EFFECTS OF URBANIZATION ON PEAK STREAMFLOWS IN FOUR CONNECTICUT COMMUNITIES, 1980-84

By Lawrence A. Weiss

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CONVERSION FACTORS AND ABBREVIATIONS

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

Multiply inch-pound unit	By	To obtain metric unit
	Length	
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
	Area	
square mile (mi^2)	2.590	square kilometer (km^2)
	Flow	
inch per hour (in./hr)	25.40	millimeter per hour (mm/hr)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
cubic foot per second per square mile [$(ft^3/s)/mi^2$]	0.01093	cubic meter per second per square kilometer [$(m^3/s)/km^2$]
	Slope	
foot per mile (ft/mi)	0.1894	meter per kilometer (m/km)
	Volume	
cubic foot per square mile (ft^3/mi^2)	0.0109	cubic meter per square kilometer (m^3/km^2)

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ABSTRACT

Peak flows for six small urban streams were obtained from rainfall and runoff data collected from 1981-84 and from a distributed-routing rainfall-runoff model that simulated storm runoff for the period 1951-80. Recurrence intervals of the simulated peak flows for these streams and three other urban streams were estimated by the log-Pearson Type III method and compared with peak flows for rural streams that were computed from regression equations. A comparison of the ratios of urban to rural peak flows shows that the ratios are outside the 95-percent confidence limits of the rural regression equations for sites where more than 50 percent of the drainage area is served by storm sewers. Peak flows for such areas can be adjusted graphically for the effects of urbanization if the streams drain less than 10 square miles and man-made storage is less than 4.5 million cubic feet per square mile.

Ratios of peak flows in urban basins to peak flows in rural basins in Connecticut are about 1.5 to 6.1 for the 2-year flood and 1.1 to 4.3 for the 100-year flood. The lower ratios, in each case, apply where 30 percent of the area is served by storm sewers, and the higher ratios apply where 90 percent of the area is served by storm sewers.

INTRODUCTION

Engineers and others who delineate flood plains, design bridges and culverts, measure erosion and deposition of streambanks, and study contamination of surface water need more flood-frequency data than are usually available. In July 1980, the U.S. Geological Survey began a cooperative study with the Connecticut Department of Transportation (CDOT), the town of Enfield, and the cities of Meriden, New Britain, and Norwalk to evaluate the effects of urbanization on peak flows in these communities.

Before 1960, information on the hydrologic characteristics of streams that drain areas of less than 10 mi^2 (square miles) in Connecticut was sparse. In 1960, CDOT and the U.S. Geological Survey began a cooperative program to collect peak-flow data for 41 streams draining rural areas of less than 10 mi^2 in the State (Weiss, 1983). Regression equations were used to determine magnitude and frequency of floods on ungaged streams in urban and rural communities because data for urban areas were sparse. Some officials of CDOT and of several urban communities were concerned that flood frequency in urban areas might be underestimated by use of the regression equations.

In 1968, Congress passed the National Flood Insurance Act. By this act, the U.S. Department of Housing and Urban Development (HUD) was authorized to hire governmental agencies and private contractors to cooperate with communities to define areas of 100-year flood-recurrence intervals. These studies are now coordinated by the Federal Emergency Management Agency (FEMA). The results of these studies are used in the development and design of flood-insurance programs. The Geological Survey has done several "flood-insurance" studies in Connecticut for FEMA.

Purpose and Scope

This report describes the results of a study to: (1) determine the 100-year flood for specific streams in the urban communities of Enfield, Meriden, New Britain, and Norwalk, and (2) develop a quantitative technique for CDOT to use in evaluating the effects of urban influences on peak flows in Connecticut. The objectives of the four communities and the State in determining the 100-year flood were to refine the meaning of "flooding potential" so that realistic flood elevations could be established and to evaluate the effects of urbanization on peak flows for use in the design of dams, bridges, and culverts in urban areas of Connecticut.

The effects of urbanization on peak flows are understood qualitatively, but quantitative data are not readily available. In Connecticut, peak-flow data have been collected for about 25 yr (years) for 96 rural basins where drainage ranges from 1 to 1,500 mi^2. Peak-flow data have been collected for six streams draining urban areas for more than 25 yr, but data from only three of these streams, Piper Brook, Mill Brook, and North Branch Park River, are used in this report (fig. 1). The data for the other three streams, South Branch Park River, Park River, and Wash Brook, were not used because of flood-control reservoirs in the headwaters.

Locations of Data-Collection Sites and Study Areas

The urban communities studied were Enfield in north-central Connecticut, Meriden and New Britain in west-central Connecticut, and Norwalk in southwestern Connecticut (fig. 1). Stream gages were on Freshwater Brook in Enfield (fig. 2; in pocket), Piper and Willow Brooks in New Britain (fig. 3; in pocket), Sodom and Harbor Brooks in Meriden (fig. 4; in pocket), and Betts Pond and Keelers Brooks in Norwalk (fig. 5; in pocket). Rain gages were at Higgins School in Enfield (fig. 2), East Street and New Britain High School in New Britain (fig. 3), Meriden Town Hall in Meriden (fig. 4), and Jefferson and Brookside Elementary Schools in Norwalk (fig. 5).

Long-term urban streamflow data (greater than 25 years) were available for Piper Brook in Newington Junction, Mill Brook in Newington, and North Branch Park River in Hartford (stations 01190100, 01190200, and 01191000, fig. 1). Daily rainfall data for 1951 to 1980 were available from Bradley International Airport in Windsor Locks, Shuttle Meadow Reservoir in New Britain, and Norwalk Gas Plant (stations 415600072411501, 413440072485801, and 410700073244101, fig. 1). Five-minute rainfall data were available for Bradley International Airport in Windsor Locks for 1951-80. Daily evaporation data for 1951-84 were available from the University of Connecticut Agricultural Experimental Station in Coventry (station 414804072205101, fig. 1) in east-central Connecticut. All the rainfall and evaporation data were supplied by the National Weather Service in computer readable format from their data storage facility in Ashville, North Carolina.

Approach

Rainfall and runoff data were collected from July 1980 through September 1984. Data from six rainfall and seven runoff (streamflow) gages were recorded at 5-minute intervals on 16-channel paper tape. The tapes were processed on a minicomputer, and the data were stored in a data-management processor called ANNIE (Lumb and others, 1989). Rainfall and evaporation data for 1951-80, as previously noted, were obtained from the National Weather Service.

Gage height for Freshwater Brook was recorded at 5-minute intervals at Enfield Street in Enfield (station 01183994, fig. 2). Instantaneous peak flows were determined at Elm Street, 2.6 mi (miles) upstream from the Enfield Street gage, with a crest-stage-indicator (CSI) pipe (station 01183993, fig. 2). The rural drainage upstream from Elm Street includes two recreational impoundments, Crescent Lake and Shaker Pond. The urban area between Elm and Enfield Streets includes many malls and shopping centers. The CSI pipe was installed at Elm Street to determine the instantaneous peak flow from the rural part of the basin. During the summer, this flow was low as the two impoundments did not spill during storms. In the spring, however, the flow was considerable because the two impoundments were spilling. The CSI data were used to verify the instantaneous peak flow at Elm Street computed by the model.

The drainage of Willow Brook in New Britain (fig. 3) is 6.46 mi^2, of which 3.17 mi^2 is upstream from Shuttle Meadow Reservoir. Flow in Willow Brook is infrequent downstream from the reservoir. Peak-flow data collected from a CSI pipe on Schultz Pond Brook at Oakwood Street in New Britain (station 01192690, fig. 3) were used to verify flow from the

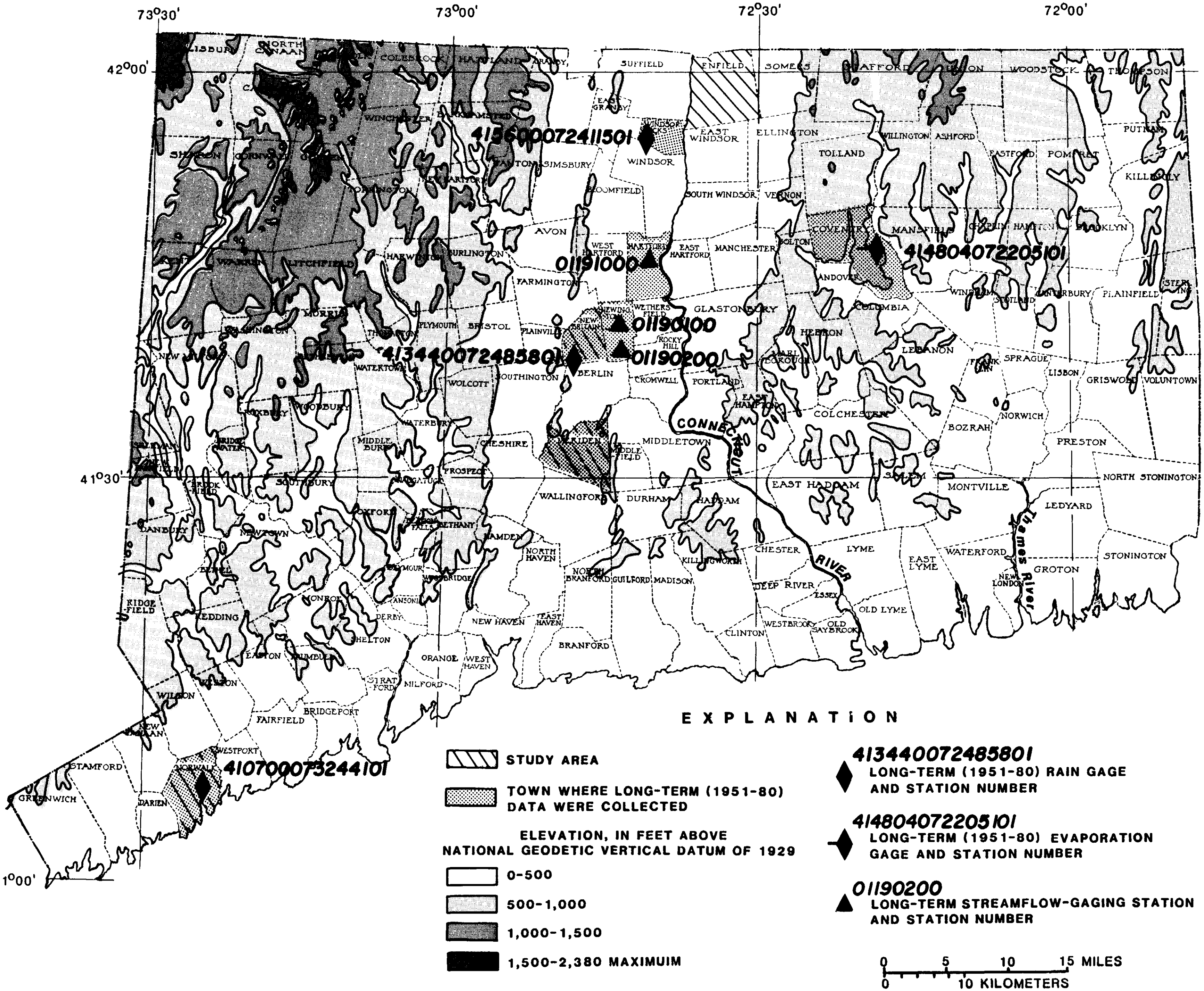

Figure 1.--Study areas, relief, and locations of long-term rain, evaporation, and stream gages.

3.17-mi^2 part of the basin. The major source of the flow in Willow Brook downstream from Buell Street (station 01192692, fig. 3) is Mason Pond Brook. A CSI pipe was installed on Mason Pond Brook at Shuttle Meadow Avenue (station 01192691, fig. 3) to collect data for verifying the modeled peak flow from this 1.39-mi^2 rural drainage basin.

Two continuous-record streamflow gages were installed on Harbor Brook in Meriden at Westfield Road and Bradley Avenue (stations 01196250 and 01196259, fig. 4). Much of the urban area in Meriden is between these two gages. Flow data from the Westfield Road gage were used in the model for the rural part of the Harbor Brook drainage basin.

Betts Pond Brook was gaged at Merrill Road (station 01209753, fig. 5) and Keelers Brook was gaged at Rowayton Avenue (station 01209775, fig. 5). Betts Pond Brook is in North Norwalk in the Norwalk River basin, and Keelers Brook is in South Norwalk in the Fivemile River basin (fig. 5). Because the drainage basins of both of these streams are urban and unregulated, no additional gaging stations were needed to adjust for peak flow from rural or regulated parts of the basin.

Rainfall and runoff data collected from 1980-84 at the six study sites and the evaporation data from the Coventry station were used to calibrate and verify the Geological Survey's Distributed-Routing Rainfall-Runoff Model, DR3M (Alley and Smith, 1982). After the model was calibrated and verified, the daily evaporation at Coventry, the daily rainfall at Bradley International Airport, Shuttle Meadow Reservoir, and Norwalk Gas Plant, and the 5-minute storm rainfall at Bradley International Airport were used in the model to simulate peak flows from 1951-80 at the six study sites (1980-84). The log-Pearson Type III technique (U.S. Water Resources Council, 1981) was used to estimate the magnitude and frequency relations of the measured and simulated peak flows. Regression equations for rural basins in Connecticut (Weiss, 1983) were applied to both the modeled urban areas and the three urban areas with long-term data, and ratios of urban to rural peak flow for specific recurrence intervals were determined. These ratios were then plotted against the percentage of area that contained storm sewers. Urban areas are defined in this study as those where more than 30-percent of the area is served by storm sewers. The 95-percent confidence limits for the regression equations for rural sites were calculated and compared to the ratios of the urban to rural peak flows for each recurrence interval. The greater the ratio, the more substantial the effect of urbanization is on peak flow. Ratios outside the 95-percent confidence limits are an indication of a significant difference between model-simulated urban and regression-derived rural peak flows.

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BASIN CHARACTERISTICS

Geology

The texture of stratified drift in Connecticut ranges from coarse boulder gravel to clay. Permeable deposits of fine to coarse sand and gravel and relatively impermeable silt and clay are commonly underlain by till, an unsorted, dense mixture of gravel, sand, silt, and clay, or bedrock. Effects of urbanization on peak flows in areas with large percentages of the land underlain by coarse-grained stratified drift compared to the effects in areas with fine-grained stratified drift, are discussed later in this report.

The Freshwater Brook basin in Enfield contains a small amount of coarse-grained stratified drift (fig. 2). The area upstream from station 01183993 contains only 0.2 mi^2 of coarse-grained stratified drift and the area between stations 01183993 and 01183994 contains none. Most of the surficial material between these two gages is very fine sand, silt, and clay that has low permeability. This part of the basin is completely urban.

Coarse-grained stratified drift underlies 10.3 percent of the Piper Brook basin in the northeastern part of New Britain (fig. 3). Willow Brook basin in the southwestern part of the city (fig. 3) contains no coarse-grained stratified drift upstream from Shuttle Meadow Reservoir. The part of the basin between station 01192692 and the reservoir contains

1.12 mi^2 of coarse-grained stratified drift, about one-third of the total drainage.

Harbor Brook in Meriden drains the eastern two-thirds of the city (fig. 4). The basin upstream from station 01196250 is rural and contains 1.43 mi^2 of coarse-grained stratified drift (about 17 percent of the total area). The part of the basin between stations 01196250 and 01196259 contains 0.96 mi^2 of this material (about 27 percent of the total area).

Betts Pond Brook in northern Norwalk drains into the Norwalk River and Keelers Brook in the southern part of the city drains into the Fivemile River (fig. 5). Betts Pond Brook drains a predominantly urban area of Norwalk, 42 percent of which is underlain by coarse-grained stratified drift. Keelers Brook also drains a predominantly urban area in South Norwalk, but only about 7 percent of the drainage is underlain by coarse-grained stratified drift.

Physical Features

The size of the drainage basins studied from 1980 to 1984 ranged from 2.25 to 11.9 mi^2 (table 1), whereas the size of the urban areas studied ranged from 2.25 to 3.58 mi^2. Slopes of the main channels of the streams in the urban areas ranged from 4.5 to 80.2 ft/mi (feet per mile). The slope of Freshwater Brook between stations 01183993 and 01183994 is slight (4.5 ft/mi) because the brook flows across a glacial lake bed in the Connecticut River basin. Upstream from station 01183993, the area is steeper (17 ft/mi) and is forested. Two large impoundments in the upper part of the Freshwater Brook drainage, Shaker Pond and Crescent Lake (fig. 2), are used for recreation, and rarely spill between late spring and early fall.

The slopes of Piper Brook (77.7 ft/mi) and the reach of Willow Brook between stations 01192689 and 01192692 (80.2 ft/mi) are the steepest slopes of the streams studied. Piper Brook is the southern headwater of the South Branch Park River, which joins the North Branch Park River at Hartford and then flows as the Park River to the Connecticut River. Willow Brook flows into the Mattabesset River, which flows into the Connecticut River at Middletown. Shuttle Meadow Reservoir, the primary source of water supply for greater New Britain, is in the southwestern, forested part of Willow Brook basin.

The slope of Harbor Brook is 18.5 ft/mi between stations 01196250 and 01196259. The area upstream from station 01196250 is drained by three tributaries: Willow, Spoon Shop, and North Brooks. North Brook flows from Bradley Hubbard Reservoir, which is used by the city of Meriden for water supply. The three streams join just upstream from Baldwin Pond to form Harbor Brook, which then joins with Sodom Brook and flows to the Quinnipiac River.

The slopes of Betts Pond and Keelers Brooks are 48.5 and 53.8 ft/mi. Both basins (fig. 5) are within 2 mi of the coastline and are within the city limits of Norwalk. Betts Pond Brook flows into the Norwalk River while Keelers Brook flows into the Fivemile River. The Norwalk and Fivemile Rivers flow into Long Island Sound.

Land Use

Freshwater Brook, flowing from northeast to southwest, drains the northern one-third of Enfield or about 11 mi^2 of the 33.8-square mile area. The area upstream from station 01183993 (fig. 2) at Elm Street is rural except for the urban Jawbucks Brook area. In the northern part of Freshwater Brook basin, vegetable farming is a major industry while downstream from station 01183993, tobacco is still cultivated. However, much of the former tobacco-growing land has been purchased for real-estate development, mostly commercial. Thirty-seven percent of the area between stations 01183993 and 01183994 is served by storm sewers, and, with the addition of roads and parking areas, 29 percent is impervious.

Piper Brook (fig. 3) drains only 18 percent of the 13.3 mi^2 of New Britain, but most of that basin is in the downtown commercial and industrial part of the city. More than 90 percent of the stream channel is in culverts. About 83 percent of the basin is served by storm sewers, and it is 26 percent impervious. Willow Brook drains 50 percent of residential New Britain. Eighty-seven percent of the area downstream from Shuttle Meadow Reservoir is served by storm sewers, but only 7 percent is impervious.

Harbor Brook drains nearly 50 percent of Meriden's 24.0 mi^2. The area upstream from station 01196250 (fig. 4) is a mixture of urban and rural development. The urban downstream area is densely covered with residential, commercial, and industrial buildings. Ninety percent of the area between stations 01196250 and 01196259 is served by storm sewers, and 33 percent is impervious.

Betts Pond Brook (fig. 5) drains 9 percent of Norwalk's 27.7 mi^2, and Keelers Brook drains 8 per-

Table 1.--*Characteristics of drainage basins and intervening areas in Connecticut*

[mi^2, square mile; mi, miles; ft/mi, feet per mile; dashes indicate not applicable]

Station number	Name of station and location	Drainage (mi^2)	Length of stream (mi)	Slope of streambed (ft/mi)	Percentage of area with coarse-grained stratified drift	Percentage of area with storm sewers
01183993	Freshwater Brook, Elm Street, Enfield	8.30	5.56	17.1	2.4	0.0
---	Intervening area	3.00	2.59	4.5	.0	37.0
01183994	Freshwater Brook, Enfield Street, Enfield	11.3	8.15	14.4	1.8	9.8
01190095	Piper Brook, East Street, New Britain	2.34	3.47	77.7	10.3	82.9
01190100	Piper Brook, Newington Junction[1]	14.4	10.0	36.0	9.9	45.0
01190200	Mill Brook, Newington[1]	2.65	2.3	16.0	28.3	50.0
01191000	North Branch Park River, Hartford[1]	25.1	11.3	16.0	9.6	31.0
01192689	Shuttle Meadow Reservoir, New Britain	3.17	--	--	.0	.0
---	Intervening area	3.29	2.42	80.2	33.2	87.4
01192692	Willow Brook, New Britain	6.46	4.20	71.1	16.5	44.5
01196250	Harbor Brook, Westfield Road, Meriden	8.32	10.7	27.4	17.2	15.0
---	Intervening area	3.58	3.12	18.5	26.7	90.0
01196259	Harbor Brook, Bradley Avenue, Meriden	11.9	13.8	23.1	20.9	27.1
01209753	Betts Pond Brook, Norwalk	2.40	2.67	48.5	41.7	45.0
01209775	Keelers Brook, Norwalk	2.25	2.28	53.8	6.7	60.3

[1] Long-term sites

cent. Both basins are mostly residential, but Keelers Brook basin also contains significant commercial development. Forty-five percent of Betts Pond Brook basin is served by storm sewers, and 9 percent is impervious. Sixty percent of Keelers Brook basin is served by storm sewers, and 30 percent of its area is impervious.

The relative effect of storm sewers is greater in Betts Pond Brook basin than in Keelers Brook basin because most of the area underlain by coarse-grained stratified drift (42 percent) is served by storm sewers. A previous Connecticut study (Weiss, 1983) showed that the peak flows are inversely related to the percentage of the drainage area underlain by coarse-grained stratified drift. Infiltration of rainfall is greater and surface runoff is smaller in the areas of coarse-grained stratified drift than in areas underlain by fine-grained stratified drift or till. Storm sewers in such areas, therefore, have a greater effect on peak flows than they would in areas underlain by less permeable fine-grained stratified drift and (or) till.

CLIMATE

The climate of Connecticut has been previously described by Weiss (1975). A brief summary follows:

From November through April, most of the precipitation in Connecticut is produced by coastal storms of extratropical origin--the so-called "northeasters." A northeaster frequently forms as a secondary low-pressure center along the Atlantic coast near North Carolina while the primary low-pressure center is dissipating over the Appalachian Mountains. Thereafter, as the secondary low-pressure center generally moves northeastward parallel to the coast, it is accompanied by intense rain, snow, or both, simultaneously in different parts of the State. When this center intensifies directly south of Long Island, it usually moves slowly, resulting in an extended period of heavy precipitation. About 20 extratropical storms affect Connecticut each year, almost all occurring during the winter. Such storms tend to be of long duration.

The second principal source of winter precipitation in Connecticut are low-pressure centers that move northeastward along the Appalachian Mountains. These centers also are usually slow moving and, consequently, produce storms of long duration. Frontal storms, such as those moving over the Appalachians, tend to be less frequent but still are of long duration.

Tropical cyclones or hurricanes may affect Connecticut anytime from June through November but are most common from mid-August to the end of September. Typically, tropical storms cause torrential rains throughout a large area. Even some of the less intense storms may cause substantial floods in Connecticut because of their slow movement. From 1871 to 1973, 20 tropical storms reached the coast of Connecticut.

The effect of topography on local precipitation is substantial. The elevation in the Litchfield Hills in northwestern Connecticut (fig. 1) ranges from 800 to 2,400 ft. These hills are connected to the Hudson and Taconic Highlands, of a similar range in elevation in adjacent New York State, by an upland area in southwestern Connecticut that ranges in elevation from 300 to 800 ft. The combined Connecticut and New York uplands deflect air masses upward, especially when the lower atmosphere is completely saturated with water.

The orographic effect of the uplands is shown by the distribution of the mean annual precipitation that ranges from 41 in. (inches) in central Connecticut, which is in a rain shadow, to 52 in. in the Litchfield Hills. A secondary area of high (49 in.) annual precipitation in southeastern Connecticut is due to the abundance of maritime storms in this coastal area. These latter statistics on rainfall are based on data from the National Weather Service.

Summer precipitation commonly occurs as showers and thunderstorms, that generally are of short duration. Thunderstorms in Connecticut occur about 18 to 35 days per year (Weiss, 1975) and the mean number of days for any point depends on its location and topography. Although the number of thunderstorms is greatest in July, the storms may occur in any month. Generally, thunderstorm activity is greatest in the western part of the State, specifically in the Litchfield Hills and parts of southwestern Connecticut where the terrain is rugged. The area of the least number of thunderstorms in Connecticut is the southeastern part of the State, an area that is often affected by air masses moving over the ocean to the south and east.

Snowmelt rarely affects annual peak flows in Connecticut because the snow tends to melt gradually, especially in urban areas. Accumulation of snow in those areas rarely exceeds 2 ft (feet), and rainfall rarely exceeds 1 in. when snow is on the ground. An exception to the effect of snowmelt on peak flows is in Litchfield Hills where snowpacks are much greater than they are elsewhere in the State. Although runoff tends to increase and infiltration tends to decrease on frozen ground, small urban basins are not affected as much as large rural basins are. On the basis of the results of our streamflow simulations for 1951 to 1980, only about 25 percent of the storms occurred when the ground was frozen. (The simulations are discussed in the section, "Calibration and Verification".) However, frozen ground conditions were not evaluated in this study.

EFFECTS OF URBANIZATION ON PEAK STREAMFLOWS

An urban environment affects the hydrology of streams in several ways. Infiltration of precipitation is much less in a drainage basin where impervious surfaces such as roofs, streets, sidewalks, and parking lots cover the land surface than it was before urbanization. As infiltration capacity is decreased, so is basin storage. Also, hydraulic conveyance is more efficient in urban areas than in rural areas, as a result of lining, straightening, or deepening of stream channels and the construction of gutters, drains, and storm sewers.

The type of urban development, such as residential, commercial, or industrial, and the type of surficial geologic material on which the urbanization occurs can greatly affect the magnitude and the frequency of flooding. The commercially developed Freshwater Brook basin between Elm and Enfield Streets includes many parking lots overlying mostly varved clay. The effect of urban development on runoff over such naturally impervious material is scant. Commercial developments in New Britain and Meriden overlie coarse-grained stratified drift. Willow Brook basin in southern New Britain, between Shuttle Meadow Reservoir and station 01192692, the location of 50 percent of southern New Britain's urbanization, has one-third of its total area underlain by permeable coarse-grained stratified drift. Similarly, 50 percent of all commercial development in Meriden is between Westfield Road and Bradley Avenue gages on Harbor Brook, on coarse-grained drift.

Two basins, Keelers and Betts Pond Brooks, in Norwalk (fig. 5) are predominantly residential. Most of the development is on coarse-grained material, but in Betts Pond Brook basin the area developed on coarse-grained stratified drift is 2.6 times that in Keelers Brook. The more an area of coarse-grained material is developed, the greater the increase in peak flow and storm runoff.

A decrease in infiltration capacity and an increase in hydraulic conveyance can cause more extensive flooding in some urban areas even at lower intensities of rainfall, cause increased runoff that cannot be adequately handled by existing culvert and bridge openings, and result in greater rates of erosion. A decrease in infiltration capacity will result in less ground-water recharge and may also reduce the base flow of streams.

Rainfall-Runoff Modeling

The complexity of hydrologic systems in urban areas and the assumption of the random nature of precipitation negates simple closed-form solutions in the simulation of peak flows. A rainfall-runoff model is a mathematical description of the physical processes that control the streamflow resulting from a given rainfall distribution. The physical and hydraulic factors that significantly affect rainfall-runoff relations include evaporation, infiltration, soil moisture, main-channel shape, and the flow routing through storage facilities and pipes.

Distributed-Routing Rainfall-Runoff Model

Urban hydrologic processes are commonly modeled in two ways. The first type of model is called a lumped-parameter model and relates rainfall to runoff based on the assumption that only the most important characteristics of the hydrologic system should be used as input parameters with the actual rainfall distribution to produce the time distribution of runoff. This approach is limited because, although the parameters represent some of the hydrologic-system properties, they rarely have a direct physical interpretation. The second type of model, which was used for this study, is called a distributed-parameter model. This type of model takes into account more of the physical processes of the hydrologic system, such as vertical infiltration and evaporation, and includes physical factors such as soil moisture before storms. These parameters are either determined by direct measurement, optimization, or a combination of both.

A distributed-parameter routing procedure has been incorporated into the DR3M, the distributed-routing rainfall-runoff model (Alley and Smith, 1982). Rainfall is used as input, and excess rainfall is determined by a system that accounts for daily soil moisture, infiltration, and evaporation between storms. The basin is considered to be a plane conveying runoff into channels and is represented by a set of overland-flow, channel, and reservoir segments. Kinematic-wave theory is used in routing the runoff over the overland-flow segments and through the channel and reservoir segments. Three techniques are available to solve the mathematical equations--the explicit and the implicit finite-difference techniques and the method-of-characteristics technique. A fixed grid is used with the implicit and the explicit finite-difference techniques. Results obtained with the implicit scheme, which contains a factor to allow for wave dispersion, were better than those obtained with the explicit scheme. The larger the basin storage, the better the results are with this technique. A moving grid is used with the method-of-characteristics technique (Alley and Smith, 1982).

Excess-rainfall components

The components of excess rainfall in the DR3M model are soil-moisture accounting, excess rainfall on pervious and impervious areas, and parameter optimization. Soil-moisture and infiltration parameters are listed in table 2. The soil-moisture-accounting component is used to measure the effect of antecedent soil moisture conditions on infiltration. This component is used to simulate moisture redistribution in the soil column and evapotranspiration from the soil. Soil moisture is considered to be a two-layer system in the model. The soil-moisture parameters are BMSN, the available inches of soil water at field capacity, where field capacity is the water content at which internal drainage ceases; EVC, a coefficient for use in converting pan evaporation to potential evaporation, the evapotranspiration rate; and RR, the proportion of daily rainfall that infiltrates the soil for the period of simulation between storms. On the day of the storm, rainfall infiltrates the upper soil-moisture zone, increasing soil-moisture storage (SMS).

The infiltration parameters are KSAT, the effective, saturated hydraulic conductivity, in inches per hour; PSP, the suction at the wetting front for soil moisture at field capacity, in inches of pressure; and RGF, the ratio of suction at the wetting front for soil moisture at the wilting point to that at field capacity, where the wilting point is the moisture content of soil when plants wilt.

Excess rainfall of pervious areas is determined from point-potential infiltration, as computed by a variation of the Green-Ampt equation (Green and Ampt, 1911). During a simulated storm, moisture is added to SMS based on--

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

where FR is the point-potential infiltration, in inches per hour;
KSAT is the effective saturated-soil hydraulic conductivity, in inches per hour;
PS is the average suction head across the wetting front, in inches of pressure; and
SMS is the soil moisture storage, in inches.

PS is varied throughout the range from wilting point to field capacity by--

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

where PSP is the effective value of PS at field capacity;
RGF is the ratio of PS at wilting point to that at field capacity; and
BMS is the antecedent base-moisture storage.

A method presented by Crawford and Linsley (1966) is used to convert FR to effective infiltration throughout the basin. The rate of generation of excess rainfall that does not infiltrate is computed by--

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

or

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

where QR is the rate of generation of excess rainfall, in inches per hour; and
SR is the supply value of rainfall for infiltration, in inches per hour.

Excess rainfall on impervious areas is the result of runoff from roofs, driveways, streets, and parking lots. Two types of impervious surfaces can be modeled. The first type, effective impervious surfaces, are impervious surfaces that are directly connected to the channels of the drainage system. Roofs that drain onto driveways and streets and

Table 2.--*Soil-moisture and infiltration parameters*

[Dashes indicate not applicable]

Parameter	Unit	Description
BMSN	inches	Maximum effective soil-moisture-storage at field capacity.
EVC	–	Coefficient for use in converting pan evaporation to potential evaporation, the evapotranspiration rate.
RR	–	Proportion of daily rainfall that infiltrates the soil for the period of simulation except for unit days.
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 suction at field capacity.

parking lots that drain into streets are examples of these surfaces. The second type, noneffective impervious surfaces, are impervious surfaces that drain to pervious surfaces, such as roofs that drain to lawns. Runoff from noneffective impervious surfaces is added to the pervious-surface runoff. The latter is the product of rainfall on pervious surfaces and model parameter RAT, which is determined as follows:

$$RAT = \frac{DA2 + DA3}{DA3}, \qquad (5)$$

where DA2 is the noneffective impervious area; and
DA3 is the pervious area.

The extent of the two types of impervious surfaces were determined by planimetric measurements of the impervious areas on 1:24,000 scale U.S. Geological Survey 7.5-minute quadrangle maps and town storm-sewer maps. The extent of the noneffective, impervious, residential areas were also determined by use of a planimeter. This determination was subjective because of the difficulty of knowing whether drain spouts were discharging to pervious grassy areas or to impervious sidewalks or driveways. Therefore, a parameter called EAC, a multiplication factor for the effective impervious area, was used to adjust for the noneffective impervious area.

The procedure used to optimize the soil-moisture parameters, infiltration parameters, and EAC is the Rosenbrock (1960) optimization procedure. The Rosenbrock procedure is a method for fitting parameters of a model so that a function, U, is minimized where:

$$U = \sum_{i=1}^{n} [\ln(S_i) - \ln(M_i)]^2, \qquad (6)$$

where
- n is number of storms;
- S_i is the simulated runoff volume of the ith value; and
- M_i is the measured runoff volume of the ith value.

The soil-moisture and infiltration parameters (table 2) should be evaluated with large storms to assure that pervious areas are contributing substantial runoff. Large storms usually occur in Connecticut in the spring. RAT and EAC are best determined with summer or small storms, which generate most of their runoff in the impervious areas where soil-moisture and infiltration parameters are insignificant in determining the final runoff.

EVC and RR or KSAT and PSP can be interactive. A value for EVC, 0.76, is that used for Connecticut by the National Oceanic and Atmospheric Administration (H.A. Thistle, University of Connecticut, written commun., 1983), and KSAT was estimated from previous unit-hydrograph analyses done to determine lag times for streams in Connecticut (Weiss, 1975). The basins are underlain by surficial deposits of till, coarse-grained stratified drift, and varved clay. Initially, values of KSAT were adjusted based on the percentage of the basin drainage area underlain by each of these materials. Therefore, initially only BMSN, RR, PSP, and RGF were optimized using the procedure described by Rosenbrock (1960). The smaller the sum of the squared deviations between the logarithms of simulated and recorded runoff volumes, the better the computed parameter.

BMSN, PSP, and RGF are directly related to field capacity. As values of RGF increase, the sensitivity of the model's estimate of infiltration to antecedent soil moisture also increases. An example of this is the Freshwater Brook basin (station 01183994) that has the maximum RGF of 20 and is underlain by varved clay whose infiltration rate (KSAT) is 0.07 in./hr (inches per hour). The initial soil moisture in a basin before a storm, for the simulations from 1951 to 1980, is based on the daily rainfall data for stations as near to the sites being modeled as possible, in order to improve the accuracy of the infiltration estimates.

Once BMSN, RR, PSP, and RGF have been optimized so that the smallest residual between measured and simulated volume is obtained, then RAT and EAC can be optimized. The value of RAT is first determined from the initial value of noneffective impervious area by use of equation (5). The value for EAC is initially set equal to 1.0. As EAC is optimized, a new value for RAT is computed. The percentage of impervious area is adjusted by multiplying by EAC. With the new values for RAT and percentage of impervious area, EAC is again reset to 1.0 and is then optimized again. This continues until EAC equals 1.0 after optimization. Through all of this procedure, the total area of DA2 + DA3 + effective impervious area is maintained.

Routing components

An example of how a basin is divided into segments is shown in figure 6. Segments are labeled as either overland flow, or as having a channel or a pipe accepting or passing flow. The percent impervious area for each overland-flow segment is given, and the flow is routed by the Saint-Venant or shallow-water equations. The equations for unsteady free-surface flow are the equation of continuity,

$$q = \frac{\partial Q}{\partial X} + \frac{\partial A}{\partial t}, \qquad (7)$$

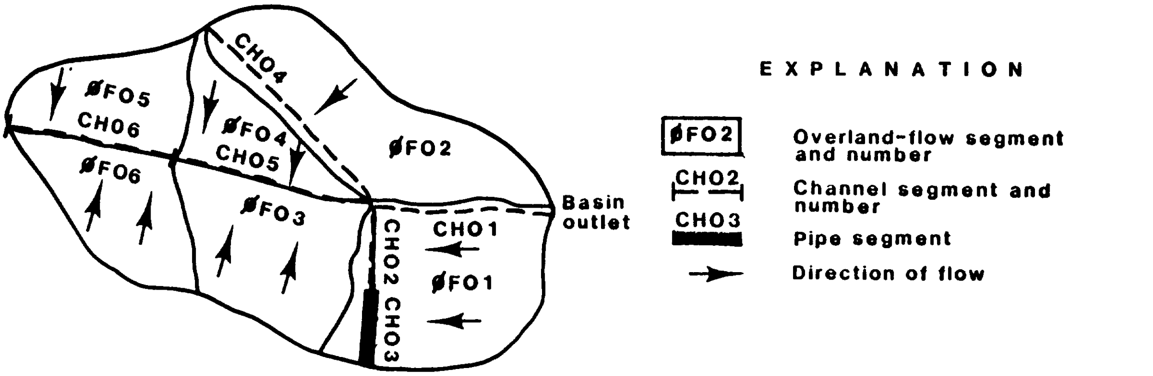
where
- q is lateral inflow per unit length;
- Q is the discharge;
- X is the distance;
- A is the flow cross-sectional area; and
- t is the time;

and the equation of momentum,

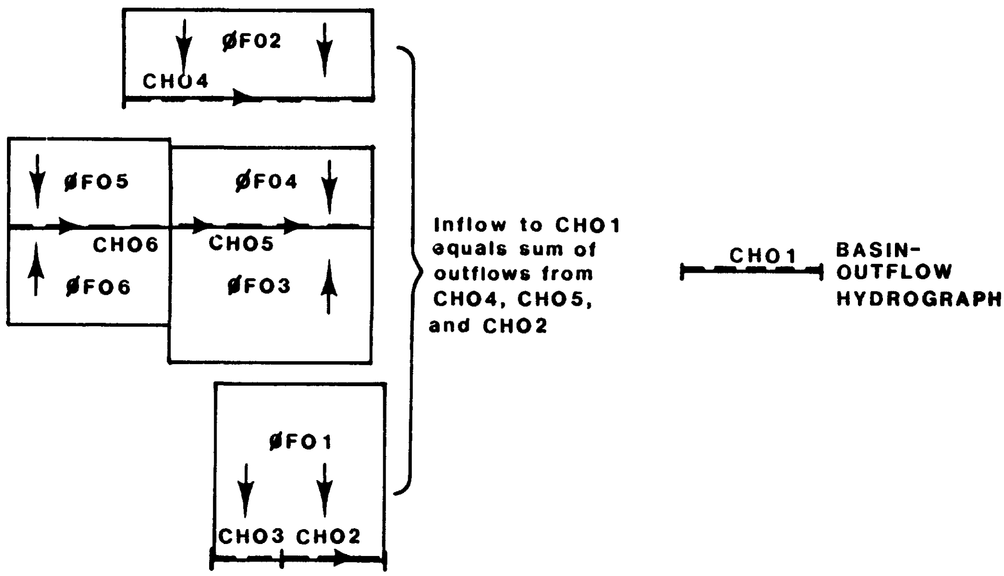
$$S_f = S_o - \left(\frac{\partial Y}{\partial X} + \frac{\partial V}{g\partial t} + \frac{\partial V}{g} \times \frac{\partial V}{\partial X}\right), \qquad (8)$$

where
- S_f and S_o are surface-water friction slope and bed slope;
- Y is the depth of flow;
- V is the velocity; and
- g is the acceleration of gravity.

Because of the complexity of basin models and the computer time needed to solve these equations



A--PLAN VIEW OF DRAINAGE BASIN



B--SCHEMATIC REPRESENTATIONS OF MODEL SEGMENTS

Segment	Inflow to Segment	
	Lateral Inflow	Upstream Inflow
ØFO1	Excess rainfall	—
ØFO2	do.	—
ØFO3	do.	—
ØFO4	do.	—
ØFO5	do.	—
CHO1	—	CHO2,CHO4,CHO5
CHO2	ØFO1	CHO3
CHO3	ØFO1	—
CHO4	ØFO2	—
CHO5	ØFO3,ØFO4	CHO6
CHO6	ØFO5,ØFO6	—

C--SEGMENT INTERRELATIONS

Figure 6.--Separation of a drainage basin into discrete overland-flow and channel segments (modified from Alley and Smith, 1982, fig. 3).

in their original form, an approximation of these equations is used in model DR3M. The acceleration terms in equation (8) are assumed to be negligible compared to the friction slope and bed slope, or S_f and S_o. S_f, can be substituted by S_o in the Manning equation resulting in--

$$Q = \frac{1.486}{n} AR^{2/3} S_o^{1/2}, \tag{9}$$

where Q is the discharge;
n is the bed friction;
A is the cross-sectional area; and
R is the hydraulic radius.

This can be further simplified to:

$$Q = \alpha A^m, \tag{10}$$

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

Routing by the implicit finite-difference method is accomplished by a four-point grid with an iterative procedure to solve for an unknown overland-flow segment. The computational box is formed by a distance interval (Δx) that varies from segment to segment and a time interval (Δt) that is constant for all segments. The implicit method solves for area (A_d) and flow (Q_d) at point d given the area and discharge at points a (A_a, Q_a), b (A_b, Q_b), and c (A_c, Q_c). The equation is written as--

$$q = \frac{W(Q_d - Q_c) + (1-W)(Q_b - Q_a)}{\Delta X} \tag{11}$$

$$= \frac{(A_d - A_b) + (A_c - A_a)}{\Delta t},$$

where W is a weighting factor for kinematic-wave dispersion. The weighting factor, W, was set equal to 0.9, which was determined in another study (Alley and Smith, 1982, p. 33) to give good closure in the solution.

Equation (11) has two unknowns, Q_d and A_d, the discharge and area at point d, which are related by equation (10). When $Q_d = \alpha A_d^m$ is substituted into equation (11), the resulting implicit finite-difference equation has one unknown and can be rearranged into the form--

$$C_o A_d^m + C_1 A_d + C_2 = 0, \tag{12}$$

where C_o is α ;

$$C_1 = \frac{\Delta X}{2W} \Delta t ;$$

and

$$C_2 = C_1 [(A_c - A_a) - A_b]$$

$$= \frac{1-W}{W} (Q_b - Q_a) - Q_c - \frac{q \Delta X}{W}.$$

The solution of equation (12) is obtained by an iterative procedure using Newton's second-order method for determining the roots of an equation.

Once the soil moisture and infiltration parameters and RAT are determined, EAC is set to 1.0 and simulations are begun. The simulations initially have alpha (α) in equation (10) set to 1.0. Values of m are equal to 1.67 for turbulent overland flow and open-channel flow and are equal to 1.0 for circular pipes. Basins, such as Freshwater Brook, which are underlain by varved clay and have sewers, may have a faster response time than those underlain by coarse-grained material, and therefore, α is greater than 1.0. Basins, such as Betts Pond Brook, which are underlain chiefly by coarse-grained stratified drift, will have a slower response time and, therefore, α is less than 1.0. The alpha term is nothing more than a timing factor.

Data requirements

Hydrologic data used in model DR3M includes values of rainfall and runoff collected at 5-minute intervals for storms and daily rainfall and evaporation for nonstorm days. Basin characteristics required as input to the model are percentage of imperviousness, channel length, bed slope, bed friction, basin area, and overland-flow area for each segment. Rainfall and runoff data were recorded at 5-minute intervals at the six rainfall and seven runoff gages (figs. 2-5). These data were used to calibrate and to verify the model. Five-minute long-term rainfall data from 1951-80 at Bradley International Airport in Windsor Locks were obtained in computer card format from the National Weather Service (NWS). Long-term records of daily rainfall at Bradley International Airport, New Britain, and Norwalk, and daily evaporation at Coventry from 1951-80 also were obtained from the NWS. Data for impervious area, stream length, drainage area, pipe sizes, and bed slopes were determined from Geological Survey 1:24,000 scale, 7.5-minute topographic maps and town-sewer maps.

All modeling was done on a minicomputer with a version of DR3M that uses a direct-access data-file system. Files are constructed by the data-management program called ANNIE (Lumb and others, 1989). This computer program is an interactive processor for hydrologic modeling that allows the user to create, check, and update input files to the model DR3M.

Application of Distributed-Routing Rainfall-Runoff Model

The modeling process, including the simulation of long-term runoff from long-term rainfall, is completed in three steps. In the first two steps, rainfall-runoff data for 1980-84 are used to (1) calibrate and (2) verify the model. In the third step, long-term rainfall data for assumed steady-state conditions in the basin are used in the calibrated and the verified model to generate annual historical peak flows before 1980. The objective of the modeling is to generate annual-peak flows for land uses representative of 1981-84 in the basin and to use these peak flows in a frequency analysis to derive the magnitude of a flood with a recurrence interval of 100 years.

Calibration and verification

Rainfall data from about 46 storms per gage were collected for model input. About 35 storms per site were actually used because recorders or clocks stopped during 11 storms. Data for about one-half of the storms were used to calibrate the model, and data for the other one-half were used to verify the model. Data used for calibration were split into spring and summer storms. There is runoff from all pervious areas during spring storms, and, therefore, such storms can be used to optimize the soil-moisture and infiltration parameters. Because pervious areas contribute little or no flow during summer storms, all the flow comes from effective and noneffective impervious areas. These storms are used in determining RAT and in refining the percentage of effective impervious area. Final model-parameter values used in the calibrated model are listed in table 3 and the results of selected model verifications are listed in table 4. Some hydrographs resulting from model verifications are shown in figures 7-12.

The parameters for soil moisture accounting and infiltration in table 3 are for either one or two soil types. Only Freshwater Brook and Harbor Brook basins were modeled for two soil types because their upstream areas consist of soil types different from those in their downstream areas.

Fifty-six segments of the Freshwater Brook basin were modeled. Upstream from Elm Street (fig. 2), the rural part of the basin includes two impoundments, Shaker Pond and Crescent Lake. The effects of the impoundments on runoff were modeled by use of the Modified Puls Method (Soil Conservation Service, 1972)--

$$\frac{2S_2}{\Delta t} + O_2 = I_1 + I_2 + \frac{2S_1}{\Delta t} - O_1, \qquad (13)$$

where
- t is the time interval;
- I_1 is the inflow to reservoir at start of Δt;
- I_2 is the inflow to reservoir at end of Δt;
- O_1 is outflow from reservoir at start of Δt;
- O_2 is the outflow from reservoir at end of Δt;
- S_1 is the change in storage at start of Δt; and
- S_2 is the change in storage at end of Δt.

The model was used to calculate values of storage and outflow that, in turn, were used to calculate values of $(2S_2/\Delta t + 0_2)$ as a function of O_2. A gage was installed at Elm Street (station 01183993) to measure the peak flows from the upstream rural drainage. The records at this site show runoff from summer storms to be rare because of storage in the upstream reservoirs. The runoff at the downstream gage at Enfield Street (station 01183994) resulting from summer storms was largely generated within the urban area downstream of Elm Street. The rainfall data were collected at Higgins School near the Enfield Street gage (fig. 2). In the rural area upstream from Elm Street, the surficial geologic material is a sandy till, but downstream, in the urban area, the material is varved clay. Therefore, the basin was modeled with two soil types to account for the difference in infiltration (table 3).

Runoff from most of the business district in New Britain drains into Piper Brook, which in this area flows through storm sewers for most of its length. The brook exits from the main sewer line about 1,000 ft upstream from East Street where the flow was gaged at station 01190095 (fig. 3). The channel of Piper Brook is constricted by vertical concrete walls, that are 13 ft apart, and the channel is concrete lined. Initially, 46 overland-flow and pipe segments were used in the modeling, but were later reduced to 4 segments. Fewer segments were used because of the difficulty in identifying the pipe segments that were actually conveying the urban runoff during storms. Identification would have required a much greater monitoring effort than was feasible.

Drawdown occurred in the stilling well at station 01190095 when flow exceeded 125 ft^3/s (cubic feet per second). This problem was overcome by the author's selecting high water marks outside the well and by his comparing all peak flows at that gage with peak flows at station 01190100 (fig. 1), located downstream at Newington Junction. Flows in excess of 125 ft^3/s were considered to be in a status of critical flow. By use of a cross section at the gage, gage height, and the equation shown below, the author was able to compute the flow at critical depth. The period that flow exceeded 125 ft^3/s was usually short and consequently there is little error in the volume of runoff measured for any storm. The equation used to compute the critical flow was--

$$Q_c = 5.67\, b d_c^{1.5}, \qquad (14)$$

where Q_c is the critical flow, in cubic feet per second;
b is the stream width, in feet; and
d_c is the critical depth, in feet.

Rainfall data for modeling were collected at East Street near station 01190095 (fig. 3). The simulated peak flows of Piper Brook for all storms evaluated were within an average error of ±24 percent. Comparisons of simulated and measured peak flows for selected storms are shown in table 4 and figure 8.

Willow Brook has two major tributaries, Schultz Pond and Mason Pond Brooks (Piper and Willow Brook drainages). Shuttle Meadow Reservoir, in the southwestern part of the Schultz Pond Brook drainage basin, rarely spills. A CSI pipe installed on Schultz Pond Brook at Oakwood Avenue (station 01192690; fig. 3) showed that runoff from the area upstream from the reservoir generally does not contribute to the peak flow at station 01192692 (fig. 3). Mason Pond Brook basin is rural, and station 01192691 (fig. 3) was used to measure peak flows and to calibrate that part of the Willow Brook basin. Rainfall was collected at New Britain High School just downstream from station 01192692 (fig. 3). Approximately one-third of New Britain's commercial district drains into Willow Brook, while the remainder of the basin is largely residential.

Sixteen segments were used to model the Willow Brook basin. The Shuttle Meadow Reservoir spilled briefly on April 19, 1983. Although this affected the storm volume, it did not contribute significantly to the peak flow for the storm because of the delay time for spilling. In a storm from June 4 to 7, 1982, 8.8 in. of rain was recorded at station 413901072464401 near station 01192692 in 3 days, and 7.73 in. in 24 hours. In the last 50 yr, this storm was second only to one over Shuttle Meadow Reservoir from August 18 to 19, 1955, that resulted in a total of 9.2 in. of rain, and 7.7 in. in 24 hours. The peak flow at station 01192692 from the June 1982 storm was 1,100 ft^3/s, and the peak flow of the August 1955 storm was 1,500 ft^3/s. Simulated peak flows were within an average error of ±11 percent of their recorded values for the 1980-84 storms used in model verification.

The headwaters of Harbor Brook, Willow, Spoon Shop, and North Brooks, which drain mostly rural areas, join near the upstream end of Baldwin Pond. Station 01196250, downstream from this pond, records the flow entering the urban area in Meriden (fig. 4). The total flow at station 01196259 located below the urban area was used to calibrate and to verify the model. The comparisons of measured and simulated discharges from runoff generated in the urban area between stations 01196250 and 01196259 for selected storms are shown in figure 10. The average error between the recorded and simulated flows was ±12 percent. Twelve overland-flow segments were used in the model between stations 01196250 and 01196259. The rainfall of June 4 to 7, 1982, generally ranged from 9 in. in northern Meriden to more than 12 in. in southern Meriden at the sewage plant on the Quinnipiac River (fig. 4). The 1,300 ft^3/s peak flow recorded at station 01196259 on June 5, 1982, was the second highest of record. The highest peak flow occurred January 25, 1979 (1,800 ft^3/s), and was caused by 3.45 in. of rain in 6 hours on frozen ground. If this had been a summer storm, the peak flow would have been lower because infiltration would have been higher than it was on the frozen ground. No adjustments were made in this study for the frozen ground conditions.

The drainages of Betts Pond Brook and Keelers Brook, which drain urban residential areas, are similar in size. However, Betts Pond Brook basin is underlain by almost three times as much coarse-grained stratified drift as Keelers Brook basin.

Table 3.--*Final model-parameter values*

[PSP, capillary potential; KSAT, saturated hydraulic conductivity; RGF, ratio that varies PSP from wilting point to field capacity; BMSN, effective soil moisture storage at field capacity; EVC, pan coefficient to convert from evaporation to potential evapotranspiration; RR, coefficient for proportioning amount of daily rainfall that infiltrates soil; EAC, multiplication factor for effective impervious area; RAT, ratio of the sum of the pervious and the noneffective impervious areas to the pervious area; W, weighting factor for kinematic-wave dispersion; in., inches; in./hr, inches per hour; mi^2, square miles; dashes indicate where values not needed]

	Parameters for soil-moisture accounting and infiltration													
	First soil type							Second soil type						
Station number	PSP (in.)	KSAT (in./hr)	RGF	BMSN (in.)	EVC	RR	EAC	PSP (in.)	KSAT (in./hr)	RGF	BMSN	EVC (in.)	RR	EAC
01183994	4.47	0.12	20.0	4.91	0.76	0.95	1.0	7.75	0.07	20.0	5.01	0.76	0.95	--
01190095	6.60	.20	14.8	5.51	.76	.76	1.0	--	--	--	--	--	--	--
01192692	2.18	.20	7.8	4.00	.76	.81	1.0	--	--	--	--	--	--	--
01196250	2.09	.15	18.1	3.12	.76	.94	1.0	--	--	--	--	--	--	--
01196259	7.87	.14	18.7	5.21	.76	.74	1.0	7.92	.20	17.8	5.79	.76	.76	--
01196259	Using 01196250 as inflow hydrograph							4.17	.30	11.8	3.74	.76	.94	1.0
01209753	4.56	.11	6.79	3.97	.76	.95	1.0	--	--	--	--	--	--	--
01209775	1.11	.27	18.8	4.51	.76	.93	1.0	--	--	--	--	--	--	--

[1] Area upstream from Shuttle Meadow Reservoir is excluded.

[2] Intervening area between stations 01196250 and 01196259.

Neither of these two basins had significant storage from impoundments nor were any parts of their basins more urban or rural than any other part. Therefore, it was not necessary to isolate parts of the basin as was done on Willow, Harbor, or Freshwater Brooks. Ten segments were used for the model of Betts Pond Brook basin and three for the model of the Keelers Brook basin. Hydrographs of measured and simulated discharges for the verification storms are shown in figures 11 (Betts Pond Brook) and 12 (Keelers Brook), and verification results for selected storms are shown in table 4. The average error between the recorded and the computed peak flows was ±14 and ±13 percent for Betts Pond and Keelers Brooks.

The calibration and the subsequent verification of the storm volumes and the peaks was successful despite problems such as storage effects on Freshwater, Willow, and Harbor Brooks in their headwaters and the intricate storm sewers of the Piper

Table 3.--*Final model-parameter values*--Continued

	Impervious and kinematic-routing parameters						
Station number	RAT	ALPHA adjust	W	Drainage (mi^2)	Pervious area (percent)	Noneffective impervious area (percent)	Effective impervious area (percent)
01183994	1.095	1.3	0.90	11.3	83.8	8.0	8.2
01190095	1.05	1.0	.90	2.34	70.1	3.5	26.4
01192692	1.139	1.0	.90	[1]3.29	82.0	11.4	6.6
01196250	1.00	.7	.90	8.32	100	0.0	0.0
01196259	1.07	.7	.90	11.9	85.0	6.0	9.0
01196259	1.00	.4	.90	[2]3.58	67.0	0.0	33.0
01209753	1.285	.7	.90	2.40	70.6	20.1	9.3
01209775	1.098	1.0	.90	2.25	63.9	6.3	29.9

Brook basin. The following criteria should be used with the model: (1) If storage in the basin headwaters is more than 4.5 million ft^3/mi^2 (cubic feet per square mile), the storage should be isolated from the model by the user's evaluating its outflow as was done on Willow and Freshwater Brooks; (2) if the storage is in the centroid of the basin, it should be incorporated into the model; however, the user's evaluation of the urban effects might be more difficult than if the storage were in the headwaters for this situation; (3) flow patterns in intricate storm-sewer systems as in Piper Brook basin, are difficult to assess, but the model does not seem to be sensitive to number of subareas used (49 compared to 4) in evaluating the timing of the peak; (4) if the upper part of a basin is rural and the lower part is urban as in Harbor Brook, then separating the two parts in the model is advantageous to assess the urban effects; (5) although no winter storms were modeled, most urban basins in Connecticut with less than 5 mi^2 of drainage, are affected by intense storms of short duration, mostly from early spring to late fall; and (6) most floods that were modeled had recurrence intervals of 10 years or less, but 100-year floods

Table 4.--*Verifications of simulated rainfall and runoff for selected storms*

Station number	Date of storm	Simulated excess rainfall (inches)	Measured direct runoff (inches)	Simulated runoff volume at outlet (inches)	Measured peak flow (cubic feet per second)	Simulated peak flow (cubic feet per second)	Error in simulated runoff volume (percent)	Error in simulated peak flow (percent)
01183994	Sept. 8, 1981	0.158	0.120	0.116	114	106	−3.3	−7.0
	Apr. 3, 1983	.189	.237	.150	67	66	−36.7	−1.5
01190095	Apr. 3, 1983	.457	.527	.455	142	139	−13.7	−2.1
	Apr. 16, 1983	.787	.869	.786	139	136	−9.6	−2.2
	Apr. 19, 1983	.517	.483	.517	123	281	+7.0	+128.0
	Aug. 12, 1983	.238	.265	.237	153	97	−10.6	−36.6
	Sept. 21, 1983	.340	.327	.339	144	176	+3.7	+22.2
01192692	Oct. 18, 1981	.231	.133	.178	230	246	+33.8	+7.0
	[1]June 5, 1982	3.810	4.874	3.715	1,095	977	−23.8	−10.8
	July 20, 1982	.183	.142	.178	140	142	+25.3	+1.4
	Aug. 9, 1982	.203	.181	.179	194	148	−1.1	−23.7
	[1]Apr. 16, 1982	.672	1.757	.671	215	259	−61.8	+20.4
Intervening area between 01196250 and 01196259	May 16, 1981	.400	.983	1.147	629	494	+68.7	−21.5
	Apr. 10, 1983	1.396	3.917	4.399	1,071	1,384	+12.3	+29.2
	June 28, 1983	.411	.403	.763	200	193	+89.3	−3.5
01209753	Apr. 3, 1982	.176	.219	.154	49	35	−29.7	−28.6
	Apr. 26, 1982	.281	.387	.257	86	69	−33.6	−19.8
	June 5, 1982	1.172	1.322	1.123	239	191	−14.3	−20.1
	June 13, 1982	.348	.346	.321	56	71	−7.2	+26.7
	Aug. 25, 1982	.173	.102	.132	45	43	+29.4	−4.4
01209775	Apr. 14, 1981	.387	.323	.233	65	48	−27.9	−26.2
	Apr. 3, 1982	.467	.504	.422	69	78	−16.3	+13.0
	Apr. 10, 1983	1.518	1.348	1.445	304	358	+7.2	+17.8

[1] Overflow from Shuttle Meadow Reservoir at end of storm, affected the direct runoff but not the peak.

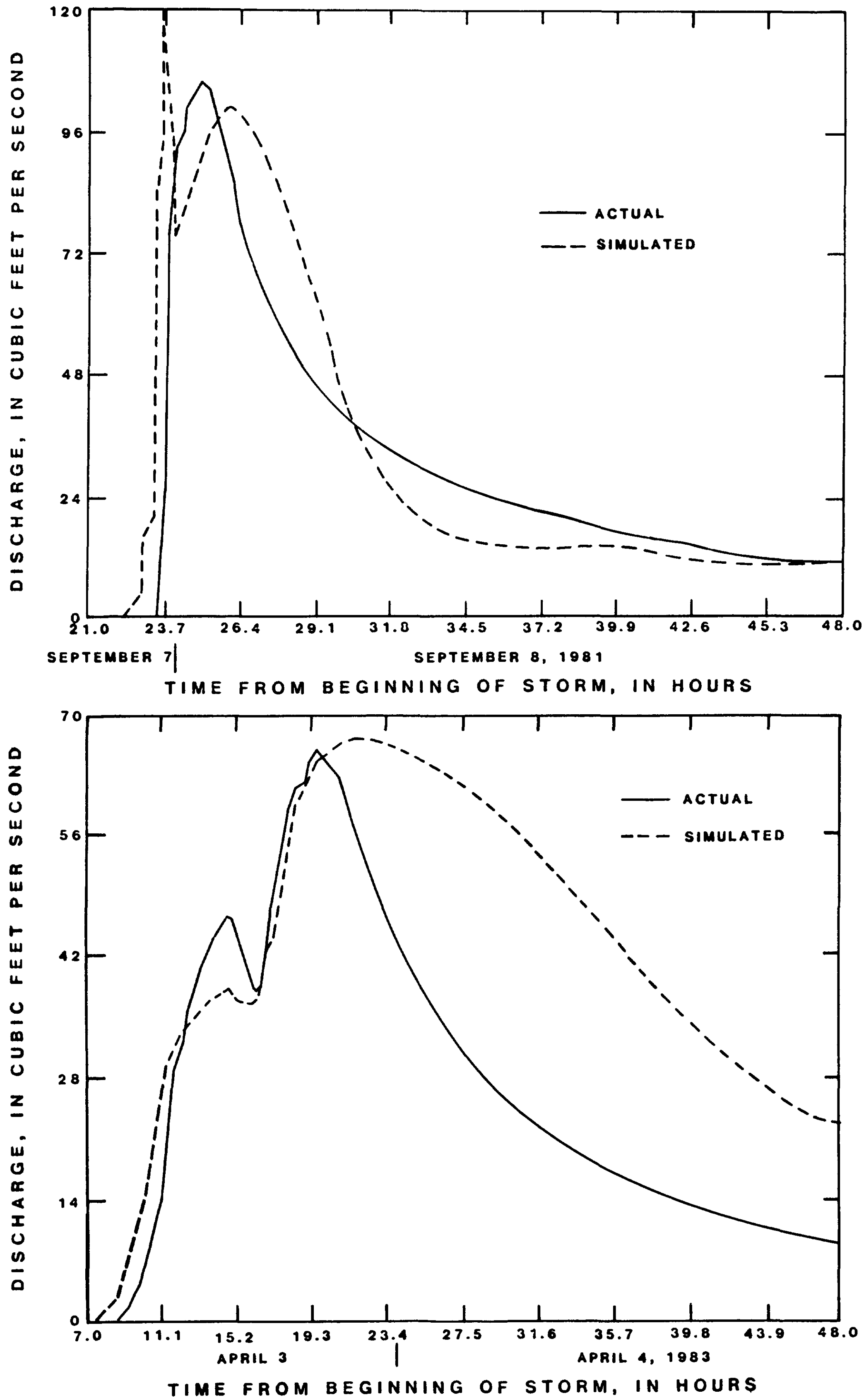
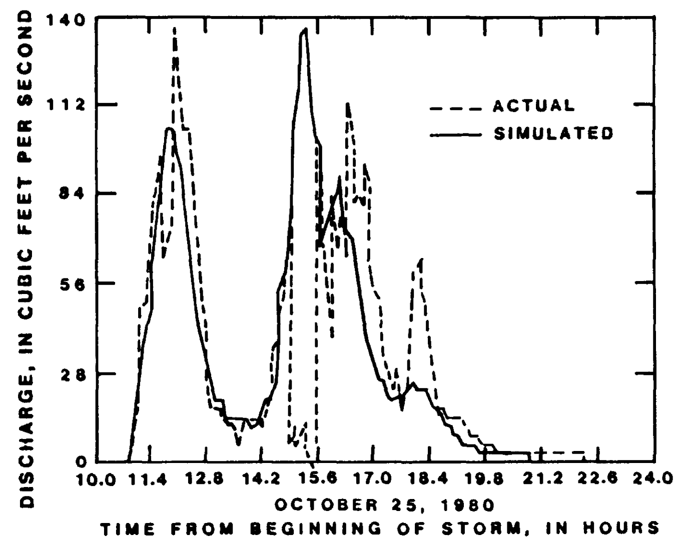
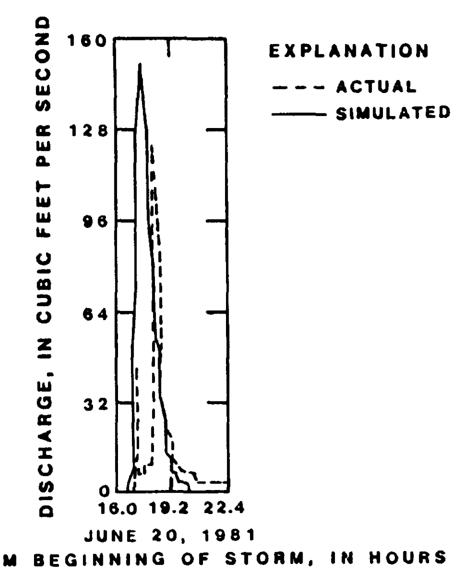


Figure 7.--Actual and simulated discharge of Freshwater Brook at Enfield, station 01183994, for storms of September 7-8, 1981, and April 3-4, 1983.





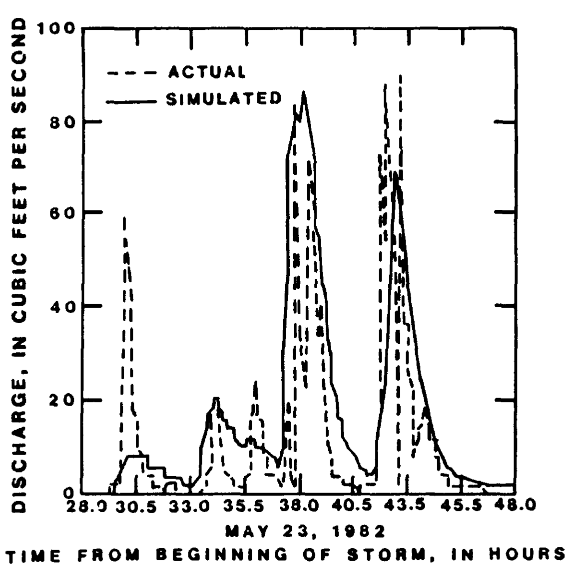
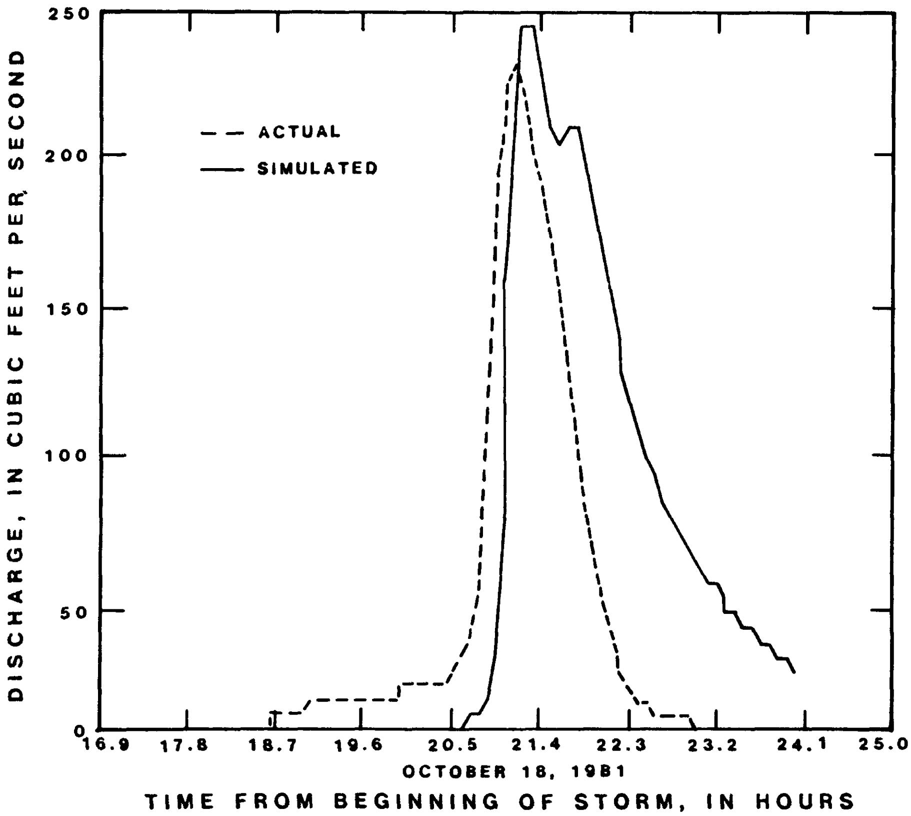


Figure 8.--Actual and simulated discharge of Piper Brook at New Britain, station 01190095, for storms of October 25, 1980, June 20, 1981, and May 23, 1982.



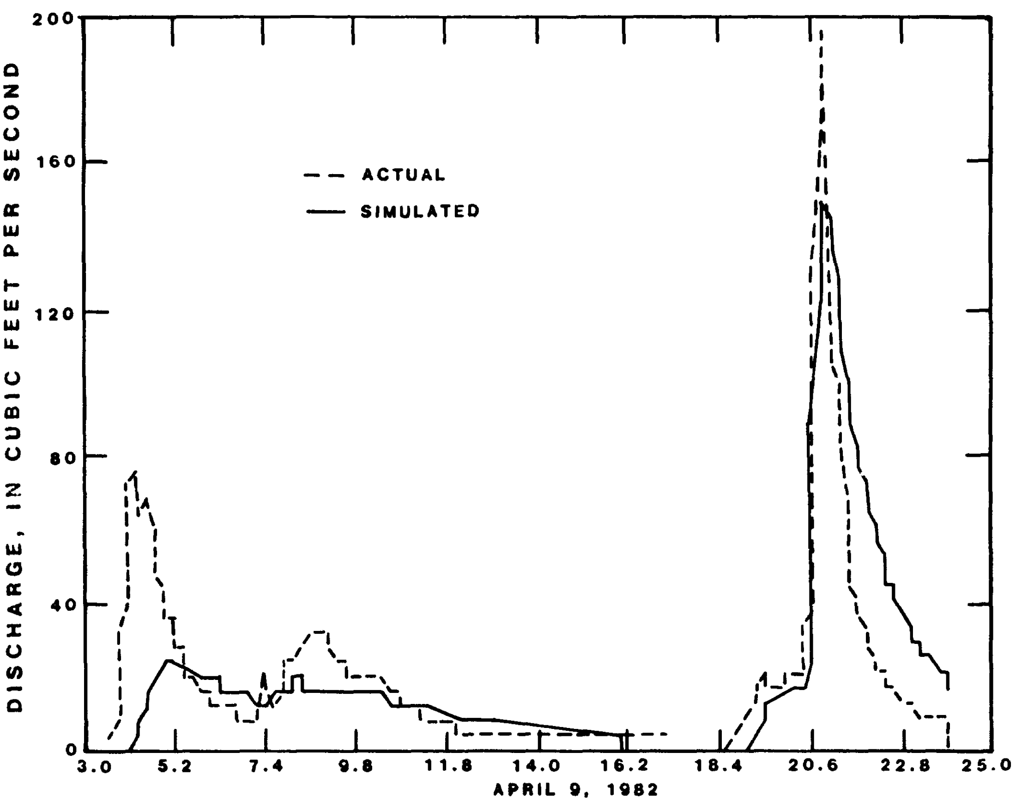


Figure 9.--Actual and simulated discharge of Willow Brook at New Britain, station 01192692, for storms of October 18, 1981 and April 9, 1982.

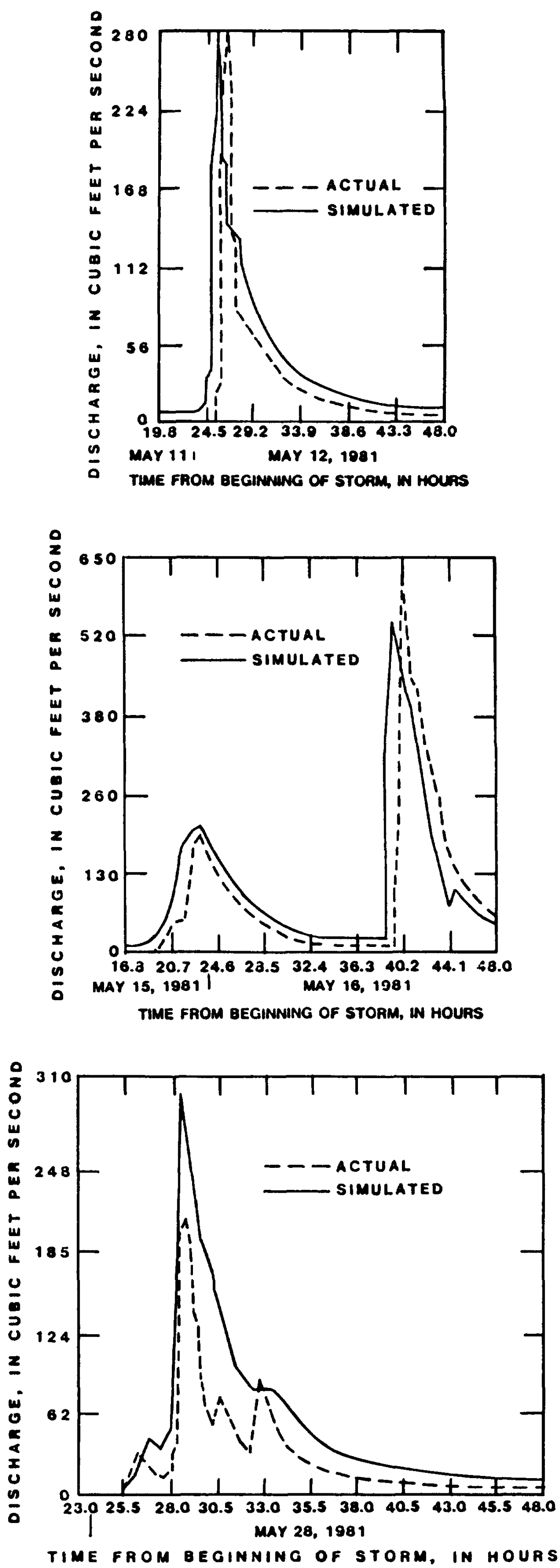


Figure 10.--Actual and simulated discharge of Harbor Brook at Meriden, resulting from runoff generated in the urban area between stations 01196250 and 01196259, for storms of May 11-12, 1981, May 15-16, 1981, and May 28, 1981.

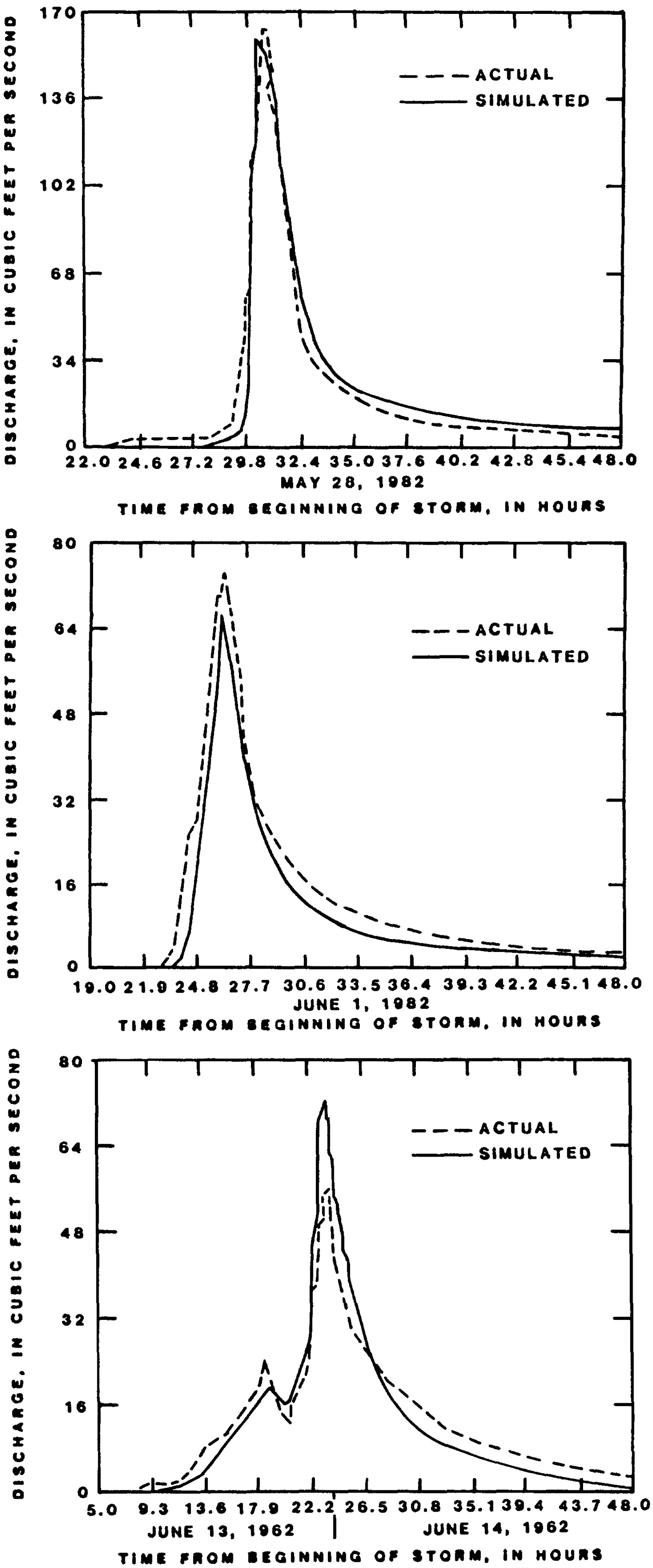


Figure 11.--Actual and simulated discharge of Betts Pond Brook at Norwalk, station 01209753, for storms of May 28, 1982, June 1, 1982, and June 13, 1982.

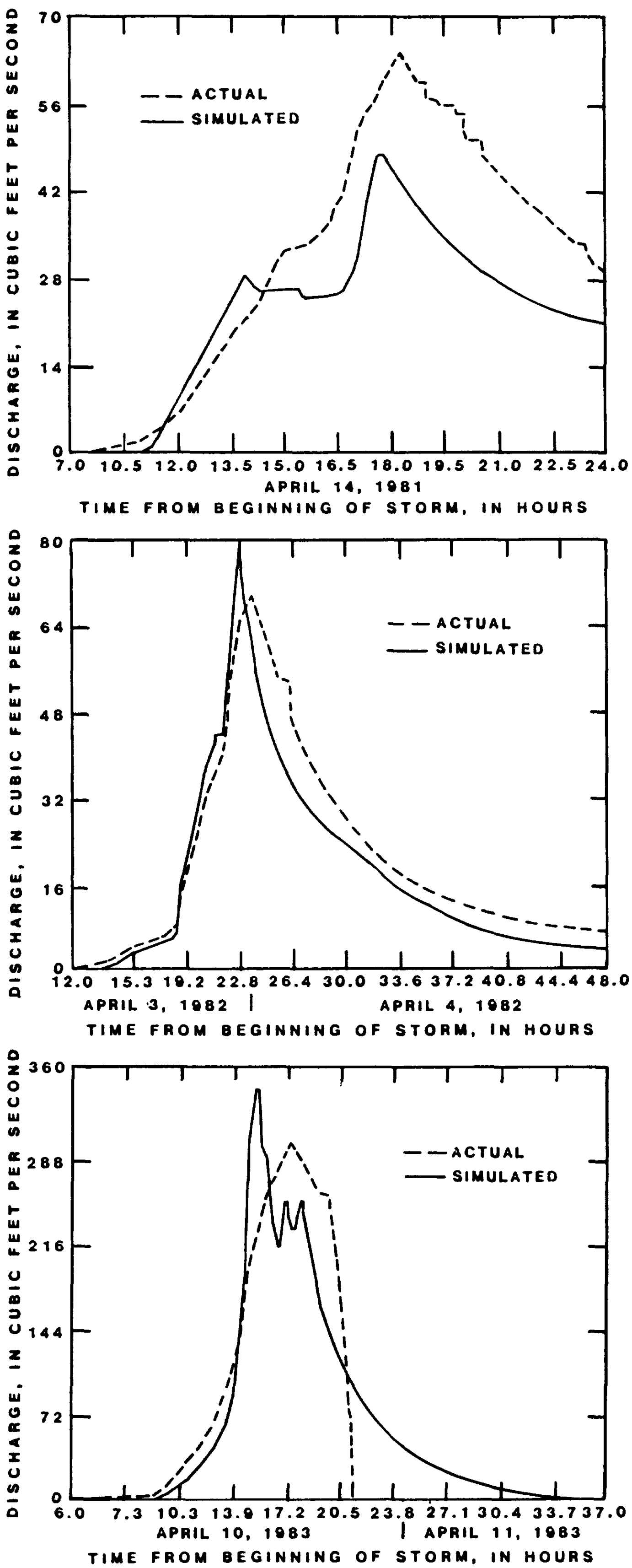


Figure 12.--Actual and simulated discharge of Keelers Brook at Norwalk, station 01209775, for storms of April 14, 1981, April 3-4, 1982, and April 10-11, 1983.

might behave differently because peak-flow timing was retarded as a result of overbank storage.

Peak-flow simulation

Long-term rainfall data are used in the simulation of annual peaks once the calibration and the verification of storm volume and peak flow are satisfied within some optimal range of error as described by Rosenbrock (1960). The given rainfall data for the most intense storms in any year together with soil moisture, infiltration, imperviousness, and timing are used as input to the model. The largest of the simulated peaks within a year is then used as the annual peak.

In order to identify the storm that caused the maximum peak flow in any one year, one must analyze the runoff of many storms and also the prior antecedent soil moisture. For each of the six modeled basins, 5-minute rainfall data from Bradley International Airport in Windsor Locks (fig. 1) were used to simulate 83 storms. Daily rainfall from 1951-80 from three hourly rainfall stations were used as input for the six basins to simulate antecedent, 30-day soil moisture. From 1951-80, data from Bradley International Airport were used for Freshwater Brook at Enfield; data from Shuttle Meadow Reservoir were used for Piper Brook and Willow Brook in New Britain and Harbor Brook in Meriden; and data from Norwalk Gas Plant were used for Betts Pond Brook and Keeler Brook stations in Norwalk. The two stations are within 1 mile of the gas plant. Daily pan-evaporation data were collected from 1951-80 by the Agricultural Experiment Station at the University of Connecticut in Coventry. The daily rainfall at the sites from 1951-80 was assumed to be similar to that at the long-term rain gages.

To test the validity of his using 5-minute storm data from Bradley International Airport to simulate peak flows in six other basins, the author compared rainfall distribution in those parts of the State. Because all the drainages of the six basins are less than 5 mi^2, only storms of short duration had to be inspected for equal magnitude of rainfall for a specific recurrence interval. On maps showing lines of equal rainfall for the 1-hour, 25-year (Weiss, 1975) and 1-hour, 100-year (Frederick and others, 1977) storms in Connecticut, Enfield, New Britain, and Meriden have similar 1-hour rainfalls. The 1-hour, 25-year rainfall at Norwalk is 16 percent higher than that at Bradley International Airport, but the 1-hour, 100-year rainfall at Norwalk is only about 4 percent higher. Therefore, the unit-rainfall data for Bradley International Airport was assumed to be a representative data set for the six basins.

The greatest rainfall for 1951-80 at Bradley International Airport was almost 14 in. on August 19, 1955. At station 01183994 in Enfield (fig. 2), just east of the airport, 9.0 in. of rainfall was recorded during the August 1955 storm. On the basis of a slope-area measurement, FEMA (1979) reported that the peak flow of the 1955 flood at station 01183994 was about 4,000 ft^3/s. This peak flow was generated by 11.5 in. of rain, 22 percent less rain than recorded at Bradley International Airport. A simulation of this storm, based on 14 in. of rainfall in the Freshwater Brook basin, resulted in a peak flow of 6,140 ft^3/s at station 01183994 (table 5) or 53.5 percent higher than that during the 1955 flood. The basin in 1955 was considerably less urbanized than during the period of this study. Therefore, the model seems to be a fairly accurate predictor of peak flows in this basin, in light of the increased rainfall generating the storm and the increased urbanization in the basin.

Another analysis of the August 19, 1955 storm was made on Willow Brook in New Britain (fig. 3). In 1955, 9.2 in. of rain recorded at Shuttle Meadow Reservoir resulted in a peak flow of 1,500 ft^3/s as calculated by a contraction measurement made by Charles Main Engineering Group of Boston, Mass. (Gregory Abrahamian, Town Engineer, City of New Britain, written commun., 1983). The simulated annual peak flow at station 01192692 was 2,600 ft^3/s (table 5) for 14 in. of rain throughout the basin or 73 and 53 percent higher than the measured flow and the rainfall of August 19, 1955. Basin imperviousness and use of storm sewers are more extensive than in 1955; this would increase the peak flow for Willow Brook for a given rainfall. This analysis and the one for station 01183994 at Enfield are included to show, in a qualitative sense, that actual floods are reasonably simulated inasmuch as the simulated floods are based on rainfall at station 415600072411501, at Bradley International Airport. The simulated annual peaks for each site are shown in table 5. During periods of historically extreme flooding, basin storage also tends to decrease the magnitude of the peak flow as streams overflow their banks. Therefore, although the simulated peak flows in table 5 may not be equal to the actual peak flows at the sites, they can be used to estimate the magnitude and the frequency of occurrence of the floods, given that the distribution of rainfall at Bradley International Airport and at these locations are similar and given the 1981-84 degree of urbanization.

Table 5.--*Simulated annual peak flows*

[All flows are in cubic feet per second. Dashes indicate no data]

	Station number							
	01183994		[1]01183994		01190095		[2]01192692	
Water year	Month and day	Peak flow	Month and day	Peak flow	Month and day	Peak flow	Month and day	Peak flow
1951	Mar. 30	307	Mar. 30	208	Mar. 30	559	Mar. 30	887
1952	May 31	89	May 31	65	May 31	128	May 31	144
1953	Mar. 12	385	Mar. 12	218	June 22	632	June 22	607
1954	Sept. 11	1,160	Sept. 11	436	Sept. 11	622	Aug. 3	1,210
1955	Aug. 19	6,140	Aug. 19	1,760	Aug. 19	1,630	Aug. 19	2,600
1956	Oct. 16	1,310	Oct. 16	487	June 2	437	Oct. 16	860
1957	June 26	55	June 26	52	July 9	417	July 9	171
1958	Dec. 20	446	Dec. 20	205	June 26	476	Dec. 20	743
1959	June 15	524	Nov. 28	234	Aug. 10	448	Aug. 10	747
1960	Sept. 12	2,540	Sept. 12	825	Oct. 24	1,510	Oct. 24	2,310
1961	Nov. 29	396	Nov. 29	175	Sept. 2	784	Sept. 2	931
1962	Aug. 17	416	Aug. 17	235	Aug. 17	801	Aug. 17	1,580
1963	Sept. 29	188	Sept. 29	158	June 28	475	June 28	503
1964	Jan. 20	283	Jan. 20	138	Aug. 12	351	Jan. 20	588
1965	Feb. 25	580	Feb. 25	274	Feb. 25	320	Feb. 25	798
1966	July 10	186	July 10	144	July 10	899	July 10	959
1967	May 25	125	May 25	91	May 25	123	May 25	177
1968	Aug. 9	133	Aug. 9	105	Aug. 9	500	Aug. 9	753
1969	Apr. 22	707	Apr. 22	312	Aug. 4	660	Aug. 4	1,030
1970	Apr. 2	556	Apr. 2	250	June 3	443	June 3	485
1971	Aug. 27	110	Aug. 27	101	Sept. 11	440	Sept. 11	320
1972	Oct. 10	285	Oct. 10	180	Aug. 27	767	Aug. 27	973
1973	June 30	482	June 30	198	Aug. 31	890	Aug. 31	1,040
1974	Sept. 29	879	Sept. 29	325	Sept. 20	628	Sept. 20	842
1975	Sept. 26	1,610	Sept. 26	563	Sept. 26	1,030	Sept. 26	1,560
1976	Apr. 1	813	Apr. 1	339	July 23	611	Apr. 1	1,120
1977	Mar. 22	209	Mar. 22	127	Mar. 22	206	Mar. 22	472
1978	Jan. 25	268	Jan. 25	159	Sept. 12	240	Jan. 25	317
1979	Jan. 24	190	Jan. 24	110	Oct. 6	245	Jan. 24	376
1980	Apr. 9	581	Oct. 3	284	Oct. 3	1,250	Oct. 3	2,030
[5]1981	Feb. 24	720	--	--	--	--	Apr. 9	460
[5]1982	June 6	575	--	--	--	--	June 6	1,100
[5]1983	Apr. 19	465	--	--	--	--	Apr. 24	284
[5]1984	May 31	520	--	--	--	--	May 29	138

[1] For urban area of 3.0 square miles between stations 01183993 and 01183994.

[2] For urban area of 3.29 square miles downstream from station 01192689.

[3] For urban area of 3.58 square miles between stations 01196250 and 01196259.

[4] For drainage of 11.9 square miles.

[5] Peaks observed during study.

Table 5.--*Simulated annual peak flows* -- Continued

Station number

Water year	[3]01196259 Month and day	Peak flow	[4]01196259 Month and day	Peak flow	01209753 Month and day	Peak flow	01207775 Month and day	Peak flow
1951	Mar. 30	832	Mar. 30	832	Mar. 30	751	Mar. 30	513
1952	June 1	138	May 31	162	May 31	50	May 31	63
1953	June 22	451	June 22	650	June 22	312	June 22	191
1954	Aug. 3	828	Sept. 11	2,390	Sept. 11	629	Sept. 11	454
1955	Aug. 19	2,330	Aug. 19	8,160	Aug. 19	1,650	Aug. 19	1,310
1956	Oct. 16	510	Oct. 16	1,640	Oct. 16	455	Oct. 16	366
1957	June 26	352	July 9	439	June 26	55	June 26	77
1958	June 26	549	Dec. 20	870	June 26	382	June 26	267
1959	Aug. 10	543	Nov. 28	914	Aug. 10	459	Aug. 10	386
1960	Oct. 24	1,490	Oct. 24	5,190	Oct. 24	899	Sept. 12	756
1961	Sept. 2	679	Nov. 29	1,210	Sept. 2	390	Sept. 2	276
1962	Aug. 17	1,030	Aug. 17	1,170	Aug. 17	636	Aug. 17	408
1963	Sept. 12	430	Sept. 12	500	Sept. 29	182	Sept. 29	149
1964	Jan. 20	348	Jan. 20	531	Jan. 20	187	Aug. 12	75
1965	Feb. 25	424	Feb. 25	972	Feb. 25	365	Feb. 25	223
1966	July 10	904	July 10	1,060	July 10	397	July 10	278
1967	May 25	166	May 25	194	May 25	78	May 25	68
1968	Aug. 9	607	Aug. 9	656	Aug. 9	291	Aug. 9	200
1969	Aug. 4	548	Aug. 4	1,170	Aug. 4	384	Aug. 4	285
1970	June 3	289	Apr. 2	620	Apr. 2	229	Apr. 2	161
1971	Sept. 12	446	Sept. 11	523	Sept. 11	229	Sept. 11	147
1972	Aug. 27	592	Aug. 27	806	Oct. 10	216	Oct. 10	171
1973	Aug. 31	878	Aug. 31	1,050	Aug. 31	459	Aug. 31	308
1974	Sept. 20	791	Sept. 29	1,430	Sept. 20	509	Sept. 20	329
1975	Sept. 26	671	Sept. 26	4,020	Sept. 26	838	Sept. 26	458
1976	July 23	708	Apr. 1	1,640	Apr. 1	476	Apr. 1	338
1977	Feb. 24	320	Mar. 22	395	Mar. 22	163	Mar. 22	113
1978	Jan. 25	258	Jan. 25	329	Jan. 25	119	Jan. 25	131
1979	Jan. 25	566	Jan. 25	1,880	Jan. 24	129	Jan. 24	97
1980	Oct. 3	1,580	Oct. 3	1,600	Apr. 10	228	Apr. 10	265
[5]1981	May 16	312	May 16	638	--	--	--	--
[5]1982	June 6	607	June 6	1,350	--	--	--	--
[5]1983	Apr. 10	121	Apr. 10	1,110	Apr. 10	444	Apr. 10	306
[5]1984	--	--	July 7	512	July 7	224	July 7	189

Peak-Flow Magnitude and Frequency

The data in table 5 were analyzed by use of the log-Pearson Type III technique and guidelines set by the U.S. Water Resources Council (1981). Results of the frequency analysis shown in table 6 include frequency data for three long-term urban sites in Connecticut. High and low outliers in the analysis were handled according to procedures recommended by the U.S. Water Resources Council (1981). The results in table 6 for Freshwater and Harbor Brooks are shown for their total drainages (rural and urban) and for just the downstream urban areas. The peak flows modeled for the period of study were in the range of a 5- to 25-year recurrence interval.

Estimating Procedures for Ungaged Urban Sites

In evaluating the effects of urbanization on peak flows for each basin, the author first computed magnitude and frequency of flow from simulated flows. These data were then compared with the peak flows computed by use of regression equations (Weiss, 1983, p. 16) that were derived from peak-flow data for 96 rural basins in Connecticut (see table 7).

The U.S. Geological Survey, in cooperation with the Connecticut Department of Environmental Protection (DEP), the U.S. Army Corps of Engineers, and many towns in Connecticut, maintains 46 continuous-record gaging stations. The records of 60 percent of these stations exceed 25 years. The

Table 6.--*Rainfall and simulated peak flows for selected recurrence intervals*

Station	Rainfall, in inches, for, indicated recurrence interval in years[1]				Peak flow, in cubic feet per second, for indicated recurrence interval, in years			
number	2	10	50	100	2	10	50	100
[2]01183994	2.55	5.15	8.8	11.5	227	542	918	1,110
[3]01183994	2.55	5.15	8.8	11.5	397	1,160	2,190	2,730
01190095	2.62	4.40	7.5	8.2	544	1,120	1,610	1,810
[4]01190100	2.55	4.35	7.25	8.0	731	1,740	3,290	4,200
[4]01190200	2.60	4.30	7.0	8.0	178	429	814	1,040
[4]01191000	2.50	4.80	8.0	10.5	1,100	2,630	5,050	6,520
[5]01192692	2.65	4.50	7.5	8.5	679	1,560	2,370	2,720
[6]01192659	2.68	4.50	7.75	9.5	547	1,200	1,840	2,130
[7]01196259	2.68	4.50	7.75	9.5	874	2,310	4,230	5,260
01209753	3.05	5.25	7.75	9.5	320	787	1,250	1,450
01209775	3.05	5.20	7.5	9.0	227	567	998	1,220

[1] From Weiss (1975).
[2] For drainage of 3.00 square miles between Elm and Enfield Streets.
[3] For drainage of 11.3 square miles.
[4] Long-term gaging station.
[5] Area above Shuttle Meadow Reservoir is excluded.
[6] For drainage of 3.58 square miles between Westfield Road and Bradley Avenue.
[7] For drainage of 22.9 square miles.

Table 7.--*Regression equations used for computing peak flows in rural basins in Connecticut*

[From Weiss, 1983. Symbols ≥ and <, "greater than or equal to" and "less than"; Q_x, peak flow, in cubic feet per second, for indicated recurrence interval, in years; A, drainage in square miles; I_x, 24-hour rainfall, in inches, for indicated recurrence interval, in years; L, main-channel stream length, in miles; S_m, main channel slope, in feet per mile (Benson, 1962); $\%A_{sd}$, percent of drainage underlain by coarse-grained stratified drift; S, standard error of estimate, in percent; Y, true average error of prediction, in percent]

Drainage area ≥100 square miles (14 sites)	Drainage area ≥10 and <100 square miles (41 sites)	Drainage area <10 square miles (41 sites)	All drainage areas (96 sites)
$Q_2 = \frac{25.6(A)^{0.85} \times (I_2)^{1.95}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.07} \times (\% A_{sd} + 1)^{0.46}}$ S = ±20.4 Y = ±26.6	$Q_2 = \frac{7.7(A)^{1.05} \times (I_2)^{1.74}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.26} \times (\% A_{sd} + 1)^{0.16}}$ S = ± 32.3 Y = ± 35.6	$Q_2 = \frac{8.1(A)^{0.88} \times (I_2)^{2.14}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.05} \times (\% A_{sd} + 1)^{0.2}}$ S = ± 45.3 Y = ± 44.4	$Q_2 = \frac{7.6(A)^{0.97} \times (I_2)^{2}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.17} \times (\% A_{sd} + 1)^{0.2}}$ S = ± 36.7
$Q_{10} = \frac{23.4(A)^{0.85} \times (I_{10})^{2.17}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.22} \times (\% A_{sd} + 1)^{0.51}}$ S = ± 30.1 Y = ± 38.5	$Q_{10} = \frac{5.6(A)^{1.05} \times (I_{10})^{1.98}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.26} \times (\% A_{sd} + 1)^{0.22}}$ S = ± 36.0 Y = ± 36.7	$Q_{10} = \frac{12.8(A)^{0.89} \times (I_{10})^{1.6}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.06} \times (\% A_{sd} + 1)^{0.17}}$ S = ± 46.4 Y = ±43.6	$Q_{10} = \frac{6.6(A)^{1.0} \times (I_{10})^{1.89}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.22} \times (\% A_{sd} + 1)^{0.19}}$ S = ± 39.2
$Q_{25} = \frac{44.7(A)^{0.87} \times (I_{25})^{1.91}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.23} \times (\% A_{sd} + 1)^{0.63}}$ S = ± 26.4	$Q_{25} = \frac{16.2(A)^{1.03} \times (I_{25})^{1.41}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.23} \times (\% A_{sd} + 1)^{0.26}}$ S = ± 39.6	$Q_{25} = \frac{63.1(A)^{0.89} \times (I_{25})^{0.71}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.06} \times (\% A_{sd} + 1)^{0.17}}$ S = ± 48.8	$Q_{25} = \frac{21.2(A)^{1.0} \times (I_{25})^{1.21}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.23} \times (\% A_{sd} + 1)^{0.2}}$ S = ± 42.2
$Q_{50} = \frac{22.7(A)^{0.96} \times (I_{50})^{2.2}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.29} \times (\% A_{sd} + 1)^{0.74}}$ S = ± 28.9 Y = ± 38.7	$Q_{50} = \frac{22.3(A)^{1.04} \times (I_{50})^{1.29}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.25} \times (\% A_{sd} + 1)^{0.3}}$ S = ± 41.6 Y = ± 42.5	$Q_{50} = \frac{72.1(A)^{0.92} \times (I_{50})^{0.67}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.09} \times (\% A_{sd} + 1)^{0.17}}$ S = ± 50.5 Y + ± 48.6	$Q_{50} = \frac{23.2(A)^{1.03} \times (I_{50})^{1.14}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.26} \times (\% A_{sd} + 1)^{0.2}}$ S = ± 44.2
$Q_{100} = \frac{39.4(A)^{0.99} \times (I_{100})^{1.91}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.38} \times (\% A_{sd} + 1)^{0.75}}$ S = ± 31.8 Y = ± 43.2	$Q_{100} = \frac{35.9(A)^{1.07} \times (I_{100})^{1.1}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.24} \times (\% A_{sd} + 1)^{0.34}}$ S = ± 45.4 Y = ± 47.7	$Q_{100} = \frac{73.5(A)^{0.91} \times (I_{100})^{0.74}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.08} \times (\% A_{sd} + 1)^{0.18}}$ S = ± 51.3 Y = ± 50.3	$Q_{100} = \frac{28.7(A)^{1.04} \times (I_{100})^{1.08}}{\left(\frac{L}{\sqrt{S_m}}\right)^{0.27} \times (\% A_{sd} + 1)^{0.22}}$ S = ± 46.8

drainage basins of these mostly rural stations exceed 10 mi^2. Additionally, in 1960, the Geological Survey and the CDOT began a peak-flow study at 50 rural drainage basins ranging in size from 1 to 10 mi^2. Regression equations for computing peak flows that were developed from the stream-gaging data and the associated standard errors of estimate are shown in table 7 under the heading "All Drainage". The equations were used to compute the floods with recurrence intervals ranging from 2 to 100 years for each urban basin in this study. Ratios of urban to rural peak flow for each area were obtained from a comparison of the simulated peak flow for each recurrence interval with the peak flows calculated by use of the equations for rural areas. These ratios can be used to adjust flood frequency for the effects of urbanization.

The ratios of urban to rural peak flow for each recurrence interval at nine sites were plotted against the percentage of the basin drained by storm sewers at each site by graphical balancing of points. The sites used were the six modeled drainage basins and three urban basins with more than 25 years of peak-flow data in north-central Connecticut. However, the nine data points available are insufficient to show the effects of urbanization in a general flood-frequency regression analysis. The ratios of flood-frequency values for urban areas to flood-frequency estimates, based on regression equations for rural areas for the same basins, increase as urbanization increases where more than 30-percent of an individual area is served by storm sewers. The results are shown in figures 13-16. For example, the ratios for the 2-year recurrence interval at specific sites ranged from about 1.5 where 33 percent of the area is served by storm sewers to 6.1 where 90 percent of the area is served by storm sewers.

Because the average error of estimate was known for the equations for rural areas (table 7) and the sample size of 96 was large, the ratios of urban to rural peak flows could be compared to the standard error of estimate of the regression equations and tested for the 95-percent confidence limit. The results are shown in table 8. Those ratios that exceed the 95-percent confidence limits (within the boundary lines in table 8) are significantly different from the average error of estimate of the equations for rural areas. Therefore, flood frequencies for urban areas with extensive parts of the basin served by storm sewers need to be adjusted. The ratios listed in table 8 and the equations in table 7, can be used to adjust peak flows for the effects of storm sewers for a given recurrence interval, if no other data are available. They should however be used only in those parts of Connecticut where the ranges of the parameters used in the model, such as imperviousness of 0 to 33 percent, drainage 2.0 to 25 mi^2, and percentage of stratified drift less than 45, are not exceeded. Furthermore, all the study sites, except for Freshwater Brook, are west of the Connecticut River, and the Freshwater Brook site is within 1 mile of the river. Rainfall distribution east of the Connecticut River may not be similar to the distribution west of the river.

Data used to calibrate and to verify models were for floods with lower peak flows than the floods simulated during record extensions. To assess how well such larger floods were simulated, the author compared the model-simulated peak flows with those computed by a three-parameter regression equation used for urban areas in the United States (Sauer and others, 1983). The equations used for the 2-and 100-year floods were--

$$UQ_2 = 13.2A^{0.21}(13 - BDF)^{-0.43}\, Q_2^{0.73}, \tag{15}$$

and

$$UQ_{100} = 7.70A^{0.15}(13 - BDF)^{-0.32}\, Q_{100}^{0.82}, \tag{16}$$

where

UQ_2	is	the 2-year flood for urban basin, in cubic feet per second;
UQ_{100}	is	the 100-year flood for urban basin, in cubic feet per second;
A	is	the drainage area, in square miles;
BDF	is	an index of urbanization;
Q_2	is	the 2-year flood for rural basin computed from regression equation in table 7, in cubic feet per second; and
Q_{100}	is	the 100-year flood for rural basin computed from regression equation in table 7, in cubic feet per second.

The simulated and the computed results and the associated errors are shown in table 9. The average positive and negative percentage errors, as well as individual errors, are well within the national average standard errors of estimate for the regression equations (15) and (16).

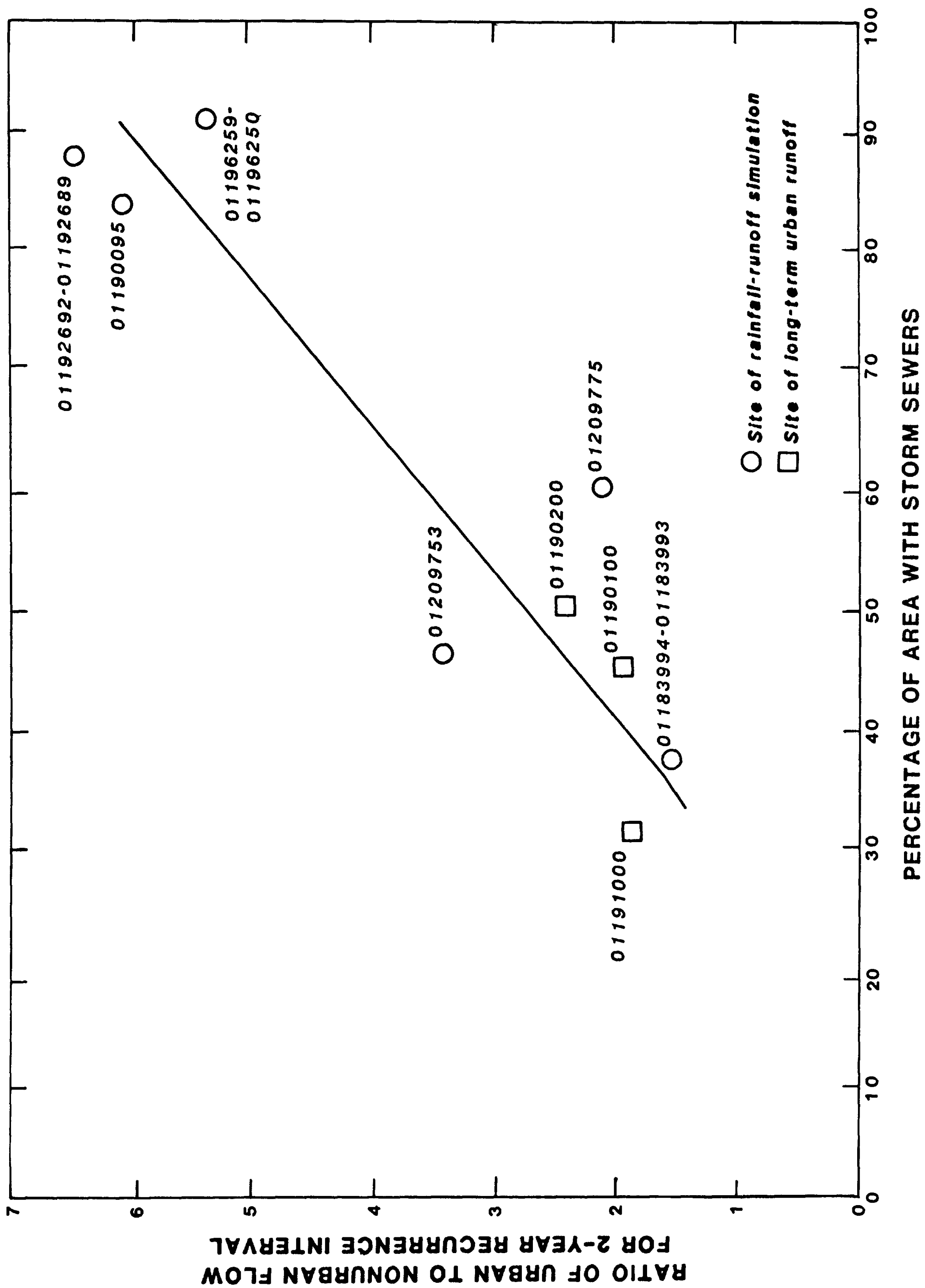


Figure 13.--Relation between floods of a 2-year recurrence interval and percentage of area with storm sewers.

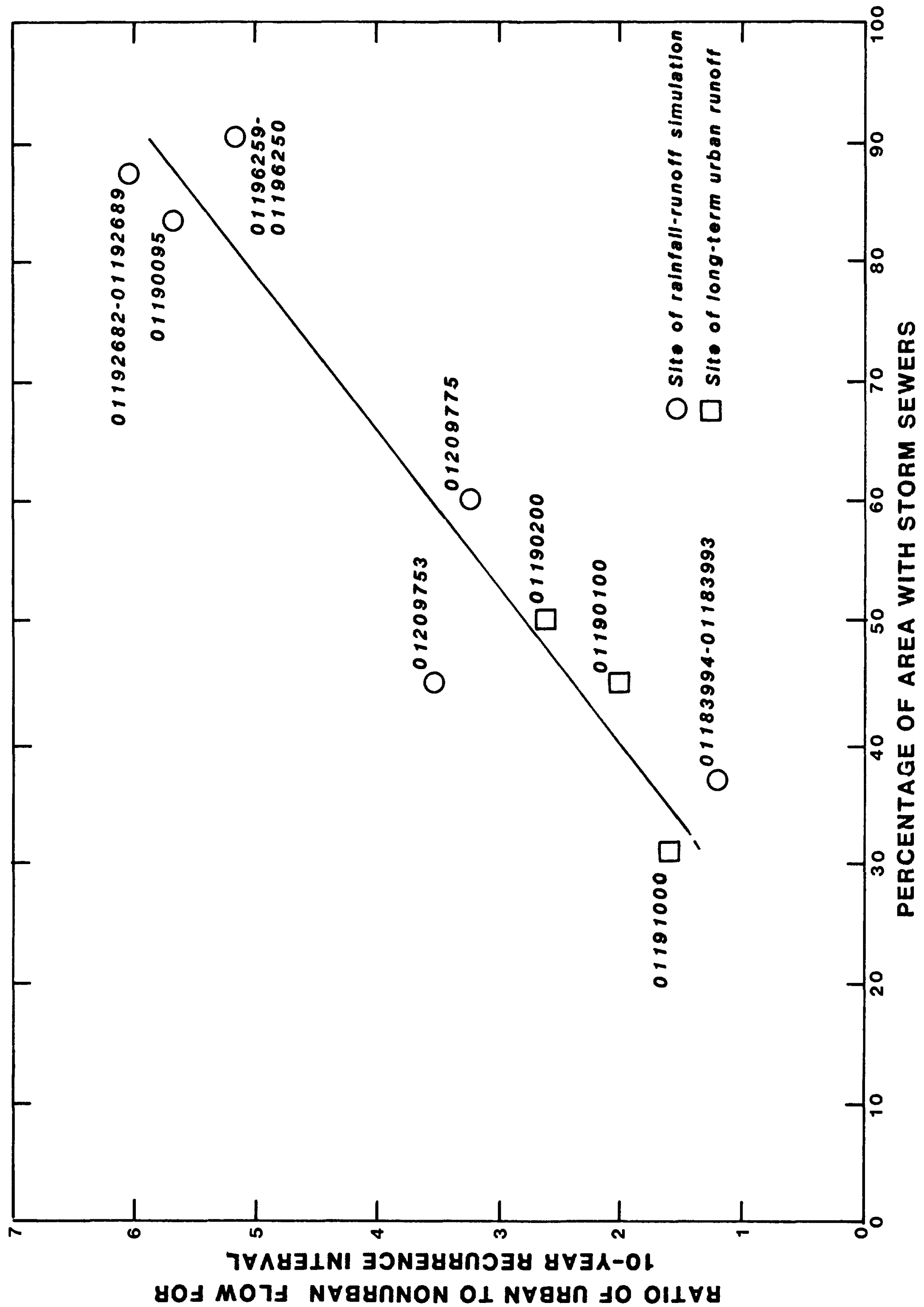


Figure 14.--Relation between floods of a 10-year recurrence interval and percentage of area with storm sewers.

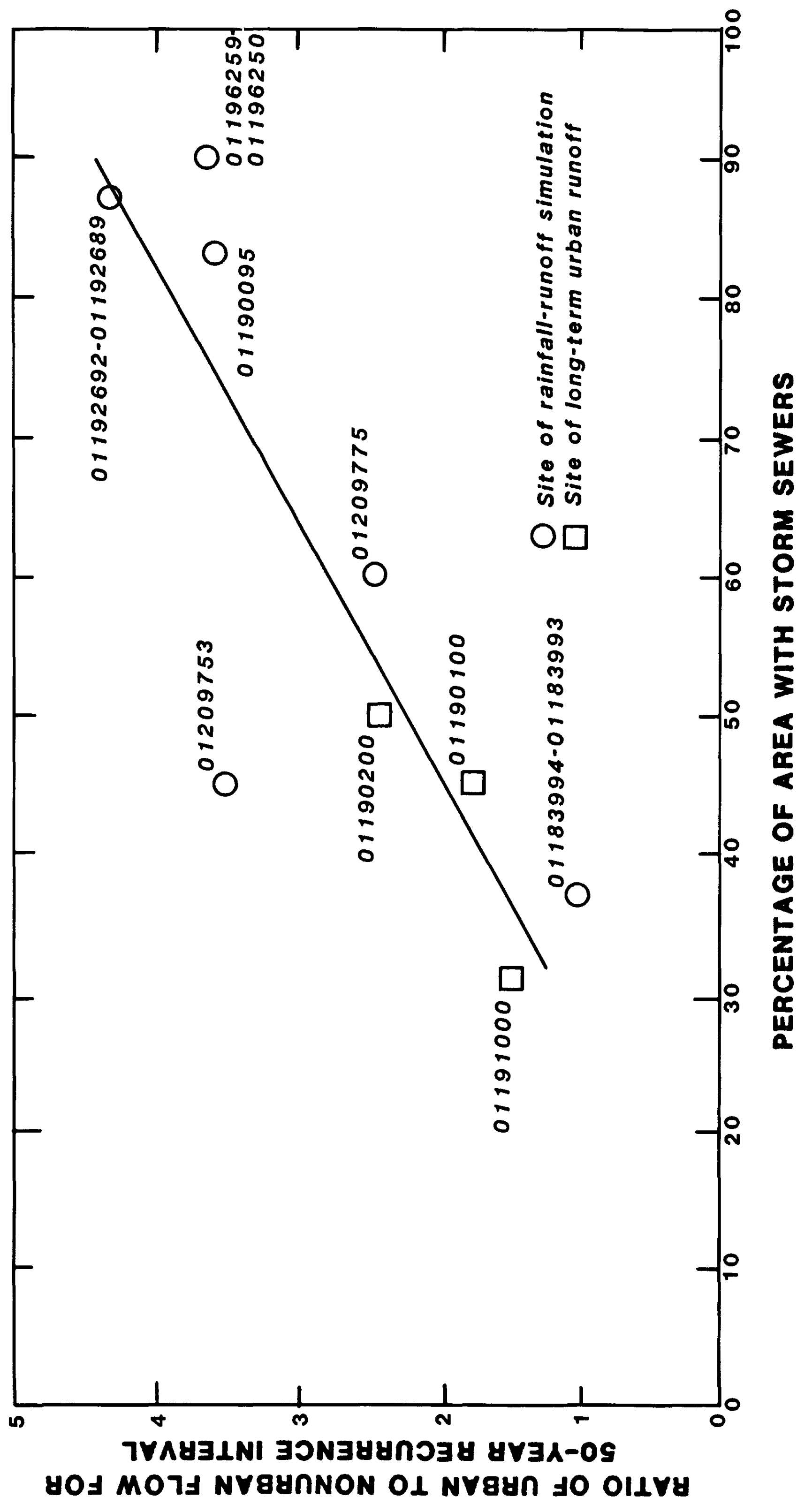


Figure 15.--Relation between floods of a 50-year recurrence interval and percentage of area with storm sewers.

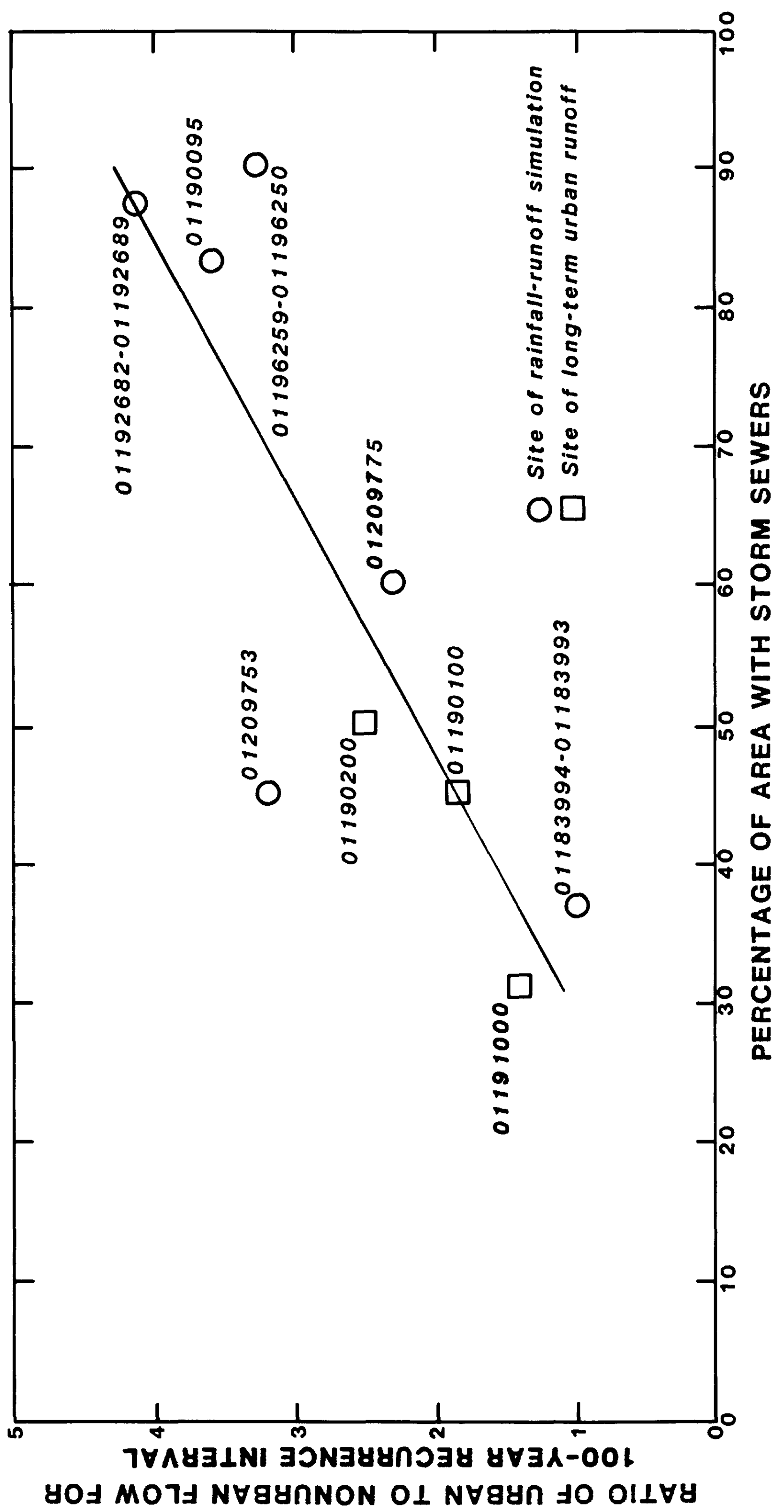


Figure 16.--Relation between floods of a 100-year recurrence interval and percentage of area with storm sewers.

Table 8.--*Statistical significance of urbanization on peak flows*

[The enclosed values on this table are outside the 95-percent confidence limits of the peak-flow regression equations developed for rural areas by Weiss (1983)]

Percentage of basin with storm sewers	Ratio of urban to rural peak flow for indicated recurrence interval			
	2-year	10-year	50-year	100-year
30	1.35	1.3	1.2	1.1
40	2.00	2.0	1.70	1.6
50	2.8	2.8	2.25	2.15
60	3.65	3.6	2.8	2.7
70	4.45	4.35	3.35	3.2
80	5.3	5.15	3.9	3.75
90	6.1	5.9	4.45	4.3

SUMMARY AND CONCLUSIONS

The relation of urbanization and peak flows was studied for six streams in four communities in Connecticut. Data were collected from July 1980 through September 1984. Seven streamflow gages and six rain gages were used to record data as input to the U.S. Geological Survey's distributed-routing rainfall-runoff model DR3M. Data for about 17 of 35 storms per station were used to calibrate and verify the model.

Long-term data used in simulating annual peak flows for 1951-80 for the six streams included daily pan evaporation from the University of Connecticut Agricultural Experimental Station in Coventry; daily rainfall from Bradley International Airport in Windsor Locks, Shuttle Meadow Reservoir in New Britain, and the Norwalk Gas Plant in Norwalk; and 5-minute rainfall from Bradley International Airport. Model DR3M was used to simulate annual peak flows for each site, and the log-Pearson Type-III method and guidelines of the U.S. Water Resources Council (1981) were used to determine magnitude and frequency of flows. These data and similar data for the three long-term stations were compared to data from peak flows computed with regression equations for rural sites. The ratio of simulated urban peak flows to computed rural peak flows for the 2-year flood ranged from about 1.5 where at least 30 percent of the area is served by storm sewers to 6.1 in areas where at least 90 percent of the area is served by storm sewers. For the 100-year flood, the ratios ranged from 1.1 where at least 30 percent of the area is served by storm sewers to 4.3 where at least 90 percent of the area is served by storm sewers.

In a comparison of the ratios, based on the 95-percent confidence limits of the standard error of estimate and the regression equations for rural areas as the criteria, the author concluded that when a site is extensively served by storm sewers and falls outside the 95-percent confidence limits, then the peak flows need to be adjusted for urbanization. When the simulated peak flows with 2- and 100-year recurrence intervals were compared to the peak flows of similar recurrence intervals that were computed using regression equations developed in a national study, the observed differences for the 2-year recurrence interval ranged from −25.9 to +15 percent and for the 100-year recurrence interval ranged from −31.1 to +34.9 percent.

Table 9.--*Peak flows determined by simulation and those computed by use of regression equations for urban areas in the United States*

[BDF, basin development factor; mi^2, square miles; ft^3/s, cubic feet per second]

Station number	Drainage (mi^2)	BDF	Rural areas[1]: Peak flow, 2-year recurrence interval (ft^3/s)	Rural areas[1]: Peak flow, 100-year recurrence interval (ft^3/s)	Urban areas: Peak flow, 2-year recurrence interval (ft^3/s) simulated	Urban areas: Peak flow, 2-year recurrence interval (ft^3/s) computed[2]	Urban areas: Peak flow, 100-year recurrence interval (ft^3/s) simulated	Urban areas: Peak flow, 100-year recurrence interval (ft^3/s) computed[3]	Difference in computed and simulated peak-flow values (percent): 2-year recurrence interval	Difference in computed and simulated peak-flow values (percent): 100-year recurrence interval
01183993 to 01183994	3.00	4	139	1,247	227	237	1,110	1,497	+4.4	+34.9
01190095	2.34	12	113	691	544	498	1,810	1,863	−8.5	+2.9
[4]01192692	3.29	23	104	654	679	503	2,720	1,874	−25.9	−31.1
01196250 to 01196259	3.58	12	102	646	547	505	2,130	1,879	−7.7	−11.8
01209753	2.40	9	92	460	320	237	1,450	1,349	−25.9	−7.0
01209775	2.25	9	107	428	227	261	998	802	+15.0	+19.6

[1] Computed by use of equations from table 7 under heading "All drainage."
[2] Computed by use of equation (15). Average standard error = ± 43 percent.
[3] Computed by use of equation (16). Average standard error = ± 46 percent.
[4] Area upstream from Shuttle Meadow Reservoir is excluded.

Use of the modeling technique in Connecticut is (1) limited by sensitivity of the model to the intricate pipe-flow systems present in the basins studied; (2) confined to the ranges in the percent of impervious cover and percent of the area underlain by coarse-grained stratified drift of the drainage basins studied; (3) limited by effects of frozen ground conditions on peak flows; and (4) limited to a lesser extent, because modeling in this study was confined to floods with less than a 25-year recurrence interval, when basin storage problems were not evident.

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GLOSSARY OF SELECTED TERMS

A. The contributing drainage, in square miles.

Base flow during a storm. That component of runoff not attributed to overland flow.

Basin development factor (BDF). An index of urbanization from a nationwide study (Sauer and others, 1983) that provides a measure of the efficiency of the drainage system. The basin is subdivided into three equal areas of upper, middle, and lower sections. Within each section, four aspects of the drainage system are evaluated and are assigned a code as follows:

1. **Channel improvements.** If channel improvements such as straightening, enlarging, deepening, and clearing are prevalent for the main drainage channels and principal tributaries (those that drain directly into the main channel), a code of one (1) is assigned. Prevalent means that at least 50 percent of the main drainage channels and principal tributaries must be improved to some degree over natural conditions. If channel improvements are not prevalent, then a code of zero (0) is assigned.

2. **Channel linings.** If more than 50 percent of the main drainage channels and principal tributaries have been lined with an impervious material, such as concrete, a code of one (1) is assigned to the lining. If less than 50 percent of a channel is lined, a zero (0) is assigned to the channel. A channel lining is indicative of channel improvements as well.

3. **Storm drains or storm sewers.** Storm drains are enclosed drainage structures (usually pipes), that are used on the secondary tributaries where the drainage is received directly from streets or parking lots. Many (some) of these drains empty into the main tributaries and channels that are either open channels or, in some basins, are also enclosed in box or pipe culverts. Where more than 50 percent of the secondary tributaries within a subarea (one-third of basin) consist of storm drains, a code of one (1) is assigned to the drains, and conversely, if less than 50 percent of the secondary tributaries consist of storm drains, then a code of zero (0) is assigned. If 50 percent or more of the main drainage channels and principal tributaries are enclosed, channel improvements and channel linings would also be assigned a code of one (1).

4. **Curb and gutter streets.** If more than 50 percent of a subarea (third of total area) is urban [residential, commercial, and (or) industrial development], and if more than 50 percent of the streets and highways in the subarea are constructed with curbs and gutters, a code of one (1) should be assigned. Otherwise, assign a code of zero (0). Frequently, drainage from curb and gutter streets will empty into storm drains.

The values of the codes assigned to items 1 through 4 are summed to obtain the total basin development factor (BDF). The range of BDF is 0 to 12.

Channel length (L), for the basin as determined from Geological Survey maps, in miles. The distance from the gaged site upstream to the watershed divide along the most well-defined and longest channel.

Channel slope (S), for the basin as determined from Geological Survey topographic maps, in feet per mile. The difference in elevation, in feet, at points 10 percent and 85 percent of the distance upstream from the gaged site along the main channel divided by the distance, in miles, along the channel between the two points.

Confidence limit. A way of indicating the reliability of an estimate. A 95-percent confidence limit means there is a 95-percent chance that the estimate lies within the prescribed limits.

Digital model parameters. The following selected acronyms pertain to parameters in the Geological Survey distributed-flow routing model DR3M:

IA. Impervious area, in percentage of total area.

EVC. Pan coefficient used in converting pan evaporation to potential evapotranspiration.

RR. Coefficient used in calculating the amount of daily rainfall that infiltrates the soil.

BMSN. Maximum effective soil-moisture storage at field capacity, in inches.

PSP. Capillary potential, or soil suction, at wetting front for field capacity, in inches.

RGF. Ratio of suction at the wetting front for soil moisture at the wilting point to suction at field capacity.

KSAT. Effective saturated hydraulic conductivity, for use in determining infiltration rates, in inches per hour.

RAT. Ratio of the sum of the pervious and noneffective impervious areas to the pervious area.

Effective impervious area. The area, as a percentage of total drainage, linked hydraulically to the stream and impervious to the infiltration of rain.

Exceedance probability. Probability that a random event will exceed a specific magnitude in a given time period. For example, a flood with a 0.01-exceedance probability is a flood that has one chance in a hundred of being exceeded in any year. This is a 100-year flood under the "recurrence-interval" terminology.

Lag time. The time from beginning (or center of mass) of rainfall to peak (or center of mass) of runoff.

Mapped impervious area. Drainage impervious to the infiltration of rain, in percentage of total drainage. Includes areas such as paved roads, paved parking lots, roofs, driveways, and sidewalks. Impervious area was determined from Geological Survey maps (1:24,000) and town-sewer maps (1 inch = 600 ft).

Overland flow. The flow of water over the land surface toward stream channels.

$\%A_{sd}$. Percentage of total drainage that is underlain by coarse-grained stratified drift.

Q_x. Discharge, in cubic feet per second, for x, the given recurrence interval, in years.

Rainfall intensity (I). Rainfall for a specified duration. As used in this report, it is the precipitation of 24-hour duration, in inches, for the drainage, determined from isopluvial maps (Weiss, 1975).

R-square. The coefficient of determination. A measure of variation in the dependent variable explained by the regression equation. R-square × 100 yields the percentage of variation explained by the regression equation. If R-square = 1, then 100 percent of the variation is explained; if R-square = 0.75, then 75 percent of the variation is explained. It is a measure of the population scatter about a curve.

Sewers in an area. Area serviced by storm sewers as taken from drainage maps supplied by various city agencies, in percentage of total drainage.

Skew. A numerical measure or index of the lack of symmetry in a frequency distribution. Also called the coefficient of skewness. Visualized as the upward (negative skew) or downward (positive) curvature of the log Pearson Type III frequency distribution curve.

Standard error of estimate (SEE). A statistical measure of accuracy based on population scatter about a curve. SEE is the square root of the variance and is graphically defined as representing approximately two-thirds of the data points falling within its limits. Normally, SEE is a value compared to the predicted value from the curve and is expressed in percent. The SEE reported with log-transformed regression equations is the average of the positive and negative antilog of the SEE in log units.

Stratified drift. A predominantly sorted sediment laid down by or in melt water from a glacier; includes gravel, sand, silt, and clay in layers.

Till. A predominantly nonsorted, nonstratified sediment deposited directly by a glacier and composed of boulders, gravel, sand, silt, and clay mixed in various proportions.

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