EFFECTS OF URBANIZATION ON STORM-RUNOFF VOLUME AND PEAK DISCHARGE OF VALLEY CREEK, EASTERN CHESTER COUNTY, PENNSYLVANIA

By Ronald A. Sloto

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CONVERSION FACTORS AND ABBREVIATIONS

For the convenience of readers who may prefer to use metric (International System) units rather than the inch-pound units used in this report, values may be converted by using the following factors:

Multiply inch-pound units	By	To obtain metric units
	Length	
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.6093	kilometer (km)
	Area	
square mile (mi^2)	2.59	square kilometer (km^2)
	Flow	
cubic foot per second (ft^3/s)	28.32	liter per second (L/s)
	0.02832	cubic meter per second (m^3/s)

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 both the United States and Canada, formerly called "Mean Sea Level of 1929."

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ABSTRACT

Peak discharge and runoff volume were simulated for 21 storms in the Valley Creek basin using the U.S. Geological Survey Distributed Routing Rainfall-Runoff Model (DR3M). Storm peak discharges ranged from 301 to 900 cubic feet per second. Rainfall was measured at three recording rain gages in the basin. Observed and simulated runoff volumes and peak discharges were compared for the upper 20.8 square miles of the basin. The average error for runoff volume was 29 percent. The average error for peak discharge was 19 percent for the 11 calibration storms and 32 percent for the 10 verification storms. Streamflow was routed to the Schuylkill River for the lower 2.6 square miles of the basin. Simulations were made to determine the effect on runoff volume and peak discharge of increasing impervious area from 9 percent to 15, 20, and 25 percent in the part of the basin most likely to be developed. For 25 percent impervious area, runoff volume would increase an average of 52 percent and peak discharge would increase an average of 55 percent for Valley Creek at the Pennsylvania Turnpike bridge. At the confluence of Valley Creek with the Schuylkill River, runoff volume would increase an average of 46 percent and peak discharge would increase an average of 50 percent.

INTRODUCTION

The Valley Creek basin in eastern Chester County, Pennsylvania, is one of the fastest growing areas in the county. This formerly agricultural area is rapidly changing to an area of high-density residential developments and industrial parks. Large tracts of farmland and woodland are being transformed into extensive areas of impervious roofs, roads, and parking lots, particularly in corporate and industrial parks. Generally, when an area undergoes urbanization, the runoff volume and peak discharge of streams increase.

Purpose and Scope

The purpose of this study is to determine the effect of urbanization on the runoff volume and peak discharge of Valley Creek. For the study, a rainfall-runoff model of the Valley Creek basin was developed to simulate floods in the basin. The calibrated model was then used to simulate the effects of increased impervious area. This study was done by the U.S. Geological Survey in cooperation with the Chester County Water Resources Authority.

This report describes the calibration and verification of a rainfall-runoff model of the Valley Creek basin using 21 storms that occurred from March 1983 to September 1985, which produced peak discharges from 301 to 900 ft3/s (cubic feet per second). It presents the results of simulations with projected increased impervious area for six of those storms to show the effect of increasing impervious area on runoff volume and peak discharge.

Description of the Valley Creek Basin

The Valley Creek basin is almost entirely in eastern Chester County in southeastern Pennsylvania with only a small part of the basin in Montgomery County (fig. 1). Valley Creek, a tributary to the Schuylkill River, drains 23.4 mi^2 (square miles).

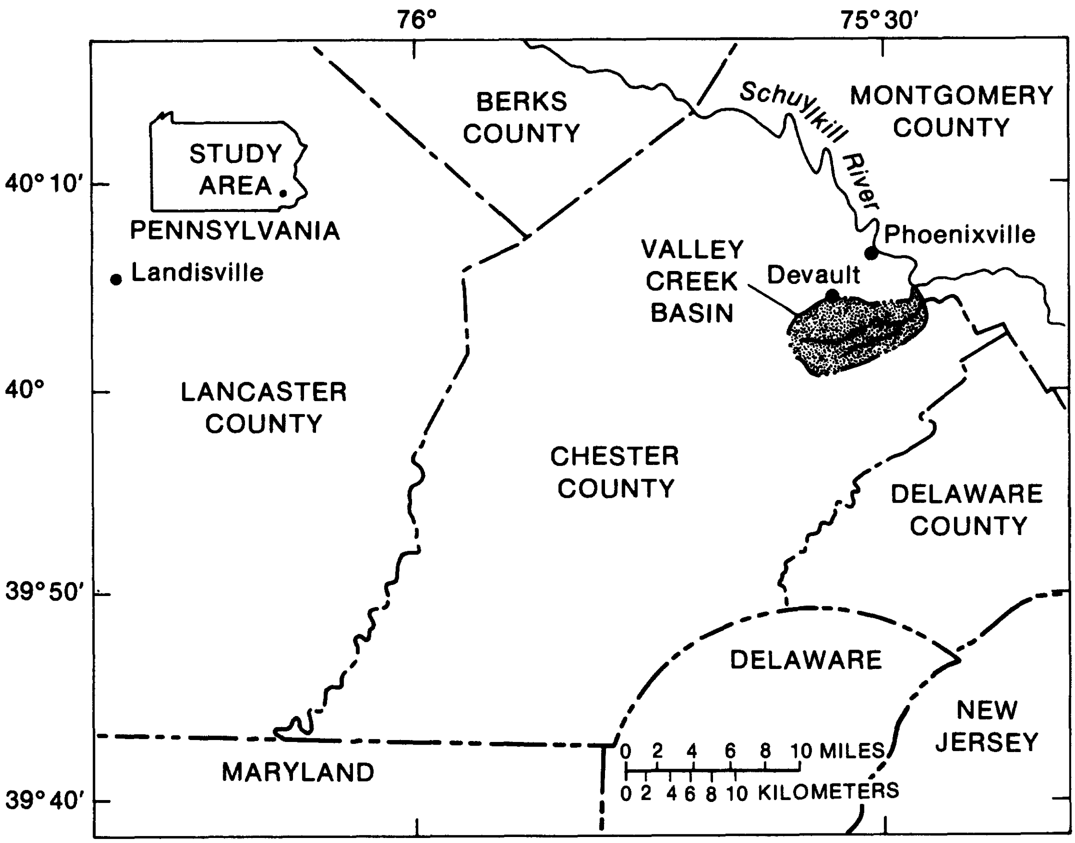


Figure 1.-- Location of Valley Creek basin.

The Valley Creek basin is in the Piedmont physiographic province. The basin occupies part of a carbonate valley, known as Chester Valley, that trends northeast across the Valley Creek basin. Chester Valley is underlain by easily erodable limestone and dolomite; it is bounded on the north and south by hills formed of more resistant crystalline rocks. The highest elevation in the basin, 668 feet above sea level, is on the northern drainage divide. The lowest elevation in the basin, 75 feet above sea level, is where Valley Creek enters the Schuylkill River.

The area has a modified humid continental climate. The normal annual temperature recorded at Phoenixville (fig. 1), a National Oceanic and Atmospheric Administration (NOAA) station, for 1951-80 is 53.1°F (11.7°C). The normal temperature for January, the coldest month is 30°F (-1.1°C). The normal temperature for July, the warmest month, is 74.9°F (23.8°C). The normal annual precipitation for 1951-80 recorded at Devault (fig. 1), a daily NOAA precipitation station in the Valley Creek basin, is 46.92 inches. Precipitation is about evenly distributed throughout the year with slightly less occurring in February, and slightly more occurring in July. Prevailing westerly winds carry most of the weather disturbances that affect the area, except for coastal storms, from the interior of the United States. Much of the summer precipitation comes from thunderstorms that produce brief periods of high-intensity rainfall. For example, a thunderstorm on August 14, 1985, produced 1.54 inches of rainfall in 45 minutes.

The Valley Creek basin lies mostly within East Whiteland and Tredyffrin Townships. These townships are undergoing rapid urbanization with land use changing from agricultural to residential and commercial. From 1950-80, population increased 387 percent in East Whiteland Township and 194 percent in Tredyffrin Township (table 1).

From 1970-80 in East Whiteland Township, acreage devoted to residential use increased 26 percent, acreage devoted to industrial use increased 34 percent, and acreage devoted to commercial use increased 12 percent; during this same period, acreage devoted to agriculture and open space decreased 16 percent. From 1970-80 in Tredyffrin Township, acreage devoted to residential use increased 16 percent, acreage devoted to industrial use increased 91 percent, and acreage devoted to commercial use increased 25 percent; during this same period, acreage devoted to agriculture and open space decreased by 23 percent (Delaware Valley Regional Planning Commission, 1984).

Table 1.--Population (1950-80) and land use (1970-80) in East Whiteland and Tredyffrin Townships

Municipality	Population1/			
	1950	1960	1970	1980
East Whiteland	1,740	5,078	7,242	8,468
Tredyffrin	7,836	16,004	23,404	23,019

	Land use, in acres2/							
	Residential		Industrial		Commercial		Agricultural and Open Space	
Municipality	1970	1980	1970	1980	1970	1980	1970	1980
East Whiteland	1,129	1,422	396	505	1,723	1,932	3,770	3,165
Tredyffrin	4,228	4,917	100	191	2,404	2,994	6,009	4,641

1/ From the Chester County Planning Commission (written commun., 1985)

2/ From the Delaware Valley Regional Planning Commission (1984)

Data-Collection Sites

Data required for storm simulation included precipitation and pan-evaporation data for model input and streamflow data for model calibration.

The Valley Creek basin is an elongated, almost rectangular, basin. The basin was divided into thirds, and a recording raingage was installed near the center of each third (fig. 2). The gages recorded rainfall at a 15-minute interval. Both daily and unit (15-minute) precipitation was required for modeling. The Mill Lane raingage was used as the daily precipitation station because it had the most complete record. Unit data from all three raingages were used for storm simulations.

Daily pan-evaporation data from the NOAA station in Landisville (fig. 1) was used for model input. The Landisville station is the nearest NOAA pan-evaporation station in the Piedmont physiographic province.

Streamflow was measured at the continuous-record station Valley Creek at Pennsylvania Turnpike bridge near Valley Forge (station number 01473169, see fig. 2). Streamflow from the upper 20.8 mi^2 of the basin is measured at this station at a 15-minute interval. The lower 2.6 mi^2 of the basin is ungaged.

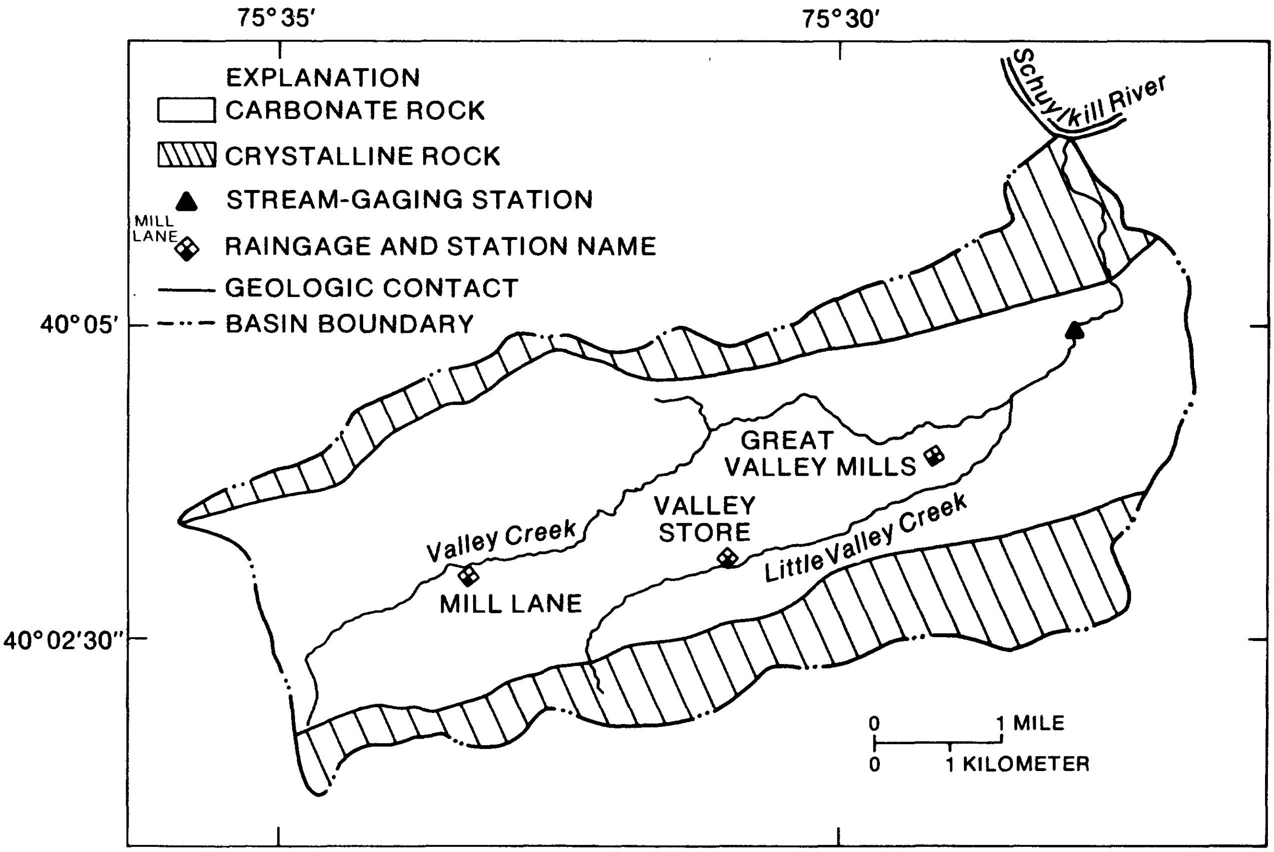


Figure 2.-- Location of data-collection sites.

Acknowledgments

The cooperation of property owners who allowed access to their property for stream channel geometry measurements is gratefully acknowledged. The author especially thanks Todd Sealman, Clarence S. Statts, Jr., East Whiteland Township, and the University of Pennsylvania for allowing the installation of raingages on their property.

EFFECTS OF URBANIZATION ON RUNOFF VOLUME AND PEAK DISCHARGE

Description of Rainfall-Runoff Model Program

The effects of urbanization on runoff volume and peak discharge was simulated by use of version II of the U.S. Geological Survey Distributed Routing Rainfall-Runoff Model (Alley and Smith, 1982), called DR3M. This computer program was used to simulate storm discharge hydrographs in the Valley Creek basin. DR3M is a deterministic, distributed-parameter model that uses many physically-based parameters, the values of which are measured in the field. DR3M combines rainfall-excess components with kinematic-wave routing. Daily and unit rainfall, and daily pan evaporation are used to compute a simulated discharge hydrograph.

Rainfall-Excess Components

The rainfall-excess components in DR3M include soil-moisture accounting, pervious-area rainfall excess, impervious-area rainfall excess, and parameter optimization.

Soil-moisture accounting

Soil-moisture and infiltration parameters are listed in table 2. The soil-moisture-accounting component measures the effect of antecedent conditions on infiltration. DR3M simulates moisture redistribution in the soil column and evapotranspiration from the soil. Soil moisture is modeled as a two-layered system. During periods between simulated storms, a part of the daily rainfall, determined by the coefficient RR, infiltrates into the upper soil-moisture zone and becomes soil-moisture storage (SMS). Evapotranspiration takes place from SMS, or from the lower soil-moisture zone, base-moisture storage (BMS), when SMS = 0. The evapotranspiration rate is determined by multiplying daily pan evaporation by a pan coefficient, EVC. Moisture from SMS drains into BMS during periods of no rainfall at a rate based on the effective hydraulic conductivity (KSAT). Storage in BMS has a maximum value, BMSN, which is equivalent to field capacity. When BMSN is exceeded, the excess moisture is assumed to enter the ground-water system.

Table 2.--Description of soil-moisture and infiltration parameters in DR3M

Parameter	Units	Description
BMSN	inches	Maximum effective soil-moisture-storage volume at field capacity
EVC	-	Coefficient that converts pan evaporation to potential evapotranspiration
KSAT	inches per hour	Effective saturated hydraulic conductivity
PSP	inches of pressure	Suction at the wetting front for soil moisture at field capacity
RGF	-	Ratio of suction at the wetting front for soil moisture at the wilting point to that at field capacity
RR	-	Proportion of daily rainfall that infiltrates into the soil for the period of simulation excluding unit days

Pervious-area rainfall excess

Point-potential infiltration is computed by a variation of the Green and Ampt (1911) equation. During a simulated storm, moisture is added to SMS based on:

$$FR = KSAT \left(1 + \frac{PS}{SMS}\right), \tag{1}$$

where FR = point-potential infiltration,
KSAT = the effective saturated-soil hydraulic conductivity, and
PS = average suction head across the wetting front.

PS is varied over the range from wilting point to field capacity by:

$$PS = PSP \left[RGF - (RGF-1) \frac{BMS}{BMSN}\right], \tag{2}$$

where PSP = effective value of PS at field capacity, and
RGF = ratio of PS at wilting point to that at field capacity.

Point-potential infiltration is converted to effective infiltration over the basin using a method presented by Crawford and Linsley (1966). The rate of generation of excess rainfall that does not infiltrate is computed by:

$$QR = \frac{SR^2}{2FR} \text{ ; if } SR \leq FR \tag{3}$$

$$QR = SR - \frac{FR}{2} \text{ ; if } SR > FR \text{ ,} \tag{4}$$

where QR = the rate of generation of rainfall excess, and
SR = the supply value of rainfall for infiltration.

Impervious-area rainfall excess

Two types of impervious surfaces can be simulated. The first type--effective impervious surfaces--are those impervious areas that are directly connected to the channel drainage system. A roof that drains onto a driveway, street, or paved parking lot that drains to a stream channel is an example of an effective impervious surface. The second type--noneffective impervious surfaces--are those impervious areas that drain to pervious areas. A roof that drains onto a lawn is an example of a noneffective impervious area.

Rain falling on noneffective impervious areas is assumed to run off onto the surrounding pervious area. In DR3M, this occurs instantaneously and the volume of runoff is uniformly distributed over the pervious area. This volume is added to the rain falling on the pervious areas prior to computation of pervious-area rainfall excess.

Parameter optimization

DR3M includes a component to optimize the soil-moisture and infiltration parameters to produce the closest match between the observed and simulated runoff volume for selected storms. This optimization procedure, an automatic fitting process that proceeds by stages, was developed by Rosenbrock (1960). An objective function, the sum of the squared deviations of the logarithms of observed and simulated storm runoff volumes, was calculated and used to evaluate the fit between observed and simulated runoff volumes.

Routing Component

The Valley Creek basin is represented by a combination of overland flow and channel segments that are described by a set of parameters. Overland flow segments receive uniformly distributed lateral inflow from excess rainfall. Channel segments receive lateral inflow from overland flow segments and upstream inflow from other segments.

Input data needed to define flow-routing parameters were obtained from field measurements, aerial photographs, and topographic quadrangle maps. Routing parameters include segment length, slope, roughness, and one or two special parameters discussed below.

Channel-segment length and slope were obtained from topographic quadrangle maps. The roughness parameter, similar to Manning's n, was estimated in the field. Special parameters for bridge openings and channel cross sections were measured in the field.

Overland flow segment length was computed by dividing the area that contributes runoff by the length of stream that drains the contributing area. Contributing areas were planimetered. Stream lengths and overland flow segment slopes were taken from topographic maps. The roughness parameter, an empirical coefficient for overland flow, was estimated. The percentage of impervious, pervious, and effective impervious areas were calculated from field measurements, aerial photographs, and topographic quadrangle maps.

DR3M routes excess rainfall for both overland flow and channel segments by applying kinematic-wave theory. The kinematic-wave model is one of a number of approximations of the dynamic-wave model. The dynamic-wave model describes one-dimensional shallow-water waves (unsteady, gradually varied, open-channel flow) and consists of the continuity equation and the equation of motion with appropriately described initial and boundary conditions. The continuity equation results from an expression of the principle of conservation of mass and is written as:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \ , \qquad (5)$$

where A = area of flow,
Q = rate of flow,
t = time,
x = distance along a segment increasing in the downstream direction, and
q = rate of lateral inflow.

In the kinematic wave approximation, the water surface slope and acceleration terms in the equation of motion are assumed to be insignificant and the equation of motion simply states that the friction slope is equal to the bed slope. By defining the friction slope with an appropriate flow resistance relationship, such as the Manning formula for turbulent flow, the equation of motion can be expressed as:

$$Q = \alpha A^m \ , \qquad (6)$$

where α and m are constants that are determined from the geometry, slope, and roughness of a channel or overland-flow plane.

The kinematic-wave equations are solved using an explicit finite-difference method. In this method, each model segment is subdivided into distance intervals. A distance interval, Δx, and a time interval, Δt, form a four-point finite-difference mesh. A and Q are solved at one point, given A and Q at the other three points. A detailed discussion of the finite-difference solution used in DR3M is given by Alley and Smith (1982, p. 12-15).

Model Calibration and Verification

Storm discharge hydrographs for the Valley Creek basin were simulated for the upper and lower parts of the basin. The upper part of the basin, above the gaging station, was calibrated and verified by comparing simulated and observed runoff volume, peak discharge, and discharge hydrographs of Valley Creek at the gaging station. Streamflow in the lower part of the basin was routed from the gaging station to the Schuylkill River. Discharge observed or simulated at the gaging station was input to the model using an input-hydrograph point corresponding to Valley Creek at the gaging station.

Basin Discretization

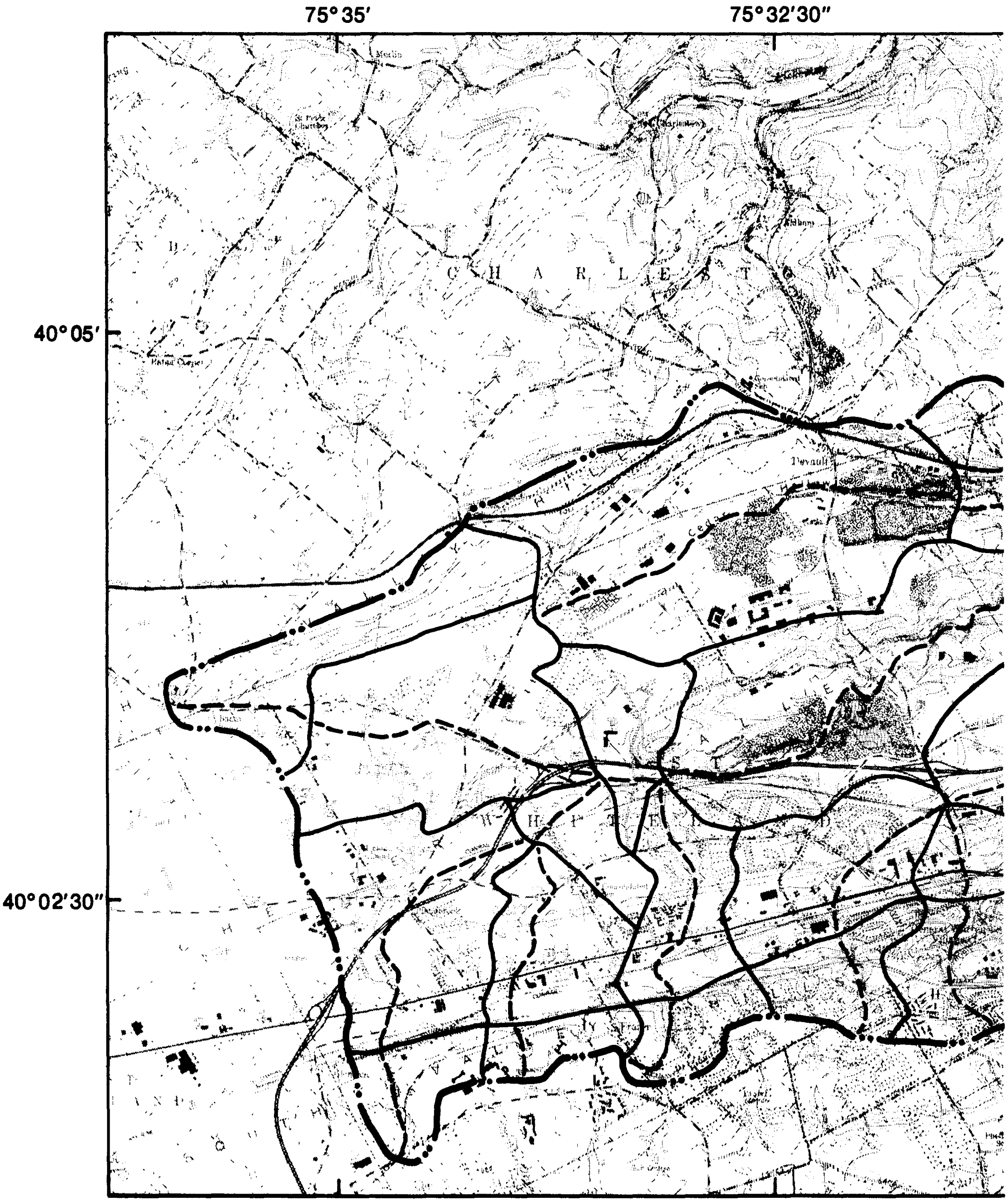
The Valley Creek basin was discretized into 119 segments for modeling (fig. 3). Above the gaging station, 99 segments were used to describe the basin: 54 overland flow segments, 44 channel segments, and one input-discharge point used to input discharge from the Cedar Hollow quarry (fig. 3). Quarry discharge was assumed to be a constant 8 ft^3/s, the average discharge measured by a weir maintained by the Warner Company (Whitcomb, T., Pennsylvania Department of Environmental Resources, written commun., 1985). Below the gaging station, 20 segments were used to describe the basin: 11 overland flow segments, eight channel segments, and one input-hydrograph point used to input discharge observed or simulated at the gaging station.

Each subbasin or stream reach between tributaries and the associated contributing area was discretized into one channel and two overland-flow segments. If the two overland-flow segments were of approximately equal size and physical character, they were averaged and described by one segment that was used twice. If a tributary crossed the contact between crystalline and carbonate rock, the tributary and the contributing area was discretized into six segments: two overland-flow segments representing contributing areas underlain by crystalline rock and the channel segment draining them, and two overland-flow segments representing contributing areas underlain by carbonate rock and the channel segment draining them. By using this method, measured values for channel geometry and roughness could be assigned to each stream reach, and the size and slope of contributing areas could be defined more exactly than if larger areas were used. This method also allowed an exact division between areas underlain by crystalline and carbonate rock.

Storms used for Modeling

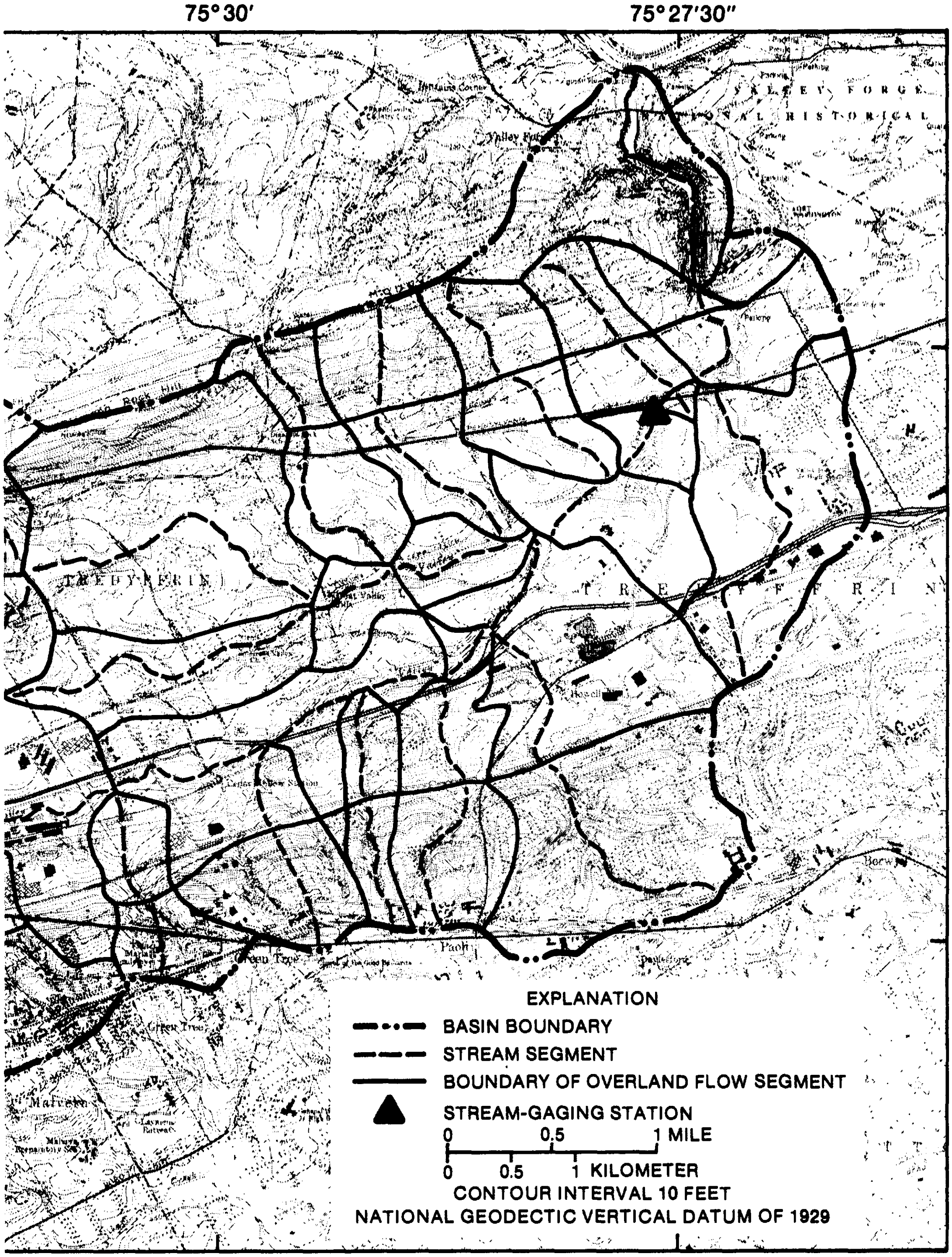
Only non-winter storms were used for modeling. DR3M contains no provisions for handling snowfall, snowmelt, or frozen ground. The raingages were not heated and could not measure snowfall or its water equivalent. Storms occurring between mid-November and mid-March were used for modeling only if the precipitation was rainfall and the ground was not frozen.

Twenty-one storms producing a peak discharge greater than the base peak of 300 ft^3/s were available for modeling. The mean annual flood is approximately 300 ft^3/s. Only storms producing a peak discharge greater than 300 ft^3/s were used for modeling because those are likely to cause property damage by flooding. These storms occurred between March 1983 and September 1985. The storm dates,



Base from U.S. Geological Survey Malvern 1:24,000, 1973, and Valley Forge 1:24,000, 1981

Figure 3.-- Discretization of Valley Creek basin into segments for modeling.


75°30′
75°27′30″
EXPLANATION
BASIN BOUNDARY
STREAM SEGMENT
BOUNDARY OF OVERLAND FLOW SEGMENT
STREAM-GAGING STATION
0 0.5 1 MILE
0 0.5 1 KILOMETER
CONTOUR INTERVAL 10 FEET
NATIONAL GEODECTIC VERTICAL DATUM OF 1929

rainfall amounts, and observed peak discharges are given in table 3. The relation between rainfall (average of the three recording stations) and observed peak discharge is shown on figure 4. The same amount of total rainfall can produce a wide range of peak discharges. Peak discharge depends primarily on the duration and intensity of rainfall, and on antecedent moisture conditions. High-intensity rainfall on saturated soil will produce the highest peak discharge.

If unit data from a raingage were missing for a modeled storm, the storm was simulated using unit rainfall substituted from one of the other raingages. The data from the raingage that produced the simulated runoff volume closest to the observed runoff volume were substituted.

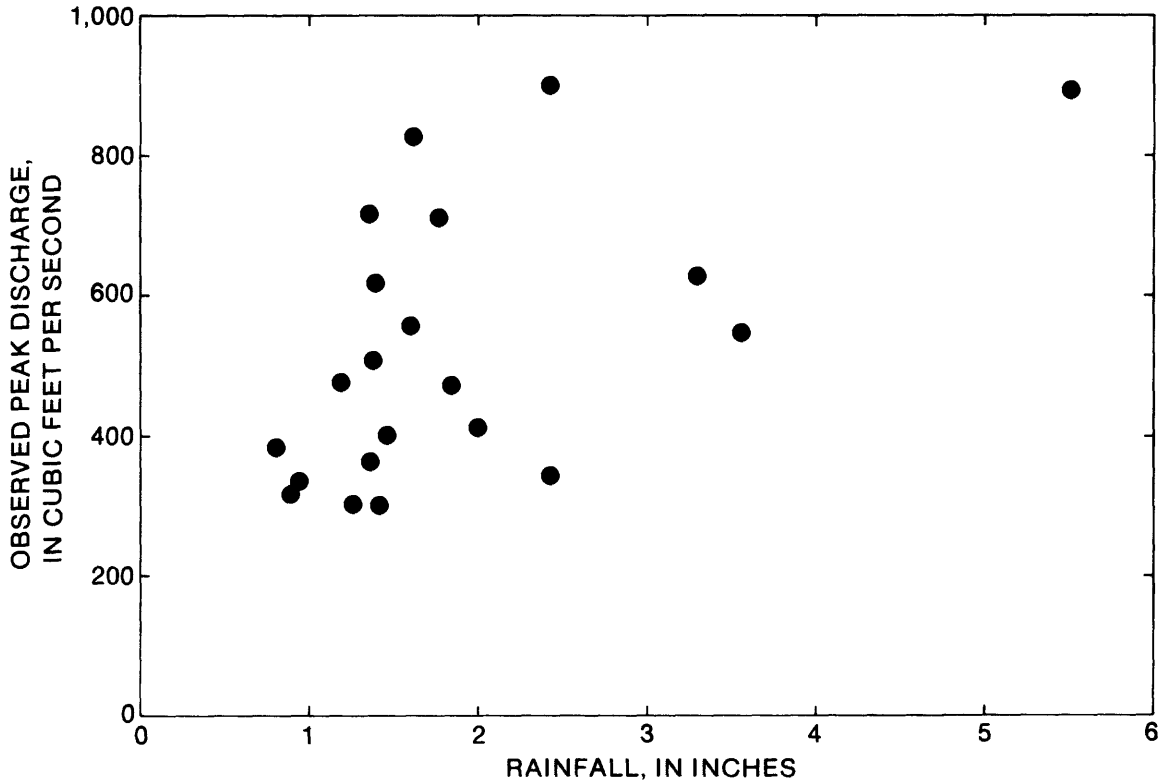


Figure 4.-- Relation between rainfall and peak discharge.

Calibration of Runoff Volumes

The soil-moisture and infiltration parameters were optimized to provide the best fit between observed and simulated runoff volumes. The optimization procedure, which calculates an objective function, in DR3M was used (Alley and Smith, 1982, p. 7).

Two sets of soil-moisture and infiltration parameters were used, one for soils derived from carbonate rock, and one for soils derived from crystalline rock (fig. 2). The initial soil-moisture and infiltration parameter values were set to those used by Sloto (1982, p. 12) for soils underlain by crystalline rock in northern Chester County. For soils underlain by carbonate rock, the upper

Table 3.--Storms used for modeling
[ft^3/s, cubic feet per second]

Storm number	Storm date	Peak discharge (ft^3/s)	Rainfall (inches)		
			Mill Lane	Valley Store	Great Valley Mills
1	March 18-19, 1983	365	1.54	1.27	1/
2	March 21, 1983	826	1.51	1.68	1/
3	March 27-28, 1983	711	1.80	1.62	1.89
4	April 9-10, 1983	617	1/	1.39	1/
5	May 21-22, 1983	302	1.22	1.26	1.29
6	June 19-21, 1983	343	1.88	2.17	3.24
7	December 12-13, 1983	627	3.06	2/	3.79
8	December 22, 1983	556	1.56	2/	1.69
9	December 28, 1983	383	.78	2/	.88
10	April 4-5, 1984	900	2.26	2.34	2.70
11	May 3-4, 1984	413	1.86	2.25	1.89
12	May 28-30, 1984	547	3.77	3.65	3.27
13	June 24-25, 1984	401	1.62	1.34	1.42
14	July 1, 1984	317	1.23	3/	.72
15	July 7, 1984	477	1.53	3/	1.02
16	July 27, 1984	508	1.41	2/	1.31
17	November 5, 1984	335	1.07	3/	.88
18	July 31, 1985	301	1.33	1.47	2/
19	August 8, 1985	472	1.39	2.06	2/
20	August 14, 1985	716	.98	1.55	2/
21	September 24-25, 1985	894	5.81	5.88	4.86

1/ Data missing. Unit data for Valley Store substituted.
2/ Data missing. Unit data for Mill Lane substitued.
3/ Data missing. Unit data for Great Valley Mills substituted.

and lower bounds for PSP, RGF, BMSN, and RR given by Alley and Smith (1982, p. 19) were used. The range for KSAT, 0.05 to 0.5 inches per hour, was based on the soil map of Chester County (Kunkle, 1963) and the U.S. Soil Conservation Service (1972) hydrologic soil group designations.

Initial soil-moisture and infiltration parameter values were estimated for soils underlain by carbonate rock. Several calibration runs were made, with parameter values being revised each time. Initially, all 21 storms were used for the optimization. After the first calibration run, storms 5 and 10 were not included in the calculation of the objective function. These storms were the major contributors to a high objective function. Simulated runoff volumes for these storms were extremely low. Calibration continued until the best possible match between observed and simulated runoff volumes was obtained. The soil-moisture and infiltration parameters for soils underlain by carbonate rock were then set and a few runs were made to calibrate soil-moisture and infiltration parameters for soils underlain by crystalline rock.

Final values for soil-moisture and infiltration parameters are given in table 4. A comparison of observed and simulated runoff volumes and the error is given in table 5. The average error for simulated runoff volumes was 29 percent. A generally accepted error criteria for simulated runoff volume for individual storms is within 50 percent if the simulated volume is less than the observed and within 100 percent if the simulated volume is greater than the observed (Doyle and Miller, 1980, p. 18; Shade, 1984, p. 12). Only storm 5 failed to meet these criteria. Observed and simulated runoff volumes are compared on figure 5.

Table 4.--Values of soil-moisture and infiltration parameters used in the model

Parameter	Value		Units
	Carbonate-rock derived soils	Crystalline-rock derived soils	
BMSN	3.56	3.23	inches
EVC	.89	.74	
KSAT	.18	.13	inches per hour
PSP	4.08	1.53	inches
RGF	21.96	21.51	
RR	.90	.89	

The relation between rainfall and observed runoff volume is shown on figure 6. A direct relation between rainfall and runoff volume for individual storms does not exist as the runoff volume depends primarily on duration and intensity of rainfall and antecedent moisture conditions. The same is true of the relation between rainfall and peak discharge (fig. 4).

Likewise, similar runoff volumes from different storms often produce a wide range in peak discharge (fig. 7). Peak discharge is a function of the distribution of runoff volume. For example, a storm producing 1 inch of runoff in 1 hour will produce a greater peak discharge than a storm producing 1 inch of runoff evenly distributed over 24 hours.

Table 5.--Observed and simulated runoff volumes

Storm number	Storm date	Runoff volume (inches) Observed	Simulated	Percent error between simulated and observed runoff volumes
1	March 18-19, 1983	0.279	0.193	-31
2	March 21, 1983	.599	.495	-17
3	March 27-28, 1983	.635	.460	-28
4	April 9-10, 1983	.498	.297	-40
5	May 21-22, 1983	.468	.177	-62
6	June 19-21, 1983	.225	.317	+41
7	December 12-13, 1983	1.060	.919	-13
8	December 22, 1983	.313	.326	+ 4
9	December 28, 1983	.187	.127	-32
10	April 4-5, 1984	1.021	.677	-34
11	May 3-4, 1984	.394	.348	-12
12	May 28-30, 1984	.987	.777	-21
13	June 24-25, 1984	.197	.186	- 6
14	July 1, 1984	.099	.123	+24
15	July 7, 1984	.275	.229	-17
16	July 27, 1984	.250	.202	-19
17	November 5, 1984	.115	.119	+ 3
18	July 31, 1985	.101	.193	+91
19	August 8, 1985	.177	.238	+34
20	August 14, 1985	.168	.282	+68
21	September 24-25, 1985	1.125	1.152	+ 2

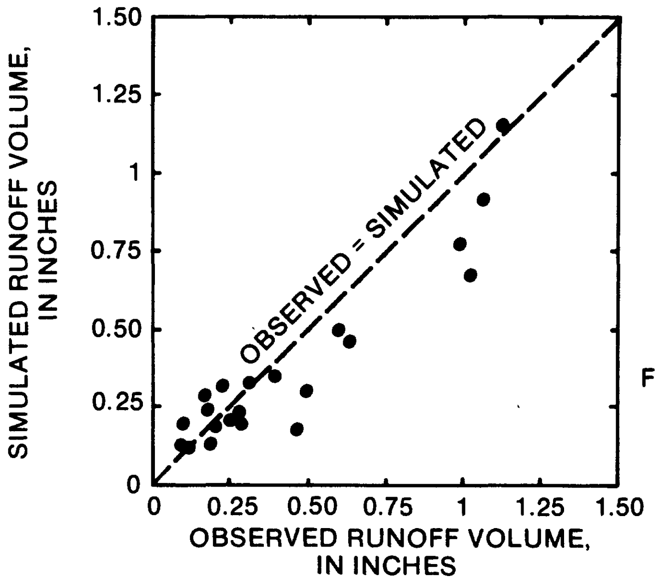


Figure 5.--Relation between observed and simulated runoff volume.

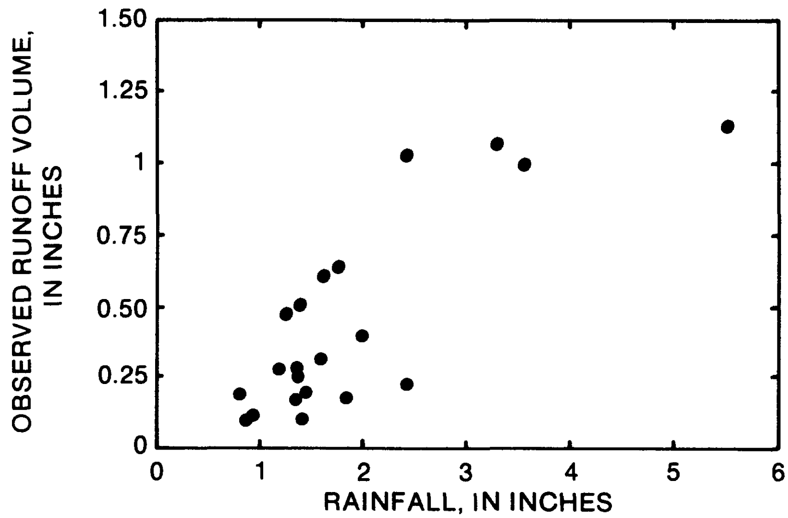


Figure 6.--Relation between rainfall and observed runoff volume.

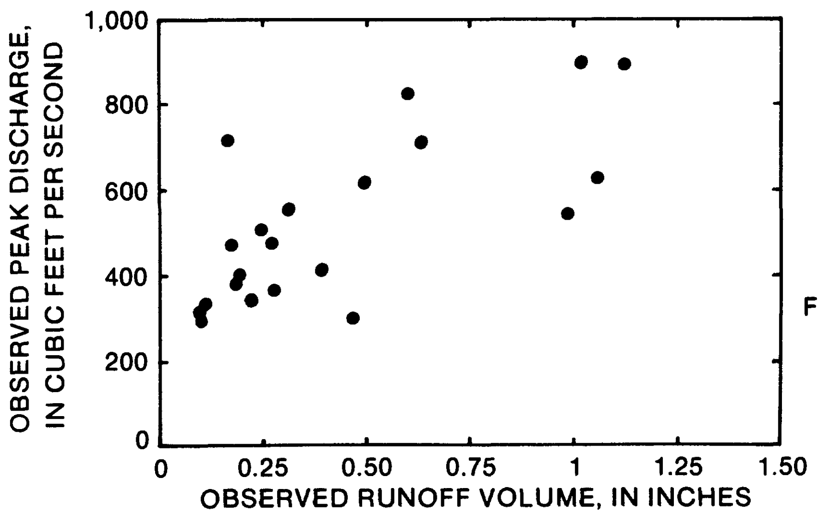


Figure 7.--Relation between observed runoff volume and peak discharge.

Calibration of peak discharge

The 21 storms selected for modeling were ranked and divided into two sets. Both sets contained storms occurring during the same period and contained a similar range of peak discharges. Eleven storms were used for calibration of peak discharge and 10 storms were used for verification of peak dishcarge. Storms used for calibration occurred between March 18, 1983, and August 14, 1985, and had peak discharges from 301 to 900 ft^3/s (table 6).

Table 6.--Storms used for calibration of peak discharge
[ft^3/s, cubic feet per second]

Storm number	Storm date	Peak discharge (ft^3/s) Observed	Simulated	Percent error
1	March 18-19, 1983	365	249	-32
2	March 21, 1983	826	878	+ 6
4	April 9-10, 1983	617	478	-23
6	June 19-21, 1983	343	290	-15
8	December 22, 1983	556	555	0
10	April 4-5, 1984	900	1,050	+17
13	June 24-25, 1984	401	248	-38
15	July 7, 1984	477	465	- 3
16	July 27, 1984	508	302	-41
18	July 31, 1985	301	337	+12
20	August 14, 1985	716	862	+20

During calibration, the α adjustment routing parameter, ALPADJ, was adjusted until the lowest average error between observed and simulated peak discharge was obtained. DR3M computes α from equation (6). The value of α contains the effects of roughness, bed slope, and cross-sectional geometry. ALPADJ is a multiplication factor for α that changes its value for every segment. The value of ALPADJ was varied from 0.2 to 1.2. A value of 0.6 produced the lowest average error. The effect of varying the value of ALPADJ on the peak discharge of storms 6, 8, and 10 is shown in figure 8. As the value of ALPADJ increases, peak discharge increases; the increase is not proportional for all storms.

Observed and simulated peak discharges are given in table 6 and compared on figure 9. The generally accepted error criteria for simulated peak discharge is within 50 percent if the simulated peak is less than the observed, and within 100 percent if the simulated peak is greater than the observed (Doyle and Miller, 1980, p. 18; Shade, 1984, p. 12). All of the simulated peak discharges met these criteria and were within 38 percent of the observed peak discharges. The average absolute error was 19 percent. Hydrographs of selected storms are shown on figure 10. The peak of record (October 1982 to September 1985) occurred on April 5, 1984. The observed peak discharge was 900 ft^3/s; the simulated peak discharge was 1,050 ft^3/s (fig. 11).

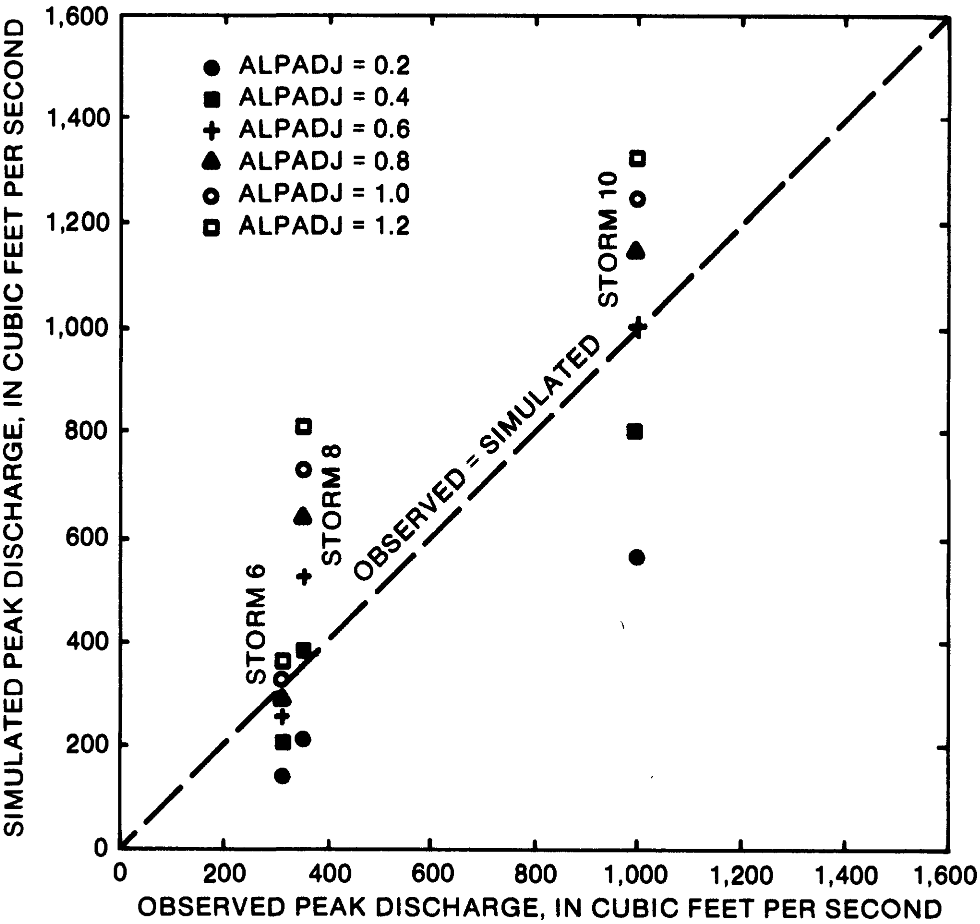


Figure 8.-- Effect of varying the α-adjustment parameter on peak discharge for selected storms.

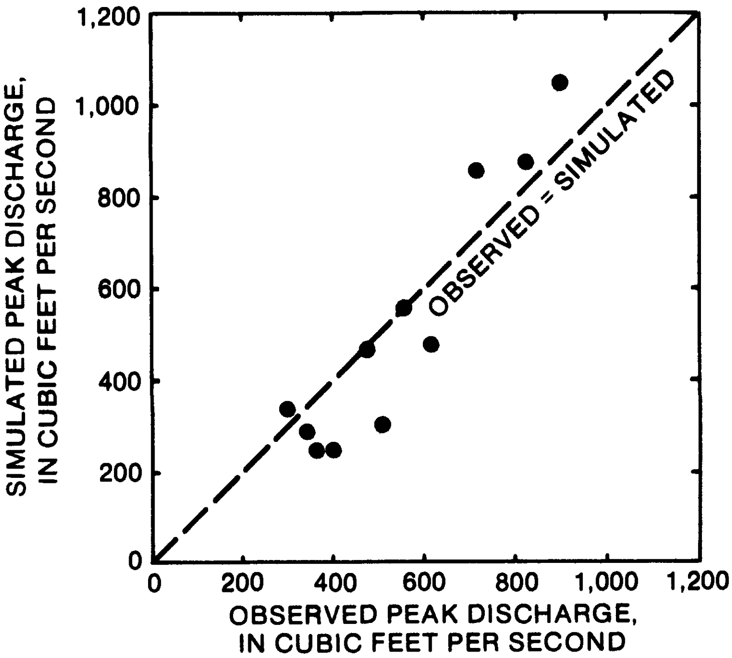
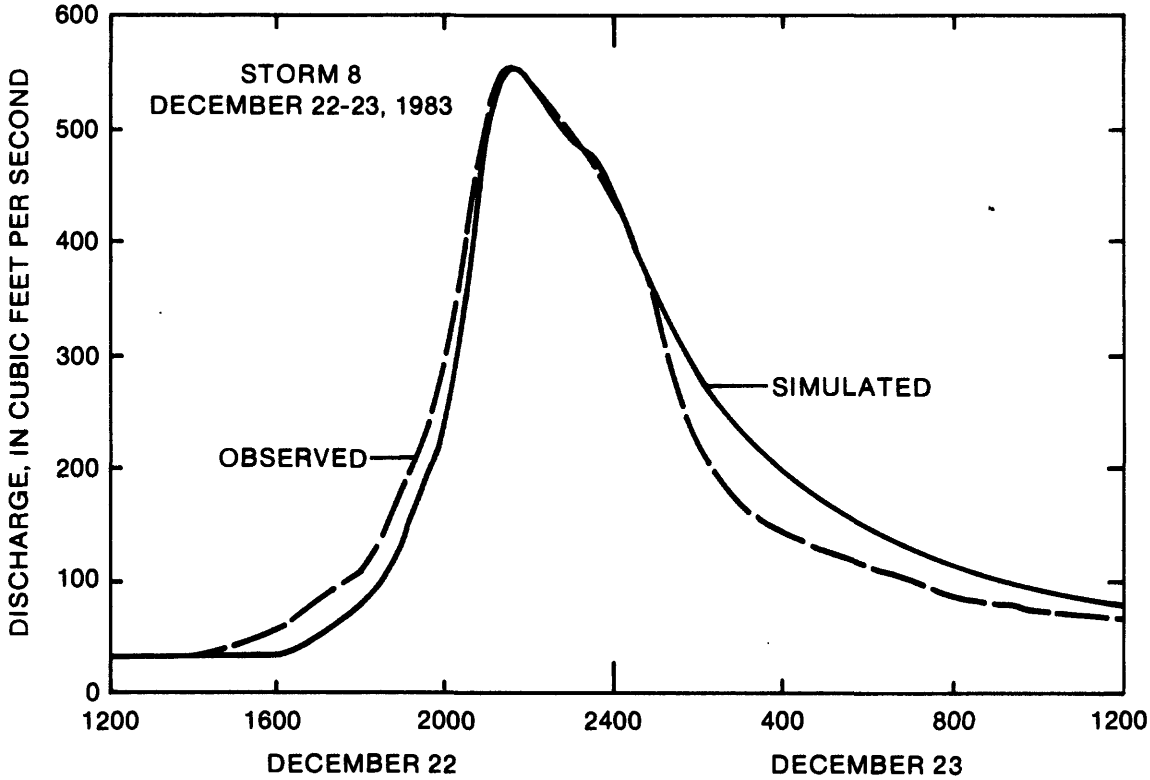


Figure 9.-- Relation between observed and simulated peak discharge for calibration storms.



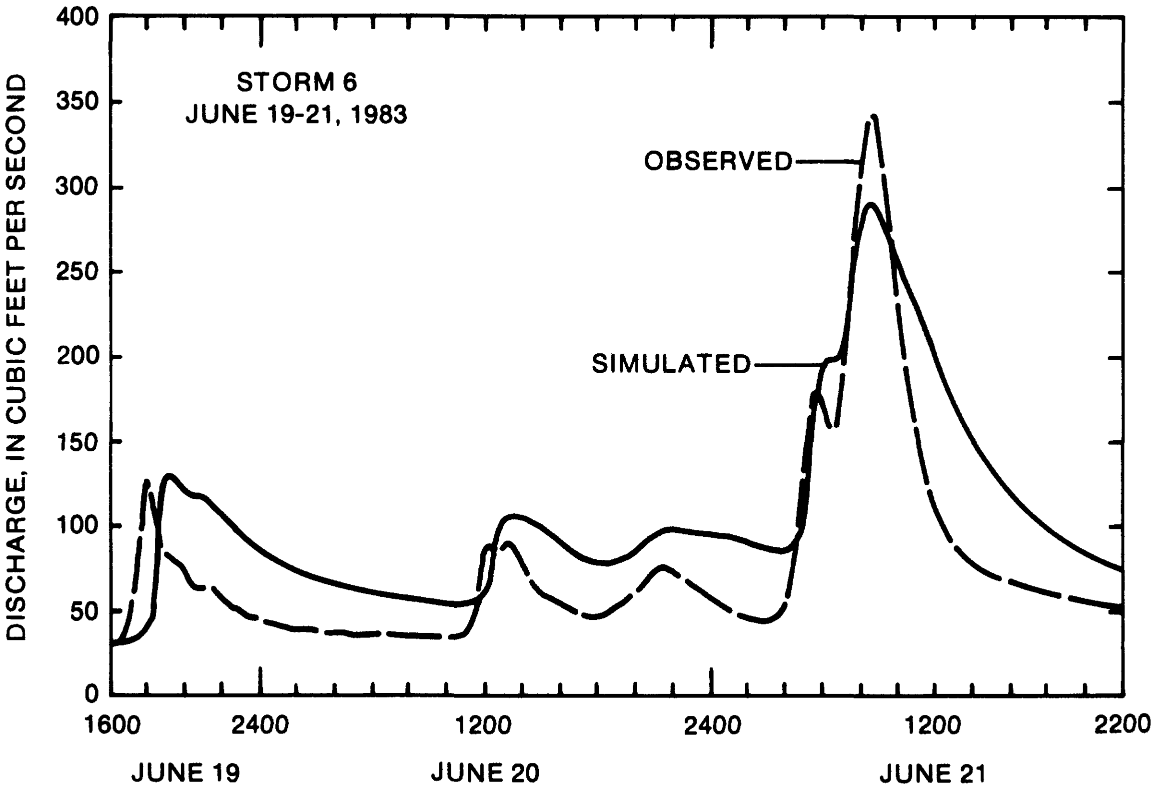


Figure 10.--Hydrographs of selected calibration storms.

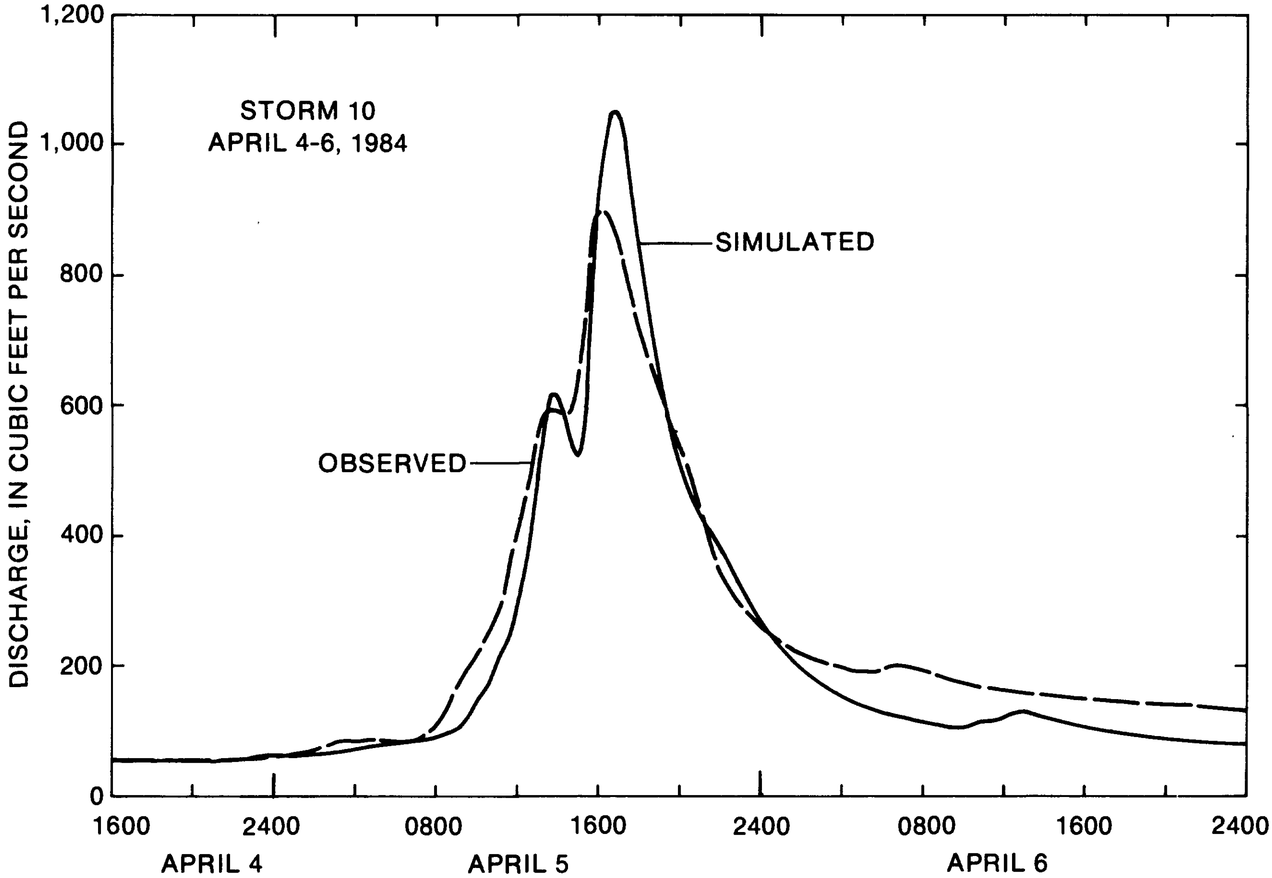


Figure 11.--Hydrograph of the April 4-6, 1985, storm.

Verification of peak discharge

Ten storms were used for verification of peak discharge (table 7). These storms occurred between March 27, 1983, and September 25, 1985, and had peak discharges from 302 to 894 ft^3/s. These storms were simulated without changing any parameter values set during model calibration. Observed and simulated peak discharges are compared on figure 12. All of the peak discharges fall within the error criteria. The average absolute error was 32 percent, which is higher than that for the calibration storms. Although the error for peak discharge was higher, the match between observed and simulated hydrograph shape was generally better. Hydrographs of selected storms are shown on figure 13. The highest peak discharge of the verification storm set, 894 ft^3/s, occurred September 26-27, 1985, and was caused by Hurricane Gloria (fig. 14). The simulated peak discharge was 1,090 ft^3/s.

Table 7.--Storms used for verification of peak discharge
[ft^3/s, cubic feet per second]

Storm number	Storm date	Peak discharge (ft^3/s)		Percent error
		Observed	Simulated	
3	March 27-28, 1983	711	957	+35
5	May 21-22, 1983	302	227	-25
7	December 12-13, 1983	627	1,210	+93
9	December 28, 1983	383	196	-49
11	May 3-4, 1984	413	444	+ 8
12	May 28-30, 1984	547	552	+ 1
14	July 1, 1984	317	211	-33
17	November 5, 1984	335	167	-50
19	August 8, 1985	472	459	- 3
21	September 24-25, 1985	894	1,090	+22

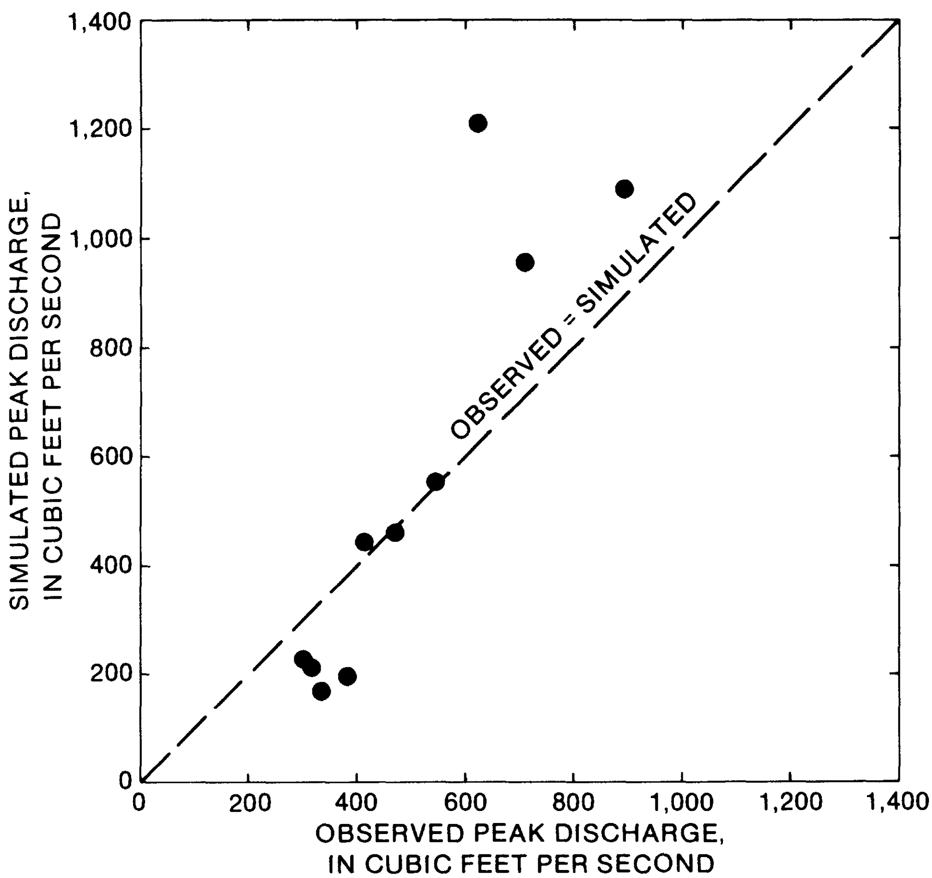


Figure 12.--Relation between observed and simulated peak discharge for verification storms.

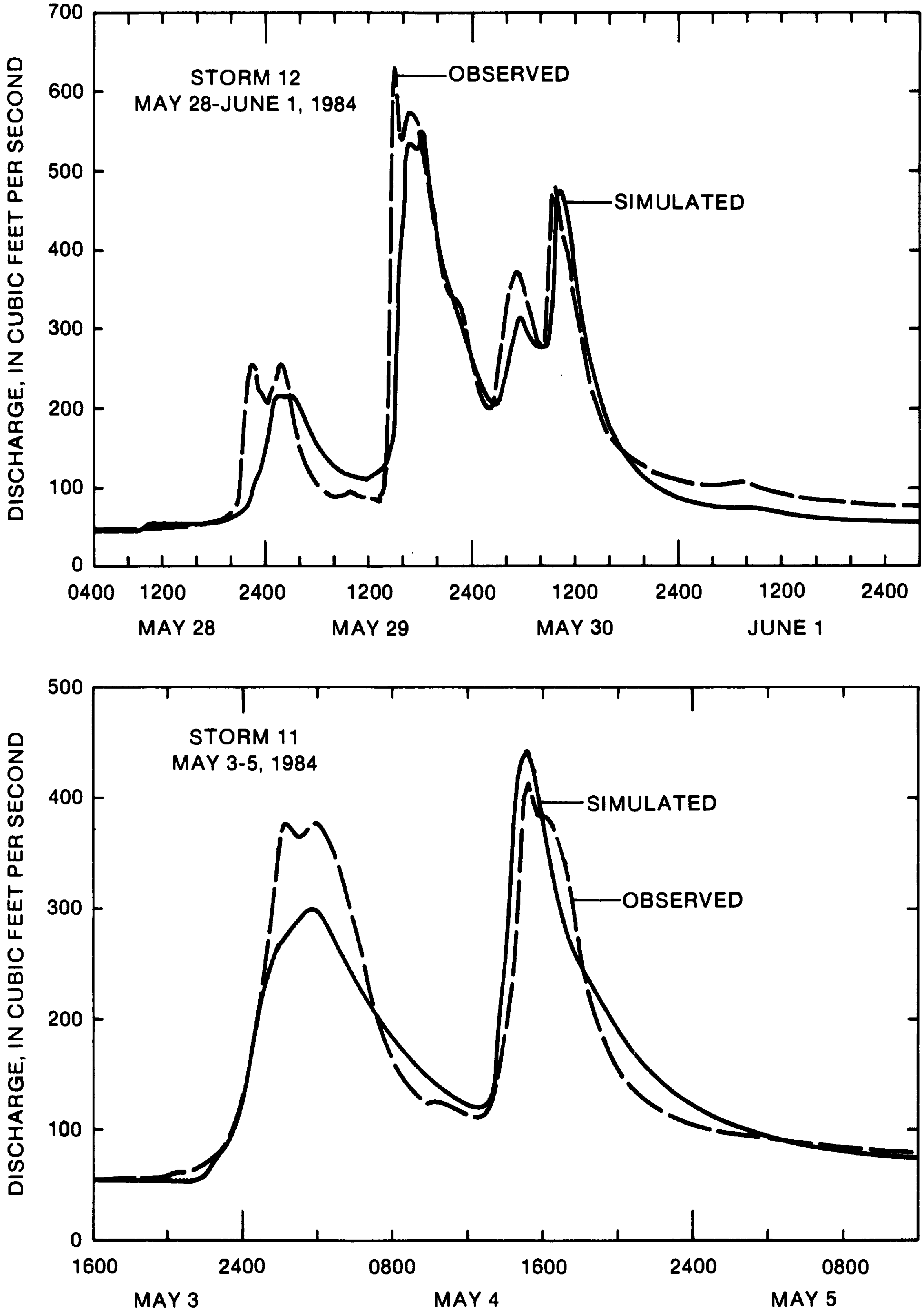


Figure 13.--Hydrographs of selected verification storms.

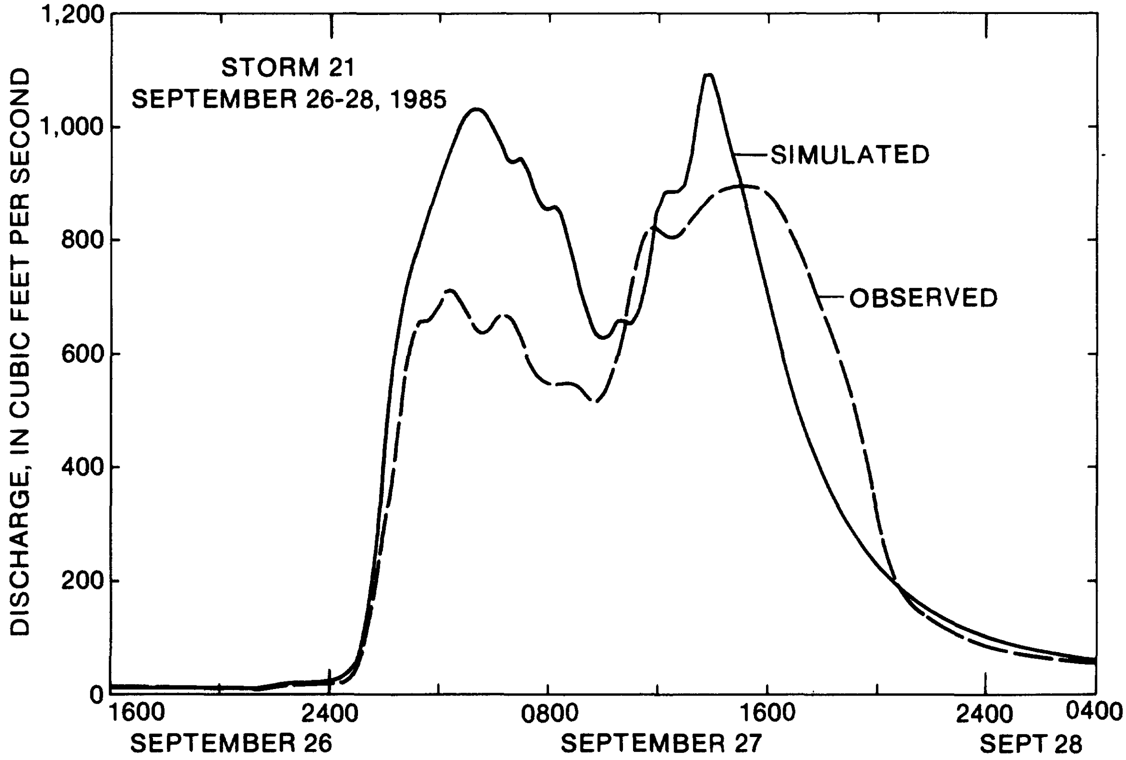


Figure 14.--Hydrograph of the September 26-28, 1985, storm (Hurricane Gloria).

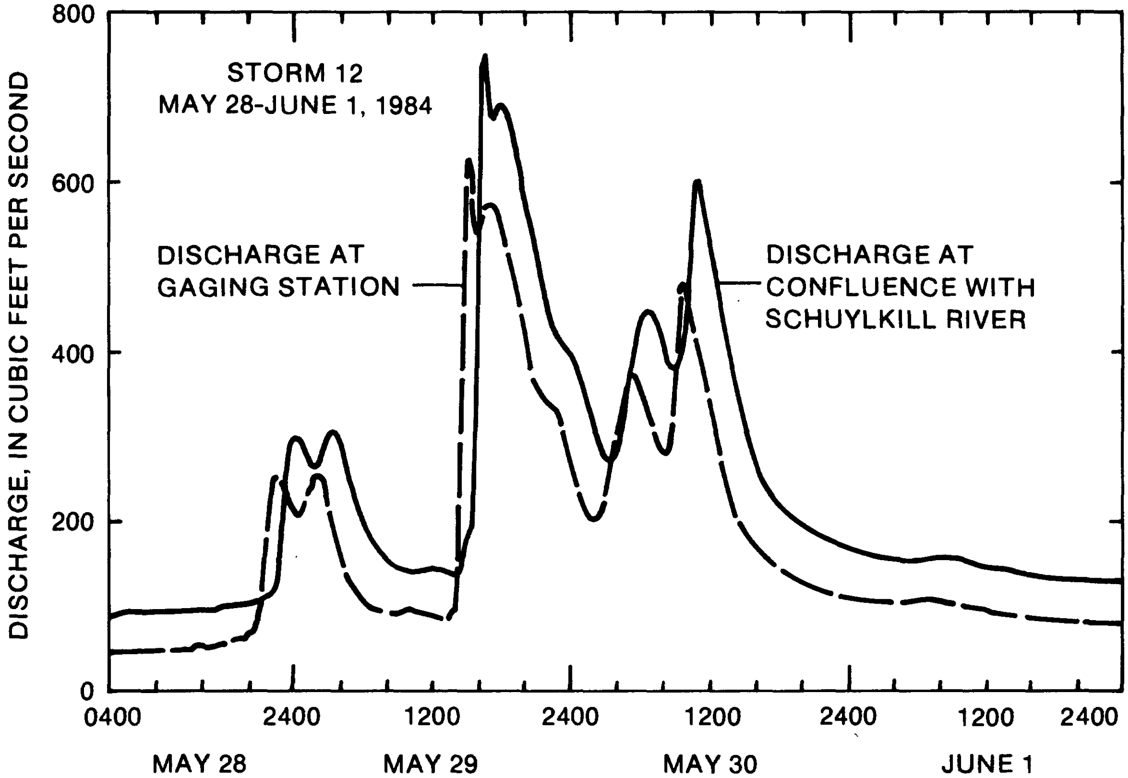
Flow Routing from the Gaging Station to the Schuylkill River

Streamflow was routed from the gaging station to the Schuylkill River using DR3M. Discharge at the gaging station, either measured or simulated, was input to DR3M with an input-hydrograph point corresponding to the location of the gaging station. Because the discharge of Valley Creek at its confluence with the Schuylkill River is not measured, the part of the basin below the gage could not be calibrated. The errors for routed runoff volume and peak discharge are not known, but depend on whether measured or simulated discharge is routed. Because the contributing area below the gage represents only 12 percent of the Valley Creek basin, the errors in routing measured streamflow are considered small. The errors in routing simulated streamflow to the confluence is probably about the same as the errors in simulated runoff volume and peak discharge at the gaging station. The same techniques were used for basin discretization and assignment of parameter values, and the same soil-moisture and infiltration parameters were used as for the calibrated part of the basin; therefore, no additional error should be introduced.

Streamflow measured at the gaging station was routed to the Schuylkill River for all 21 storms. Table 8 gives the simulated peak discharge and runoff volume (rainfall excess) of Valley Creek at the confluence with the Schuylkill River. Peak discharges at the confluence were 4 to 37 percent higher than at the gaging station, with an average increase of 16 percent. Typical hydrographs for routed storms are shown on figure 15.

Table 8.--Simulated peak discharge and runoff volume of Valley Creek at the confluence with the Schuylkill River
[ft^3/s, cubic feet per second]

Storm number	Storm date	Peak discharge (ft^3/s)		Simulated runoff volume (inches)	
		Observed at Valley Creek at Pa. Turnpike Bridge	Simulated at Valley Creek at confluence with the Schuylkill River	Valley Creek at Pa. Turnpike Bridge	Valley Creek at confluence with the Schuylkill River
1	March 18-19, 1983	365	381	0.196	0.165
2	March 21, 1983	826	1,010	.485	.606
3	March 27-28, 1983	711	856	.456	.560
4	April 9-10, 1983	617	709	.297	.321
5	May 21-22, 1983	302	320	.179	.185
6	June 19-21, 1983	343	408	.325	.445
7	December 12-13, 1983	627	777	.909	1.119
8	December 22, 1983	556	675	.327	.403
9	December 28, 1983	383	447	.129	.151
10	April 4-5, 1984	900	1,190	.671	.959
11	May 3-4, 1984	413	500	.350	.336
12	May 28-30, 1984	547	752	.774	.770
13	June 24-25, 1984	401	455	.193	.144
14	July 1, 1984	317	354	.126	.070
15	July 7, 1984	477	530	.226	.142
16	July 27, 1984	508	554	.205	.163
17	November 5, 1984	335	365	.124	.083
18	July 31, 1985	301	343	.198	.171
19	August 8, 1985	472	535	.242	.181
20	August 14, 1985	716	771	.274	.208
21	September 24-25, 1985	894	944	1.155	.924



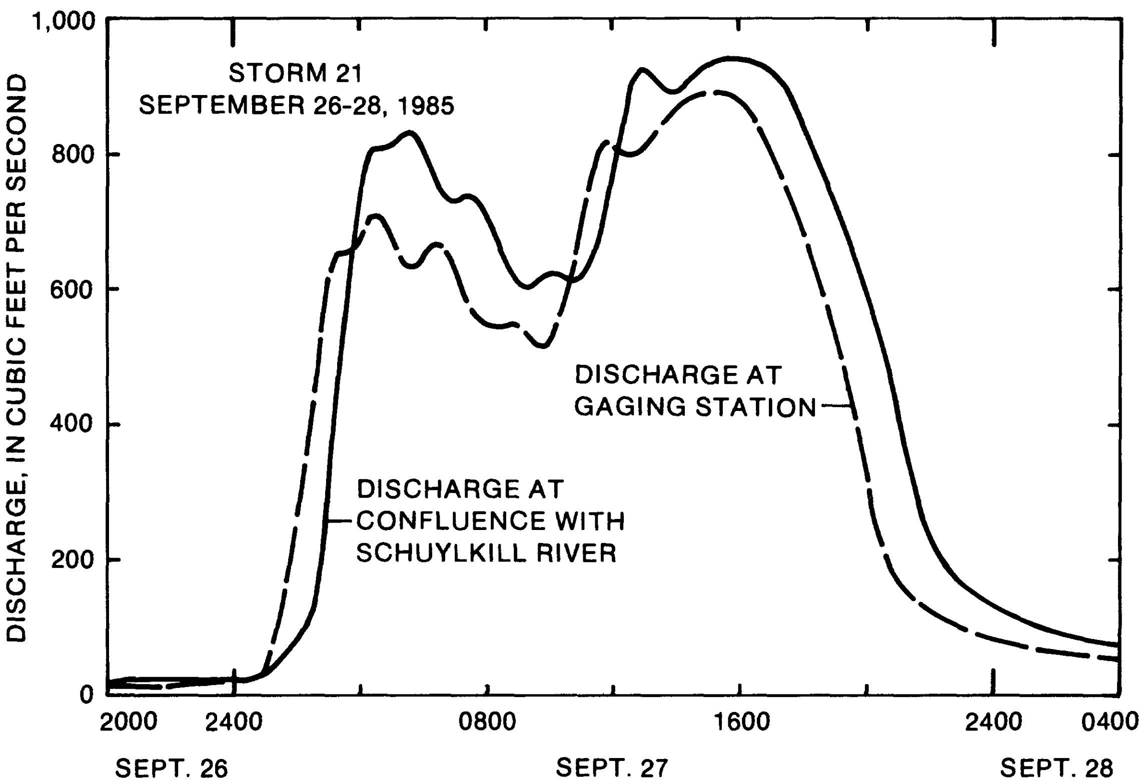


Figure 15.--Hydrographs of Valley Creek at the confluence with the Schuylkill River for selected storms.

Results of Model Simulations

As the Valley Creek basin continues to undergo urbanization, pervious farmland and woodland is being converted to extensive areas of impervious roofs, parking lots, and roadways, particularly in corporate and industrial parks. Generally, as impervious area increases, flood peaks and runoff volumes also increase. Model simulations of selected storms were made to estimate the expected magnitude of increase in flood peak and runoff volume resulting from the conversion of pervious to impervious surfaces.

Almost all of the development in the Valley Creek basin is taking place in Chester Valley, particularly along U.S. Route 202. Because extensive development is unlikely on the steeper slopes of the crystalline rocks to the north and south of Chester Valley, impervious area was increased only in Chester Valley for model simulations.

About 9 percent of Chester Valley is covered by impervious surfaces. Impervious area was increased to 15, 20, and 25 percent, and discharge hydrographs were simulated for six selected storms. These storms were selected because the simulated peak discharges were within 8 percent of observed peak discharges. No additional flood control in the basin was assumed for model simulations. Construction of proper flood controls as urbanization progresses may help to minimize the effects of increasing impervious area. However, simulation studies have shown that some flood controls may actually increase peak discharge (Sloto, 1982, p. 31; Sloto, 1985, p. 10).

When impervious area was increased to 15 percent, runoff volume increased from 9 to 18 percent, with an average increase of 14 percent (table 9). Peak discharge increased 9 to 17 percent, with an average increase of 14 percent (table 10). When impervious area was increased to 20 percent, runoff volume increased 19 to 39 percent, with an average increase of 30 percent. Peak discharge increased 16 to 43 percent, with an average increase of 31 percent. When impervious area was increased to 25 percent, runoff volume increased 32 to 67 percent, with an averge increase of 52 percent. Peak discharge increased 26 to 80 percent, with an average increase of 55 percent. The increase in both runoff volume and peak discharge was greatest for storm 19. Typical hydrographs are shown on figure 16.

An increase in impervious area is not the only factor that causes the increase in peak discharges in table 9. Rainfall intensity and antecedent rainfall also affect the peak discharge (table 11). Storms 11, 15, and 19 had simulated peak discharges of 444 to 465 ft^3/s for simulations with 9 percent impervious area. For simulations with 25 percent impervious area, peak discharges ranged from 558 to 837 ft^3/s (fig. 17). Storm 15, which had the highest peak discharge for the simulation with 25 percent impervious area, had the highest antecedent rainfall and the second highest rainfall intensity. Storm 19, which had the second highest peak discharge for the simulation with 25 percent impervious area, had the highest rainfall intensity, but a low antecedent rainfall. Storm 11, which had the lowest peak discharge for the simulation with 25 percent impervious area, had the lowest rainfall intensity and no antecedent rainfall.

Runoff volumes and peak discharges were also simulated for Valley Creek at the confluence with the Schuylkill River with increased impervious area. Simulated streamflow (table 10) was routed to the confluence of Valley Creek with the Schuylkill River. When impervious area was increased to 15 percent, runoff volume increased 6 to 18 percent, with an average increase of 12 percent (table 12). Peak discharge increased 8 to 16 percent, with an average increase of 13 percent (table 13). When impervious area was increased to 20 percent, runoff volume increased 14 to 37 percent, with an average increase of 27 percent. Peak discharge increased 15 to 41 percent, with an average increase of 27 percent. When impervious area was increased to 25 percent, runoff volumes increased 25 to 63 percent, with an average increase of 46 percent. Peak discharge increased 25 to 78 percent, with an average increase of 50 percent. Typical hydrographs are shown on figure 18.

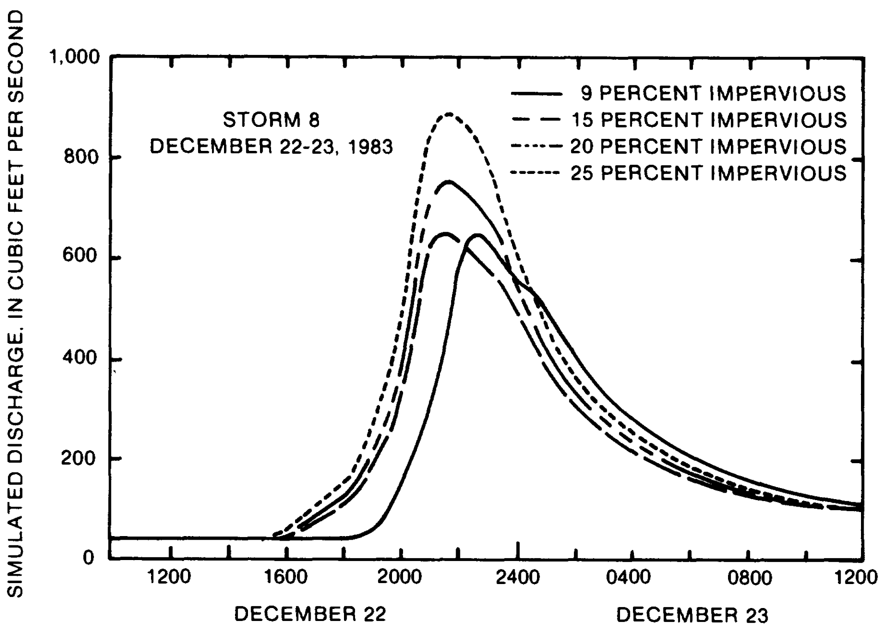
Peak discharges simulated with increased impervious area may be greater than those that might actually occur. Model-simulated peak discharges were greater than the observed for both the calibration and verification storms having peak discharges greater than 600 ft^3/s (tables 6 and 7). Therefore, it is possible that simulated peak discharges greater than 600 ft^3/s for model simulations with increased impervious area also are higher than what might actually occur.

Table 9.--Simulated runoff volume of Valley Creek at the Pennsylvania Turnpike bridge for selected storms with increased impervious area in the Valley Creek basin

		Percent impervious area						
		9	15		20		25	
Storm number	Storm date	Runoff volume (inches)	Runoff volume (inches)	Percent increase	Runoff volume (inches)	Percent increase	Runoff volume (inches)	Percent increase
2	March 21, 1983	0.485	0.526	9	0.575	19	0.639	32
8	December 22, 1983	.327	.371	14	.423	29	.491	50
11	May 3-4, 1984	.350	.405	16	.470	34	.556	59
12	May 28-30, 1984	.774	.870	12	.986	27	1.142	48
15	July 7, 1984	.226	.258	14	.300	33	.357	58
19	August 8, 1985	.242	.286	18	.336	39	.403	67
Average increase				14		30		52

Table 10.--Simulated peak discharge of Valley Creek at the Pennsylvania Turnpike bridge for selected storms with increased impervious area in the Valley Creek basin
[ft^3/s, cubic feet per second]

		Percent impervious area						
		9	15		20		25	
Storm number	Storm date	Peak discharge (ft^3/s)	Peak discharge (ft^3/s)	Percent increase	Peak discharge (ft^3/s)	Percent increase	Peak discharge (ft^3/s)	Percent increase
2	March 21, 1983	878	975	11	1,110	26	1,310	49
8	December 22, 1983	555	644	16	741	34	883	59
11	May 3-4, 1984	444	482	9	514	16	558	26
12	May 28-30, 1984	552	628	14	721	31	850	54
15	July 7, 1984	465	546	17	665	43	837	80
19	August 8, 1985	459	537	17	616	34	734	60
Average increase				14		31		55



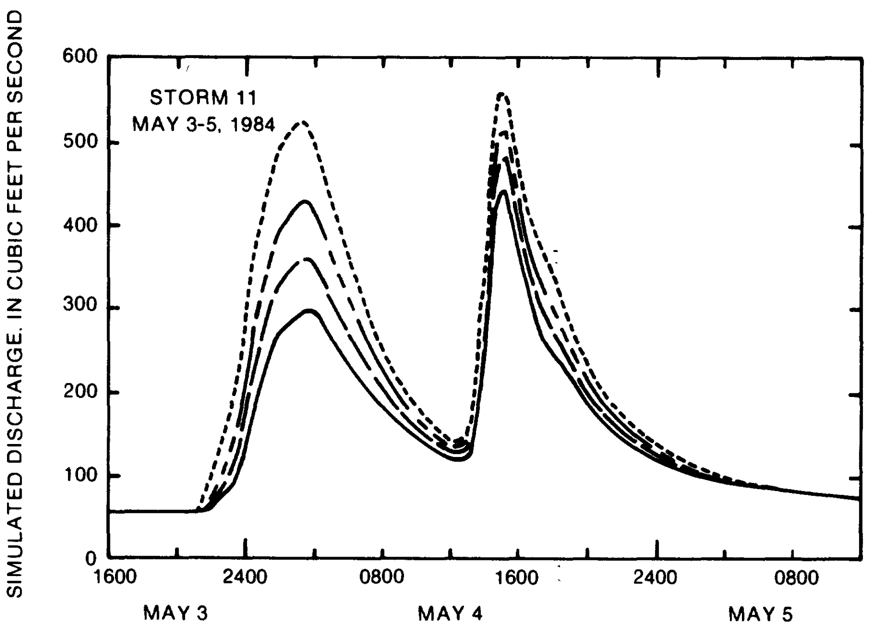


Figure 16.--Simulated hydrographs of Valley Creek at the Pennsylvania Turnpike bridge for selected storms showing the effects of increased impervious area in the Valley Creek basin.

Table 11.--Rainfall intensity and antecedent rainfall for storms 11, 15, and 19
[ft^3/s, cubic feet per second]

Storm number	Storm date	Simulated peak discharge (ft^3/s) Percentage of impervious area 9	25	Total rainfall (inches)	Maximum 1-hour rainfall intensity (inches)	Two-day antecedent rainfall (inches)
11	May 3-4, 1984	444	558	2.25	0.43	0.0
15	July 7, 1984	465	837	1.53	1.22	.72
19	August 8, 1985	459	734	2.06	1.54	.04

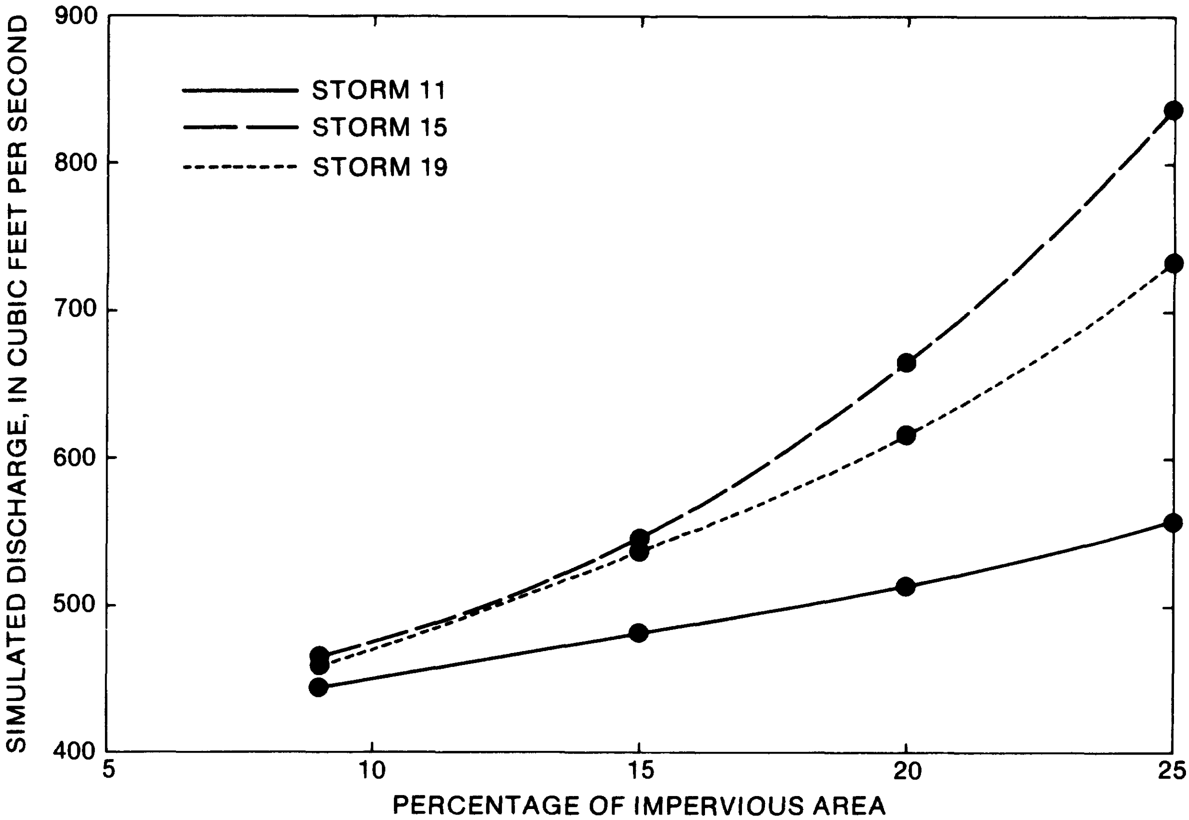


Figure 17.--Simulated peak discharge of Valley Creek at the Pennsylvania Turnpike bridge for storms 11, 15, and 19 with increased impervious area in the Valley Creek basin.

Table 12.--Simulated runoff volume of Valley Creek at the confluence with the Schuylkill River for selected storms with increased impervious area in the Valley Creek basin

		Percent impervious area						
		9	15		20		25	
Storm number	Storm date	Runoff volume (inches)	Runoff volume (inches)	Percent increase	Runoff volume (inches)	Percent increase	Runoff volume (inches)	Percent increase
2	March 21, 1983	0.508	0.536	6	0.579	14	0.635	25
8	December 22, 1983	.335	.375	12	.421	26	.480	43
11	May 3-4, 1984	.347	.397	14	.454	31	.530	53
12	May 28-30, 1984	.776	.858	11	.960	24	1.097	41
15	July 7, 1984	.219	.244	11	.281	28	.331	51
19	August 8, 1985	.231	.273	18	.317	37	.376	63
Average increase				12		27		46

Table 13.--Simulated peak discharge of Valley Creek at the confluence with the Schuylkill River for selected storms with increased impervious area in the Valley Creek basin
[ft^3/s, cubic feet per second]

		Percent impervious area						
		9	15		20		25	
Storm number	Storm date	Peak discharge (ft^3/s)	Peak discharge (ft^3/s)	Percent increase	Peak discharge (ft^3/s)	Percent increase	Peak discharge (ft^3/s)	Percent increase
2	March 21, 1983	1,040	1,140	10	1,250	20	1,450	39
8	December 22, 1983	639	729	14	826	29	968	51
11	May 3-4, 1984	481	520	8	555	15	601	25
12	May 28-30, 1984	604	672	11	765	27	898	49
15	July 7, 1984	474	551	16	668	41	844	78
19	August 8, 1985	490	568	16	646	32	766	56
Average increase				13		27		50

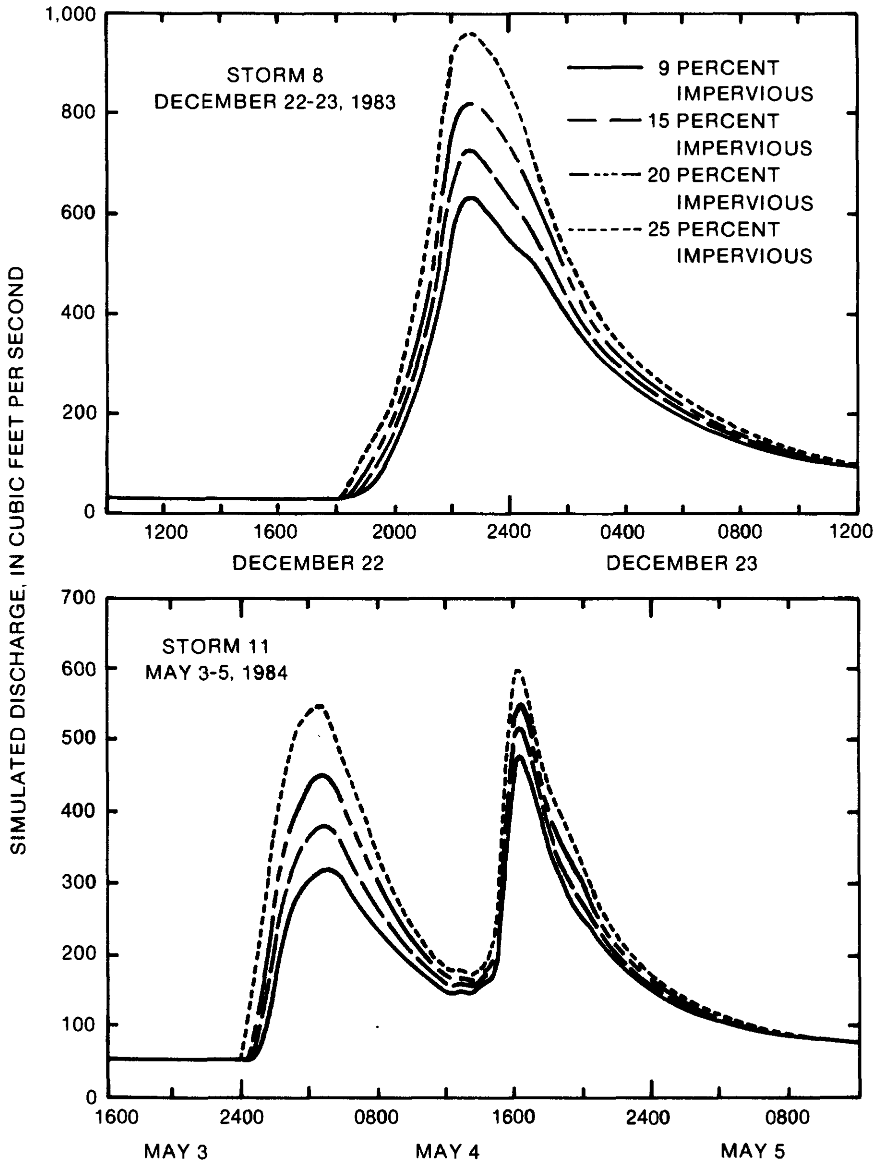


Figure 18.--Simulated hydrographs of Valley Creek at the confluence with the Schuylkill River for selected storms showing the effects of increased impervious area in the Valley Creek basin.

SUMMARY

Peak discharge and runoff volume were simulated for 21 storms in the Valley Creek basin by using version II of the U.S. Geological Survey Distributed Routing Rainfall-Runoff Model (DR3M). Rainfall was measured at three recording raingages in the basin. Simulated runoff volumes and peak discharges were compared with those observed at stream-gaging station Valley Creek at the Pennsylvania Turnpike bridge near Valley Forge (station number 01473169).

Two sets of soil-moisture and infiltration parameters were used, one for soils derived from carbonate rock, and one for soils derived from crystalline rock. Soil-moisture and infiltration parameters were calibrated by using 19 storms. The average error for simulated runoff volume was 29 percent.

The set of storms for calibration of peak discharge contained 11 storms occurring between March 1983 and August 1985, with discharges ranging from 301 to 900 ft^3/s. The average error for peak discharge was 19 percent.

The set of storms for verification of peak discharge contained 10 storms occurring between March 1983 and September 1985, with discharges ranging from 302 to 894 ft^3/s. The average error for peak discharge was 32 percent, which is higher than that for the calibration set of storms. However, the match between observed and simulated hydrograph shape was better than that for the calibration storms.

Streamflow was routed from the gaging station to the confluence of Valley Creek with the Schuylkill River using DR3M. Discharge measured or simulated at the gaging station was input to DR3M with the use of an input-hydrograph point corresponding to the location of the gaging station. Streamflow measured at the gaging station was routed to the Schuylkill River for 21 storms. Peak discharges at the confluence were 4 to 37 percent higher than at the gaging station, with an average increase of 16 percent.

Simulations were made to determine the effect on runoff volume and peak discharge of increasing impervious area in Chester Valley resulting from continuing urbanization. Impervious area was increased from 9 percent to 15, 20, and 25 percent, and discharge hydrographs for six storms were simulated. The six storms were selected because the simulated peak discharges were within 8 percent of the observed peak discharge. For 25 percent impervious area, runoff volume would increase an average of 52 percent, and peak discharge would increase an average of 55 percent for Valley Creek at the Pennsylvania Turnpike bridge. At the confluence of Valley Creek with the Schuylkill River, runoff volume would increase an average of 46 percent, and peak discharge would increase an average of 50 percent. However, because simulated peak discharges were greater than observed for storms having peak discharges greater than 600 ft^3/s, simulated peak discharges greater than 600 ft^3/s for simulations with increased impervious area are probably higher than what might actually occur.

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