EFFECTS OF URBANIZATION ON FLOOD CHARACTERISTICS IN NASHVILLE-DAVIDSON COUNTY, TENNESSEE

by Herman C. Wibben

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Water-Resources Investigations 76-121

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Discharges corresponding to 2-, 5-, 10-, 25-, 50-, 100-year floods from the modeled basins were compared with discharges from regional equations for estimating peak discharge rates from rural basins. Basin lag times of the urban basins were compared with those of nearby rural basins. The analyses indicated that in a fully-developed residential area, the flood peaks and the basin lag times will not be significantly different from those expected from an undeveloped area. Data were not sufficient to determine if an increase in flood peaks would occur from extremely small basins with extremely intensive development.

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UNITED STATES DEPARTMENT OF THE INTERIOR

Thomas S. Kleppe, Secretary

GEOLOGICAL SURVEY

V. E. McKelvey, Director

For additional information write to:

U.S. Geological Survey
Room A-413 U.S. Courthouse
Nashville, Tennessee 37203

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ABSTRACT

Streamflow data from 14 basins in Davidson County were extended in time by use of a digital model of the hydrologic system. The basins ranged in size from 1.58 to 64.0 square miles (4.09 to 165.8 square kilometers) and ranged in extent of man-made impervious cover from 3 to 37 percent. The flood-frequency characteristics were defined by weighting frequency curves based on simulated discharges with those based on observed discharges. The average record length of the three rain-gages used in simulation was 72 years, and the average record length of observed discharges was 11 years.

Discharges corresponding to 2-, 5-, 10-, 25-, 50-, and 100-year floods from the modeled basins were compared with discharges from regional equations for estimating peak discharge rates from rural basins. Lag times between rainfall and runoff in the urban basins were compared with those of nearby rural basins. The analyses indicated that in a fully-developed residential area, the flood peaks and the basin lag times will not be significantly different from those expected from an undeveloped area. Data were not sufficient to determine if an increase in flood peaks would occur from extremely small basins with extremely intensive development.

INTRODUCTION

As urban development takes place in a previously rural basin, substantial changes in flood characteristics of the basin often occur. The effect of the changes, in general, is to increase the magnitude and frequency of flooding. The impact of this increased flooding can be minimized if it is considered in the planning and design of buildings and drainage structures. Adequate design is not possible if the magnitude of change in flooding is not known.

Most urban areas lack sufficient streamflow information to determine the effects of urbanization upon the flood characteristics of its streams. Design techniques currently used in urban areas consist largely of using empirical equations developed decades ago. Variables within these equations are normally selected from curves or tables based on small quantities of data from various locations. The curves or tables may not be at all representative of the locale in which they are being used since they reflect average values. Recent studies have shown that the effect of urbanization can vary greatly from area to area.

A study in Houston, Texas by Johnson and Sayre (1973) indicated that changing a rural basin into an urban basin having 35 percent impervious area would increase the magnitude of the 2-year flood about nine times and the magnitude of the 50-year flood about five times. A similar study in Dallas, Texas by Dempster (1974) indicated considerably different results. The Dallas study indicated that a fully developed residential urban basin would increase the flood peak at the 2-year recurrence interval by about 1.4 times and at the 50-year recurrence interval by about 1.2 times the discharge from the same basin under rural conditions. Anderson (1970), in a study of Fairfax County, Virginia, found increases in flood peaks due to urbanization that were about halfway between those from the Houston and Dallas studies. The results of the above studies point out the extent of the potential error that can result from transferring output from the study of one urban area to another without having local data to verify such a transfer.

Purpose and Scope of the Urban Study

The purpose of this report is to assess the effects of urbanization upon magnitude and frequency of floods in Nashville-Davidson County, Tennessee. Data from urban basins are compared with data from rural basins in and around Davidson County to quantify the increase in flooding. The discussion and presentation of results provide information needed to design drainage systems and facilitate optimum land-use planning.

The author acknowledges the assistance of Mr. Glenn Bowles, Environmental Planner, Metropolitan Government of Nashville-Davidson County Planning Commission in computing

the areal extent of man-made impervious cover in the study basins. Daily precipitation data, 5-minute incremental storm data, and evaporation data for historical periods were obtained from the National Oceanic and Atmospheric Administration at Asheville, North Carolina.

Use of Metric Units of Measurement

The analysis and compilations in this report were made with English units of measurement. The equivalent metric units are given in the text and illustrations where appropriate. English units only are shown in tables where, because of space limitations, the dual system of English units and metric units would not be practicable. To convert English units to metric units, the following conversion factors should be used:

Multiply English units	by	To obtain metric units
inches (in)	25.4	millimeters (mm)
feet (ft)	0.305	meters (m)
miles (mi)	1.619	kilometers (km)
square miles (mi^2)	2.59	square kilometers (km^2)
feet per mile (ft/mi)	0.189	meters per kilometer (m/km)
cubic feet per second (ft^3/s)	0.0283	cubic meters per second (m^3/s)

BACKGROUND OF PROJECT

The U.S. Geological Survey, in cooperation with the Metropolitan Government of Nashville-Davidson County, began a program aimed at meeting future needs for information on the flood hydrology in Davidson County in July 1963. This program entailed delineating extent and frequency of flooding likely to be experienced in the areas covered by 88 flood inundation maps. Water-surface profile information used to delineate the flood boundaries are presented in a report by Conn and Boyd (1975). The project also included collection of streamflow and rainfall data in four tributary basins in the county.

In 1974 a second cooperative program between the Geological Survey and the Metropolitan Government of Nashville-Davidson County was initiated to provide planners a means for assessing the impact of basin development alternatives upon flooding. Data available for analysis consisted mainly of the data from the earlier cooperative program. Length of record at the gaging stations averaged only about 11 years. In addition, the data were collected during an unusually dry period. Because of these two factors, time-sampling bias of the observed data resulted in the potential for introducing considerable error in any flood-frequency analysis based directly on the observed data. Data from 14 of the stations fulfilled requirements of input to the U. S. Geological Survey rainfall-runoff model. Consequently, the approach used in this second program to minimize the time-sampling bias of the Nashville data base was to calibrate the Geological Survey rainfall-runoff model with observed data from the 14 stations and subsequently, to simulate a series of annual peaks on the basis of long-term climatological data. The average length of record simulated was 72 years. Determination of flood-frequency characteristics was made utilizing the simulated data.

Figure 1 shows the locations of the gaging stations which have data available for model use. Some physical characteristics of the modeled basins along with their respective record periods are presented in table 1.

PEAK-FLOW SIMULATIONS

Description of the Model

The U. S. Geological Survey rainfall-runoff model is a parametric simulation model based on bulk-parameter approximations to the physical laws governing infiltration, soil-moisture accretion and depletion, and surface streamflow. It was developed by Dawdy, Lichty, and Bergmann (1972) for use with point rainfall data and daily potential evapotranspiration data to predict flood volumes and peak rates of runoff for small drainage basins.

The model deals with three components of the hydrologic cycle-antecedent moisture, infiltration, and surface flow routing. A schematic outline of the model is shown in figure 2. Brief descriptions of the model parameters are listed below.

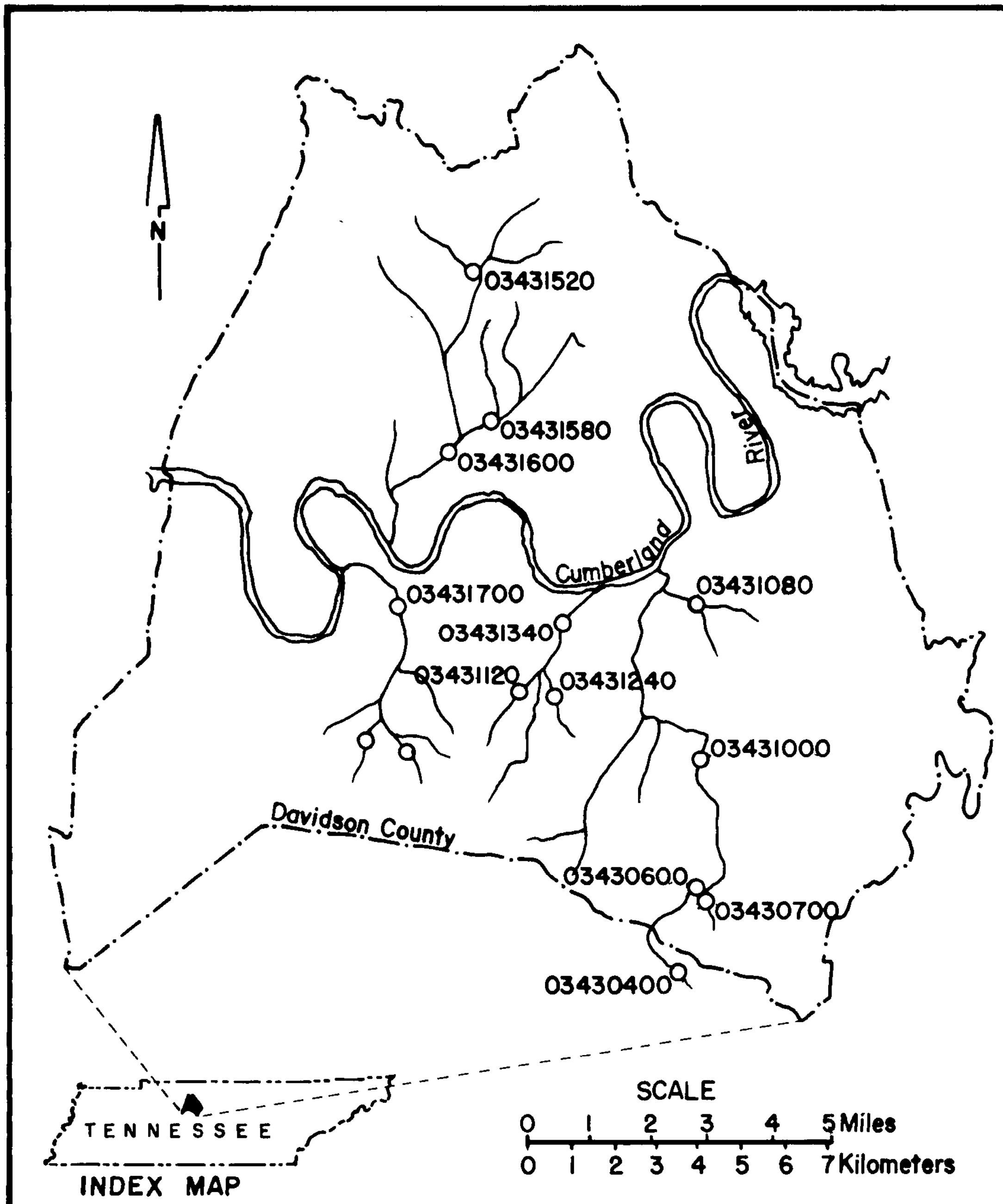


Figure 1.--Location of Metropolitan Nashville-Davidson County and the numbered gaging stations selected for modeling.

Table 1. -- Stations in the Nashville area used to calibrate the U. S. Geological Survey rainfall-runoff model.

Station Number	Stream and Location	Drainage area (mi^2)	L (mi)	S (ft/mi)	Impervious area (percent)	Basin lag time (hrs)	Length of record (years)
03430400	Mill Creek at Nolensville, Tenn.	12.0	4.34	30.6	3.0	1.7	10
03430600	Mill Creek at Hobson Pike, near Antioch, Tenn.	43.0	10.71	16.1	3.0	4.2	10
03430700	Indian Creek at Pettus Road, at Nashville, Tenn.	3.86	2.51	45.9	3.0	1.6	9
03431000	Mill Creek near Antioch, Tenn.	64.0	17.0	11.4	4.2	5.4	21
03431080	Sims Branch at Elm Hill Pike, near Donelson, Tenn.	3.92	3.03	57.8	22.4	1.1	10
03431120	West Fork Browns Creek at General Bates Drive, at Nashville, Tenn.	3.30	3.35	77.1	22.3	0.9	10
03431240	East Fork Browns Creek at Baird Ward Printing Company at Nashville, Tenn.	1.58	2.36	65.6	37.3	1.1	10
03431340	Browns Creek at Factory Street, at Nashville, Tenn.	13.2	6.51	42.6	31.5	1.9	10
03431520	Claylick Creek at Lickton, Tenn.	4.13	3.01	69.3	8.2	1.5	10
03431580	Ewing Creek at Knight Road, near Bordeaux, Tenn.	13.3	4.50	46.7	14.2	2.0	10
03431600	Whites Creek at Tucker Road, near Bordeaux, Tenn.	51.6	11.13	21.5	8.0	3.5	10
03431630	Richland Creek at Lynnwood Blvd., at Bell Meade, Tenn.	2.21	1.96	119.0	11.7	1.3	9
03431650	Vaughns Gap Branch at Percy Warner Blvd., at Belle Meade, Tenn.	2.66	2.38	83.3	14.9	0.7	10
03431700	Richland Creek at Charlotte Avenue, at Nashville, Tenn.	24.3	7.90	33.0	21.3	2.4	10

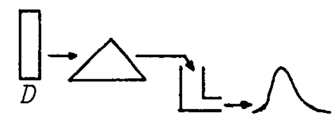
ANTECEDENT-MOISTURE ACCOUNTING COMPONENT	INFILTRATION COMPONENT	ROUTING COMPONENT
Saturated- unsaturated soil mositure regimes	Philip infiltration equation	Modified Clark instantaneous unit hydrograph
<u>Parameter</u> <u>Variable</u> *EVC* *BMS* *RR* *SMS* *BMSM* *DRN*	$\frac{di}{dt} = (l + \frac{P\ (m-m\ )}{i}$ <u>Parmeter</u> <u>Variable</u> *PSP* *BMS* *KSAT* *SMS* *RGF*	 <u>Parameter</u> *KSW* *TC* *TP*
	INPUT DATA	
Daily rainfall Daily pan evaportion Initial condition	Unit rainfall *BMS* *SMS*	Rainfall excess
	OUTPUT DATA	
BMS *SMS*	Rainfall excess	Discharge

Figure 2.--Schematic outline of the model, showing components, parameters, and variables.

Parameter identifier code	Units	Application
PSP	Inches	Represents the combined effects of soil moisture content and suction at the wetting front for soil moisture at field capacity.
RGF		Ratio of PSP for soil moisture at wilting point to that at field capacity.
KSAT	Inches per hour	The minimum saturated value of hydraulic conductivity used to determine infiltration soil rates.
BMSM	Inches	Soil moisture-storage volume at field capacity.
EVC		Coefficient to convert pan evaporation to potential evapotranspiration values.
DRN	Inches per hour	A constant drainage rate for redistribution of soil moisture.
RR		Proportion of daily rainfall that infiltrates the soil.
KSW	Hours	Time characteristic for linear reservoir storage.
TC	Minutes	Time base of the triangular translation hydrograph.
TP	Minutes	Time to peak of triangular translation hydrograph.

The antecedent-moisture component determines the initial infiltration rate for a storm. Input to this component are daily rainfall and daily evaporation, and output is the amount of base-moisture storage (BMS) and infiltrated surface-moisture storage (SMS).

The infiltration component uses the Philip (1954) equation, which is believed to be a somewhat better approximation to the differential equation for saturated flow than the classical Horton (1940) exponential-decay-infiltration equation. Input are storm rainfall, BMS, and SMS. This component computes the amount of storm rainfall that infiltrates the soil and determines rainfall excess as output.

The third component, surface-runoff routing, is based on a modification of the Clark (1945) form of the instantaneous unit hydrograph. Input is the rainfall excess computed in the infiltration component, and output is the storm-runoff hydrograph. First, the precipitation excess is converted into a triangular translation hydrograph representing the effects of varying travel times in the basin smoothed by storage. In the second step, successive flow rates of the translation hydrograph are attenuated by routing through linear storage.

Calibration of the model for a basin involves trial and error adjustment of parameter values in order to improve the comparison between observed input and simulated output. The comparison is made by testing for the minimum value of an objective function, which is based on the sum of the squared deviations of the logarithms of peak flows, storm volumes, or some combination of both. Starting values of parameters must be computed or estimated, and maximum and minimum parameter limits must be set. The observed rainfall and evaporation data serve as input and are used to generate a streamflow sequence that is compared with the observed streamflow record. Three separate phases of the calibration optimize on three different objective functions. During phase one direct runoff volumes are used in the objective function, and parameters pertaining to the first two components of the model are varied. In phase two, the routing phase, peak flows are used in the objective function, and the hydrograph shape parameters are optimized. Volumes routed are the observed direct runoff volumes so that errors introducted by rainfall data are eliminated. In phase three peak flows are again used in the objective function while the parameters affecting the moisture-accounting and infiltration components are varied.

The current version of the model has been adapted for use on urban basins. Percent impervious cover is input to the model. The impervious area is assumed to be uniformly distributed throughout the basin and is assumed to be capable of storing 0.05 in (1.27 mm) of precipitation. All precipitation in excess of 0.05 in (1.27 mm) that falls on the impervious area is assumed to be direct runoff.

Calibration Results

Final model parameters for the calibrated basins are shown in table 2. The model calibrations are described in more detail in an earlier report (Wibben, 1976). Average error of peak discharge simulation was 38 percent and no simulation bias was evident from the results of the final calibrations. Accuracy of simulated peaks was better for large peaks than for smaller ones. This trend is assumed to be the result of an effectively simpler model simulating the larger peaks. Saturated soil conditions exist during many of the larger storms. Under these conditions, most of the parameters within the antecedent-moisture-accounting component and several of those within the infiltration component have a negligible effect upon losses. These parameters are effectively ignored by the model and as such, any error introduced by them would be negligible. On the other hand, these very parameters are the ones that should have major impact in simulating the smaller events. Calibration results from the basins larger than 15 mi^2 (38.8 km^2) were noticeably poorer than those from smaller basins. The source of the increased error seems to be in simulation of precipitation excess, and its cause lies in the rainfall variation over these larger basins.

Parameters affecting the loss components of the model were constrained during calibration within the range of reasonable occurrence in the field. With only a few exceptions, final parameter values are within those limits. The constraints were applied to prevent unreasonable parameter distortion from interaction during calibration. Although several of the parameters resulted in variations consistent with their physical occurrence, it seems improbable that parameter values for ungaged basins can be predicted with accuracy. KSAT is a good example of the effects of parameter interaction. Basins having thin, tight soils generally produced smaller values of KSAT than did basins having thicker, more permeable soils. This trend was consistent with expectations. Variation in KSAT between adjacent basins, however, was frequently over 100 percent, even though physical characteristics of the basin were nearly the same.

Because hydrograph shape parameters were either computed from selected hydrographs or severely constrained during optimization, very little of the parameter interaction so prevalent in the loss parameters was present in the routing parameters. Phase 2 errors were generally in the range of 15 to 20 percent, with some as low as 10 percent.

Table 2. Summary of calibrated model parameters.

Station number	PSP (in.)	KSAT (in./hr)	DRN	RGF	BMSM (in.)	EVC	RR	KSW (hrs)	TC (min)	TP/TC	Effective impervious area (percent)
03430400	1.260	0.029	0.124	14.900	3.980	0.829	0.911	0.640	125	0.5	0
03430600	2.520	0.061	0.260	10.000	1.580	0.590	0.910	1.860	275	0.5	0
03430700	2.230	0.087	0.462	4.720	2.520	0.811	0.899	0.830	90	0.5	0
03431000	1.260	0.078	0.400	6.900	3.460	0.900	0.900	2.710	323	0.5	0
03431080	5.850	0.112	0.131	15.500	1.580	0.791	0.980	0.420	80	0.5	5
03431120	2.020	0.087	0.634	6.990	1.050	0.845	0.759	0.300	75	0.5	0
03431240	7.990	0.123	0.152	24.700	7.880	0.829	0.979	0.830	30	0.5	10
03431340	8.940	0.082	0.526	15.300	8.990	0.829	0.980	0.830	130	0.5	5
03431520	5.820	0.077	0.532	12.300	6.580	0.747	0.911	0.750	95	0.5	0
03431580	1.940	0.043	0.437	8.390	2.660	0.829	0.870	0.750	150	0.5	5
03431600	3.250	0.062	0.242	4.400	4.730	0.622	0.203	1.150	280	0.5	0
03431630	2.520	0.075	0.913	20.800	5.000	0.822	0.928	0.750	70	0.5	5
03431650	7.770	0.128	0.123	9.740	3.460	0.720	0.980	0.310	40	0.5	2
03431700	2.690	0.090	0.345	19.900	3.940	0.845	0.878	0.833	190	0.5	5

In studies of urban areas from other parts of the U.S., data frequently indicated that not all the man-made impervious cover was effective in producing additional runoff. Essentially the same situation was indicated by data in the vicinity of Nashville. At all the urban basins calibrated, small storm direct runoff volumes were overestimated when the measured impervious area was used as input to the model. The phase one results from station 03431080 shown in figure 3 are typical.

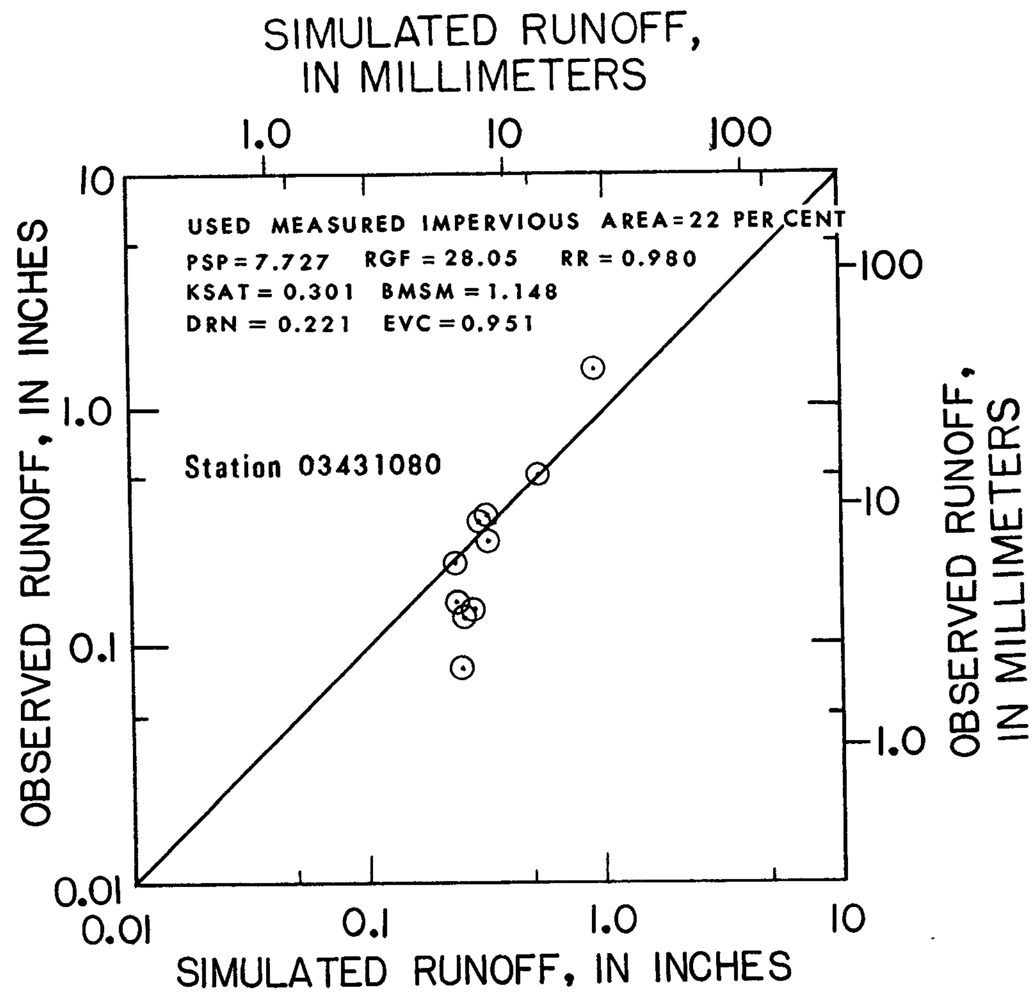


Figure 3.--Results of calibration of storm volumes using measured impervious area.

When a better fit of the smaller storm volumes was achieved through parameter optimization larger storm volumes frequently were underestimated. In addition to this bias, several model parameters, mainly PSP, KSAT, and RGF, were forced outside a reasonable range of occurrence during optimization. The net effect of the unusual parameter values was to reduce runoff from the pervious areas to compensate for the increased runoff from the impervious areas. Evidently the impervious surfaces were storing more than 0.05 in (1.27 mm) of precipitation or portions of the flow from the impervious surfaces were subsequently infiltrated while being routed over pervious areas enroute to the stream channels.

The approach used in selected final parameters was based on the assumption that only part of the impervious area was effective in increasing runoff. This approach was similar to that used by Durbin (1974) in his study of the Upper Santa Ana Valley in California. During subsequent calibrations, the percent impervious area was successively reduced until the model would reproduce small runoff events. The average effective impervious area of the other urban basins was 22 percent of the measured impervious area. The values of effective impervious area for individual basins were fairly close to those given by Durbin's curve relating the effective impervious area in drainage basins to the area affected by urban development and are included in table 2. Figure 4 shows the phase one results from station 03431080 using a reduced value of impervious area.

Flood-Frequency Determination

Log-Pearson type III discharge-frequency curves were computed from observed annual peaks in accordance with Water Resources Council (1976) recommendations for each of the fourteen gaging stations. Log-Pearson type III discharge-frequency curves were also computed from annual peaks simulated by the Geological Survey rainfall-runoff model using the calibrated parameters, long-term rainfall data from Nashville, Chattanooga, and Knoxville, and evaporation data from Center Hill Dam. The average record length of the precipitation data was 72 years. The three simulated frequency curves for each gaging station were combined into a composite simulated frequency curve by prorating them inversely with the distances from the long-term rain gages to the streamflow site. The observed frequency curves and the composite simulated frequency curves were then combined using a weighting technique based on their relative accuracy

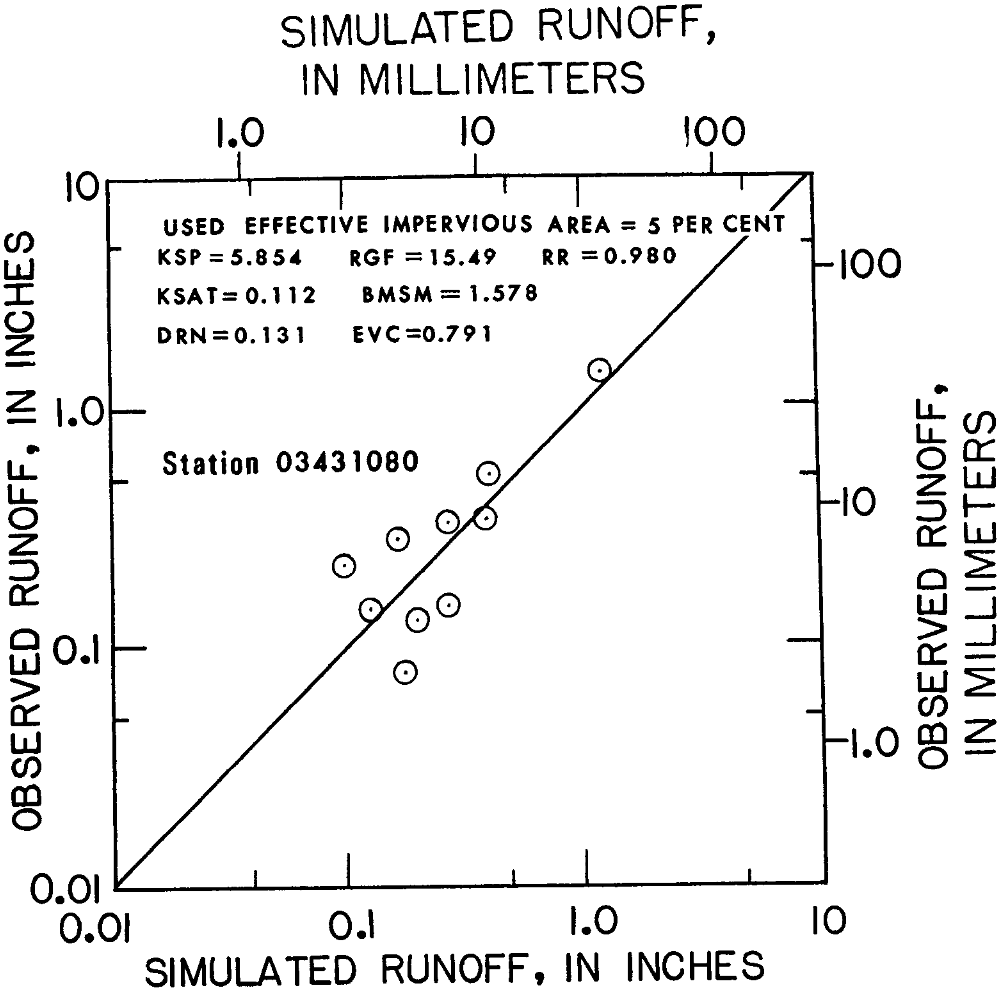


Figure 4.--Results of calibration of storm volumes using reduced value of impervious area.

Table 3.--Summary of t-year discharges for modeled basins.

Station number	Drainage area (mi^2)	Q_2 (ft^3/s)	Q_5 (ft^3/s)	Q_{10} (ft^3/s)	Q_{25} (ft^3/s)	Q_{50} (ft^3/s)	Q_{100} (ft^3/s)
03430400	12.0	3880	5550	6680	8120	9150	10200
03430600	43.0	4930	7290	8910	10900	12300	13700
03430700	3.86	790	1190	1480	1850	2100	2350
03431000	64.0	6890	10600	13100	16400	18800	21300
03431080	3.92	698	1280	1730	2310	2770	3250
03431120	3.30	1100	1830	2360	3060	3580	4110
03431240	1.58	231	343	426	546	637	737
03431340	13.2	1910	2700	3250	4030	4600	5200
03431520	4.13	806	1430	1900	2530	3020	3540
03431580	13.3	3130	4350	5170	6200	6920	7660
03431600	51.6	6300	9630	12000	15000	17200	19200
03431630	2.21	474	768	977	1240	1430	1620
03431650	2.66	594	970	1260	1660	1940	2240
03431700	24.3	3110	4970	6310	8000	9180	10400

at select recurrence intervals. The measure of relative accuracy used is described in more detail by Wibben (1976). It is analgous to a variance analysis except that the expected value of mean square error was used as an indicator of error rather than regression variance.

Discharges resulting from application of the weighting technique are presented in table 3 for select recurrence intervals. They represent the best estimate available of the flood-frequency characteristics of the modeled basins.

GENERAL EFFECTS OF URBANIZATION

Previous studies of the hydrology of urban streams have indicated that hydrographs, and consequently flood peaks, are affected two ways by the urban development. One is that quantities of storm flow are generally increased because of reduced infiltration at those parts of basins covered by impervious surfaces. The other is that improvements to drainage systems normally increase their hydraulic efficiency such that storm flows leave the basins in a shorter period of time, thus increasing the peak discharges. A commonly used measure of a basin's hydraulic efficiency is basin lag time which is defined as the average time interval between the centroid of rainfall excess and the centroid of resultant runoff. Decreases in basin lag time can result from improvements to the overland flow system, such as storm sewers and drainage ditches, as well as improvements to the channel flow system that would increase the conveyance of the channels.

Index of Development

To compare flood characteristics of basins having various degrees of development, some index of development was needed. The areal extent of man-made impervious cover was chosen as that index. Impervious cover was felt to be a reasonable indicator of potential increased runoff. Studies by Putnam (1972) and Johnson and Sayre (1973) indicated that impervious cover should also be a reasonable indicator of the hydraulic improvements in a basin. Insufficient information was available to accurately determine the extent of storm sewering and to subsequently relate it to impervious area.

The percent impervious area within each basin was determined by: (1) Delineating areas of similar development on 1:48,000 scale maps as determined from Metropolitan Government of Nashville-Davidson County Planning Commission property maps, recent aerial photographs, and visual inspectations; (2) Selecting representative parcels from each of the development types and measuring the extent of impervious cover; (3) Planimetering the areas of similar development types within each basin and multiplying the areas by their respective percentage of impervious cover; and (4) Summing the results of (3) and dividing by the basin drainage area.

The values of impervious area (table 1) reflect conditions as of mid-1974. No previous computations of impervious area or any other index of urbanization have been made. Consequently, a slight bias toward overestimating impervious area at the time of data collection may be present. Planning Commission officials (G. R. Bowles, oral commun., 1974) indicated that the bias would generally be less than 2 percent.

Average percent of impervious area used by the Planning Commission for various land-use types, which are similar to those used in other urban areas, are as follows:

Land-use type	Impervious area (percent)
Low-density residential	18
Medium-density residential	25
High-density residential, Apartments, Warehousing, Wholesaling	50
Manufacturing and Storage	60
Commercial, Retail arterial	80
Commercial retail concentrations	90

LOCAL EFFECTS OF URBANIZATION

Analysis of Lag Time

Lag times were computed for the Nashville-area stations, as well as 15 rural gaging stations (Wibben, 1976) from the model routing parameters. The rural stations are listed in table 4 along with some physical basin characteristics. O'Kelly (1955) showed that lag time, the time from the centroid of precipitation excess to the centroid of direct runoff, is equal to one-half the time base of the isosceles triangle translation hydrograph plus the linear storage constant. In terms of model parameters, this relation is

$$T_L = KSW = 1/2\ TC, \qquad (1)$$

where T_L is basin lag time in hours and the units of both KSW and TC are in hours. The computed lag times for the Nashville-area stations and the nearby rural stations are given, respectively, in table 1 and table 4.

Table 4.-- Rural gaging stations surrounding Davidson County for which lag times have been determined

Station Number	Drainage area (mi^2)	L (mi)	S (ft/mi)	Basin lag time (hrs)
03313600	0.95	1.55	73.9	1.1
03313620	3.03	3.12	52.8	1.2
03427830	0.17	0.56	100.3	0.5
03427840	3.54	4.58	73.9	1.5
03435020	9.32	4.00	46.5	1.9
03435030	15.1	6.70	28.0	3.1
03435600	3.50	3.52	51.7	1.1
03597300	4.99	4.20	49.6	1.8
03597400	9.59	6.08	31.7	2.4
03597450	0.73	1.70	132.0	0.8
03597500	16.3	8.30	25.7	3.3
03597550	1.86	4.02	58.1	1.7
03604080	1.52	2.16	105.6	1.3
03604090	6.02	3.14	73.9	1.5
03604100	10.1	5.23	49.6	2.3

A multiple regression analysis using basin lag time and basin characteristics for the combined data for urban and rural basins resulted in the following equation for lag time:

$$T_L = 2.25\ (L/\ S)^{0.52} \qquad (2)$$

where

T_L is the basin lag time in hours,

L is the length of principal stream in miles,

S is the difference in principal stream elevation, in feet per mile, at points 10 and 85 percent of the stream length, in miles.

The data points and the regression equation are plotted on figure 5. The standard error of equation 2 was 21.2

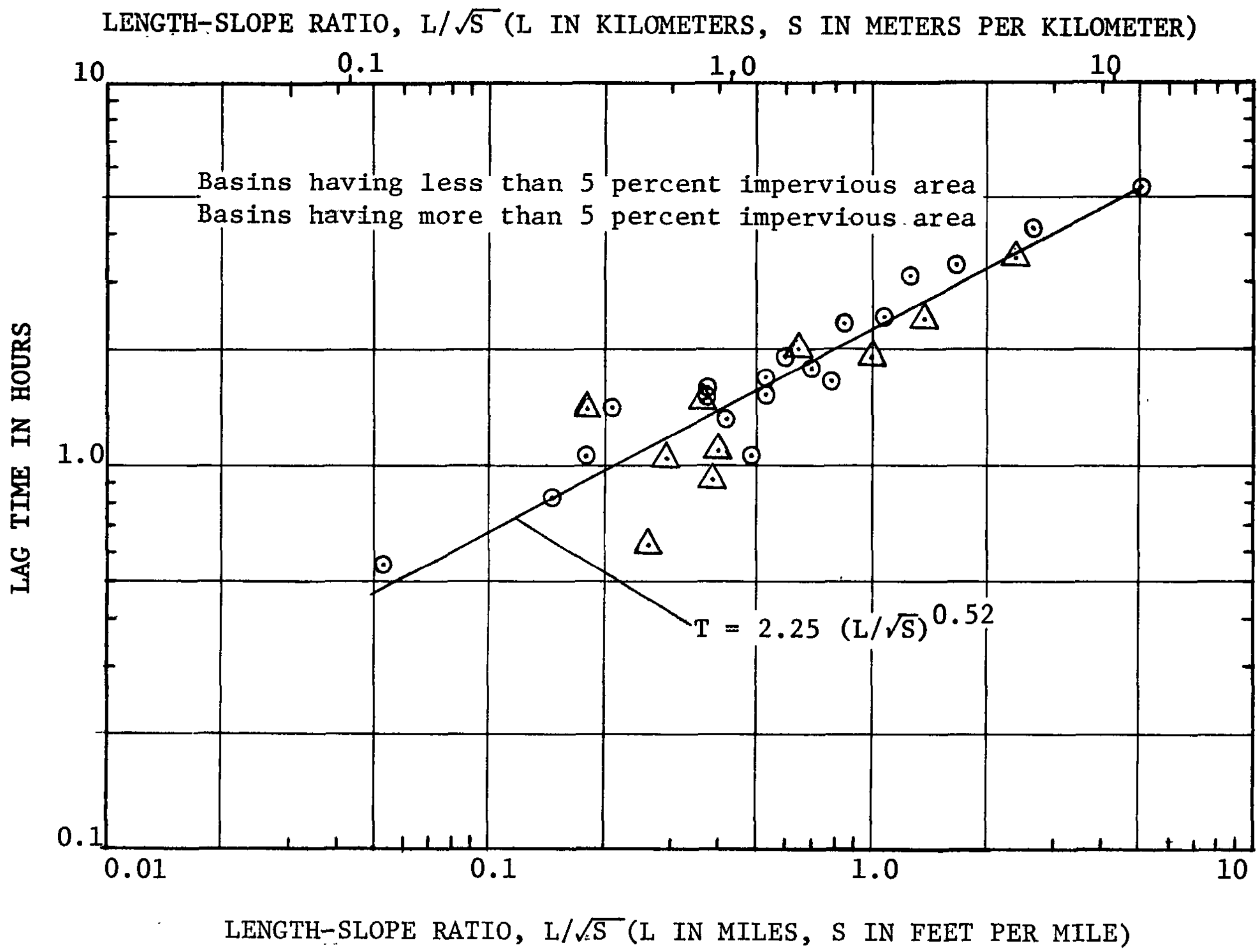


Figure 5.--Relation between lag time and basin length-slope ratio for Nashville-area rural and urban basins.

percent. Percent of impervious area dropped out of the regression model at the 5 percent level of significance. Figure 5 indicates that lag times for basins having more than 5 percent impervious area are not significantly less than those for rural basins.

The lack of difference in lag times is attributed partly to the efficiency of the natural channels in Davidson County. Current development practices increase channel conveyance very little. Major changes, such as lining, widening, or deepening channels, are rare except for localized reaches where channel improvements are made to replace conveyance lost to flood-plain encroachment. Extensive storm sewering to help relieve local drainage problems is not provided throughout most basins. In addition, the large number of stream crossings by roads has a tendency to increase channel storage which could actually increase lag times. This tendency is illustrated by the Sugartree Creek 50-year flood profile (Conn and Boyd, 1975) presented in figure 6. The elevated profile at Woodmont Lane and Estes Road reflect additional storage at these road crossings. Equation 2 should provide reasonable estimates of lag time for basins within Davidson County whose basin characteristics are within the range of those used in the regression model and whose drainage systems have not been significantly altered.

Comparison of Peak Discharges with Regional Estimates

Peak discharges were computed for each of the modeled basins using regression equations (Randolph and Gamble, 1976) for estimating discharges for selected recurrence intervals from ungaged rural basins. The computed discharges for the 2-, 5-, 10-, 25-, 50-, and 100-year recurrence intervals are plotted respectively, in figures 7 through 12, against the station discharges from table 3. Percent of impervious area is shown as a third variable on the figures. Data in the plots show that t-year floods from the urban streams are not significantly larger than those expected from rural basins.

Impact on Storm Volumes

As mentioned previously, urbanization generally increases flood peaks by a combination of increasing storm runoff and increasing hydraulic efficiency. Studies of figures 7 through 12 indicate that flood peaks have not been significantly increased due to urbanization. In addition, the data indicate that little or no change in basin lag times have occurred due to urbanization. Therefore, it

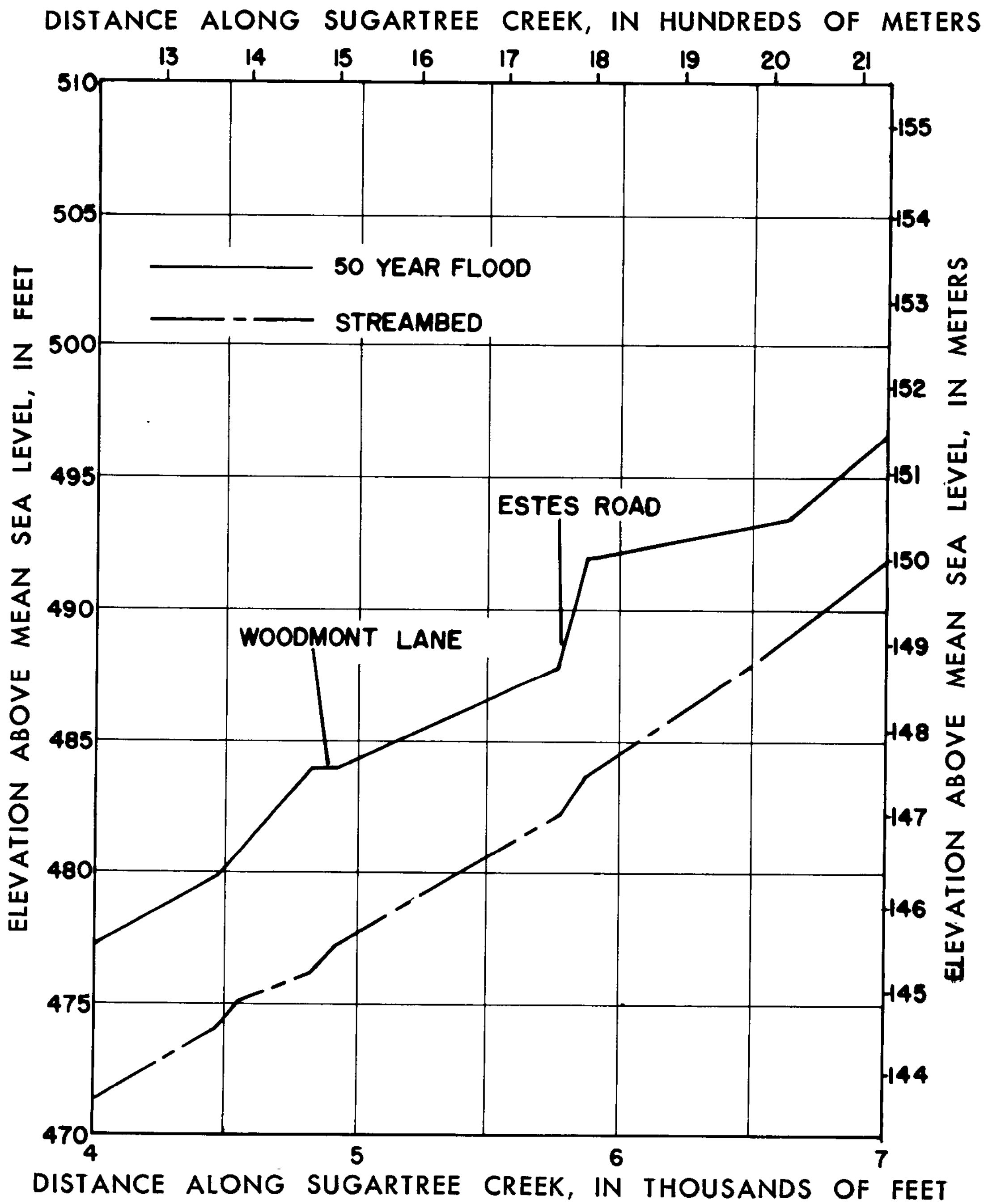


Figure 6.--Water-surface profile of 50-year flood along Sugartree Creek.

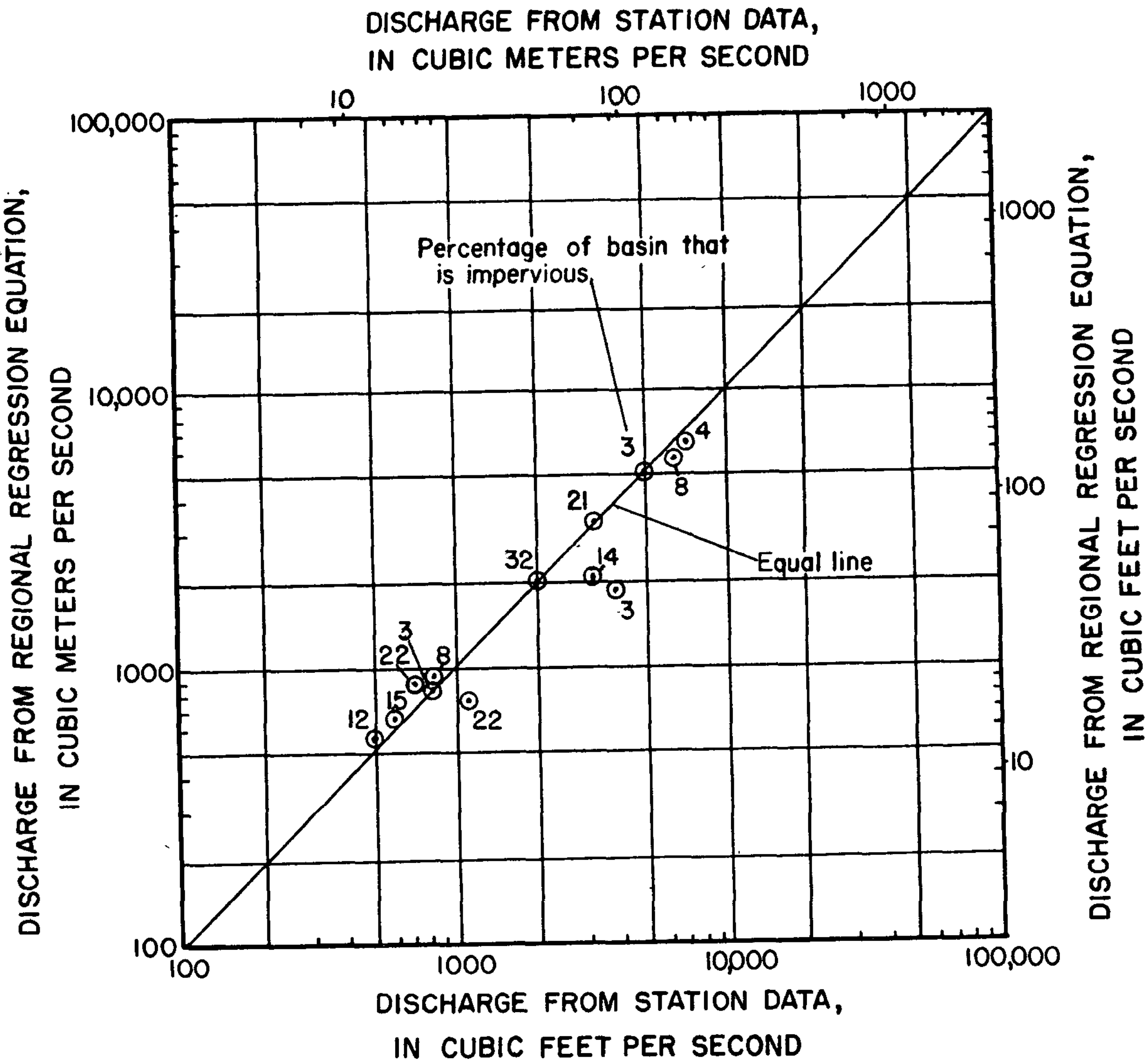


Figure 7.--Relation between discharges from regional regression equation and those from Nashville-area streams of 2-year recurrence interval.

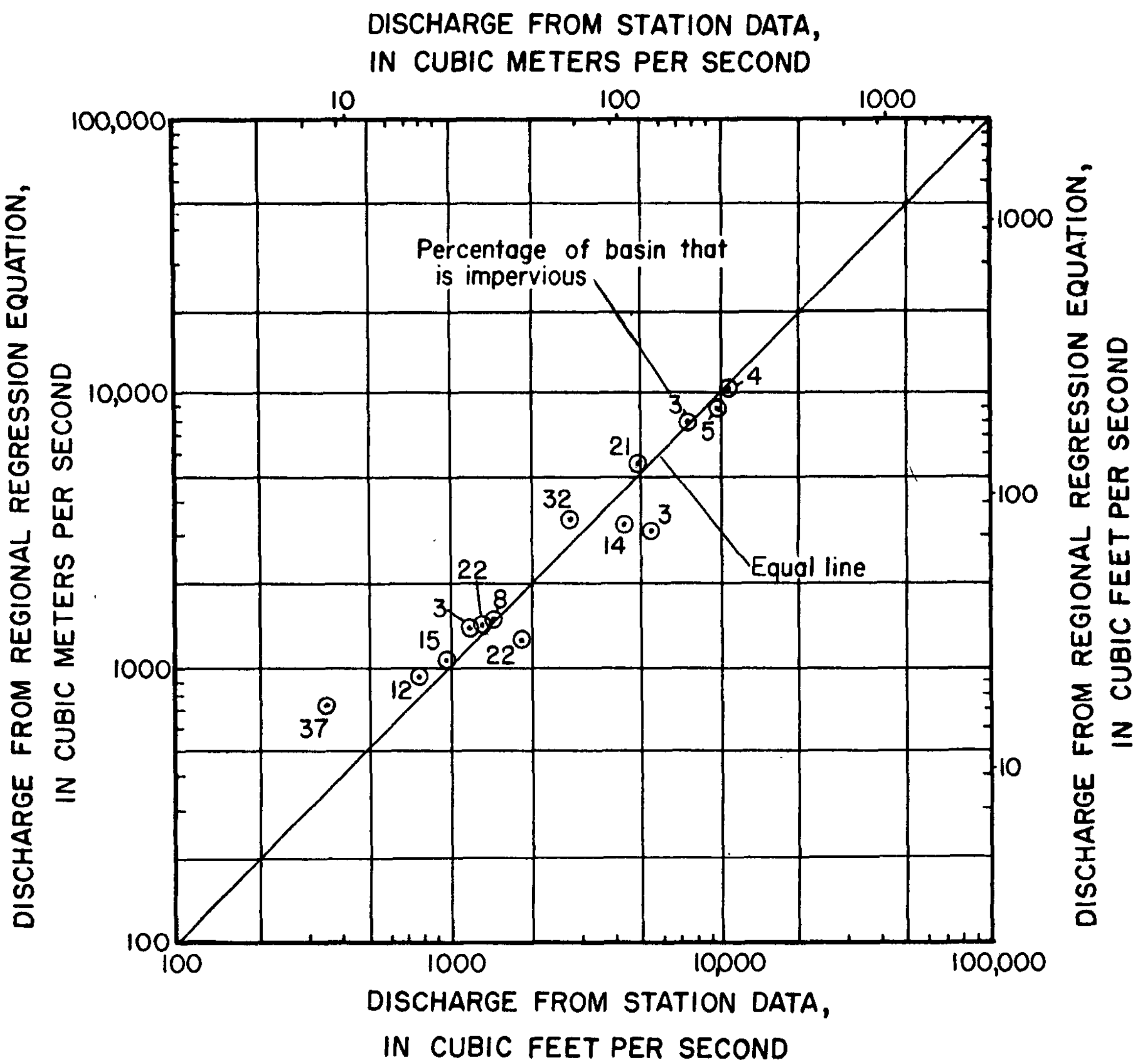


Figure 8.--Relation between discharges from regional regression equation and those from Nashville-area streams of 5-year recurrence interval.

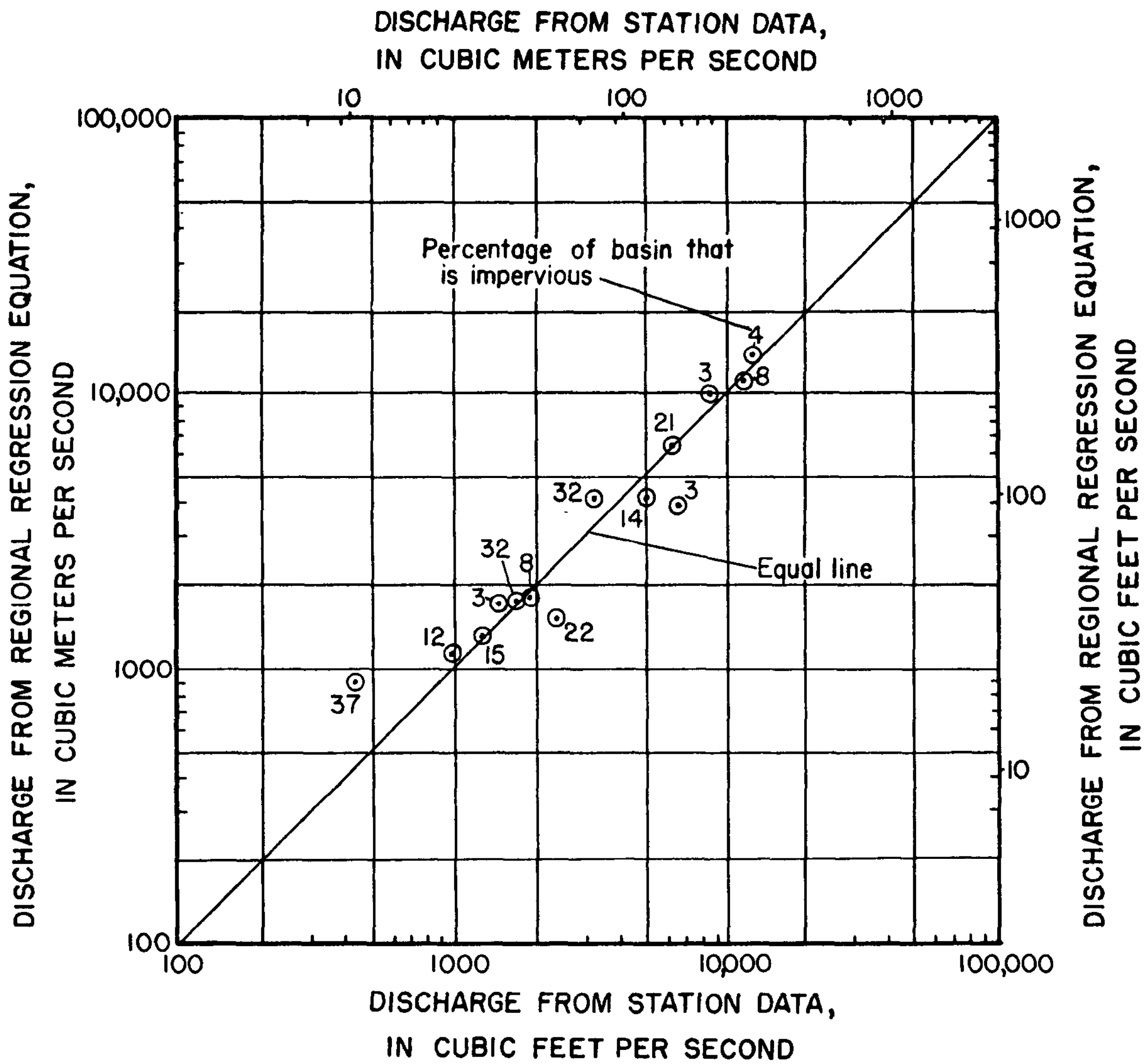


Figure 9.--Relation between discharges from regional regression equation and those from Nashville-area streams of 10-year recurrence interval.

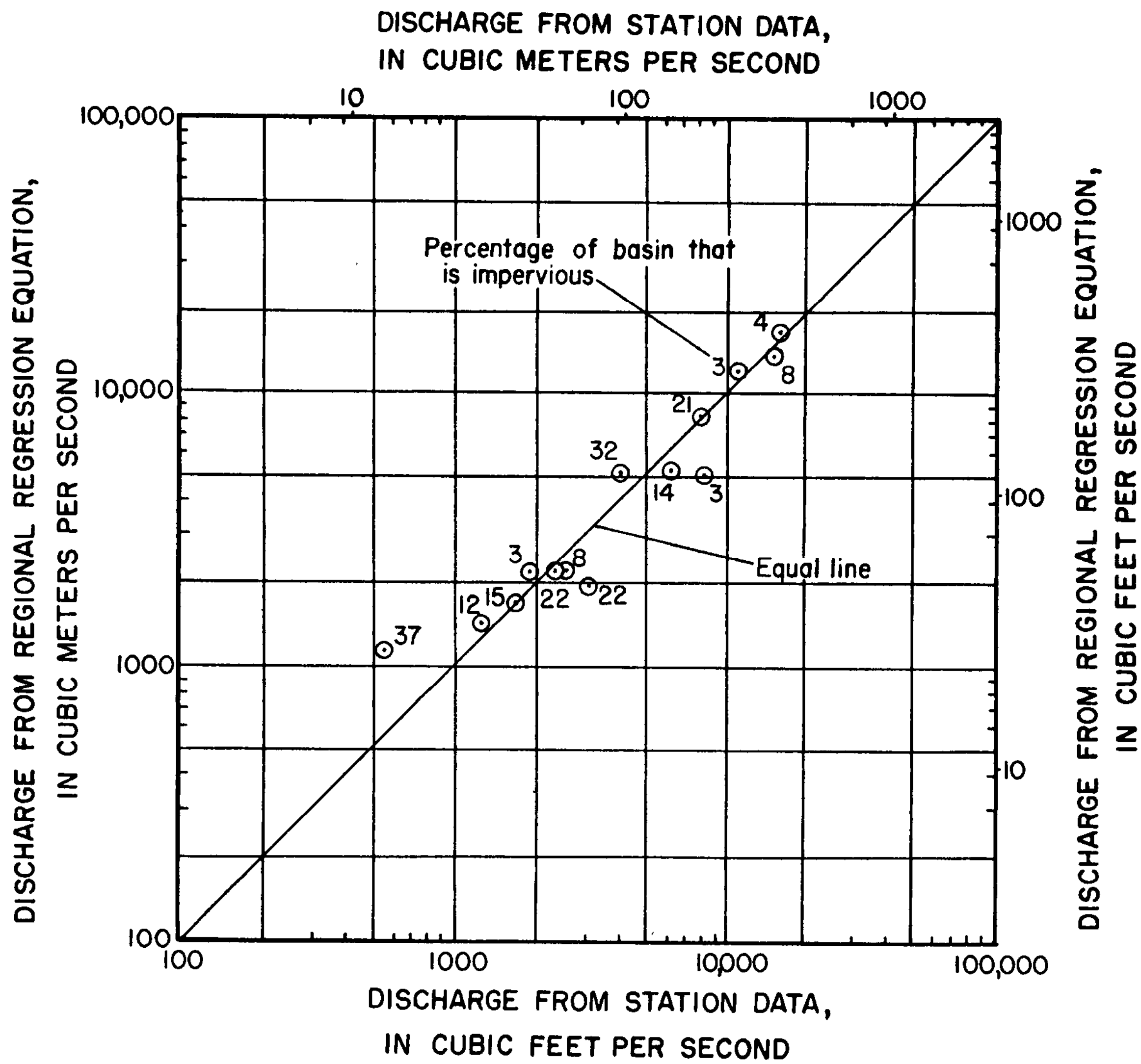


Figure 10.--Relation between discharges from regional regression equation and those from Nashville-area streams of 25-year recurrence interval.

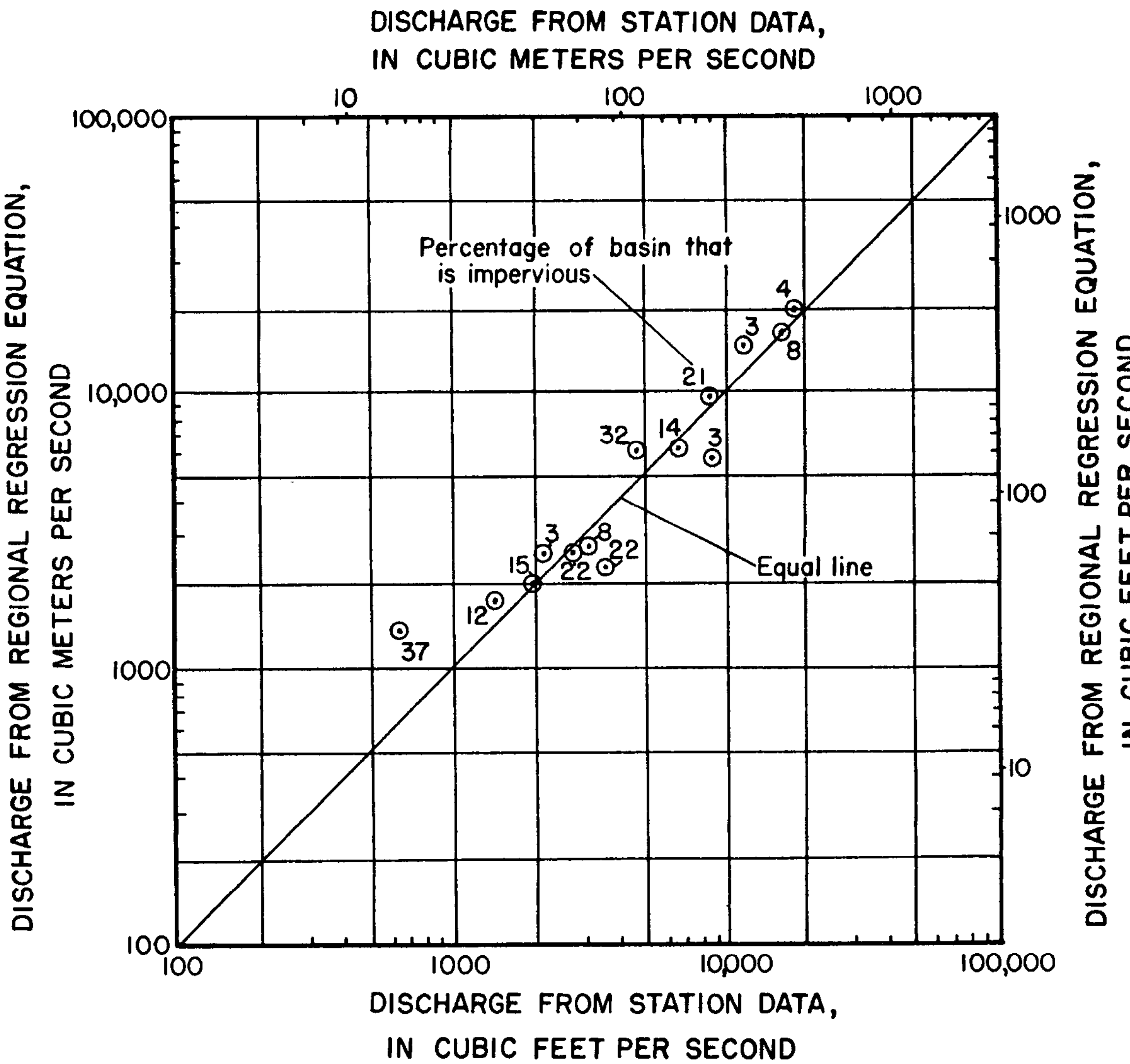


Figure 11.--Relation between discharges from regional regression equation and those from Nashville-area streams of 50-year recurrence interval.

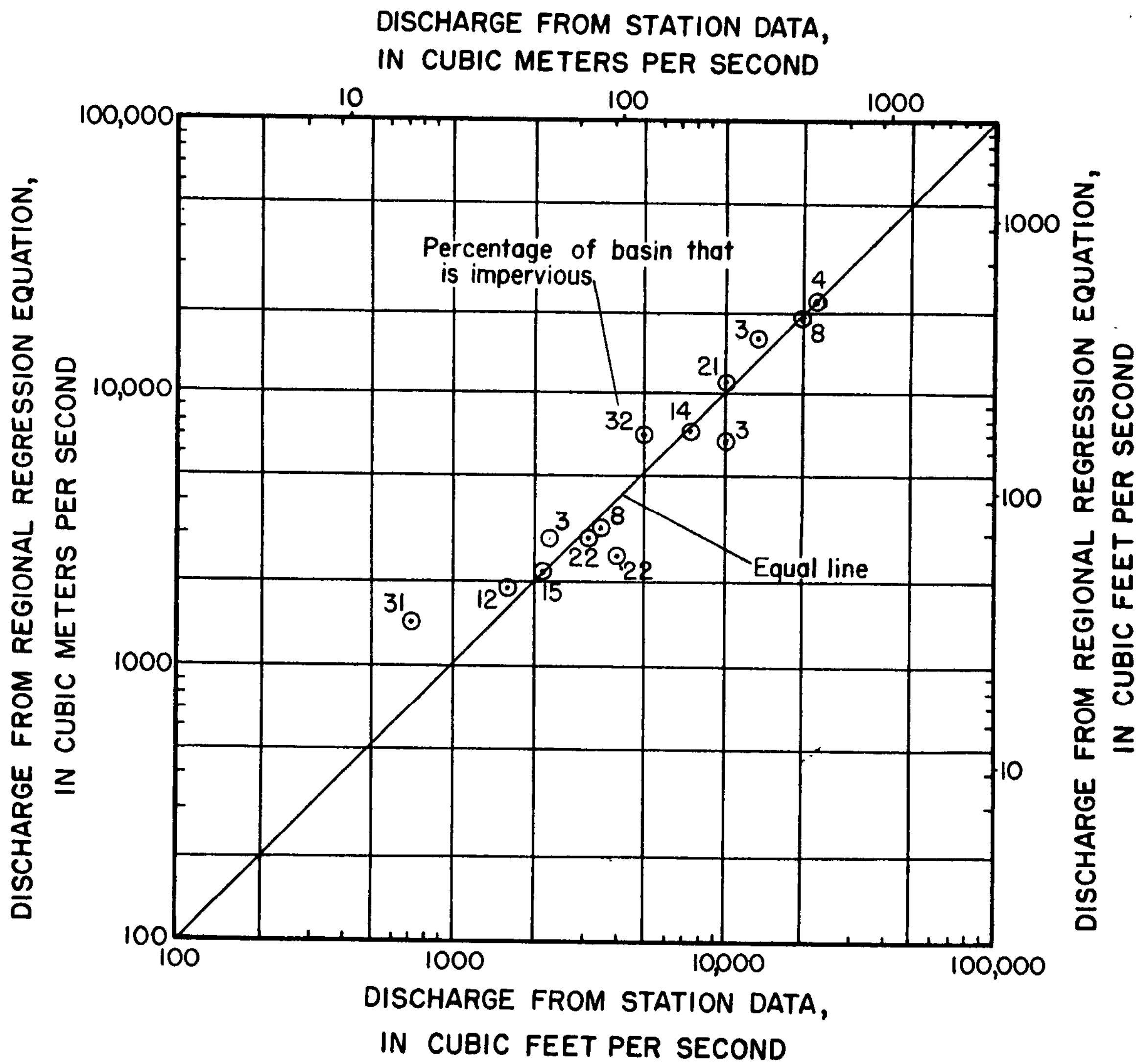


Figure12.--Relation between discharges from regional regression equation and those from Nashville-area streams of 100-year recurrence interval.

follows that within limits of urbanization reflected by the data, storm runoff volumes are not significantly increased.

The lack of impact on storm volumes is apparently due to the shallow soil cover and low permeability over most of Davidson County. The landscape is composed of loamy and clayey soils, the depth of which ranges from less than 1 foot to about 4 feet, and rock outcrops are numerous.

DETERMINATION OF FLOOD FREQUENCY

The urban basins studied range in size from 1.58 to 64.0 mi^2 (4.09 to 165.8 km^2) and in average impervious area from 3 to 37 percent. The regional equations developed by Randolph and Gamble (1976) that are applicable to the Nashville area are listed below:

	standard error of estimate (percent)	equivalent years of record	
$Q_2 = 319\ (A)^{.733}$	33	3	(3)
$Q_5 = 512\ (A)^{.725}$	30	4	(4)
$Q_{10} = 651\ (A)^{.723}$	30	6	(5)
$Q_{25} = 836\ (A)^{.720}$	31	8	(6)
$Q_{50} = 977\ (A)^{.720}$	32	8	(7)
$Q_{100} = 1125\ (A)^{.719}$	34	9	(8)

where

Q_t is that discharge, in ft^3/s, likely to be exceeded at an average interval of t years.

A is the contributing drainage area in mi^2.

Flood frequency at gaged sites can be determined by a combined use of the regression equations and the gaging-station frequency curve. The recommended procedure (U.S. Water Resources Council, 1976) is to compute the discharge for the desired recurrence interval as a weighted average of the station value and the regression value.

The weighted average is based on length of record of the station data and equivalent years of record for the regression value as indicated above. For modeled stations, the average length of record (Ng) used in this study was 25 years for all flood levels. The equation,

$$\log Q_{t(w)} = \frac{\log Q_{t(g)}(N_g) + \log Q_{t(r)}\ (Nu)}{Ng + Nu}$$

is used to compute the weighted average, where

$Q_t(w)$ = the weighted station discharge for recurrence interval t,

$Q_t(g)$ = the station discharge from the gaging station frequency curve for recurrence interval t,

$Q_t(r)$ = the regression discharge for recurrence interval t,

N_g = the number of years of gage record used to compute $Q_t(g)$, and

N_u = the equivalent years of record for $Q_t(r)$ indicated above.

The weighted values can be used directly for design purposes at gage sites.

When the site for which flood magnitudes are desired is located between two gages on the same stream, compute the regionally weighted discharge for the desired recurrence interval for each gage and estimate the discharge at the site by interpolation on the basis of contributing drainage area. Interpolation may be done by plotting discharge versus drainage area on logarithmic paper for the two gaged sites, connecting the two parts with a straight line, and then entering the relation with the value of drainage area at the site where information is desired. If the drainage area at the downstream gage is more than three times that at the upstream gage, use of one of the following procedures is recommended.

Flood discharges at sites which are relatively near a gaging station on the same stream can be calculated by a combined use of the regression equations and the nearby station data. The station value can be transferred upstream or downstream by the equation,

$$Q'_{t(w)} = \left(\frac{Au}{Ag}\right)^{b} Q_t(w),$$

and a weighted value can be calculated by the equation,

$$Q''_{t(w)} = \left(\frac{2\Delta A}{Ag}\right)Q_{t(r)} + \left(1-\frac{2\Delta A}{Ag}\right)Q'_{t(w)}$$

where

$Q'_{t(w)}$ = the weighted station discharge, $Q_{t(w)}$, transferred upstream or downstream to the ungaged site,

$Q''_{t(w)}$ = the final weighted discharge at the ungaged site for recurrence interval t,

$Q_{t(r)}$ = the discharge at the ungaged site from the regression equation for recurrence interval t,

Au = the drainage area at the ungaged site

Ag = the drainage area at the gaged site

ΔA = the absolute difference between Au and Ag, and

b = the regression coefficient (exponent) of drainage area for recurrence interval t.

When the drainage area at the desired site differs by more than 50 percent from that at the gaged site, the regional estimate of Q_t should be used.

Limitations

Equations 3 through 8 should provide reliable estimates of t-year floods for rural basins in Davidson County and for urban basins that are within the size and development range of the gaged urban basins. They are not applicable for small basins that are highly developed or basins that have undergone extensive drainage system improvements. Use of the equations for urban basins draining less than 1.5 mi^2 (3.88 km^2) or for basins containing more than 50 percent impervious area is not recommended.

SUMMARY AND CONCLUSIONS

The U. S. Geological Survey rainfall-runoff simulation model was applied to 14 gaged basins in Davidson County to reduce the time-sampling bias of the observed data. The basins ranged in size from 1.58 to 64.0 mi^2 (4.09 to 165.8 km^2) and in development from 3 to 37 percent impervious area. Calibrated model parameters were used with long-term climatological data to simulate annual peak discharges and to derive discharge-frequency curves for the basins. Six selected recurrence-interval (t-year) floods, based on observed and simulated data, were weighted to provide estimates of the flood-frequency characteristics of the 14 stations.

The t-year floods from the gaged urban basins in Davidson County are not significantly larger than those from rural basins. Lag times between rainfall and runoff in the urban basins showed little or no decrease as compared to those of rural basins. Consequently, regional equations for estimating peak runoff from rural basins should be reliable estimators of t-year floods from urban basins in Davidson County within the size and development range of the gaged urban basins.

Data are needed from smaller basins, with more intense development than was sampled, to determine whether the results of this report would be applicable to them.

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