per year (in/yr)	25.40	millimeter per year
feet per mile (ft/mi)	0.1894	meter per kilometer
square mile (mi^2)	2.590	square kilometer
acre	0.4047	hectare
acre-foot (acre-ft)	1.233	cubic meter
acre-inch per hour (acre-in./hr)	0.10275	cubic meter per hour
cubic foot (ft^3)	0.02832	cubic meter
cubic foot per second (ft^3/s)	0.02832	cubic meter per second

Temperature can be converted between degrees Fahrenheit (°F) and degrees Celsius (°C) as follows:

$$°F = 9/5\ (°C) + 32$$

$$°C = 5/9\ (°F - 32)$$

Sea level: In this report "sea level" refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929)--a geodetic datum derived from a general adjustment of the first-order level nets of the United States and Canada, formerly called Sea Level Datum of 1929.

Acronyms

AMC I	Antecedent Moisture Condition I
AMC II	Antecedent Moisture Condition II
AMC III	Antecedent Moisture Condition III
CN	Curve Number
EPA	Environmental Protection Agency
NOAA	National Oceanic and Atmospheric Administration
NRCS	Natural Resources Conservation Service (formerly the Soil Conservation Service)
PC	Personal Computer
SWFWMD	Southwest Florida Water Management District
SWMM	Storm Water Management Model
TR-20	Soil Conservation Service Technical Release No. 20
USGS	U.S. Geological Survey

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ABSTRACT

Hydrologists use several traditional techniques for estimating peak discharges and runoff volumes from ungaged watersheds. However, applying these techniques to watersheds in west-central Florida requires that empirical relationships be extrapolated beyond tested ranges. As a result there is some uncertainty as to their accuracy.

Sixty-six storms in 15 west-central Florida watersheds were modeled using (1) the rational method, (2) the U.S. Geological Survey regional regression equations, (3) the Natural Resources Conservation Service (formerly the Soil Conservation Service) TR-20 model, (4) the Army Corps of Engineers HEC-1 model, and (5) the Environmental Protection Agency SWMM model. The watersheds ranged between fully developed urban and undeveloped natural watersheds. Peak discharges and runoff volumes were estimated using standard or recommended methods for determining input parameters. All model runs were uncalibrated and the selection of input parameters was not influenced by observed data.

The rational method, only used to calculate peak discharges, overestimated 45 storms, underestimated 20 storms and estimated the same discharge for 1 storm. The mean estimation error for all storms indicates the method overestimates the peak discharges. Estimation errors were generally smaller in the urban watersheds and larger in the natural watersheds.

The U.S. Geological Survey regression equations provide peak discharges for storms of specific recurrence intervals. Therefore, direct comparison with observed data was limited to sixteen observed storms that had precipitation equivalent to specific recurrence intervals. The mean estimation error for

all storms indicates the method overestimates both peak discharges and runoff volumes. Estimation errors were smallest for the larger natural watersheds in Sarasota County, and largest for the small watersheds located in the eastern part of the study area.

The Natural Resources Conservation Service TR-20 model, overestimated peak discharges for 45 storms and underestimated 21 storms, and overestimated runoff volumes for 44 storms and underestimated 22 storms. The mean estimation error for all storms modeled indicates that the model overestimates peak discharges and runoff volumes. The smaller estimation errors in both peak discharges and runoff volumes were for storms occurring in the urban watersheds, and the larger errors were for storms occurring in the natural watersheds.

The HEC-1 model overestimated peak discharge rates for 55 storms and underestimated 11 storms. Runoff volumes were overestimated for 44 storms and underestimated for 22 storms using the Army Corps of Engineers HEC-1 model. The mean estimation error for all the storms modeled indicates that the model overestimates peak discharge rates and runoff volumes. Generally, the smaller estimation errors in peak discharges were for storms occurring in the urban watersheds, and the larger errors were for storms occurring in the natural watersheds. Estimation errors in runoff volumes; however, were smallest for the 3 natural watersheds located in the southernmost part of Sarasota County.

The Environmental Protection Agency Storm Water Management model produced similar peak discharges and runoff volumes when using both the Green-Ampt and Horton infiltration methods. Estimated peak discharge and runoff volume data calculated with the Horton method was only slightly higher than those calculated with the Green-Ampt method. The mean estimation error for all the storms modeled indicates the model using the Green-Ampt infiltration method overestimates peak discharges and slightly underestimates runoff volumes. Using the Horton infiltration method, the model overestimates both peak discharges and runoff volumes. The smaller estimation errors in both peak discharges and runoff volumes were for storms occurring in the five natural watersheds in Sarasota County with the least amount of impervious cover and the lowest slopes. The largest errors were for storms occurring in the three small natural watersheds in the eastern part of the study area. The mean estimation errors for peak discharge ranged from an underestimation of 63 percent to an over estimation of 224 percent. For runoff volume, the mean estimation errors range from an underestimation of 63.3 percent to an overestimation of 267 percent.

INTRODUCTION

Low topographic relief and intense or prolonged rainfall events associated with tropical storms can produce recurring problems with storm-water flooding in west-central Florida. These naturally-occurring problems are being further compounded by rapid increases in population and the accompanying development.

Local, state, and federal agencies have recognized the potential impacts of population growth and development, and have imposed regulations on storm-water discharges. To comply with regulations and permit requirements, hydrologist commonly use several traditional techniques for estimating peak discharge and volume of stormwater runoff from ungaged watersheds; however, applying these techniques to watersheds in west-central Florida requires that empirical relationships be extrapolated beyond tested ranges. Watersheds in this area typically have flatter slopes, more permeable soils, lower stream gradients, higher ground water levels, and larger wetland areas than watersheds used in the development of many of these empirical relationships. Rainfall events are typically short duration, high intensity thunder storms rather than the 24 hr, specific recurrence interval storm used for design purposes. Because watersheds and rainfall events in west-central Florida are not typical of those used to develop most standard techniques there is uncertainty as to the accuracy of the estimates.

Underestimating the peak flow or storm volume can cause detrimental environmental and possibly severe economic consequences, while overestimation can result in unnecessary economic burdens on the community. The U.S. Geological Survey (USGS) began a cooperative study in April 1991 with the Sarasota County Environmental Stormwater Utility to better understand the uncertainty of five of these traditional estimating techniques when applied to low-gradient watersheds common in west-central Florida.

Purpose and Scope

The specific objectives of this report are to: (1) describe the methods used to collect rainfall and runoff data from 15 low-gradient watersheds in west-central Florida, (2) describe the techniques used to estimate peak discharge and runoff volumes for specific storms that occurred in these watersheds, and (3) present comparisons of the estimated and observed peak discharges and runoff volumes for those storms.

The overall purposes of the study were to evaluate the reliability and accuracy of techniques commonly used to estimate stormwater runoff when those techniques are applied to the low-gradient watersheds found in west-central Florida, and, to develop techniques to estimate runoff from watersheds with characteristics that are outside the range for which traditional techniques can be reliably applied. A subsequent report will present modifications of techniques for estimating stormwater runoff.

Data from 15 watersheds in Sarasota, Hardee, Hillsborough, and Pinellas Counties were used for this study. Continuous rainfall and stream-flow data were collected for each watershed. Stage and discharge relationships were developed for each station. Seven new gaging stations were installed in Sarasota County and data collected during previous investigations in 8 watersheds in Hillsborough, Pinellas, and Hardee Counties were also used. Physiographic characteristics for each watershed were compiled from aerial photographs, U.S. Geological Survey topographic maps, County drainage maps, consultant's reports, and field observations.

Estimated peak discharges and runoff volumes were computed using techniques commonly applied by design engineers. The design techniques were applied according to quidelines specified in user manuals or standard engineering textbooks, as though no field data were available. Computed estimates were compared to observed peak discharges and runoff volumes, so that the accuracy of the techniques could be evaluated.

Acknowledgments

This study was coordinated with J.P. Marchand, Deputy Director of the Sarasota County Environmental Stormwater Utility. The authors gratefully acknowledge the cooperation and assistance of Mr. Marchand and his staff, without whom this study would not have been conducted. Thanks are also extended to the land owners who allowed access to the streams and construction of the rainfall/runoff stations, particularly the Palmer and Taylor Ranches. Special thanks are extended to Stephen M. Suau, consulting engineer, Kimley-Horn and Associates, Inc., for providing extensive information and maps for the Catfish Creek and South Creek watersheds.

Description of the Study Area

The study area includes watersheds located in Sarasota, Hardee, Hillsborough, and Pinellas Counties (fig. 1) and lies within the Gulf Coastal Lowlands and Central Highlands physiographic regions (fig. 2) described by White (1970). The Gulf Coastal Lowlands consist of a broad, sandy, gently sloping marine plain containing creeks, swamps, and sloughs. Land-surface elevation in this region ranges from sea level to about 70 ft (feet) above sea level. The watersheds lying within the Central Highlands are located in the area where Hillsborough, Manatee, and Hardee Counties intersect (fig. 1). This part of the Central Highlands consists of relatively flat plains that contain swamps and widely branching streams. Land-surface elevation generally ranges from 100 to 130 ft above sea level in the vicinity of these watersheds (Lewelling and Wylie, 1993, p.4).

The climate in west-central Florida is subtropical and is characterized by long humid summers and mild winters. Mean monthly temperatures range from about 50 $^{\circ}$ F (degrees Fahrenheit) in January to about 90 $^{\circ}$ F in August. Mean annual (normal) rainfall is 53.10 inches (in.) in St. Petersburg, 55.67 in. in the Sarasota area, and 53.09 in. in Wauchula (National Oceanic and Atmospheric Administration (NOAA), 1992). More than one-half of all rainfall occurs from June to September as high intensity, short duration thunder storms or occasional hurricanes or tropical storms. Rainfall from winter frontal activity is generally of longer duration and lower intensity. Figure 3 shows the mean monthly rainfall in the St. Petersburg, Sarasota, and Wauchula areas.

Watershed Descriptions

The 15 watersheds included in this study range in size from 0.14 mi^2 (square miles) to 15.22 mi^2. Six watersheds are urban, six are natural, and the remaining three have varying degrees of development associated with them. Table 1 summarizes the characteristics for each watershed.

Twelve of the watersheds are located near the coast. Nine of these drain directly to coastal waters. The remaining three (1,2 and 3), located in Hillsborough County, drain to the Hillsborough River. Three watersheds are located inland; two are in eastern Hillsborough County (6 and 7) and drain to the Little Manatee River. The third watershed is located in Hardee County (8) and drains through tributaries to the Peace River.

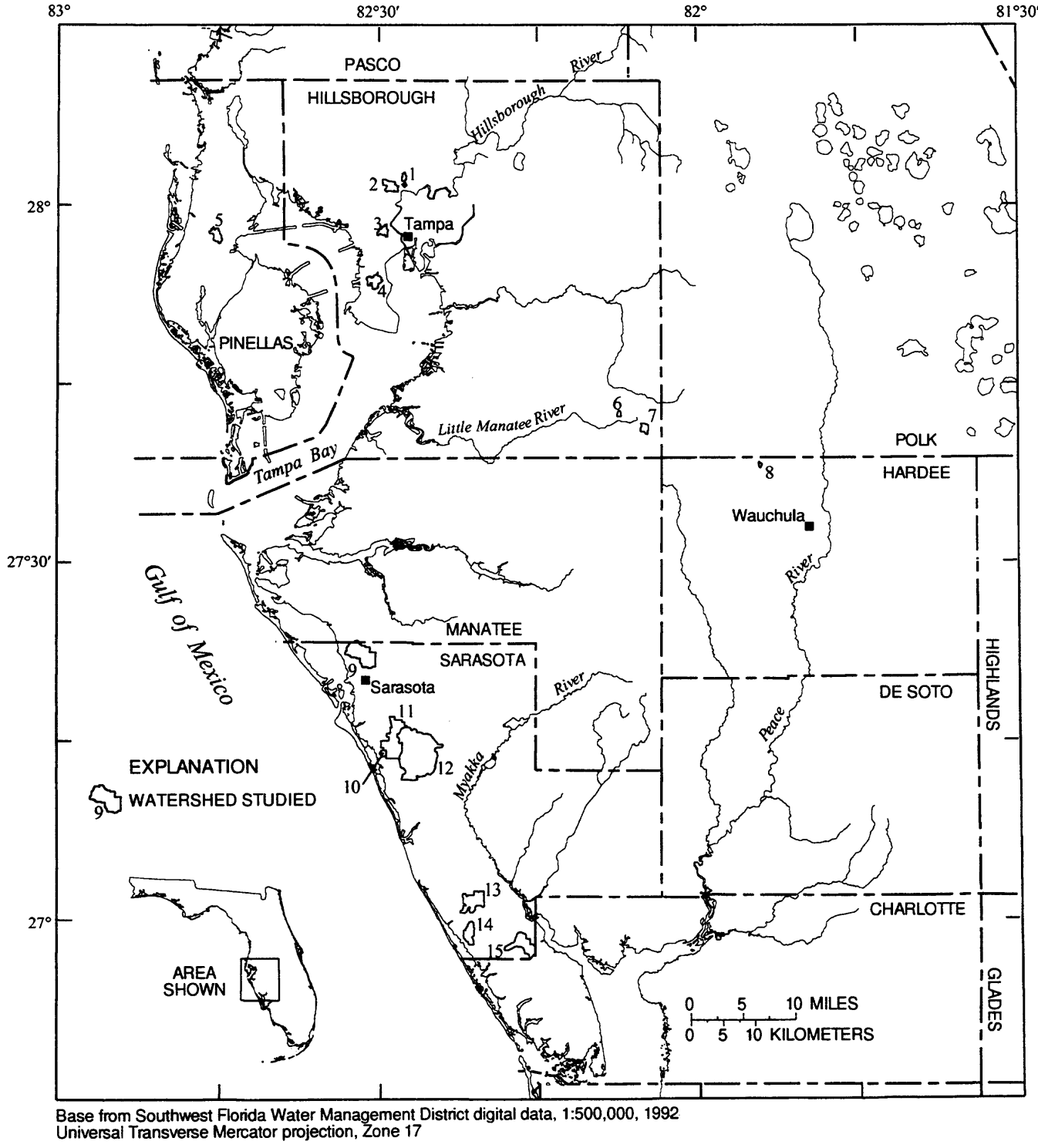


Figure 1. Location of the study area. (Watersheds listed on table 1)

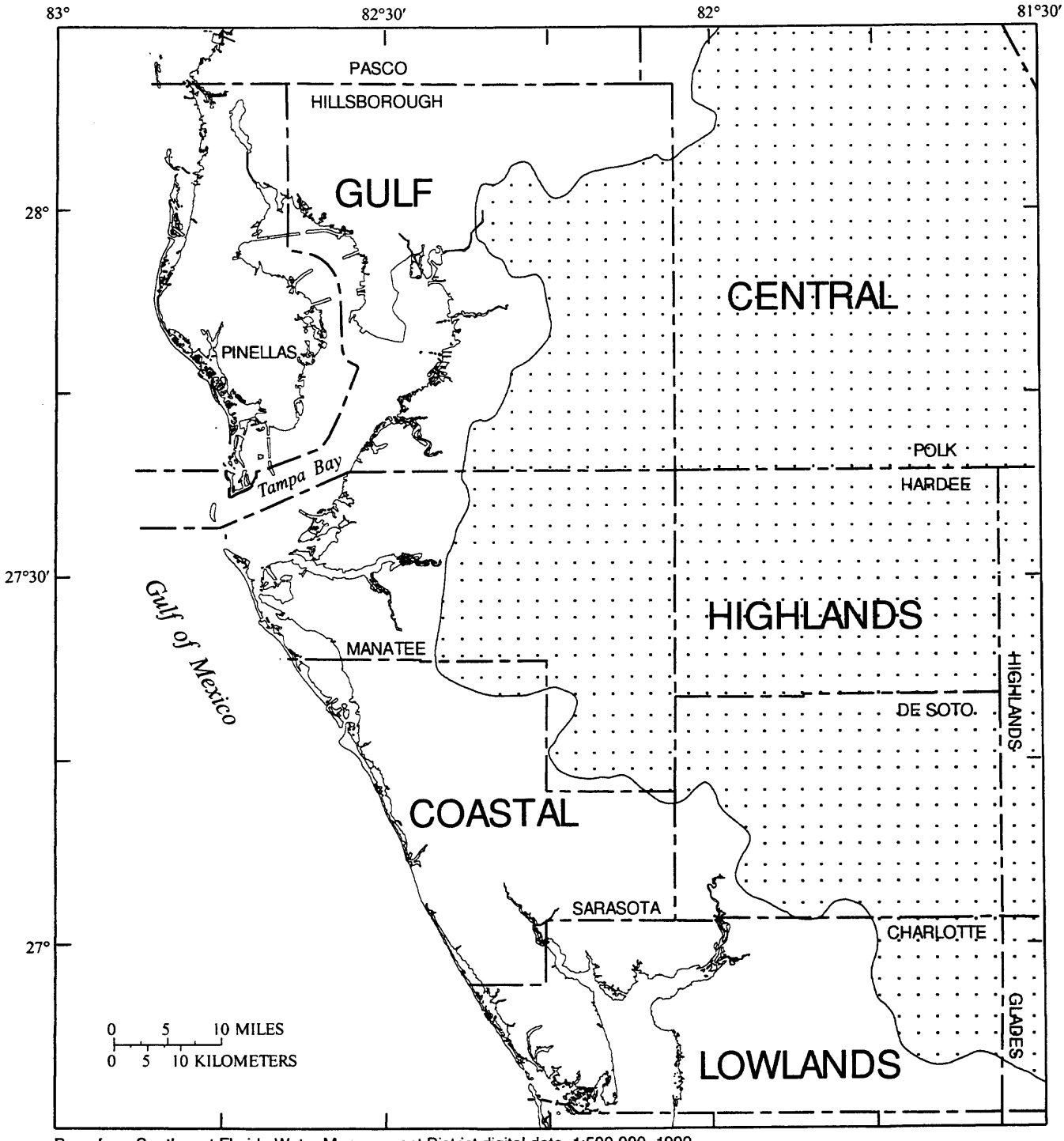


Figure 2. Physiography of the study area (White, 1970).

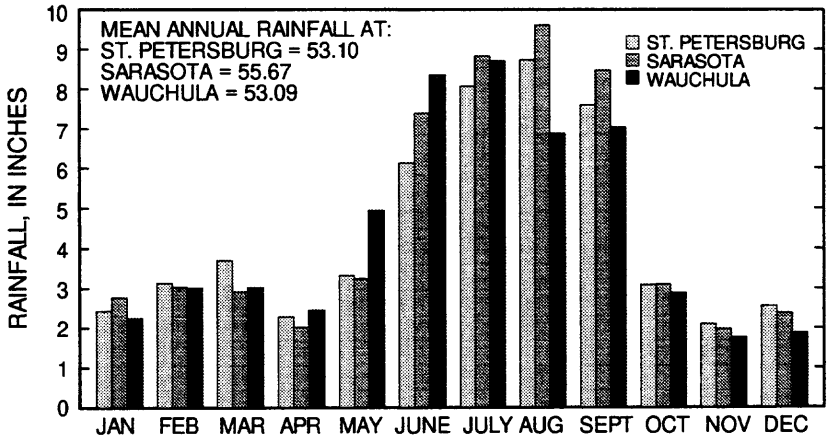


Figure 3. Mean monthly rainfall (1951-80) at St. Petersburg, Sarasota, and Wauchula, Florida.

Table 1. Watershed characteristics

[DA, drainage area (mi^2); SL, slope (ft/mi); EIA, effective impervious area (mi^2); Wet, wetlands; Res, residential; Com, commercial (includes commercial, industrial, and roads); Ag, agricultural; Pas, pasture or rangeland; For, forest or woodland; Open, open space; U, urban; N, natural; M, mixed]

						Land use, in percent of total area							
Map no.	Identification no.	Watershed name	Watershed classification	DA	SL	EIA	Wet	Res	Com	Ag	Pas	For	Open
1	02306002	Arctic Street storm drain	U	0.34	12.3	40.0	0	50.0	50.0	0	0	0	0
2	02306006	Kirby Street drainage ditch	U	1.15	8.1	5.5	3.1	72.3	11.1	0	0	0	13.5
3	02306021	St. Louis Street drainage ditch	U	0.51	10.2	9.0	0	68.0	16.0	0	0	0	16.0
4	02306071	Gandy Boulevard drainage ditch	U	1.29	4.6	20.0	0.9	42.3	33.4	0	0	0	23.4
5	02307731	Allen Creek	U	1.79	23.4	20.0	0.9	63.0	20.0	0	0	0	16.1
6	274215082072000	IMC Creek	N	0.17	47.0	0	0	0	0	0	67.0	33.0	0
7	274141082051300	Grace Creek	N	0.66	26.0	0	0	0	0	33.0	33.0	34.0	0
8	273806081535000	CFI-3 Creek	N	0.14	36.0	0	0	0	0	0	67.0	33.0	0
9	02299861	Walker Creek	M	4.78	6.3	40.0	1.0	52.0	16.0	0	0	16.0	15.0
10	02299742	Clower Creek	U	0.35	3.7	85.0	0.1	14.9	85.0	0	0	0	0
11	02299741	Catfish Creek	M	4.77	3.5	10.0	0.5	25.0	10.0	0	10.0	30.0	25.0
12	02299737	South Creek	N	15.2	2.9	0	31.0	10.0	0	0	35.0	24.0	0
13	02299684	Forked Creek	N	2.7	2.8	0	15.0	0	0	30.0	55.0	0	0
14	02299681	Gottfried Creek	M	2.00	1.4	10.0	15.0	50.0	10.0	0	0	0	25.0
15	02299680	Rock Creek	N	2.6	2.9	0	25.0	0	0	0	0	25.0	0

The Arctic Street Storm Drain (site 1, fig. 1), located in Hillsborough County, within the City of Tampa, drains a 0.34 mi^2 area. The watershed is urban and land use is about evenly divided between older single-family residences and commercial businesses. Land surface elevation ranges from about 50 ft above sea level at the southern boundary of the watershed to about 30 ft above sea level at the gaging station. The watershed is about 2,650 ft wide by 7,400 ft long (fig. 4) and has a slope of 12.3 ft/mi. Drainage is through an underground storm-sewer system (Lopez and Michaelis, 1979, p. 5) that is about 6,600 ft in length. Maximum recorded discharge from the watershed was 142.0 ft^3/s (Lopez and Woodham, 1983).

The Kirby Street drainage ditch (site 2, fig. 1), located in Hillsborough County, in the City of Tampa, drains a 1.15 mi^2 area. Over 70 percent of land use in the watershed is residential. About 13.5 percent of the watershed contains open areas and the remaining area is commercial. Land surface elevation ranges from 50 ft above sea level at the western boundary of the watershed to 25 ft above sea level at the gaging station. The watershed is about 5,500 ft wide by 6,000 ft long (fig. 5) and has slope of 8.1 ft/mi. The stream channel is about 12,700 ft in length and is well defined. Maximum recorded discharge from the watershed was 192 ft^3/s (Lopez and Woodham, 1983).

The St. Louis Street drainage ditch (site 3, fig. 1), located in Hillsborough County, within the City of Tampa, drains a 0.51 mi^2 area. Older single-family residences account for 68 percent of the land use in the watershed. The remaining area is about equally divided between commercial and open land. Land surface elevation ranges from 35 ft above sea level at the northern and western watershed boundaries to about 25 ft above sea level at the gaging station. The watershed is about 5,200 ft wide by 4,500 long (fig. 6) and has a slope of 10.2 ft/mi. Drainage is through an underground storm-sewer system (Lopez and Michaelis, 1979, p. 7) that is about 5,900 ft in length. Maximum recorded discharge from the basin was 357 ft^3/s (Lopez and Woodham, 1983).

The Gandy Boulevard drainage ditch (site 4, fig. 1), located in Hillsborough County, within the City of Tampa, roughly in the center of the Interbay Peninsula, drains a 1.29 mi^2 area. Land use is divided between small, single-family residences, commercial centers with large parking lots and lightly vegetated or grassy open areas. Land surface elevation ranges from 15 ft above sea level at the eastern boundary of the watershed to about 6 ft above sea level at the gage. The watershed is about 7,500 ft wide by 6,500 ft long (fig. 7) and has a slope of 4.6 ft/mi. Drainage is through a combination of open ditches and underground storm-sewers (Lopez and Michaelis, 1979, p. 7) that is about 8,600 ft in length.

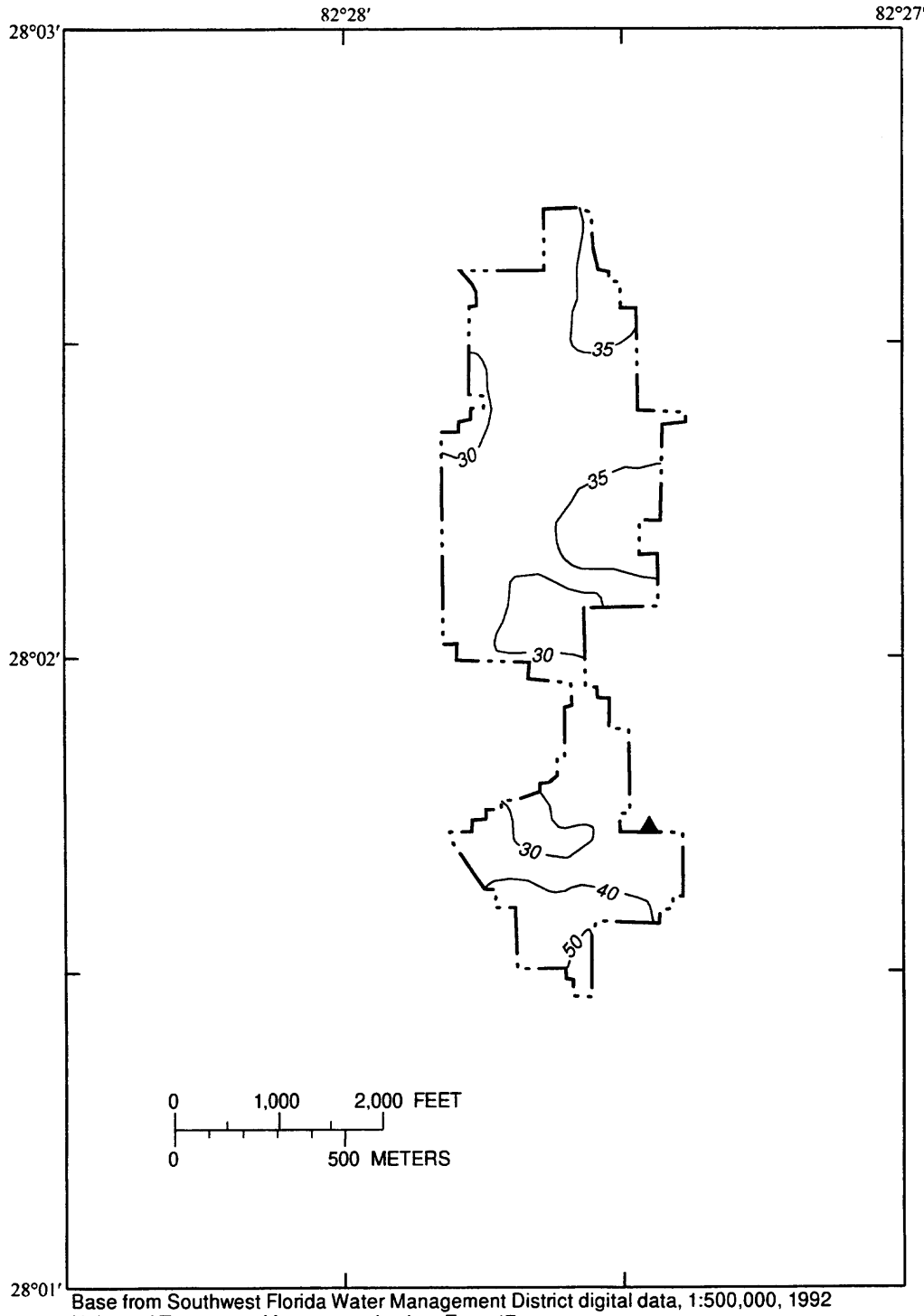
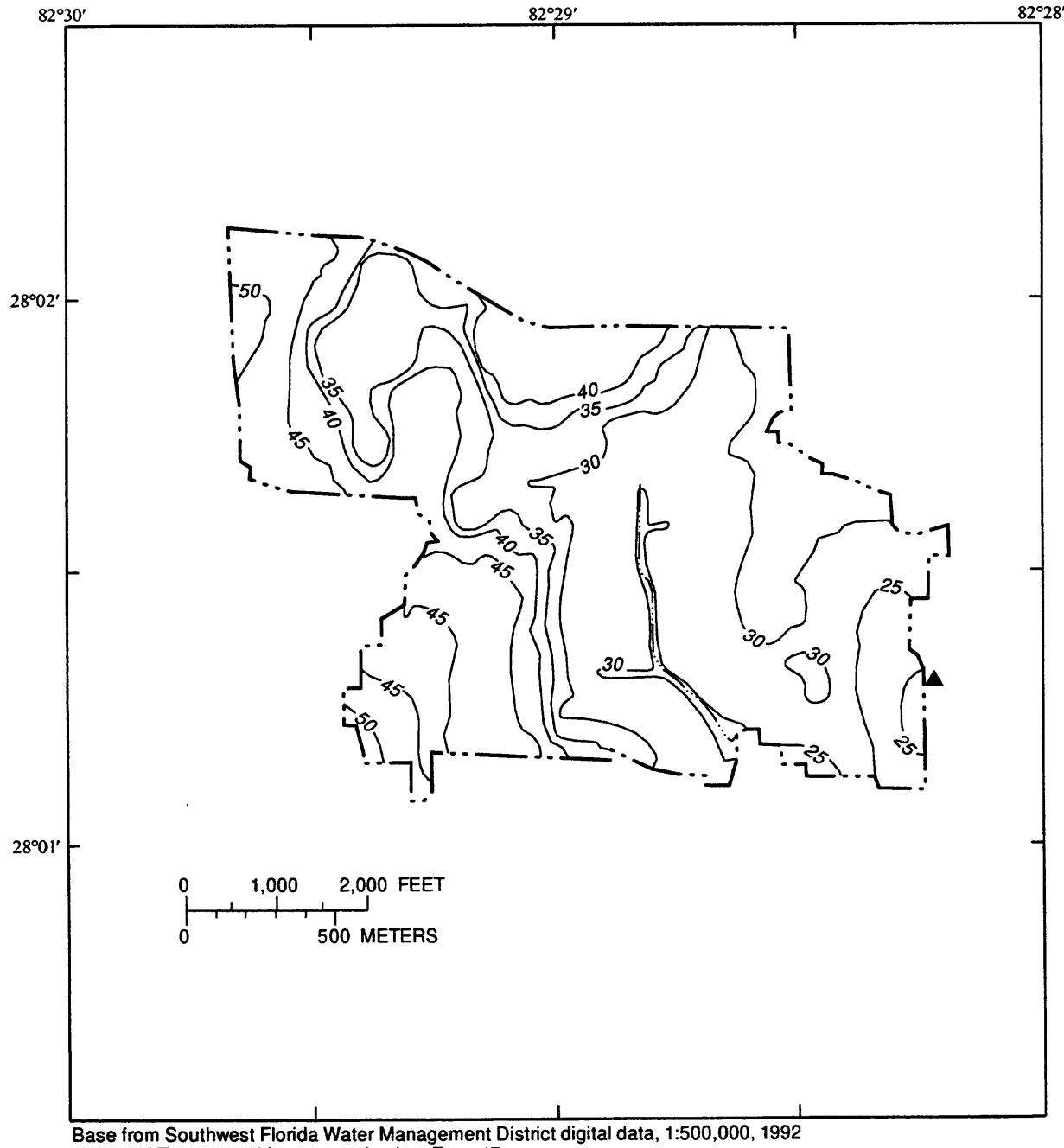


Figure 4. Arctic Street storm drain watershed.



Base from Southwest Florida Water Management District digital data, 1:500,000, 1992
Universal Transverse Mercator projection, Zone 17

EXPLANATION

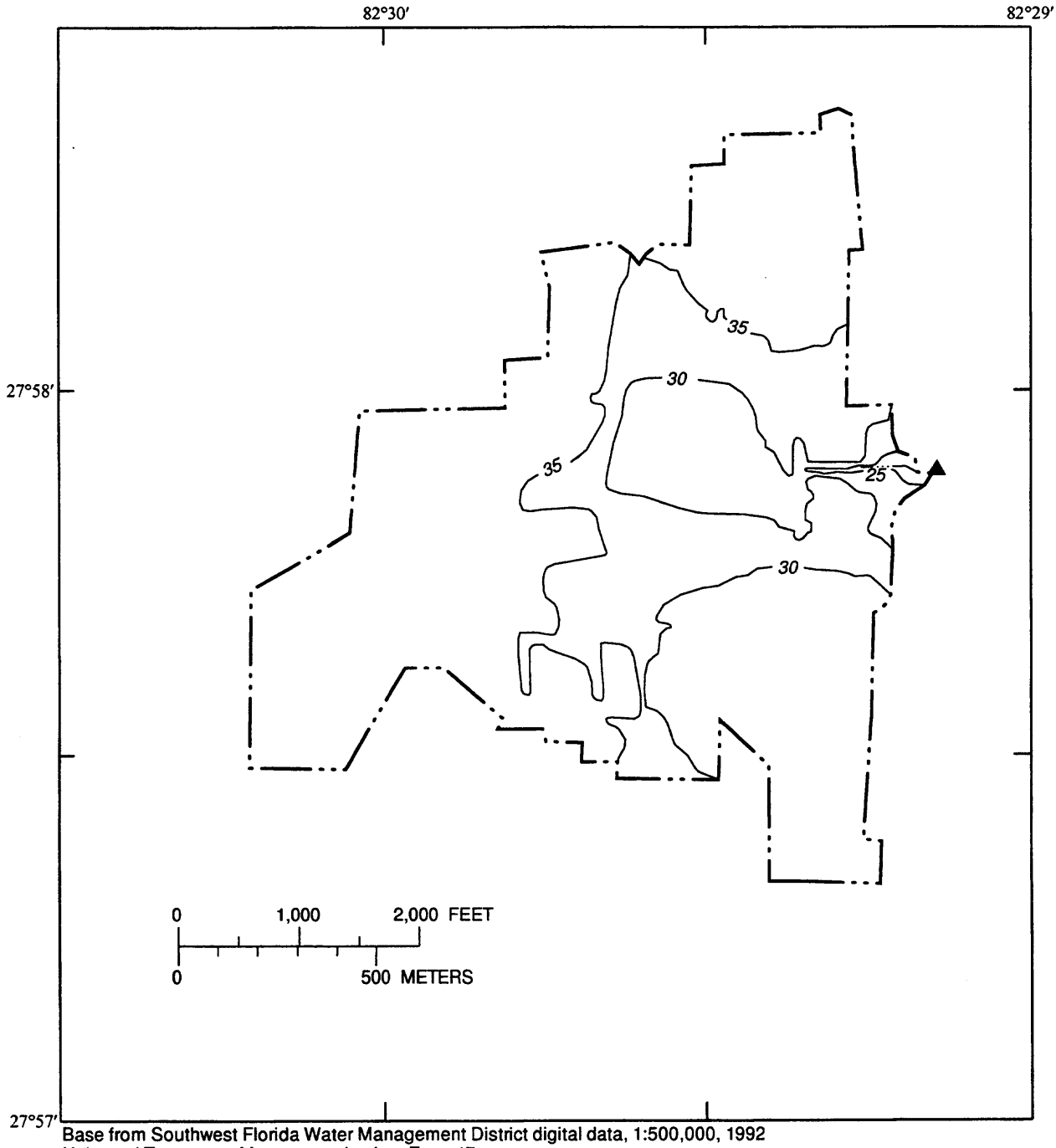
TOPOGRAPHIC CONTOUR- Showing altitude of land surface. Contour intervals 5 feet. Datum is sea level

APPROXIMATE WATERSHED BOUNDARY ABOVE THE GAGING STATION

APPROXIMATE LOCATION OF THE STREAM CHANNEL

STREAM GAGING STATION

Figure 5. Kirby Street drainage ditch watershed.



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Universal Transverse Mercator projection, Zone 17

EXPLANATION

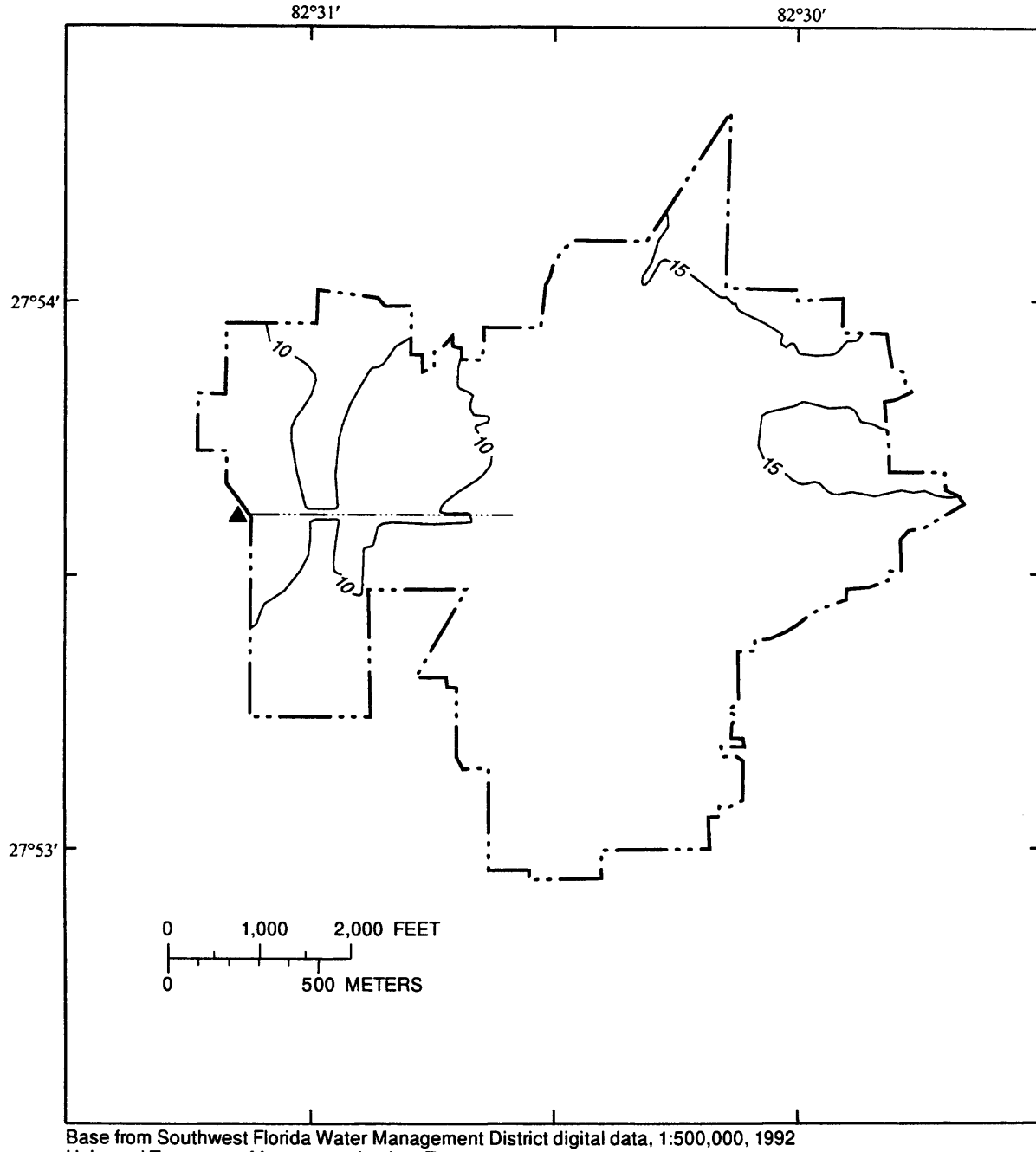
TOPOGRAPHIC CONTOUR- Showing altitude of land surface. Contour intervals 5 feet. Datum is sea level

APPROXIMATE WATERSHED BOUNDARY ABOVE THE GAGING STATION

APPROXIMATE LOCATION OF THE STREAM CHANNEL

STREAM GAGING STATION

Figure 6. St. Louis Street drainage ditch watershed.



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Universal Transverse Mercator projection, Zone 17

EXPLANATION

TOPOGRAPHIC CONTOUR-Showing altitude of land surface. Contour intervals 5 feet. Datum is sea level

APPROXIMATE WATERSHED BOUNDARY ABOVE THE GAGING STATION

APPROXIMATE LOCATION OF THE STREAM CHANNEL

STREAM GAGING STATION

Figure 7. Gandy Boulevard drainage ditch watershed.

Maximum recorded discharge from the watershed was 692 ft^3/s (Lopez and Woodham, 1983).

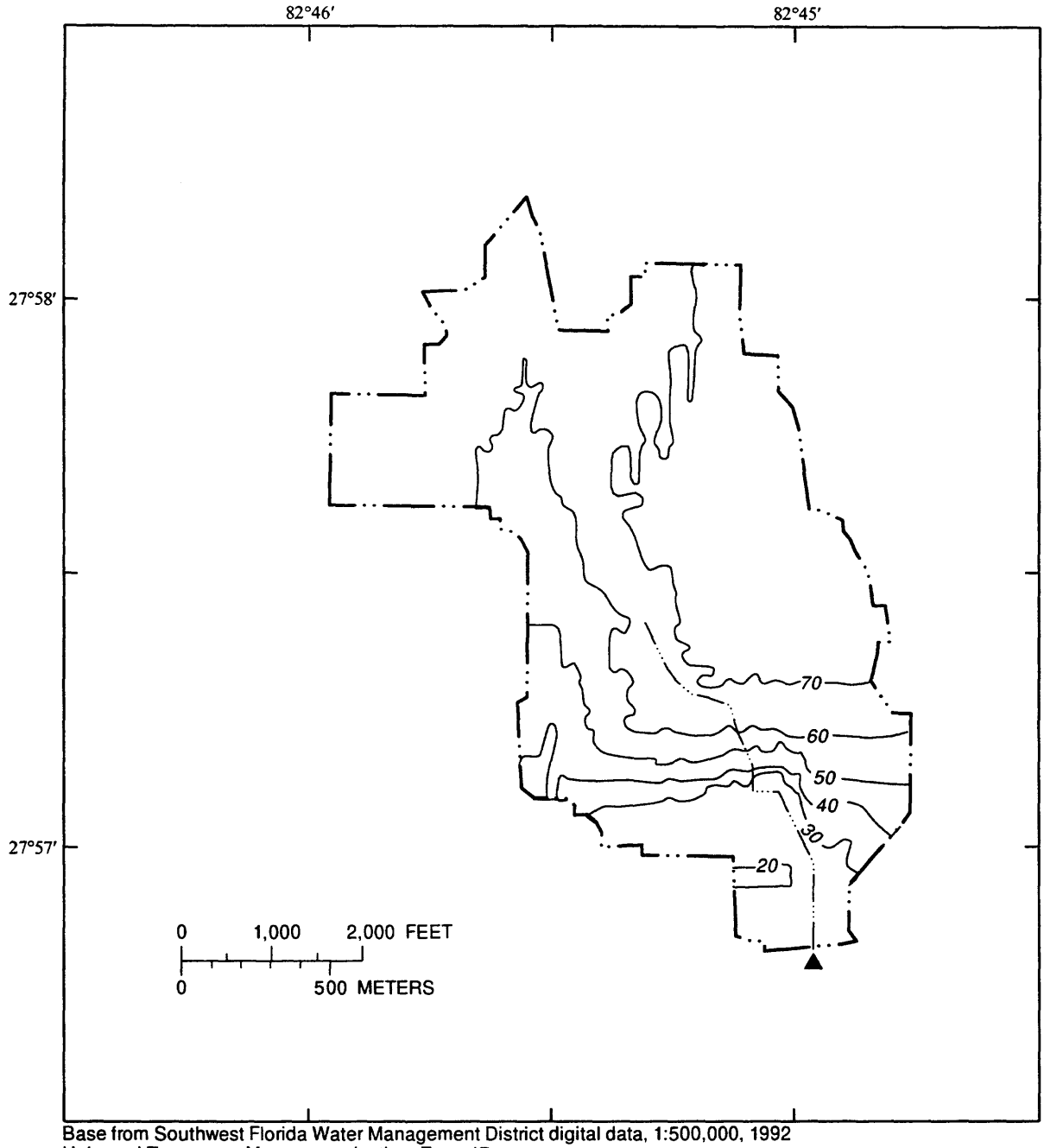
The Allen Creek watershed (site 5, fig. 1), located in northwestern Pinellas County, drains a 1.79 mi^2 area. Land use is predominantly residential. Land surface elevation ranges from 75 ft above sea level in the northern part of the watershed to 20 ft above sea level at the gaging station. The watershed is about 4,500 ft wide by 6,600 ft long (fig. 8) and has a slope of 23.4 ft/mi. The northern part of the watershed is drained by an underground storm sewer, and the southern part by an open ditch (Lopez and Michaelis, 1979, p. 11). The main channel is about 7,400 ft in length. Maximum recorded discharge from the watershed was 852 ft^3/s (Lopez and Woodham, 1983).

The IMC Creek watershed (site 6, fig. 1), located in southeastern Hillsborough County, drains a 0.17 mi^2 area and consists of gently sloping pastureland and a wooded area of cultivated pine trees. Land surface elevations range from about 115 ft above sea level at the watershed boundary to 95 ft above sea level at the gaging station. The watershed is approximately 2,000 ft wide by 2,800 ft long (fig. 9) and has a slope of 47.0 ft/mi. Drainage channels are poorly defined, except in the area of the gaging station. Maximum recorded discharge from the watershed was 10.5 ft^3/s (Lewelling and Wylie, 1993, p. 13).

The Grace Creek watershed (site 7, fig. 1), located in southeastern Hillsborough County, drains a 0.66 mi^2 area. Land use in the watershed is divided equally between citrus, pastureland, and wooded areas. Topography is flat to gently sloping with land surface elevation ranging from 135 ft above sea level at the watershed boundary to 110 ft above sea level at the gaging station. The watershed is approximately 4,200 ft wide by 4,400 ft long (fig. 10), and has a slope of 26.0 ft/mi. The main stream channel is about 7,200 ft in length and is generally well defined throughout the watershed. Maximum recorded discharge from the watershed was 47.0 ft^3/s (Lewelling and Wylie, 1993, p. 17).

The CFI-3 Creek watershed (site 8, fig. 1), located in northwestern Hardee County, drains a 0.14 mi^2 area. Pastureland covers 67 percent of the watershed. The remaining land is covered by palmetto scrub and forest. Land surface elevation ranges from 130 ft above sea level at the watershed boundary to about 115 ft above sea level at the gaging station. The watershed is approximately 2,200 ft wide by 2,100 ft long, (fig. 11) and has a slope of 36.0 ft/mi. The stream channel is about 2,200 ft in length and is well defined. Maximum recorded discharge from the watershed was 62.7 ft^3/s (Lewelling and Wylie, 1993, p. 21).

The Walker Creek watershed (site 9, fig. 1), located in Sarasota County, within the City of



Base from Southwest Florida Water Management District digital data, 1:500,000, 1992
Universal Transverse Mercator projection, Zone 17

EXPLANATION

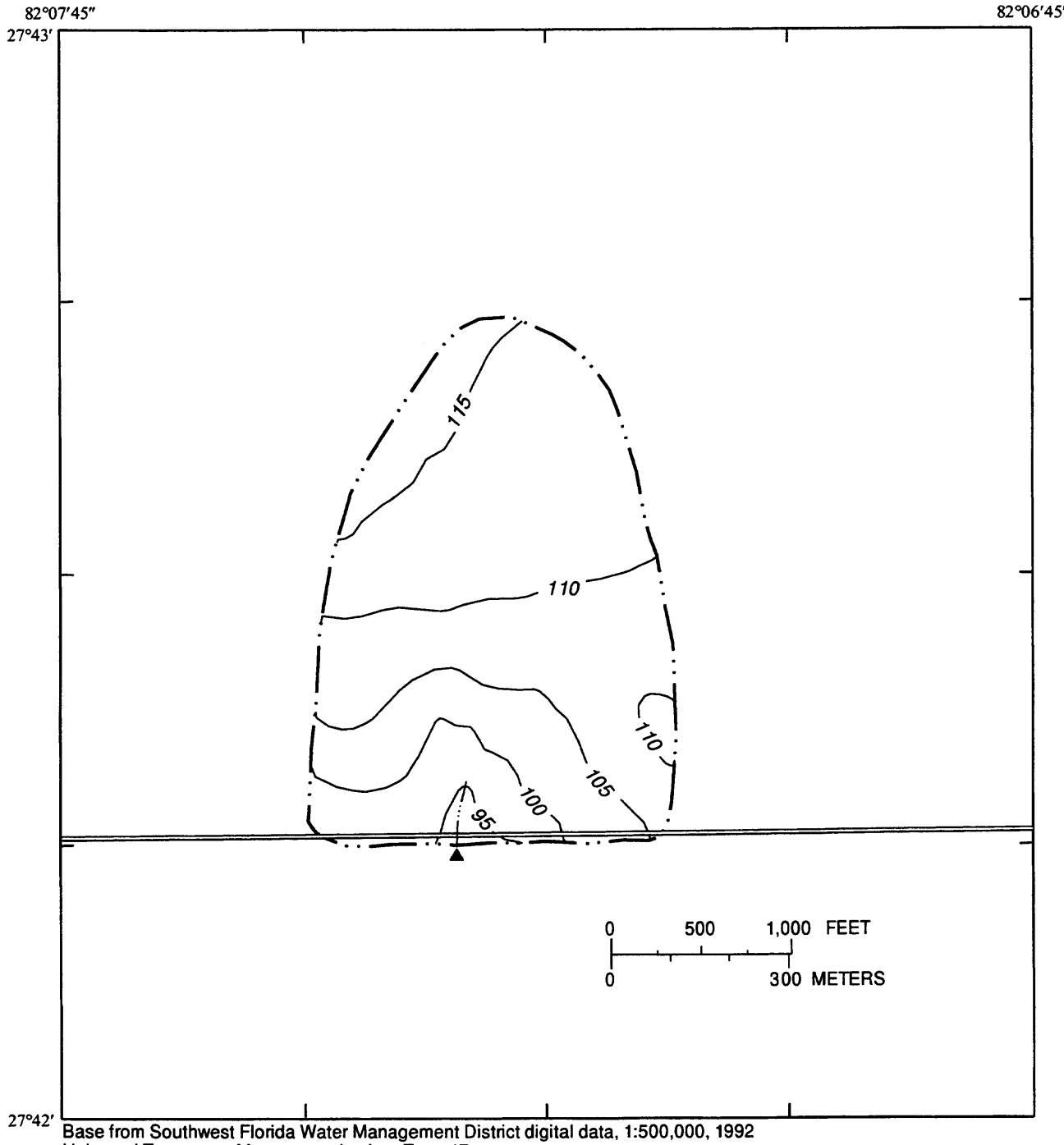
TOPOGRAPHIC CONTOUR-Showing altitude of land surface. Contour intervals 10 feet. Datum is sea level

APPROXIMATE WATERSHED BOUNDARY ABOVE THE GAGING STATION

APPROXIMATE LOCATION OF THE STREAM CHANNEL

STREAM GAGING STATION

Figure 8. Allen Creek watershed.



Base from Southwest Florida Water Management District digital data, 1:500,000, 1992
Universal Transverse Mercator projection, Zone 17

EXPLANATION

TOPOGRAPHIC CONTOUR- Showing altitude of land surface. Contour intervals 5 feet. Datum is sea level

APPROXIMATE WATERSHED BOUNDARY ABOVE THE GAGING STATION

APPROXIMATE LOCATION OF THE STREAM CHANNEL

STREAM GAGING STATION

Figure 9. IMC Creek watershed.

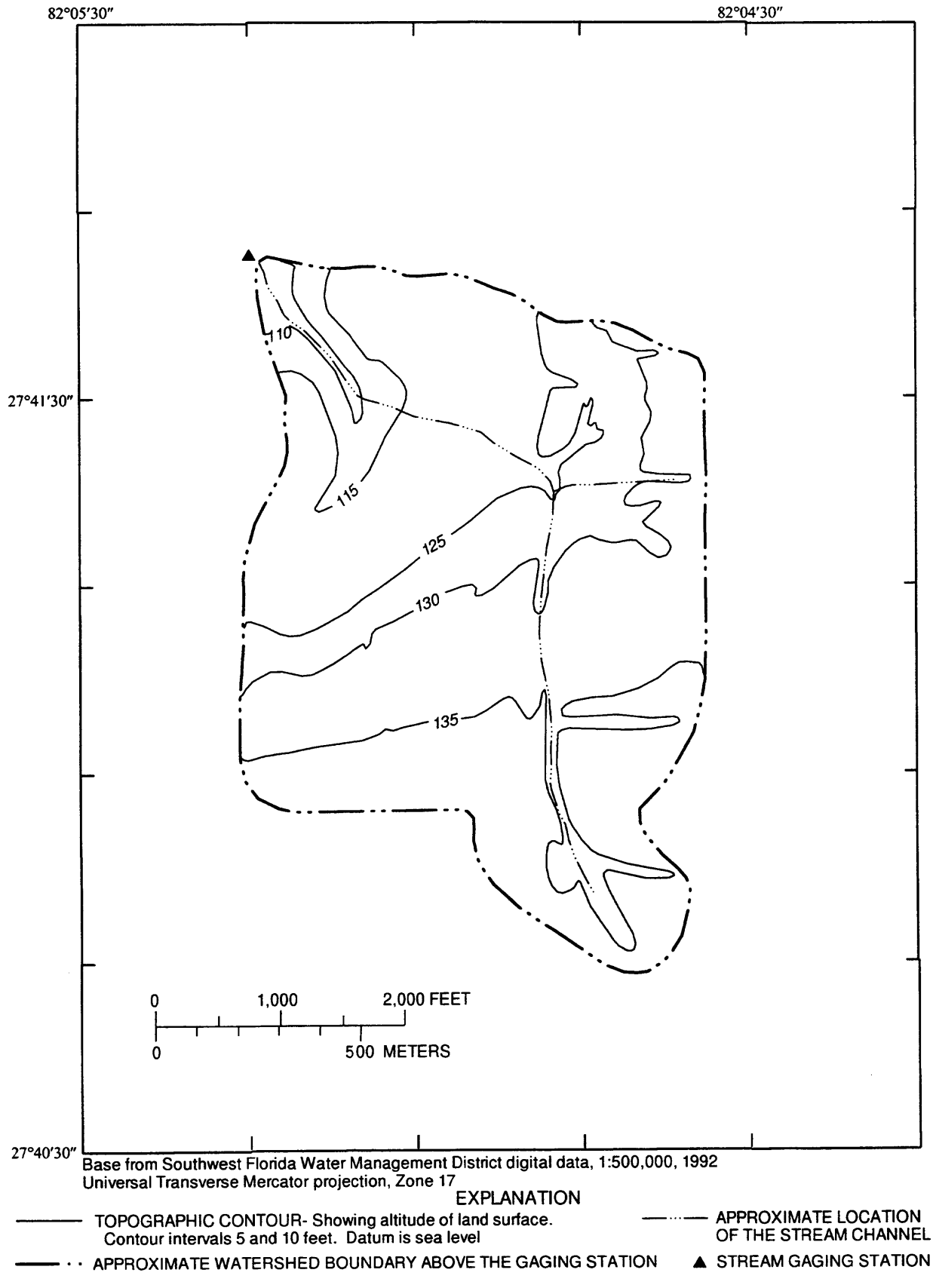


Figure 10. Grace Creek watershed (modified from Lewelling and Wylie, 1993).

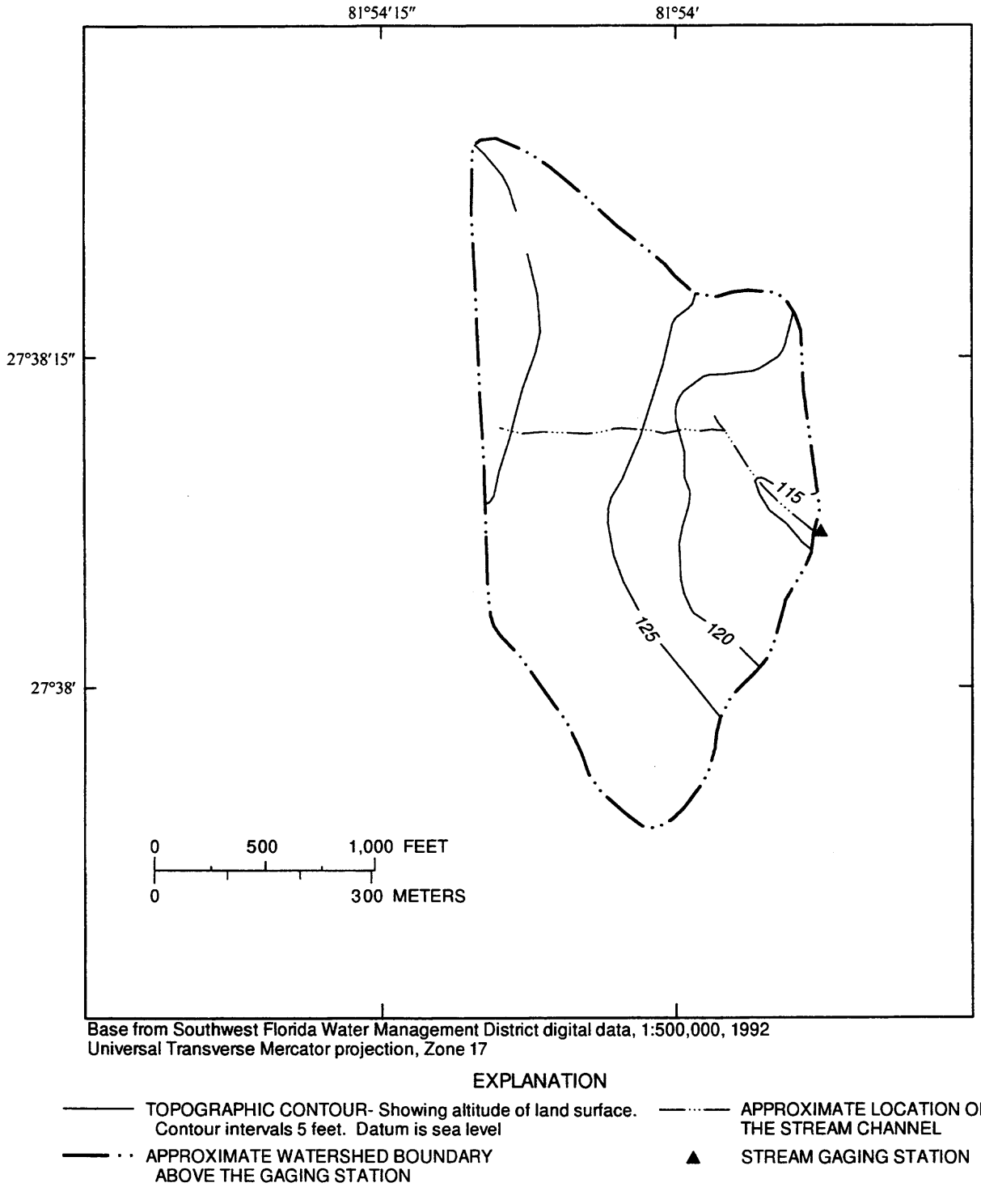


Figure 11. CFI-3 Creek watershed (modified from Lewelling and Wylie, 1993).

Sarasota, drains a 4.78 mi^2 area. Residential development accounts for 52 percent of the land use in the watershed. About 16 percent is commercial or industrial. The remaining area consists of wooded or grassy open areas. Ponds or wetlands cover less than 1 percent. Land surface elevation ranges from 40 ft above sea level on the eastern side of the watershed to 10 ft above sea level at the gaging station. The watershed is about 4,500 ft wide by 9,000 ft long and is subdivided by 3 tributaries that converge upstream from the gaging station (fig. 12). All stream channels are well defined and the slope of each tributary averages about 6.3 ft/mi. The lengths of the western, eastern, and southern tributaries are 9,800 ft, 15,500 ft, and 12,500 ft respectively. Maximum recorded discharge from the watershed was 971 ft^3/s.

The Clower Creek watershed (site 10, fig. 1), located in west-central Sarasota County, drains a 0.35 mi^2 area. Land use is predominantly commercial and high-density residential. About 85 percent of the watershed is covered by impervious surfaces. Retention ponds cover about 0.1 percent of the watershed. Land surface elevation averages about 15 ft above sea level throughout most of the watershed, except in the western part of the watershed where the land surface elevation drops to about 5 ft above sea level (fig. 13). The watershed is approximately 3,200 ft square and has a slope of about 3.7 ft/mi. Drainage is through an underground storm-sewer system in the vicinity of the shopping malls and the trailer park. The length of the drainage system is about 3,000 ft. Maximum recorded discharge from the watershed was 205 ft^3/s.

The Catfish Creek watershed (site 11, fig. 1), borders the Clower Creek watershed and drains a 4.77 mi^2area. About 50 percent of the watershed has been developed as golf and country club communities. Commercial development makes up about 10 percent. The remainder is undeveloped. Land surface elevation ranges from 35 ft above sea level in the northern part of the watershed to about 15 ft above sea level at the gaging station. The watershed is about 7,000 ft wide by 21,500 ft long (fig. 14). Slope in the watershed is about 3.5 ft/mi. The main channel is about 23,500 ft in length. Maximum discharge from the watershed for the period of record (October 1992 to September 1993) is 300 ft^3/s; however, an instantaneous discharge of 467 ft^3/s was measured on June 27, 1992.

The South Creek watershed (site 12, fig. 1), borders the Catfish Creek watershed and drains a 15.22 mi^2 area. A golf course resort, and residential and commercial development make up about 15 percent of the watershed in the northern part. The remainder is undeveloped. Ponds or wetlands cover about 31 percent of the total area. Land surface elevation ranges from 35 ft above sea level to 15 ft at the

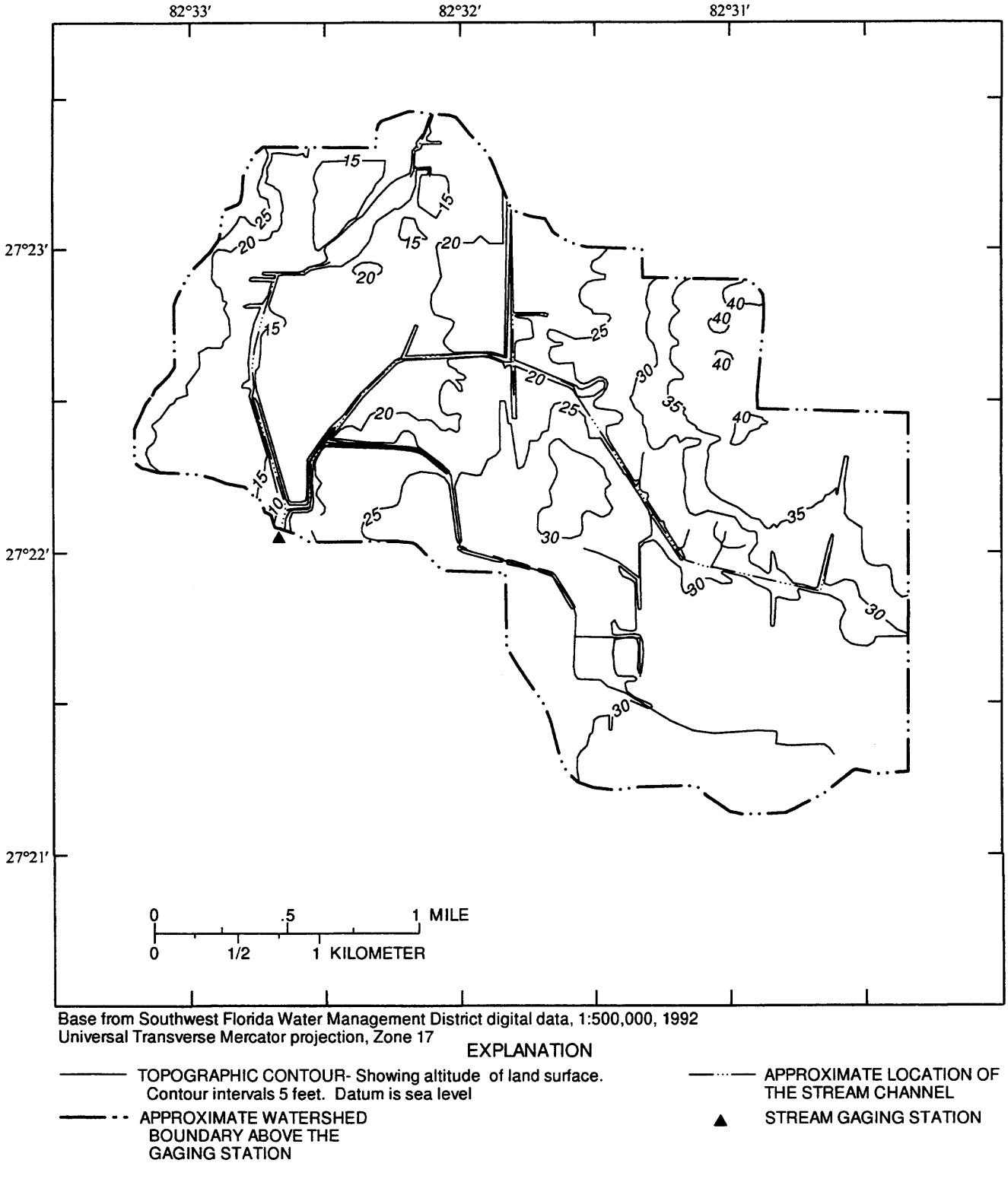


Figure 12. Walker Creek watershed.

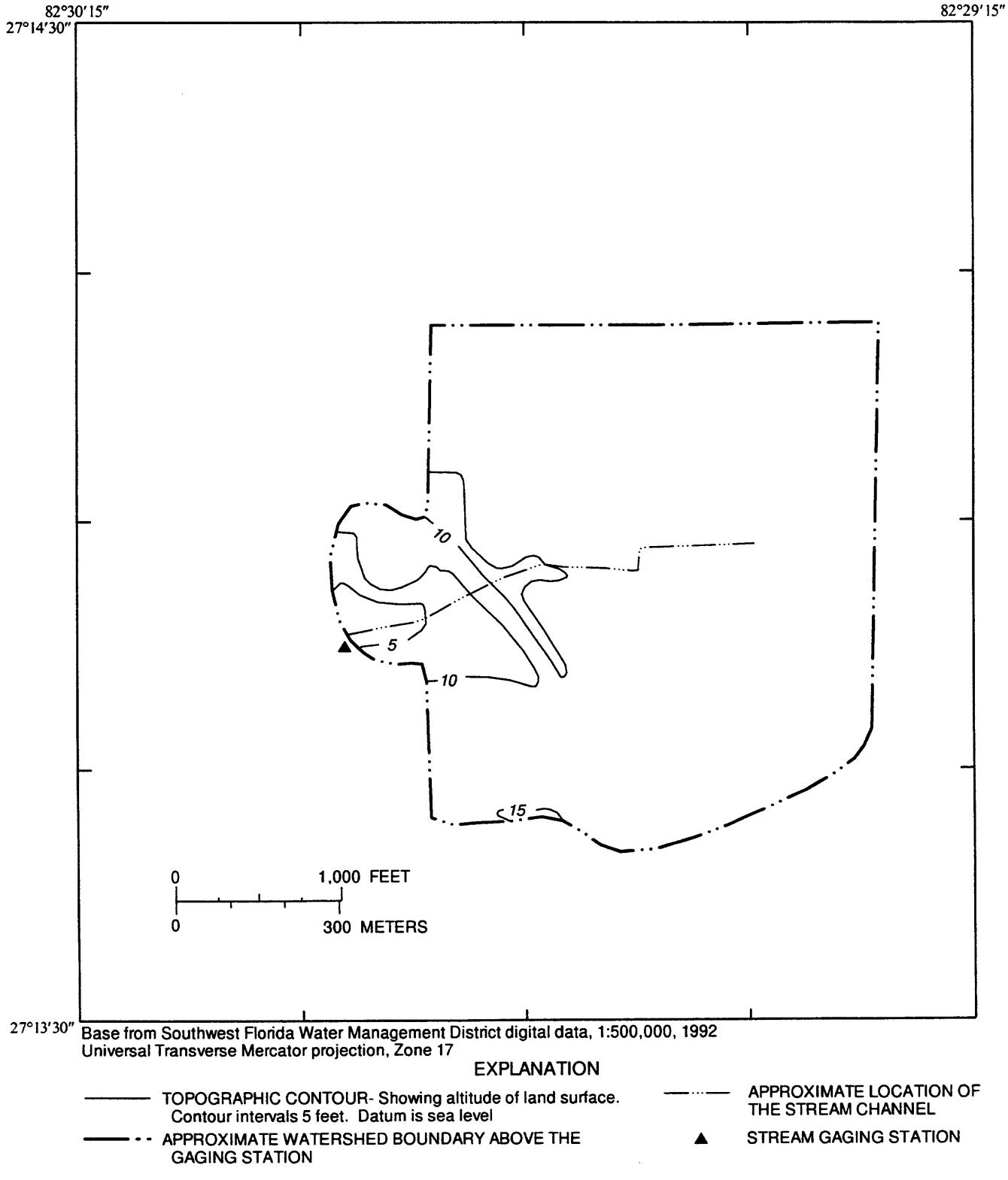


Figure 13. Clower Creek watershed.

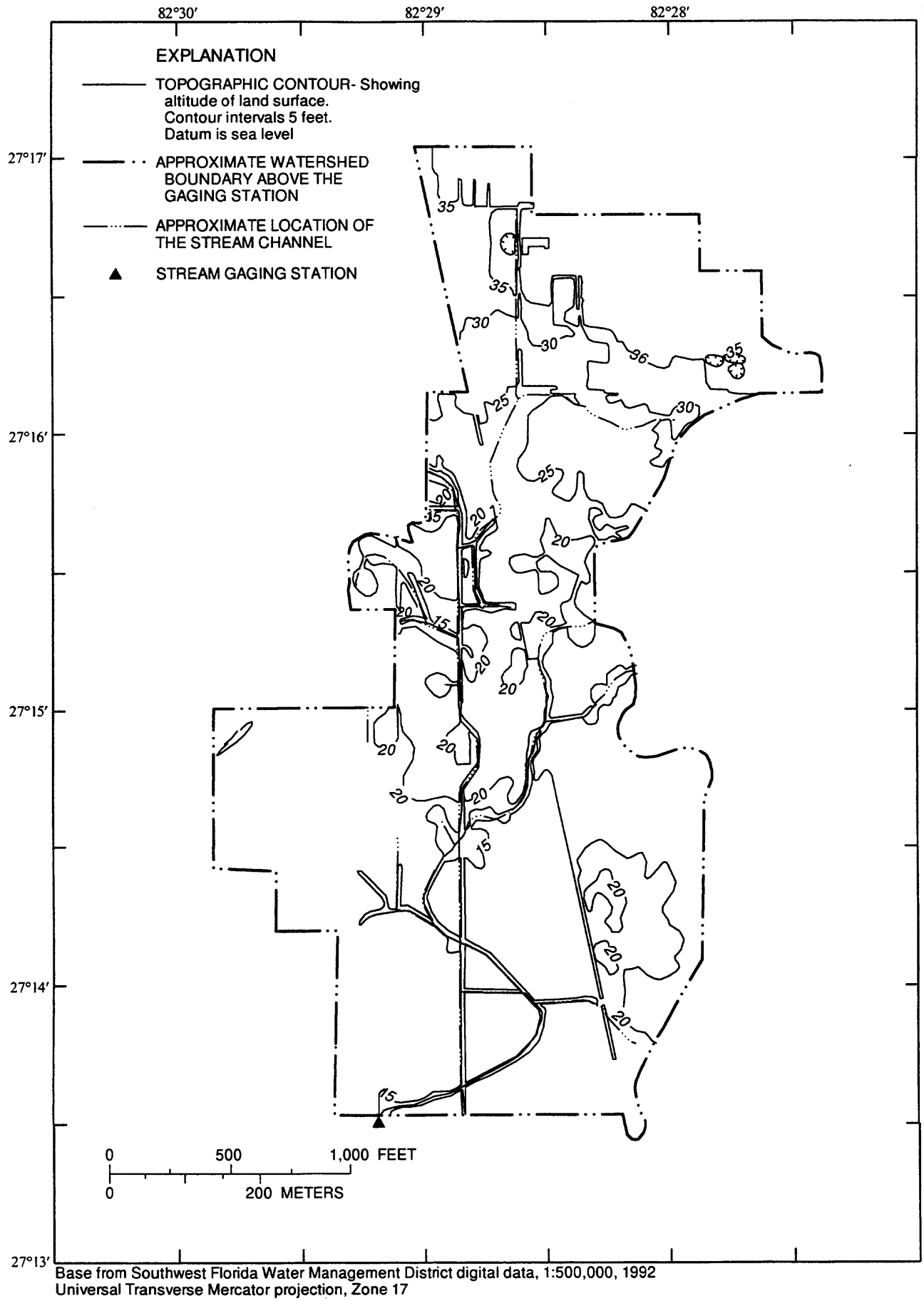


Figure 14. Catfish Creek watershed.

gaging station. The watershed is about 19,000 ft wide by 26,000 ft long. Average watershed slope is about 2.9 ft/mi. It is subdivided by 3 tributaries that converge upstream from the gaging station (fig. 15). The lengths of the main channel, lateral no.1, and lateral no. 2 are approximately 23,000 ft., 16,000 ft and 15,900 ft respectively. Maximum recorded discharge from the watershed was 442 ft^3/s.

The Forked Creek watershed (site 13, fig. 1), located in the southern part of Sarasota County, drains a 2.72 mi^2 area. About 30 percent of the watershed is cultivated in citrus and sod. Extensive cross ditching is present in this part of the watershed. The remainder is undeveloped native pastureland. Ponds or wetlands cover about 15 percent of the watershed. Land surface elevation ranges from 15 ft above sea level at the northwestern and southwestern watershed boundary to about 10 ft above sea level at the gage. The watershed is approximately 10,500 ft wide by 10,000 ft long (fig. 16). The slope in the watershed averages about 2.8 ft/mi. The main channel is about 12,400 ft long. Maximum recorded discharge from the watershed was 287 ft^3/s.

The Gottfried Creek watershed (site 14, fig. 1), located in the southern part of Sarasota County, drains a 2.0 mi^2 area. Land use is primarily residential with some commercial development. About 25 percent of the watershed consists of undeveloped land. Ponds or wetlands cover about 15 percent. Land surface elevation averages about 13 ft above sea level. The watershed is about 5,000 ft wide by 11,000 long (fig. 17). The slope is about 1.4 ft/mi. The main channel is about 11,000 ft in length. Maximum recorded discharge from the watershed was 119 ft^3/s.

The Rock Creek watershed (site 15, fig. 1), located in the southeastern corner of Sarasota County (fig. 1) is also known as Ainger Creek, and drains a 2.63 mi^2 area. It is an undeveloped natural watershed consisting of native pastureland and palmetto prairies, pine woods, and wetlands. Wetlands cover about 25 percent of the watershed. Land surface elevation averages about 12 ft above sea level. The watershed is approximately 6,000 ft wide by16,000 ft long (fig. 18). The slope is about 2.9 ft/mi. The main channel is about 16,000 ft in length. Maximum recorded discharge from the watershed was 109 ft^3/s.

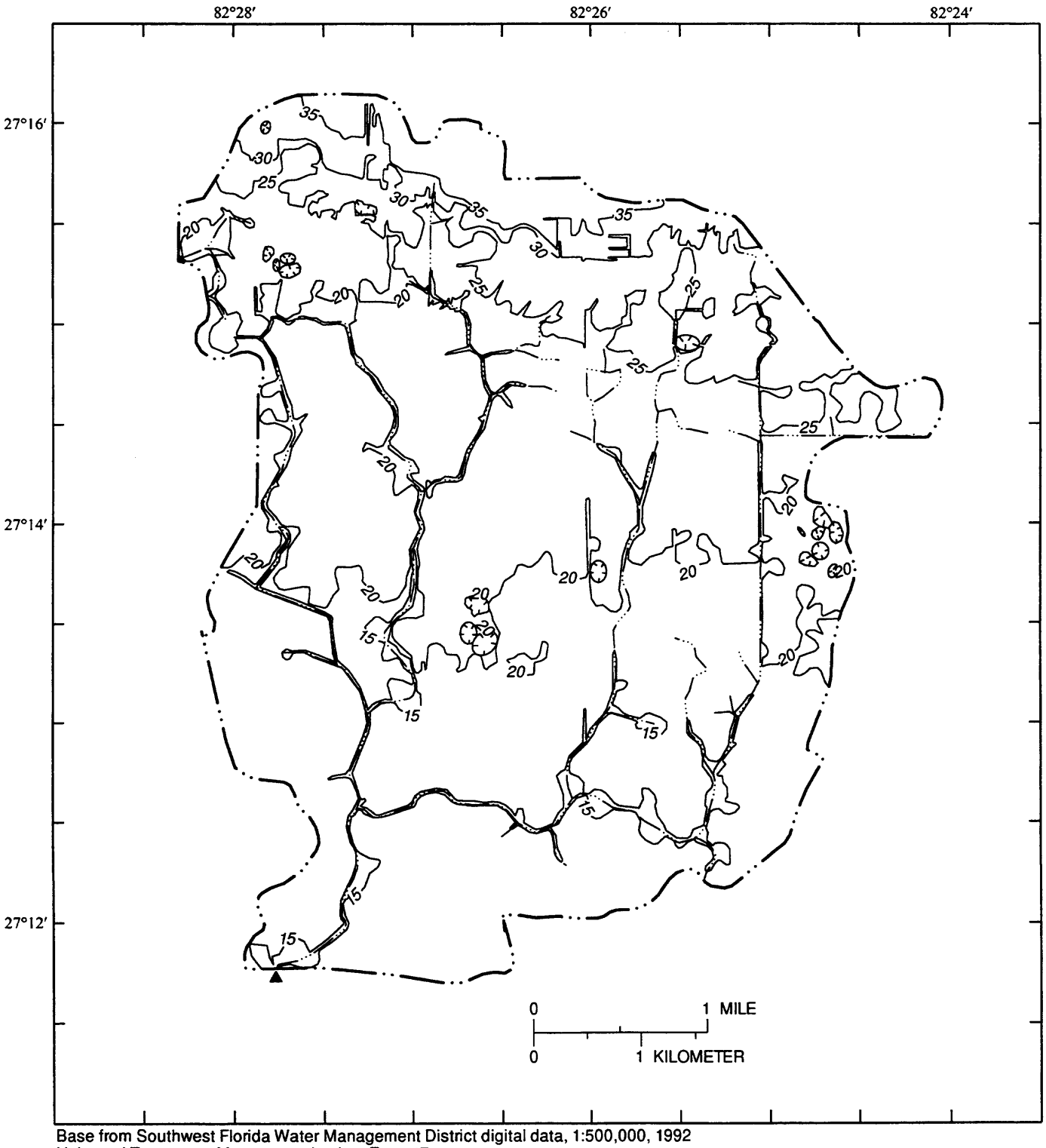


Figure 15. South Creek watershed.

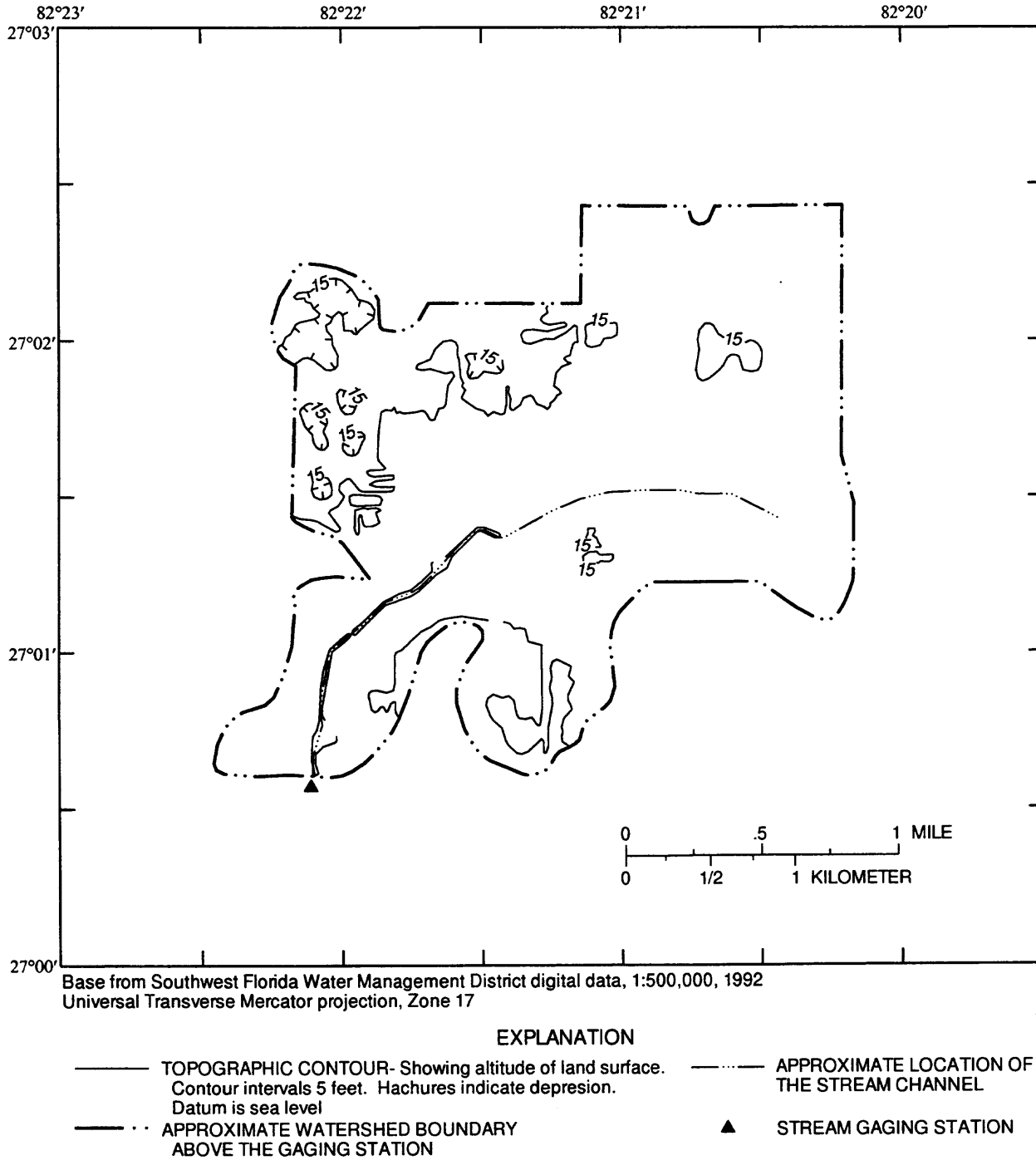


Figure 16. Forked Creek watershed.

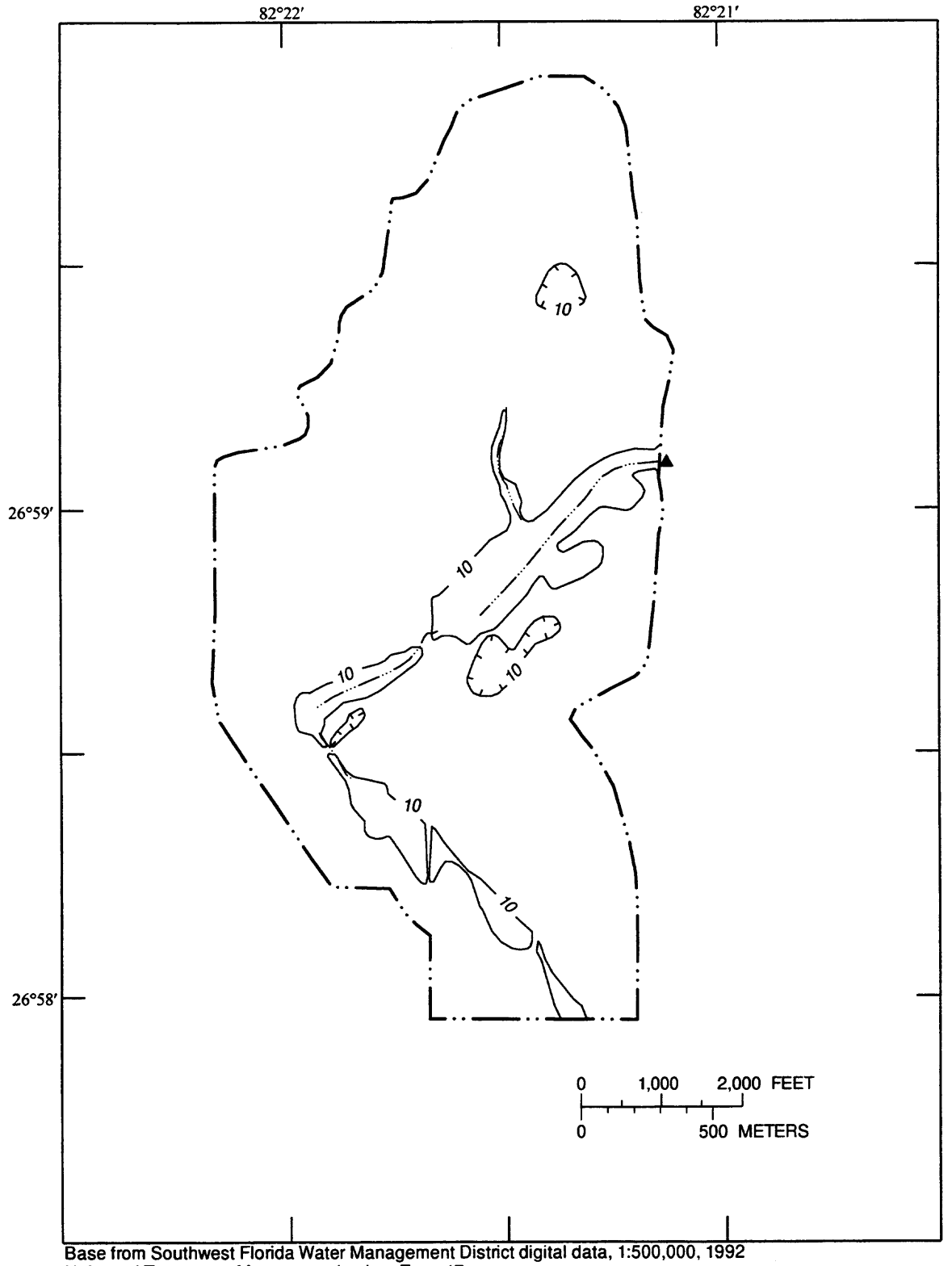


Figure 17. Gottfried Creek watershed.

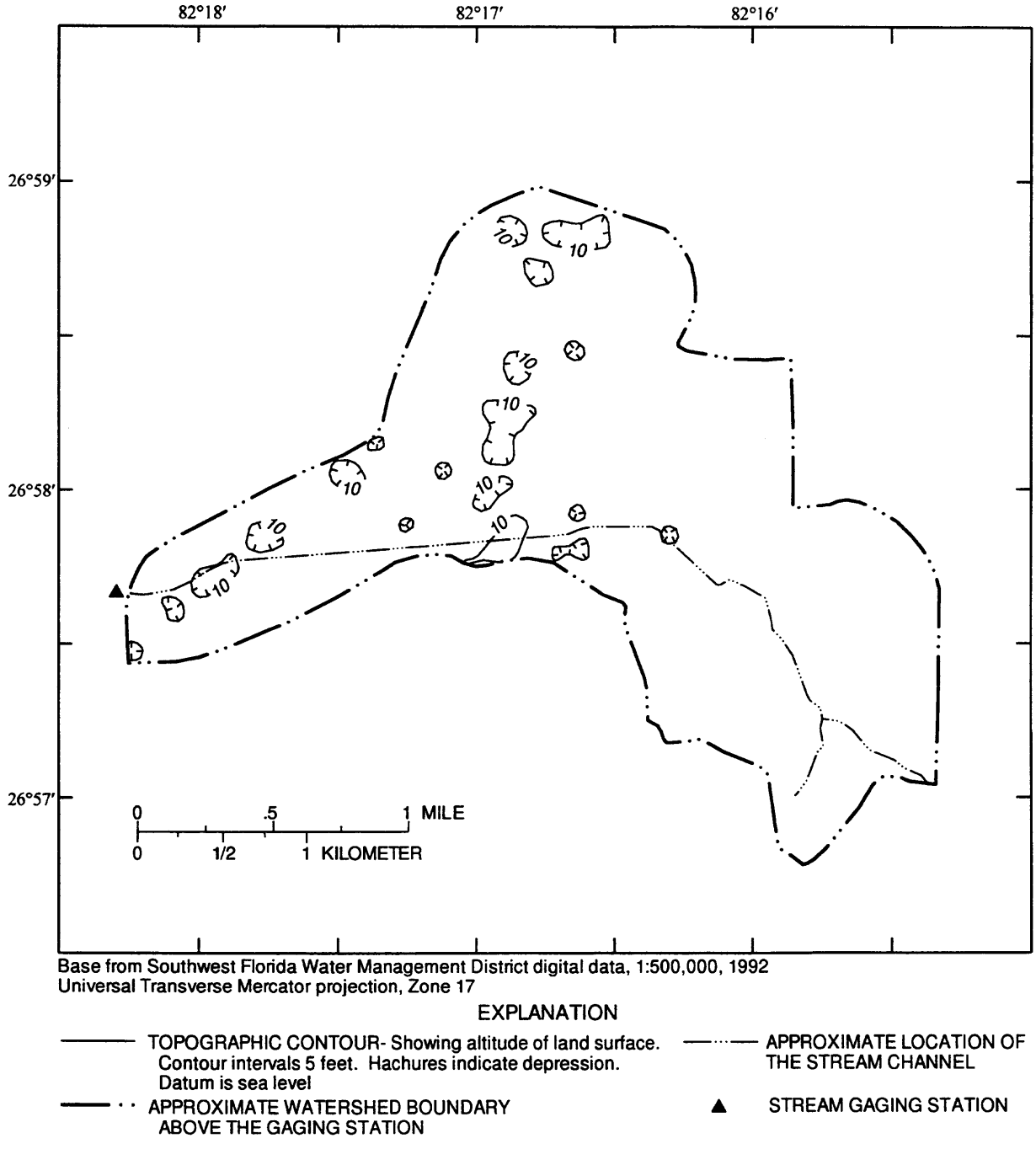


Figure 18. Rock Creek (Ainger Creek) watershed.

METHODS

Engineers engaged in the design of structures that require knowledge of peak discharges and storm water runoff volumes typically estimate these values using traditional techniques at ungaged watersheds. These techniques require one or more of the following: (1) estimation of watershed characteristics; (2) synthetic or design storm precipitation of specific recurrence intervals; and (3) extrapolation of empirical relationships beyond tested ranges. Estimation techniques used in this study were applied using computer programs and information readily available to engineers. Input parameters were estimated using each technique's recommended procedures. Recorded rainfall was used in most of these techniques to make comparisons between estimated and recorded peak discharges and runoff volumes. Rainfall amounts used for specific recurrence intervals were taken from the NRCS Publication No. 4-33137 (Soil Conservation Service, 1978) and Weather Bureau Technical Paper No. 49 (Weather Bureau, 1964). Table 2 shows these rainfall depths, in inches, for storms of given duration and recurrence intervals in west-central Florida. Rainfall depths for actual and synthetic storms were assumed to be uniform across the entire watershed. Evaluation of recorded discharge data was not used to influence the selection of input parameters. Comparison of estimates made using standard or accepted practices with actual measured rainfall and runoff from specific storms was, instead used to evaluate the reliability and accuracy of these techniques.

Rainfall and runoff data collected as part of the Tampa Bay area urban watershed study (Lopez and Woodham, 1983) and the study of unmined and reclaimed basins in phosphate mining areas (Lewelling and Wylie, 1993) were evaluated. Eight of these watersheds and 7 new sites established in Sarasota County were included in the study. Four hundred and fifty eight storms from these watersheds were evaluated. Sixty-six storms were selected for use in the evaluation of design techniques.

Five urban watersheds in Pinellas and western Hillsborough Counties, three natural watersheds in eastern Hillsborough and Hardee Counties, and the Clower Creek watershed in Sarasota County were modeled as single basin watersheds. The remaining six watersheds in Sarasota County were modeled with multiple subbasins.

Table 2. Rainfall depths for storms of given duration and recurrence intervals for west-central Florida, in inches

	Rainfall recurrence interval, in years					
Duration	**2**	**5**	**10**	**25**	**50**	**100**
30 minute	1.8	2.4	2.5	2.8	3.0	3.5
1 hour	2.2	2.8	3.0	3.4	3.8	4.0
2 hour	2.7	3.5	4.0	4.5	5.0	5.5
3 hour	3.0	4.0	4.5	5.0	5.5	6.5
6 hour	3.5	4.5	5.5	6.0	7.0	8.0
12 hour	4.5	5.5	6.5	8.0	9.0	10.0
24 hour	5.0	7.0	8.0	9.5	10.5	12.0
2 day	6.0	8.0	9.0	11.0	12.0	14.0
4 day	7.0	9.0	10.0	14.0	15.0	17.0

Watershed Characteristics

The following criteria were used in selecting the watersheds included in this study: (1) watersheds were small with relatively flat topography; (2) land use in the watersheds was typical of the types of development in watersheds in west-central Florida; (3) land use did not change during the data collection periods; and (4) a stage-discharge relation could be developed at the gaging station.

Watershed boundaries were delineated by outlining natural drainage divides and then modifying them to reflect changes resulting from development. Some of the watershed boundaries for Forked (fig. 16) and Rock Creeks (fig. 18) and possibly South Creek (fig. 15) are poorly defined because of low topographic relief. Delineation of these boundaries is uncertain because they may vary depending on rainfall intensity and water-level elevation in the wetlands along these boundaries. USGS 71/2-minute series topographic maps, Southwest Florida Water Management District (SWFWMD) 1:2400 aerial photographic maps that were interpreted to 1 ft topographic contours, drainage maps supplied by Sarasota County and private consulting companies, and field observations were used to make these determinations. Drainage areas and the area of lakes, ponds, wetlands, the various land use categories were determined by planimetering.

The channel slopes used in this report are the average slope of the main channel between points 10 and 85 percent of the distance from the gage to the watershed divide. They were determined from USGS topographic maps and SWFWMD topographic data. The main-channel length is the distance between the gaging station and the watershed or subbasin divide, or the confluence of a tributary with the main channel and the watershed or subbasin divide.

The concept of the time of concentration (T_c) is used for many runoff estimation methods and is related to watershed characteristics. The time of concentration is commonly considered to be the time it takes a flood wave to travel from the most distant part of the watershed to the point of discharge.

Data Collection

Rainfall, stage and discharge data were collected at the coastal watersheds in Hillsborough and Pinellas Counties from 1975 to 1980, at the eastern Hillsborough County and the Hardee County watersheds from 1988 to 1990, and at the Sarasota County watersheds from 1991 to 1993. Data collection stations were installed and operated using USGS standard methods and techniques (Carter

and Davidian, 1968).

Rainfall data were collected at the Hillsborough, Pinellas and Hardee County stations using an 8-in diameter standard calibrated funnel that drained into a 3-inch diameter pipe (Lopez and Michaelis, 1979, p22.). A digital recorder with a float and tape assembly was used to record rainfall accumulation to the nearest 0.01 in. Tipping-bucket rain gages and electronic data loggers were used to collect rainfall data at the stations in Sarasota County. Rainfall accumulation was also recorded to the nearest 0.01 in. Data were recorded at 5-minute intervals for the small urban watersheds in Hillsborough and Pinellas Counties and at 15-minute intervals for the stations in eastern Hillsborough, Hardee, and Sarasota Counties.

Stage at all the stations except Arctic Street (site 1, fig. 1), in Hillsborough County, was measured in stilling wells installed in the stream channel. Stilling wells are metal or polyvinylchloride (PVC) pipes, open to the channel through a series of holes which allow water in the stilling well to rise to the same level as the stream stage while dampening fluctuations caused by wind or turbulence. A gas-purged servo-controlled manometer or bubble gage (Buchanan and Somers, 1969) was used at the Arctic Street gaging station. Digital recorders or electronic data loggers were used at all stations and stage data were recorded to the nearest 0.01 ft. Data were recorded at 5-minute intervals at the small urban watersheds in Hillsborough and Pinellas Counties, and at the Clower Creek station in Sarasota County. Stage data for all other gaging stations were recorded at 15-minute intervals.

Recorded stage data were used to compute discharge at each station by means of a discharge rating. A discharge rating is the relation of the discharge to the stage at a gaging station (Kennedy, 1984). Discharge ratings were developed for each gaging station by plotting a series of discharge measurements against corresponding stage data throughout the range in stage experienced at the station. As many discharge measurements as possible were made during or immediately following major storm events to define the upper end of the rating curve. Discharge measurements were made using standard USGS methods described by Buchanan and Somers (1969).

Estimating Procedures

The five estimation techniques used to calculate peak discharges and runoff volumes are: (1) the Rational Method, (2) the USGS regional regression equations for Florida, (3) the NRCS (formerly the SCS) TR-20 model, (4) the Army Corps of Engineers HEC-1 model, and (5) the Environmental

Protection Agency (EPA) Storm Water Management model (SWMM).

All estimates were made using programs executed on a microcomputer (PC). A spread sheet program was used for the estimates using the rational method and the USGS regression equations. PC versions of the TR-20, HEC-1 and SWMM models were obtained directly from the Natural Resources Conservation Service, Army Corps of Engineers and the Environmental Protection Agency respectively.

The Rational Method

The rational method provides estimates of peak discharges. Volumes can be computed from the calculated peak discharge using a dimensionless unit hydrograph representative of the basin, if one has been developed. However, this was not done for this study.

The rational method is widely used to estimate peak discharge for design of sewers and culverts in sewered areas or natural watersheds with drainage areas less than 5 mi^2 (Williams. 1950). It is simple to understand and is easy to apply. The method uses the approximation that 1 acre-inch/hr is equal to 1 ft^3/s and assumes: (1) the maximum runoff that results from a storm has a duration equal to the time of concentration; and (2) the rate of runoff equals a percentage of the average rate of rainfall (Williams, 1950, p.309); and (3) rainfall intensity is constant. The method uses the following equation:

$$Q = CIA \qquad (1)$$

where

Q = the peak runoff, in acre-in/hr or ft^3/s;

C = coefficient of runoff;

I = average rainfall intensity, in in/hr;

A = area of the watershed, in acres.

Two parameters, watershed area and the average rainfall intensity, necessary for estimating peak

flow by this method, are easily measured or estimated. The coefficient of runoff (C); however, is not easily measured, and are typically subjective, based on watershed characteristics. The values of C used in this report were obtained from procedures and data tables for urban and agricultural areas presented by Williams (1950, p.314-315) and Viessman (1989, p.311).

In addition to the assumptions already mentioned, the discharge frequency is assumed equal to the selected rainfall frequency (U.S. Water Resources Council, 1981). This assumption is probably not accurate because peak flows reflect the combined effects of rainfall intensity, duration, and antecent moisture conditions as well as rainfall volume.

The U.S. Geological Survey Regional Regression Equation Method

The USGS regional regression equations were developed using a multiple linear regression analysis to relate peak discharges from 182 watersheds throughout Florida to various basin characteristics (Bridges,1982). The watersheds were between 1.83 and 3,066 mi^2 in size, had slopes between 0.15 and 24.2 ft/mi, and had wetland areas that ranged between 0 and 28.2 percent. The solution of these equations, therefore, provides a peak discharge rate for a watershed with an average of these characteristics. The most significant basin characteristics were drainage area, lake area, and channel slope.

The State of Florida was divided into three hydrologic regions and a separate equation was developed for each region. All of the watersheds used in this study are within Region A. The equation for Region A has the following form:

$$Q_T = CDA^{B_1}SL^{B_2}(LK + 3.0)^{B_3} \qquad (2)$$

where

Q_T = the discharge for a recurrence interval of T-years, in cubic feet per second

C = the regression constant;

DA = the drainage area, in square miles;

SL = the channel slope, in feet per mile;

LK = lake area (or wetland areas) plus a constant of 3, in percent;

B_1, B_2, B_3 = exponents of the regression.

The regression constant and exponents were obtained from Bridges (1982, p.9) and are shown in table 3 of this report. The peak discharge for storm events of the 2, 5, 10, 25, 50, and 100 year recurrence intervals were computed by substituting the basin characteristics for each watershed into equation 2, using the exponents for each recurrence interval.

An equation developed by Stricker and Sauer (1982, p.19) was derived by multiple linear regression analysis to estimate runoff volumes associated with peak discharges for storms with specific recurrence intervals. It was developed separately from the Florida regional equations by relating runoff volumes to flood peaks and basin characteristics for 55 watersheds located in Pennsylvania, Missouri, Oklahoma, Oregon and Texas. The equation has the following form:

$$V = 0.0142\,(A)^{-0.75}\,(LT)^{0.63}\,(Q_p)^{0.72} \qquad (3)$$

where

V = runoff volume, in inches;

A = contributing drainage area, in square miles;

LT = lag time, in hours;

Q_p = peak discharge, in cubic feet per second.

The watershed response time, or lag time (LT) is required for this estimate. The lag time is generally considered to be the elapsed time between the center of rainfall excess and the center of the runoff volume. LT was estimated by an equation originally developed by Sauer and others (1981) and simplified by Stricker and Sauer (1982, p.2).

Table 3. Regression constant and exponents used for the U.S. Geological Survey regression equations, region A

Recurrence interval T, in years	Regression constant, c	Regression Exponents		
		B_1	B_2	B_3
2	93.4	0.756	0.268	-0.803
5	192	.722	.255	- .759
10	274	.708	.248	- .738
25	395	.696	.240	- .717
50	496	.690	.234	- .705
100	609	.685	.227	- .695

The equation has the following form:

$$LT = 0.85\,(L/(SL^{0.5}))^{0.62}\,(13 - BDF)\,0.47 \qquad (4)$$

where

LT = lag time, in hours;

L = watershed length, in miles;

SL = the main channel slope, in feet per mile;

BDF = basin development factor, determined using methods developed by Sauer and others (1981). The BDF will range from 0 to 12.

Peak discharges estimated from equation 2 and the lag time estimated from eq. 4 can be substituted into eq. 3 to estimate the runoff volume for a specified recurrence interval.

The Natural Resources Conservation Service TR-20 Model

One of the most commonly used methods for estimating peak discharges and runoff volumes was developed by the Natural Resources Conservation Service. It is relatively simple and can be applied to a wide range of watershed conditions. Although computations for the method can be done manually, they are frequently accomplished using a digital computer as described in TR-20 (Technical Release No. 20). The TR-20 method is a single-event model that computes direct runoff, storm hydrographs, and routes the flow through stream channels and reservoirs. It combines hydrographs at subbasin boundaries (if the watershed has been subdivided) and computes peak discharge, time of occurrence and runoff volume (Soil Conservation Service, 1983). Another NRCS program, TR-55 (Soil Conservation Service, 1986) further simplifies the curve number method; however, the program cannot be used for watersheds or watershed subbasins having times of concentration greater than 2 hrs. This program was not used in this study because times of concentration for most watersheds or watershed subbasins in Sarasota County were longer than 2 hrs.

Rainfall from 66 storms were used to generate peak flow and volume estimates. The model calculates runoff from rainfall by using the NRCS runoff equation and a watershed storage parameter calculated as a function of a curve number (CN). The NRCS runoff equation has the following form:

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \qquad (5)$$

where

Q = watershed runoff, in inches;

P = rainfall, in inches;

S = maximum retention after runoff begins, in inches;

The development of these procedures is outlined in chapter 10, NEH-4 (National Engineering Handbook, Section 4-Hydrology, Soil Conservation Service, 1985). The CN is determined from watershed characteristics, such as soils, land use, amount of impervious area, interception, and surface storage. Larger or more complex watersheds were divided into subbasins to more accurately define the CN. CN's were determined using procedures contained in chapters 7, 8, and 9 of the NEH-4 (SCS, 1985). The calculated CN assumes average watershed antecedent moisture conditions (AMC=II). Antecedent moisture conditions can be varied in the model to account for dry conditions (AMC=I) or wet conditions (AMC=III). Most model simulations made in this study use AMC=II condition. The NRCS (Soil Conservation Service, 1986, Florida Bulletin No. 210-7-2) recommends that AMC=II be used for Florida. However, some estimates were made using AMC=III because of wet conditions existing in the watersheds resulting from summer thunder storms that closely followed proceeding storms.

The peak discharge is determined by converting runoff from the watershed or watershed subbasin to a runoff hydrograph using a dimensionless unit hydrograph and the peak rate equation. Chapter 16 of the NEH-4 (Soil Conservation Service, 1985) describes the hydrograph development method and peak rate equation used by the NRCS. The time of concentration (T_c) of the watershed is used in this procedure and is defined in chapter 15 of NEH-4 as the time it takes for runoff to travel from the hydraulically most distant part of the watershed to the watershed outlet. The T_c is related to the watershed lag time (L) by the following empirical relationship:

$$L = 0.6T_c \quad (6)$$

where

L = watershed lag time, in hours;

T_c = Time of concentration, in hours.

The watershed lag time is the elapsed time between the center of rainfall volume and the center of runoff volume and was estimated for these simulations using the following SCS equation:

$$L = \frac{l^{0.8}(S+1)^{0.7}}{1900Y^{0.5}} \quad (7)$$

where

L = watershed lag time, in hours;

l = hydraulic length of the watershed, in feet;

$S = \frac{1000}{CN'} - 10$, in inches (where CN' is approximately equal to CN);

Y = average watershed land slope, in percent

The standard dimensionless unit hydrograph built into the model was not used to estimate peak discharge rates and runoff volumes for this study because it has a peak rate factor of 484. The NRCS (Soil Conservation Service, 1986) recommends that a dimensionless unit hydrograph with a peak rate factor of 284 be used in Florida. The 284 unit hydrograph was used in all estimates made using the TR-20 model.

Watershed subbasin hydrographs were routed through reservoirs or stream reaches where necessary and added algebraically at the confluence. Reservoir routing uses the storage-indication method which is based on the hydrologic storage routing equation (Soil Conservation Service, 1985, Chapter 17). The starting elevation for routing, or the pool elevation when runoff begins had to be specified. Estimates of elevation, discharge, and storage were computed from SWFWMD aerial

photographs with 1 ft topographic contours. Hydrographs are routed through a stream reach using the Modified Attenuation-Kinematic method, which combines the hydrologic storage equation with a kinematic model (Soil Conservation Service, 1983).

The Army Corps of Engineers HEC-1 Model

The HEC-1 model was developed by the Army Corp of Engineers as a single event model designed to simulate the surface runoff response of a watershed to precipitation by representing the watershed as an interconnected system of hydrologic and hydraulic components. The model components are based on simple mathematical relationships that are intended to represent average conditions for the meteorologic, hydrologic and hydraulic processes. The HEC-1 model gives the user choices of methods to calculate precipitation, interception/infiltration (precipitation loss), transformation of rainfall to runoff, and flood hydrograph routing (Hydrologic Engineering Center, 1990).

Measured rainfall for actual storms was used for the HEC-1 simulations. Precipitation loss can be calculated by five different methods within the HEC-1 model, but the curve number method was used because the other methods require input parameters or coefficients that are difficult to estimate for ungaged watersheds or are more appropriate for small cultivated agricultural watersheds. Average antecedent moisture conditions were used in this procedure. An equivalent AMC=III CN was computed for storm events when wet conditions existed in the watersheds.

Rainfall excess was transformed to runoff by the unit hydrograph method. Three synthetic unit hydrograph methods are available within the model, the Snyder, Clark, and NRCS methods. The Snyder and Clark methods require input of storage and peaking coefficients which are difficult to estimate for ungaged watersheds typical of those in west-central Florida. Therefore, the NRCS unit hydrograph was used. The standard unit hydrograph (484 peak rate factor) is contained within the HEC-1 model. The source code would have had to be modified and the program recompiled to enter the 284 peak rate factor into the unit hydrograph program. Most users would not go through the process of making these changes before applying the model; therefore, the standard 484 unit hydrograph was used for the simulations.

Watershed subbasin hydrographs were routed through stream reaches or reservoirs and combined where necessary. Reservoir routing was accomplished using the storage routing method (Hydrologic

Engineering Center, 1990). Storage volume and the elevation where runoff begins are required input parameters for this method. They were estimated using SWFWMD aerial photographs with 1 ft topographic contours. Hydrograph flood routing can be accomplished using one of 6 methods; however, only the Muskingum-Cunge and kinematic wave channel routing methods require input parameters that can be easily measured or estimated. The kinematic wave method is most appropriately used in urban watersheds where flood wave attenuation is not significant. The Muskingum-Cunge method was used for this study because many of the watersheds, including some urban watersheds in Pinellas and western Hillsborough Counties have stream channels where flood wave attenuation is significant, due to low slopes, natural densely vegetated channels, or tailwater control.

The Environmental Protection Agency Storm Water Management Model

The Storm Water Management model, developed by the Environmental Protection Agency, can be used as either a single-event or continuous simulation model. For this study, the model was used only as a single-event model. It simulates storm events by using rainfall and watershed characterization. The model is organized in the form of "blocks." There are four computational blocks and 6 service blocks in the model. Up to 25 blocks can be run sequentially; however, the model is typically run using only the executive block and one or two computational blocks. A detailed explanation of the model's properties, processes and requirements are contained in the user's manual (Huber and Dickinson, 1988). The runoff and extended transport (extran) computational blocks and the executive and graph service blocks were used for this study.

The runoff block generates surface runoff in response to rainfall. The block accepts rainfall and calculates infiltration, surface detention, and overland and channel flow. Rainfall from measured storms was used as input. The SWMM model has two options for calculating infiltration; the Green-Ampt equation, and an integrated form of Horton's equation. Both the Green-Ampt and Horton's equations were used in separate simulations. Except for the urban watersheds in Pinellas and western Hillsborough Counties and the Clower Creek watershed in Sarasota County, infiltration was also routed through subsurface pathways. Infiltration can be routed through the unsaturated and saturated zones to a channel or junction, or be lost as evapotranspiration or to a deep groundwater zone. Subsurface routing was not used in the urban watersheds because of the high percentage of impervious area and the presence of sewered drainage systems. Overland flow is calculated in the runoff block by

approximating the watersheds as non-linear reservoirs by coupling a spatially-lumped continuity equation with Manning's equation.

The runoff block of the SWMM model cannot simulate backwater effects on flood hydrographs being routed through watersheds with multiple subbasins. The extran block solves the equations, accounting for backwater effects as well as flow reversal, pressure flow, and surcharging (backup, storage, and slower release of water) at junctions (Roesner and others, 1988). Significant backwater and some surcharging occurs in the watersheds in Sarasota County. The Walker, Catfish, South, Forked, Gottfried, and Rock Creek watersheds in Sarasota County were modeled using multiple subbasins which allowed for a greater degree of spatial detail. Extran channel routing, therefore, was used for all simulations where the watersheds were modeled with multiple subbasins. Channel routing was not used for single basin watersheds.

COMPARISON OF OBSERVED TO ESTIMATED RUNOFF

Estimated peak discharges and runoff volumes for 66 storms in 15 watersheds were compared with observed peak discharges and runoff volumes using the USGS regional regression equations, TR-20, HEC-1, and SWMM explained previously. Peak discharge only was calculated using the rational method. The regression method uses input parameters based on synthetic rainfall events for specific recurrence intervals rather than actual rainfall depths. Therefore, only 16 of the observed storms that matched equivalent recurrence interval synthetic storms were available from which direct comparisons could be made.

The Rational Method

The rational method overestimated peak discharges for most storms (table 4, and fig. 19). Forty-five storms were overestimated, twenty were underestimated, and one estimated discharge was the same as the observed.

Errors between estimated and observed peak discharges were generally smaller for the six urban watersheds than for the natural or mixed watersheds. Errors were 211 percent or less, and averaged about 11 percent in the urban watersheds, except for the Kirby Street watershed, which had errors as high as 637 percent, and averaged about 525 percent. Unlike other urban watersheds, the Kirby

Table 4. Input parameters and comparison of estimated and observed peak discharges using the rational method

[C, coefficient of runoff; I, rainfall intensity (in/hr); A, drainage area; cfs, cubic feet per second; E, early; L, late; -, negative values represent underestimations; U, urban; N, natural; M, mixed]

Watershed name	Watershed classification	Input parameters			Peak discharge (cfs)		Error		Date of storm
		C	I	A	Estimated	Observed	cfs	Percent	
Arctic Street storm drain	U	0.4	1.50	218	131	120	11	9.2	08/03/76
		0.4	1.67	218	146	133	13	9.8	08/04/76
		0.4	2.24	218	195	137	58	42.3	09/26/77
		0.4	0.93	218	81	142	-61	-43.0	05/20/78
Kirby Street drainage ditch	U	0.3	1.90	736	420	57	363	637	07/19/75
		0.3	1.81	736	400	95	305	321	08/30/75
		0.3	3.12	736	689	96	593	618	08/15/78
St. Louis Street drainage ditch	U	0.5	0.86	326	140	357	-217	-60.8	05/15/76
		0.5	1.24	326	202	226	-24	-10.6	06/18/76
		0.5	1.57	326	256	326	-70	-21.5	06/29/77
Gandy Boulevard drainage ditch	U	0.5	0.92	826	380	223	157	70.4	06/18/75
		0.5	0.44	826	182	301	-119	-39.5	07/11/75
		0.5	1.56	826	644	207	437	211	08/07/75
		0.5	0.49	826	202	692	-490	-70.8	05/15/76
		0.5	0.77	826	318	410	-92	-22.4	05/17/76
Allen Creek	U	0.5	1.15	1203	692	341	351	103	07/28/76
		0.5	0.74	1203	445	379	66	17.4	07/01/77E
		0.5	0.82	1203	493	819	-326	-39.8	07/01/77L
		0.5	0.70	1203	421	335	86	25.7	07/03/77
		0.5	0.15	1203	102	89	13	14.6	12/02/77
		0.5	0.28	1203	168	286	-118	-41.3	02/18/78
IMC Creek	N	0.3	0.32	109	10	11	-1	-9.1	11/23/88
		0.3	2.40	109	78	5	73	1460	07/12/89
		0.3	0.41	109	13	4	9	225	02/23/90
		0.3	1.29	109	42	9	33	367	07/21/90
Grace Creek	N	0.2	0.70	422	59	59	0	0.0	08/07/88
		0.2	0.99	422	84	40	44	110	08/23/88
		0.2	1.20	422	101	16	85	531	07/12/90
		0.2	0.33	422	28	25	3	12.0	07/14/90
CFI-3 Creek	N	0.3	0.28	90	8	19	-11	-57.9	07/05/89
		0.3	0.15	90	4	7	-3	-42.9	02/23/90
		0.3	1.05	90	28	6	22	367	06/02/90
Walker Creek	M	0.4	0.16	3059	196	971	-775	-79.8	June 92
		0.4	1.28	3059	1566	438	1128	258	07/23/92
		0.4	0.71	3059	1738	398	1340	337	08/07/92
		0.4	1.59	3059	1946	334	1612	483	09/04/92
		0.4	1.95	3059	2386	278	2108	758	09/05/92
		0.4	0.46	3059	563	199	364	183	09/25/92
		0.4	0.65	3059	795	292	503	172	09/26/92
		0.4	0.41	3059	502	235	267	114	01/15/93
		0.4	0.49	3059	600	319	281	88.1	04/01/93
		0.4	0.53	3059	649	237	412	174	07/01/93
Clower Creek	U	0.6	0.39	224	52	77	-25	-32.5	02/05/92
		0.6	0.14	224	19	205	-186	-90.7	June 92
		0.6	1.09	224	146	66	80	121	09/02/92
		0.6	0.78	224	105	110	-5	-4.5	09/13/92
		0.6	0.50	224	67	42	25	59.5	01/14/93
		0.6	0.90	224	121	60	61	102	03/13/93

Table 4. Input parameters and comparison of estimated and observed peak discharges using the rational method

[C, coefficient of runoff; I, rainfall intensity (in/hr); A, drainage area; cfs, cubic feet per second; E, early; L, late; -, negative values represent underestimations; U, urban; N, natural; M, mixed] (Continued)

Watershed name	Watershed classification	Input parameters			Peak discharge (cfs)		Error		Date of storm
		C	I	A	Estimated	Observed	cfs	Percent	
		0.6	0.81	224	109	116	-7	-6.0	04/01/93
Catfish Creek	M	0.3	0.56	3053	513	70	443	633	01/14/93
		0.3	0.22	3053	201	76	125	164	01/15/93
		0.3	0.91	3053	833	140	693	495	03/13/93
		0.3	0.81	3053	742	300	442	147	04/01/93
South Creek	N	0.2	0.17	9875	328	442	-114	-25.8	June 92
		0.2	0.47	9875	928	143	785	549	09/06/92
		0.2	0.43	9875	849	96	753	784	09/13/92
		0.2	0.98	9875	1936	94	1842	1960	03/13/93
		0.2	0.52	9875	1126	168	958	570	04/01/93
Forked Creek	N	0.3	0.13	1741	68	287	-219	-76.3	June 92
		0.3	0.92	1741	481	45	436	969	08/09/92
Gottfied Creek	M	0.3	0.12	1280	46	119	-73	-61.3	June 92
		0.3	0.54	1280	207	21	186	886	08/11/92
		0.3	0.11	1280	42	18	24	133	10/02/92
Rock Creek	N	0.3	0.27	1683	136	109	27	24.8	June 92
		0.3	0.49	1683	247	24	223	929	09/25/92
		0.3	0.10	1683	50	25	25	100	10/02/92

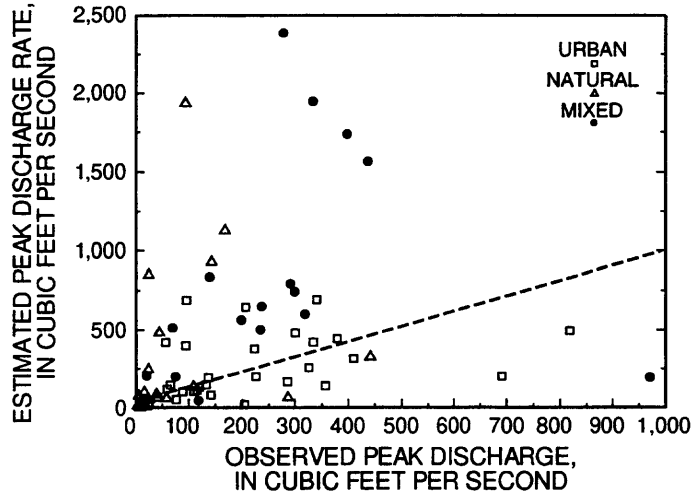


Figure 19. Comparison of observed and estimated peak discharge rates using the rational method.

watershed does not drain through storm sewers and has less impervious area and more wetland area; however, it is not substantially different from the St. Louis Street watershed, which produced more accurate estimates.

Peak discharges for most storms for the six natural watersheds were overestimated. One storm in the IMC Creek watershed was overestimated by 1460 percent and one in the South Creek watershed was overestimated by 1960 percent. The average for the 6 natural watersheds was 410 percent.

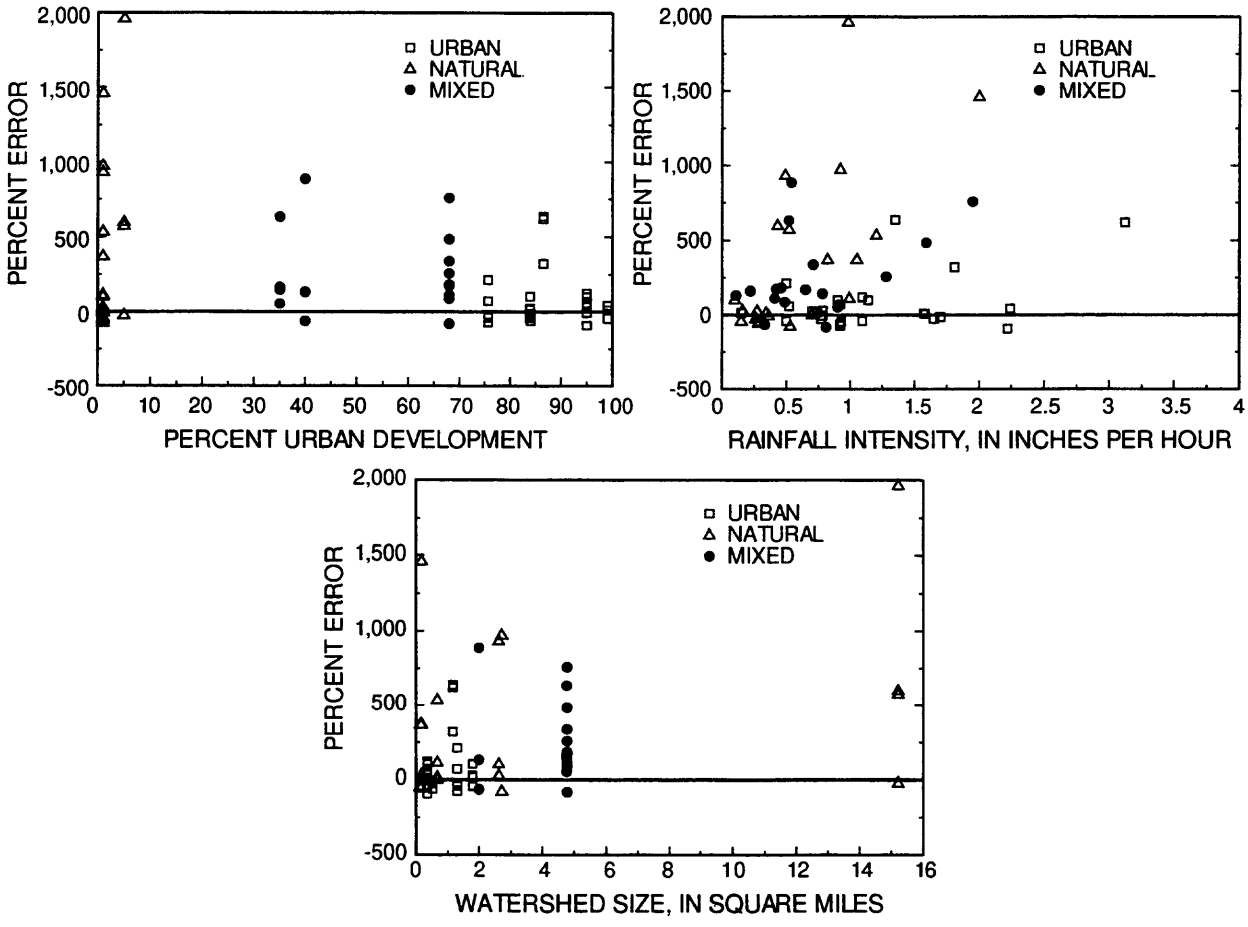
For the 3 mixed land use watersheds, peak discharges were overestimated for 15 of 17 storms. Errors ranged from an underestimation of 79.8 percent to an overestimation of 886 percent, and averaged about 287 percent.

Infiltration, surface detention, and time of concentration are controlling influences in larger natural watersheds. The rational method does not directly use watershed characteristics in the calculation of peak discharges; therefore, it is not sensitive to these characteristics.

Figure 20 shows comparisons of the error (in percent) between estimated and observed peak discharge for the modeled storms and the amount of urban development, rainfall intensity, and watershed size. The amount of urban development present in the watershed had the most effect on the accuracy of estimated peak discharges. Estimation errors decrease as urban development increases. There appears to be some correlation between the percent error and the rainfall intensity when rainfall intensity is below 0.5 in/hr. There is no apparent correlation above 0.5 in/hr and the range of differences is much greater. Williams (1950, p. 317) states that computed peak discharge rates for short duration, high intensity storms may be higher than those computed for low-intensity storms using this method. Most of the storm events modeled during this study were short duration, high intensity summer thunder storms, increasing the probability that peak discharges would be overestimated. The size of the watershed seems to have no correlation with the error.

The U.S. Geological Survey Regional Regression Equation Method

The USGS regression equations can not use rainfall from specific storms to calculate peak discharges. The method is based on the flood frequency distributions of gaged stream flows; therefore, direct comparison of estimated and observed discharges from actual storms cannot be made. However, observed peak discharge and runoff volumes from actual storm events were compared to the estimates of equivalent storm events for specific rainfall recurrence intervals. The flood frequency and rainfall



igure 20. Comparison of the percent error between observed and estimated peak discharge rates with watershed evelopment, rainfall intensity, and watershed size using the rational method.

recurrence intervals were assumed to be equal. Appendix I presents the estimated peak discharge rates and runoff volumes for storms with a 2, 5, 10, 25, 50, and 100 year recurrence interval for the 15 watersheds included in the study. Of the 66 measured storms, only 16 had rainfall with a recurrence interval equivalent to the estimated runoff interval. Peak discharges and runoff volumes for the observed storms and the estimates for the equivalent storms are compared in table 5 and figure 21.

The regional regression equations developed by Bridges (1982) overestimated peak discharge for 6 storms and underestimated it for 10 storms. For the same storms, the runoff equations developed by Stricker and Sauer (1982) overestimated the runoff volume for 4 storms and underestimated it for 12 storms.

Average errors for the estimated peak discharges and runoff volumes were 25 and 32 percent less than observed for the urban watersheds, and 14 and 79 percent for the mixed watersheds. In the natural watersheds the error varied considerably. The errors for the South, Forked, and Rock creek watersheds were 12 and 92 percent less than observed discharges and runoff volumes. In the IMC and CFI-3 watersheds in eastern Hillsborough and Hardee Counties, errors averaged 711 and 307 percent greater than observed peak discharges and runoff volumes. Large errors in the IMC and CFI-3 watersheds could be caused by drainage area and slopes which are outside the range of those used to develop the regression equations. The smallest errors in peak discharges were for storms occurring in the South Creek and Rock Creek watersheds. The runoff volume for storms in these watersheds; however, was greatly underestimated. The watershed characteristics for these watersheds more closely resembled the watershed characteristics for sites used to develop the peak discharge regression equations, but are outside the range of watershed characteristics for the sites used to develop the runoff volume regression equations.

Comparison of the error (in percent) between estimated and observed peak discharge and runoff volume and the percentage of development in the watershed, the watershed size, watershed slope and the percentage of the watershed covered by wetlands are shown in figures 22 and 23. There appears be a slight correlation between the percent error for the peak discharges and these watershed characteristics. The errors become smaller as the watershed development, size, and wetland areas increase, and as the watershed slope decreases. There is no similar correlation for runoff volume. When the IMC and CFI-3 watersheds are not included in the comparisons, correlation between the percent error and watershed characteristics is not evident for peak discharges or runoff volumes.

Table 5. Comparison of peak discharges and runoff volumes estimated using the U.S. Geological Survey regional regression equations and equivalent observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; -, negative values represent underestimations; U, urban; N, natural; M, mixed]

Watershed name	Watershed classification	Recurrence interval	Peak discharge (cfs)				Runoff volume (in)				Date of storm
					Error				Error		
			Estimated	Observed	cfs	Percent	Estimated	Observed	inches	Percent	
Arctic Street storm drain	U	2	18	142	-124	-87.3	0.16	1.30	-1.14	-87.7	05/20/78
Kirby Street drainage ditch	U	2	75	57	18	31.6	.59	0.30	0.29	96.7	07/19/75
		2	75	95	-20	-21.0	.59	.77	-.18	-23.4	08/30/75
		5	157	96	61	63.5	1.00	.76	.24	31.6	08/15/78
Gandy Boulevard drainage ditch	U	2	57	223	-166	-74.4	.24	.50	-.26	-52.0	06/18/75
IMC Creek	N	5	62	5	57	1140	.72	.17	.55	324	07/12/89
CFI-3 Creek	N	2	23	6	17	283	.39	.10	.29	290	06/02/90
Walker Creek	M	2	164	438	-274	-62.6	.26	.91	-.65	-71.4	07/23/92
		2	164	334	-170	-50.9	.26	.77	-.51	-66.2	09/04/92
		50	845	971	-126	-13.0	.84	6.89	-6.05	-87.8	June 92
Clower Creek	U	2	24	110	-86	-78.2	.18	2.31	-2.13	-92.2	09/13/92
		100	182	205	-23	-11.2	.76	17.08	-16.32	-95.6	June 92
South Creek	N	100	432	442	-10	-2.26	.42	4.30	-3.88	-90.2	June 92
Forked Creek	N	50	164	287	-123	-42.9	.52	8.54	-8.02	-93.9	June 92
Gottfied Creek	M	50	205	119	86	72.3	.61	6.70	-6.09	-90.9	June 92
Rock Creek	N	50	118	109	9	8.26	.50	5.64	-5.14	-91.1	June 92

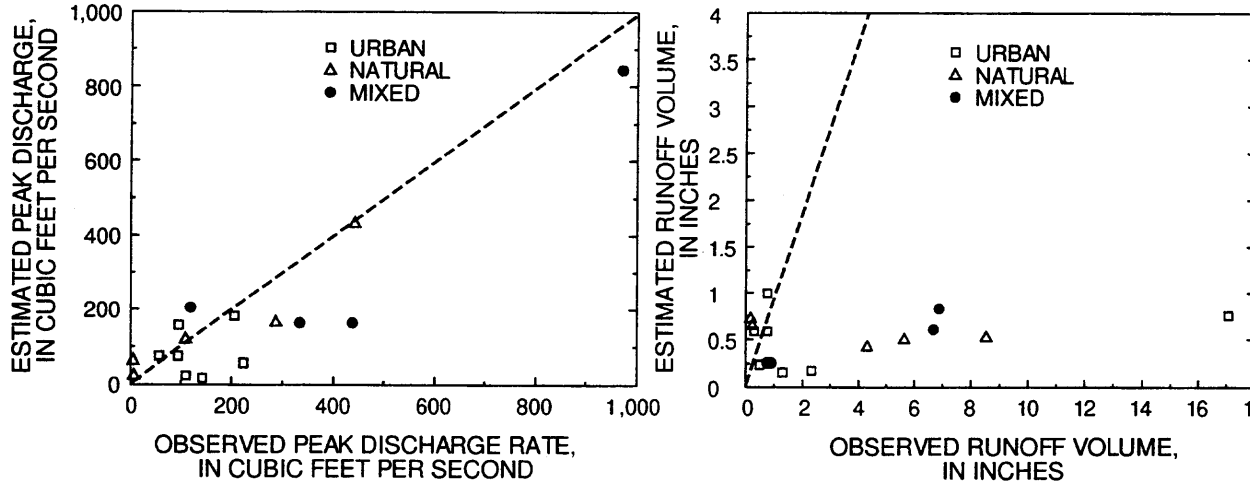
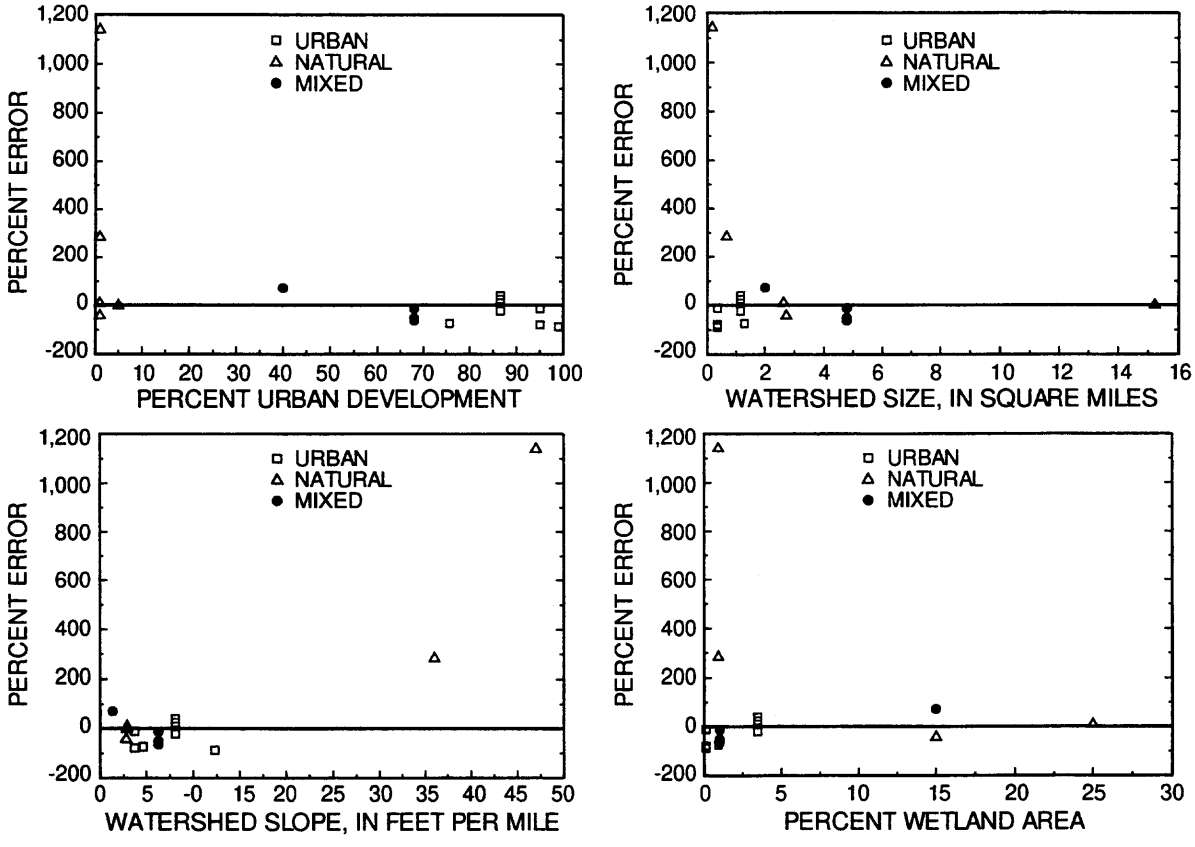


Figure 21. Comparison of observed and estimated peak discharge rates and runoff volumes using the U.S. Geological Survey regional regression equations.



gure 22. Comparison of the percent error between observed and estimated peak discharge rates with watershed velopment, size, slope, and wetland area using the U.S. Geological Survey regional regression equations.

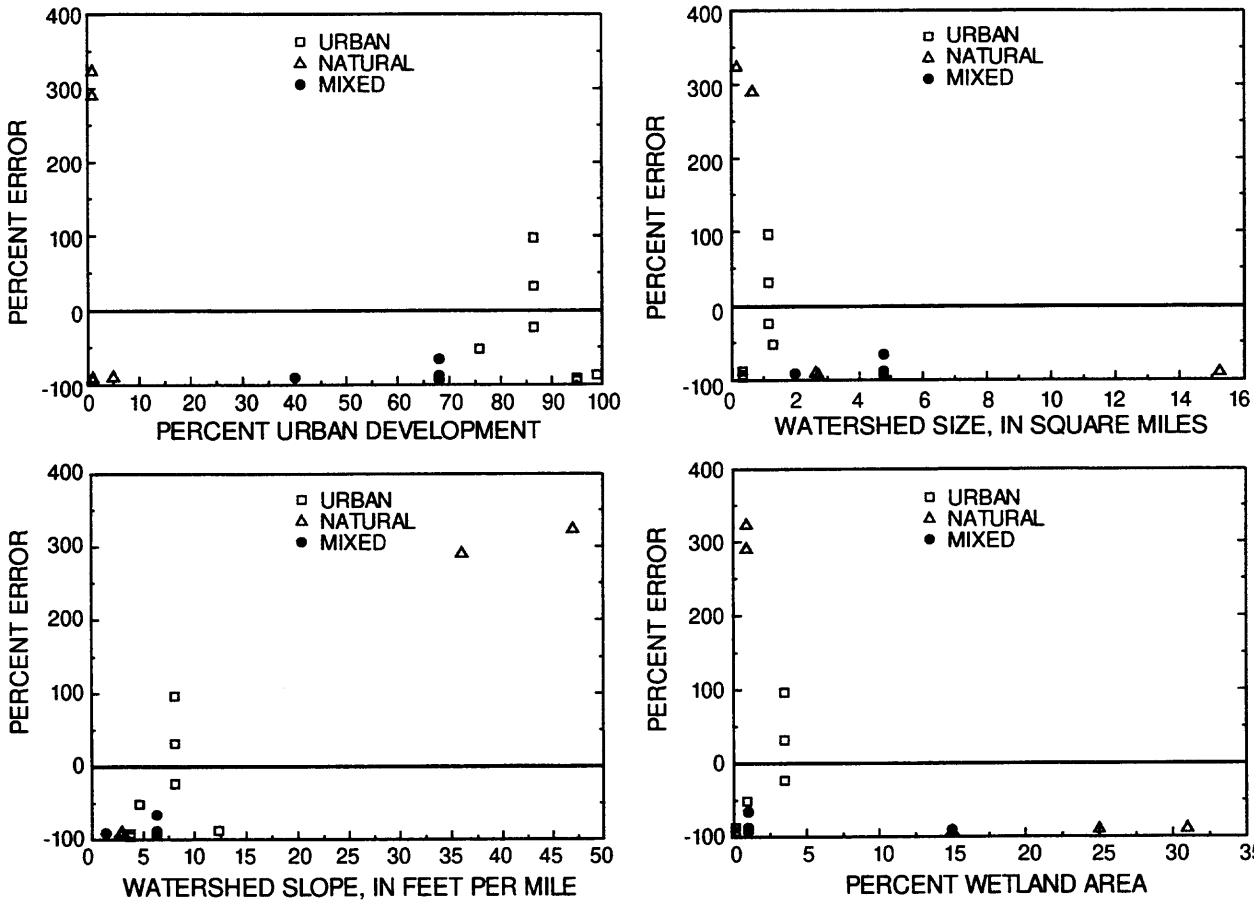


Figure 23. Comparison of the percent error between observed and estimated runoff volumes with watershed development, size, slope, and wetland area using the U.S. Geological Survey regional regression equations.

Most of the basin characteristics for the watersheds included in this study fall within the ranges of those used by Bridges (1982) to develop the regression equations for peak discharge; however, many of them fall at the extremes of these ranges, increasing the probability of error. The characteristics of many watersheds in west-central Florida are outside the range of those used by Stricker and Sauer (1982) to develop the runoff volume equation; therefore, use of the runoff equation may not produce reliable estimates for the watersheds in west-central Florida.

The Natural Resources Conservation Service TR-20 Model

The TR-20 model calculates peak discharge, runoff volume, and time to peak and outputs a simulated flood hydrograph. Peak discharges were overestimated for 45 storms and underestimated for 21 storms (table 6 and fig. 24). Runoff volumes were overestimated for 44 storms and underestimated for 22 storms, but overestimates of runoff volume did not occur for many of the same storms as overestimates of peak discharge.

The average errors between estimated and observed peak discharges and runoff volumes are smaller for the six urban watersheds than for the 6 natural watersheds included in this study. The average errors for peak discharges and runoff volumes for the six urban watersheds were about 13 and 25 percent greater than observed discharges and runoff volumes, respectively. The average errors for the 6 natural watersheds were about 98 and 76 percent greater than observed discharges and runoff volumes, respectively. The average errors for the 3 watersheds with mixed characteristics were 47 and 50 percent greater than the observed peak discharges and runoff volumes, respectively. The smallest estimation error for peak discharges was 0.7 percent greater than the observed discharge and was calculated for a storm occurring in the Gandy Boulevard watershed. The largest error was 583 percent greater than the observed discharge and was calculated for a storm occurring in the CFI-3 watershed. The smallest and largest runoff volume errors were also in the Gandy Boulevard and CFI-3 Creek watersheds, respectively. The estimation error for runoff volume was 0.41 percent less than the observed runoff volume for a storm occurring in the Gandy Boulevard watershed and 1,020 percent greater than the observed runoff volume for a storm occurring in the CFI-3 Creek watershed (table 6).

The curve number is used by the TR-20 model as a composite index of watershed characteristics, therefore the error between observed and estimated peak discharges and runoff volumes were compared to the watershed curve number rather than to individual watershed characteristics. A weighted average

Table 6. Comparison of peak discharges and runoff volumes estimated using the Natural Resources Conservation Service TR-20 model and observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; E, early; L, late; -, negative values represent underestimations; U, urban; N, natural; M, mixed]

Watershed name	Watershed classification	Peak discharge (cfs)				Runoff volume (in)				Date of storm
		Estimated	Observed	Error cfs	Error Percent	Estimated	Observed	Error Inches	Error Percent	
Arctic Street storm drain	U	109	120	-11	-9.17	1.06	0.76	0.30	39.7	08/03/76
		168	133	35	26.3	1.38	.97	.41	42.3	08/04/76
		81	137	-56	-40.9	.61	.58	.03	5.17	09/26/77
		197	142	55	38.7	1.85	1.30	.55	42.3	05/20/78
Kirby Street drainage ditch	U	38	57	-19	-33.3	.39	.30	.09	30.0	07/19/75
		114	95	19	20.0	1.19	.77	.42	54.6	08/30/75
		27	96	-69	-71.9	.27	.76	-.49	-64.5	08/15/78
St. Louis Street drainage ditch	U	168	357	-189	-52.9	2.10	.92	1.18	128	05/15/76
		89	226	-137	-60.6	1.00	.40	.60	150	06/18/76
		100	326	-226	-69.3	1.16	.45	.71	158	06/29/77
Gandy Boulevard drainage ditch	U	290	223	67	30.0	.67	.50	.17	34.0	06/18/75
		266	301	-35	-11.6	1.06	1.19	-.13	-10.9	07/11/75
		144	207	-63	-30.4	.39	.71	-.32	-45.1	08/07/75
		812	692	120	17.3	2.45	2.46	-.01	-0.41	05/15/76
		413	410	3	0.73	.95	.87	.08	9.20	05/17/76
Allen Creek	U	483	341	142	41.6	.39	.69	-.30	-43.5	07/28/76
		549	379	170	44.8	.52	.60	-.08	-13.3	07/01/77E
		931	819	112	13.7	.98	1.64	-.66	-40.2	07/01/77L
		398	335	63	18.8	.41	.51	-.10	-19.6	07/03/77
		196	89	107	120	.23	.18	.05	27.8	12/02/77
		866	286	580	203	.78	.71	.07	9.86	02/18/78
IMC Creek	N	1	11	-10	-90.9	.02	.66	-.64	-97.0	11/23/88
		30	5	25	500	.71	.17	.54	318	07/12/89
		10	4	6	150	.52	.36	.16	44.4	02/23/90
		14	9	5	55.6	.34	.47	-.13	-27.7	07/21/90
Grace Creek	N	70	59	11	18.6	.80	1.00	.20	20.0	08/07/88
		28	40	-12	-30.0	.28	.72	-.44	-61.1	08/23/88
		34	16	18	113	.34	.23	.11	47.8	07/12/90
		56	25	31	124	.92	.54	.38	70.4	07/14/90
CFI-3 Creek	N	7	19	-12	-63.2	.28	.44	-.16	-36.4	07/05/89
		22	7	15	214	.87	.29	.58	200	02/23/90
		41	6	35	583	1.12	.10	1.02	1020	06/02/90
Walker Creek	M	2058	971	1087	112	13.00	6.89	6.11	88.7	June 92
		380	438	-58	-13.2	1.01	.91	.10	11.0	07/23/92

Table 6. Comparison of peak discharges and runoff volumes estimated using the Natural Resources Conservation Service TR-20 model and observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; E, early; L, late; -, negative values represent underestimations; U, urban; N, natural; M, mixed] (Continued)

Watershed name	Watershed classification	Peak discharge (cfs)				Runoff volume (in)				Date of storm
				Error				Error		
		Estimated	Observed	cfs	Percent	Estimated	Observed	Inches	Percent	
		189	398	-209	-52.5	.49	.96	-.47	-49.0	08/07/92
		330	334	-4	-1.20	.89	.77	.12	15.6	09/04/92
		464	278	186	66.9	1.17	.41	.76	185	09/05/92
		254	199	55	27.6	.74	.56	.18	32.1	09/25/92
Walker Creek (cont.)		301	292	9	3.08	.75	.74	.01	1.35	09/26/92
		231	235	-4	-1.70	.72	.76	-.04	-5.26	01/15/93
		386	319	67	21.0	1.38	.94	.44	46.8	04/01/93
		334	237	97	40.9	.98	.46	.52	113	07/01/93
Clower Creek	U	83	77	6	7.79	1.60	1.45	.15	10.3	02/05/92
		237	205	32	15.6	17.10	17.08	.02	.12	June 92
		59	66	-7	-10.6	1.10	1.07	.03	2.80	09/02/92
		125	110	15	13.6	2.65	2.31	.34	14.7	09/13/92
		59	42	17	40.5	1.30	.67	.63	94.0	09/14/92
		89	60	29	48.3	1.90	1.37	.53	38.7	03/13/92
		161	116	45	38.8	3.90	2.90	1.00	34.5	04/01/92
Catfish Creek	M	127	70	57	81.4	.48	.21	.27	129	01/14/93
		119	76	43	56.6	.58	.25	.33	132	01/15/93
		215	140	75	53.6	.88	.49	.39	79.6	03/13/93
		509	300	209	69.7	2.41	1.41	1.00	70.9	04/01/93
South Creek	N	1964	442	1522	344	13.33	4.30	9.03	210	June 92
		218	143	75	52.4	.27	.13	.14	108	09/06/92
		139	96	43	44.8	.26	.39	-.13	-33.3	09/13/92
		234	94	140	149	.88	.69	.19	27.5	03/13/32
		238	168	70	41.7	.81	1.27	-.46	-36.2	04/01/93
Forked Creek	N	277	287	-10	-3.48	8.16	8.54	-.38	-4.45	June 92
		49	45	4	8.89	.37	.82	-.45	-54.6	08/09/92
Gottfied Creek	M	423	119	304	255	12.40	6.70	5.70	85.1	June 92
		18	21	-3	-14.3	.08	.32	-.24	-75.0	08/11/92
		34	18	16	88.9	.42	.50	-.08	-16.0	October 92
Rock Creek	N	178	109	69	63.3	8.29	5.64	2.65	47.0	June 92
		15	24	-9	-37.5	.01	.24	-.23	-95.8	09/25/92
		12	25	-13	-52.0	.19	.78	-.59	-75.6	October 92

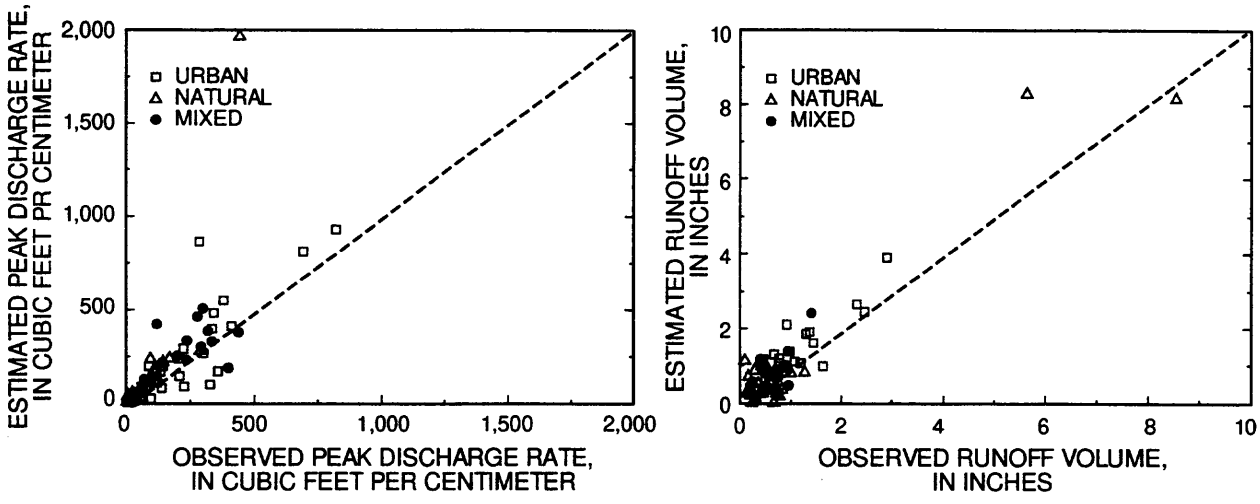


Figure 24. Comparison of the observed and estimated peak discharge rates and runoff volumes calculated using the NRCS TR-20 model.

was used for the curve number where watersheds were modeled with multiple subbasins. These comparisons show the error decreases as the curve number increases. A weaker correlation exists between the errors in estimated runoff volumes and the curve number. Curve numbers are generally lower in the natural watersheds and higher in the urban watersheds (fig. 25).

Typical observed and simulated hydrographs for storms occurring in the Gandy Boulevard and the Clower Creek watersheds are shown in figure 26. Both watersheds are urban, and drain through storm sewer systems. Simulated hydrographs for storms in these watersheds matched the corresponding observed hydrographs more accurately than simulated hydrographs for storms in any of the remaining watersheds included in the study. The predicted peak discharge occurred about 30 minutes after the observed peak for most storms in the Gandy Boulevard watershed, and between 1 and 1.5 hours after the observed peaks in the Clower Creek watershed.

Simulated storm hydrographs for the remaining 4 urban watersheds in Pinellas and western Hillsborough Counties did not accurately match the observed hydrographs (fig. 27). The rising limb of the observed hydrographs for the Arctic Street and Kirby Street watersheds were steep and peaked rapidly. The simulated hydrographs rose slower and peaked from 2 to 3 hours after observed peaks. The observed hydrographs in the St. Louis Street watershed had very steep rising and falling limbs, and peak discharges occurred about 2 hours before the predicted peak. The rising limb of the observed hydrographs for the Allen Creek watershed did not rise as fast or peak as early as the simulated hydrographs. The simulated hydrographs peaked about 1 to 2 hours before the observed peaks. The model consistently overestimated peak discharges in the Allen Creek watershed (table 6).

Observed and simulated hydrographs resulting for 3 different types of storms in the Walker Creek watershed are shown in figure 28. The Walker Creek watershed has a mixture of natural and urban areas and most runoff is through a series of improved open ditches. Simulated and observed hydrographs matched more closely for high intensity, summer thunder storms (Sept 4, 1992) than the hydrograph for a high intensity 4 day storm resulting from a local low pressure system (June 1992) or the hydrographs for winter frontal storms (Jan 15, 1993).

The model overestimated the peak discharge for the 4-day storm (June 1992) by about 100 percent. Two major peaks occurred during this storm (fig. 28). The first simulated peak was predicted to occur about 4 hours after the first observed peak and the second simulated peak was predicted to occur about 4.5 hours before the second observed peak. Runoff volume for this storm was underestimated by 89 percent.

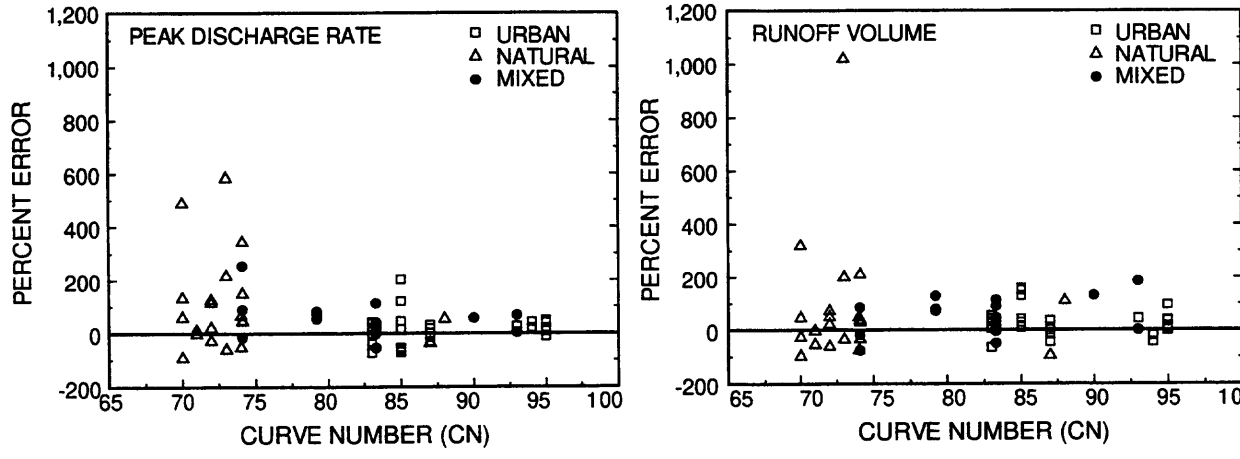


Figure 25. Comparison of the percent error between observed and estimated peak discharge rates and runoff volumes with the average watershed curve number using the NRCS TR-20 model.

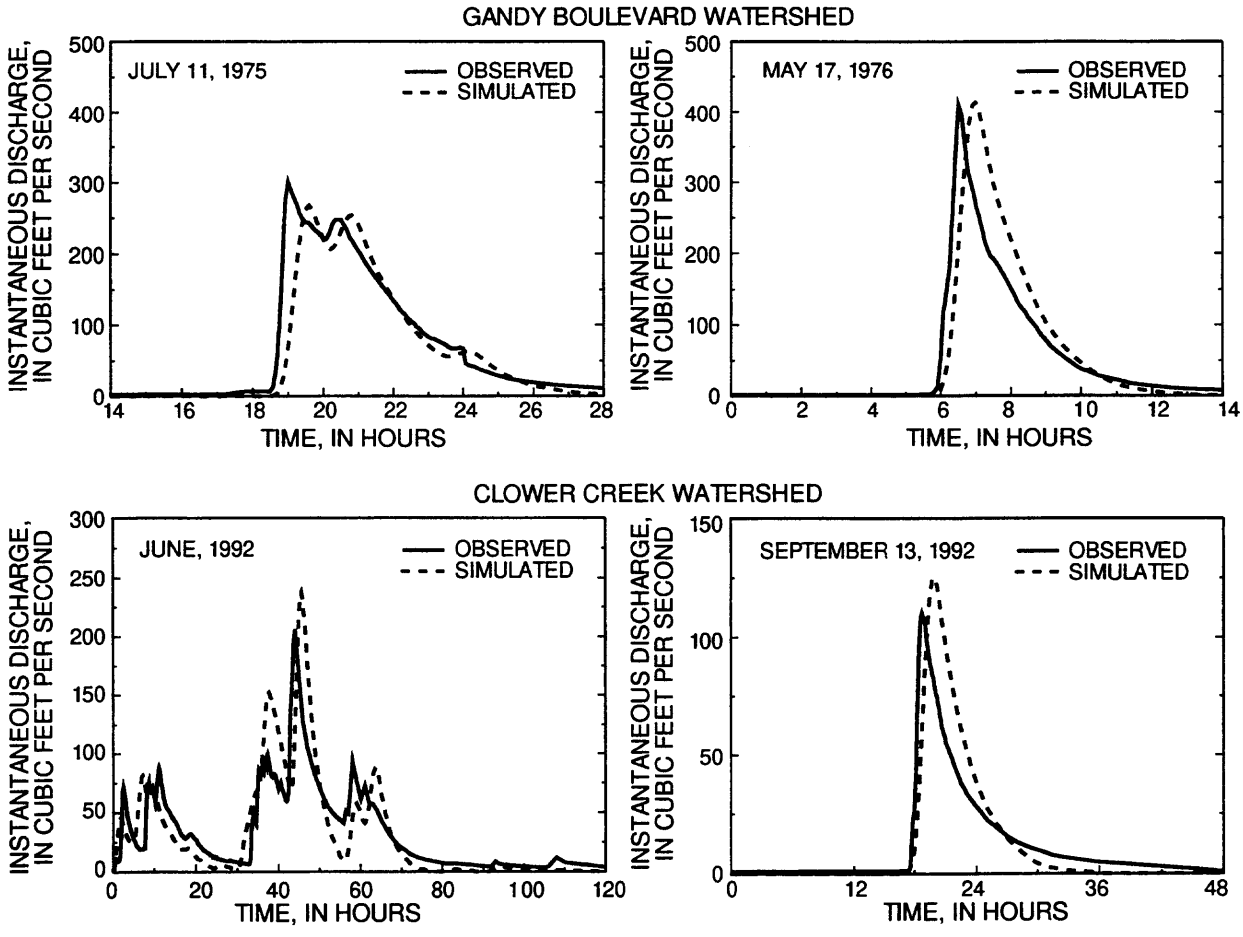


Figure 26. Typical observed hydrographs for storms occurring in the Gandy Boulevard and Clower Creek watersheds and corresponding hydrographs simulated using the NRCS TR-20 model.

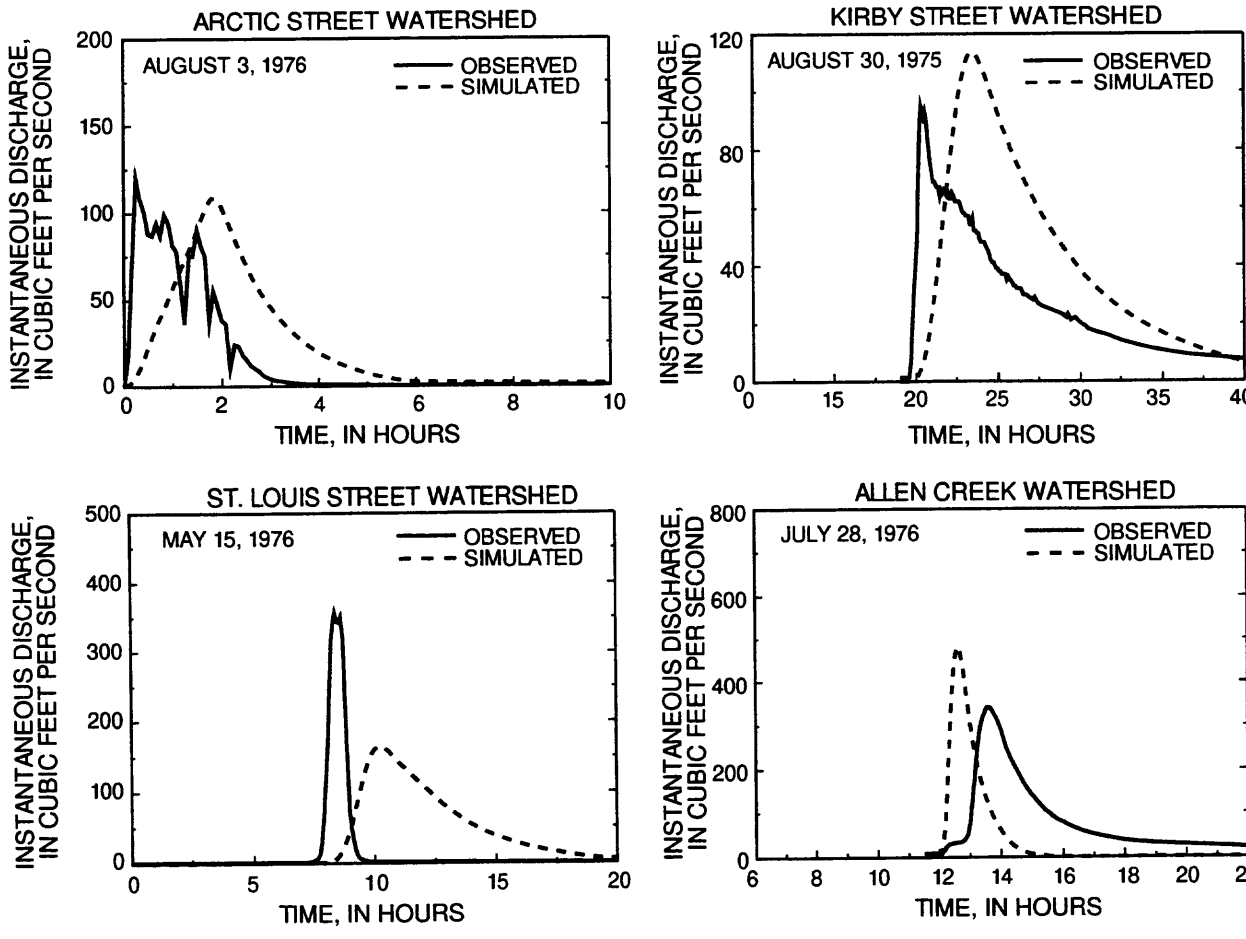


Figure 27. Typical observed hydrographs for storms occurring in the Arctic Street, Kirby Street, St. Louis Street, and Allen Creek watersheds, and corresponding hydrographs simulated using the NRCS TR-20 model.

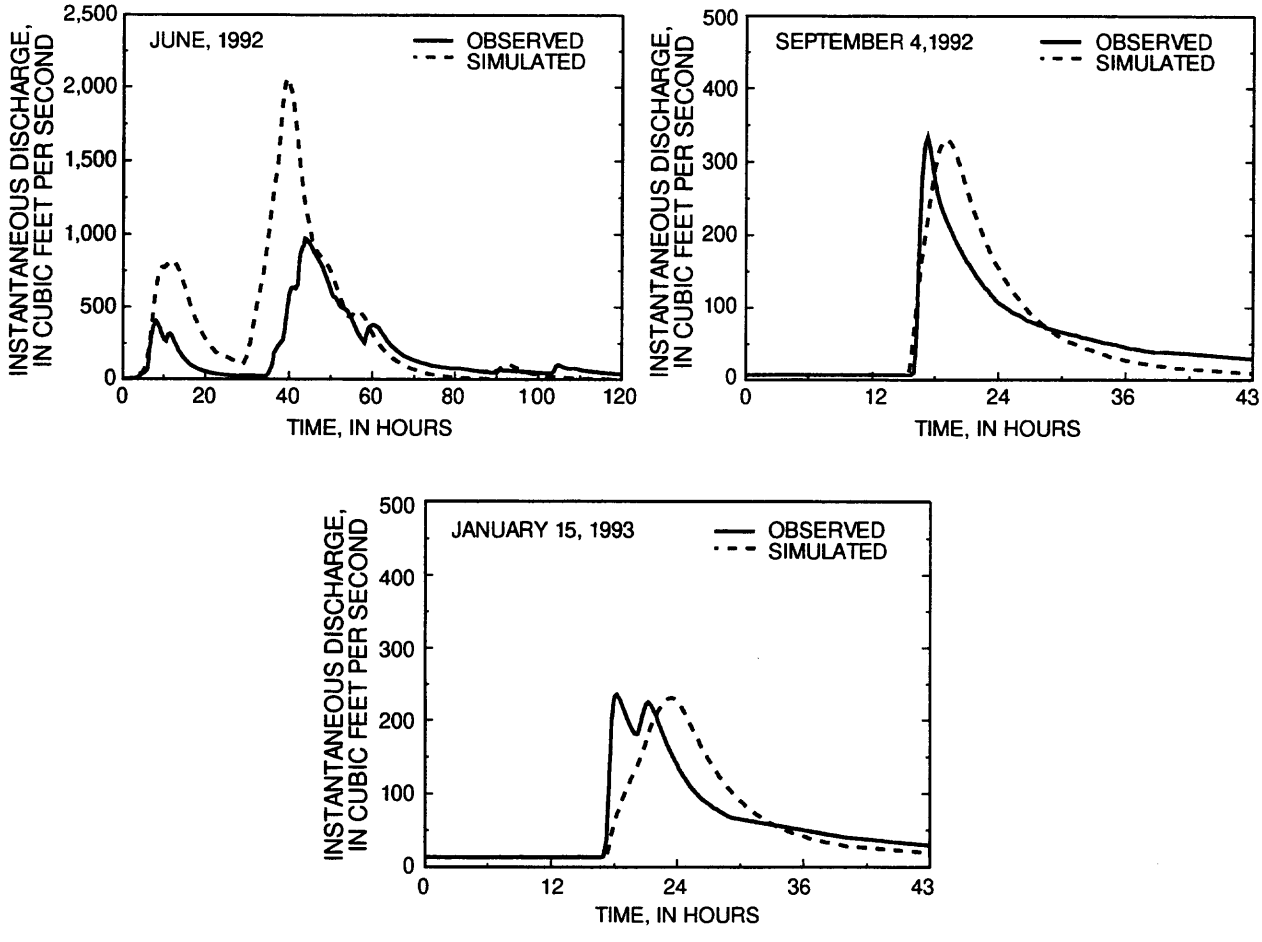


Figure 28. Typical observed hydrographs for storms occurring in the Walker Creek watershed and corresponding hydrographs simulated using the NRCS TR-20 model.

The estimated and observed peak discharge and runoff volume for the January 15, 1993 winter frontal storm, differed only by 1.7 and 5.3 percent (table 6.), respectively. However, the shape of the hydrographs were very different (fig. 28). The observed hydrograph had steeper rising and falling limbs and showed 2 distinct peaks corresponding to periods of heavier rainfall. The first observed peak occurred more than 5 hours before the model predicted the only single peak for the storm. A frontal storm that occurred on April 1, 1993 (not shown) produced a similar hydrograph. The model, when applied to this basin for frontal type storms, did not accurately match the observed storm hydrograph and does not appear to be sensitive to variable rainfall intensity.

The size and shape of the simulated hydrographs for the summer thunder storms closely matched the observed hydrograph (fig. 28). Predicted peak discharges occurred 1.5 to 2.5 hours after the observed peaks for the 7 summer storms modeled.

Typical observed and simulated hydrographs for the Catfish Creek and Gottfried Creek watersheds, also watersheds with mixed land use, are shown on figure 29. The shape of the simulated hydrograph for Catfish Creek is similar to the observed hydrograph, however, the model consistently overestimated the peak discharges and runoff volumes (table 6). Predicted peak discharges occurred between 1 and 2 hours after the observed peaks. There are numerous stormwater management practices in place in this watershed which include control structures and cultivation of aquatic plants in the stream channels. Such management practices slow streamflow and are probably the cause for the consistent overestimation of peak discharge and runoff volume by the model. The observed hydrograph for Gottfried Creek has a long time to peak and a long, relatively flat recession limb. The simulated hydrograph has a much shorter time to peak and a steep recession limb. Predicted peak discharge occurred about 16 hours before the observed peak for the October, 1992 storm. The low stream gradient (1.4 ft/mi), surface detention, subsurface storage and flow, aquatic weed growth, and occasional tidal backwater conditions effect the shape of the observed storm hydrograph.

Typical observed and simulated hydrographs for storms occurring in the IMC, CFI-3, and Grace Creek watersheds, the three inland natural watersheds, are shown in figure 30. The observed hydrographs have lower peaks and longer, flatter recession limbs than the simulated hydrographs, indicating rainfall is being stored in the watershed, then released at a slower rate. Soil is permeable in these watersheds and there are no surface impoundments or wetland areas; therefore, storage in the permeable surficial deposits, subsurface flow, and gradual release of water from the surficial aquifer system to the stream attenuates the storm hydrograph in these watersheds. The TR-20 model can not

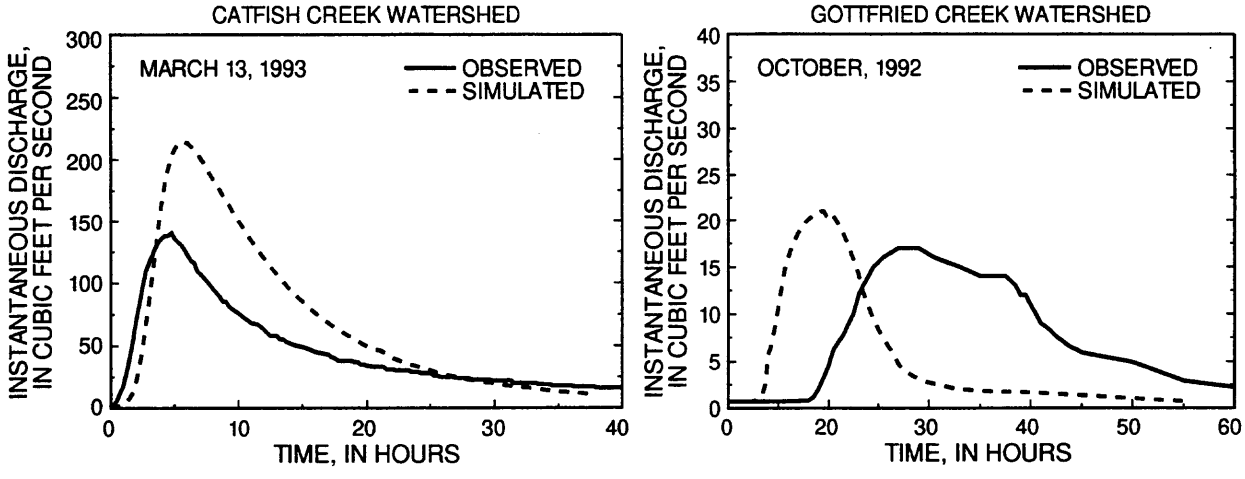


Figure 29. Typical observed hydrographs for storms occurring in the Catfish Creek and Gottfried Creek watersheds and corresponding hydrographs simulated using the NRCS TR-20 model.

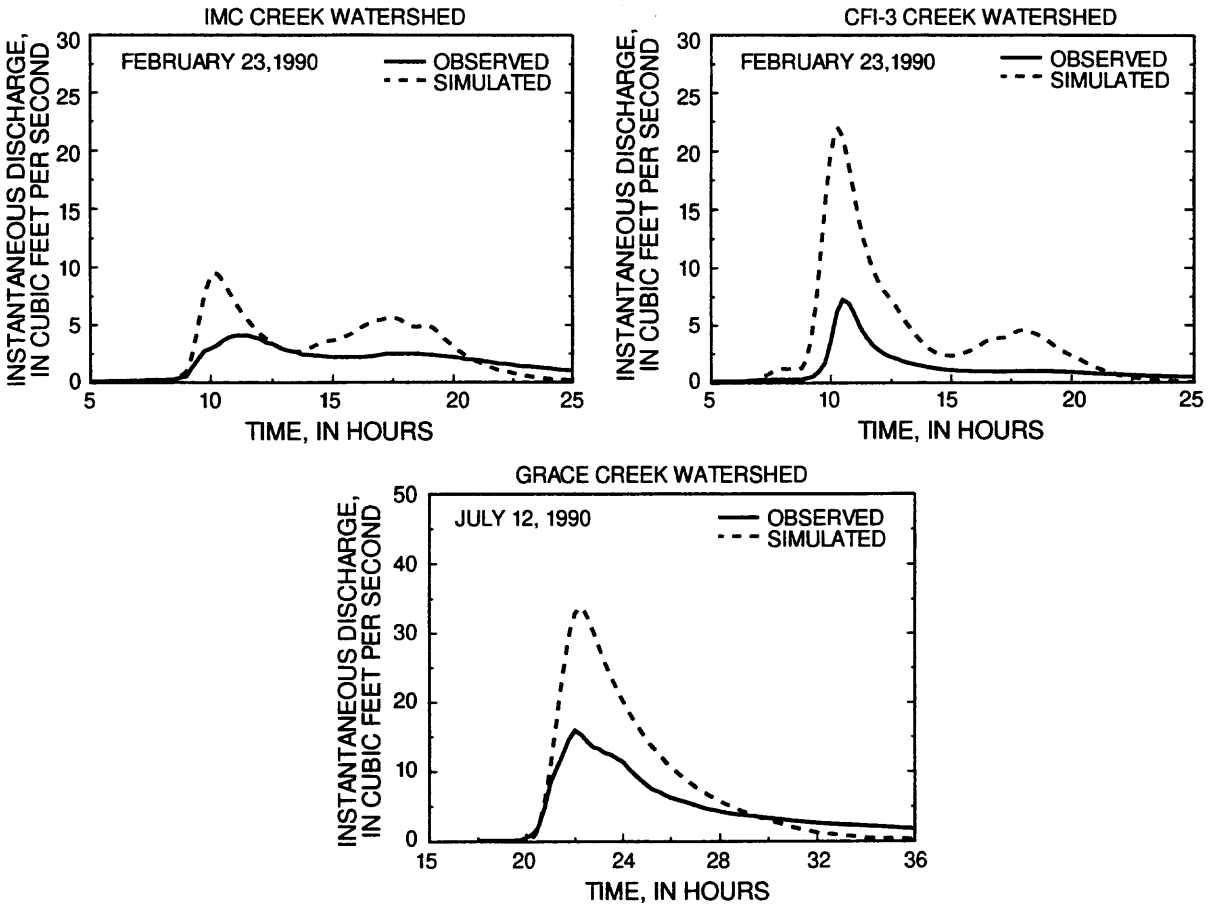


Figure 30. Typical observed hydrographs for storms occurring in the IMC, CFI-3, and Grace Creek watersheds and corresponding hydrographs simulated using the NRCS TR-20 model.

calculate subsurface storage and flow.

Typical observed and simulated hydrographs for the South, Forked and Rock Creek watersheds are shown on figure 31. These watersheds are natural watersheds which are characterized by low slopes, large wetland areas and high water tables. Observed hydrographs in the 3 watersheds are similar in shape, characterized by flat peaks and long recession limbs. Simulated hydrographs do not match the observed hydrographs in either size or shape. Surface detention, subsurface storage and flow, and discharge from the surficial aquifer system to the stream influence the shape of the observed hydrographs in these watersheds.

The Army Corps of Engineers HEC-1 Model

The HEC-1 model calculates a peak discharge, runoff volume, and time to peak, and outputs a simulated flood hydrograph. Peak discharges were overestimated for 55 storms and underestimated 11 storms (table 7 and fig. 32). Runoff volumes were overestimated for 44 storms and underestimated for 22 storms but overestimates of peak discharge did not occur for many of the same storms as overestimates of runoff volumes.

The average errors between estimated and observed peak discharge rates and runoff volumes are smaller for the six urban watersheds than for the six natural watersheds. The average errors for peak discharges and runoff volumes for the urban watersheds of Arctic Street, Kirby Street, St. Louis Street, Gandy Boulevard, Allen Creek and Clower Creek were about 88 and 25 percent greater than observed peak discharge and runoff volumes. The average errors for the six natural watersheds were about 201 and 74 percent greater than observed peak discharges and runoff volumes. The average errors for the three watersheds with mixed characteristics were 98 percent greater than observed peak discharges and 43 percent greater than observed runoff volumes. The smallest estimation error for peak discharges was 2.5 percent greater than the observed peak discharge and was calculated for a storm occurring in the Grace Creek watershed. The largest error was 1,017 percent greater than the observed peak discharge and was calculated for a storm occurring in the CFI-3 watershed. The smallest and largest runoff volume errors were calculated for storms in the Gandy Boulevard and CFI-3 Creek watersheds. The error for runoff volume was 0.41 percent less than the observed runoff volume for a storm occurring in the Gandy Boulevard watershed and 1,020 percent greater than the observed runoff volume for a storm occurring in the CFI-3 Creek watershed (table 7).

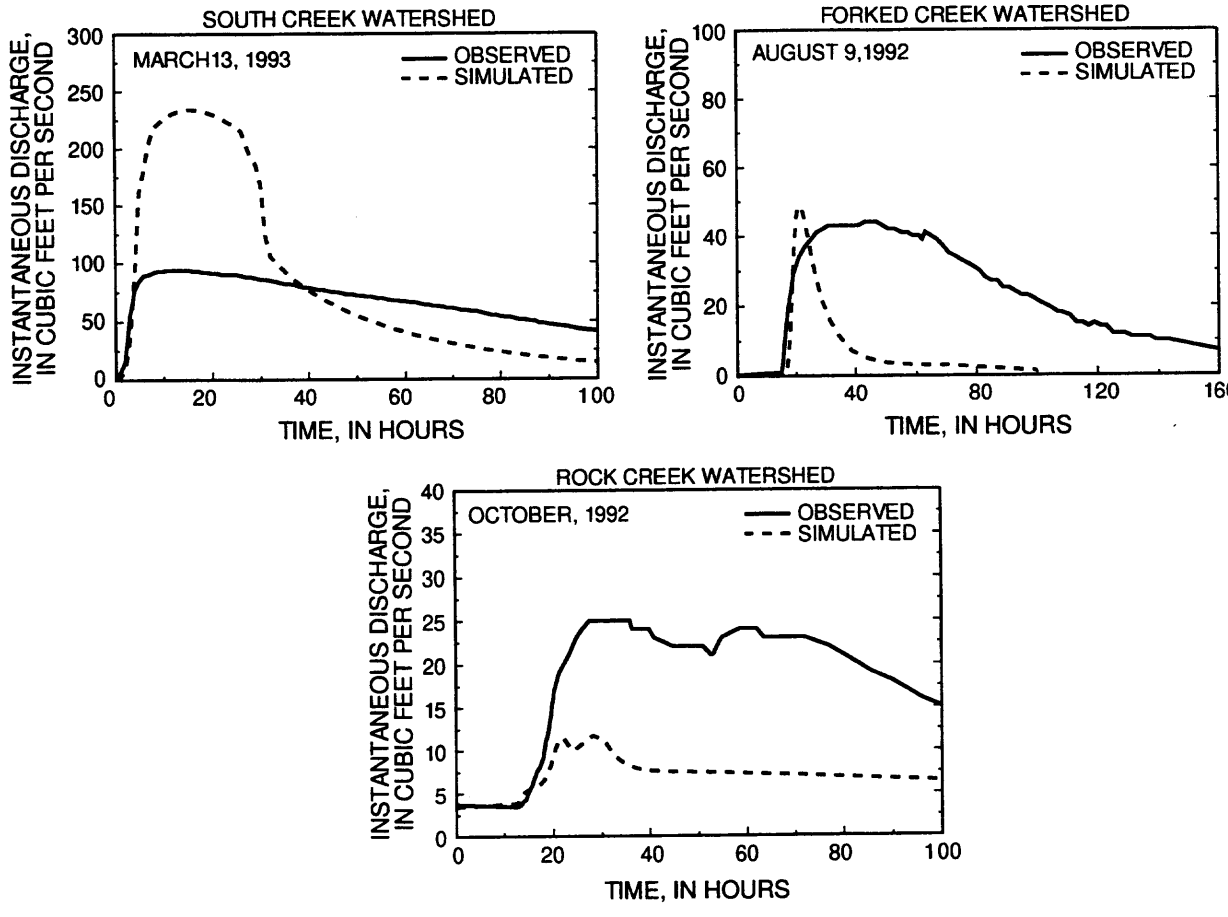


Figure 31. Typical observed hydrographs for storms occurring in the South, Forked, and Rock Creek watersheds and corresponding hydrographs simulated using the NRCS TR-20 model.

Table 7. Comparison of peak discharges and runoff volumes estimated using the U.S. Army Corps of Engineers HEC-1 model and observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; E, early; L, late; -, negative values represent underestimations; U, urban; N, natural; M, mixed]

Watershed name	Watershed classification	Peak discharge (cfs)				Runoff volume (in)				Date of storm
				Error				Error		
		Estimated	Observed	cfs	Percent	Estimated	Observed	Inches	Percent	
Arctic Street storm drain	U	154	120	34	28.3	1.06	0.76	0.30	39.5	08/03/76
		301	133	168	126	1.40	.97	.43	44.3	08/04/76
		147	137	10	7.30	.61	.58	.03	5.17	09/26/77
		339	142	197	139	1.86	1.30	.56	43.1	05/20/78
Kirby Street drainage ditch	U	77	57	20	35.1	.40	.30	.10	33.3	07/19/75
		218	95	123	129	1.19	.77	.42	54.5	08/30/75
		61	96	-35	-36.5	.27	.76	-.49	-64.5	08/15/78
St. Louis Street drainage ditch	U	303	357	-54	-15.1	2.10	.92	1.18	128	05/15/76
		167	226	-59	-26.1	1.00	.40	.60	150	06/18/76
		184	326	-142	-43.6	1.16	.45	.71	158	06/29/77
Gandy Boulevard drainage ditch	U	523	223	300	135	.67	.50	.17	34.0	06/18/75
		474	301	173	57.5	1.06	1.19	-.13	-10.9	07/11/75
		267	207	60	29.0	.39	.71	-.32	-45.1	08/07/75
		1407	692	715	103	2.45	2.46	-.01	-0.41	05/15/76
		737	410	327	79.8	.95	.87	.08	9.20	05/17/76
Allen Creek	U	764	341	423	124	.39	.69	-.30	-43.5	07/28/76
		897	379	518	137	.52	.60	.08	-13.3	07/01/77E
		1433	819	614	75.0	.99	1.64	-.65	-39.6	07/01/77L
		677	335	342	102	.41	.51	-.10	-19.6	07/03/77
		267	89	178	200	.23	.18	.05	27.8	12/02/77
		1280	286	994	348	.78	.71	.07	9.86	02/18/78
IMC Creek	N	2	11	-9	-81.8	.05	.66	-.61	-92.4	11/23/88
		52	5	47	940	.71	.17	.54	318	07/12/89
		16	4	12	300	.49	.36	.13	36.1	02/23/90
		24	9	15	167	.34	.47	-.13	-27.7	07/21/90
Grace Creek	N	101	59	42	71.2	.80	1.00	.20	20.0	08/07/88
		41	40	1	2.50	.28	.72	-.44	-61.1	08/23/88
		48	16	32	200	.34	.23	.11	47.8	07/12/90
		71	25	46	184	.92	.54	.38	70.4	07/14/90
CFI-3 Creek	N	10	19	-9	-47.4	.28	.44	-.16	-36.4	07/05/89
		34	7	27	386	.87	.29	.58	200	02/23/90
		67	6	61	1017	1.12	.10	1.02	1020	06/02/90
Walker Creek	M	2463	971	1492	154	12.80	6.89	5.91	85.8	June 92
		553	438	115	26.3	1.01	.91	.10	11.0	07/23/92

Table 7. Comparison of peak discharges and runoff volumes estimated using the U.S. Army Corps of Engineers HEC-1 model and observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; E, early; L, late; -, negative values represent underestimations; U, urban; N, natural; M, mixed] (Continued)

Watershed name	Watershed classification	Peak discharge (cfs)				Runoff volume (in)				Date of storm
				Error				Error		
		Estimated	Observed	cfs	Percent	Estimated	Observed	Inches	Percent	
		250	398	-148	-37.2	.49	.96	-.47	-49.0	08/07/92
		446	334	112	33.5	.89	.77	.12	15.6	09/04/92
		600	278	322	116	1.17	.41	.76	185	09/05/92
		336	199	137	68.8	.74	.56	.18	32.1	09/25/92
Walker Creek (cont.)		408	292	116	39.7	.75	.74	.01	1.35	09/26/92
		285	235	50	21.3	.72	.76	-.04	-5.26	01/15/93
		508	319	189	59.2	1.38	.94	.44	46.8	04/01/93
		459	237	222	93.7	.98	.46	.52	113	07/01/93
Clower Creek	U	136	77	59	76.6	1.63	1.45	.18	12.4	02/05/92
		309	205	104	50.7	17.19	17.08	.11	.64	June 92
		108	66	42	63.6	1.14	1.07	.07	6.54	09/02/92
		224	110	114	103	2.55	2.31	.24	10.4	09/13/92
		104	42	62	148	1.33	.67	.66	98.5	09/14/92
		155	60	95	158	1.96	1.37	.59	43.1	03/13/92
		268	116	152	131	3.90	2.90	1.00	34.5	04/01/92
Catfish Creek	M	200	70	130	186	.46	.21	.25	119	01/14/93
		197	76	121	159	.54	.25	.29	116	01/15/93
		354	140	214	153	.86	.49	.37	75.5	03/13/93
		837	300	537	179	2.34	1.41	.93	66.0	04/01/93
South Creek	N	1710	442	1268	287	12.75	4.30	8.45	197	June 92
		138	143	-5	-3.50	.04	.13	-.09	-69.2	09/06/92
		142	96	46	47.9	.23	.39	-.16	-41.0	09/13/92
		245	94	151	161	.93	.69	.24	34.8	03/13/32
		244	168	76	45.2	.84	1.27	-.43	-33.9	04/01/93
Forked Creek	N	1365	287	1078	376	13.19	8.54	4.65	54.4	June 92
		56	45	11	24.4	.26	.82	-.56	-68.3	08/09/92
Gottfied Creek	M	485	119	366	308	12.30	6. 70	5.60	83.6	June 92
		19	21	-2	-9.52	.08	.32	-.24	-75.0	08/11/92
		40	18	22	122	.42	.50	-.08	-16.0	October 92
Rock Creek	N	300	109	191	175	10.62	5.64	4.98	88.3	June 92
		22	24	-2	-8.33	.14	.24	-.10	-41.7	09/25/92
		18	25	-7	-28.0	.33	.78	-.45	-57.7	October 92

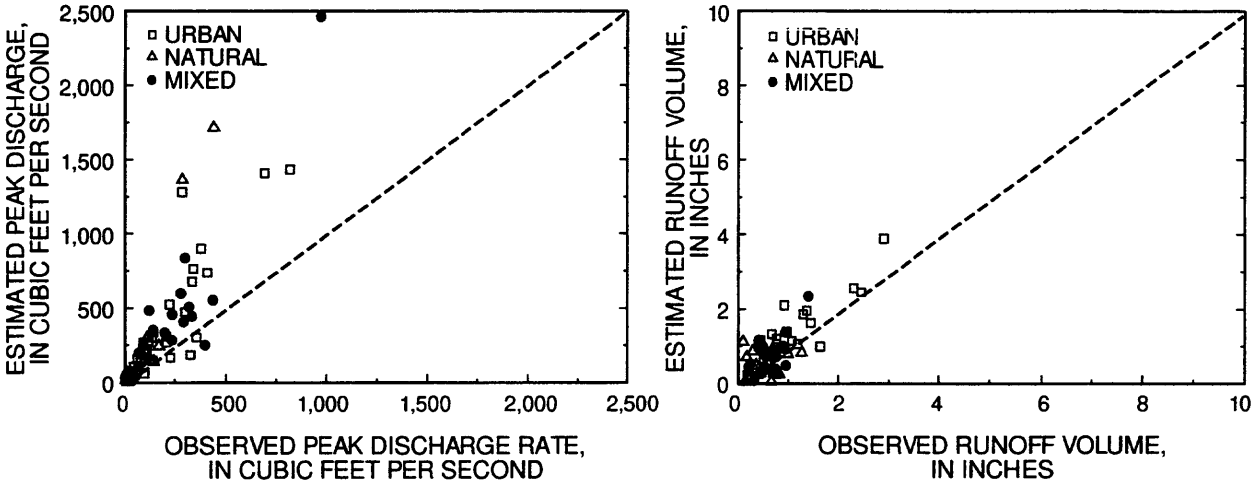


Figure 32. Comparison of the observed and estimated peak discharge rates and runoff volumes calculated using the U.S. Army Corps of Engineers HEC-1 model.

Errors between estimated and observed peak discharges and runoff volumes were compared to the average watershed curve number rather than to individual watershed characteristics because the NRCS curve number method was used to calculate runoff in this model. Comparisons indicate decreasing error in peak discharges and runoff volumes as the curve number increases (fig. 33).

Typical observed and simulated hydrographs for storms occurring in the Gandy Boulevard, and Clower Creek watersheds are shown on figure 34. Both watersheds are urban and drain through storm sewers. The model overestimated peak discharges for all storms in these watersheds. The initial rising limb of the simulated hydrographs matches the observed hydrographs in these watersheds, but the peak was overpredicted and the recession limbs are shorter than the recession limbs of the observed hydrographs. Predicted peak discharges occur about an hour after the observed peak discharges.

Simulated storm hydrographs for the remaining 4 urban watersheds did not match observed hydrographs (fig. 35). Observed hydrographs for the Arctic Street and Kirby Street watersheds respond quickly and peaked rapidly. Whereas the simulated hydrographs rose slower, overpredicted the peak, and peaked about 2 to 3 hours after the observed peak. The model consistently underestimated peak discharges and overestimated runoff volumes in the St. Louis Street watershed. The observed hydrographs had steep rising and falling limbs and short duration times indicating little or no surface- or ground-water storage occurs in this watershed; whereas the simulated hydrographs did not rise or fall as steeply and the duration time was about 3 times as long. Simulated peak discharges occurred about 2 hours after the observed peak. In the Allen Creek watershed, the model consistently overestimated peak discharges and predicted the peak to occur between 1 to 2 hours before the observed peaks.

Observed and simulated hydrographs for 3 different storm types that occurred in the Walker Creek watershed are shown in figure 36. The Walker Creek watershed has a mixture of natural and urban areas. The storms included a high intensity 4-day storm resulting from a local low pressure system (June, 1992), winter frontal storms (Jan 15, 1993), and high intensity, short duration summer thunderstorms (Sept 4, 1992).

The model overestimated the peak discharge and runoff volume for the 4-day storm by about 154 percent and 86 percent, respectively (table 7). Simulated peaks occurred about 3 hours after the observed peaks; however, the general shape of the simulated hydrograph matches the observed hydrograph, except the peaks are overpredicted. Runoff volume was overestimated by about 86 percent.

The estimated and observed peak discharge and runoff volume for the January 15, 1993 storm, a winter frontal storm, differed by 21.3 and 5.3 percent (table 7), respectively. The observed hydrograph

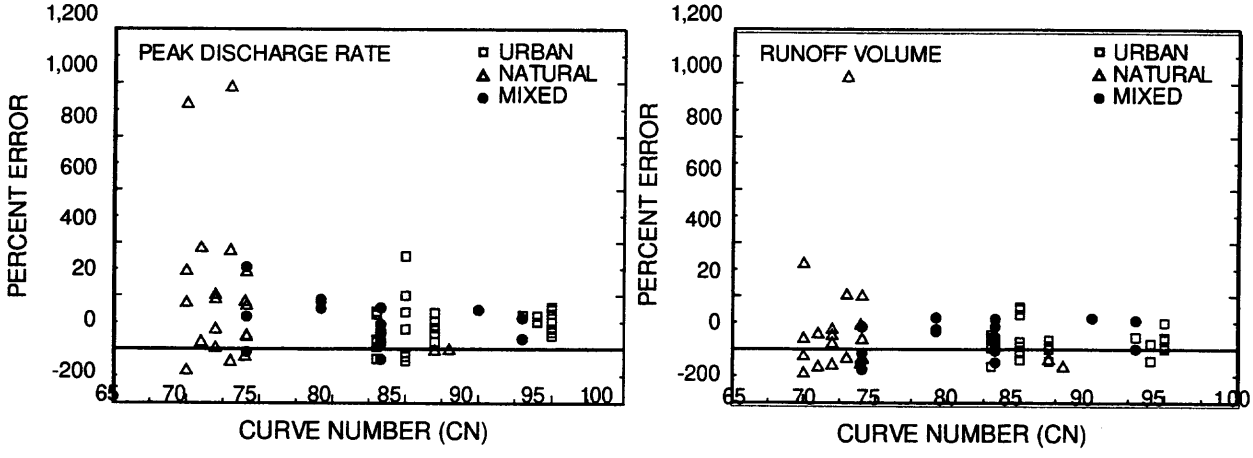


Figure 33. Comparison of the percent error between observed and estimated peak discharge rates and runoff volumes with the average curve number using the U.S. Army Corps of Engineers HEC-1 model.

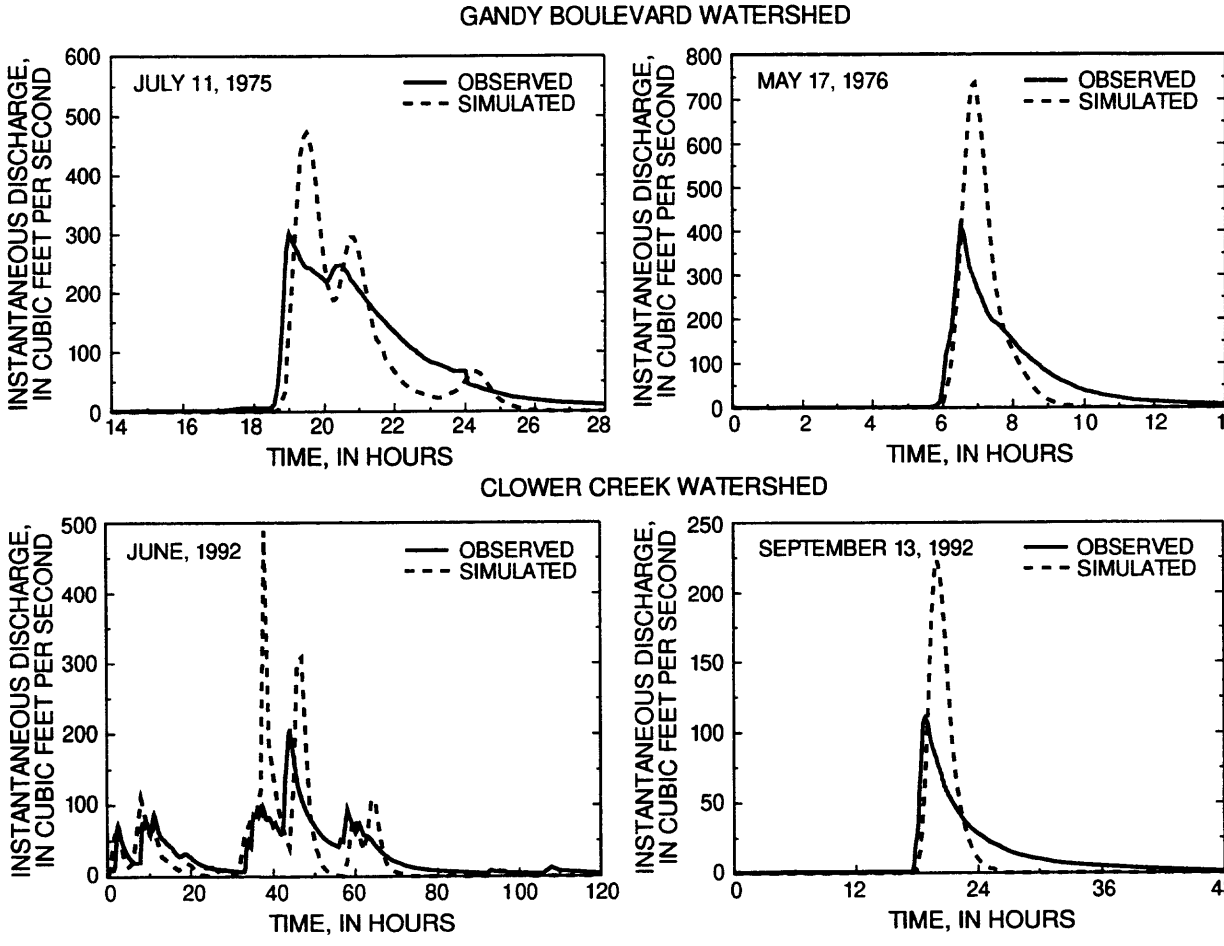


Figure 34. Typical observed hydrographs for storms occurring in the Gandy Boulevard and Clower Creek watersheds and corresponding hydrographs simulated using the U.S. Army Corps of Engineers HEC-1 model.

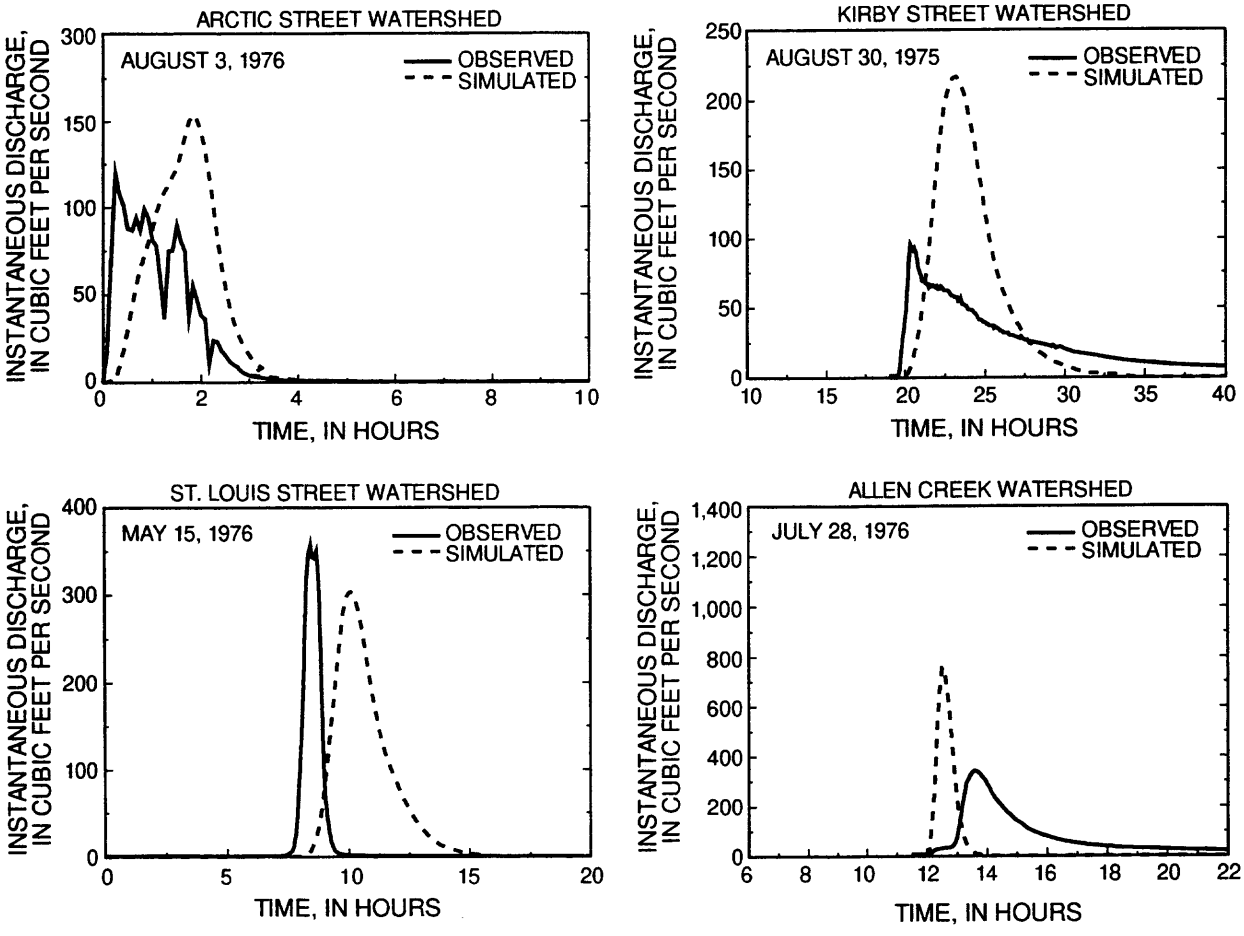


Figure 35. Typical observed hydrographs for storms occurring in the Arctic Street, Kirby Street, St. Louis Street, and Allen Creek watersheds and corresponding hydrographs simulated using the U.S. Army Corps of Engineers HEC-1 model.

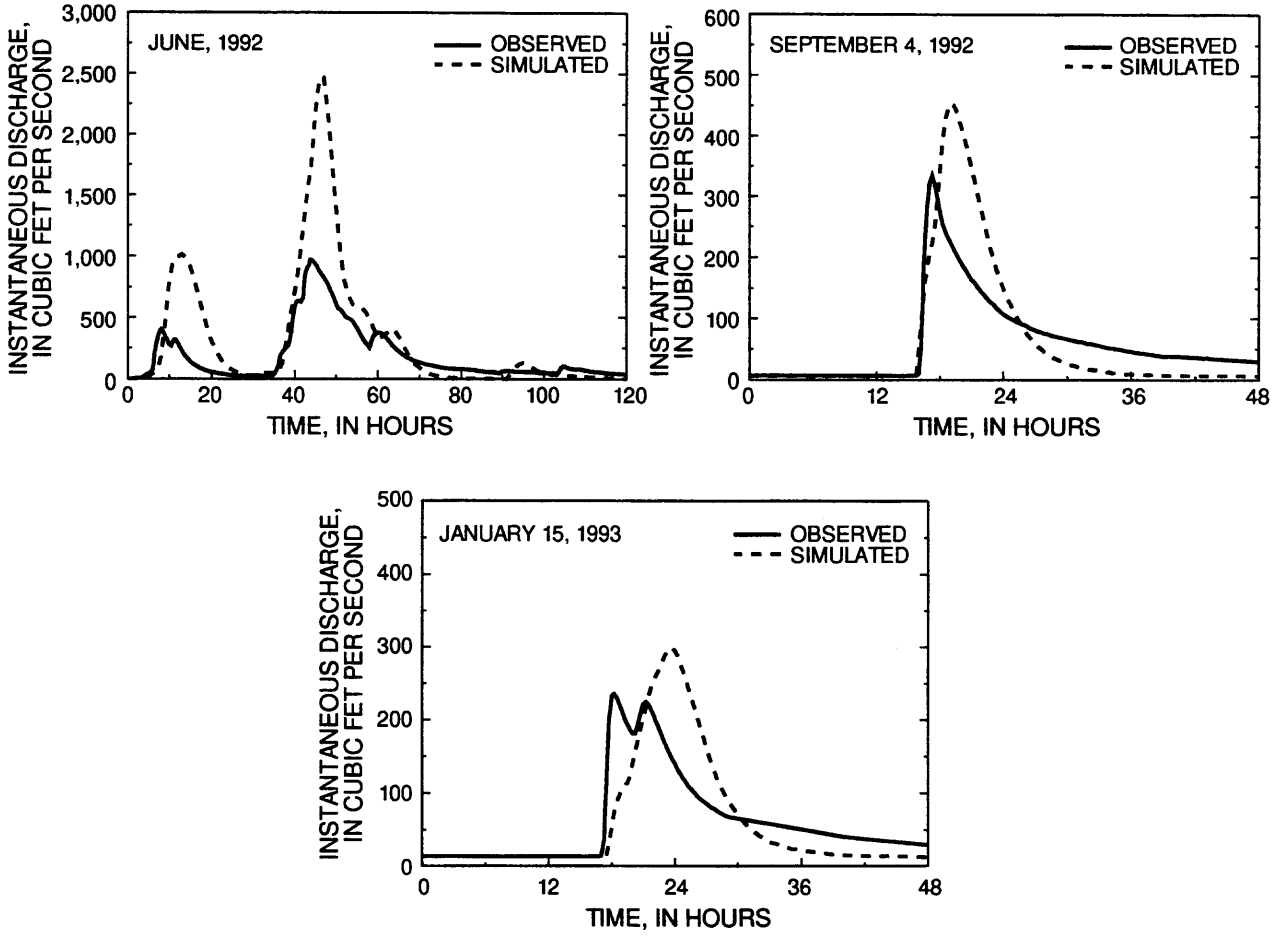


Figure 36. Typical observed hydrographs for storms occurring in the Walker Creek watershed and corresponding hydrographs simulated using the U.S. Army Corps of Engineers HEC-1 model.

had steeper rising and falling limbs and showed 2 distinct peaks corresponding to periods of heavier rainfall. The first observed peak occurred more than 5 hours before the model predicted a peak; the model also only predicted a single peak for the storm. A frontal storm that occurred on April 1, 1993 (not shown) produced a similar hydrograph. After the initial rapid decline, the later part of the recession limb of the observed hydrograph became flatter and remained higher than the simulated hydrograph, indicating a slower release of groundwater to the stream. The model, when applied to this watershed for frontal type storms, did not accurately match the observed storm hydrograph and does not appear to be sensitive to variable rainfall intensity.

The observed hydrographs for summer thunderstorms occurring in the Walker creek watershed had slightly steeper rising limbs than the simulated hydrographs. The recession limbs initially fell rapidly, than flattened and remained higher than the simulated hydrographs, indicating a ground water contribution to the stream. Peak discharges and runoff volumes were overestimated by about 34 and 16 percent, respectively and the predicted peaks occurred about 1.5 hours after observed peaks.

Typical observed and simulated hydrographs for the Catfish Creek and Gottfried Creek watersheds, also watersheds with mixed land use, are shown on figure 37. The model consistently overestimated the peak discharges and runoff volumes for the Catfish Creek watershed (table 7). The falling limb of the simulated hydrograph is steeper than the observed hydrograph and predicted peak discharges to occur between 1 to 2 hours after the observed peaks. There are numerous stormwater management practices in place in this watershed which include control structures and cultivation of aquatic plants in the stream channels. Such management practices slow streamflow and are probably the cause for the consistent overestimation of peak discharge and runoff volume by the model. The observed hydrograph for Gottfried Creek has a long time to peak and a long relatively flat recession limb. The simulated hydrograph peaks sooner, has a steep recession limb and has a peak discharge that is about twice the observed discharge. Low stream gradient, surface detention, subsurface storage and flow, aquatic weed growth and periodic tidal backwater conditions effect the shape of the storm hydrographs. The model predicted the peak to occur about 16 hours before the observed peak.

Typical observed and simulated hydrographs for storms occurring in the IMC, CFI-3, and Grace Creek watersheds, the three inland natural watersheds, are shown in figure 38. Simulated hydrographs for the these watersheds poorly match the observed hydrographs. The observed hydrographs have lower peaks and longer, flatter recession limbs, which indicates rainfall is being stored in the watershed, then released at a slower rate. Soil is permeable in these watersheds and there are no surface impoundments

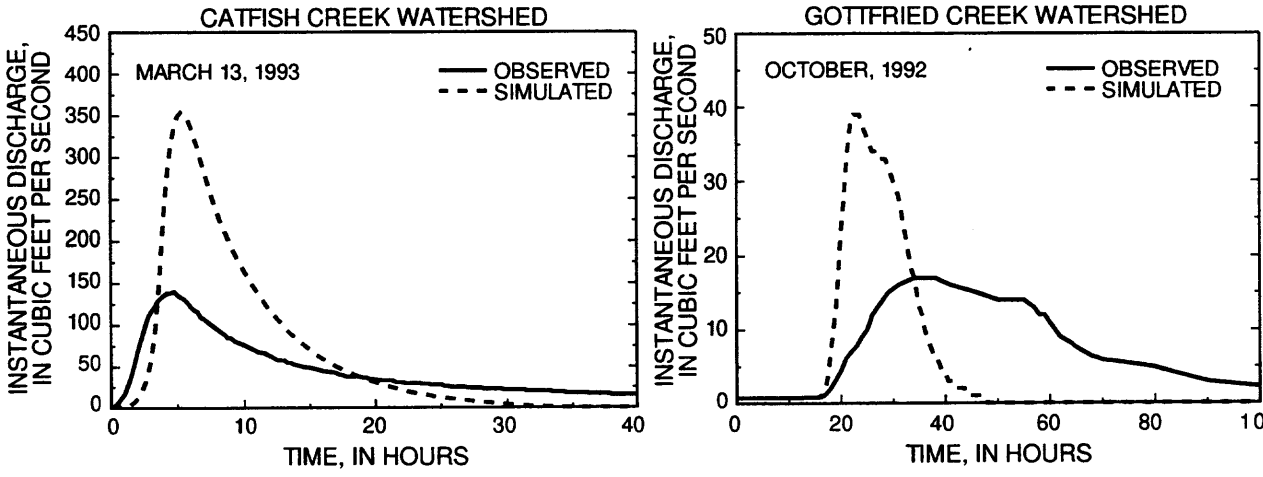


Figure 37. Typical observed hydrographs for storms occurring in the Catfish Creek and Gottfried Creek watershed and corresponding hydrographs simulated using the U.S. Army Corps of Engineers HEC-1 model.

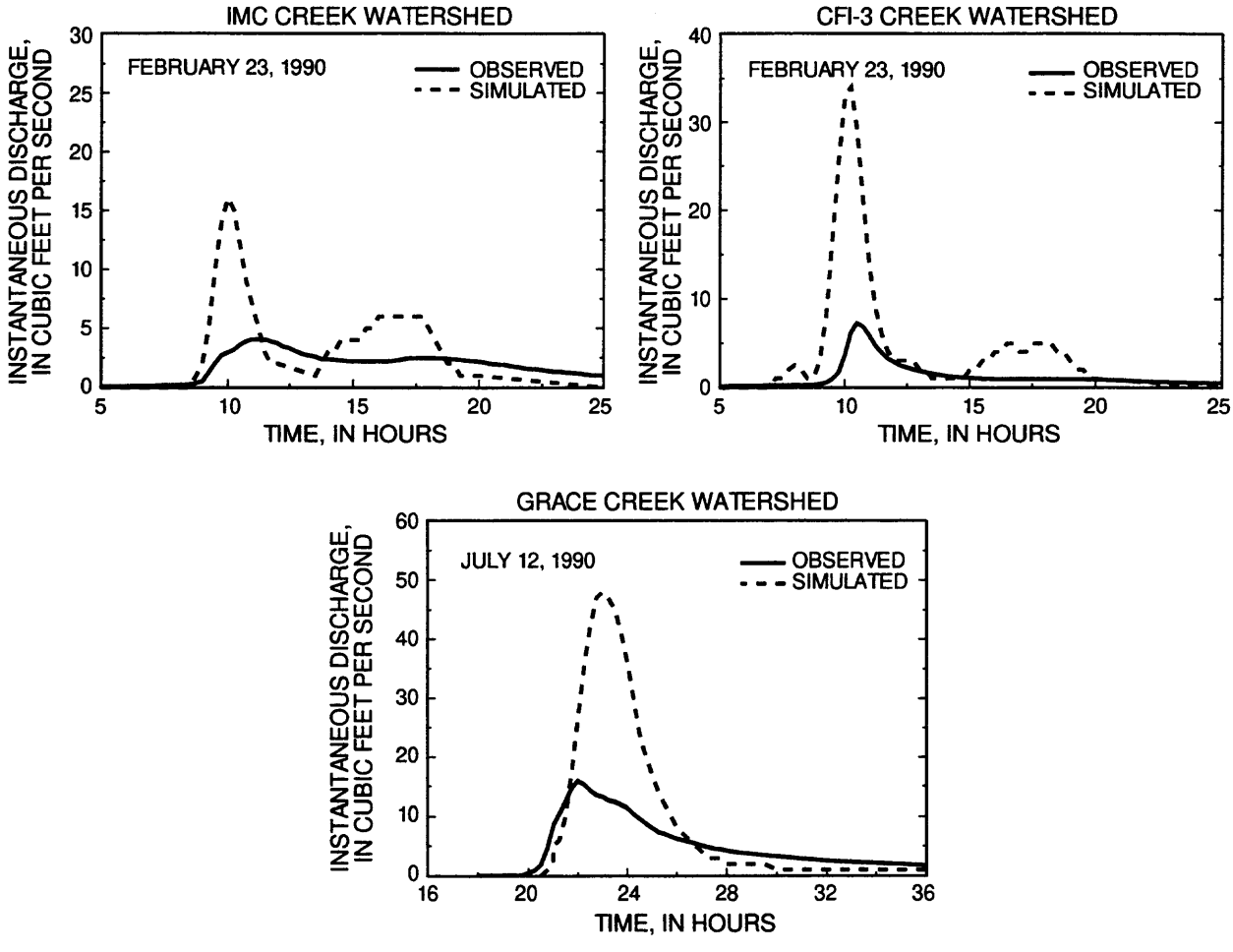


Figure 38. Typical observed hydrographs for storms occurring in the IMC, CFI-3, and Grace Creek watersheds and corresponding hydrographs simulated using the U.S. Army Corps of Engineers HEC-1 model.

or wetland areas. Attenuation of storm hydrographs is caused by storage in the permeable surficial deposits, subsurface flow, and gradual release of water from the surficial aquifer system to the stream.

Typical observed and simulated hydrographs for the South, Forked and Rock Creek watersheds are shown on figure 39. These watersheds are natural watersheds which are characterized by low slopes, large wetland areas and high water tables. Observed hydrographs in these 3 watersheds are similar in shape. Simulated hydrographs do not match the observed hydrographs in either size or shape. Peak discharges are overestimated in the South and Forked Creek watersheds and underestimated in the Rock Creek watershed. Simulated peak discharges occur before observed peak discharges. Surface detention, subsurface storage and flow, and discharge from the surficial aquifer system to the stream influence the shape of the observed hydrographs in these watersheds. The HEC-1 model does not calculate subsurface storage and flow.

The Environmental Protection Agency Storm Water Management Model

Uncalibrated model runs were used to simulate storm events. Selected input parameters for these model runs are listed in appendix II and III. Comparisons of estimated and observed peak discharges and runoff volumes, calculated with both the Green-Ampt and Horton infiltration methods (tables 8 and 9, figs. 40 and 41) show the similar results. Peak discharges for most storms were overestimated. Estimates of runoff volumes appear to better match measured runoff volumes. The model overestimated the peak discharge for 44 storms, underestimated the peak for 20 storms, and was the same for 2 storms, using the Green-Ampt method. When the Horton infiltration method was used, the model overestimated the peak discharge for 44 storms and underestimated the peak for 22 storms. Runoff volumes were overestimated for 22 storms, underestimated for 42 storms, and was the same for 2 storms, using the Green-Ampt infiltration method. When the Horton infiltration method was used, runoff volumes were overestimated for 25 storms, underestimated for 39 storms and was the same for 2 storms. Choice of the Green-Ampt or the Horton infiltration method did not greatly affect the estimates of peak discharges and runoff volumes, although the peaks and volumes calculated with the Horton method were slightly higher than those calculated with the Green-Ampt method.

When using the Green-Ampt infiltration method, the average error for the urban watersheds was 19 percent greater than observed peak discharges and 28 percent less than observed runoff volumes. The average error for the natural watersheds was about 105 percent greater than observed peak

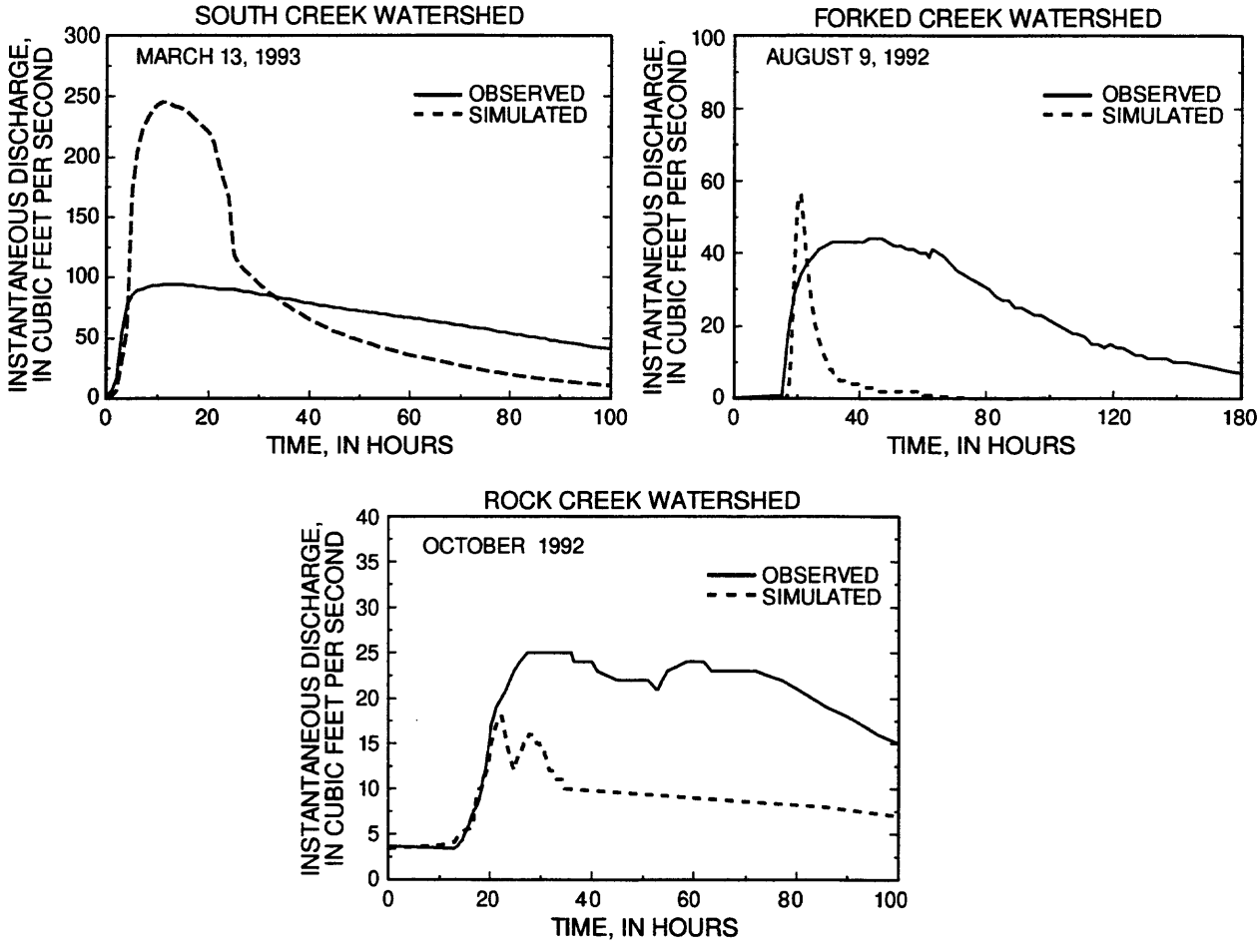


Figure 39. Typical observed hydrographs for storms occurring in the South, Forked, and Rock Creek watersheds and corresponding hydrographs simulated using the U.S. Army Corps of Engineers HEC-1 model.

Table 8. Comparison of peak discharges and runoff volumes estimated using the Environmental Protection Agency Storm Water Management model with the Green-Ampt infiltration method, and observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; E, early; L late; -, negative values represent underestimations; U, urban; N, natural; M, mixed]

Watershed name	Watershed classification	Peak discharge (cfs)				Runoff volume (in)				Date of storm
		Estimated	Observed	Error cfs	Error Percent	Estimated	Observed	Error Inches	Error Percent	
Arctic Street storm drain	U	121	120	1	0.83	1.01	0.76	0.25	32.9	08/03/76
		174	133	41	30.8	.84	.97	-.13	-13.4	08/04/76
		182	137	45	32.8	.73	.58	.15	25.9	09/26/77
		241	142	99	69.7	1.49	1.30	.19	14.6	05/20/78
Kirby Street drainage ditch	U	92	57	35	61.4	.15	.30	-.15	-50.0	07/19/75
		139	95	44	46.3	.33	.77	-.44	-57.1	08/30/75
		100	96	4	4.17	.13	.76	-.63	-82.9	08/15/78
St. Louis Street drainage ditch	U	128	357	-229	-64.1	.96	.92	.04	4.35	05/15/76
		117	226	-109	-48.2	.39	.40	-.01	-2.50	06/18/76
		76	326	-250	-76.7	.45	.45	0	0.00	06/29/77
Gandy Boulevard drainage ditch	U	251	223	28	12.6	.34	.50	-.16	-32.0	06/18/75
		285	301	-16	-5.32	.45	1.19	-.74	-62.2	07/11/75
		203	207	-4	-1.93	.25	.71	-.46	-64.8	08/07/75
		537	692	-155	-22.4	.98	2.46	-1.48	-60.2	05/15/76
		354	410	-56	-13.7	.43	.87	-.44	-50.6	05/17/76
Allen Creek	U	171	341	-170	-49.9	.16	.69	-.53	-76.8	07/28/76
		362	379	-17	-4.49	.32	.60	-.28	-46.7	07/01/77E
		475	819	-344	-42.0	.52	1.64	-1.12	-68.3	07/01/77L
		156	335	-179	-53.4	.17	.51	-.34	-66.7	07/03/77
		125	89	36	40.4	.21	.18	.03	16.7	12/02/77
		374	286	88	30.8	.40	.71	-.31	-43.7	02/18/78
IMC Creek	N	2	11	-9	-81.8	.03	.66	-.63	-95.5	11/23/88
		29	5	24	480	.39	.17	.22	129	07/12/89
		4	4	0	0.00	.06	.36	-.30	-83.3	02/23/90
		13	9	4	44.4	.13	.47	-.34	-72.3	07/21/90
Grace Creek	N	111	59	52	88.1	.55	1.00	-.45	-45.0	08/07/88
		40	40	0	.0	.16	.72	-.56	-77.8	08/23/88
		51	16	35	219	.14	.23	-.09	-39.1	07/12/90
		31	25	6	24.0	.42	.54	-.12	-22.2	07/14/90
CFI-3 Creek	N	0.1	19	-18.9	-99.5	.01	.44	-.43	-97.7	07/05/89
		10	7	3	42.9	.12	.29	-.17	-58.6	02/23/90
		52	6	46	767	.86	.10	.76	760	06/02/90
Walker Creek	M	1650	971	679	69.9	9.57	6.89	2.68	38.9	June 92
		1270	438	832	190	.90	.91	-.01	-1.10	07/23/92

Table 8. Comparison of peak discharges and runoff volumes estimated using the Environmental Protection Agency Storm Water Management model with the Green-Ampt infiltration method, and observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; E, early; L late; -, negative values represent underestimations; U, urban; N, natural; M, mixed] (Continued)

Watershed name	Watershed classification	Peak discharge (cfs)				Runoff volume (in)				Date of storm
				Error				Error		
		Estimated	Observed	cfs	Percent	Estimated	Observed	Inches	Percent	
		649	398	251	63.1	.59	.96	-.37	-38.5	08/07/92
		970	334	636	190	.80	.77	.03	3.90	09/04/92
		783	278	505	182	.63	.41	.22	53.7	09/05/92
		533	199	334	168	.65	.56	.09	16.1	09/25/92
Walker Creek (cont.)		496	292	204	69.9	.47	.74	-.27	-36.5	09/26/92
		420	235	185	78.7	.70	.76	-.06	-7.89	01/15/93
		729	319	410	129	.94	.94	0	0.00	04/01/93
		664	237	427	180	.84	.46	.38	82.6	07/01/93
Clower Creek	U	99	77	22	28.6	.94	1.45	-.51	-35.2	02/05/92
		463	205	258	126	15.30	17.08	-1.78	-10.4	June 92
		89	66	23	34.8	.68	1.07	-.39	-36.4	09/02/92
		227	110	117	106	1.84	2.31	-.47	-20.3	09/13/92
		75	42	33	78.6	.75	.67	.08	11.9	09/14/92
		105	60	45	75.0	1.26	1.37	-.11	-8.03	03/13/92
		294	116	178	153	3.15	2.90	.25	8.62	04/01/92
Catfish Creek	M	79	70	9	12.9	.18	.21	-.03	-14.3	01/14/93
		55	76	-21	-27.6	.13	.25	-.12	-48.0	01/15/93
		137	140	-3	-2.14	.34	.49	-.15	-30.6	03/13/93
		300	300	0	0.00	1.03	1.41	-.38	-27.0	04/01/93
South Creek	N	651	442	209	47.3	9.29	4.30	4.99	116	June 92
		154	143	11	7.69	.16	.13	.03	23.1	09/06/92
		132	96	36	37.5	.30	.39	-.09	-23.1	09/13/92
		207	94	113	120	.76	.69	.07	10.1	03/13/32
		193	168	25	14.9	.68	1.27	-.59	-46.5	04/01/93
Forked Creek	N	263	287	-24	-8.36	8.84	8.54	.34	3.51	June 92
		39	45	-6	-13.3	1.29	.82	.47	57.3	08/09/92
Gottfied Creek	M	244	119	125	105	11.70	6.70	5.00	74.6	June 92
		23	21	2	9.50	.15	.32	-.17	-53.1	08/11/92
		17	18	-1	-5.60	.29	.50	-.21	-42.0	October 92
Rock Creek	N	272	109	163	150	12.50	5.64	6.86	122	June 92
		17	24	-7	-29.2	.10	.24	-.14	-58.3	09/25/92
		12	25	-13	-52.0	.37	.78	-.41	-52.6	October 92

Table 9. Comparison of peak discharges and runoff volumes estimated using the Environmental Protection Agency Storm Water Management model with the Horton infiltration method, and observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; E, early; L, late; -, negative values represent underestimations; U, urban; N, natural; M, mixed]

Watershed name	Watershed classification	Peak discharge (cfs)				Runoff volume (in)				Date of storm
				Error				Error		
		Estimated	Observed	cfs	Percent	Estimated	Observed	Inches	Percent	
Arctic Street storm drain	U	124	120	4	3.33	1.06	0.76	0.30	39.5	08/03/76
		174	133	41	30.8	.87	.97	-.10	-10.3	08/04/76
		182	137	45	32.8	.74	.58	.16	27.6	09/26/77
		244	142	102	71.8	1.61	1.30	.31	23.8	05/20/78
Kirby Street drainage ditch	U	92	57	35	61.4	.17	.30	-.13	-43.3	07/19/75
		142	95	47	49.5	.42	.77	-.35	-45.5	08/30/75
		99	96	3	3.13	.14	.76	-.62	-81.6	08/15/78
St. Louis Street drainage ditch	U	144	357	-213	-59.7	1.59	.92	.67	72.8	05/15/76
		125	226	-101	-44.7	.76	.40	.36	90.0	06/18/76
		86	326	-240	-73.6	.88	.45	.43	95.6	06/29/77
Gandy Boulevard drainage ditch	U	256	223	33	14.8	.41	.50	-.09	-18.0	06/18/75
		289	301	-12	-3.99	.57	1.19	-.62	-52.1	07/11/75
		208	207	1	.48	.28	.71	-.43	-60.6	08/07/75
		544	692	-148	-21.4	1.22	2.46	-1.24	-50.4	05/15/76
		361	410	-49	-12.0	.55	.87	-.32	-36.8	05/17/76
Allen Creek	U	176	341	-165	-48.4	.18	.69	-.51	-73.9	07/28/76
		370	379	-9	-2.37	.39	.60	-.21	-35.0	07/01/77E
		488	819	-331	-40.4	.69	1.64	-.95	-57.9	07/01/77L
		161	335	-174	-51.9	.18	.51	-.33	-64.7	07/03/77
		127	89	38	42.7	.21	.18	.03	16.7	12/02/77
		374	286	88	30.8	.44	.71	-.27	-38.0	02/18/78
IMC Creek	N	3	11	-8	-72.7	.05	.66	-.61	-92.4	11/23/88
		33	5	28	560	.60	.17	.43	253	07/12/89
		3	4	-1	-25.0	.06	.36	-.30	-83.3	02/23/90
		16	9	7	77.8	.23	.47	-.24	-51.1	07/21/90
Grace Creek	N	125	59	66	112	.67	1.00	-.33	-33.0	08/07/88
		42	40	2	5.00	.57	.72	-.15	-20.8	08/23/88
		62	16	46	288	.23	.23	.00	0.00	07/12/90
		29	25	4	16.0	.41	.54	-.13	-24.1	07/14/90
CFI-3 Creek	N	10	19	-9	-47.4	.05	.44	-.39	-88.6	07/05/89
		5	7	-2	-28.6	.05	.29	-.24	-82.8	02/23/90
		57	6	51	850	1.16	.10	1.06	1060	06/02/90
Walker Creek	M	1650	971	679	69.9	9.60	6.89	2.71	39.3	June 92
		1270	438	832	190	.92	.91	.01	1.10	07/23/92

Table 9. Comparison of peak discharges and runoff volumes estimated using the Environmental Protection Agency Storm Water Management model with the Horton infiltration method, and observed peak discharges and runoff volumes

[cfs, cubic feet per second; in, inches; E, early; L, late; -, negative values represent underestimations; U, urban; N, natural; M, mixed] (Continued)

Watershed name	Watershed classification	Peak discharge (cfs)				Runoff volume (in)				Date of storm
		Estimated	Observed	Error cfs	Error Percent	Estimated	Observed	Error Inches	Error Percent	
		671	398	273	68.6	.62	.96	-.34	-35.4	08/07/92
		967	334	633	190	.81	.77	.04	5.19	09/04/92
		816	278	538	193	.68	.41	.27	65.9	09/05/92
		540	199	341	171	.65	.56	.09	16.1	09/25/92
Walker Creek (cont.)		513	292	221	75.7	.48	.74	-.26	-35.1	09/26/92
		421	235	186	79.1	.70	.76	-.06	-7.89	01/15/93
		746	319	427	134	.98	.94	.04	4.26	04/01/93
		674	237	437	184	.85	.46	.40	84.8	07/01/93
Clower Creek	U	96	77	19	24.7	.91	1.45	-.54	-37.2	02/05/92
		463	205	258	126	15.30	17.08	-1.78	-10.4	June 92
		87	66	21	31.8	.67	1.07	-.40	-37.4	09/02/92
		223	110	113	103	1.82	2.31	-.49	-21.2	09/13/92
		74	42	32	76.1	.74	.67	.07	10.4	09/14/92
		102	60	42	70.0	1.24	1.37	-.13	-9.49	03/13/92
		290	116	174	150	3.13	2.90	.23	7.93	04/01/92
Catfish Creek	M	75	70	5	7.14	.21	.21	.00	0.00	01/14/93
		56	76	-20	-26.3	.15	.25	-.10	-40.0	01/15/93
		132	140	-8	-5.71	.41	.49	-.08	-16.3	03/13/93
		316	300	16	5.33	1.25	1.41	-.16	-11.3	04/01/93
South Creek	N	662	442	220	49.8	9.24	4.30	4.94	115	June 92
		148	143	5	3.50	.15	.13	.02	15.4	09/06/92
		113	96	17	17.7	.27	.39	-.12	-30.8	09/13/92
		199	94	105	112	.75	.69	.06	8.70	03/13/32
		196	168	28	16.7	.70	1.27	-.57	-44.9	04/01/93
Forked Creek	N	264	287	-23	-8.01	8.88	8.54	.34	3.98	June 92
		36	45	-9	-20.0	1.41	.82	.59	72.0	08/09/92
Gottfied Creek	M	244	119	125	105	11.80	6.70	5.10	76.1	June 92
		19	21	-2	-9.50	.14	.32	-.18	-56.3	08/11/92
		17	18	-1	-5.60	.30	.50	-.20	-40.0	October 92
Rock Creek	N	269	109	160	147	12.60	5.64	6.96	123	June 92
		17	24	-7	-29.2	.10	.24	-.14	-58.3	09/25/92
		12	25	-13	-52.0	.37	.78	-.41	-52.6	October 92

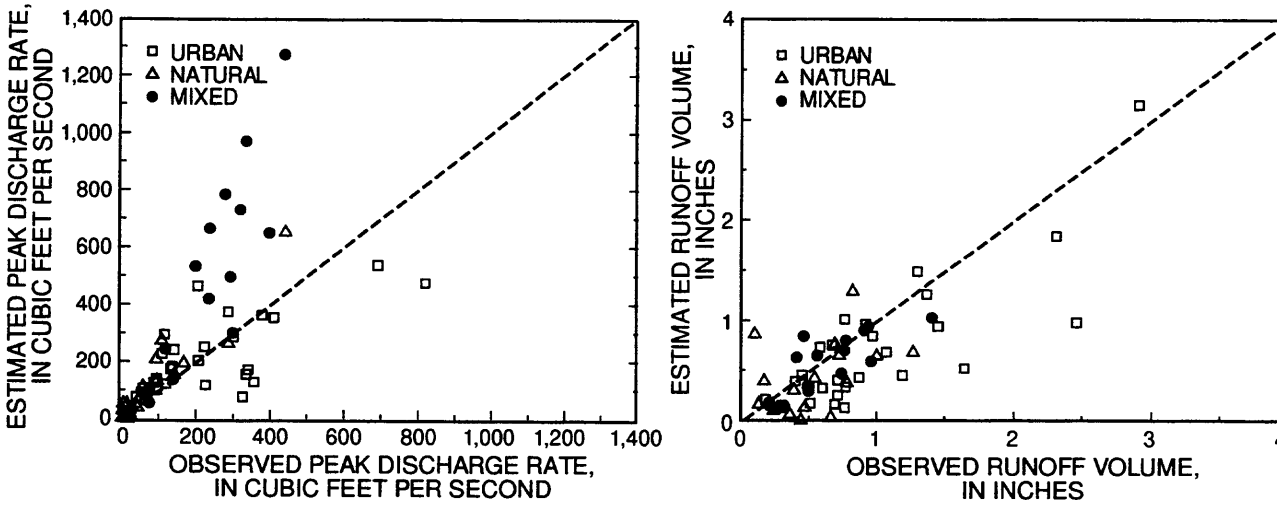


Figure 40. Comparison of observed and estimated peak discharge rates and runoff volumes for the Environmental Protection Agency Storm Water Management model, using the Green-Ampt infiltration method.

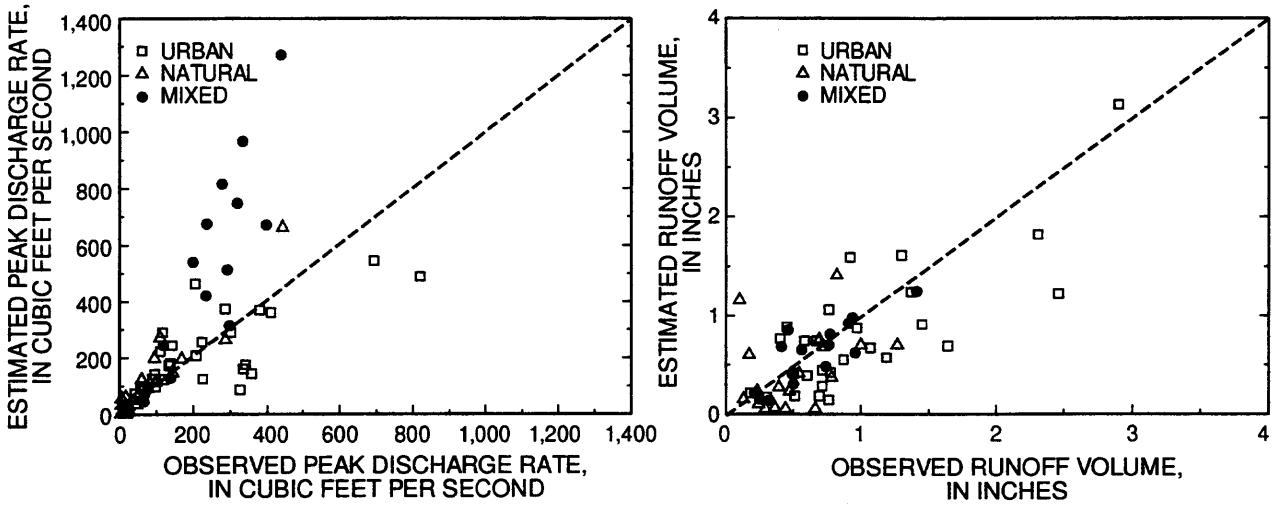


Figure 41. Comparison of observed and estimated peak discharge rates and runoff volumes for the Environmental Protection Agency Storm Water Management model, using the Horton infiltration method.

discharges and 26 percent greater than observed runoff volumes. The average error for the three mixed land use watersheds was 67 percent greater than observed discharges and 4 percent less than observed runoff volumes. When using the Horton infiltration method, the average error for the urban watersheds was 26 percent greater than observed peak discharges and 14 percent greater than observed runoff volumes. The average error for the natural watersheds was about 137 percent greater than observed peak discharges and 59 percent greater than observed runoff volumes. The average error for the three mixed land use watersheds was 67 percent greater than observed peak discharges and 9 percent greater than observed runoff volumes.

Figures 42 and 43 show the percent error between estimated and observed peak discharges, as a function of the percent impervious area, watershed slope, and watershed size calculated using the Green-Ampt and Horton infiltration methods, respectively. Very little correlation between these basin characteristics and the percent error in peak discharges is evident. The percent error between estimated and observed runoff volumes, as a function of the percent impervious area, watershed slope and watershed size calculated using the Green-Ampt and Horton infiltration methods are shown in figures 44 and 45, respectively. The range in errors generally decreased with increasing impervious area. No direct correlation between the percent error and the watershed slope or size is apparent.

Observed and simulated hydrographs typical of storms occurring in the Gandy Boulevard and Clower Creek watersheds are show in figure 46. The watersheds are urban and drain through underground storm sewers. Simulated hydrographs for the Gandy Boulevard watershed closely matched the observed peak; however, runoff volume was underestimated. In the Clower Creek watershed, the simulated peaks exceeded the observed peaks by more than 100 percent. The time of the peaks and the runoff volumes was similar.

The observed and simulated hydrograph for the Arctic Street watershed, also an urban watershed, had similar shapes (fig. 47). The first peak on the simulated hydrograph matched the observed peak. Subsequent peaks were greater than the observed peaks; however, the timing of each peak was simulated accurately.

Simulated storm hydrographs for the remaining 3 urban watersheds in Pinellas and western Hillsborough Counties did not accurately match the observed hydrographs (fig. 47). Peak discharge on the simulated hydrograph for the Kirby Street watershed was 49.5 percent greater than on the observed hydrograph (table 9), although the time to peak was predicted accurately. The falling limb of the simulated hydrograph was much steeper than that of the observed hydrograph. The rising limb of the

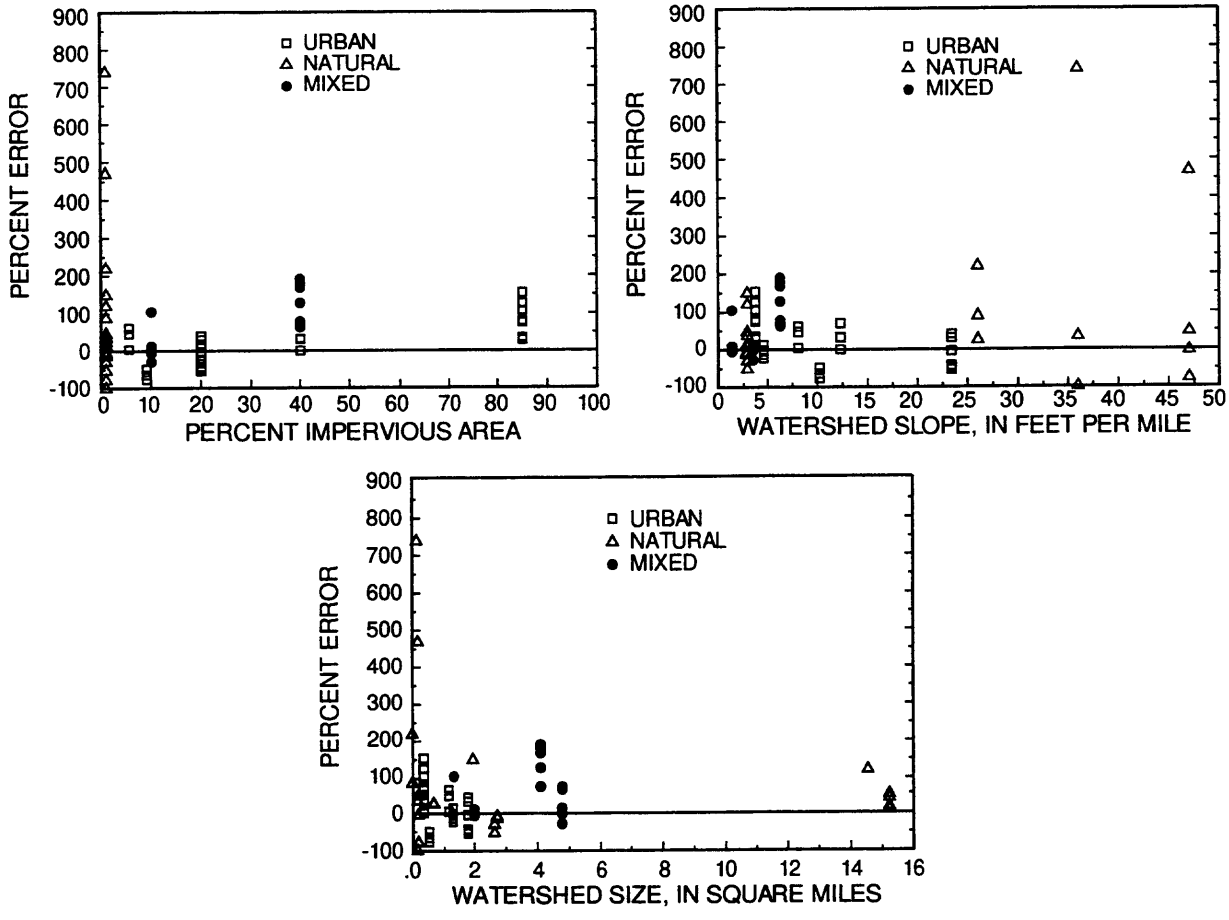


Figure 42. Comparison of the percent error between estimated and observed peak discharge rates with percent impervious area, average watershed slope, and watershed size, for the Environmental Protection Agency StormWater Management model, using Green-Ampt infiltration method.

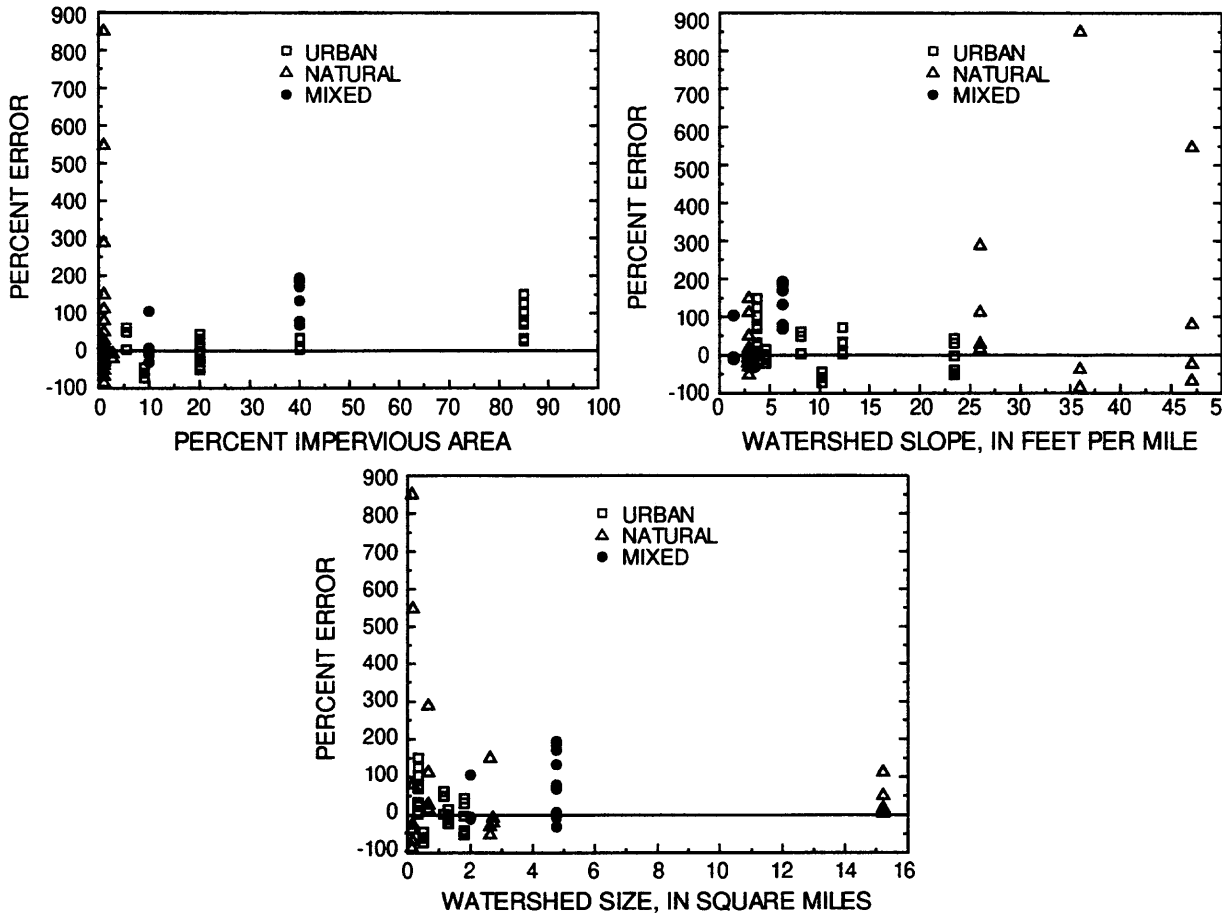


Figure 43. Comparison of the percent error between estimated and observed peak discharge rates with impervious area, average watershed slope, and watershed size, for the Environmental Protection Agency Storm Water Management model, using the Horton infiltration method.

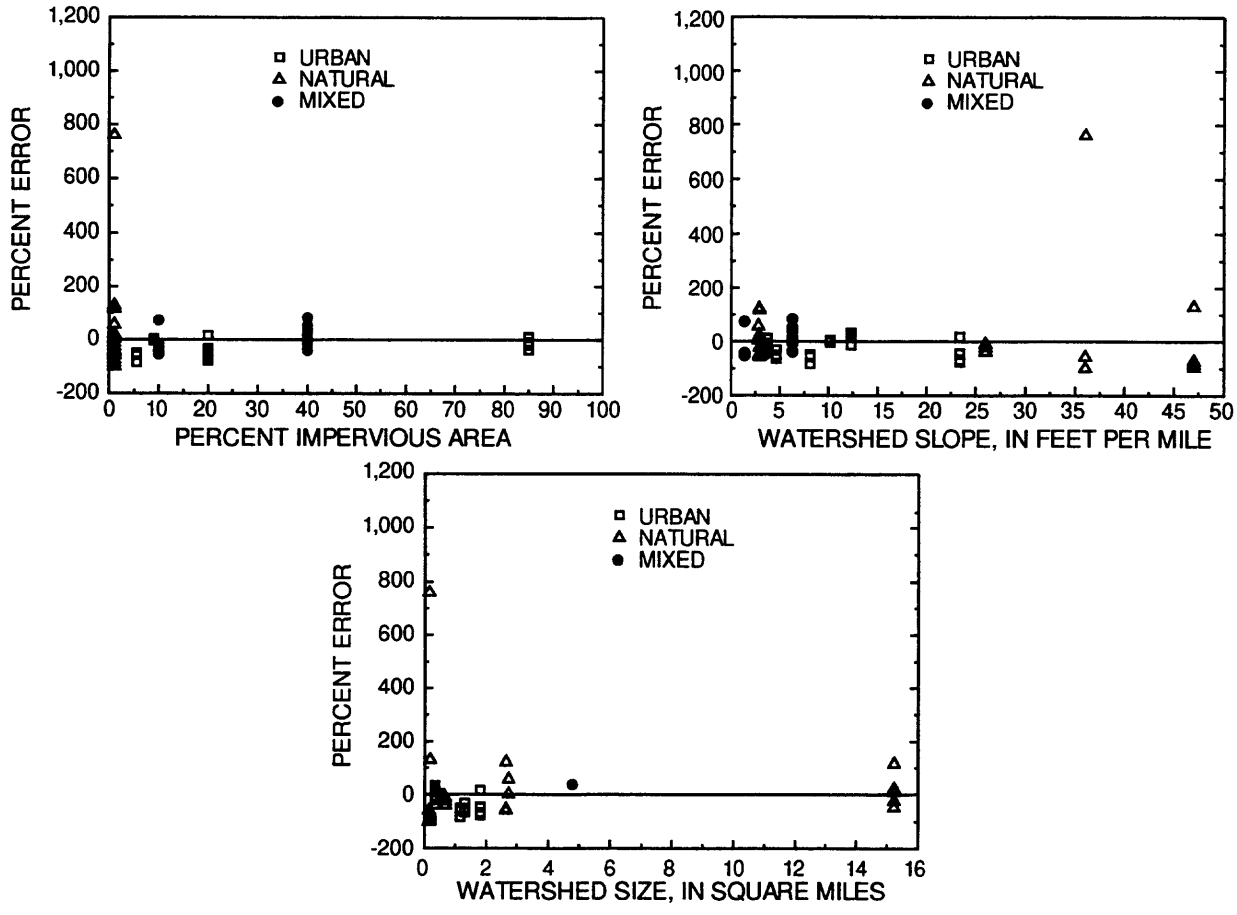


Figure 44. Comparison of the percent error between estimated and observed runoff volumes with percent impervious area, average watershed slope, and watershed size, for the Environmental Protection Agency Storm Water Management model, using the Green-Ampt infiltration method.

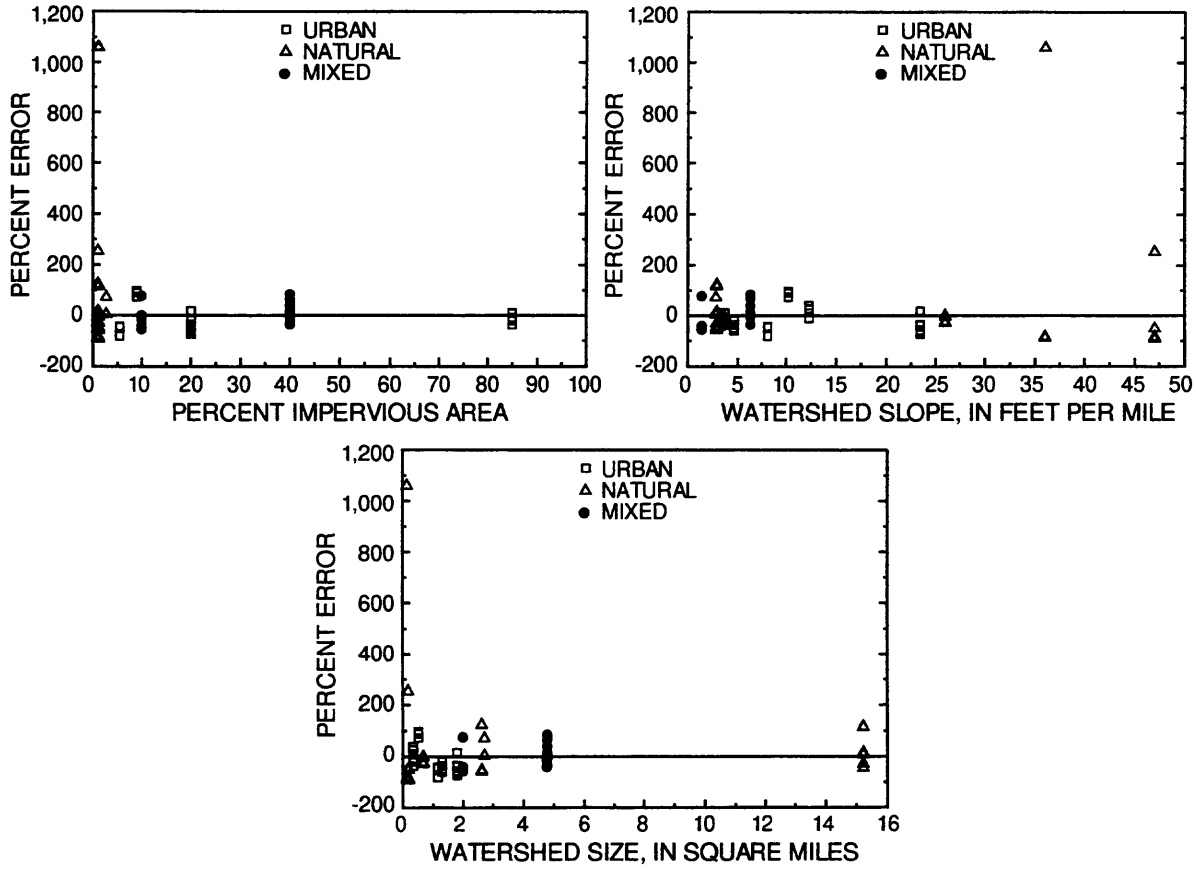


Figure 45. Comparison of the percent error between estimated and observed runoff volumes with percent impervious area average watershed slope, and watershed size, for the Environmental Protection Agency Storm Water Management model, using the Horton infiltration method.

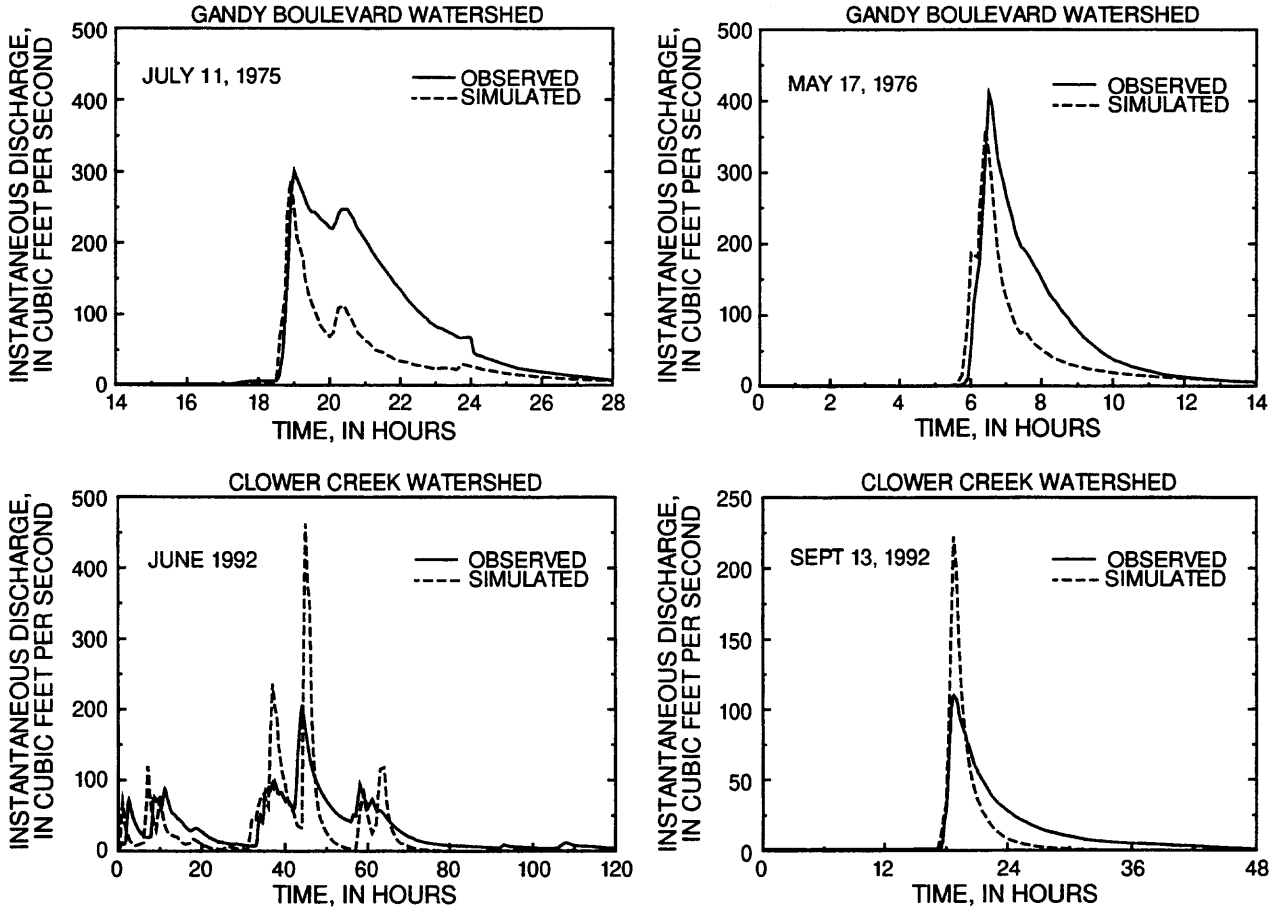


Figure 46. Typical observed hydrographs for storms occurring in the Gandy Boulevard and the Clower Creek watersheds and corresponding hydrographs simulated using the Environmental Protection Agency Storm Water Management model.

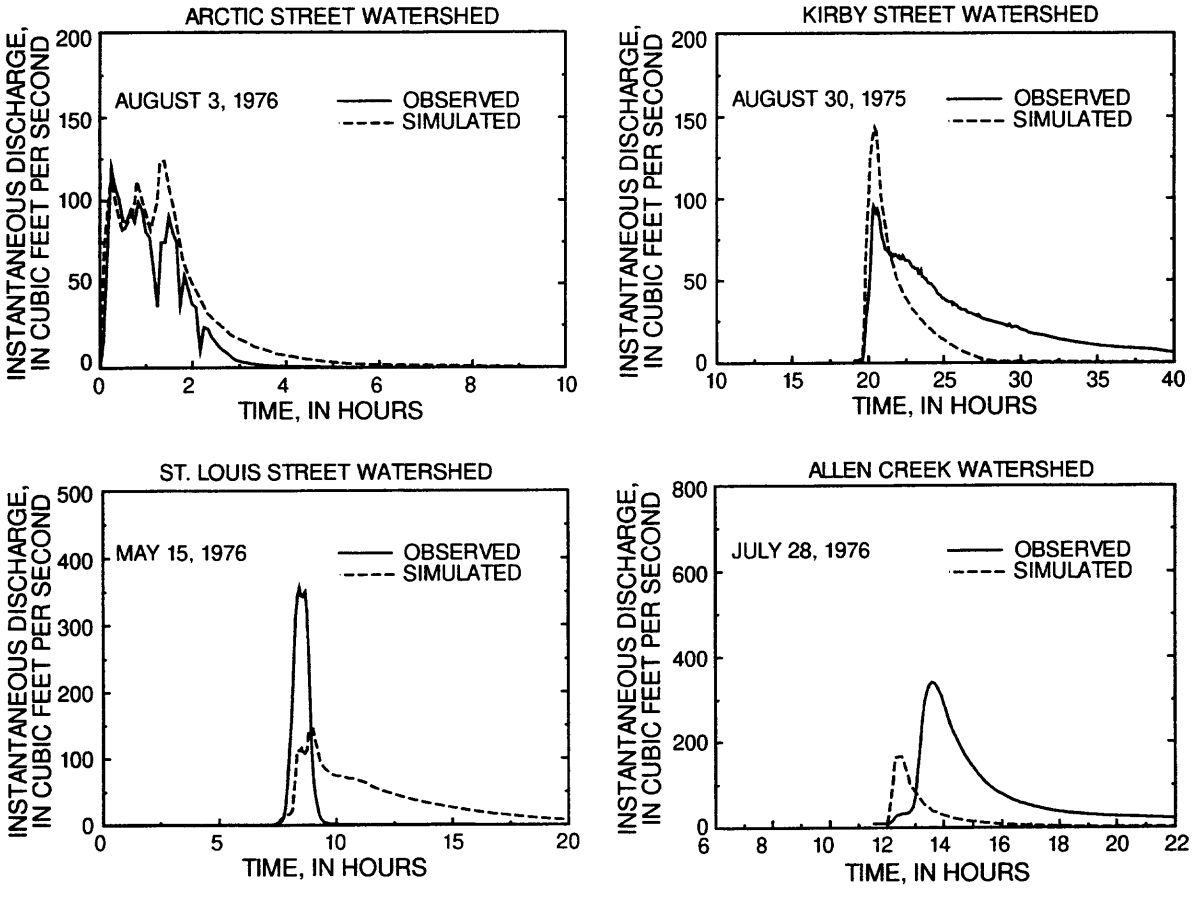


Figure 47. Typical observed hydrographs for storms occurring in the Arctic Street, Kirby Street, St. Louis Street and Allen Creek watersheds and corresponding hydrographs simulated using the Environmental Protection Agency Storm Water Management model.

simulated hydrograph for the St. Louis Street watershed was steep and matched the observed hydrograph; however, neither the peak discharge or the falling limb of the simulated hydrograph matched the observed hydrograph. The general shape of the simulated hydrograph for the Allen Creek watershed matched the observed hydrograph; however, the peak occurred about 1.5 hours before the observed peak and the runoff volume was underestimated by about 48 percent.

Observed and simulated hydrographs resulting from three different types of storms in the Walker Creek watershed are shown on figure 48. Land use in the Walker Creek watershed is mixed. The simulated hydrograph for a 4 day, high intensity storm (June, 1992) matched the observed hydrograph more accurately than the short duration, high intensity, summer thunderstorm (Sept 4, 1992), or the winter frontal storm (Jan 15, 1993). Peak discharges were overestimated for all three type storms; however, overestimations were greater for the summer thunder storms than for the other two storm types. The time to peak between the simulated and observed hydrographs differed by less than one hour for the three storm types. The falling limbs of the simulated hydrographs for the summer thunderstorms and the winter frontal storms did not match the observed hydrographs.

Observed and simulated hydrographs for the Catfish Creek and Gottfried Creek watersheds, also mixed land use watersheds are shown in figure 49. Peak discharge was only slightly underestimated for both watersheds, but the shape of the simulated hydrograph better matched the observed hydrograph for the Catfish Creek watershed than for the Gottfried Creek watershed. The predicted peak discharge occurred 1 hour before the observed discharge in the Catfish Creek watershed and 13.5 hours before the observed peak in the Gottfried Creek watershed.

Typical observed and simulated hydrographs for storms occurring in the three natural watersheds of IMC Creek, CFI-3 Creek and Grace Creek are shown on figure 50. The simulated hydrographs for the IMC Creek and CFI-3 Creek watersheds had lower peak discharges and runoff volumes than the observed hydrographs; whereas in the Grace Creek watershed, peak discharge was greater than the observed. All three simulated hydrographs had steep rising and falling limbs, while the observed hydrographs had long flat recession limbs. The long recession limbs on the observed hydrographs indicate that rainfall is being stored in the watershed and released to the stream at a slow rate.

Typical observed and simulated hydrographs for the South Creek, Forked Creek, and Rock Creek watersheds, natural watersheds, characterized by low slopes, large wetlands, and high water tables, are shown on figure 51. The shape of the observed hydrographs in all three watersheds are similar, with long, flat recession limbs. The simulated hydrographs for these watersheds do not match the observed

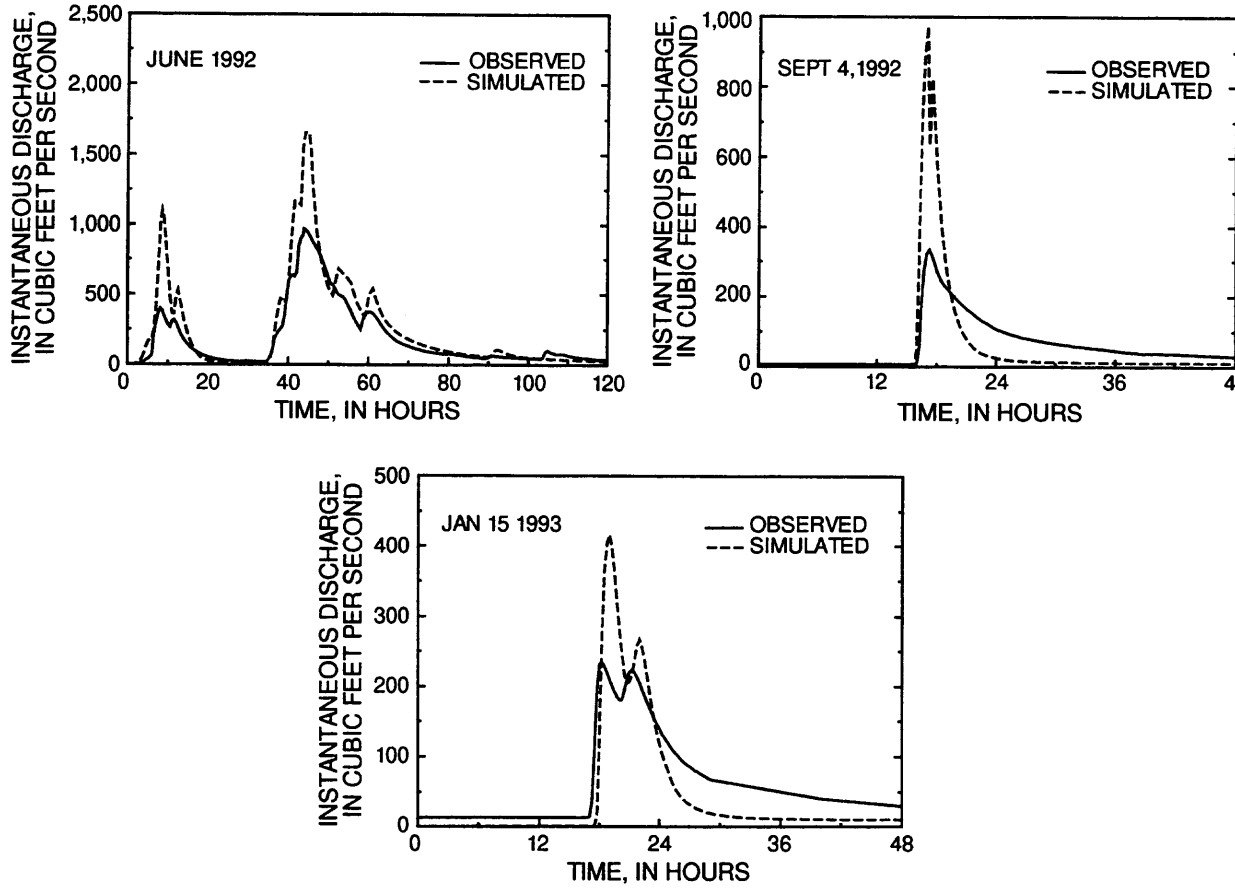


Figure 48. Typical observed hydrographs for storms occurring in the Walker Creek watershed and corresponding hydrographs simulated using the Environmental Protection Agency Storm Water Management model.

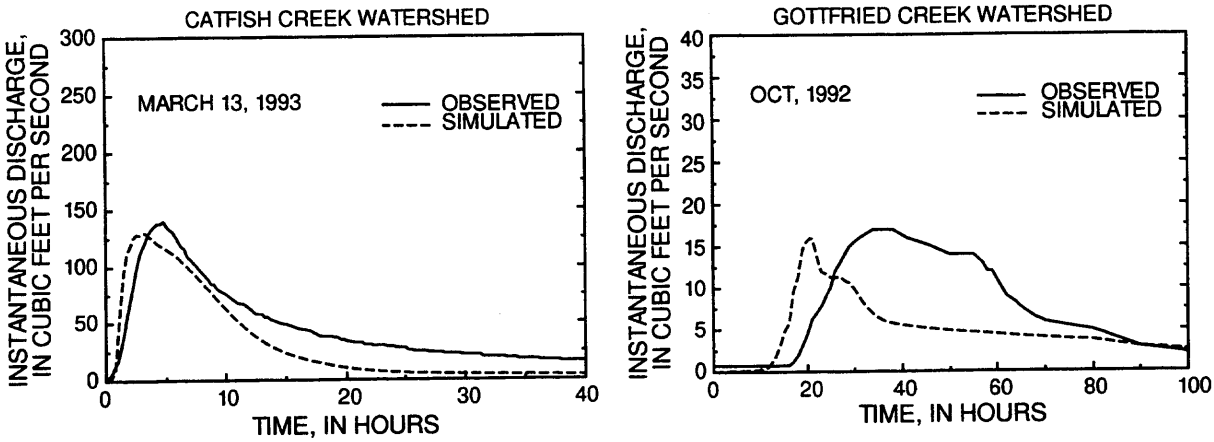


Figure 49. Typical observed hydrographs for storms occurring in the Catfish Creek and Gottfried Creek watersheds and corresponding hydrographs simulated using the Environmental Protection Agency Storm Water Management model.

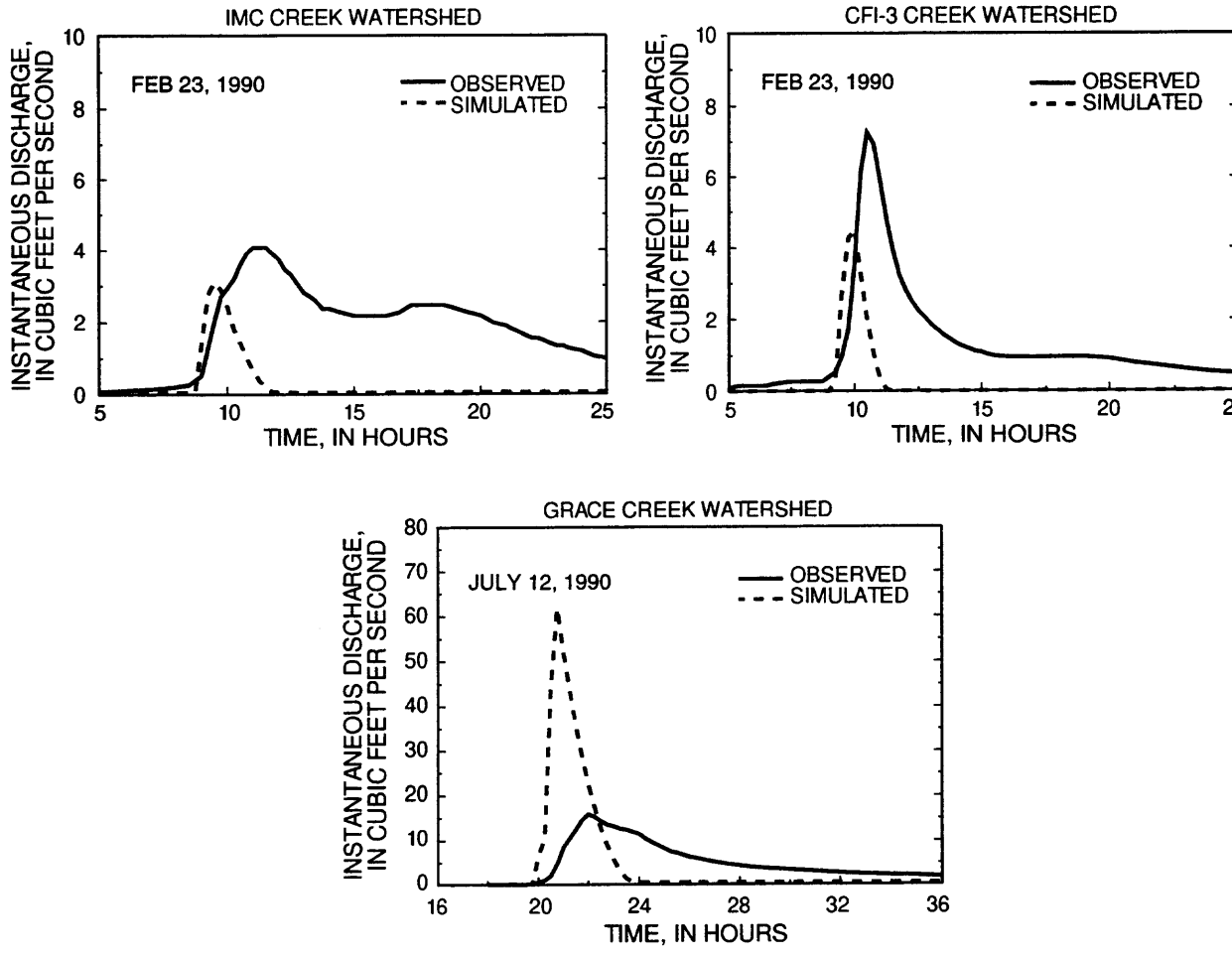


Figure 50. Typical observed hydrographs for storms occurring in the IMC, CFI-3 and Grace Creek watersheds and corresponding hydrographs simulated using the Environmental Protection Agency Storm Water Management model.

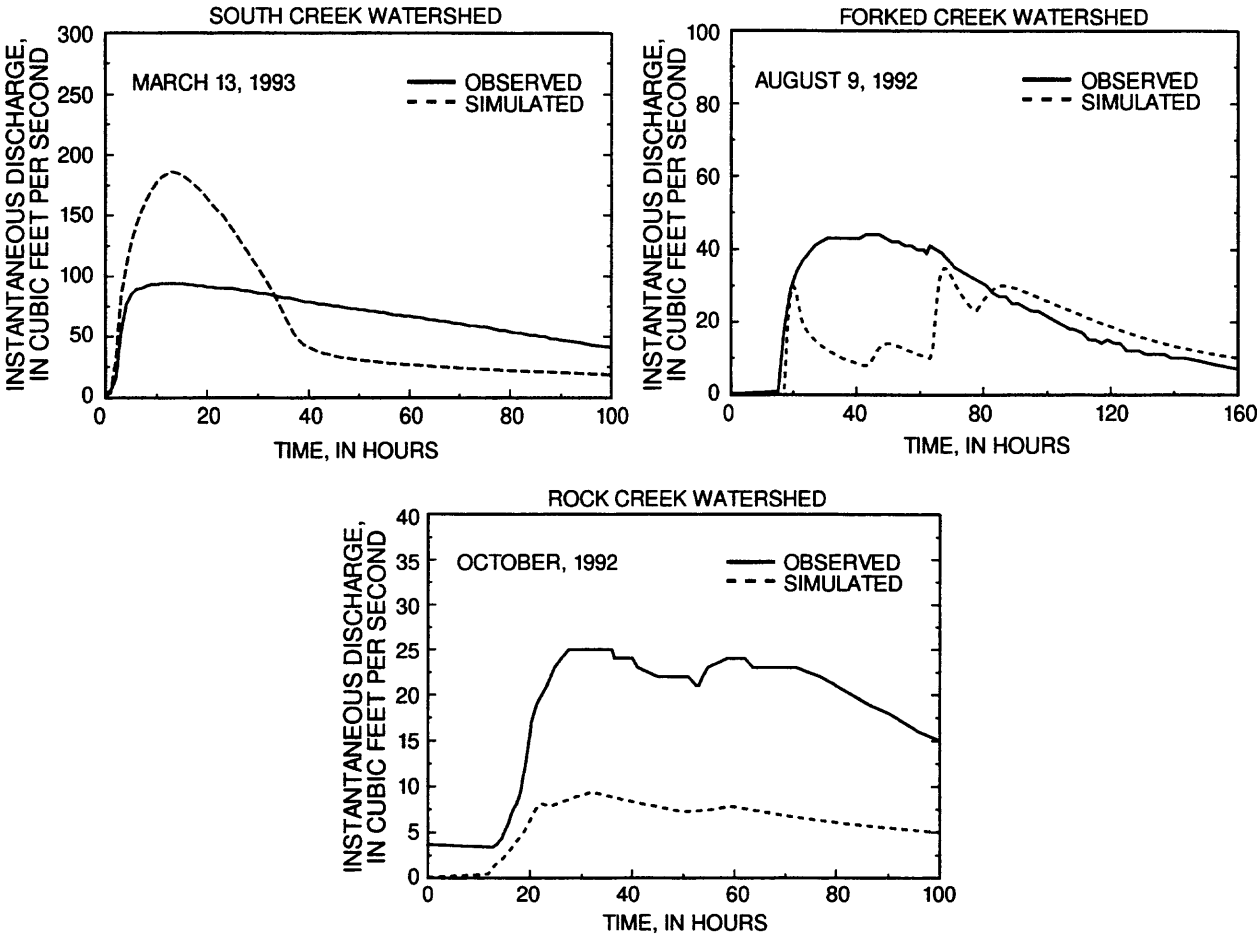


Figure 51. Typical observed hydrographs for storms occurring in the South, Forked, and Rock Creek watersheds and corresponding hydrographs simulated using the Environmental Protection Agency Storm Water Management model.

hydrographs in either size or shape. Peak discharges are overestimated for the South Creek watershed and underestimated for the Forked Creek and Rock Creek watersheds. Surface detention, subsurface storage and flow, and discharge from the surficial aquifer system influence the shape of the observed hydrographs in these watersheds.

STATISTICAL ANALYSIS OF DATA

The mean estimation error indicates the tendency of a method to under- or overestimate observed data. The mean estimation error, in percent, was calculated for each watershed, all urban watersheds, all natural watersheds, all mixed watersheds, and for all watersheds for each method using the following equation:

$$\xi = 100\left[\frac{\sum_{i=1}^{n}\left(\frac{est_i - obs_i}{obs_i}\right)}{n}\right] \qquad (8)$$

where:

ξ = mean estimation error, in percent;

est_i = estimated peak discharge for event i, in cubic feet per second or runoff volume, in inches;

obs_i = observed peak discharge for event i, in cubic feet per second or runoff volume, in inches;

n = number of storm events.

Mean estimation errors for peak discharge for all storms in each watershed for the rational method ranged from an underestimation of 31 percent to an overestimation of 767 percent. The smallest mean estimation error was calculated for storms in the urban watershed of Allen Creek. The largest was for storms in the natural watershed of South Creek (table 10). The mean estimation error for all storms occurring in the urban watersheds was an overestimation of about 67 percent. The error was an

Table 10. The mean estimation error for each watershed, all urban watersheds, all natural watersheds, all mixed watersheds and for all the watersheds, in percent, for peak discharges calculated using five common design techniques

[-, negative values represent underestimations; --, error could not be computed, U, urban; N, natural; M, mixed]

		Peak Discharge					
						Environmental Protection Agency Storm Water Management Model Infiltration Method	
Watershed Name	Watershed Classification	Rational Method	USGS Regression Equations	TR-20 Model	HEC-1 Model	Green-Ampt	Horton
Arctic Street storm drain	U	4.57	-87.3	3.73	75.2	33.5	34.7
Kirby Street drainage ditch	U	525	24.7	-28.4	42.5	37.3	38.0
St. Louis Street drainage ditch	U	-31.0	--	-60.9	-28.3	-63.0	-59.3
Gandy Boulevard drainage ditch	U	29.7	-74.4	1.21	80.9	-6.15	-4.42
Allen Creek	U	13.3	--	73.6	164	-13.1	-11.6
Clower Creek	U	21.3	-44.7	22.0	104	86.0	83.1
All Urban Watersheds		67.1	-25.3	12.5	88.0	19.6	20.2
IMC Creek	N	511	1140	153	331	111	135
Grace Creek	N	163	--	56.4	114	82.8	105
CFI-3 Creek	N	88.7	283	245	452	237	258
South Creek	N	767	-2.26	126	108	45.5	39.9
Forked Creek	N	446	-42.9	2.70	200	-10.8	-14.0
Rock Creek	N	351	8.26	-8.73	46.2	22.9	21.9
All Natural Watersheds		416	277	104	201	83.7	93.9
Walker Creek	M	249	-42.2	20.3	57.5	132	135
Catfish Creek	M	360	--	65.3	169	-4.21	-4.88
Gottfried Creek	M	319	72.3	110	140	36.3	30.0
All Mixed Watersheds		287	-13.6	46.7	98.4	83.1	83.9
All Watersheds		235	72.2	50.4	127	56.4	60.0

overestimation of 416 percent for all storms occurring in the natural watersheds and an overestimation of 287 percent for all storms occurring in the watersheds with mixed land use. The mean estimation error for all the storms modeled was an overestimation of 235 percent. This method overestimates peak discharges for all watershed types included in this study.

Mean estimation errors calculated for peak discharge for the USGS regression equations ranged from an underestimation of 87.3 percent to an overestimation of 1,140 percent (table 10). The smallest and largest errors were calculated for the South Creek and IMC Creek watersheds, respectively. The South Creek watershed is a large, natural watershed with low topographic relief. The IMC Creek watershed is a very small, natural watershed with fairly steep topographic relief. The mean estimation error for all storms occurring in the urban watersheds was an underestimation of about 25 percent. The error was an overestimation of 277 percent for all storms occurring in the natural watersheds and an overestimation of about 14 percent for all storms occurring in the watersheds with mixed land use. The mean estimation error for all the storms modeled was an overestimation of about 72 percent. The mean estimation errors calculated for runoff volume for the USGS regression equations ranged from an underestimation of 93.9 percent to an overestimation of 324 percent (table 11). The mean estimation error for all storms occurring in the urban watersheds was an underestimation of about 32 percent. The error was an overestimation of about 68 percent for all storms occurring in the natural watersheds and an underestimation of about 68 percent for all storms occurring in the watersheds with mixed land use. The mean runoff estimation error for all the storms modeled was an underestimate of 12.5 percent. The mean estimation errors indicate the regression equations have a tendency to overestimate peak discharges in all the watersheds included in this study. Runoff volumes for storms in the urban and mixed watersheds were underestimated and storms in the natural watersheds were overestimated. The regression equations have a general tendency to underestimate runoff volume.

Mean estimation errors calculated for peak discharge for the TR-20 model ranged from an underestimation of 60.9 percent to an overestimation of 245 percent (table 10). The mean estimation error for all storms occurring in the urban watersheds was an overestimation of 12.5 percent. The error was an overestimation of 104 percent for all storms occurring in the natural watersheds and an overestimation of about 47 percent for all storms occurring in the mixed watersheds. The mean estimation error for all the storms modeled was an overestimation of about 50 percent. The mean estimation errors calculated for runoff volume for the TR-20 model ranged from an underestimation of 41.5 percent to an overestimation of 395 percent (table 11). The mean estimation error for all storms

Table 11. The mean estimation error for each watershed, all urban watersheds, all natural watersheds, all mixed watersheds and for all the watersheds, in percent, for runoff volumes calculated using four common design techniques

[-, negative values represent underestimations; --, error could not be computed, U, urban; N, natural; M, mixed]

Watershed name	Watershed classification	Runoff Volume				
		USGS regression equations	TR-20 model	HEC-1 model	Environmental Protection Agency Storm Water Management Model infiltration method	
					Green-Ampt	Horton
Arctic Street storm drain	U	-87.7	32.4	33.0	15.0	20.1
Kirby Street drainage ditch	U	35.0	6.70	7.77	-63.3	-56.8
St. Louis Street drainage ditch	U	--	145	145	0.62	86.1
Gandy Boulevard drainage ditch	U	-52.0	-2.64	-2.64	-54.0	-43.6
Allen Creek	U	--	-13.2	-13.1	-47.6	-42.1
Clower Creek	U	-93.9	27.9	29.4	-12.8	-13.9
All Urban Watersheds		-31.8	24.6	25.2	-27.6	-14.3
IMC Creek	N	324	59.4	58.5	-30.5	6.55
Grace Creek	N	--	19.3	19.3	-46.0	-19.5
CFI-3 Creek	N	290	395	395	201	296
South Creek	N	-90.2	55.2	17.5	15.9	12.7
Forked Creek	N	-93.9	-29.5	-6.95	30.4	38.0
Rock Creek	N	-91.1	-41.5	-3.70	3.70	4.03
All Natural Watersheds		67.8	75.8	74.2	21.4	47.1
Walker Creek	M	-75.1	43.9	43.6	11.1	13.8
Catfish Creek	M	--	103	94.1	-30.0	-16.9
Gottfried Creek	M	-90.9	-1.97	-2.47	-6.83	-6.73
All Mixed Watersheds		-79.1	49.7	47.4	-1.72	2.97
All Watersheds		-12.5	47.3	46.5	-5.35	9.69

occurring in the urban watersheds was an overestimation of about 25 percent. The error was an overestimation of about 76 percent for all storms occurring in the natural watersheds and an overestimation of about 50 percent for all storms occurring in the mixed land use watersheds. The mean estimation error for all the storms modeled was an overestimation of about 47 percent. The model has a tendency to overestimate peak discharges and runoff volumes for storms occurring in the all the watershed types.

Mean estimation errors calculated for peak discharge for the HEC-1 model ranged from an underestimation of 28.3 percent to an overestimation of 452 percent (table 10). The mean estimation error for all storms occurring in the urban watersheds was an overestimation of 88 percent. The error was an overestimation of 201 percent for all storms occurring in the natural watersheds and an overestimation of about 98 percent for all storms occurring in the mixed watersheds. The mean estimation error for all the storms modeled was an overestimation of 127 percent. The mean estimation errors calculated for runoff volume for the HEC-1 model ranged from an underestimation of 13.1 percent to an overestimation of 395 percent (table 11). The mean estimation error for all storms occurring in the urban watersheds was an overestimation of about 25 percent. The error was an overestimation of about 74 percent for all storms occurring in the natural watersheds and an overestimation of about 47 percent for all storms occurring in the mixed watersheds. The mean estimation error for all the storms modeled was an overestimation of 46.5 percent. The model has a tendency to overestimate peak discharges and runoff volumes for storms occurring in all the watershed types.

Mean estimation errors calculated for peak discharge for the EPA SWMM model with the Green-Ampt infiltration method, ranged from an underestimation of 63.0 percent to an overestimation of 237 percent (table 10). The mean estimation error for all storms occurring in the urban watersheds was an overestimation of about 20 percent. The error was an overestimation of about 84 percent for all storms occurring in the natural watersheds and an overestimation of about 83 percent for all storms occurring in the mixed land use watersheds. The mean estimation error for all the storms modeled was an overestimation of about 56 percent. The mean estimation errors calculated for runoff volume for the EPA SWMM model with the Green-Ampt infiltration method, ranged from an underestimation of 63.3 percent to an overestimation of 201 percent (table 11). The mean estimation error for all storms occurring in the urban watersheds was an underestimation of about 28 percent. The error was an overestimation of about 21 percent for all storms occurring in the natural watersheds and an

underestimation of under 2 percent for all storms occurring in the mixed land use watersheds. The mean estimation error for runoff volume for all the storms modeled was an underestimation of about 5 percent. The model with the Green-Ampt infiltration method has a tendency to overestimate peak discharges and slightly underestimate runoff volume for storms occurring in the watersheds included in the study.

Mean estimation errors calculated for peak discharge for the EPA SWMM model with the Horton infiltration method, ranged from an underestimation of 59.3 percent to an overestimation of 258 percent (table 10). The mean estimation error for all storms occurring in the urban watersheds was an overestimation of about 20 percent. The error was an overestimation of about 94 percent for all storms occurring in the natural watersheds and an overestimation of about 84 percent for all storms occurring in the mixed land use watersheds. The mean estimation error for all the storms modeled was an overestimation of 60 percent. The mean estimation errors calculated for runoff volume for the EPA SWMM model with the Horton infiltration method, ranged from an underestimation of 56.8 percent to an overestimation of 296 percent (table 11). The mean estimation error for all storms in the urban watersheds was an underestimation of about 14 percent, an overestimation of about 47 percent in the natural watersheds, and an underestimation of about 3 percent in the mixed land use watersheds. The mean estimation error for all the storms modeled was an overestimation of about 10 percent.

The model with the Horton infiltration method overestimates peak discharges for all watersheds included in the study. It underestimates runoff volumes in the urban watersheds, and overestimates runoff volumes in the natural and mixed watersheds.

The standard estimation error quantifies the absolute magnitude of the error, in percent. It could not be calculated for individual watersheds with less than 2 equivalent storms; however, all equivalent storms were used to calculate the standard error for each of the watershed types and for all the watersheds, all natural watersheds, all mixed watersheds. The standard estimation error was calculated using the following equation:

$$\varepsilon = 100\left[\frac{\sum_{i=1}^{n}\left(\frac{est_i - obs_i}{obs_i}\right)^2}{n-1}\right]^{0.5} \qquad (9)$$

where:

ε = standard estimation error, in percent;

est_i = estimated peak discharge for event i, in cubic feet per second or runoff volume, in inches;

obs_i = observed peak discharge for event i, in cubic feet per second or runoff volume, in inches;

n = number of storm events.

The standard estimation errors calculated for peak discharges for the five common design techniques are shown in table 12. For the urban watersheds, the USGS regression equations, the TR-20 model and the EPA SWMM model using both the Green-Ampt and the Horton infiltration methods had standard estimation errors less than 65 percent. The rational method and the HEC-1 model had standard estimation errors of 193 and 121 percent, respectively. The TR-20, HEC-1 models, and the EPA SWMM model with the Green-Ampt infiltration method, had standard estimation errors that ranged between 207 and 358 percent for the natural watersheds. The rational method and the USGS regression equations had standard errors of 695 and 588 percent, respectively. The USGS regression equations for peak discharge and the TR-20 model had standard errors of less than 100 percent for the mixed watersheds. The SWMM model with both infiltration methods and the HEC-1 model had standard errors that ranged between 116 and 133 percent. A standard error of 404 percent was calculated for the mixed watersheds using the rational method. When the standard estimation error was calculated for all the watersheds, the TR-20 model, and the SWMM model with both infiltration methods had errors that ranged between 128 and 152 percent. The HEC-1 model had an error of 223 percent, and the USGS regression equations and the rational method had a standard errors greater than 300 percent.

The standard estimation errors for runoff volumes for the methods other than the rational method, are shown in table 13. The SWMM model with both the Green-Ampt and Horton infiltration methods had standard estimation errors of about 26 and 44 percent, respectively, for the urban watersheds. The TR-20 and HEC-1 models had standard errors of about 60 percent, and the USGS regression equation had a standard error of about 81 percent for the urban watersheds. All the methods except the SWMM model with the Green-Ampt infiltration method had standard error greater than 200 percent, for the natural watersheds. Standard errors of about 42 percent were calculated for the mixed watersheds using

Table 12. Summary of the standard estimation error for each watershed, all urban watersheds, all natural waterseds, all mixed watersheds and for all watersheds, in percent, for peak discharges calculated using five common design techniques

[--, standard estimation error could not be computed, U, urban; N, natural; M, mixed]

		Peak Discharge					
						Environmental Protection Agency StormWater Management Model infiltration method	
Watershed name	Watershed classification	Rational method	USGS regression equations	TR-20 model	HEC-1 model	Green-Ampt	Horton
Arctic Street storm drain	U	35.7	--	36.3	110	47.9	49.0
Kirby Street drainage ditch	U	667	52.3	57.8	98.0	54.5	55.8
St. Louis Street drainage ditch	U	46.2	--	75.1	37.5	78.5	74.1
Gandy Boulevard drainage ditch	U	119	--	23.8	99.2	14.8	14.5
Allen Creek	U	54.9	--	109	206	44.1	43.5
Clower Creek	U	79.5	79.0	31.8	120	104	101
All Urban Watersheds		193	64.4	57.8	121	60.9	59.6
IMC Creek	N	879	--	308	580	282	329
Grace Creek	N	313	--	99.0	162	137	179
CFI-3 Creek	N	264	--	441	770	548	602
South Creek	N	1127	--	192	168	67.7	62.5
Forked Creek	N	972	--	9.55	377	15.7	21.5
Rock Creek	N	661	--	63.7	125	114	112
All Natural Watersheds		695	588	207	358	217	244
Walker Creek	M	351	57.8	50.4	81.2	150	154
Catfish Creek	M	481	--	76.5	196	17.6	16.4
Gottfried Creek	M	635	--	191	234	74.7	74.7
All Mixed Watersheds		404	63.0	84.2	133	116	118
All Watersheds		452	308	128	223	139	152

Table 13. Summary of the standard estimation error for each watershed, all urban watersheds, all natural watersheds, all mixed watersheds, and for all watersheds, in percent, for runoff volumes calculated using four common design techniques

[--, standard estimation error could not be computed, U, urban; N, natural; M, mixed]

		Runoff Volume				
					Environmental Protection Agecy Storm Water Management Model infiltration method	
Watershed name	Watershed classification	USGS regression equations	TR-20 model	HEC-1 model	Green-Ampt	Horton
Arctic Street storm drain	U	--	41.6	42.5	26.7	31.6
Kirby Street drainage ditch	U	73.8	63.4	64.2	79.5	72.8
St. Louis Street drainage ditch	U	--	179	179	3.55	106
Gandy Boulevard drainage ditch	U	--	29.1	29.1	61.8	51.5
Allen Creek	U	--	31.4	31.3	62.2	56.5
Clower Creek	U	133	44.4	46.6	23.7	24.5
All Urban Watersheds		80.6	60.0	60.5	44.2	26.5
IMC Creek	N	--	194	193	112	165
Grace Creek	N	--	61.6	61.6	58.0	26.5
CFI-3 Creek	N	--	735	735	544	755
South Creek	N	--	121	109	64.8	64.2
Forked Creek	N	--	54.8	87.3	57.4	72.1
Rock Creek	N	--	92.5	80.2	103	103
All Natural Watersheds		231	254	252	184	252
Walker Creek	M	102	82.2	81.9	40.0	42.1
Catfish Creek	M	--	123	112	37.3	25.7
Gottfried Creek	M	--	81.0	80.2	71.2	72.7
All Mixed Watersheds		92.1	86.4	83.3	42.3	42.2
All Watersheds		136	152	151	108	145

the SWMM model with both infiltration methods. The TR-20 model and the HEC-1 model had standard errors of 86.4 and 83.3 percent, respectively, for the mixed watersheds. The USGS regression equations for runoff volume produced a standard estimation error of about 92 percent. When the standard estimation error was calculated for all the watersheds, the four techniques produced errors that ranged between 108 percent and 152 percent.

SUMMARY AND CONCLUSION

Measured peak discharges and runoff volumes were compared to estimated values using the rational method, the USGS regional regression equations, the NRCS TR-20 model, the U.S. Army Corp of Engineers HEC-1 model and the EPA SWMM model. Sixty-six storms in 15 watersheds located in west-central Florida were estimated. Observed rainfall was used with all these techniques except the USGS regression equations, which calculates runoff from rainfall for specified recurrence intervals. Estimated peak discharge and runoff data were then compared to observed data. Six of the watersheds are urban, 6 are natural, and 3 watersheds have varying degrees of natural, agricultural or urban characteristics. They range in size from 0.14 to 15.22 mi^2, with slopes that range from 1.4 ft/mi to 47 ft/mi.

Peak discharges and runoff volumes were calculated with each of these techniques except for the rational method which only provides a peak discharge. Techniques were applied using recommended or customary procedures. The choice of input parameters was not influenced by observed data.

The rational method is usually applied in sewered or natural watersheds with drainage areas less than 5 mi^2, where infiltration, surface detention, and time of concentration are not large influences. The rational method overestimated peak discharge rates for most of the storms modeled. Estimation errors were generally smaller for storms occurring in the six urban watersheds. The largest error was for a storm occurring in the South Creek watershed which is a large natural watershed, and contains over 30 percent wetland areas. Examination of estimation errors and watershed characteristics indicate that errors decrease as the amount of urban development in the watershed increases. The mean estimation error for all the storms modeled indicates the method has a tendency to overestimate peak discharge for the watersheds included in the study. The largest errors were for storms occurring in the natural watersheds. The smallest errors were for storms occurring in the urban watersheds.

The USGS regional regression equations are used to determine flood flow for specific recurrence intervals; therefore, direct comparison of estimated and observed discharges from actual storms could not be made. However, observed peak discharges and runoff volumes from 16 storms with rainfall comparable to specific recurrence intervals could be used to compare estimated runoff to measured runoff. This method underestimated peak discharge and runoff volumes for most individual storms. Mean estimation errors for peak discharge indicate the method was more accurate for the urban watersheds than for the natural and mixed watersheds. When the runoff volume regression equations were used, mean estimation errors indicate the method was more accurate for the urban watersheds than for the natural and mixed watersheds. The mean estimation error for all the storms modeled indicates the regression equations have a tendency to overestimate peak discharge and underestimate runoff volume for the watersheds included in the study. The watershed characteristics in this study are closer to the watershed characteristics used to develop the peak discharge regression equations, but differ from those used to develop the runoff volume regression equations. The runoff regression equation may not be applicable to the type of watersheds located in west-central Florida.

Peak discharges and runoff volumes for most storms were overestimated using the TR-20 model. The average errors between observed and estimated discharges and runoff volumes are smaller for the six urban watersheds than for the six natural watersheds using this method. Mean estimation errors for peak discharge indicate the method is more accurate for the urban watersheds than for the mixed or natural watersheds. Mean estimation errors for runoff volume data indicate the method is more accurate for the urban and mixed watersheds than for the natural watersheds. The mean estimation errors for all the storms modeled indicate the model has a tendency to overestimate peak discharges and runoff volumes for the watersheds. Examination of estimation errors and curve numbers indicate errors decrease as the average curve number for the watershed increases.

Peak discharges and runoff volumes for most storms were overestimated using the U.S. Army Corp of Engineers HEC-1 model. The average errors between observed and estimated discharges and runoff volumes are smaller for the six urban watersheds than for the six natural watersheds using this method. Mean estimation errors for peak discharge and runoff data indicate the method is more accurate for the urban watersheds than for the mixed or natural watersheds. The mean estimation error for all the storms modeled indicates the model has a tendency to overestimate peak discharge rates and runoff volumes for the watersheds included in the study. Examination of estimation errors and curve numbers indicates that errors decrease as the average curve number for the watershed increases.

The EPA SWMM model was run using both the Green-Ampt and the Horton infiltration methods, in separate simulations. Estimates calculated with the Horton method were slightly higher than those calculated with the Green-Ampt method. The average errors between observed and estimated peak discharges and runoff volumes are smaller for the six urban watersheds than for the six natural watersheds using the Green-Ampt infiltration method. Mean estimation errors for peak discharge indicate the Green-Ampt infiltration method is more accurate for the urban watersheds than for the mixed or natural watersheds. Mean estimation errors for runoff volume data indicate the Green-Ampt method is more accurate for the mixed watersheds than for the urban and natural watersheds. The mean estimation errors for all the storms modeled indicates the model, with the Green-Ampt infiltration method has a tendency to overestimate peak discharges and slightly underestimate runoff volumes. The mean estimation errors for peak discharges calculated using model with the Horton infiltration method, indicate the method is more accurate for the urban watersheds than for the mixed or natural watersheds. Mean estimation errors for runoff volume indicate that the Horton infiltration method is more accurate for the urban and mixed watersheds than for the natural watersheds. Comparison of estimation errors for peak discharge rates with watershed characteristics indicates very little correlation. Comparison of estimation errors for runoff volumes; however, indicates that errors generally decrease as the impervious area of the watershed increases. No correlation between runoff volume errors and other watershed characteristics is evident.

Evaluation of the standard estimation errors indicate the TR-20 model was more accurate than the other models for estimating peak discharges. The SWMM model with the Green-Ampt infiltration method was more accurate than the other models for estimating runoff volumes.

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Apendix 1. Estimated peak discharges and runoff volumes for synthetic storms for the 2, 5, 10, 25, 50, and 100 year recurrence intervals using the U.S. Geological Survey regional regression equations

[cfs, cubic feet per second; in, inches]

Watershed name	Recurrence interval	Estimated	
		Peak discharge (cfs)	Runoff volume (in)
Arctic Street storm drain	2	18	0.16
	5	40	.29
	10	60	.38
	25	89	.50
	50	113	.60
	100	140	.70
Kirby Street drainage ditch	2	75	.59
	5	157	1.00
	10	226	1.30
	25	327	1.70
	50	411	2.00
	100	502	2.30
St. Louis Street drainage ditch	2	43	.19
	5	93	.33
	10	135	.43
	25	196	.56
	50	247	.66
	100	303	.77
Gandy Boulevard drainage ditch	2	57	.24
	5	121	.40
	10	175	.53
	25	256	.69
	50	324	.82
	100	398	.95
Allen Creek	2	113	.24
	5	232	.40
	10	331	.51
	25	476	.66
	50	594	.78
	100	721	.90
IMC Creek	2	28	.41
	5	62	.72
	10	90	.94
	25	132	1.24
	50	166	1.46
	100	202	1.68

Apendix 1. Estimated peak discharges and runoff volumes for synthetic storms for the 2, 5, 10, 25, 50, and 100 year recurrence intervals using the U.S. Geological Survey regional regression equations (Continued)

[cfs, cubic feet per second; in, inches]

Watershed name	Recurrence interval	Estimated	
		Peak discharge (cfs)	Runoff volume (in)
CFI-3 Creek	2	23	.39
	5	50	.69
	10	74	.91
	25	108	1.19
	50	136	1.41
	100	166	1.63
Grace Creek	2	68	.38
	5	141	.64
	10	204	.83
	25	294	1.07
	50	368	1.27
	100	447	1.46
Walker Creek	2	164	.26
	5	332	.43
	10	471	.55
	25	676	.71
	50	845	.84
	100	1030	.97
Clower Creek	2	24	.18
	5	53	.32
	10	78	.42
	25	115	.55
	50	147	.66
	100	182	.76
Catfish Creek	2	156	.40
	5	315	.67
	10	448	.86
	25	645	1.12
	50	808	1.32
	100	988	1.52
South Creek	2	57	.10
	5	124	.17
	10	187	.23
	25	271	.30
	50	347	.35
	100	432	.42

Apendix 1. Estimated peak discharges and runoff volumes for synthetic storms for the 2, 5, 10, 25, 50, and 100 year recurrence intervals using the U.S. Geological Survey regional regression equations (Continued)

[cfs, cubic feet per second; in, inches]

Watershed name	Recurrence interval	Estimated	
		Peak discharge (cfs)	Runoff volume (in)
Forked Creek	2	26	.14
	5	57	.24
	10	85	.32
	25	128	.44
	50	164	.52
	100	205	.61
Gottfried Creek	2	34	.17
	5	74	.29
	10	109	.39
	25	161	.51
	50	205	.61
	100	253	.71
Rock Creek	2	18	.13
	5	40	.23
	10	61	.31
	25	92	.42
	50	118	.50
	100	148	.59

Appendix II

Selected Input parameters to the U.S. Environmental Protection Agency Storm Water Management Model

Appendix 2. Selected input paramaeters to the U.S. Environmental Protection Agency Storm Water Management model. Ranges of values are given where varying initial soil moisture conditions were modeled

[DCIA, directly connected impervious area (percentage of the basin); IMPN, impervious area Manning's number; PERVN, pervious area Manning's number; IDS, impervious area depression storage (inches/impervious area); PDS, pervious area depression storage (inches/pervious area); F_o, maximum infiltration rate (in/hr); F_c, minimum infiltration rate (in/hr); SUCT, average capillary suction (in.); --, not applicable]

Watershed name	Area (acres)	Width (ft)	DCIA	Watershed slope (%)	IMP N	PERVN	IDS	PDS	F_o	F_c	SUCT
Arctic Street storm drain	218	11000	40	0.00233	0.012	0.25	0.0	0.0	3.0	0.30	4.0
Kirby street drainage ditch	736	25344	5.5	0.00153	0.010	0.35	0.1	0.1	3.0	0.30	4.0
St. Louis Street drainage ditch	326	11827	9.0	0.00193	0.010	0.16	0.0	0.0	1.0	0.10	8.0
Gandy Boulevard drainage ditch	826	17200	20	0.00087	0.010	0.29	0.0	0.0	1.0	0.10	8.0
Allen Creek	1203	14800	20	0.00443	0.012	0.28	0.0	0.0	0.5-1.0	0.10	8.0
IMC Creek	109	2000	0	0.00890	--	0.37	--	0.0	2.0-3.0	0.20	4.0
CFI-3 Creek	90	3960	0	0.00682	--	0.37	--	0.0	2.0-3.0	0.30	4.0
Grace Creek	422	10800	0	0.00492	--	0.32	--	0.0	3.0	0.30	4.0
Walker Creek	13	760	25	0.00300	0.015	0.45	0.1	0.1	1.0-3.0	0.30	4.0
	423	9200	30	0.00050	0.015	0.45	0.1	0.1	1.0-3.0	0.30	4.0
	326	7000	40	0.00100	0.015	0.45	0.5	0.5	1.0-3.0	0.30	4.0
	109	1580	20	0.00400	0.015	0.45	0.1	0.1	1.0-3.0	0.30	4.0
	755	7000	15	0.00100	0.015	0.45	0.2	0.2	1.0-3.0	0.30	4.0
	486	7200	40	0.00300	0.015	0.45	0.3	0.3	1.0-3.0	0.30	4.0
	346	5400	25	0.00100	0.015	0.45	0.1	0.1	1.0-3.0	0.30	4.0
	346	9000	40	0.00080	0.015	0.45	0.2	0.2	1.0-3.0	0.30	4.0
	256	4500	40	0.00200	0.015	0.45	0.1	0.1	1.0-3.0	0.30	4.0
Clower Creek	223	5300	85	0.00070	0.012	0.35	2.0	0.5	4.0	0.10	4.0

Appendix 2. Selected input paramaeters to the U.S. Environmental Protection Agency Storm Water Management model. Ranges of values are given where varying initial soil moisture conditions were modeled (Continued)

[DCIA, directly connected impervious area (percentage of the basin); IMPN, impervious area Manning's number; PERVN, pervious area Manning's number; IDS, impervious area depression storage (inches/impervious area); PDS, pervious area depression storage (inches/pervious area); F_o, maximum infiltration rate (in/hr); F_c, minimum infiltration rate (in/hr); SUCT, average capillary suction (in.); --, not applicable]

Watershed name	Area (acres)	Width (ft)	DCIA	Watershed slope (%)	IMP N	PERVN	IDS	PDS	F_o	F_c	SUCT
Catfish Creek	128	11000	5	0.00090	0.015	0.50	0.0	0.0	1.0-3.0	0.20	4.0
	160	6400	0	0.00030	--	0.30	--	0.1	1.0-3.0	0.20	4.0
	96	3400	5	0.00030	0.015	0.30	0.0	0.1	1.0-3.0	0.20	4.0
	262	5500	0	0.00050	--	0.35	--	0.1	1.0-3.0	0.20	4.0
	134	6000	0	0.00030	--	0.45	--	0.1	0.5-1.0	0.10	4.0
	13	600	30	0.00050	0.015	0.30	0.0	0.0	0.5-1.0	0.10	4.0
	26	2200	10	0.00050	0.015	0.30	0.0	0.0	0.5-1.0	0.10	4.0
	70	3400	5	0.00060	0.015	0.35	0.0	0.1	1.0-3.0	0.20	4.0
	77	4000	0	0.00030	--	0.40	--	0.1	1.0-3.0	0.20	4.0
	70	4400	10	0.00050	0.015	0.40	0.0	0.0	1.0-3.0	0.20	4.0
	333	6400	10	0.00080	0.015	0.40	0.0	0.1	1.0-3.0	0.20	4.0
	243	10340	0	0.00030	--	0.30	--	0.1	0.5-1.0	0.10	4.0
	102	3200	0	0.00030	--	0.35	--	0.0	0.5-1.0	0.10	4.0
	154	5610	0	0.00030	--	0.35	--	0.0	1.0-3.0	0.20	4.0
	250	5000	15	0.00060	0.015	0.20	0.0	0.1	1.0-3.0	0.20	4.0
	96	2660	0	0.00050	--	0.45	--	0.0	1.0-3.0	0.20	4.0
	442	2800	0	0.00020	--	0.45	--	0.1	1.0-3.0	0.20	4.0
	173	7200	20	0.00060	0.015	0.45	0.0	0.0	1.0-3.0	0.20	4.0
	128	5100	20	0.00070	0.015	0.45	0.0	0.0	1.0-3.0	0.20	4.0
	96	2600	0	0.00050	--	0.45	--	0.0	1.0-3.0	0.20	4.0

Appendix 2. Selected input paramaeters to the U.S. Environmental Protection Agency Storm Water Management model. Ranges of values are given where varying initial soil moisture conditions were modeled (Continued)

[DCIA, directly connected impervious area (percentage of the basin); IMPN, impervious area Manning's number; PERVN, pervious area Manning's number; IDS, impervious area depression storage (inches/impervious area); PDS, pervious area depression storage (inches/pervious area); F_0, maximum infiltration rate (in/hr); F_c, minimum infiltration rate (in/hr); SUCT, average capillary suction (in.); --, not applicable]

Watershed name	Area (acres)	Width (ft)	DCIA	Watershed slope (%)	IMP N	PERVN	IDS	PDS	F_o	F_c	SUCT
South Creek	269	6720	0	0.00030	--	0.40	--	0.0	3.0	0.05	4.0
	595	5720	5	0.00030	0.015	0.40	0.0	0.2	3.0	0.05	4.0
	832	7920	5	0.00050	0.015	0.40	0.0	0.0	3.0	0.10	4.0
	883	29100	0	0.00050	--	0.30	--	0.0	5.0	0.25	4.0
	256	12800	0	0.00040	--	0.30	--	0.0	5.0	0.25	4.0
	429	14000	0	0.00030	--	0.40	--	0.0	3.0	0.05	4.0
	262	10000	3	0.00040	0.015	0.40	0.0	0.1	3.0	0.05	4.0
	378	15600	0	0.00030	--	0.35	--	0.1	5.0	0.25	4.0
	512	12600	0	0.00030	--	0.35	--	0.1	3.0	0.10	4.0
	755	36000	1	0.00040	0.015	0.40	0.0	0.2	3.0	0.05	4.0
	640	18000	0	0.00030	--	0.35	--	0.1	5.0	0.25	4.0
	698	15200	0	0.00040	--	0.30	--	0.1	5.0	0.25	4.0
	416	6000	1	0.00040	0.015	0.35	0.0	0.1	3.0	0.05	4.0
	33	3200	0	0.00030	--	0.40	--	0.2	3.0	0.05	4.0
	1300	27000	0	0.00040	--	0.35	--	0.1	3.0	0.10	4.0
	704	9800	1	0.00060	0.015	0.35	0.0	0.1	3.0	0.05	4.0
	474	9200	0	0.00060	--	0.40	--	0.1	3.0	0.10	4.0
Forked Creek	58	2000	0	0.00060	--	0.25	--	0.1	3.0	0.05	4.0
	173	4800	0	0.00050	--	0.40	--	0.1	5.0	0.25	4.0
	877	5000	0	0.00060	--	0.25	--	0.2	5.0	0.25	4.0
	224	4200	0	0.00020	--	0.35	--	0.0	3.0	0.05	4.0
	102	2000	0	0.00040	--	0.35	--	0.2	3.0	0.05	4.0
	205	7650	0	0.00040	--	0.30	--	0.1	5.0	0.25	4.0
	166	8200	0	0.00060	--	0.40	--	0.1	5.0	0.25	4.0

Appendix 2. Selected input paramaeters to the U.S. Environmental Protection Agency Storm Water Management model. Ranges of values are given where varying initial soil moisture conditions were modeled (Continued)

[DCIA, directly connected impervious area (percentage of the basin); IMPN, impervious area Manning's number; PERVN, pervious area Manning's number; IDS, impervious area depression storage (inches/impervious area); PDS, pervious area depression storage (inches/pervious area); F_0, maximum infiltration rate (in/hr); F_c, minimum infiltration rate (in/hr); SUCT, average capillary suction (in.); --, not applicable]

Watershed name	Area (acres)	Width (ft)	DCIA	Watershed slope (%)	IMP N	PERVN	IDS	PDS	F_0	F_c	SUCT
Gottfried Creek	326	4700	0	0.00030	--	0.40	--	0.1	3.0	0.05	4.0
	64	2900	0	0.00050	--	0.40	--	0.1	3.0	0.05	4.0
	58	1200	10	0.00030	0.015	0.40	0.2	0.2	5.0	0.25	4.0
	192	2700	5	0.00050	0.015	0.40	0.2	0.2	3.0	0.05	4.0
	58	3000	2	0.00050	0.015	0.35	0.2	0.2	5.0	0.25	4.0
	19	600	0	0.00050	--	0.35	--	0.1	4.0	0.10	4.0
	38	1400	0	0.00050	--	0.40	--	0.1	5.0	0.25	4.0
	115	1500	10	0.00040	0.015	0.30	0.1	0.1	4.0	0.10	4.0
	64	1500	5	0.00050	0.015	0.40	0.1	0.1	3.0	0.05	4.0
	83	1200	10	0.00050	0.015	0.30	0.1	0.1	4.0	0.10	4.0
	77	900	0	0.00050	--	0.40	0.1	3.0	3.0	0.05	4.0
	77	2000	10	0.00040	0.015	0.30	0.1	0.1	5.0	0.25	4.0
	109	3000	10	0.00040	0.015	0.30	6.0	6.0	5.0	0.25	4.0
Rock Creek	589	12000	0	0.00070	--	0.35	--	0.5	3.0	0.05	4.0
	832	6000	0	0.00050	--	0.35	--	0.8	3.0	0.05	4.0
	262	9000	0	0.00050	--	0.35	--	0.1	3.0	0.05	4.0

Appendix III

Selected Ground Water Input Parameters to the U.S. Environmental Protection Agency Storm Water Management Model

Appendix 3. Selected ground water input paramaeters to the U.S. Environmental Protection Agency Storm Water Management model. Ranges of values are given where varying initial soil moisture conditions were modeled

[IMD, initial moisture deficit of soil (in/in); A_1, ground water flow coefficient (in/hrft); HKSAT, saturated subsurface hydraulic conductivity (in/hr); TH_1, initial upper zone moisture content (in/in); --, not applicable]

Watershed name	IMD	Depth to water table (ft)	A_1	B_1	Soil porosity (in/in)	Wilting point (in/in)	Field capacity (in/in)	HKSAT	TH_1
Arctic Street storm drain	0.30	--	--	--	--	--	--	--	--
Kirby street drainage ditch	0.30	--	--	--	--	--	--	--	--
St. Louis Street drainage ditch	0.25	--	--	--	--	--	--	--	--
Gandy Boulevard drainage ditch	0.25	--	--	--	--	--	--	--	--
Allen Creek	0.15-0.25	--	--	--	--	--	--	--	--
IMC Creek	0.25-0.30	2.6-4.4	4.06E-06	2	0.40	0.12	0.26	5.35	0.24-0.28
Grace Creek	0.25-0.30	1.8-2.0	1.58E-06	2	0.40	0.12	0.26	0.57	0.25-0.28
CFI-3 Creek	0.30	0.9-2.9	5.49E-05	2	0.40	0.12	0.26	0.65	0.25-0.28
Walker Creek	0.20-0.30	1.5-2.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
	0.20-0.30	1.5-2.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
	0.20-0.30	1.5-2.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
	0.20-0.30	1.5-2.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
	0.20-0.30	2.5-3.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
	0.20-0.30	1.5-2.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
	0.20-0.30	1.5-2.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
	0.20-0.30	1.5-2.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
	0.20-0.30	1.5-2.0	5.00E-05	2	0.40	0.13	0.26	2.00	0.25-0.28
Clower Creek	0.20	--	--	--	--	--	--	--	--

Appendix 3. Selected ground water input paramaeters to the U.S. Environmental Protection Agency Storm Water Management model. Ranges of values are given where varying initial soil moisture conditions were modeled (Continued)

[IMD, initial moisture deficit of soil (in/in); A_1, ground water flow coefficient (in/hrft); HKSAT, saturated subsurface hydraulic conductivity (in/hr); TH_1, initial upper zone moisture content (in/in); --, not applicable]

Watershed name	IMD	Depth to water table (ft)	A_1	B_1	Soil porosity (in/in)	Wilting point (in/in)	Field capacity (in/in)	HKSAT	TH_1
Catfish Creek	0.10-0.20	1.4-2.2	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.4-2.2	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-2.0	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.4-2.2	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-2.0	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-2.0	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-2.0	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.7-2.5	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	0.9-1.7	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.0-1.8	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.7-2.5	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.4-2.2	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-2.0	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-2.0	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-2.0	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	0.7-1.5	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-2.0	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	0.7-1.5	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	0.7-1.5	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30
	0.10-0.20	1.2-1.5	5.00E-05	2	0.40	0.17	0.26	6.00	0.25-0.30

Appendix 3. Selected ground water input paramaeters to the U.S. Environmental Protection Agency Storm Water Management model. Ranges of values are given where varying initial soil moisture conditions were modeled (Continued)

[IMD, initial moisture deficit of soil (in/in); A_1, ground water flow coefficient (in/hrft); HKSAT, saturated subsurface hydraulic conductivity (in/hr); TH_1, initial upper zone moisture content (in/in); --, not applicable]

Watershed name	IMD	Depth to water table (ft)	A_1	B_1	Soil porosity (in/in)	Wilting point (in/in)	Field capacity (in/in)	HKSAT	TH_1
South Creek	0.20	1.5-3.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5-3.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.5-3.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.0-2.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.0-3.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5-2.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5-2.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5-2.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0-2.5	5.00E-03	2	0.40	0.17	0.26	20.00	0.25
	0.20	1.5-2.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.0-2.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.0-3.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0-2.5	5.00E-03	2	0.40	0.17	0.26	20.00	0.25
	0.20	1.0-1.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5-2.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0-2.0	5.00E-03	2	0.40	0.17	0.26	20.00	0.25
	0.20	1.5-2.5	5.00E-03	2	0.40	0.17	0.26	20.00	0.25
Forked Creek	0.20	1.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0	5.00E-03	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.0	5.00E-03	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.0	5.00E-03	2	0.40	0.17	0.26	10.00	0.25

Appendix 3. Selected ground water input paramaeters to the U.S. Environmental Protection Agency Storm Water Management model. Ranges of values are given where varying initial soil moisture conditions were modeled (Continued)

[IMD, initial moisture deficit of soil (in/in); A_1, ground water flow coefficient (in/hrft); HKSAT, saturated subsurface hydraulic conductivity (in/hr); TH_1, initial upper zone moisture content (in/in); --, not applicable]

Watershed name	IMD	Depth to water table (ft)	A_1	B_1	Soil porosity (in/in)	Wilting point (in/in)	Field capacity (in/in)	HKSAT	TH_1
Gottfried Creek	0.20	1.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5	5.00E-03	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	3.0	5.00E-03	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.5	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0	5.00E-03	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.0	5.00E-03	2	0.40	0.17	0.26	10.00	0.25
	0.20	2.5	5.00E-03	2	0.40	0.17	0.26	10.00	0.25
Rock Creek	0.20	1.0	1.00E-04	2	0.40	0.17	0.26	10.00	0.25
	0.20	0.7	1.00E-03	2	0.40	0.17	0.26	10.00	0.25
	0.20	1.0	1.00E-02	2	0.40	0.17	0.26	10.00	0.25