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

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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:

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

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:

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

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:

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

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:

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

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]

		Input parameters			Peak discharge (cfs)				
							Error		
Watershed name	Watershed classification	C	I	A	Estimated	Observed	cfs	Percent	Date of storm
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)

					Peak discharge (cfs)				
	Watershed	Input parameters					Error		Date of
Watershed name	classification	C	I	A	Estimated	Observed	cfs	Percent	storm
		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

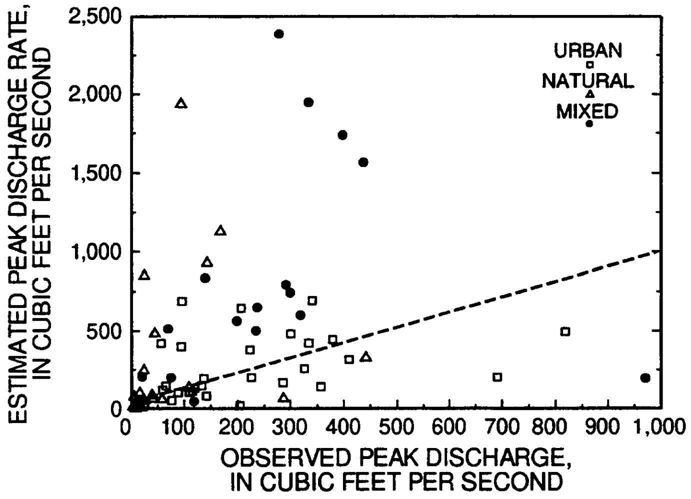


Figure 19. Comparison of observed and estimated peak discharge rates using the rational method.

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.

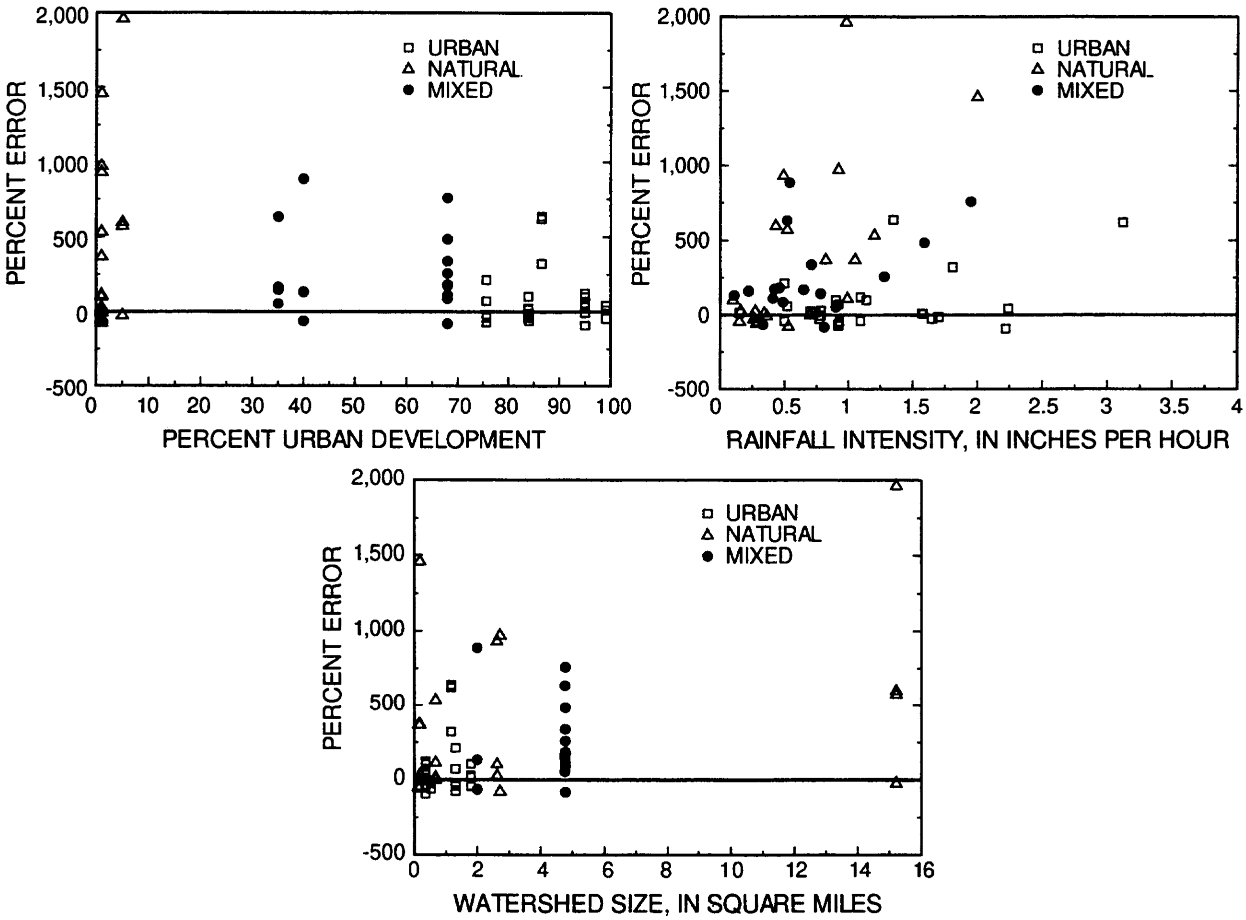
For the 3 mixed land use watersheds, peak discharges were overestimated for 15 of 17 storms. Errors ranged from an underestimation of 79.8 percent to an overestimation of 886 percent, and averaged about 287 percent.

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]

			Peak discharge (cfs)				Runoff volume (in)				
					Error				Error		
Watershed name	Watershed classification	Recurrence interval	Estimated	Observed	cfs	Percent	Estimated	Observed	inches	Percent	Date of storm
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

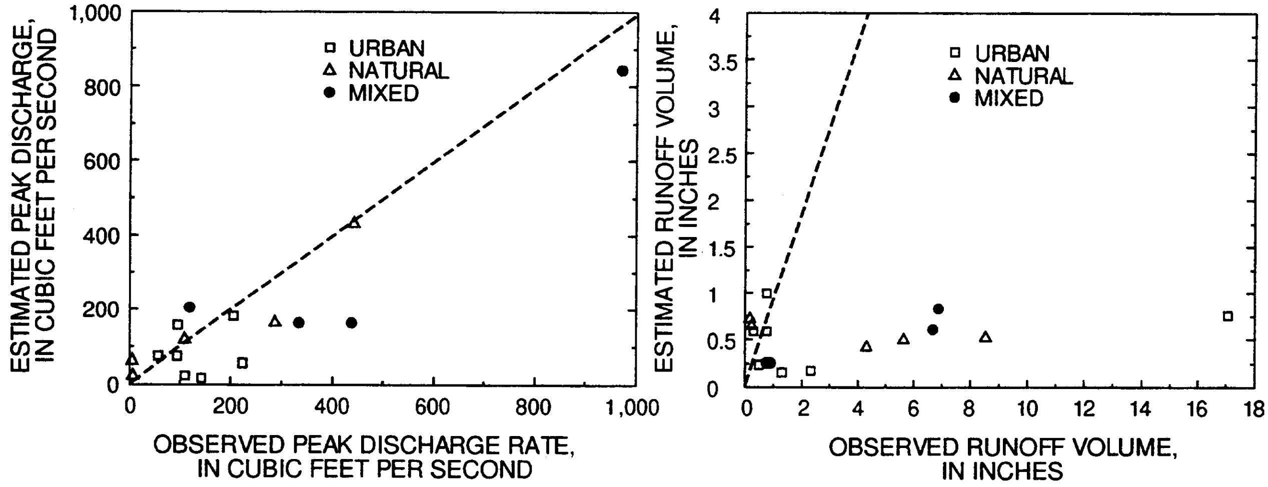
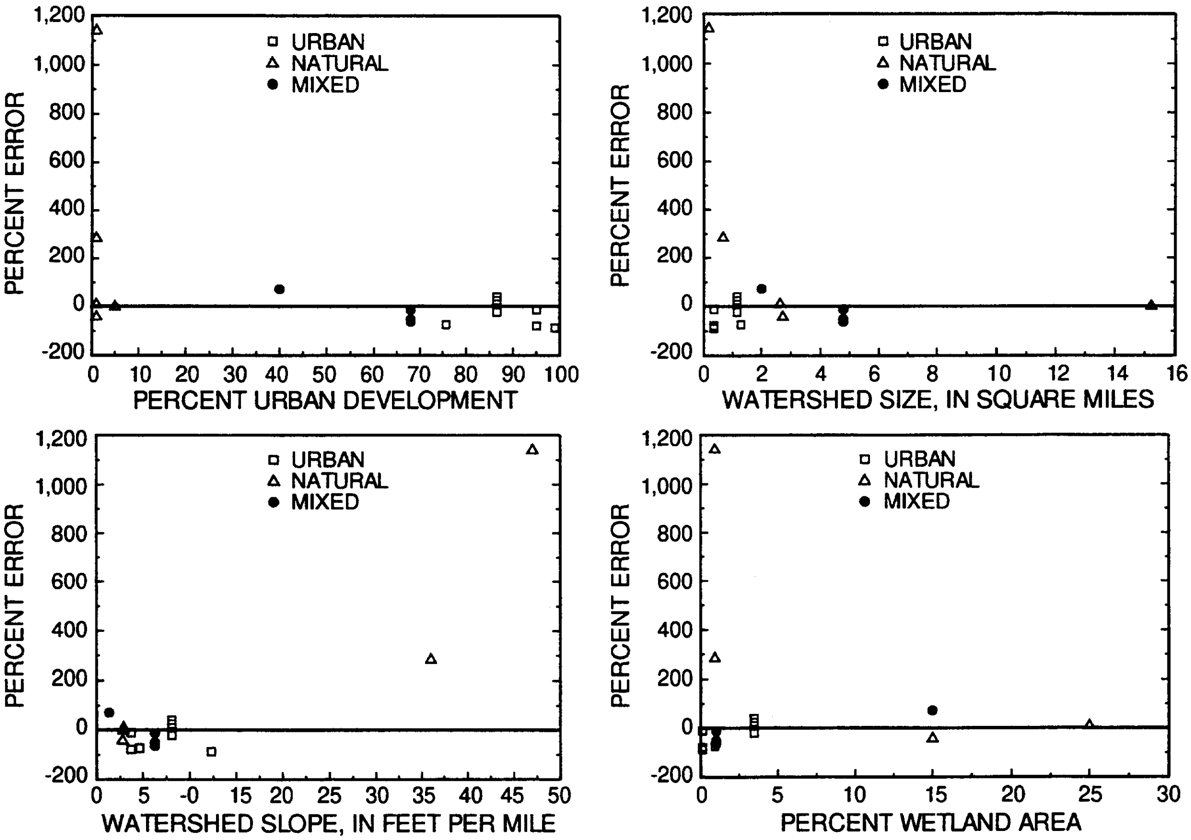


Figure 21. Comparison of observed and estimated peak discharge rates and runoff volumes using the U.S. Geological Survey regional regression equations.



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.

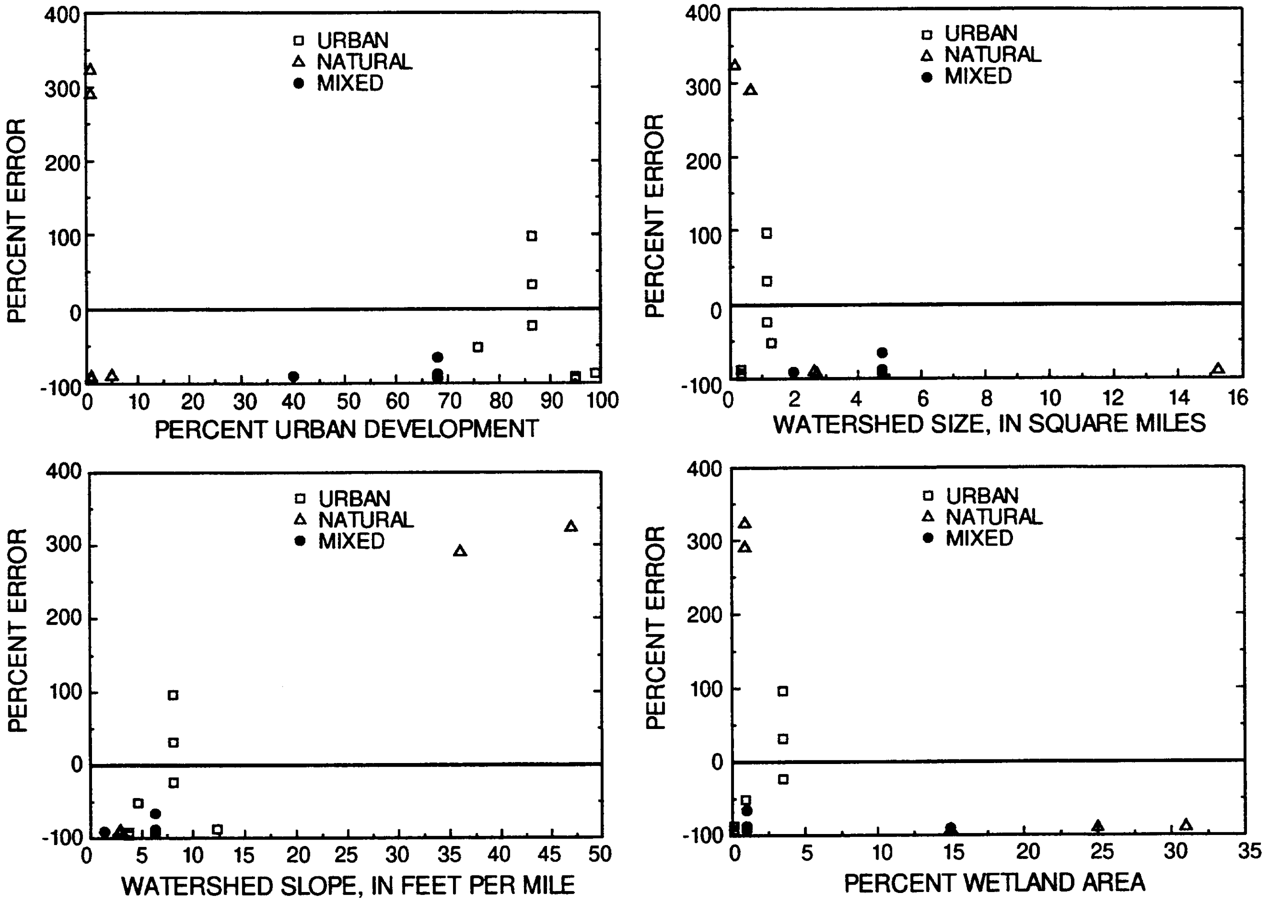


Figure 23. Comparison of the percent error between observed and estimated runoff volumes with watershed development, size, slope, and wetland area using the U.S. Geological Survey regional regression equations.

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
				Error				Error		
		Estimated	Observed	cfs	Percent	Estimated	Observed	Inches	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) Estimated	Observed	Error cfs	Error Percent	Runoff volume (in) Estimated	Observed	Error Inches	Error Percent	Date of storm
		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

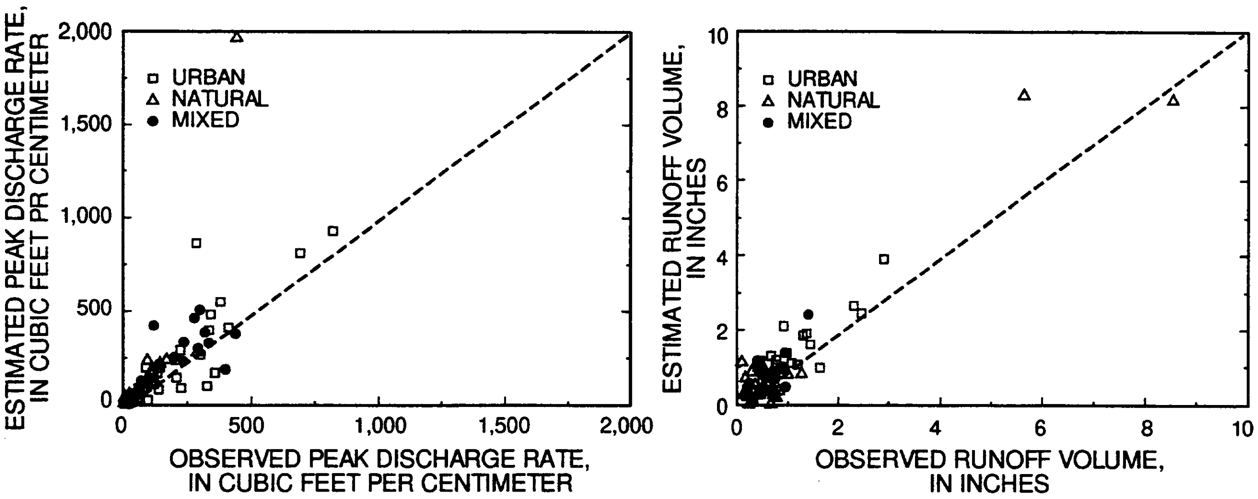


Figure 24. Comparison of the observed and estimated peak discharge rates and runoff volumes calculated using the NRCS TR-20 model.

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.

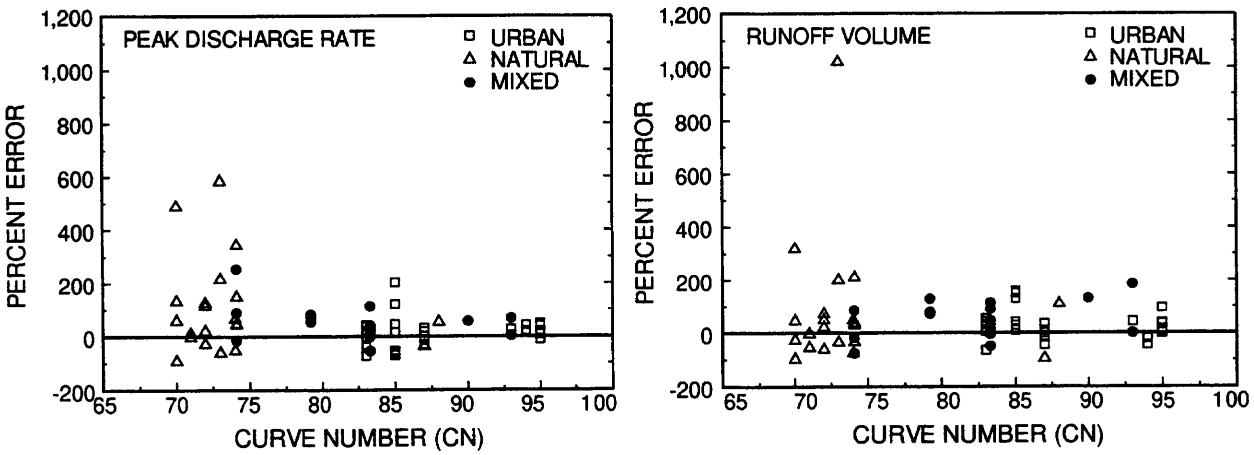


Figure 25. Comparison of the percent error between observed and estimated peak discharge rates and runoff volumes with the average watershed curve number using the NRCS TR-20 model.

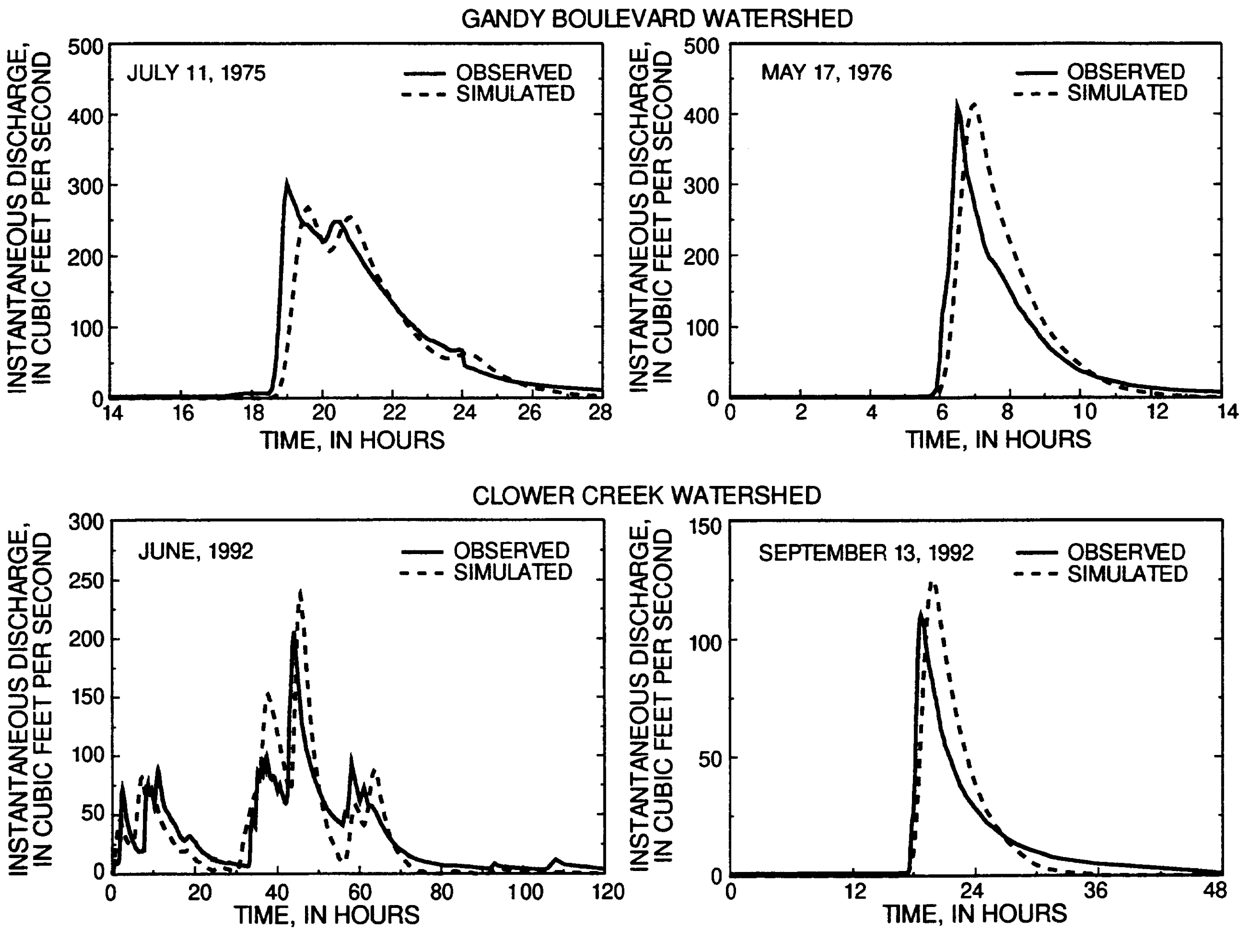


Figure 26. Typical observed hydrographs for storms occurring in the Gandy Boulevard and Clower Creek watersheds and corresponding hydrographs simulated using the NRCS TR-20 model.

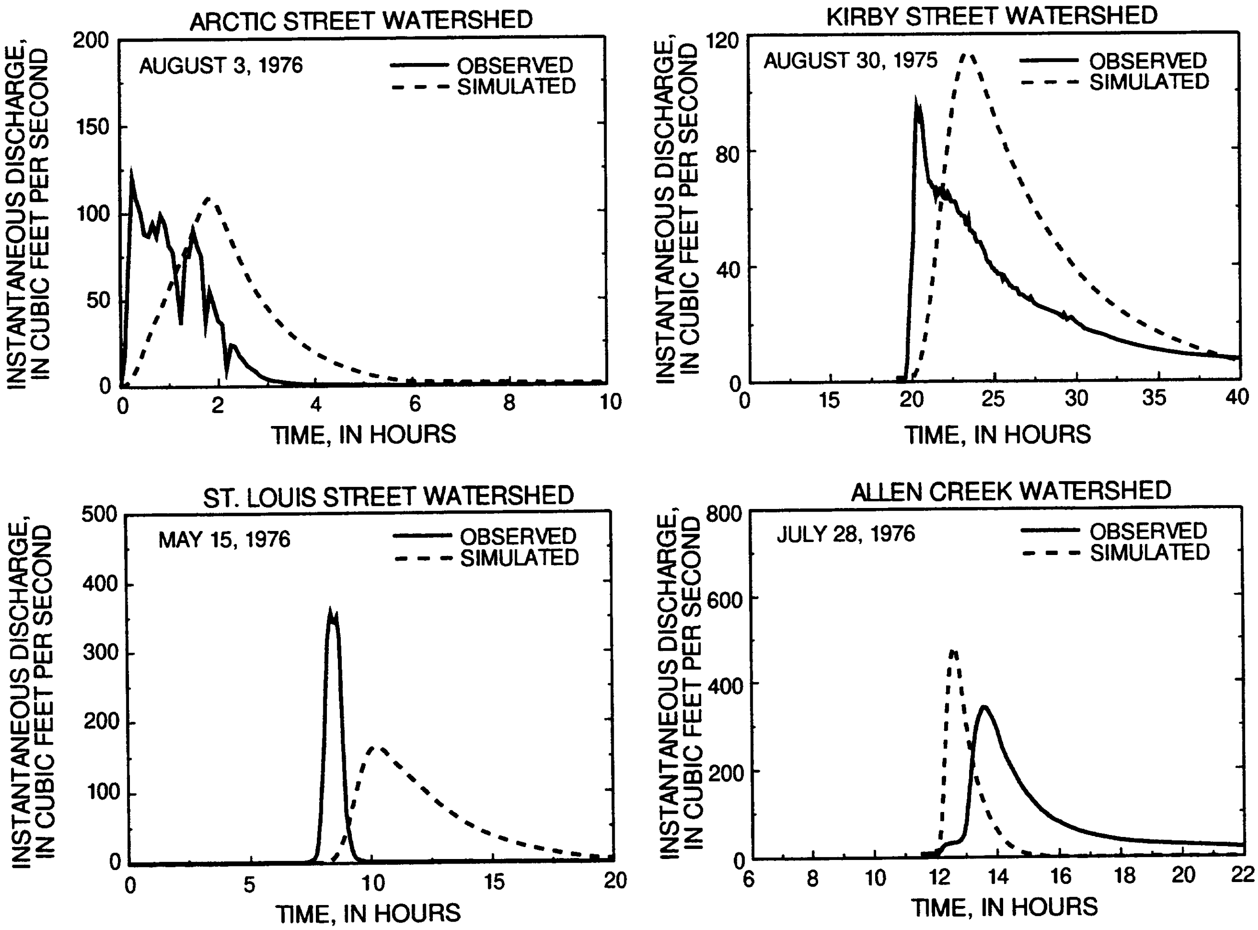


Figure 27. Typical observed hydrographs for storms occurring in the Arctic Street, Kirby Street, St. Louis Street, and Allen Creek watersheds, and corresponding hydrographs simulated using the NRCS TR-20 model.

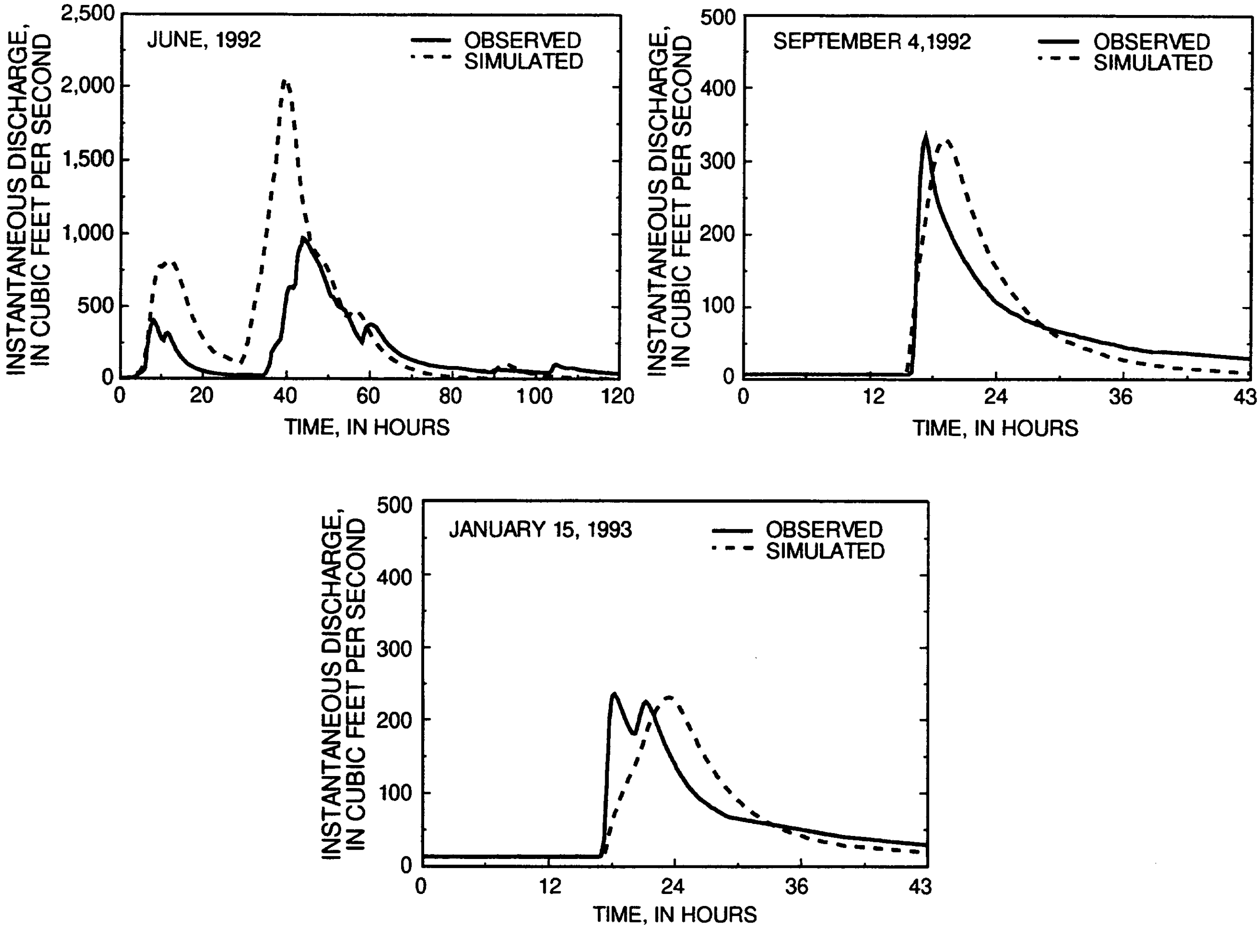


Figure 28. Typical observed hydrographs for storms occurring in the Walker Creek watershed and corresponding hydrographs simulated using the NRCS TR-20 model.

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

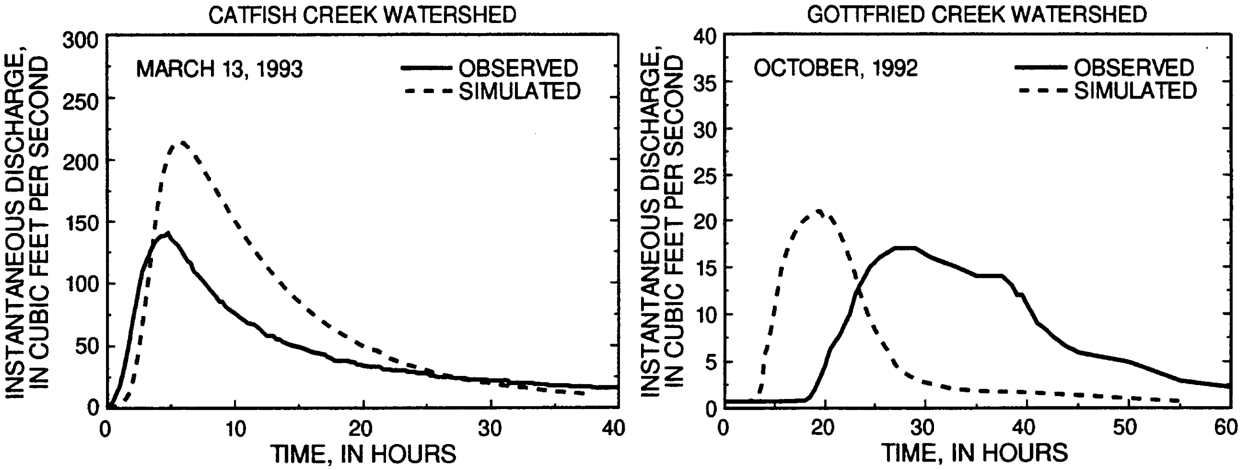


Figure 29. Typical observed hydrographs for storms occurring in the Catfish Creek and Gottfried Creek watersheds and corresponding hydrographs simulated using the NRCS TR-20 model.

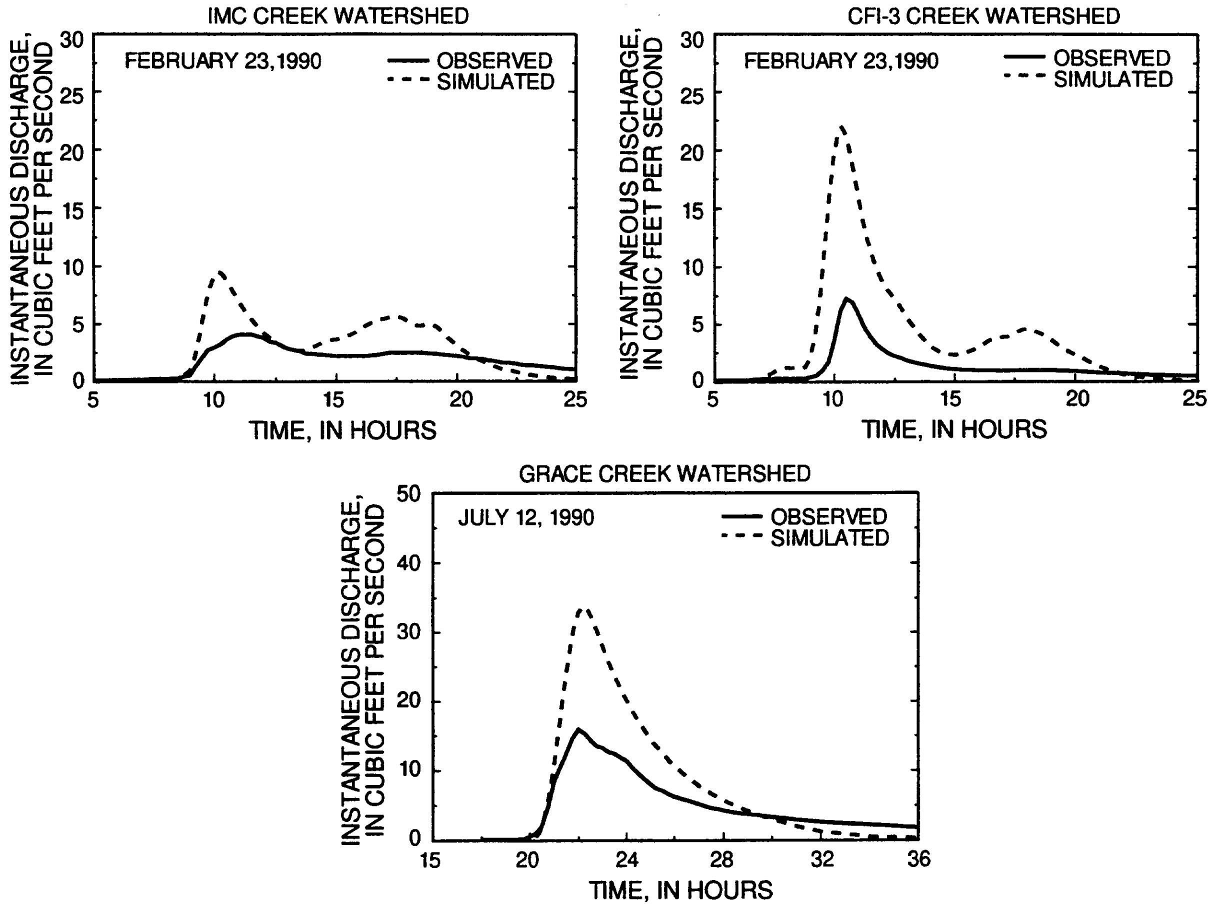


Figure 30. Typical observed hydrographs for storms occurring in the IMC, CFI-3, and Grace Creek watersheds and corresponding hydrographs simulated using the NRCS TR-20 model.

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).