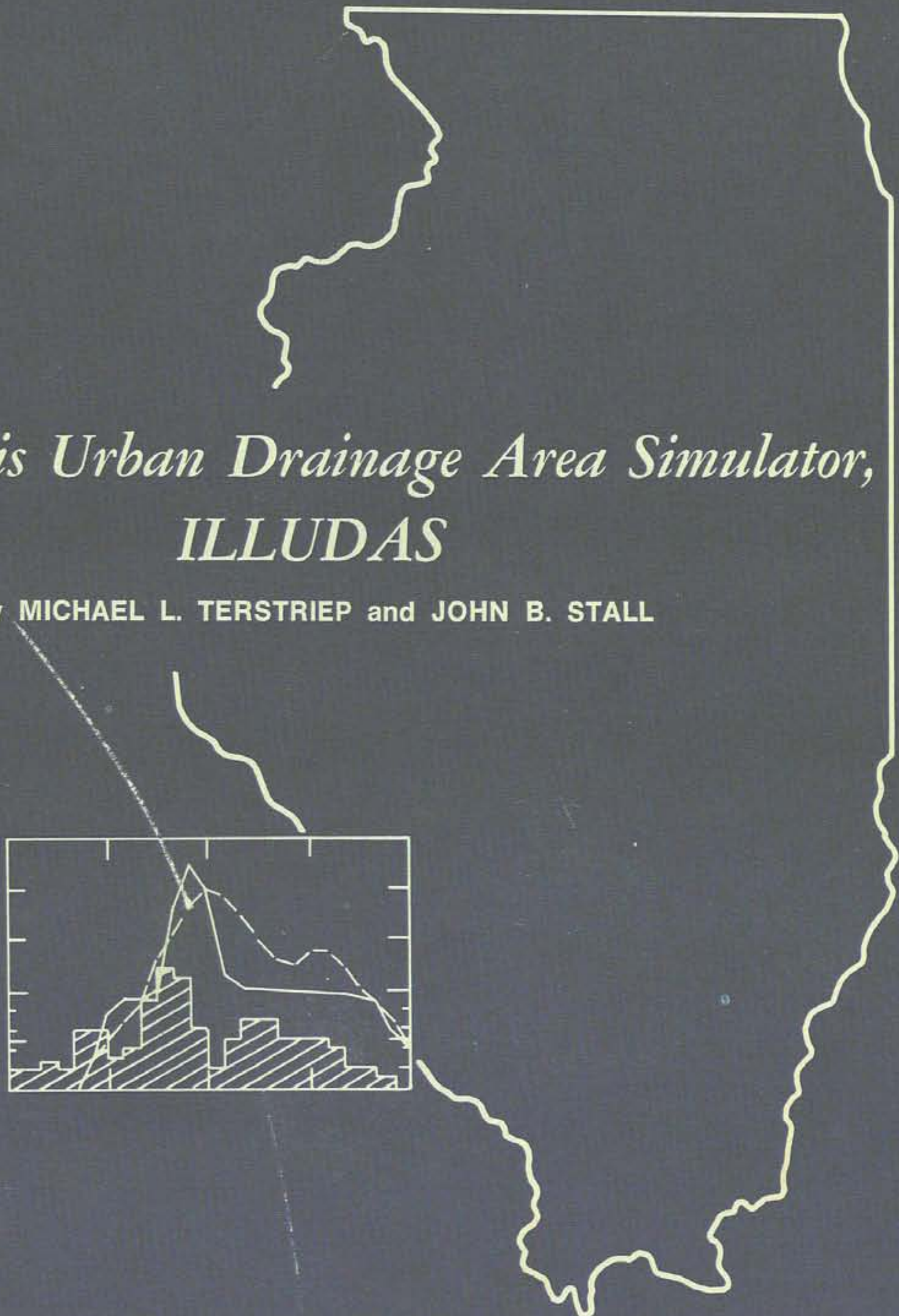
ISWS-74-BUL58
BULLETIN 58
STATE OF ILLINOIS
DEPARTMENT OF REGISTRATION AND EDUCATION

The Illinois Urban Drainage Area Simulator, ILLUDAS

by MICHAEL L. TERSTRIEP and JOHN B. STALL

ILLINOIS STATE WATER SURVEY
URBANA
1974



BULLETIN 58

The Illinois Urban Drainage Area Simulator, ILLUDAS

by MICHAEL L. TERSTRIEP and JOHN B. STALL

Title: The Illinois Urban Drainage Area Simulator, ILLUDAS.
Abstract: This report presents an objective method for the hydrologic design of storm drainage systems in urban areas and for the evaluation of an existing system. The method, based on a digital model known as ILLUDAS, uses storm rainfall and physical basin parameters to predict storm runoff from both paved areas and grassed areas. ILLUDAS utilizes the directly connected paved area concept of the RRL method but also recognizes and reproduces runoff from grassed and nonconnected paved areas. Included are a description of the theoretical development of the model, verification of the model by its application to 21 existing urban basins (from 0.39 acres to 8.3 square miles) and 2 rural basins, and a users manual that describes in detail the actual use of ILLUDAS in design applications. ILLUDAS is available in the form of a 700-card FORTRAN IV deck. A description of the input deck in its proper order and the actual content and format for each card is presented. This model will provide consulting engineers with an objective and reliable method for urban storm drainage design that requires little more input data than a rational method solution.

Reference: Terstriep, Michael L., and John B. Stall. The Illinois Urban Drainage Area Simulator, ILLUDAS. Illinois State Water Survey, Urbana, Bulletin 58, 1974.
Indexing Terms: Drainage design, hydrologic simulation, urban drainage, urban hydrology.

STATE OF ILLINOIS
HON. DANIEL WALKER, Governor

DEPARTMENT OF REGISTRATION AND EDUCATION
DEAN BARRINGER, Ph.D., Director

STATE WATER SURVEY DIVISION
WILLIAM C. ACKERMANN, D.Sc, Chief

URBANA
1974

Printed by authority of the State of Illinois—Ch. 127, IRS, Par. 58.29
(8-74-1750)

CONTENTS

The Illinois Urban Drainage Area Simulator, ILLUDAS

by Michael L. Terstriep and John B. Stall

ABSTRACT

This report presents an objective method for the hydrologic design of storm drainage systems in urban areas. The method is based on a digital model to be known as ILLUDAS, the Illinois Urban Drainage Area Simulator. ILLUDAS uses an observed or specific temporal rainfall pattern uniformly distributed over the basin as the primary input. The basin is divided into sub-basins, one for each design point in the basin. Paved-area and grassed-area hydrographs are produced from each sub-basin by applying the rainfall pattern to the appropriate contributing areas. These hydrographs are combined and routed downstream from one design point to the next until the outlet is reached. Pipe sizes are determined at each design point Detention storage can be included as part of the design in any sub-basin.

Included in this report are a description of the theoretical development of the model, the verification of the model by its application to 21 existing urban basins and 2 rural basins, and a users manual that describes in detail the application of ILLUDAS.

INTRODUCTION

The construction cost for storm drainage systems in urban areas of the United States estimated by the American Public Works Association (1966) is $2.5 billion per year. This monumental expenditure represents the amount that city dwellers pay in order that storm runoff water can be adequately collected and removed from the rooftops and streets of an urban area and emptied into a convenient natural stream outside of the city limits.

When storm rainfall occurs in a rural area, much of it soaks into the earth, and the remainder runs off to the nearest stream. The excess surface runoff may cause some temporary flooding on the land surface along ditches, drainageways, and small stream channels. When a city is constructed, much of the natural landscape is covered with rooftops, paved streets, and other paved areas. The remaining natural earth is usually covered with grass lawns. Several researchers have shown the effects of urbanization on the storm runoff of a region. Stall et al. (1970) showed that the complete transformation of a 3.5 square mile rural basin in east-central Illinois to an intensely urbanized basin would increase the flood peak by about 4 times for the 50-year recurrence interval. It would increase the mean annual flood by about 8 times.

An artificial storm drainage system for an urban area usually includes a collection network of storm drains consisting of underground conduits. Current engineering design practice utilizes almost exclusively the *rational method* for determining the required hydraulic capacity of these storm drainage systems. Design practice in 32 cities has been summarized by Ardis et al. (1969). The rational method is described in most hydrologic textbooks and is given by Chow (1964) as being $Q = CIA$ where Q is the peak discharge in cubic feet per second (cfs), C is a runoff coefficient depending on the characteristics of the drainage basin, I is the rainfall intensity in inches per hour, and A is the drainage area in acres. The term *rational* is used because the units of the quantities are numerically consistent. The method has widespread acceptance but its use still relies heavily on engineering judgment.

Practicing engineers have recognized the need for an improved method for understanding the storm rainfall-runoff process in urban areas. The American Society of Civil Engineers (1969) gave an extremely high priority to the need for better knowledge of the rainfall-runoff-quality process in urban drainage systems. Under this impetus a number of different models have been developed in recent years for accommodating the storm rainfall-runoff process for an urban region. A critical review of about 12 of these models has been provided by Linsley (1971), who states as one conclusion: "The present limited amount of urban hydrologic data is a serious deterrent to development and testing of storm runoff models. It seems unlikely that any significant improvement in current models is possible until more data and better quality data are available."

Background

A literature search early in the Water Survey's studies of urban runoff disclosed a design method developed by the British Road Research Laboratory (RRL) and used successfully in Britain. The RRL method described by Watkins (1962) looked promising, but it did need to be verified with data collected in the United States. Rainfall-runoff data together with the appropriate physical basin information were not generally available from urban basins. For preliminary study, three basins having such data were selected: Boneyard Creek in Champaign-Urbana, Illinois, Oakdale Avenue in Chicago, and the South Parking Lot in Baltimore. The RRL method was applied to these basins, and the results described by Terstriep and Stall (1969) were promising. However, they concluded that further verification of the RRL method would be required before it could be generally recommended.

Additional observed data from urban basins were acquired from Louisville, Houston, Dallas, and Baltimore. At this point contract support was received from the federal Environmental Protection Agency to obtain similar data from, and to test the RRL model on, basins in 10 cities. Stall and Terstriep (1972) in the final report from that contract indicated that the RRL method provided an accurate means of predicting runoff from paved areas, but could not be recommended for all urban basins unless a grassed-area component of runoff was provided. The grassed-area component is included in ILLUDAS.

Plan of Report

Part 1 of this report presents the development of what is called the Illinois Urban Drainage Area Simulator (ILLUDAS). ILLUDAS uses storm rainfall and physical basin parameters to predict storm runoff from both paved areas and grassed areas. Part 2 of this report contains the results obtained from applying ILLUDAS to 21 urban basins and 2 rural basins, and Part 3 is a users manual describing the actual use' of ILLUDAS in design applications.

The ultimate goal of this study has been to provide consulting engineers with an objective and reliable method for urban storm drainage design that requires little more input data than a rational method solution, and at the same time allows the user to examine various alternatives in arriving at a final design.

Acknowledgments

This study has been carried out by the authors as part of their regular duties in the Illinois State Water Survey's Hydrology Section, John B. Stall, Head. The work was under the general supervision of Dr. William C. Ackermann, Survey Chief. Many Water Survey personnel made substantial contributions, notably Floyd A. Huff, who provided valuable assistance in the determination of design storms for use in Illinois. Robert A. Sinclair provided the computer programming expertise for the project. Illustrations were prepared by William Motherway, Jr., and Linda Riggin, under the direction of John W. Brother, Jr. Mrs. J. L. Ivens, assisted by Mrs. P. A. Motherway, edited the final report and contributed much to its value.

Help and advice were provided by L. Scott Tucker of the American Society of Civil Engineers, whose earlier report (Tucker, 1969) listed various sources of rainfall-runoff data on urban basins. Special acknowledgment is made to the various offices of the U. S. Geological Survey, to state and city officials, and to others who provided data on the individual basins. These include: for Woodoak basin, Westbury, New York, Gerald E. Seaburn, USGS; for Sewer District 8, Bucyrus, Ohio, Richard Noland of Burgess and Niple, Ltd., consulting engineers, Columbus; for Echo Park Avenue basin, Los Angeles, California, Lyall A. Pardee, City Engineer, Department of Public Works, and Irving R. Cole and Walter R. Naydo, Division of Storm Drainage Design, Bureau of Engineering; for Crane Creek basin, Jackson, Mississippi, Kenneth V. Wilson and James Hudson, USGS; for Tripps Run basin, Falls Church, Virginia, Pat L. Soule, Fairfax office USGS; for Tar Branch, Winston-Salem, and Third Fork, Durham, North Carolina, Arthur L. Putnam, Durham office USGS, and Larry S. Kerr, Street Engineer, Durham; for Dry Creek basin, Wichita, Kansas, David Richards, Lincoln (Nebraska) USGS office and Wichita office personnel; for Wingohocking basin, Philadelphia, Joseph V. Radziul, Chief, Research Division of City Water Department, and William Green, Planning Division.

Also for basins in Louisville, Kentucky, Frank Druml, U. S. Army Corps of Engineers; for area basins of Baltimore, Maryland, Dr. John C. Geyer and Dennis Horn, the Johns Hopkins University; for Mt. Washington basins, Cincinnati, Ohio, Jesse Cohen, FEPA, and Tom Young, city traffic engineer; for Turtle Creek basin, Dallas, Texas, Hampton Couch and John Montgomery, USGS; for Hunting Bayou basins, Houston, Texas, Robert E. Smith and Emil G. Kaminski, USGS, and Don Van Sickle, of Turner, Collie, and Braden, Inc.; for the Boneyard Creek basin, Champaign-Urbana, Illinois, Delbert Winget and the late Davis Ellis, USGS. Soils information was obtained from the State Soils Scientist, U. S. Soil Conservation Service, for states represented in this report.

Part 1. Theory and Development

Paved-Area Runoff

The dominant feature of the RRL method is that it accommodates runoff only from the paved areas of the basin that are *directly connected* to the storm drainage system. Grassed areas are excluded from consideration as are paved areas that are not directly connected. ILLUDAS utilizes the directly connected paved area concept of the RRL method but also recognizes and reproduces runoff from grassed and nonconnected paved areas.

The principal elements in the computation of runoff from directly connected paved areas are as follows. Equal time increments of rainfall are applied to the directly connected paved area in a small sub-basin of the total urban basin. Next a computation is made of the travel time required for each increment of runoff to reach the inlets at the downstream end of the sub-basin. In this way a surface hydrograph is provided for each sub-basin. These surface hydrographs from each sub-basin are accumulated in a downstream order through the basin. This cumulation of inflow hydrographs is routed through each section of pipe to account for the temporary storage within each pipe section. The result is a computed outflow hydrograph from each section of pipe, and ultimately a hydrograph at the outlet of the total basin.

ILLUDAS is applied by first dividing the basin to be studied into sub-basins. A sub-basin is normally a homogeneous portion of the basin tributary to a single inlet or set of inlets that constitute a design point in the drainage network. Two physical factors must be evaluated for each sub-basin. First, the paved area directly connected to the storm drainage system must be determined; second, the travel time from the farthest point on the paved area to the design point must be calculated.

The various elements and steps used in developing a runoff hydrograph from the contributing paved area of an urban sub-basin are illustrated in figure 1. Extending down the middle of the sub-basin map in figure la is a city street with a pair of inlets at the lower end that allow water to enter a storm drain network. Shown also are rooftops along this residential street. The area shaded has been determined by a field survey to be directly connected to the street. In each case about half of the driveway has been considered to be contributing. The flow from roof No. 1 is not connected to the street, but the flow from roof No. 2 reaches the drainage system either by way of the driveway which drains into the street or by a direct underground connection.

After the directly connected paved area has been determined, calculations are made to determine the time-of-travel for the runoff from various parts of the paved area to the inlets at the downstream end of the sub-basin. During experimental studies, the velocity and travel times for overland flow were based on an equation developed by Hicks (1944) as described by Jens and McPherson (1964). In the present program, travel times on the paved area are computed in two steps. As the first step, flow of 0.5 to 1.0 cfs per acre of contributing paved area is assumed to exist in the street gutters.

The second step is to apply Manning's equation to compute the velocity of flow in the gutters. With these velocities, travel times are computed at various points on the paved area in each sub-basin. These travel times are plotted on the paved area, and by connecting points of equal travel time a series of isochrones are drawn on the paved area, as shown in figure la. The directly connected paved area between these isochrones is measured and designated areas PA1, PA2, PA3, PA4, and PA5. These various areas are accumulated and plotted against travel time to the inlet as shown in figure lb. This time-area curve shows the amount of paved area within the sub-basin that is contributing water at the storm drain inlet at any time after the beginning of runoff. In the computer program described later, the time-area curve was assumed to be a straight line connecting the origin and the end-point of the curve. The end-point, as illustrated in figure lb, represents the travel time from the farthest point of the directly connected paved area, and the total amount of the directly connected paved area.

In constructing the runoff hydrograph for each sub-basin, the input is the rainfall pattern as a series of intensities of equal duration, as shown in figure 1c. The rainfall input can be an actual event or a. design storm. The time increment used should be the same as the time interval between the isochrones. In general this time interval, At, which is used throughout the computations, should be as short as the quality of rainfall data will allow, but a longer At may be more convenient to use for very large basins or very long storms.

Figure 1d shows the losses for the same time intervals used for rainfall. For application to a paved area, the losses consist of initial wetting and depression storage. These losses are combined and treated as an initial loss to be subtracted from the beginning of the rainfall pattern. In figure 1d the entire initial loss, L1, occurs during the first minute or first time increment.

After subtracting these losses from the rainfall pattern, the remainder of the rainfall will appear as runoff from the paved area. This runoff is shown in figure 1e and is referred to as the paved-area supply rate (PASR).

The ordinates of the paved-area hydrograph are computed by applying the paved-area supply rate to the time vs paved-area curve with the series shown in figure 1f. The hydrograph shown in figure 1f occurs at the sub-basin inlets illustrated in figure la. Such a hydrograph is developed for each sub-basin, and after being combined with the corresponding grassed-area hydrograph (described in the next section) becomes an input into the drainage network at a particular point.

Grassed-Area Runoff

Computation of a grassed-area hydrograph for each sub-basin closely parallels that of the paved-area hydrograph. Figure 2a represents the same sub-basin used to illustrate paved-area runoff. The shaded area represents the con-

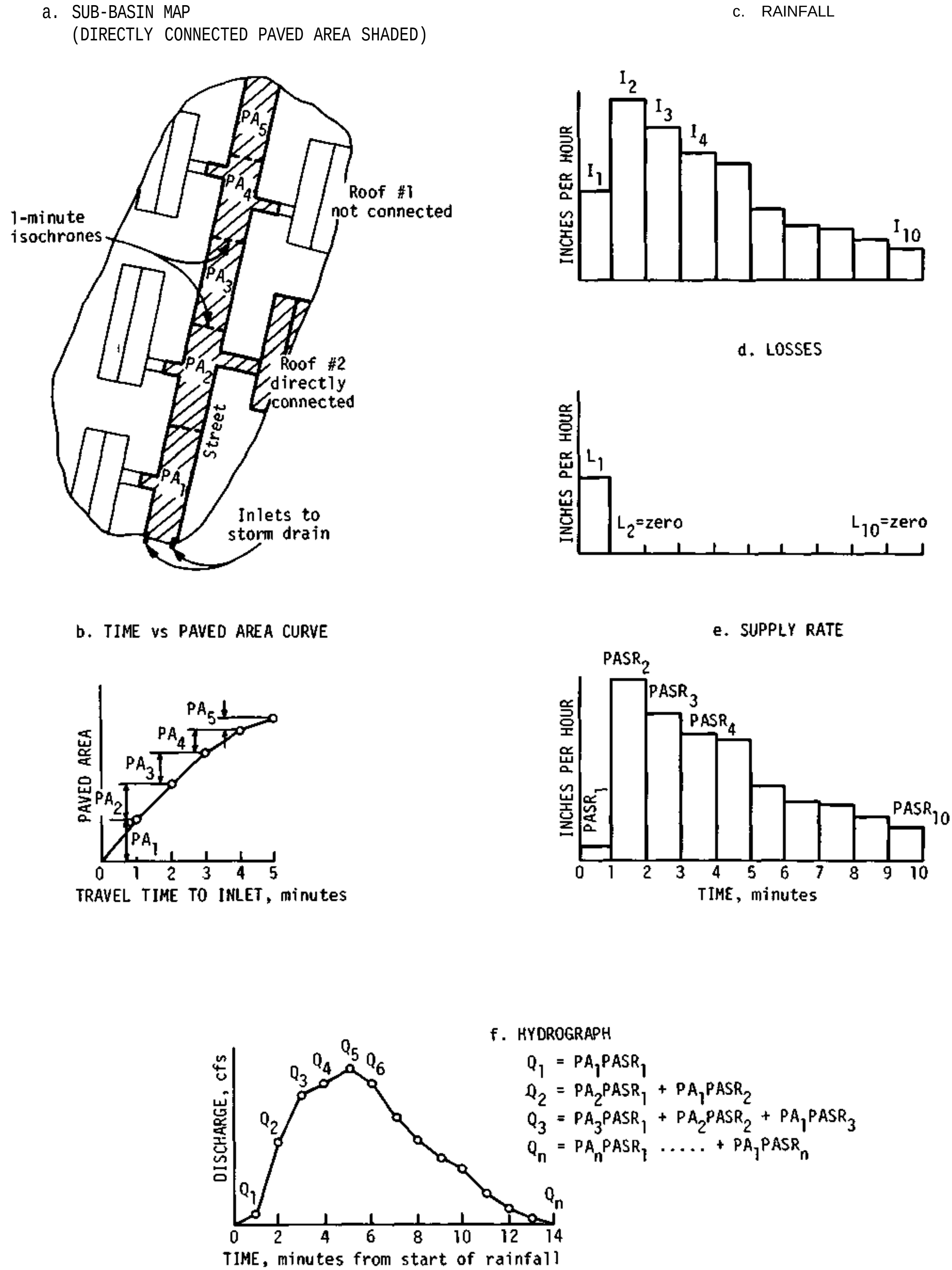


Figure 1. Elements in the development of the paved-area hydrograph

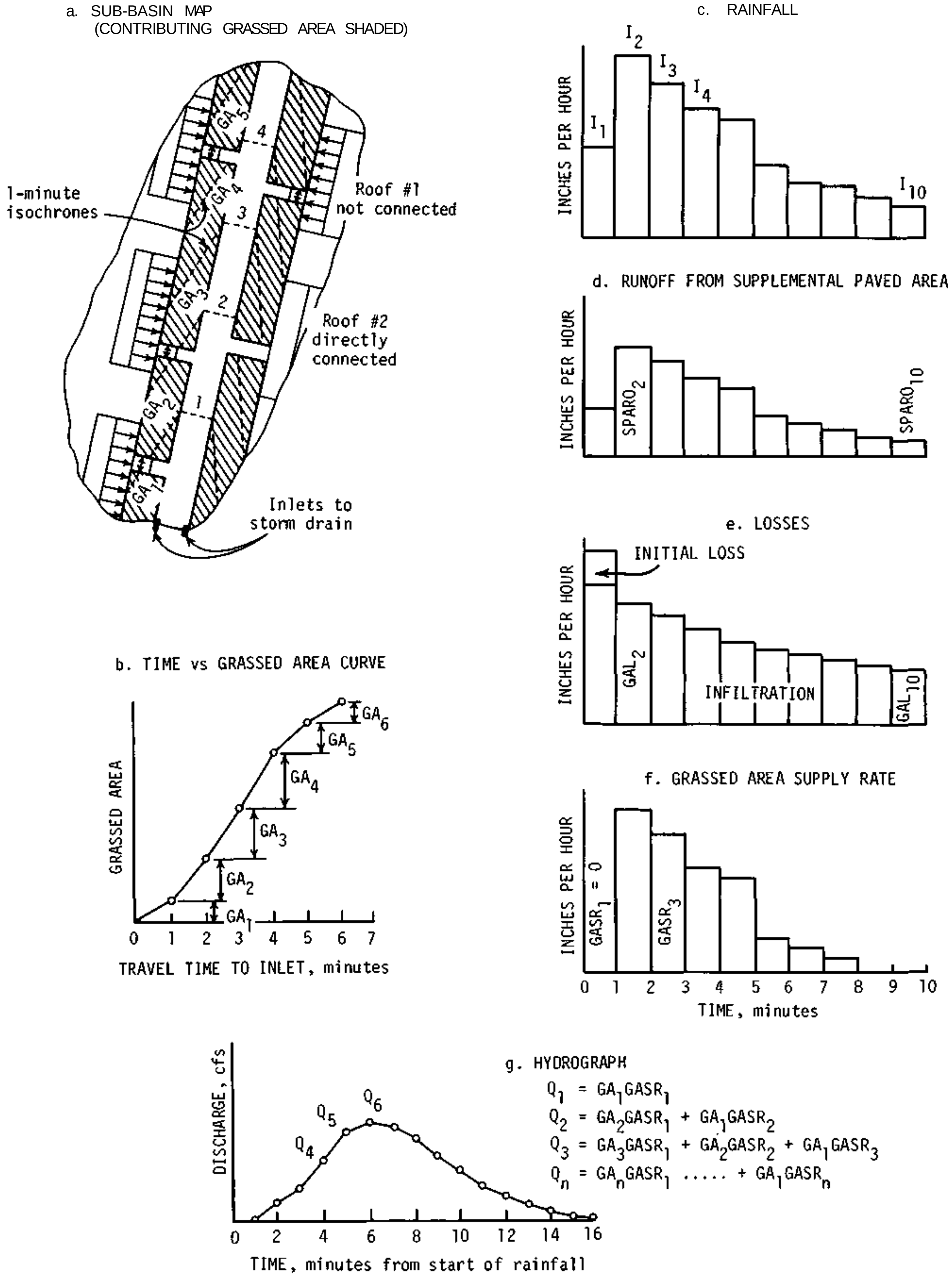


Figure 2. Elements in the development of the grassed-area hydrograph

tributing grassed area which in this case includes only the front yards of a residential neighborhood. More grassed area could contribute to runoff, but front yards are typically graded to drain quickly to the street. Runoff from back and side yards often drains gradually to a common back lot line and then laterally to the nearest street. The travel time required for this long flow path virtually removes such grassed areas from consideration during relatively short intense storms normally used for drainage design.

After the contributing grassed area has been determined, the time vs grassed-area curve in figure 2b can be constructed. The location on the street of the 1-minute isochrones in figure 2b was determined earlier for the paved-area runoff. Travel times on the grass strip itself are equivalent to the time of equilibrium in the following equations by Izzard (1946).

$$q_e = 0.0000231\ IL \qquad (1)$$

where

q_e = discharge of overland flow, in cfs per foot of width, at equilibrium

/ = supply rate in inches per hour assumed to be 1.0 in this study

L = length of overland flow in feet

$$t_e = 0.033\ KL \qquad q_e^{-0.67} \qquad (2)$$

where

t_e = time of equilibrium in minutes

$$K = (0.0007\ 1 + \qquad c)S^{-0.33} \qquad (3)$$

S = surface slope in feet per foot

c = coefficient having a value of 0.046 for bluegrass turf

Since the equilibrium condition is reached asymptotically, Izzard (1946) arbitrarily set the time of equilibrium as the time when q reaches $0.97q_e$. The time of equilibrium determined by these equations was found to be in close agreement with empirical equations for time of concentration developed by Hicks (1944).

After the travel times at various points on the contributing grassed area have been computed and plotted, the 1-minute isochrones in figure 2a are drawn. The contributing grassed areas within time zones GA_1 through GA_6 are then measured and plotted cumulatively against time-of-travel from the inlet as shown in figure 2b. This time-area curve shows the amount of grassed area within the sub-basin that is contributing water at the storm drain inlet at any time after the beginning of runoff. In the computer program described later, the time-area curve was assumed to be a straight line connecting the origin and the end-point of the curve. The end-point, as illustrated in figure 2b, represents the travel time from the farthest point on the contributing grassed area, and the total amount of contributing grassed area.

As in the case of paved-area runoff, rainfall is the primary input for development of the grassed-area hydrograph. The rainfall pattern illustrated in figure 2c is the same as that in figure 1c. The modifications that must be made in changing the rainfall pattern to a grassed-area supply rate are much more complex than in the paved-area case. The procedure followed in ILLUDAS is first to add in supplemental paved-area runoff (SPARO, figure 2d) and then to subtract initial and infiltration losses (GAL, figure 2e).

Rain falling on the supplemental paved area (which is the paved area not directly connected) is assumed to run off onto the surrounding grassed area. ILLUDAS assumes that this occurs instantly and that the volume of runoff is uniformly distributed over the contributing grassed area. Because of these simplifying assumptions, the SPARO can be expressed as inches on the grassed area by the following equation and added directly into the rainfall pattern.

SPARO (inches on SPA) x (SPA/GA) = SPARO (inches on GA)

where

SPA = total supplemental paved area
GA = total contributing grassed area

The losses illustrated in figure 2e include an initial loss, usually 0.2 inches, to account for depression storage plus infiltration. The grassed-area supply rate in figure 2f is obtained by subtracting these losses from the sum of rainfall plus the supplemental paved-area runoff. The determination of infiltration losses will be covered later.

The ordinates of the grassed-area hydrograph are computed by applying the grassed-area supply rate to the time vs grassed-area curve with the series in figure 2g. The hydrograph shown in figure 2g occurs at the sub-basin inlets illustrated in figure 2a. Such a hydrograph is developed for each sub-basin and combined with the corresponding paved-area hydrograph. These combined hydrographs become the surface hydrographs from each sub-basin and are point inputs into the drainage network.

If the sub-basin in question happens to be at the uppermost end of a series of pipes or open channel reaches, the surface hydrograph is entered into the system by routing it downstream to the next input point. If the sub-basin occurs somewhere below the upper end, its surface hydrograph is combined with the upstream hydrograph and the resulting combined hydrograph is routed downstream to the next input point. If the sub-basin is located at the confluence of two or more pipes, the surface hydrograph is combined with the converging hydrographs before routing downstream to the next input point.

Infiltration

In an urban basin, the area that is not paved is most often covered with bluegrass turf. When rain falls on this turf, there are two principal losses, the first being depression storage and the second being infiltration into the soil. In ILLUDAS provision is made for depression storage to be filled and satisfied before any infiltration takes place. Depression storage is normally taken to be 0.20 inches, but provision is made in ILLUDAS for this to be varied.

The dominant and far more complex loss of rainfall falling on bluegrass turf is that caused by infiltration. To estimate infiltration losses, extensive use has been made of concepts of infiltration described by Holtan and Musgrave (1947), Holtan (1961), and Holtan et al. (1967). This research has been carried out by the Agricultural Research Service and is based on extensive data from research watersheds. The theoretical approach to evaluating infiltration rates has been based on using the physical properties of the soil for estimating the water storage available in the soil mantle and evaluating the role of this water storage in the infiltration of rain water into and through the soil mantle.

Water Storage in Soil

The amount of water that can be stored in the soil mantle is dependent first upon the total pore space available in the soil between the soil particles. This is commonly expressed as porosity in percent of soil volume. The total pore space available within the soil mantle represents the maximum volume of water that can be stored in this soil mantle. When this entire pore space is filled with water, the soil is said to be *saturated.* The total water in storage in the soil mantle is divided into three principal parts. The first of these is *gravitational water.* This is water which will drain out of the soil by gravity. When the gravitational water has been depleted, the soil mantle is said to be at *field capacity.* This is commonly considered to be the condition for which there is a soil moisture tension of 0.3 bars on the soil moisture. This means the moisture is held by the soil against a pressure 30 percent of atmospheric pressure.

The second principal type of water storage within the soil mantle is *ET water,* or that water which can be removed by plants by the process of evapotranspiration. When this water has been depleted by evapotranspiration, the soil is said to be at the *wilting point.* This is commonly considered to be a soil moisture tension of 15 bars, or 15 atmospheres. The third element of water storage in a soil is called *hygroscopic water.* This is water held within the soil which cannot be removed by gravity or by evapotranspiration by plants. This hygroscopic water is only removed by evaporation, or in the laboratory by drying. When this water in the soil is depleted, the soil is said to be *air dry.*

It has been shown by Holtan et al. (1967) that these various water storages within a soil profile can be calculated on the basis of the physical properties of the soil. Subsequently, moisture tension data for soils sampled at 200 Agricultural Research Service experimental watersheds or plots at 34 locations in the United States have been published by the U. S. Department of Agriculture (1968). The data are useful for calculating the probable rates of infiltration. As a first step in doing this, it is necessary to compute the actual available water storages.

This set of calculations is shown in table 1 for Alexis silt loam soil sampled at Monticello, Illinois, and published by the U. S. Department of Agriculture (1968). The calculations in table 1 follow those described by Holtan (1961). In table 1 calculations are shown for four soil horizons within Alexis silt loam. Item 3 shows the saturated conductivity in inches per hour which can generally be considered the ultimate constant infiltration rate through this soil profile. As shown, this conductivity for the fourth soil horizon is considerably smaller than that for the first, second, and third horizons. Consequently, judgment is used to determine that the *zone of principal hydrologic activity* is confined to the first three soil horizons.

In the lower portion of table 1, porosity data are used to calculate the water storage in inches which comprises for this soil the gravitational water G and the ET water. The total water storage affecting infiltration is considered to be the sum of G and ET water which is called the water storage S available to infiltration (a total of 6.95 inches in table 1). Also shown in table 1 are the total water storage in the soil (item 13) and the hygroscopic water not available to infiltration (item 14). The various storages available in Alexis silt loam as calculated in table 1 are shown in figure 3.

Table 1. Computation of Water Storages Available in Alexis Silt Loam

Item	Description	Value			
	Soil horizons	1st	2nd	3rd	4th
1	Depth to bottom, *inches*	12	16	22	31
2	Thickness, *inches*	12	4	6	9
3	Saturated conductivity, *inches per hour*	1.30	3.74	2.53	0.46
4	Total porosity, *percent*	49.4	50.6	40.4	40.0
	Available storage				
5	At 0.3 bar tension, field capacity, *percent*	33.0	32.9	38.5	38.0
6	At 15 bars tension, wilting point, *percent*	13.1	15.3	20.8	23.9
	Gravitational water, G				
7	*percent*	16.4	17.7	1.90	
8	*inches*	1.97	0.71	0.11	
	ET water, or water a plant can withdraw				
9	*percent*	19.9	17.6	17.7	
10	*inches*	2.39	0.70	1.06	
	Water storage, S, available to infiltration				
11	*percent*	36.3	35.3	19.6	
12	*inches*	4.36	1.41	1.18	
13	Total water storage in soil, *inches*	5.93	2.02	2.42	3.60
14	Hygroscopic water, not available to infiltration, *inches*	1.57	0.61	1.24	

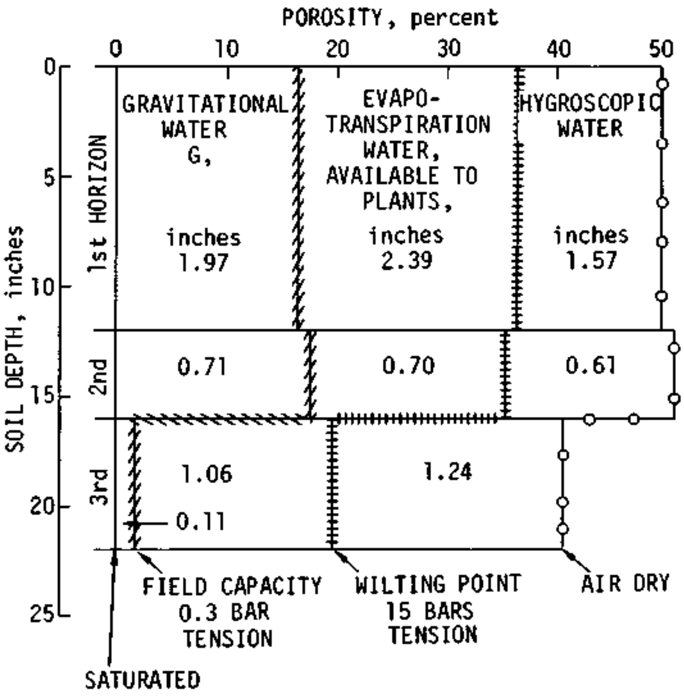


Figure 3. Water storages available within Alexis silt loam

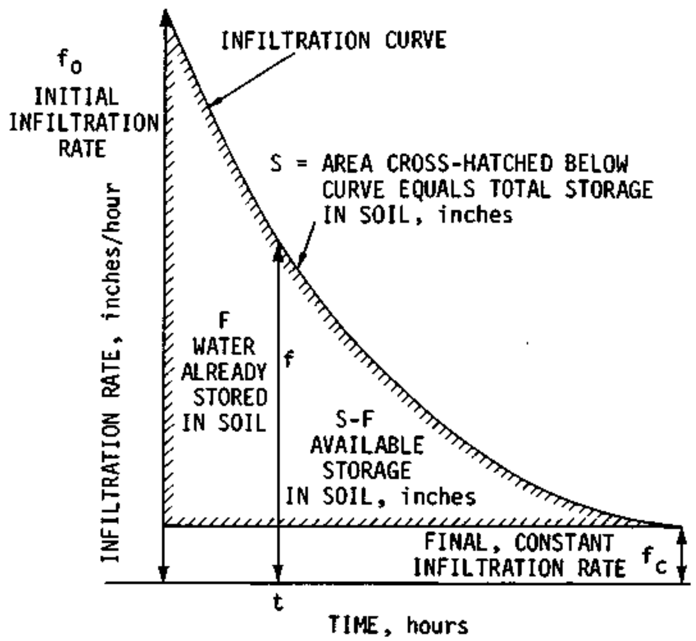


Figure 4. Diagram of infiltration curve and infiltration rates as related to storage in soil

Computed Infiltration

Knowledge of the water storage available to infiltration within a soil mantle makes it possible to compute the infiltration rate at any time *t* by methodology described by Holtan (1961):

$$f = a(S - F)^n + f_c \tag{4}$$

where

- f = infiltration rate at time *t*, in inches per hour
- a = a vegetative basal factor reflecting the efficiency a crop root system makes of soil porosity for storing water; a = 1.0 for bluegrass turf
- n = a constant = 1.4
- S = storage available in the soil mantle in inches (storage at the total soil porosity minus storage at the wilting point)
- F = water already stored in the soil at time *t*, in excess of the wilting point, in inches (amount accumulated from infiltration prior to time r)
- $(S — F)$ = storage space remaining in the soil mantle at the time *t*, in inches
- f_c = final constant infiltration rate, in inches per hour (generally equivalent to the saturated conductivity, in inches per hour, of the tightest horizon present in the soil profile)

With equation 4 it is possible to compute an infiltration curve based on the physical properties of the soil. Figure 4 shows the general interrelationship between the various infiltration rates and storage factors involved in equation 4.

Table 2 shows a computation of an infiltration curve with equation 4 for Alexis silt loam in which a water storage 5 of 6.95 inches is available as calculated in table 1. The computations in table 2 provide a series of infiltration rates in inches per hour at various times in hours. This computed infiltration curve for Alexis silt loam is the uppermost dashed line in figure 5.

Also shown in figure 5 are various other observed and computed infiltration curves including a computed infiltration curve for Ipava silt loam. In the lower part of figure 5 are results of actual infiltration rates observed on bluegrass turf at Elmwood, Illinois, described by Holtan and Musgrave (1947). Additional curves are shown for Tama silt

Table 2. Computation of Infiltration Curve for Alexis Silt Loam

$$f = 1(6.95\text{-}F)^{1.4} + 0.50$$

Available storage, *S - F* (*inches*)	*ΔF* (*inches*)	Water stored, *F* (*inches*)	*(S - F)*$^{1.4}$	Infiltration rate: *f* (*inches per hour*)	Infiltration rate: *favg* (*inches per hour*)	Time: *Δt** (*hours*)	Time: *t* (*hours*)
6.95		0	15.0	15.5			0
6.00	0.95	0.95	12.3	12.8	14.1	0.07	0.07
5.0	1.0	1.95	9.5	10.0	11.4	0.09	0.16
4.0	1.0	2.95	7.0	7.5	8.7	0.11	0.27
3.0	1.0	3.95	4.65	5.15	6.3	0.16	0.43
2.0	1.0	4.95	2.64	3.14	4.2	0.24	0.67
1.0	1.0	5.95	1.0	1.50	2.3	0.43	1.10
0	1.0	6.95	0	0.50	0.7	1.43	2.53

**Incremental time*, $\Delta t = \Delta F \div f_{avg}$

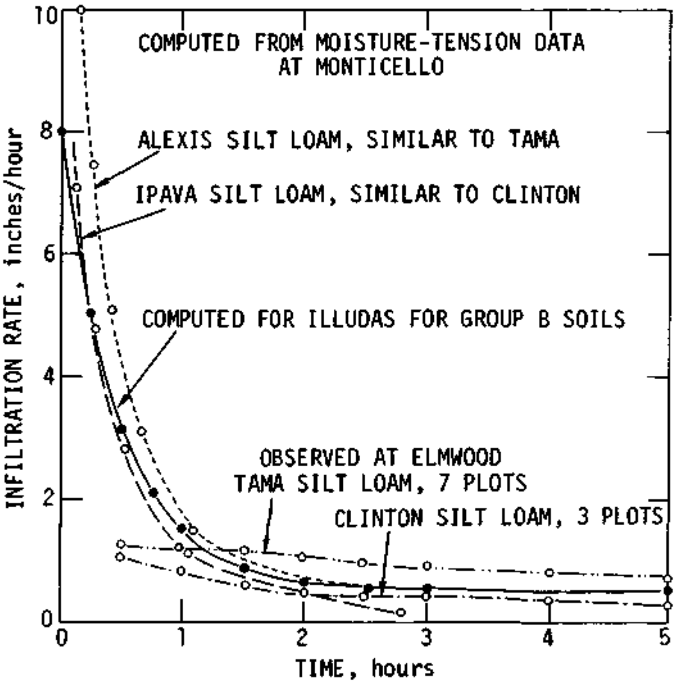


Figure 5. Infiltration curves for bluegrass turf

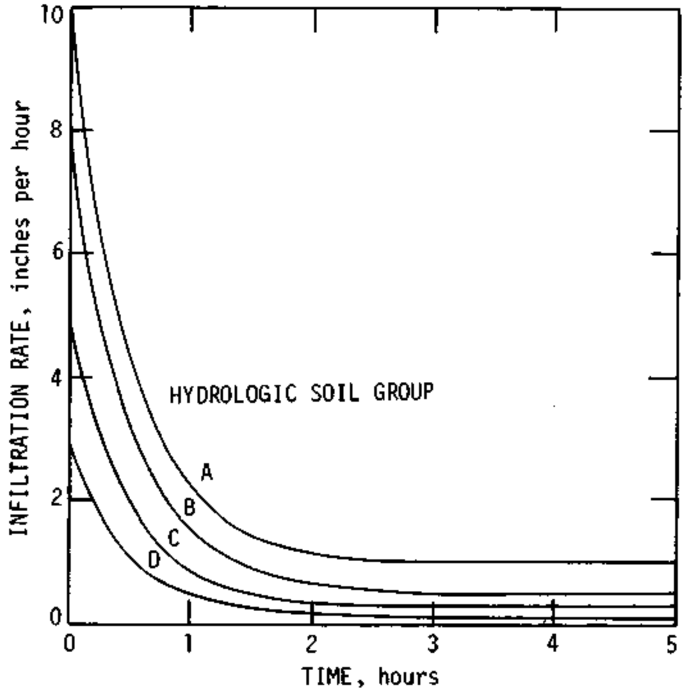


Figure 6. Standard infiltration curves for bluegrass turf used in ILLUDAS for soils of four hydrologic groups

loam and Clinton silt loam. There is a general physical similarity between Alexis and Tama silt loams and a similarity between Ipava and Clinton silt loams. All of these soils are graded as hydrologic group B and all occur in central Illinois. The curve ultimately computed for use in ILLUDAS for group B soils is shown as a solid line in figure 5.

The U. S. Soil Conservation Service describes the four hydrologic soil groups as follows:

- A — Low runoff potential, high infiltration rates (consist of sand and gravel)
- B — Moderate infiltration rates and moderately well drained
- C — Slow infiltration rates (may have a layer that impedes downward movement of water)
- D — High runoff potential, very slow infiltration rates, (consist of clays with a permanent high water table and a high swelling potential)

Standard infiltration curves have been devised for use in ILLUDAS for soils of hydrologic groups A, B, C, and D. These curves were calculated from the Horton equation as given by Chow (1964) as

$$f = f_c + (f_0 - f_c)e^{-kt} \qquad (5)$$

where

f_O = initial infiltration rate, inches per hour
e = base of natural logs
k = a shape factor selected as k = 2
t = time from start of rainfall

This equation is solved in ILLUDAS by the Newton-Raphson technique.

Table 3 lists the various factors selected for use in solving equation 5 for calculating the standard infiltration curves for bluegrass turf used in ILLUDAS shown in figure 6. Values of

Table 3. Factors Used in Equation 8 for Calculating the Standard Infiltration Curves for Bluegrass Turf

Item	Value			
Hydrologic soil group				
USDA designation	A	B	C	D
ILLUDAS designation	1 2		3	4
Final constant infiltration rate, f_c, *inches per hour*	1.0	0.50	0.25	0.10
Initial infiltration rate, f_O, *inches per hour*	**10**	8	**5**	3
Depression storage, *inches*	0.20	0.20	0.20	0.20
Shape factor, k, of infiltration curve	2	2	2	2
Available storage capacity, S, in soil mantle, *inches,* for four antecedent conditions				
Bone dry, condition 1	6	4	3	2
Rather dry, condition 2	4	2.5	2	1.3
Rather wet, condition 3	2	1	1	0.5
Saturated, condition 4	0	0	0	0
Infiltration accumulated, F, in soil mantle, *inches,* at start of rainfall				
Bone dry, condition 1	0	0	0	0
Rather dry, condition 2	2	1.5	1	0.7
Rather wet, condition 3	4	3	2	1.5
Saturated, condition 4	6	4	3	2

f_C and f_O were selected arbitrarily. The shape factor k was selected as equal to 2 to provide the shape best reflecting natural conditions as shown in figure 5. Selected representative values for the available storage capacity, S, for four antecedent moisture conditions were used to provide values of infiltration accumulated, F, in inches for each condition.

In order to use the standard infiltration curves shown in figure 6 to determine the grassed-area losses for use in ILLUDAS, it was necessary to evaluate the antecedent moisture conditions actually prevailing on the urban basin at the time of a particular storm. An arbitrary selection of antecedent moisture conditions (AMC) that would have general value was made, as shown in table 4. Each condition is based on the total rainfall that occurred during the 5 days preceding the storm. The values in table 4 were used throughout the calibration portion of this research and for the 23 basins studied. Results have been generally favorable.

Table 4. Antecedent Moisture Conditions for Bluegrass Lawns

ILLUDAS number	Description	Total rainfall during 5 days preceding storm *(inches)*
1	Bone dry	0
2	Rather dry	0 to 0.5
3	Rather wet	0.5 to 1
4	Saturated	over 1

Routing Procedure

After the paved-area and grassed-area hydrographs are combined as a single surface hydrograph from each sub-basin, this hydrograph becomes a point input into the drainage network. A simple storage routing technique is used to pass the hydrograph from one input point to the next. For this technique, a determinate relationship must exist between discharge and storage for the reach of channel or pipe between the input points. Such a relationship is developed by first using Manning's equation to compute a stage-discharge curve for the cross section in question. Since the length and geometry of the reach are known, the required discharge-storage relationship may be computed by assuming uniform flow in the particular reach. Errors incurred by this assumption are minimized by keeping the time increment and reach length as short as practical. ILLUDAS provides routing through circular, trapezoidal, and rectangular sections.

Figure 7 shows two curves, $0Q_{1in}Q_{2in}$ which is a section of the inflow hydrograph at the upper end of the reach, and $0Q_{1out}Q_{2out}$ which is a section of the outflow hydrograph at the lower end of the reach. S_1 and S_2 are the total storage at times t and $2t$ respectively. From figure 7, area $0ac$ = areas $0ae + 0ec$ or

$$1/2 \quad (Q_{1in}t) = S_1 + 1/2 \quad (Q_{1out}t) \qquad (6)$$

Since Q_{1in} and t are known, the right side of equation 6 may be evaluated. Because S_1 is known in terms of Q_{1out} from the discharge-storage relationship described eariler, the equation

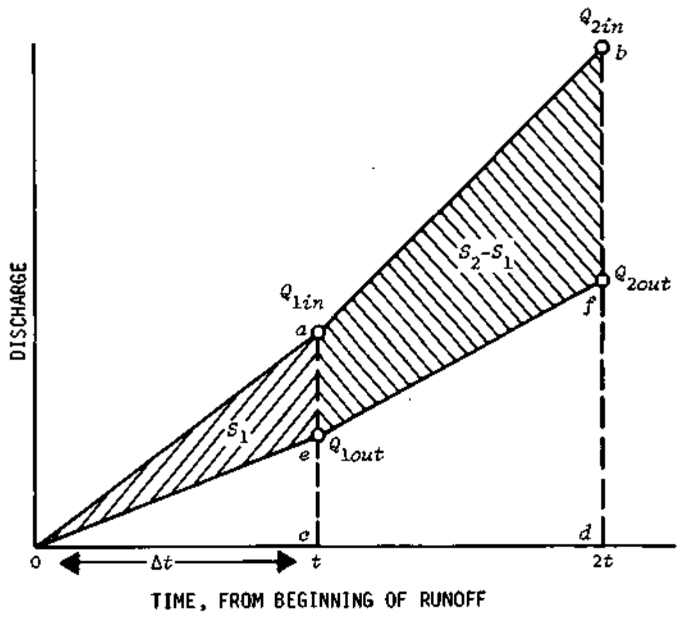


Figure 7. Elements in the storage routing technique

can be solved for Q_{1out} For the period t to $2t$, area $abcd$ = areas $efdc + abfe$, or

$$t/2 \ (Q_{1in} + Q_{2in}) = t/2 \ (Q_{1out} + Q_{2out}) + (S_2 - S_1) \quad (7)$$

$$t/2 \ (Q_{1in} + Q_{2in} - Q_{1out}) + S_1 = t/2 \ Q_{2out} + S_2 \quad (8)$$

Since the left side of equation 8 is known, the right side may again be solved for Q_{2out} from the discharge-storage relationship. By this step-by-step procedure all ordinates of the downstream or routed hydrograph may be determined.

Flood Water Detention Basins

In the 1970s it has become common practice, where needed, to provide artificial, man-made detention basins to provide for the temporary storage of flood waters from an urban area during flood times. After the flood peak recedes, water from the detention basin is emptied into the storm drainage system. The provision of such basins can be economical because the basin cuts down greatly on the maximum required capacity of the pipes removing the storm drainage water. A schematic view of a typical basin is shown in figure 8.

Temporary flood detention can also be provided along open drainageways in natural channels throughout an urban region if desired. This has been accomplished in Madison, Wisconsin, by city ordinance. Here a greenway is defined as being an open area of land, the primary purpose of which is to carry storm water on the ground surface in lieu of an enclosed storm drain. Where these greenways or drainageways are shown on the official master plan of the city, developers are required to provide that these areas be reserved for the greenway for acquisition by the city or by the township. Drainageways are to have a minimum width of 200 feet. It is required

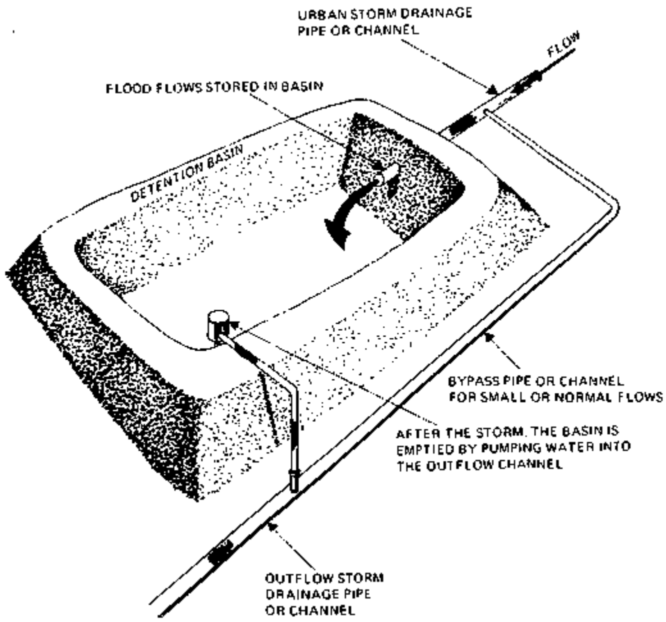


Figure 8. Schematic view of a detention basin for an urban basin

where possible that storm water drainage be maintained by landscaped open channels adequate to accommodate the maximum expected storm flows.

Rice (1971) has shown that urban storm water can be retained in decorative ponds or depressed areas integrated into the landscape plan, in small ponds forming part of the major drainage system, or in off-channel detention basins in the greenbelt. Release rates from detention basins can be made suitable to the channel capacity. An example of the use of a floodwater detention basin has been described by Antoine (1964). On Deer Creek in St. Louis a flood problem was alleviated by diverting flood flows into a rock quarry. For a 20-year recurrence interval storm, a total of 36.5 million cubic feet of water was temporarily stored in the quarry. In this way the storm drainage system was not called upon to accommodate the design peak of the 20-year storm of 31,850 cfs. The flow in the storm drainage channel was limited to 16,000 cfs which was allowable in the existing channel. After flood waters receded, the quarry was de-watered by pumping.

Regulations of the Metropolitan Sanitary District of Greater Chicago (1970) now require storage for storm water runoff as part of any new residential development exceeding 10 acres in size. The release rate of storm water from such developments is not to exceed the storm water runoff rate from the area in its natural undeveloped state. Detention storage is to be provided to handle the runoff from a 100-year rainfall for all durations, minus that volume discharged at the approved release rate into the natural channel. During 1972 about 135 permits were issued by the Metropolitan Sanitary District for the construction of such on-site detention basins.

ILLUDAS assists the user in the design of detention basins in several ways. First, if an existing system is being analyzed, ILLUDAS accumulates flows greater than the capacity of the existing pipe for each reach in the basin. The maximum volume of flow thus accumulated is equivalent to the detention storage required to keep the system operating at capacity during passage of the design storm. These accumulated flows are reported on the output and serve to pinpoint the location and severity of flooding in the basin.

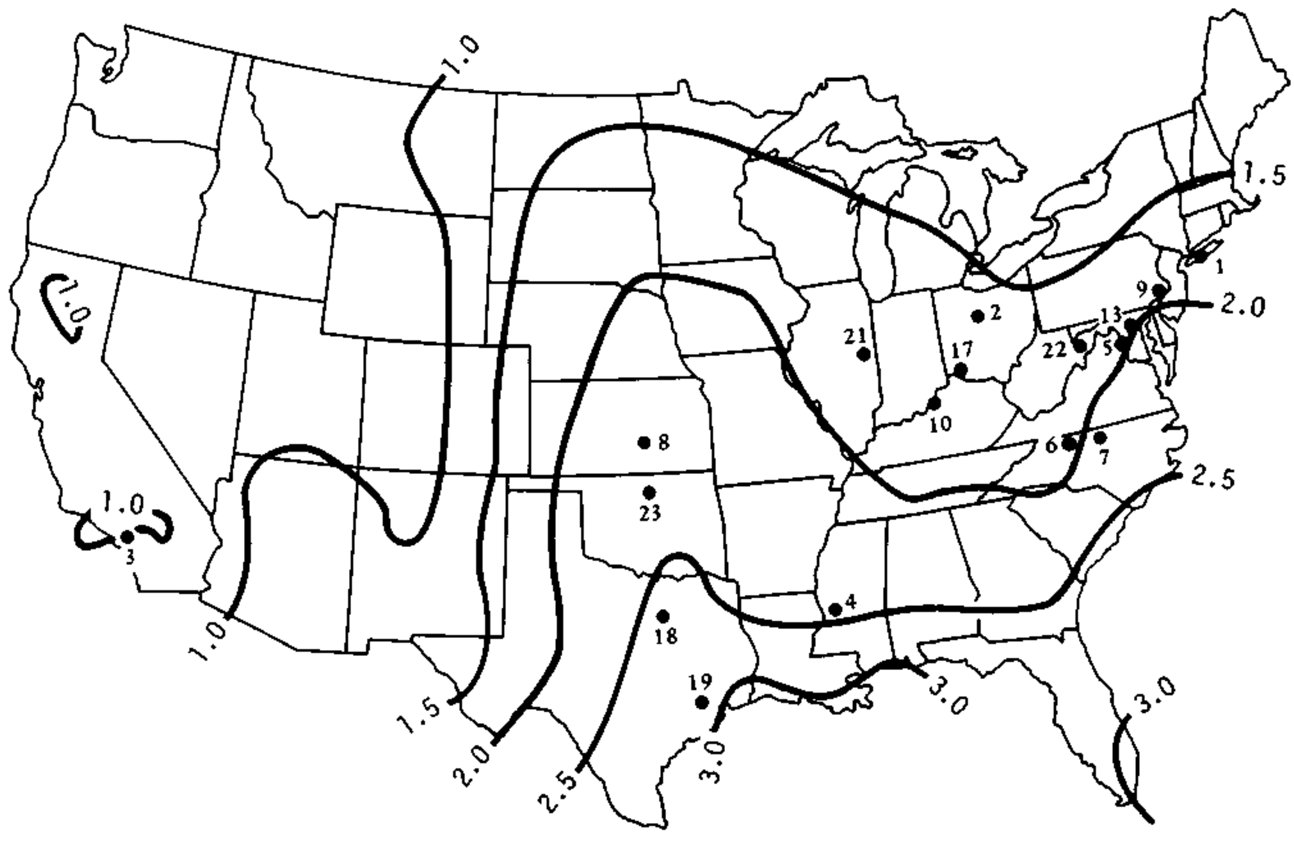
If a new drainage system is being designed, the user may specify the volume of detention storage allowable at any point in the basin. ILLUDAS will then incorporate that volume of storage into the design by allowing incoming flows to fill the allowable storage. The outlet capacity needed to make effective use of this storage will also be provided by ILLUDAS.

As an additional option the user may limit flow through a given reach by specifying a small outlet pipe size or a maximum discharge through the reach, and ILLUDAS will report the volume of detention storage accumulated during passage of the design storm.

Part 2. Verification

The results of ILLUDAS applications to 21 urban and 2 rural basins are presented in this section. The distribution of the basins is indicated by the map in figure 9. As an aid in comparing the relative intensities of the many storms studied, figure 9 also shows the 5-year 1-hour storm rainfall from Technical Paper 40, U. S. Weather Bureau (1961). The 21 urban basins range in size from 0.39 acres to 8.3 square miles and are from 21 to 100 percent impervious or paved. The availability of rainfall-runoff as well as basin data was the primary consideration in selecting these basins. It was also felt desirable to have basins of less than 10 square miles with no major agricultural areas.

In the following pages the basins are presented in the order shown on figure 9. For each there is a brief description of the basin, soil type, instrumentation, and any distinguishing features. There is also a discussion of the results for each basin. A table similar to table 5 presents the basic data used on each basin, the observed storm data, and the computed results. A figure similar to figure 10 is also included for each basin. These figures are in three parts, showing (a) computed peaks plotted against observed peaks, (b) computed runoff vs observed runoff, and (c) a complete computed and observed hydrograph and the hyetograph for one storm. Where available, a general view of the basin and an aerial photograph are also included.



1 Woodoak Drive Basin, Westbury, Long Island, New York
2 Sewer District No. 8 Basin, Bucyrus, Ohio
3 Echo Park Avenue Basin, Los Angeles, California
4 Crane Creek Basin, Jackson, Mississippi
5 Tripps Run Tributary Basin, Falls Church, Virginia
6 Tar Branch Basin, Winston-Salem, North Carolina
7 Third Fork Basin, Durham, North Carolina
8 Dry Creek Basin, Wichita, Kansas
9 Wingohocklng Basin, Philadelphia, Pennsylvania
10 First Street Basin, Louisville, Kentucky
11 Seventeenth Street Basin, Louisville, Kentucky
12 Northwestern Basin, Louisville, Kentucky
13 Montebello No. 4 Basin, Baltimore, Maryland
14 Northwood Basin, Baltimore, Maryland
15 Gray Haven Basin, Baltimore, Maryland
16 South Parking Lot No. 1 Basin, Baltimore, Maryland
17 Mt. Washington Basin, Cincinnati, Ohlo
18 Turtle Creek Basin, Dallas, Texas
19 Hunting Bayou Basin at Cavalcade Street, Houston, Texas
20 Hunting Bayou Basin at Falls Street, Houston, Texas
21 Boneyard Creek Basin, Champaign-Urbana, Illinois
22 Watershed No. 4, Moorefield, West Virginia
23 Watershed W-1, Stillwater, Oklahoma

Figure 9. Distribution of the 21 urban and 2 rural basins, and the 5-year 1-hour storm rainfall from Technical Paper 40, U. S. Weather Bureau (1961)

Woodoak Drive Basin, Westbury, Long Island, New York

Woodoak Drive basin is a 14.7-acre residential area all of which drains to one set of inlets (figure 10). Because of its small size and the fact that only one length of pipe exists, it was not necessary to divide the basin into smaller sub-basins. Street slopes are less than 1 percent and yard slopes were estimated to be less than 3 percent. The dominant soil in the area is Haven loam which is classified in hydrologic group B. The existence of highly permeable gravelly sand at a depth of 2 or 3 feet accounts for the success of recharge basins which are common in this area of Long Island *{note figure 12).*

Most driveways in the area are paved and are either full-width drives or narrow strips of concrete that accommodate car tires. Roof drains appear to flow onto full-width drives where such drives exist and onto the grass in other cases. The directly connected paved area therefore consists of the streets, all driveway aprons, all full-width driveways, and the front half of roofs located adjacent to full-width drives.

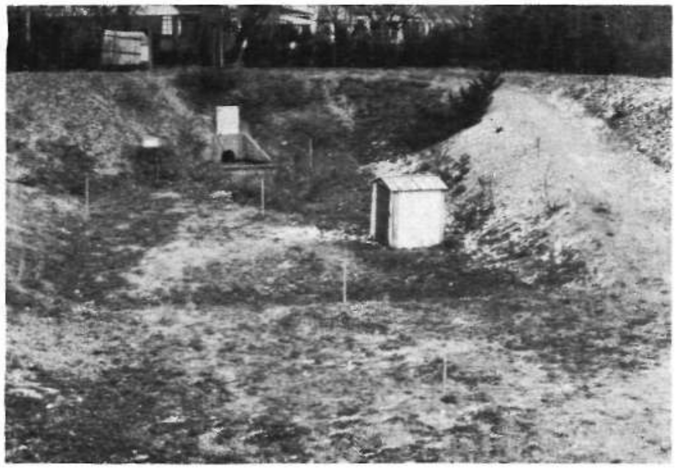


Figure 10. Outlet of the single storm drain in the Woodoak Drive basin

Data

Flow measurements are made at 5-minute intervals by a digital stage recorder located behind a V-notch weir in a 24-inch concrete pipe. The instrumentation, including a water table measuring system, are described in detail by Seaburn (1970). Flow data were provided for this project in printed form with discharge in cfs at 5-minute intervals.

Rainfall is recorded on a weighing-bucket gage located about 300 yards southeast of the basin. Copies of the original weekly charts were provided for this study. The charts were replotted with a larger time scale and read at 10-minute intervals.

Results

ILLUDAS works well on this basin (figure 11) as did the RRL method (see Stall and Terstriep, 1972). Table 5 shows that grassed-area runoff occurs only on storm number 10. There appears to be a correlation between high antecedent moisture (AMC) and negative runoff errors in the computed results. This correlation seems to indicate that there is grassed-area runoff occurring during these *wet period* storms. Such runoff could indicate either that the ILLUDAS infiltration curve for group B soils is too high or that a more impervious soil mantle is developing in this residential area.

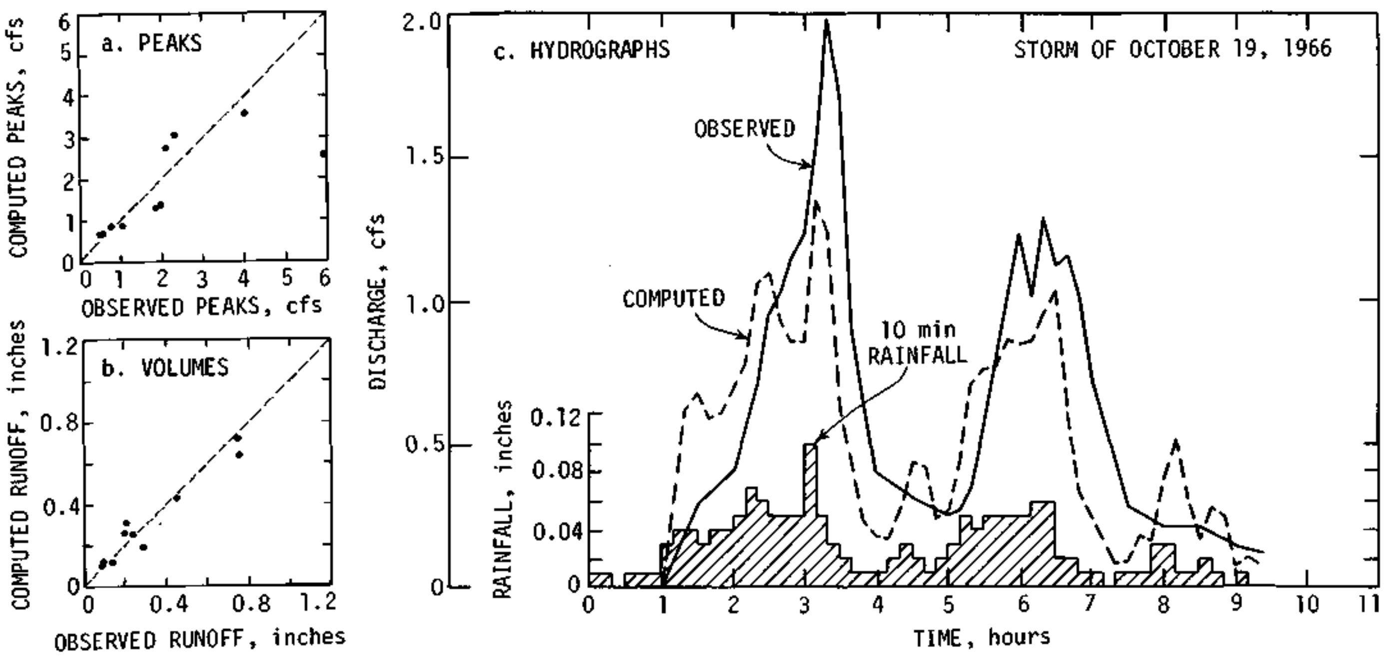


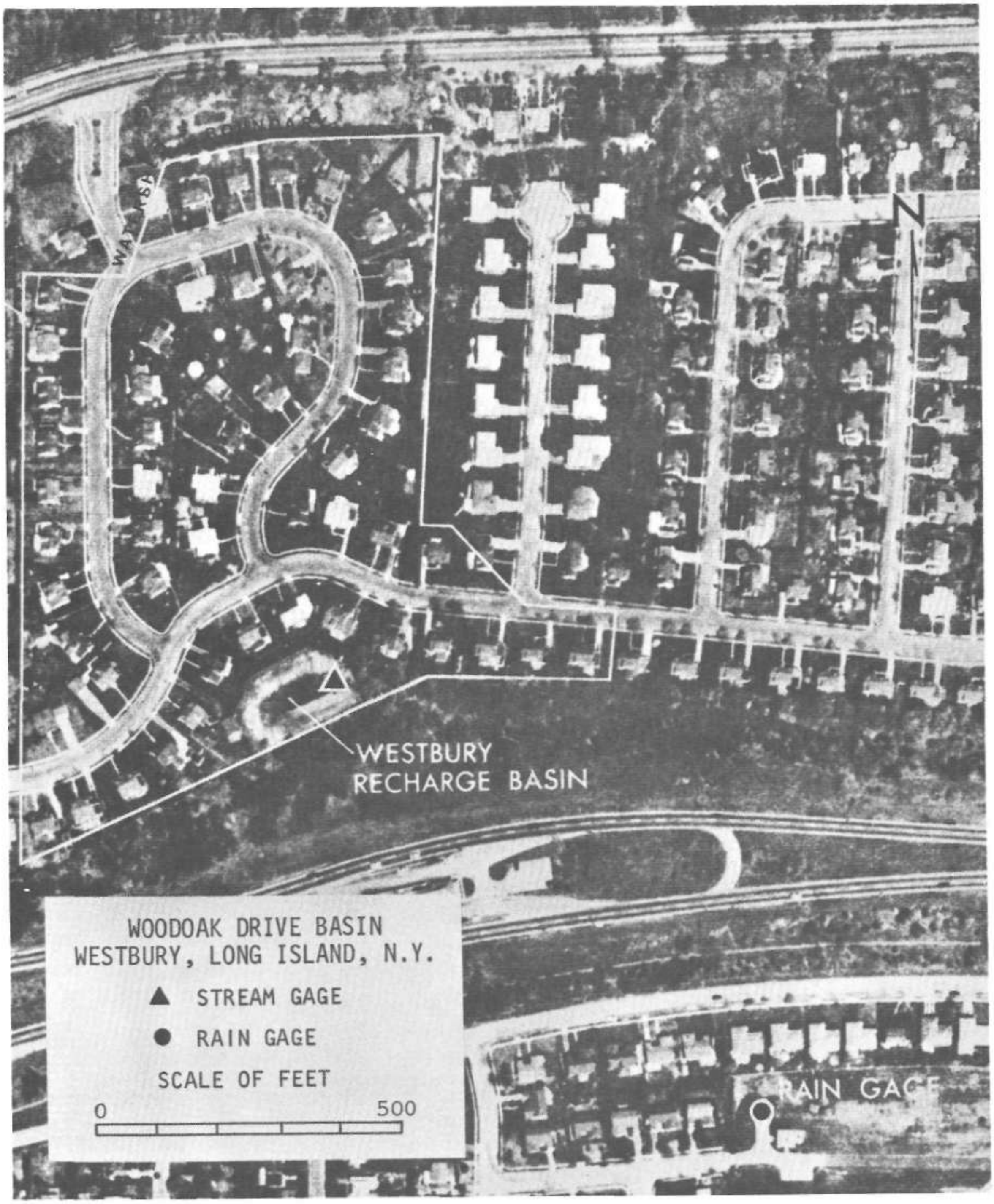
Figure 11. ILLUDAS results for Woodoak Drive basin

Table 5. Storm Data and Results for Woodoak Drive Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
14.7 acres	4.9 acres	2.85 acres	1.29 acres	2.48 acres
	33.9 percent	19.4 percent	8.8 percent	16.9 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	91466	2	1.42	0.76	0.24	0.17	0.25	0.0	4.7	0.85	12.3
2	92166	1	3.48	5.93	0.75	0.22	0.64	0.0	-15.6	2.53	-57.3
3	101966	2	2.38	1.98	0.45	0.19	0.43	0.0	-4.9	1.35	-31.7
4	102066	4	0.76	1.84	0.14	0.18	0.12	0.0	-11.4	1.27	-30.9
5	42766	3	1.49	0.59	0.20	0.13	0.26	0.0	30.4	0.68	16.0
6	50667	2	0.65	0.53	0.09	0.15	0.10	0.0	9.7	0.69	29.4
7	50767	4	1.13	1.07	0.29	0.26	0.19	0.0	-32.8	0.86	-19.9
8	82567	2	1.74	2.30	0.21	0.12	0.31	0.0	45.1	3.02	31.3
9	82667	4	0.72	2.10	0.10	0.14	0.12	0.0	16.1	2.73	30.2
10	52968	3	3.60	4.00	0.75	0.21	0.72	8.4	-4.1	3.57	-10.8
Mean values						0.18			17.5		27.0

Computed peaks were high in 5 cases, average + error = 23.9 percent
Computed peaks were low in 5 cases, average – error = 30.1 percent
Computed runoff volumes were high in 5 cases, average + error = 21.2 percent
Computed runoff volumes were low in 5 cases, average – error = 13.8 percent



(Photo by Lockwood, Kessler, & Bartlett)

Figure 12. Aerial photo of Woodoak Drive basin

Sewer District No. 8 Basin, Bucyrus, Ohio

The 206-acre No. 8 Sewer District basin lies within the older section of Bucyrus, Ohio. Land use varies from residential to commercial and heavy industrial. The entire basin is served by a combined sewer system, but there does not appear to be an adequate number of inlets to drain the basin properly. Dry-weather flow is intercepted above the gage. The combined sewer system is quite extensive and was represented in the model by dividing the basin into 42 sub-basins. With the exception of a few short roadside swales, there is no open channel drainage. Street slopes are generally less than 1 percent and yard slopes less than 3 percent. Principal soils in the area are Bennington silt loam and Luray silty clay loam. These soils are both classified in hydrologic group C-D.

Determination of the directly connected paved area was complicated by the lack of curb and gutter on many streets in the northern part of the basin. Runoff from many of these streets (figure 13) would apparently find its way into adjacent low-lying yards and vacant lots or be ponded on the street. A sample inspection of roofs during field investigations indicated that about 60 percent of the residential roofs are directly connected to the combined sewer system. The directly connected paved area thus consisted of all streets with curb and gutter, a 10-foot strip for streets without curb and gutter, driveways on curbed and guttered streets, major buildings, and 60 percent of the residential roofs.

Data

Instrumentation, as described by Burgess and Niple (1969), consisted of a Stevens Type-F stage recorder located behind an 8-foot rectangular weir. The data were provided in the form of plotted discharge hydrographs. Since the distance between the 42-inch outfall pipe and the weir was necessarily small, approach velocities would have had an effect on the measurement of high flows.

Figure 13. Flat terrain and indeterminate drainage pattern typical of much of the upstream basin of Sewer District No. 8

Rainfall data were collected on a Bendix weighing-bucket gage with a 24-hour chart. The data received for this study had been digitized at 10-minute intervals.

Results

ILLUDAS badly overpredicts the measured flows on this basin (figure 14 and table 6). The primary reason for this is the amount of ponding that occurs throughout the basin. No attempt was made to include this detention in the analysis of the basin. The ponding was probably greatly increased by undersized inlets and inlets in poor condition. Another factor in the overprediction has to be the actual measured flows. The outlet from the basin discharges directly behind and within 10 to 12 feet of the 8-foot weir used to measure flows (figure 15). During high flows, approach velocities at the weir are surely affected by the relatively high outlet velocity from the pipe.

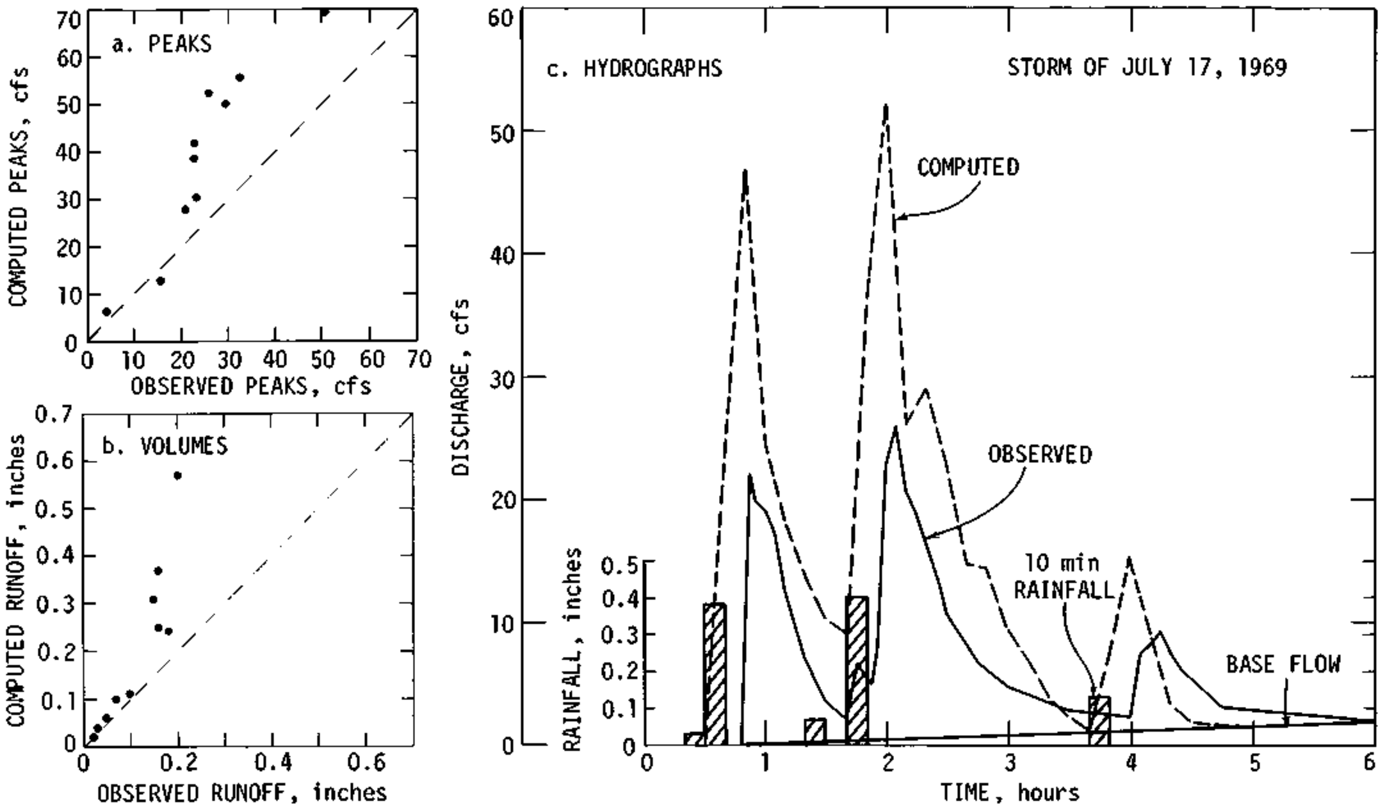


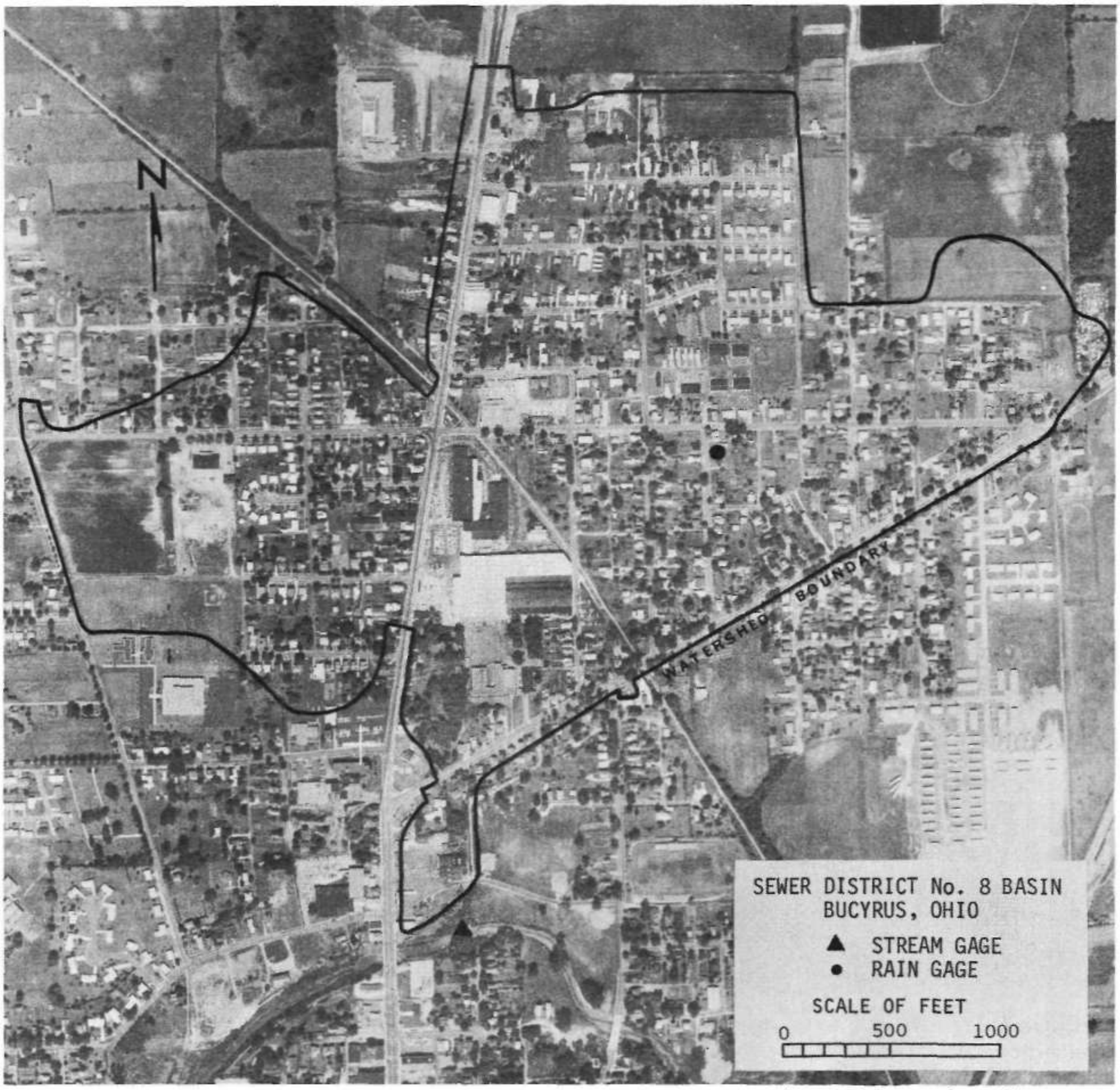
Figure 14. ILLUDAS results for Sewer District No. 8 basin

Table 6. Storm Data and Results for Sewer District No. 8 Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
206 acres	43 acres	37.5 acres	5.5 acres	40 acres
	21 percent	18.2 percent	2.7 percent	19.4 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	32469	2	0.31	4.4	0.03	0.10	0.04	0	18.7	6.1	37.9
2	40569	2	1.47	22.8	0.16	0.11	0.25	1	61.6	41.9	83.6
3	51769	3	1.37	32.4	0.16	0.11	0.37	36	132.4	55.4	70.9
4	61369	2	1.20	29.5	0.18	0.15	0.24	15	32.3	50.1	69.7
5	71169	4	1.55	50.8	0.20	0.13	0.57	50	184.2	69.1	36.1
6	71769	4	1.01	25.8	0.15	0.14	0.31	44	111.3	52.1	101.8
7	72769	1	0.40	20.9	0.05	0.12	0.06	1	12.3	27.9	33.4
8	80969	3	0.51	22.7	0.07	0.14	0.10	21	33.5	38.6	70.2
9	81669	1	0.70	23.1	0.10	0.14	0.11	0	8.2	30.1	30.3
10	90669	2	0.23	15.9	0.02	0.09	0.02	0	11.6	12.9	-19.1
Mean values						0.13			60.6		55.3

Computed peaks were high in 9 cases, average + error = 59.3 percent
Computed peaks were low in 1 case, average — error = 19.1 percent
Computed runoff volumes were high in 10 cases, average + error = 60.6 percent
Computed runoff volumes were low in 0 cases



(USDA photos of 1970)

Figure 15. Aerial photo of Sewer District No. 8

Echo Park Avenue Basin, Los Angeles, California

The Echo Park Avenue basin is primarily a residential area with commercial strips along the main streets. The basin has a deep valley configuration (figure 16). Runoff flows down very steep side streets to an interceptor flowing north-to-south along the center of the valley. Minimum slopes in the basin occur down the center of the valley where they vary from 2 to 4 percent. On the side streets, slopes approach 20 percent, and on landscaped areas slopes of 30 percent are not uncommon. The dominant soil in the basin according to a 1916 survey is Altamont loam. Under natural, undisturbed conditions, this soil would be in hydrologic group B or C depending on the depth to bedrock and the degree to which the rock is weathered.

Surveys by the city of Los Angeles fixed the total paved area at 136 acres. These surveys showed that 54 percent of the total paved area was in streets and parking, and that the other 46 percent was in roofs. An additional roof survey indicated that 40 percent of the roofs are connected to the streets and 60 percent to the lawns. The directly connected paved area thus consisted of 73 acres of streets and parking (136 x 0.54) and 25 acres of connected roofs [(136 — 73) x 0.40] for a total of 98 acres. The remainder of the paved area was included in the supplemental paved area. All grassed area was considered to be contributing grassed area.

Figure 16. Street view in Echo Park Avenue basin

Data

Stage hydrographs are recorded in a 51-inch concrete storm sewer by the Bureau of Engineering at the City of Los Angeles. The original charts along with a rating table based on Manning's equation assuming uniform flow and a 0.013 'n' value were furnished by the bureau. Concerning the Echo Park data, Crawford (1971) has recently commented that "the flow data could be in error by more than 20 percent due to uncertainty in the roughness and the supercritical flow velocities in the sewer."

Rainfall is recorded on a weighing-bucket type gage *(see location in figure 18)* on a standard 24-hour chart. These charts were provided by the Bureau of Engineering and were digitized, as a regular part of this project, by the Water Survey Model 3400 autotrol. A 4-minute interval was used for rainfall. Because of the short entry times and quick response of this basin, an even shorter time interval would have been desirable.

Results

Table 7 shows that grassed-area runoff occurs on 12 of the 18 storms. The total runoff volumes computed by ILLUDAS are acceptable, but the peaks are consistently underestimated *(note figure 17).* The 4-minute minimum time interval is too long to represent properly the short inlet times occurring on many of the sub-basins used on Echo Park. The extreme steepness of this basin appears to be a boundary condition for the use of ILLUDAS. For these conditions, 1-minute rainfall amounts would be desirable, and the sub-basins should be checked for supercritical flow during the computation of inlet times.

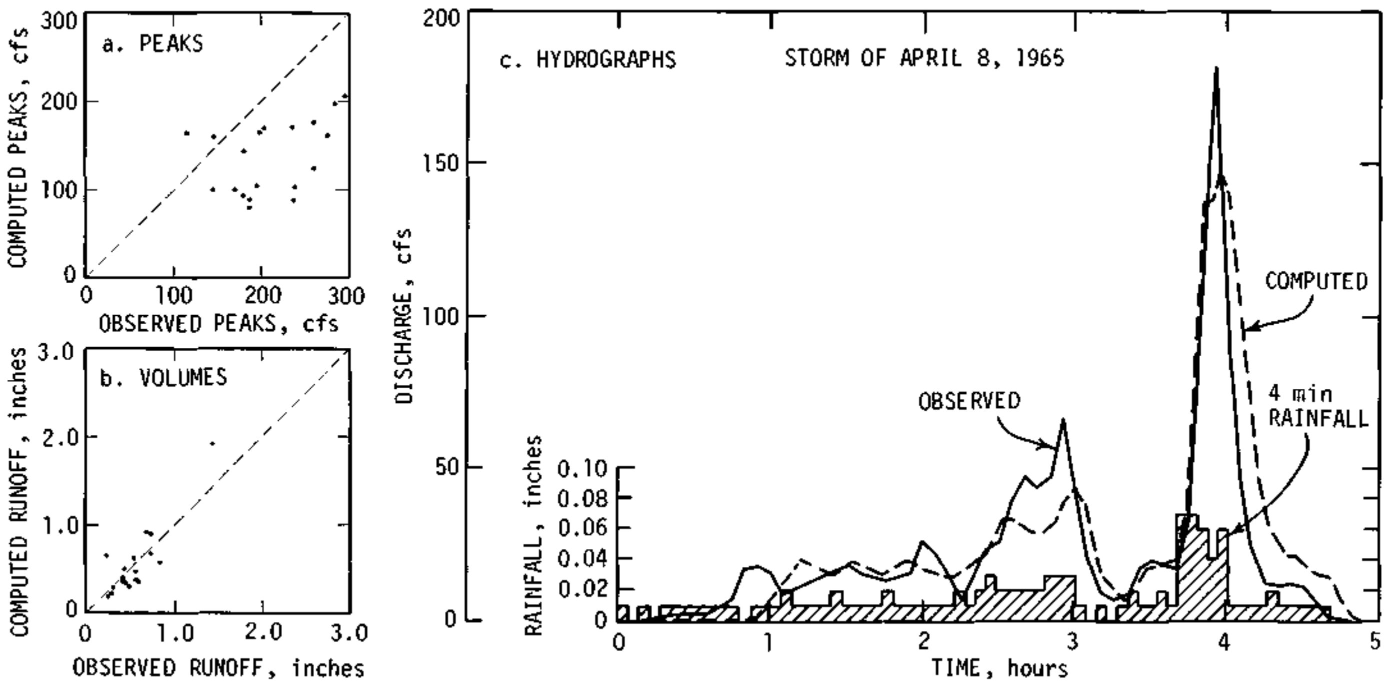


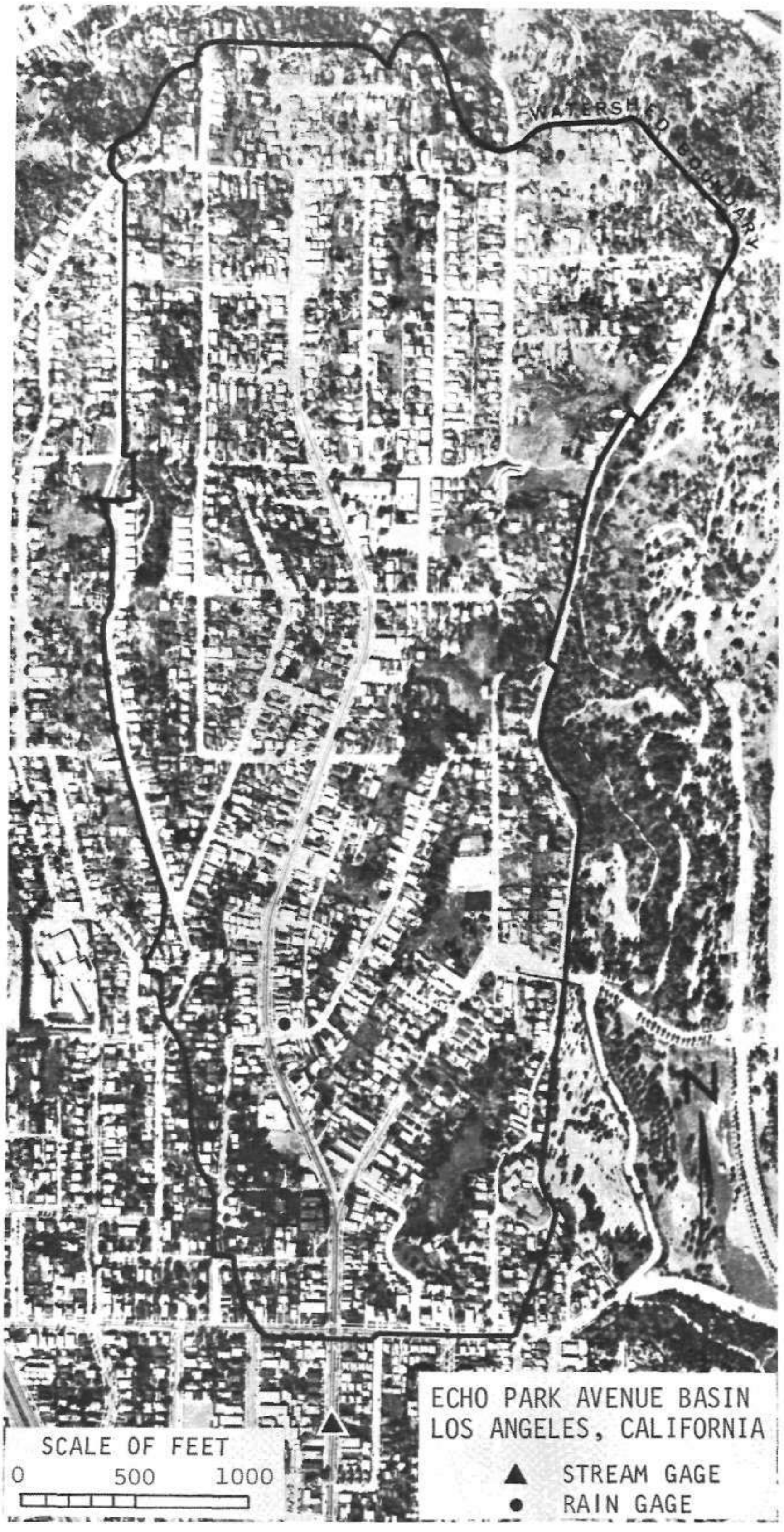
Figure 17. ILLUDAS results for Echo Park Avenue basin

Table 7. Storm Data and Results for Echo Park Avenue Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
252 acres	136 acres	97.7 acres	38.3 acres	116 acres
	53.8 percent	38.8 percent	15.2 percent	46.0percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	20358	2	0.66	275	0.29	0.45	0.22	0	-26.6	161	-41.4
2	20458	4	1.10	260	0.56	0.51	0.47	18	-16.1	176	-32.2
3	21958	1	3.43	295	1.43	0.42	1.91	32	33.1	207	-29.9
4	21262	4	0.68	234	0.42	0.61	0.40	44	-5.0	171	-27.2
5	21962	4	1.54	204	0.68	0.44	0.92	40	35.8	170	-16.9
6	20963	1	2.38	170	0.73	0.31	0.90	3	22.9	100	-40.9
7	12164	2	1.06	178	0.41	0.38	0.37	0	-9.5	93	-47.6
8	12264	4	0.54	187	0.25	0.47	0.18	8	-27.5	80	-56.8
9	40865	4	1.11	181	0.44	0.39	0.50	22	14.8	143	-21.0
10	40965	4	1.30	199	0.74	0.57	0.67	31	-10.2	165	-16.8
11	111967	1	0.88	260	0.49	0.55	0.30	0	-38.2	125	-51.9
12	112067	4	0.49	284	0.31	0.62	0.28	46	-9.4	198	-30.4
13	12669	4	0.85	187	0.56	0.66	0.37	21	-34.2	88	-52.6
14	20669	2	1.01	238	0.60	0.59	0.35	0	-41.7	105	-55.9
15	21569	2	1.00	196	0.45	0.45	0.35	0	-23.2	106	-46.0
16	22569	4	1.33	146	0.84	0.63	0.57	17	-32.7	100	-31.2
17	30470	4	1.35	146	0.55	0.41	0.61	22	10.9	160	9.2
18	122170	4	1.35	116	0.24	0.18	0.65	26	171.6	163	41.4
Mean values						0.48			31.3		36.1

Computed peaks were high in 2 cases, average + error = 25.3 percent
Computed peaks were low in 16 cases, average — error = 37.4 percent
Computed runoff volumes were high in 6 cases, average + error = 48.2 percent
Computed runoff volumes were low in 12 cases, average — error =22.8 percent



(Photo by Geotronics, Long Beach, California, 1968)

Figure 18. Aerial photo of Echo Park Avenue basin

Crane Creek Basin, Jackson, Mississippi

The Crane Creek basin is a 273-acre residential area. Two large schools, a church, and an apartment complex have a significant effect on the paved area runoff. There are large open areas around the schools and in the floodplain in the lower part of the basin. Street slopes range from 1 to 30 percent and yard slopes vary from 2 to 6 percent. The drainage system as represented in the model has 11 open channel reaches with a total length of 5700 feet and 15 closed conduits with a total length of 6800 feet. The primary soil in the basin is a Loring silt loam which is classified in hydrologic group C. In the floodplain area, Falaya series soils of hydrologic group D should be expected.

The absence of curb and gutter on many streets complicates the determination of directly connected paved area. All such streets have well-maintained roadside ditches (figure 19) and conceivably the contributing roadway could include everything between the roadside ditches. For this study, however, a 20-foot strip of contributing area was used for streets without curb and gutter. In addition to the streets, all major buildings, parking lots, and an approximation of residential driveways were included in the directly connected paved area. The supplemental paved area consists of about one-half of the residential roofs which generally drain onto grass. The contributing grassed area consists of the front yard strip plus about one-half of the large grassed areas *(note figure 21).*

Data

The instrumentation for this basin is typical for a USGS installation. One digital recorder provides hydrographs at a rated culvert on Crane Creek. Another digital recorder provides rainfall at the same site. The recorders operate from the same clock at 5-minute intervals. Rainfall and discharge data were provided in both tabular and plotted form for this study. Urban runoff effects for several basins in Jackson have been published elsewhere by Wilson (1968).

Figure 19. Ditch along street in Crane Creek basin

Results

Both the peaks and the runoff volume predictions are low but acceptable (figure 20 and table 8). Table 8 shows that there is significant grassed runoff in 12 of the 17 storms. The consistent nature of the runoff error indicates that the directly connected paved area used for Crane Creek is somewhat low. In general ILLUDAS does a good job on this basin.

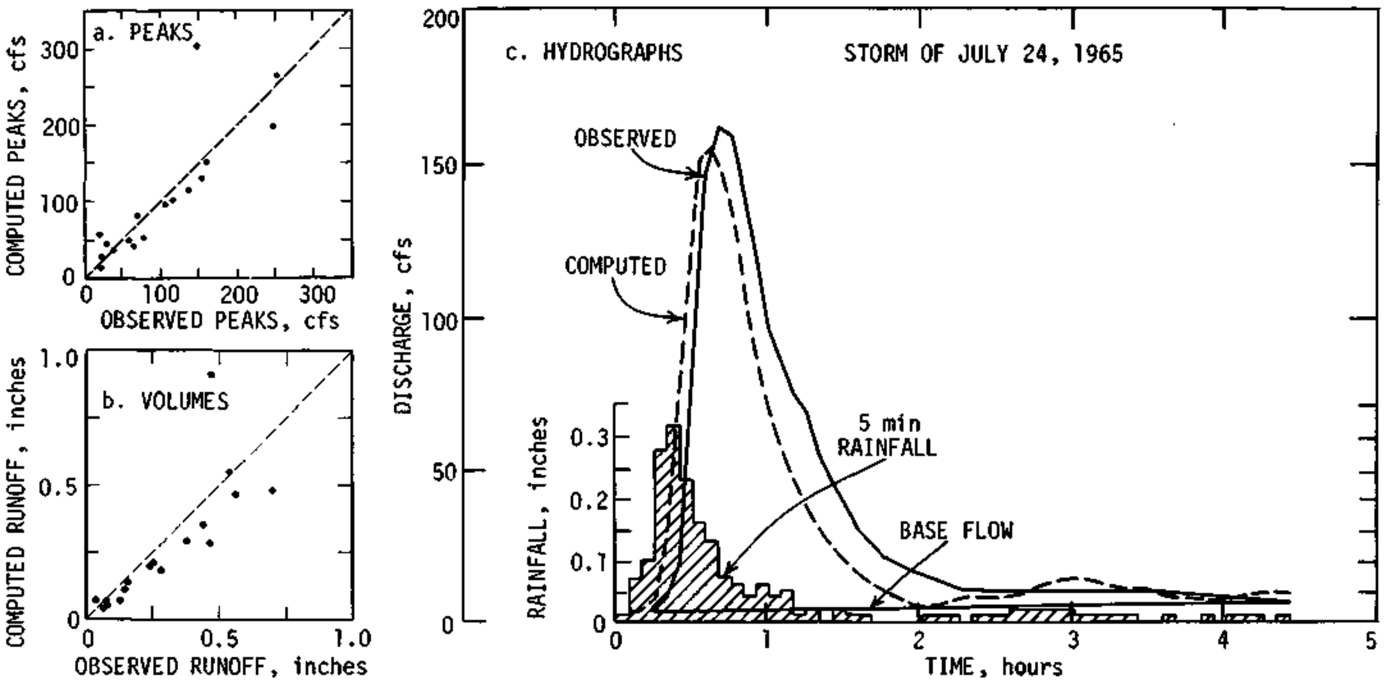


Figure 20. ILLUDAS results for Crane Creek basin

Table 8. Storm Data and Results for Crane Creek Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
273 acres	65.5 acres	39.7 acres	14.2 acres	128.8 acres
	23.9 percent	14.5 percent	5.2 percent	47.1 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	51565	2	0.38	39	0.07	0.19	0.04	0	-50.9	38	-2.0
2	62465	2	1.44	106	0.24	0.17	0.20	9	-19.8	97	-8.5
3	62565	4	0.78	65	0.15	0.20	0.11	19	-29.5	42	-34.4
4	72465	2	2.00	161	0.42	0.21	0.35	29	-15.0	152	-5.2
5	81265	4	1.81	149	0.47	0.26	0.91	75	95.2	302	102.7
6	82065	2	0.64	29	0.08	0.12	0.07	0	-13.0	46	55.2
7	91065	1	1.71	78	0.25	0.15	0.21	0	-15.8	54	-30.9
8	91165	4	1.27	253	0.54	0.42	0.55	72	2.6	256	1.0
9	92265	3	0.58	20	0.04	0.07	0.07	19	96.9	59	191.1
10	100665	2	1.14	70	0.16	0.14	0.14	5	-13.3	83	18.4
11	10466	4	1.79	60	0.46	0.26	0.29	26	-36.5	50	-17.2
12	20166	4	0.45	23	0.08	0.17	0.05	14	-30.1	29	23.0
13	22666	3	0.65	22	0.13	0.20	0.07	0	-48.2	15	-33.1
14	30366	4	0.57	137	0.29	0.50	0.19	66	-35.0	115	-16.1
15	42066	2	3.23	154	0.70	0.22	0.48	15	-31.5	132	-14.3
16	42666	4	1.09	116	0.38	0.35	0.29	54	-25.5	103	-11.4
17	52366	4	1.16	248	0.56	0.48	0.47	70	-15.2	196	-21.1
Mean values						0.24			33.8		34.4

Computed peaks were high in 6 cases, average + error = 65.2 percent
Computed peaks were low in 11 cases, average - error = 17.6 percent
Computed runoff volumes were high in 3 cases, average + error = 64.9 percent
Computed runoff volumes were low in 14 cases, average — error = —27.1 percent

Figure 21. Aerial photo of Crane Geek basin

Tripps Run Tributary Basin, Falls Church, Virginia

Tripps Run is primarily a residential basin, but there is a significant amount of commercial development adjacent to U. S. Route 50 which crosses the basin in an east-west direction *(note figure 24).* North of Route 50 the residential area is relatively dense compared with the large lots and open areas to the south. The streets south of Route 50 are asphalt strips laid on existing grade without curb and gutter or roadside ditches, as illustrated in figure 22. Of the 15 reaches used to represent the drainage system, 5 were open channels with a combined length of 2370 feet and 10 were closed conduits with a combined length of 8325 feet. Storm drainage information was difficult to obtain. In several locations missing data had to be filled with what seemed appropriate. Street slopes in the basin vary from 1 to 6 percent and yard slopes vary from 3 to 10 percent. Dominant soils in the general area of the basin are Appling and Louisburg in hydrologic group B and Colfax in hydrologic group C.

The directly connected paved area includes all of the streets, all of the commerical area, and driveways in the residential areas. Supplemental paved area includes about one-half of the residential roofs. Contributing grassed area includes the front yard strip plus large grassed areas in the southern part of the basin.

Figure 22. Typical large lawns in the downstream portion of Tripps Run Tributary basin

Data

The USGS provided the data for this study in the form of original charts from a Stevens graphical stage-recorder located on a rated culvert. The recorder was equipped with a second pen that recorded blips from a tipping bucket raingage on the same chart. The time scale of 0.2 inches per hour was adequate to define the stage hydrographs but not for accurate timing of bucket tips. The 0.1-inch tipping bucket was a further limitation on this data. Since it was recognized that a different interpretation of the rainfall between bucket tips was possible, the most intense storms from the available data were read at 10-minute intervals.

Results

As shown in figure 23 and table 9, the results for this basin are erratic. ILLUDAS does a good job on some storms and very poorly on others. The fact that the runoff ratio varied from 0.09 to 0.89 for the 10 storms studied demonstrates the volatile nature of this basin. Urban basins of this size are sensitive to short duration high intensity rainfall that could not be determined from the available data.

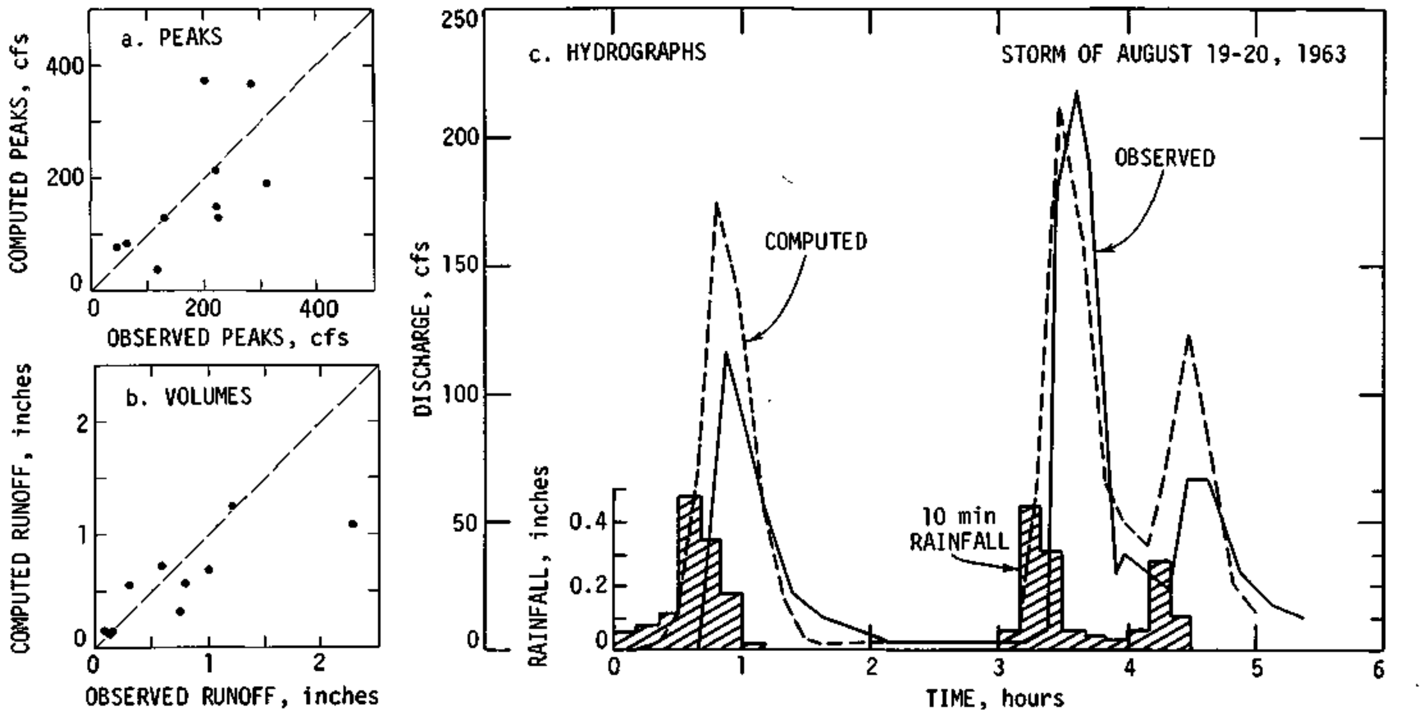


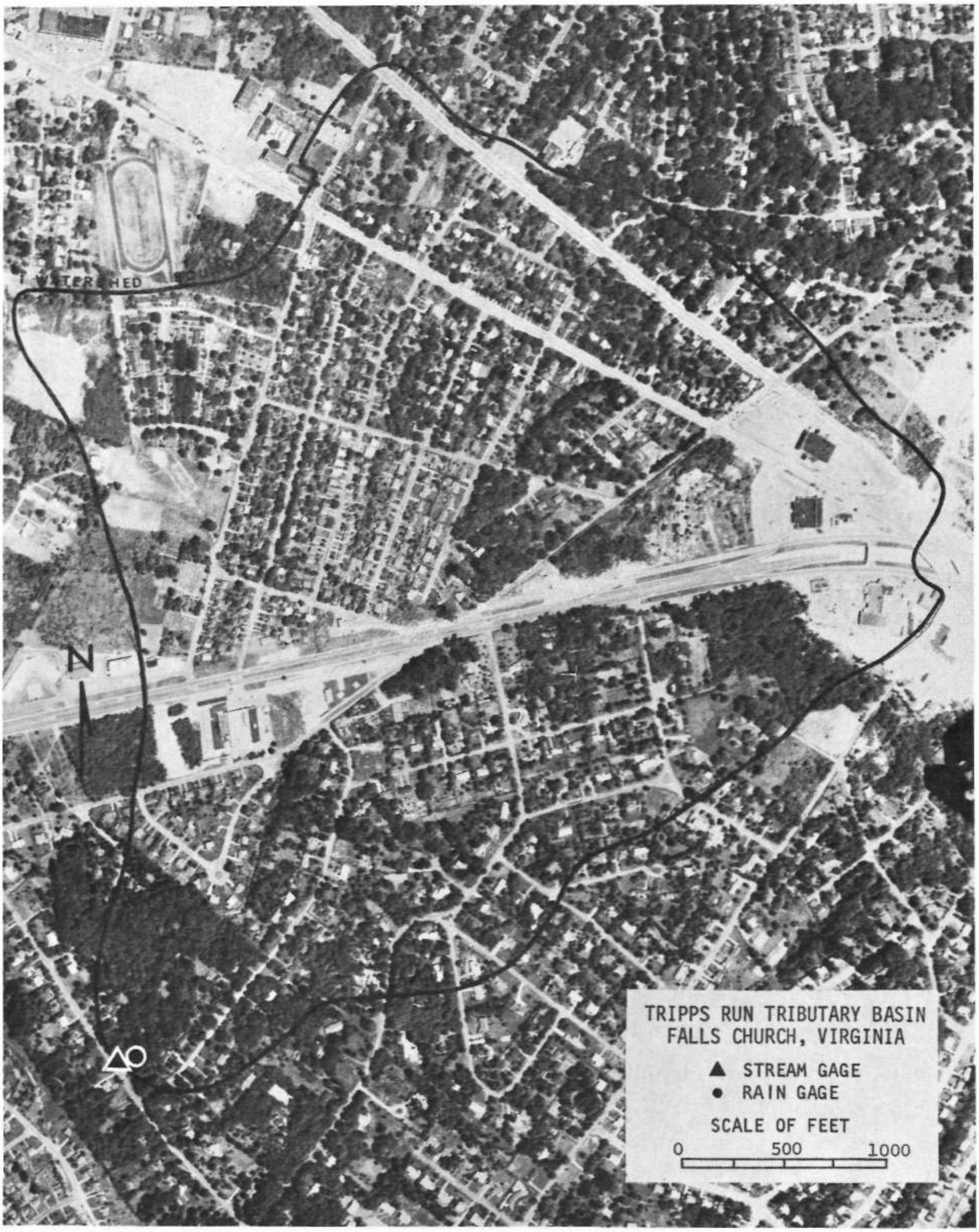
Figure 23. ILLUDAS results for Tripps Run Tributary basin

Table 9. Storm Data and Results for Tripps Run Tributary Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
322 acres	100 acres	56.9 acres	15.2 acres	150 acres
	31 percent	17.7 percent	4.7 percent	46.6 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	31163	3	1.70	118	0.74	0.43	0.32	11	-56.9	38	-67.6
2	62963	1	2.75	225	0.78	0.28	0.57	17	-27.2	130	-42.3
3	81963	2	2.55	219	0.58	0.23	0.72	39	24.6	213	-2.6
4	82063	4	2.45	285	1.23	0.50	1.25	67	1.3	368	29.2
5	60265	3	0.85	47	0.08	0.09	0.15	11	97.3	78	66.6
6	81865	2	0.85	131	0.17	0.20	0.14	6	-18.1	130	-0.7
7	82665	3	1.35	203	0.30	0.22	0.55	59	84.7	374	84.5
8	100765	1	3.10	221	1.00	0.32	0.68	22	-31.9	149	-32.6
9	82467	4	2.55	312	2.27	0.89	1.09	60	-52.1	191	-38.8
10	102567	1	0.90	62	0.17	0.19	0.14	0	-18.1	84	36.0
Mean values						0.34			41.2		40.1

Computed peaks were high in 4 cases, average + error = 54.1 percent
Computed peaks were low in 6 cases, average — error = —30.8 percent
Computed runoff volumes were high in 4 cases, average + error = 52.0 percent
Computed runoff volumes were low in 6 cases, average — error = —34.0 percent



(USDA photo of May 25, 1962)

Figure 24. Aerial photo of Tripps Run Tributary basin

Tar Branch Basin, Winston-Salem, North Carolina

A large part of downtown Winston-Salem and major industrial areas lie within the boundaries of the Tar Branch basin and account for the high percentage of paved area. The remainder of the basin is light commercial or residential (figure 25). In order to represent the extensive storm drainage system in reasonable detail, 103 sub-basins were used. Of the 103 reaches, 15 were open channels with a combined length of 7200 feet. Pipes in the system ranged from 10 inches up to 72 inches in diameter. Information on the drainage system was not complete, and storm drain slopes were assumed to be the same as street slopes in many cases. Street slopes are highly variable. In the downtown area they are gentle, but they range up to 10 percent in other parts of the basin. Yard slopes are also variable ranging from 3 to 10 percent. The dominant soil in the basin is a Pacolet fine sandy loam which in the undisturbed state is in hydrologic group B.

The directly connected paved area consists of all of the downtown commercial area, all other streets, and other major buildings and parking lots. Residential roofs are not generally connected to the drainage system and private driveways are usually not paved.

Figure 25. Street view in Tar Branch basin

Data

The instrumentation on this basin was the USGS standard installation for urban basins. Two digital recorders punch the rainfall and stage synchronously at 5-minute intervals. In this case the instruments are located on an open channel above a rated culvert *(note location in figure 27).* For this study the rainfall and the discharge were both provided in tabular form at 5-minute intervals.

Results

Considering the variability in the runoff ratios shown in table 10 for this basin (0.17 to 0.88), ILLUDAS does a good fitting job. The authors feel that the scatter apparent in figure 26 a and b and the variable runoff ratios are caused to a large degree by variability of storm rainfall across the basin. This would also account for the several low runoff ratios (0.17 to 0.25) that occurred on the 17 storms analyzed. Grassed area contributes on only 4 of the 17 storms analyzed and then not significantly.

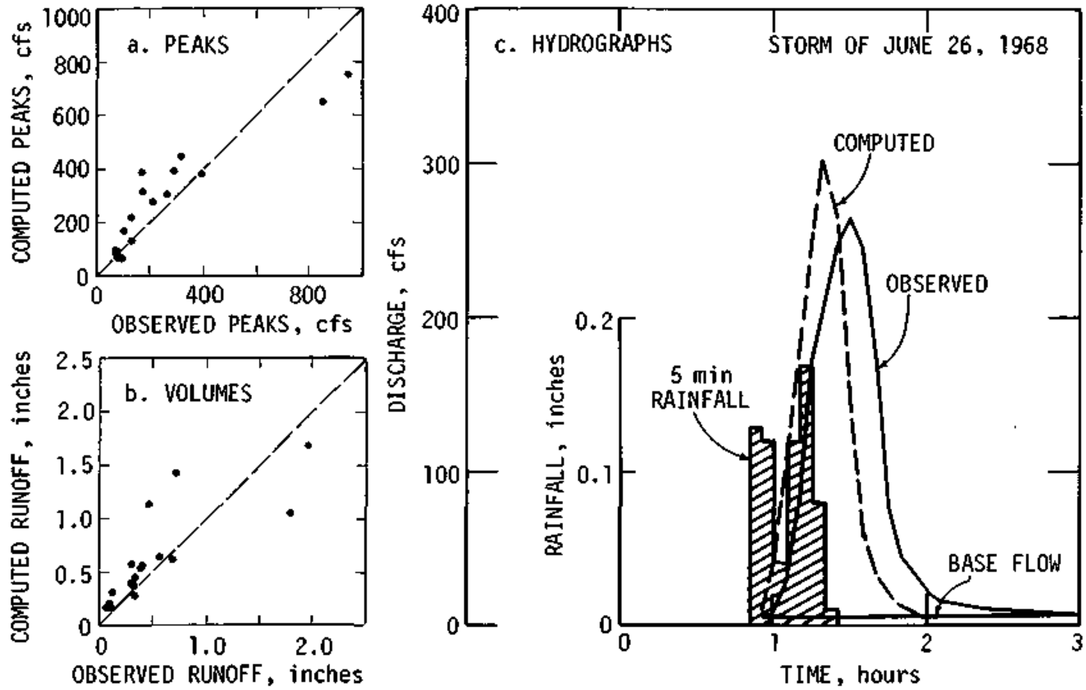


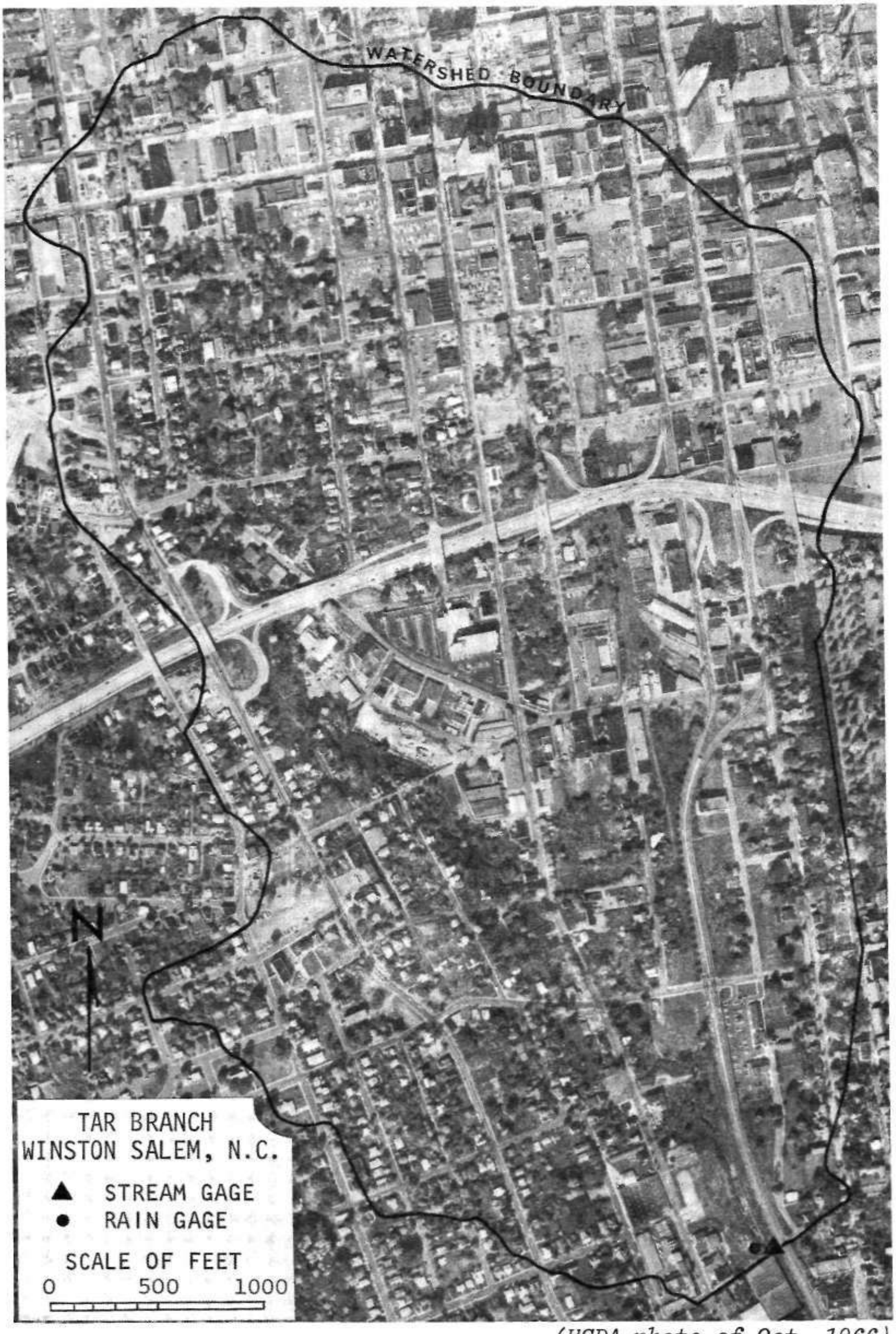
Figure 26. ILLUDAS results for Tar Branch basin

Table 10. Storm Data and Results for Tar Branch Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
384 acres	227 acres	195 acres	10.2 acres	36.2 acres
	59 percent	51 percent	2.6 percent	9.5 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	62668	2	0.67	265	0.34	0.51	0.29	0	-14.9	301	13.7
2	71268	2	1.42	397	0.69	0.48	0.62	0	-10.3	380	-4.2
3	101868	4	2.88	175	0.71	0.25	1.43	1	100.4	315	80.3
4	61169	4	3.03	945	1.97	0.65	1.68	12	-14.5	748	-20.8
5	61569	4	2.22	171	0.46	0.21	1.14	4	145.5	385	125.5
6	61869	4	0.38	210	0.12	0.31	0.16	6	37.9	274	30.6
7	62169	2	0.82	316	0.33	0.40	0.37	0	14.6	445	40.8
8	72869	2	1.38	290	0.53	0.39	0.65	0	21.4	393	35.6
9	61370	3	2.05	857	1.80	0.88	1.05	6	-41.6	648	-24.3
10	121867	1	1.00	97	0.35	0.35	0.45	0	28.0	62	-36.1
11	31668	4	1.14	132	0.38	0.33	0.53	0	39.0	123	-6.1
12	42368	2	0.52	73	0.10	0.20	0.21	0	110.4	79	7.7
13	41569	2	0.88	71	0.30	0.34	0.39	0	32.2	91	28.7
14	52769	3	1.22	105	0.41	0.33	0.56	0	38.7	165	56.9
15	60969	2	0.26	134	0.09	0.34	0.08	0	-6.3	118	-12.3
16	80369	3	0.43	80	0.07	0.17	0.17	0	123.3	96	19.6
17	121069	4	1.23	82	0.31	0.25	0.57	0	83.4	66	-20.1
Mean values						0.38			50.7		33.1

Computed peaks were high in 10 cases, average + error = 43.9 percent
Computed peaks were low in 7 cases, average — error = 17.7 percent
Computed runoff volumes were high in 12 cases, average + error = 64.6 percent
Computed runoff volumes were low in 5 cases, average — error = 17.5 percent



(USDA photo of Oct. 1966)

Figure 27. Aerial photo of Tar Branch basin

Third Fork Basin, Durham, North Carolina

The Third Fork basin contains a variety of land uses. There is a high-density commercial area and a significant industrial area along the northern watershed boundary. The residential area, which makes up most of the basin, is itself highly variable ranging from simple frame homes on dirt streets to homes on large lots. Surrounding the channel in the southern part of the basin are over 100 acres of open park. Soils in the floodplain are primarily Cangaree loams. Although these are classified in hydrologic group B, the high water table in this area could add significantly to the runoff potential. Upland soils consist of White Store soils and are classified in hydrologic group D. With the exception of a few pipes in the upper reaches of the basin, all drainage is by open channel (figure 28). Of the 39 reaches used to describe the storm drainage system, only 8 are closed conduits. Street and channel slopes are moderate, ranging from less than 1 to about 5 percent. Yard slopes range from 5 to 10 percent.

The total paved area of the basin was determined by zoning out the 100-percent paved areas and the park area, and measuring sample blocks in the remaining area. The residential area was divided into 3 zones; low income, middle income, and high income. It was assumed that 0, 10, and 12.5 percent, respectively, of these roof areas were connected to the storm drainage system. One-half of the remaining roof area made up the supplemental paved areas. In the areas where paved streets did not exist a 15-foot strip was assumed to be connected to the system. The directly connected paved area thus consisted of 147 acres of commercial area, 126 acres of streets, and 20 acres of residential roofs and driveways. Contributing grassed area was limited to a 30 to 50 foot front yard strip and large park areas adjacent to the main channels.

Data

This is a standard USGS installation. Two digital recorders operating from the same clock punch the stage hydrograph and the rainfall at 5-minute intervals. The stage hydrograph is recorded at a rated culvert section in an open channel *(note location in figure 30).* For this study both rainfall and discharge were provided in tabular form at 5-minute intervals.

Figure 28. Open channel which is representative of most of the drainage in Third Fork basin

Results

ILLUDAS provided several excellent fits such as the one in figure 29c. The general tendency to underpredict the peaks and runoff volumes indicated in figure 29 a and b could have been compensated for by an increase in the directly connected paved area or in the contributing grassed area. In many residential areas back yards were able to drain directly to open drainage channels and could have been added to the contributing grassed area. The grassed area contributed to runoff in 8 of the 15 storms analyzed, as shown in table 11. In 4 of these 8 storms the grassed area contribution accounted for at least 25 percent of the total runoff.

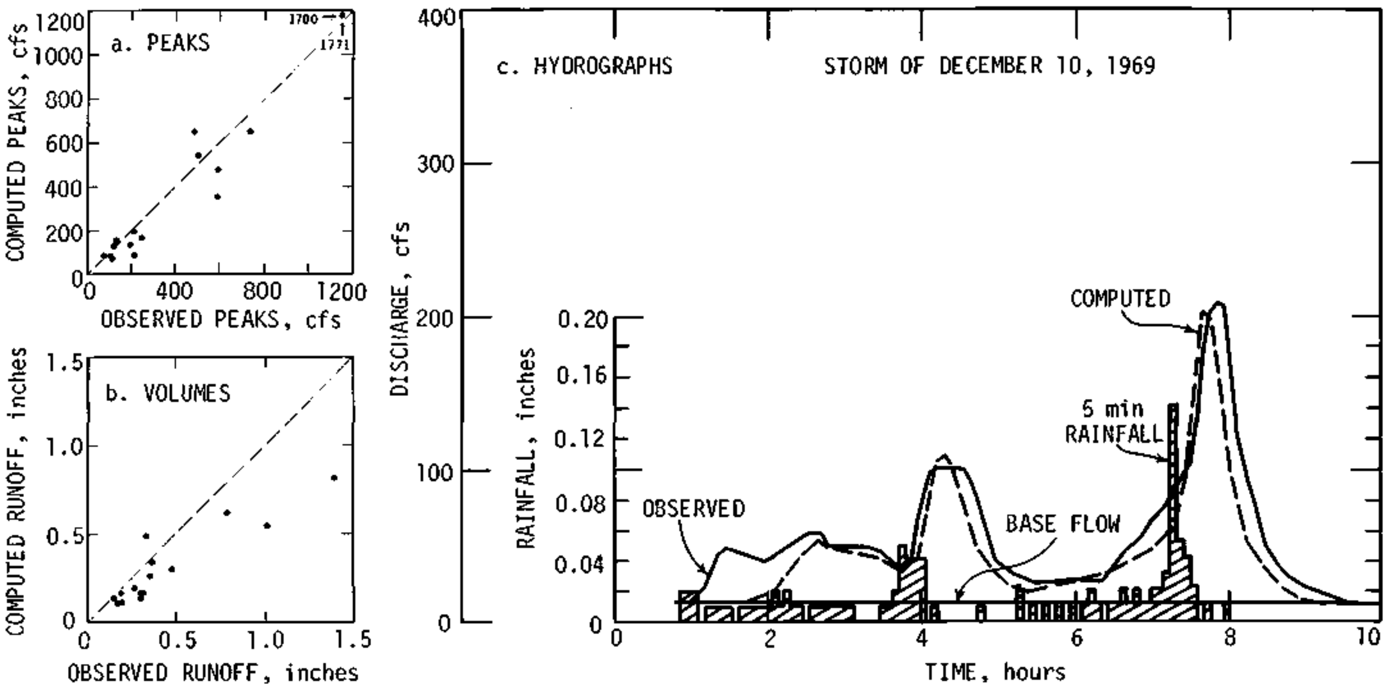


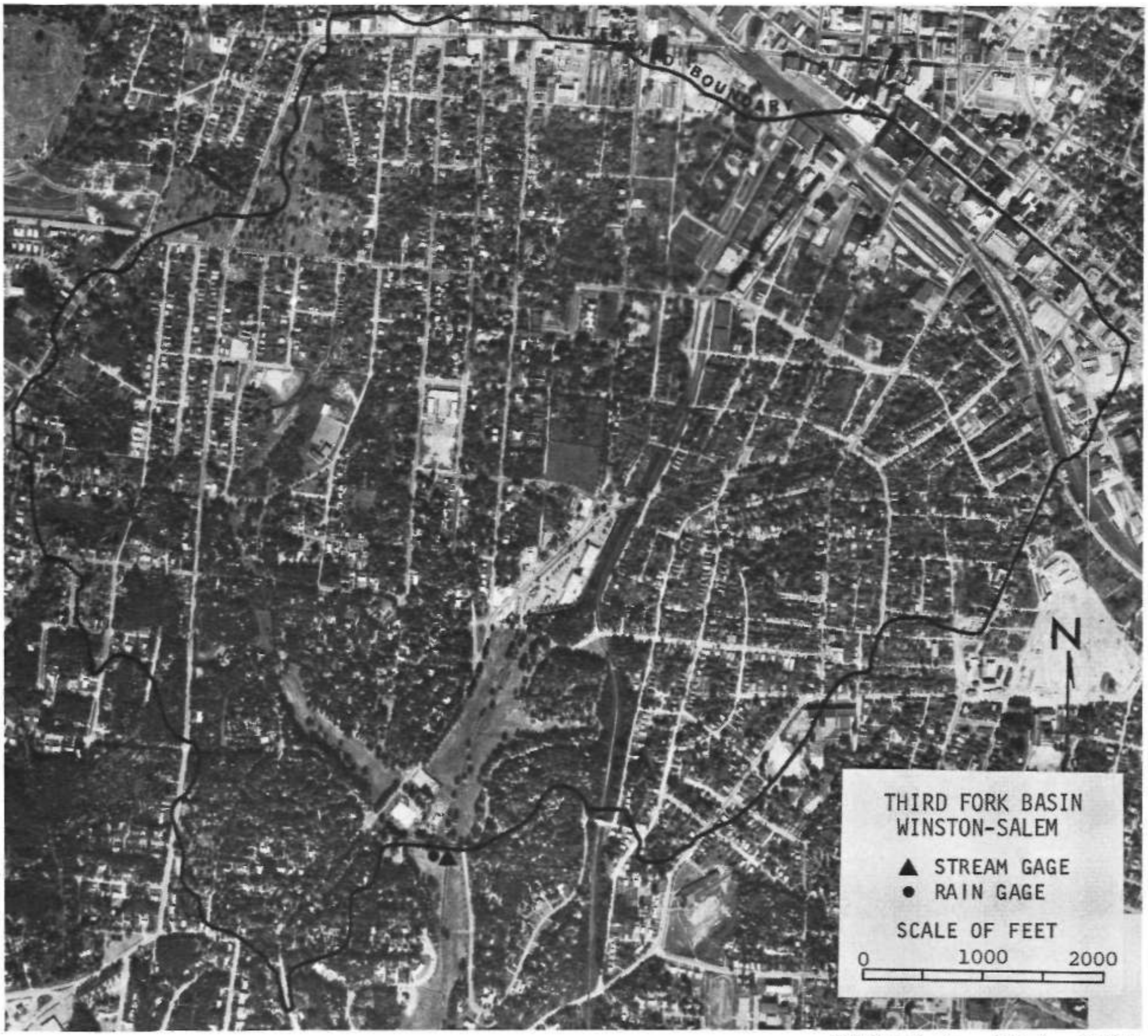
Figure 29. ILLUDAS results for Third Fork basin

Table 11. Storm Data and Results for Third Fork Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
1075 acres	397 acres	293 acres	56 acres	233 acres
	37 percent	27 percent	5.2 percent	21.6 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	60969	2	0.64	77	0.15	0.23	0.14	0	-1.7	86	12.0
2	61569	4	1.80	500	0.79	0.44	0.62	25	-21.8	540	8.0
3	72869	3	0.97	485	0.33	0.35	0.49	36	47.6	646	33.3
4	80169	3	0.72	137	0.19	0.26	0.17	1	-11.1	154	12.1
5	80369	4	0.83	593	0.48	0.58	0.30	35	-37.5	470	-20.7
6	80469	4	0.50	199	0.19	0.38	0.11	3	-41.0	139	-30.1
7	81369	3	0.53	120	0.17	0.31	0.11	0	-31.0	129	7.2
8	81569	3	1.96	1700	1.39	0.71	0.82	39	-40.9	1771	4.2
9	90269	1	0.73	593	0.32	0.43	0.17	0	-46.4	346	-41.7
10	91769	1	1.36	732	0.37	0.27	0.34	0	-7.8	654	-10.6
11	92469	4	0.60	217	0.31	0.52	0.13	1	-56.8	83	-61.7
12	121069	3	1.05	205	0.36	0.34	0.26	3	-26.9	199	-2.7
13	122169	1	0.83	105	0.27	0.32	0.19	0	-27.2	87	-17.3
14	122569	3	0.73	116	0.32	0.44	0.17	0	-48.0	75	-35.1
15	21670	1	2.11	245	1.01	0.48	0.54	0	-46.7	162	-33.8
Mean values						0.40			32.8		22.0

Computed peaks were high in 6 cases, average + error = 12.8 percent
Computed peaks were low in 9 cases, average — error = —28.2 percent
Computed runoff volumes were high in 1 case, average + error = 47.6 percent
Computed runoff volumes were low in 14 cases, average — error = —31.8 percent



(USDA photo of Oct. 1966)

Figure 30. Aerial photo of Third Fork basin

This is primarily a residential basin with a few strips of commercial area. There is no underground storm drainage in the basin. Runoff is transported via streets to either the East or West Branches of Dry Creek. The East Branch has had some improvement but is essentially a natural stream. The West Branch flows for much of its length through specially modified street cross sections which are in effect a concrete canal. This is illustrated in figure 31. As a result, flow down the West Branch is much faster than flow down the East Branch. Street slopes are quite flat, averaging less than 0.5 percent. Yard slopes vary from 2 to 8 percent. Dominant soils in the area are Dale silt loam and Farnum loam, both classified in hydrologic group C.

Twenty-three sample blocks were used to determine the paved area of the basin. The directly connected paved area includes all of the streets, major buildings and parking lots, and 25 percent of the remaining paved area. Supplemental paved area consists of one-half of the residential roof area. Contributing grassed area includes the 50 to 200 foot strip along the stream channels and the front yards in residential areas.

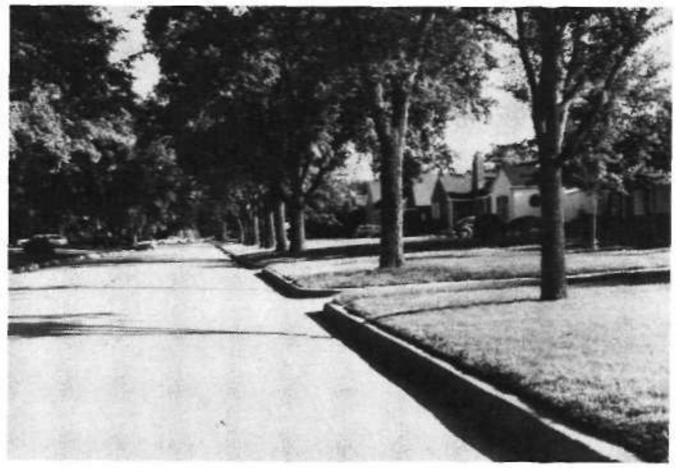


Figure 31. View of street typical of those which carry most surface runoff in Dry Creek basin

Data

Both rainfall and stage data on this basin are collected by digital punch-type recorders located on a rated bridge on Dry Creek *(note location in figure 33). A* graphical stage recorder originally installed was found impractical because of the rapid changes in stage. The data provided for this project were for 1964 through 1969, but only the 1964-1965 data were used because, after a dry period during 1966-1967, there appeared to be a shift in the rating curve. This shift has been fairly well documented by USGS personnel, but there is still some question about the 1968-1969 data.

Results

The wide scatter in figure 32 a and b makes any generalization of the results for this basin difficult. ILLUDAS shows heavy grassed-area contribution for storms 2, 3, 4, and 7 in table 12. In all of these storms, however, ILLUDAS overpredicts the runoff volume. For the other storms in which little or no grassed-area runoff is indicated ILLUDAS underpredicts. There appears to be an inverse relationship between the antecedent moisture condition and the runoff ratio rather than the expected direct relationship. Data on more storms would be needed to form any firm conclusions about this basin.

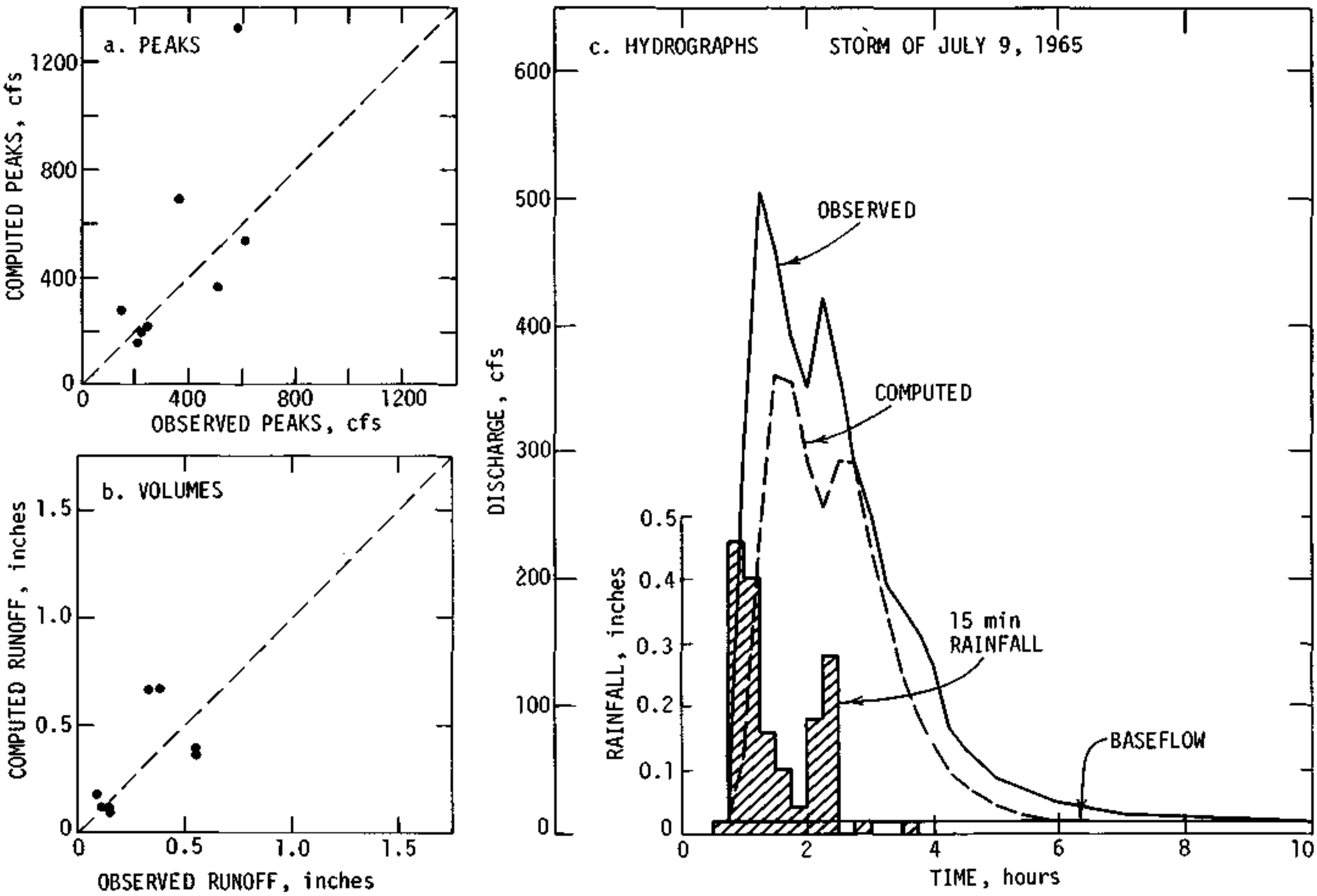


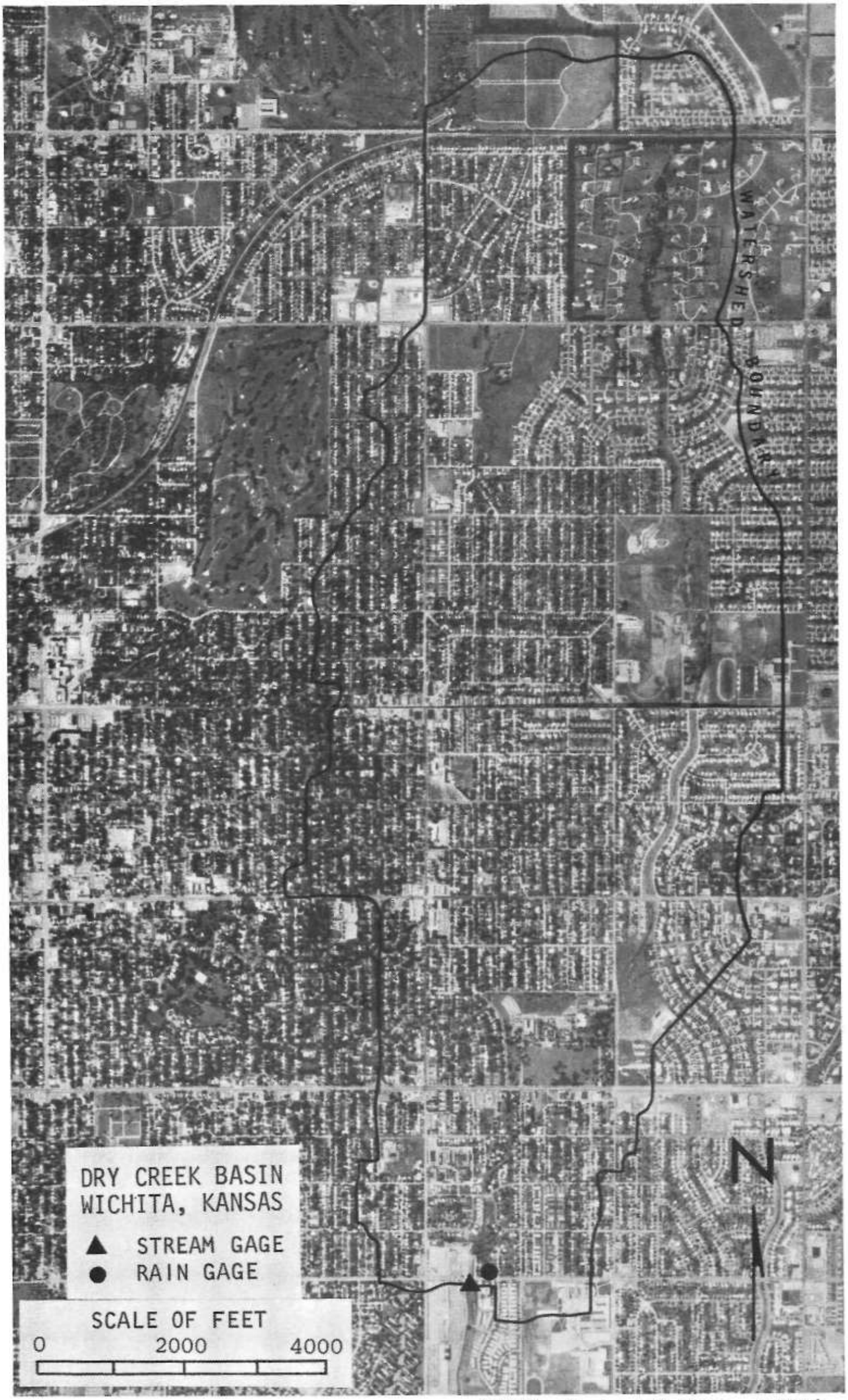
Figure 32. ILLUDAS results for Dry Creek basin

Table 12. Storm Data and Results for Dry Creek Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
1882 acres	583 acres	365 acres	147 acres	673 acres
	31 percent	19 percent	7.8 percent	35.7 percent

		Observed Storm Data					Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	50564	2	0.72	250	0.15	0.21	0.12	0	-21.0	219	-12.6
2	80964	4	2.20	365	0.34	0.15	0.66	45	95.2	689	88.9
3	82764	4	1.88	580	0.39	0.21	0.67	58	71.3	1321	127.9
4	91964	4	0.56	226	0.11	0.19	0.12	27	12.0	197	-12.9
5	22865	2	0.60	212	0.15	0.25	0.09	0	-39.1	155	-26.8
6	51365	2	1.98	608	0.55	0.28	0.39	8	-28.9	537	-11.6
7	52465	4	0.74	148	0.08	0.11	0.18	30	111.3	272	83.6
8	70965	3	1.68	505	0.56	0.33	0.37	17	-33.9	361	-28.6
Mean values						0.22			51.6		49.1

Computed peaks were high in 3 cases, average + error = 100.1 percent
Computed peaks were low in 5 cases, average — error =18.5 percent
Computed runoff volumes were high in 4 cases, average + error = 72.4 percent
Computed runoff volumes were low in 4 cases, average - error =30.7 percent



(USDA photo of June, 1970)

Figure 33. Aerial photo of Dry Creek basin

Wingohocking Basin, Philadelphia, Pennsylvania

Wingohocking is the largest and the most highly urbanized basin in this report. There are a few areas of separate single-family residences in the basin, but row-houses are by far the most common (figure 34). Extensive commercial and industrial areas also exist in the basin. There are no open channels. An extensive combined sewer system with arch-shaped pipes up to 21 by 24 feet provides storm drainage. A sanitary interceptor sewer picks up dry-weather flow just above the gage. The basin is represented in the model by 128 separate sub-basins ranging in size from 1.2 to 117 acres. Street slopes generally range from 0.5 to 2 percent and yard slopes from 3 to 10 percent. Soils in the area are either in the Chester Complex group which is classified 85 percent hydrologic group B and 15 percent C-D, or Howell Complex which is classified 75 percent hydrologic group B and 25 percent C-D.

Paved areas were based on studies previously made by the city of Philadelphia and confirmed during this study by measuring sample blocks on aerial photographs. All of the paved areas in the basin, including residential rooftops, are directly connected to the drainage system. Large park areas and undeveloped areas in the basin were not well drained. Only the front yards in single family areas were included in the contributing grassed area.

Data

The flow measurement program, as described by Tucker (1969), was established by the U. S. Public Health Service in 1963. A graphical stage recorder was installed 450 feet upstream from a low broad-crested weir. The weir, which is 87 feet upstream from the outfall, was rated by a physical model built and tested at Swarthmore College in 1964. The Research and Development unit of the Philadelphia Water Department took over the gage in July 1967 and again built a model of the weir at the city's Northeast Water Pollution Control Plant. For use in this project discharge hydrographs were provided in digital form.

Figure 34. Typical row-houses in the Wingohocking basin

The city also operates a network of recording raingages. Four of these gages were used for the Wingohocking basin. These are shown on the map in figure 36 as 1, Roosevelt; 2, Heintz; 3, Queen Lane; and 4, Harrowgate. All of the raingages were of the weighing-bucket type. As a part of this project the original charts were digitized for 5-minute intervals with the Water Survey's autotrol model 3400 X-Y digitizer.

Results

Table 13 and figure 35 a and b clearly show that ILLUDAS overpredicts both peak flows and runoff volumes on this basin. The abrupt change in the nature of the measured runoff between August 1967 and June 1968 may or may not be significant, but it is interesting to note that prior to 1968 the average runoff ratio for the 13 storms analyzed is 0.32 and for the 3 storms available since 1968, the average runoff ratio is 0.72. The ILLUDAS predictions on these last three storms is acceptable.

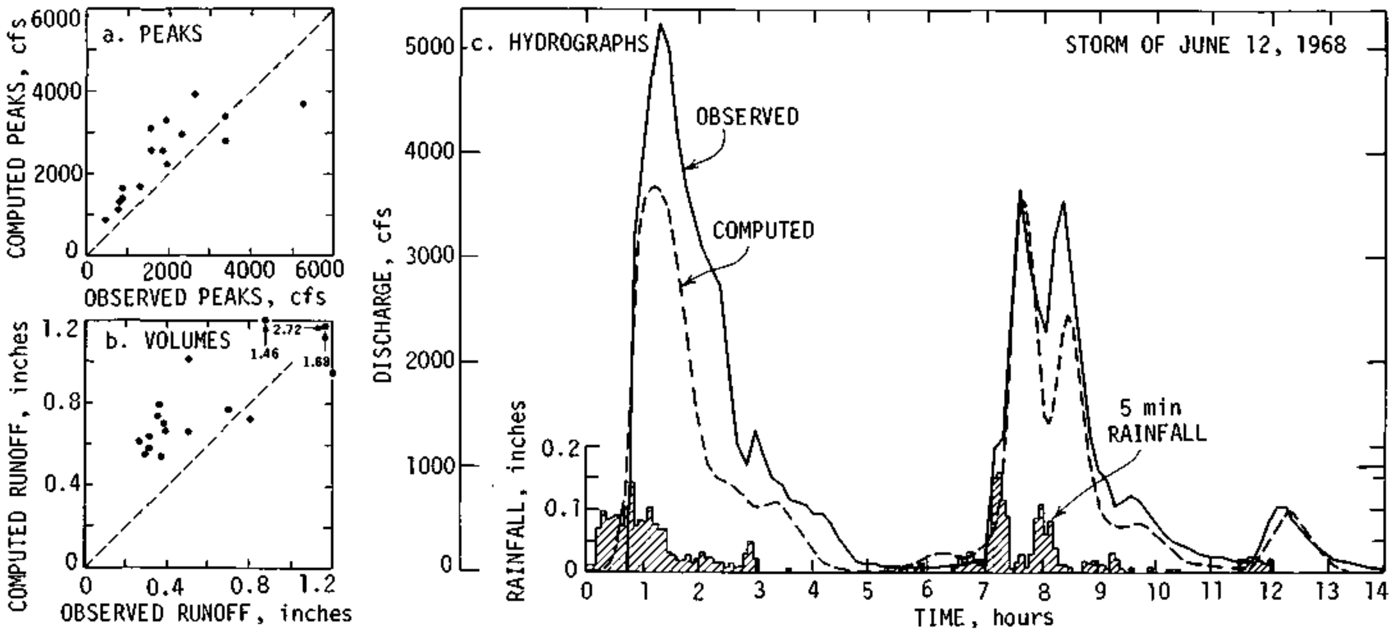


Figure 35. ILLUDAS results for Wingohocking basin

Table 13. Storm Data and Results for Wingohocking Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
5326 acres	3246 acres	3246 acres	0 acres	368 acres
	61 percent	61 percent		7 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	42764	2	1.02	470	0.27	0.26	0.55	0	107.2	849	80.6
2	42964	3	1.00	860	0.37	0.37	0.54	0	45.9	1400	62.8
3	92864	4	1.26	1145	0.38	0.31	0.70	0	81.5	1690	47.6
4	112564	2	1.41	1960	0.36	0.26	0.79	0	116.5	2225	13.6
5	71165	2	2.52	1860	0.89	0.35	1.46	0	64.2	2583	38.9
6	80465	2	1.02	789	0.31	0.30	0.58	0	86.6	1124	42.5
7	80965	4	1.97	1960	0.50	0.25	1.13	0	124.6	3301	68.4
8	82165	3	1.16	800	0.31	0.27	0.64	0	106.9	1308	63.5
9	92465	1	1.22	1570	0.39	0.32	0.67	0	72.3	3101	97.5
10	100765	1	1.11	880	0.26	0.23	0.61	0	137.1	1667	89.5
11	70267	3	1.38	2325	0.70	0.51	0.77	0	10.6	2967	27.6
12	72967	3	1.20	1587	0.50	0.41	0.66	0	33.7	2692	69.6
13	80967	3	1.34	2640	0.35	0.26	0.74	0	112.7	3928	48.8
14	61268	4	3.23	5248	2.72	0.84	1.88	0	-30.8	3680	-29.9
15	72468	3	1.68	3402	1.20	0.71	0.95	0	-20.5	2806	-17.5
16	80168	2	1.31	3402	0.80	0.61	0.72	0	-9.2	3417	0.5
Mean values						0.39			72.5		50.0

Computed peaks were high in 14 cases, average + error = 53.8 percent
Computed peaks were low in 2 cases, average — error = —23.5 percent
Computed runoff volumes were high in 1 3 cases, average + error = 84.7 percent
Computed runoff volumes were low in 3 cases, average — error = —19.8 percent

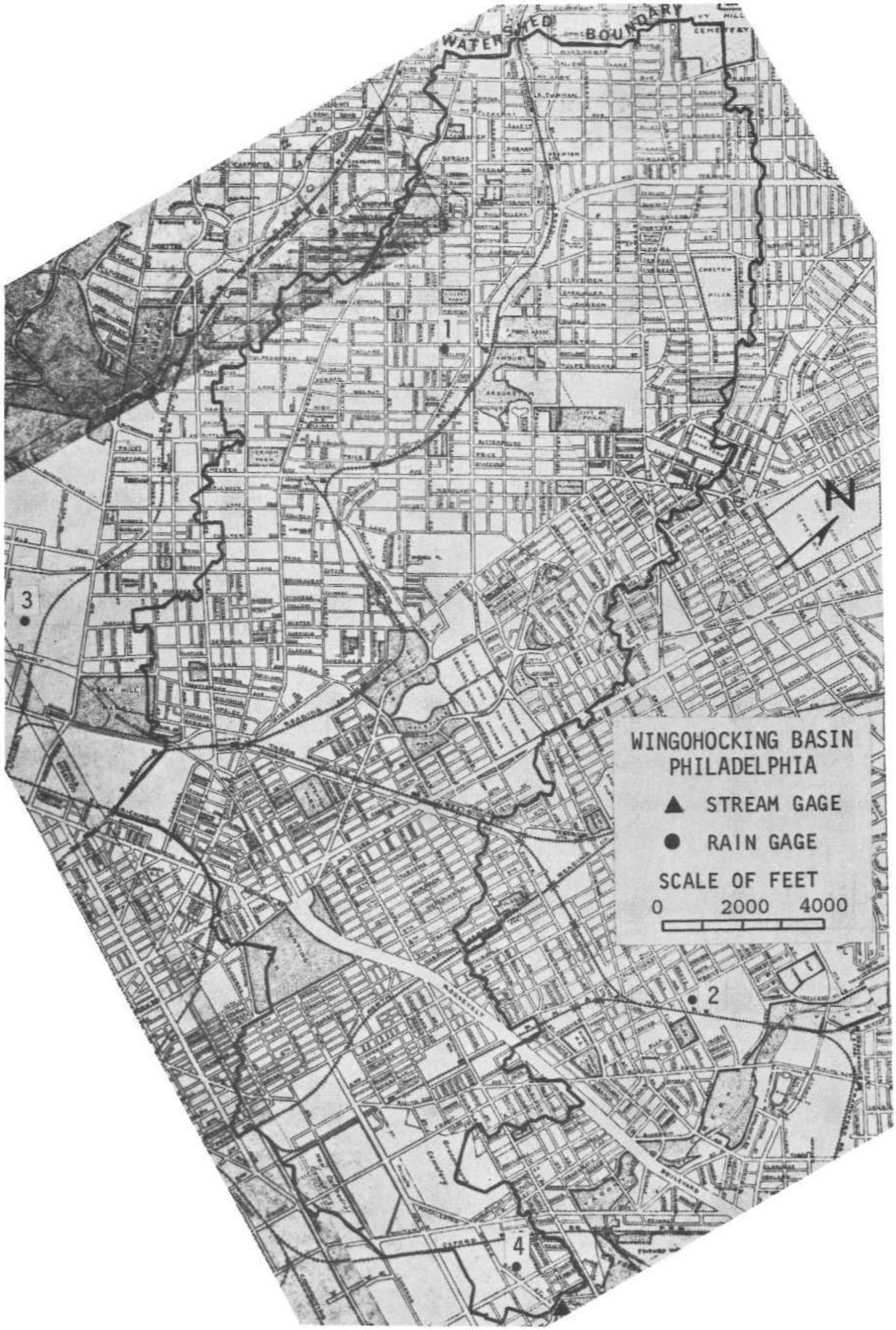


Figure 36. Street map of the Wingohocking basin

First Street Basin, Louisville, Kentucky

Louisville Gaging Program

Data for the following three basins in Louisville, Kentucky, were collected by the U. S. Army Corps of Engineers, Louisville District, in compliance with a June 1944 request from the Ohio River Division of the Corps. The purpose of the investigation was to determine runoff characteristics of several areas in order to design nine pumping stations for the removal of storm runoff from the low-lying areas of Louisville. To provide information on sewer runoff rates 12 Friez FW-1 water stage recorders were installed in 6 Louisville sewers. The charts operated at 14.4 inches per day and were changed weekly. Discharge rating curves for the gages were developed from measurements of sewer discharge made by the salt-velocity method. In addition to the sewer gaging stations, the Corps operated a network of 10 Friez recording raingages of the Universal or Ferguson types. Three of the six basins gaged by the Corps were selected for analysis in this study.

The First Street basin is a highly developed commercial and industrial area in the old northern sector of Louisville (figures 37 and 39). At the time the data were collected the basin was classified as 57 percent commercial, 23 percent industrial, and 20 percent residential with a population of 1000. All of the storm drainage is underground and the trunk line is oversize compared with other storm drains in Louisville. Soils in the urban area of Louisville are not well defined, but are generally in the Wheeling-Weinbach-Sciotoville association. These soils are hydrologic soil groups B, C, and C, respectively. Group B was used in the First Street basin analysis. Surface slopes in the basin are flat. The directly connected paved area consists of the streets, alleys, and driveways, and 80 percent of the roofs. The supplemental paved area includes the remainder of the roofs. All grassed area was considered contributing because of the small size of the yards and other grassed areas.

Figure 37. Alley in First Street basin

Results

Very little can be concluded from two storms, but ILLUDAS does do a good job on these. In both cases about 12 percent of the runoff volume comes from the grassed area (table 14). Figure 38c shows that the timing of the runoff is acceptable.

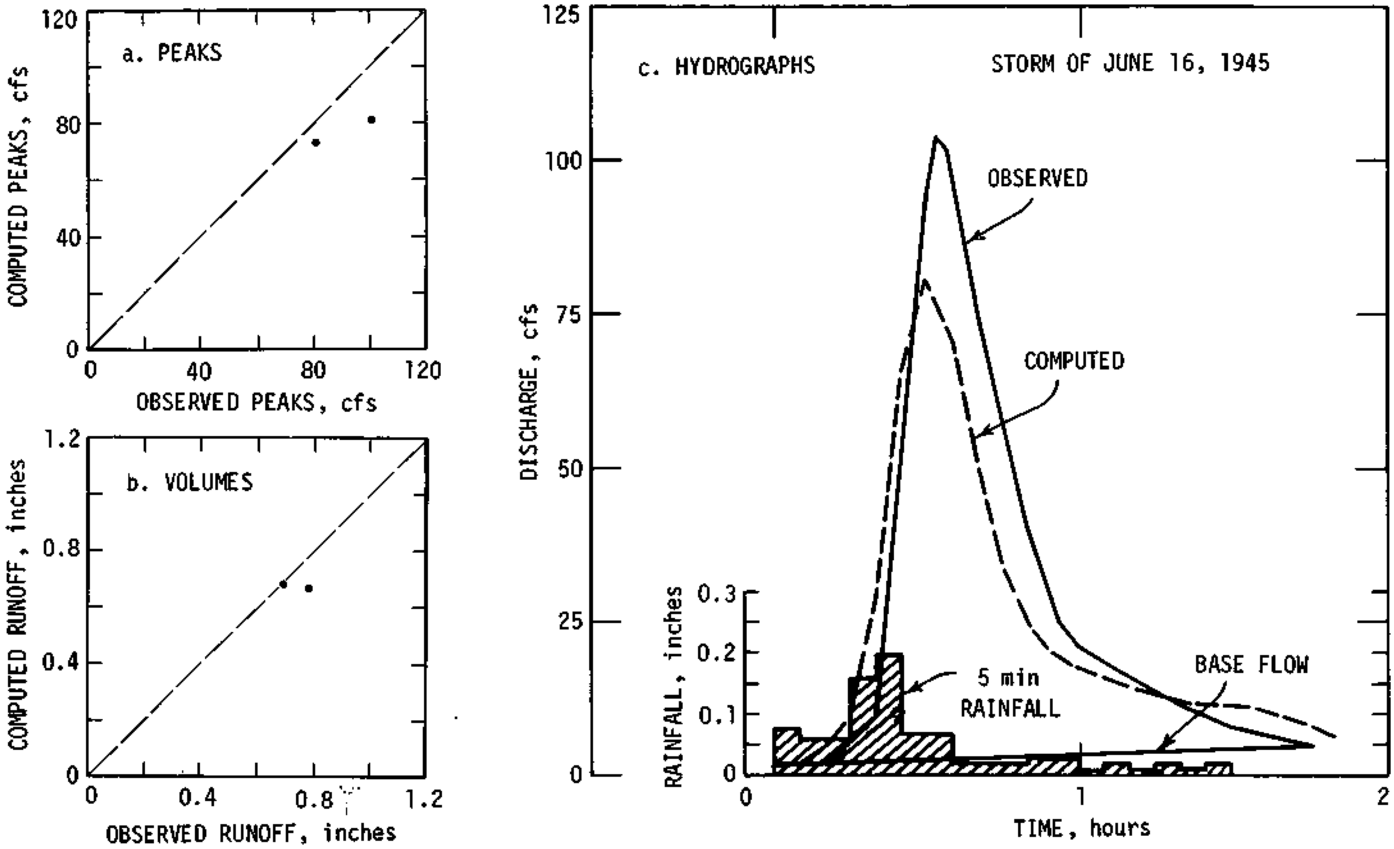


Figure 38. ILLUDAS results for First Street basin

Table 14. Storm Data and Results for First Street Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
61.2 acres	49.3 acres	39.6 acres	9.7 acres	11.9 acres
	80.5 percent	64.7 percent	15.8 percent	19.4 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	60845	3	1.02	81	0.78	0.77	0.67	12	-14.2	73	-9.7
2	61645	3	1.04	104	0.69	0.66	0.68	12	-0.9	81	-22.1
Mean values						0.71			7.5		15.9

Computed peaks were high in 0 cases
Computed peaks were low in 2 cases, average — error = 15.9 percent
Computed runoff volumes were high in 0 cases
Computed runoff volumes were low in 2 cases, average — error = 7.5 percent

(USDA photo, 1951)

Figure 39. Aerial photo of First Street basin

Seventeenth Street Basin, Louisville, Kentucky

The Seventeenth Street basin is in the same general area of Louisville as the First Street basin but it is more residential. At the time the data were collected the basin was classified as 15 percent commercial, 28 percent industrial, and 57 percent residential, with a population of approximately 3000. Most of the residential area is old and consists of small one-family homes on narrow lots with small lawn areas (figure 40). An aerial view is shown in figure 42. The surface slopes are flat and the soil is considered to be in hydrologic group B from the Wheeling-Weinbach-Sciotoville association. The directly connected paved area includes the streets, alleys, drives, commerical roofs, and about 70 percent of the residential roofs. The remainder of the paved area was included in the supplemental paved area and one-half of the grassed area was considered contributing.

Figure 40. Street scene typical of Seventeenth Street basin

Results

ILLUDAS does a good job of predicting runoff hydrographs on this basin. Table 15 indicates that there is considerable grassed-area runoff in several of the storms. The general shape of the hydrograph and timing of the peak are acceptable as indicated in figure 41c.

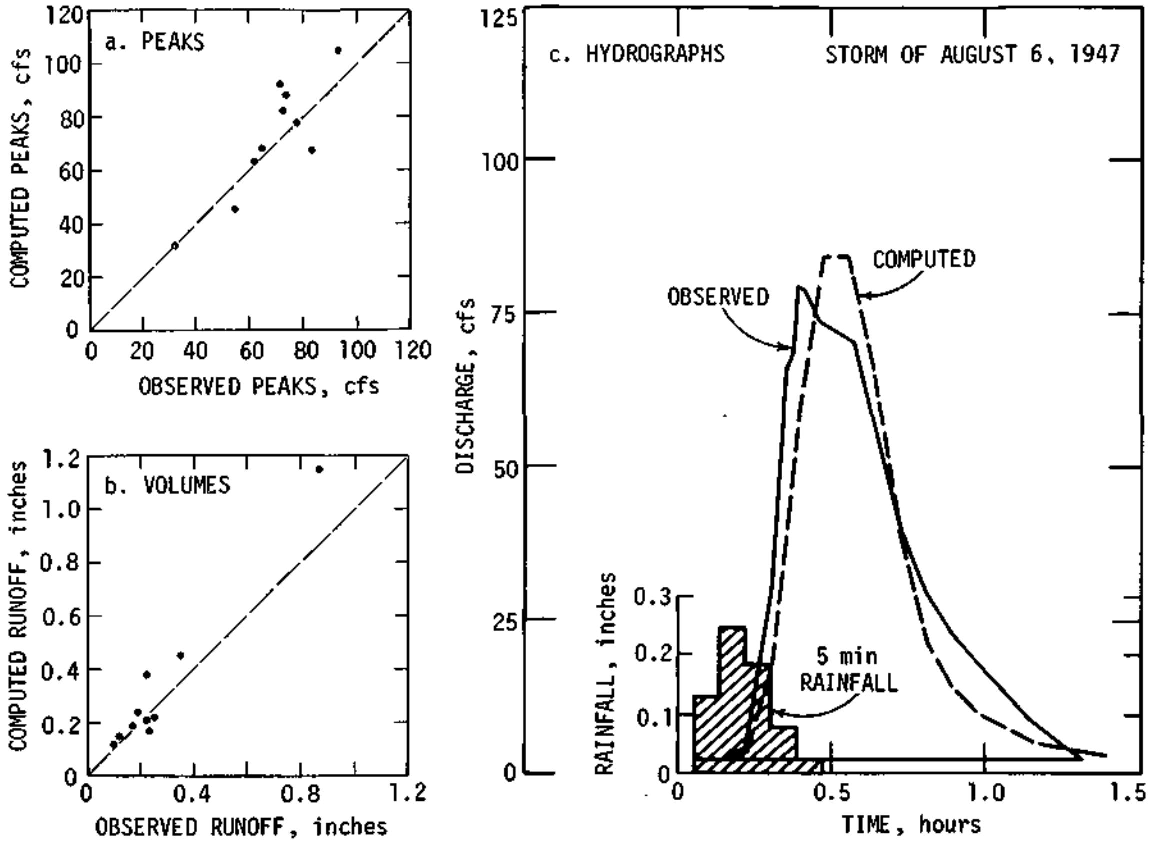


Figure 41. ILLUDAS results for Seventeenth Street basin

Table 15. Storm Data and Results for Seventeenth Street Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
141.1 acres	118.2 acres	58.4 acres	20.2 acres	19.8 acres
	83.7 percent	41.4 percent	14.3 percent	**14.0** percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	50245	2	0.47	54.6	0.12	0.25	0.15	0	28.2	45.3	-17.0
2	52945	2	0.39	32.0	0.10	0.26	0.12	0	17.5	32.1	0.5
3	61045	4	0.52	62.0	0.22	0.41	0.21	20	-0.8	63.0	1.7
4	91345	2	1.15	74.0	0.35	0.30	0.45	3	28.0	87.5	18.3
5	61346	1	0.51	83.5	0.23	0.44	0.17	2	-24.1	66.9	-19.8
6	61946	2	0.55	65.5	0.17	0.31	0.19	0	7.8	67.6	3.3
7	61946	3	0.91	72.8	0.22	0.24	0.38	13	72.6	92.5	27.1
8	52447	4	0.50	73.0	0.19	0.38	0.24	32	27.3	82.2	12.6
9	53047	4	2.25	93.0	0.87	0.38	1.30	32	50.5	105.7	13.7
10	80647	2	0.61	78.0	0.25	0.40	0.22	3	-12.3	77.9	-0.1
Mean values						0.34			26.9		11.4

Computed peaks were high in 7 cases, average + error = 11.0 percent
Computed peaks were low in 3 cases, average — error = —12.3 percent
Computed runoff volumes were high in 7 cases, average + error = 33.1 percent
Computed runoff volumes were low in 3 cases, average — error = 12.4 percent

(USDA photo, 1951)

Figure 42. Aerial photo of Seventeenth Street basin

Northwestern Basin, Louisville, Kentucky

This is the largest Louisville basin selected for analysis in this study. It is 90 percent residential, 5 percent commercial, 4 percent industrial, and 1 percent parks. The commercial area is composed of a number of small shopping centers. Industrial development is centered around the railroad yards visible in the aerial view in figure 45. The residential area varies from small one-family homes on very narrow lots (figure 43) to more modern developments with larger lots near Shawnee Park. The population within the area is approximately 25,000. A hydrologic soil group of C was assigned to the basin to account for the dominant Weinbach-Sciotoville soils. Surface slopes in the basin are relatively flat. The directly connected paved area is composed of the streets, 20 percent of the alleys, 90 percent of the commercial area, and 20 percent of the residential roofs. The supplemental paved area consists of one-half of the unconnected residential roofs. The contributing grassed area includes the residential lawns but not the park areas.

Figure 43. Street scene typical of Northwestern basin

Results

Figures 44 a and b show that the predicted runoff amounts are consistently somewhat low and that the computed peaks are scattered. The contributing paved area (table 16) could be raised slightly to improve these results. Figure 44c is typical of the shape of the predicted hydrographs. In general ILLUDAS does an acceptable job of predicting runoff hydrographs for this basin.

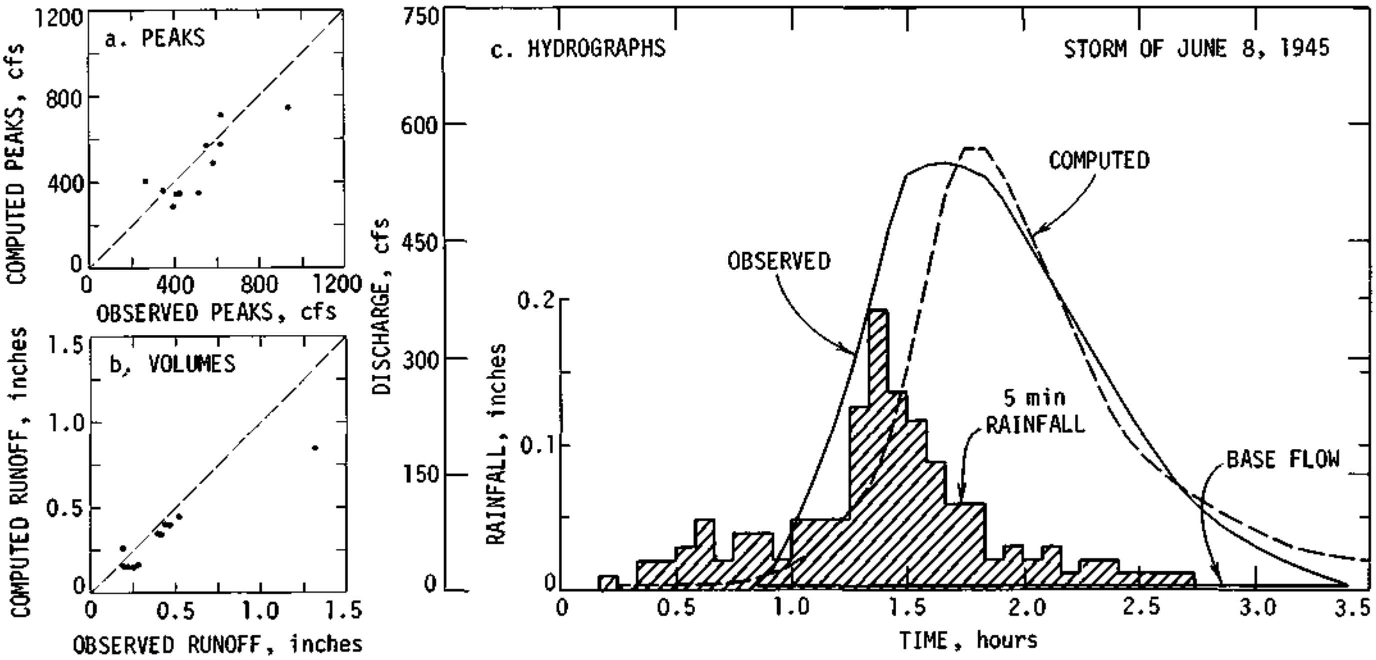


Figure 44. ILLUDAS results for Northwestern basin

Table 16. Storm Data and Results for Northwestern Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
1213 acres	498 acres	353 acres	22 acres	257 acres
	41 percent	29 percent	1.8 percent	21 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	91345	2	1.26	580	0.42	0.34	0.34	0	-20.4	490	-15.5
2	61845	4	0.75	262	0.19	0.26	0.26	28	33.0	407	55.3
3	33045	2	1.31	423	0.39	0.30	0.35	0	-10.9	348	-17.8
4	80546	2	1.47	615	0.46	0.31	0.40	2	-12.8	577	-6.2
5	60845	3	1.39	548	0.52	0.37	0.45	17	-13.9	567	3.4
6	61946	2	0.63	395	0.25	0.40	0.15	0	-38.9	285	-27.9
7	72248	4	0.96	620	0.44	0.46	0.39	36	-11.0	715	15.3
8	32447	2	0.70	512	0.28	0.40	0.17	0	-38.5	347	-32.3
9	72647	1	0.66	342	0.19	0.28	0.16	0	-13.9	358	4.8
10	60147	3	0.57	416	0.22	0.39	0.16	17	-26.9	348	-16.3
11	60147	4	2.01	939	1.32	0.66	0.85	35	-35.7	741	-21.0
Mean values						0.38			23.3		19.6

Computed peaks were high in 4 cases, average + error = 19.7 percent
Computed peaks were low in 7 cases, average — error = —19.6 percent
Computed runoff volumes were high in 1 case, average + error = 33.0 percent
Computed runoff volumes were low in 10 cases, average — error = —22.3 percent

(USDA photo, 1951)

Figure 45. Aerial photo of Northwestern basin

Baltimore Area Basins

Data for the following four Baltimore, Maryland, basins came from the Johns Hopkins University Storm Drainage Research Project. Under the direction of Dr. John C. Geyer, this program was in operation from 1948 to early 1968. In all, rainfall and/or runoff data were collected on 52 urban basins. Knapp et al. (1963) described the instrumentation developed at Johns Hopkins for measuring runoff at storm-water inlets. One-minute rainfall and runoff increments were available for the basins used in this report. These basins were inspected and the data obtained for this report during a visit to the Johns Hopkins campus. Subsequently the daia for Northwood and Gray Haven were published in two reports by Tucker (1968, 1969).

At Montebelio five inlets had been gaged within a small area. Gage No. 4 was selected because many more significant storms were available on this basin. The basin consists of one side of a residential street some 350 feet in length and the front half of a line of row or group houses as shown in figure 46. Runoff is measured by a tipping-bucket gage on the same chart. The street slope is about 2 percent but the grass slope varies greatly (figure 46). Soils are Beltsville series and are classified in hydrologic group C. A map of the basin is shown in figure 47.

Figure 46. Street scene in the Montebelio No. 4 basin

Results

ILLUDAS does an excellent job of predicting the runoff hydrographs from this small inlet area (figure 48 and table 17). The hydrograph shape and the timing of the peak are excellent.

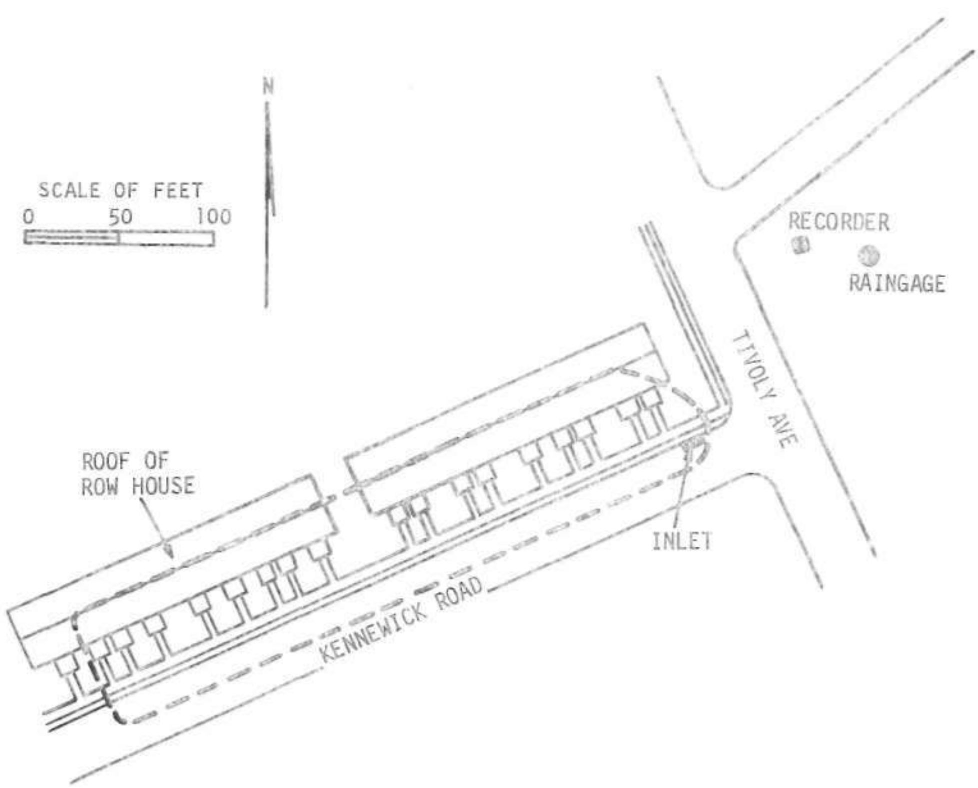


Figure 47. Map of Montebelio No. 4 basin

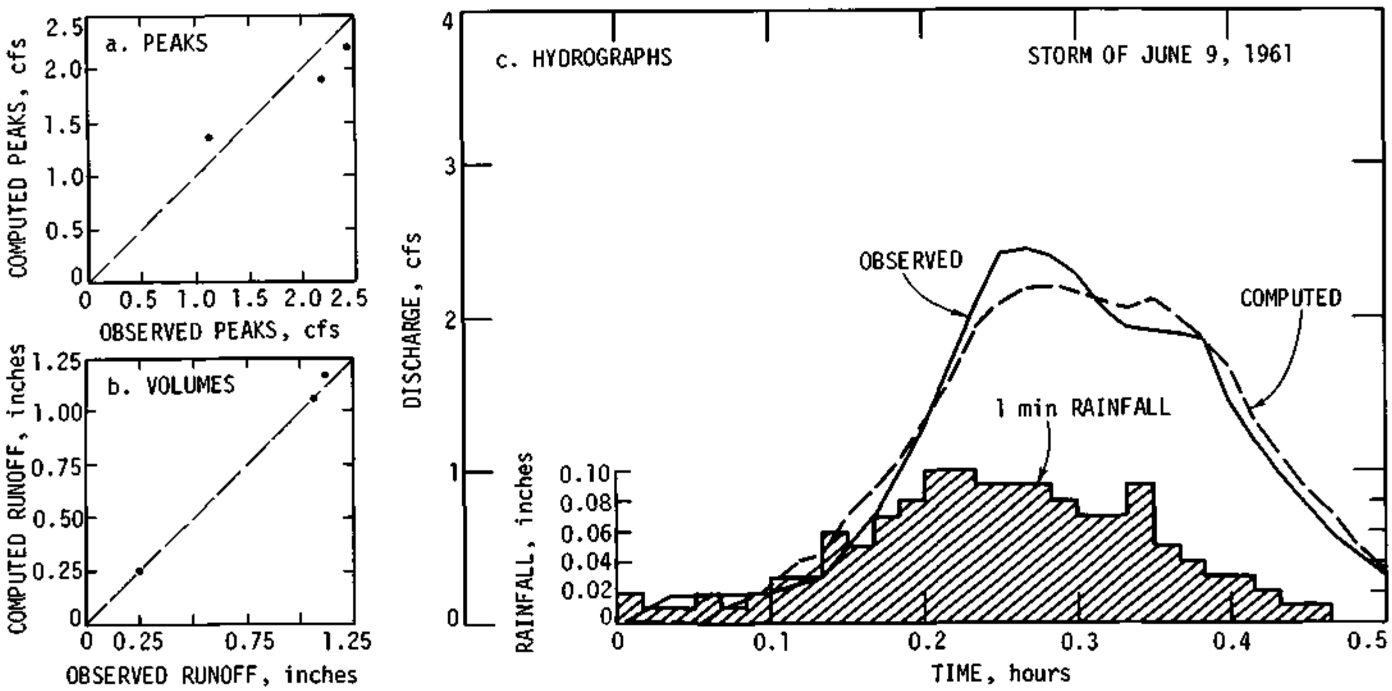


Figure 48. ILLUDAS results for Montebello No. 4 basin

Table 17. Storm Data and Results for Montebello No. 4 Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
0.54 acres	0.39 acres	0.34 acres	0.05 acres	0.15 acres
	72.2 percent	63.0 percent	9.2 percent	27.8 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	60961	2	1.44	2.44	1.06	0.74	1.06	21	-0.2	2.20	-9.6
2	90761	2	0.49	1.12	0.25	0.51	0.25	5	1.6	1.35	20.6
3	62063	1	1.85	2.19	1.12	0.60	1.16	6	4.0	1.90	-13.1
Mean values						0.62			1.9		14.4

Computed peaks were high in 1 case, average + error = 20.6 percent
Computed peaks were low in 2 cases, average — error = -11.3 percent
Computed runoff volumes were high in 2 cases, average + error = 2.8 percent
Computed runoff volumes were low in 1 case, average — error = -0.2 percent

Northwood Basin, Baltimore, Maryland

The Northwood drainage area is composed of a residential area containing group houses and a shopping center that occupies about 37 percent of the basin (figure 51). The area is drained by a 48-inch reinforced concrete pipe outletting into a concrete Parshall flume with a 12-foot throat width (figure 49). A tipping-bucket raingage records synchronously with the stage on the flume. Average grades in the basin are about 3 percent and the drainage boundaries are well defined. Sassafras series soils are dominant in the basin and are classified in hydrologic group B. The contributing paved area consists of the streets, parking lots, alleys, and about one-half of the roof area. All remaining paved area was included in the supplemental paved area. Because of the narrow yards and well-drained backyards, all grassed area was included in the contributing grassed area.

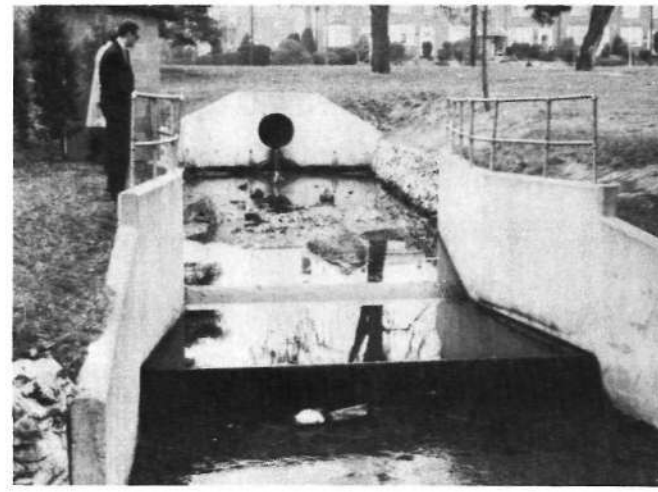


Figure 49. Parshall flume for measuring flows from the Northwood basin

Results

For the three storms analyzed ILLUDAS does an acceptable job of predicting the runoff hydrograph. Even though the predicted runoff volume is much too high for storm number 3 in table 18, the hydrograph shapes (figure 50c) and timing are good.

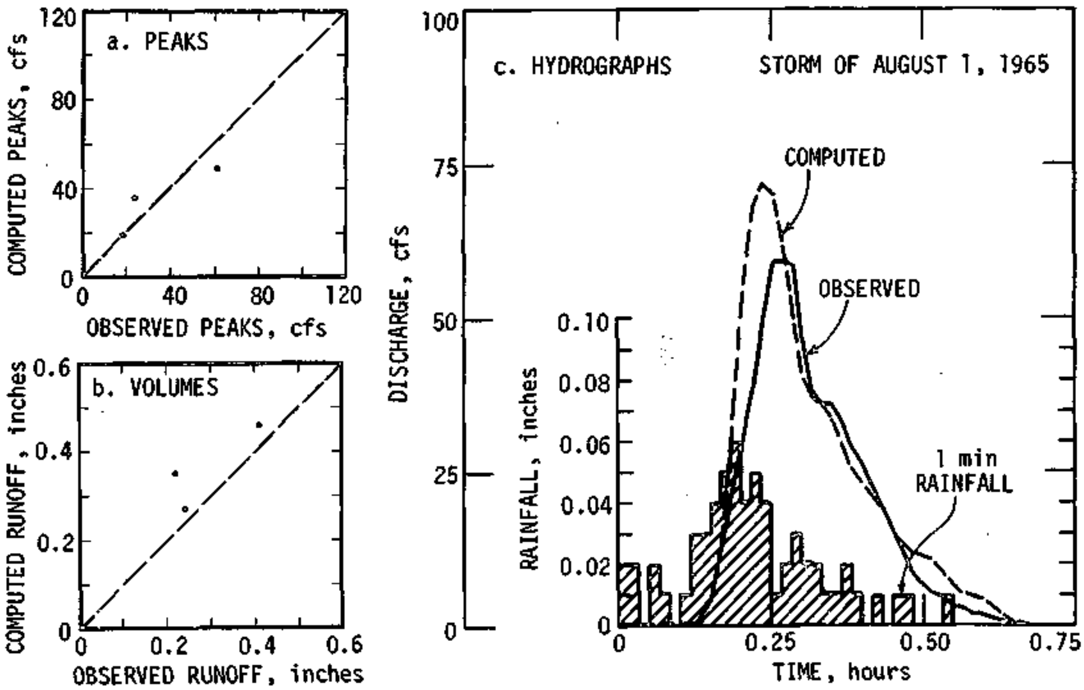


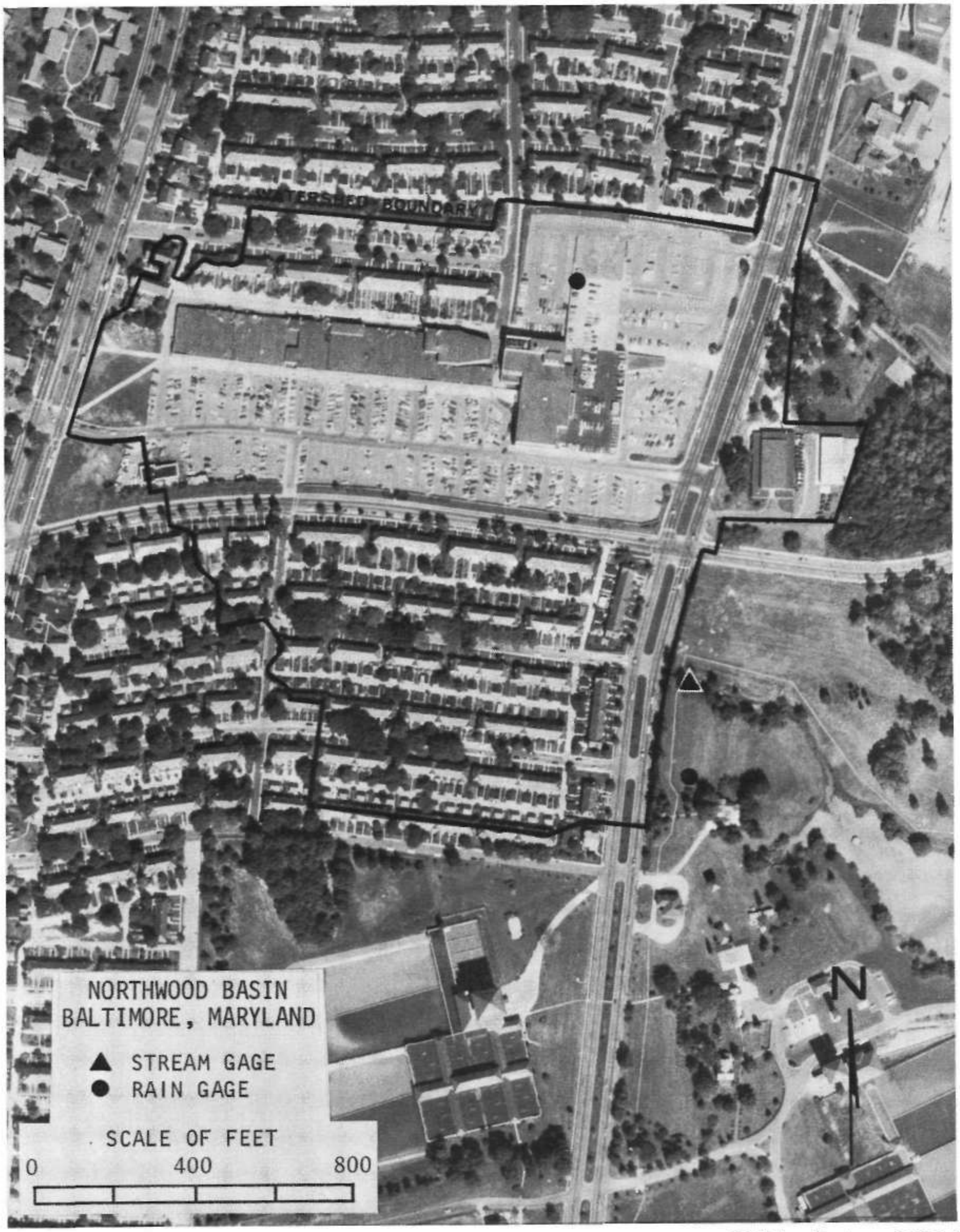
Figure 50. ILLUDAS results for Northwood basin

Table 18. Storm Data and Results for Northwood Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
47.4 acres	32.5 acres	22.7 acres	9.8 acres	14.9 acres
	68.6 percent	47.9 percent	20.7 percent	31.4 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	70565	3	0.74	107.0	0.41	0.55	0.46	29	12.1	107.6	0.5
2	80165	2	0.61	59.3	0.24	0.39	0.27	5	15.0	71.0	19.8
3	80465	3	0.64	96.3	0.22	0.34	0.35	21	63.7	83.8	-13.0
Mean values						0.43			30.2		**11.1**

Computed peaks were high in 2 cases, average + error = 10.1 percent
Computed peaks were low in 1 case, average — error = 13.0 percent
Computed runoff volumes were high in 3 cases, average + error = 30.2 percent
Computed runoff volumes were low in 0 cases



(USDA photo, 1971)

Figure 51. Aerial photo of Northwood basin

Gray Haven Basin, Baltimore, Maryland

The Gray Haven drainage area is a homogeneous residential area containing group houses on lots of 2000 to 3000 square feet (figure 5 3). Ground slopes are gentle, averaging about 0.5 percent. Subsurface drainage is provided throughout the basin and finally through a 42-inch reinforced concrete pipe into a 6-foot concrete Parshall flume. Stage measurements from the flume are recorded synchronously with a tipping-bucket raingage. Grassed areas in the basin are very gentle in slope averaging 0.5 percent. The soils are generally Sassafras series which are classified as hydrologic group B. The streets, alleys, and one-half of the roofs are included in the directly connected paved area. The remainder of the paved area is included in the supplemental paved area. All of the grassed area is considered contributing.

Results

Table 19 and figure 52 show that ILLUDAS does an excellent job of predicting the runoff hydrographs for this basin. Grassed-area runoff occurred on only 2 of the 9 storms analyzed.

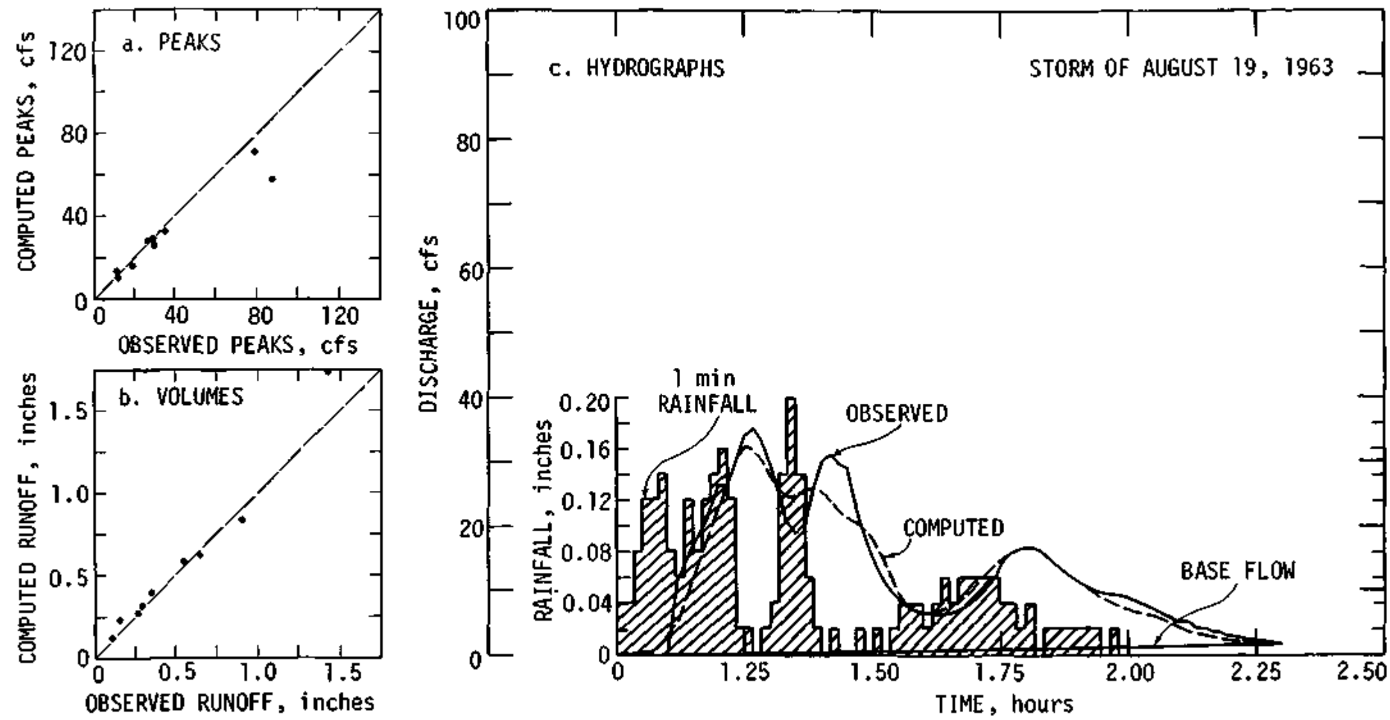


Figure 52. ILLUDAS results for Gray Haven basin

Table 19. Storm Data and Results for Gray Haven Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
23.3 acres	12.1 acres	10.3 acres	1.8 acres	11.2 acres
	52 percent	44 percent	8 percent	48 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	61063	4	2.20	79.0	1.43	0.65	1.73	47	20.9	71.0	-9.7
2	62063	1	1.42	29.6	0.55	0.38	0.58	0	6.2	29.0	-1.9
3	62963	1	0.80	27.3	0.29	0.37	0.31	0	4.8	28.2	3.4
4	80163	3	1.44	88.0	0.91	0.64	0.83	30	-8.9	57.9	-34.2
5	81363	1	0.61	11.2	0.15	0.24	0.22	0	51.3	14.6	30.4
6	81963	2	1.51	35.0	0.65	0.43	0.62	0	-5.2	32.4	-7.3
7	80265	2	0.99	30.2	0.35	0.35	0.39	0	12.4	26.2	-13.3
8	81266	2	0.37	12.4	0.11	0.30	0.12	0	7.1	10.4	-15.8
9	81266	3	0.72	19.4	0.27	0.37	0.27	0	1.9	16.3	-16.0
Mean values						0.41			13.2		14.7

Computed peaks were high in 2 cases, average + error = 16.9 percent
Computed peaks were low in 7 cases, average — error = 14.0 percent
Computed runoff volumes were high in 7 cases, average + error = 14.9 percent
Computed runoff volumes were low in 2 cases, average — error = 7.0 percent

(USDA photo, 1971)

Figure 53. Aerial photo of Gray Haven basin

South Parking Lot No. 1, Baltimore, Maryland

This basin is an asphalt parking lot at the Johns Hopkins University. It is completely surrounded by an asphalt curb and bermto define the drainage boundary (figures 54 and 55). The average slope of the basin is 1.7 percent. Runoff measurements are made with a weir installed in the storm-water inlet. Rainfall measurements are made with a tipping-bucket raingage. Synchronous records of rainfall and runoff events are obtained in 1-minute increments.

Figure 54. General view of South Parking Lot No. 1 basin

Results

ILLUDAS does an excellent job of fitting this basin. The dynamic nature of the basin makes the 1-minute data a necessity. Even with the 1-minute time increment, the shapes of the computed hydrographs (figure 56c) are not as close to the measured hydrographs as one might expect from a small 100 percent impervious basin. The runoff ratios shown in table 20 are quite variable for a basin of this nature.

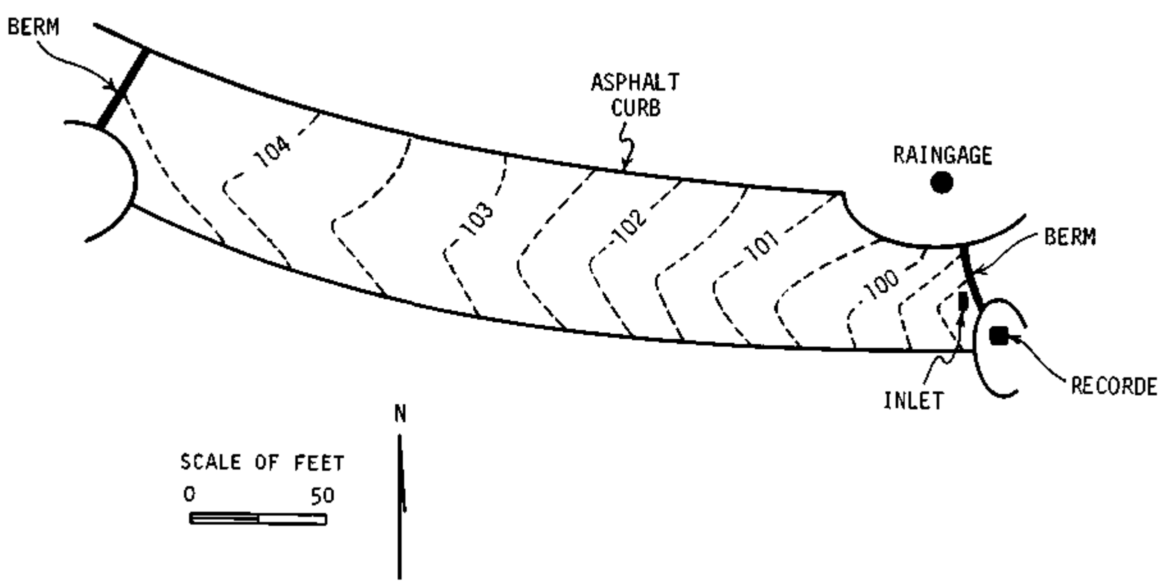


Figure 55. Map of South Parking Lot No. 1 basin

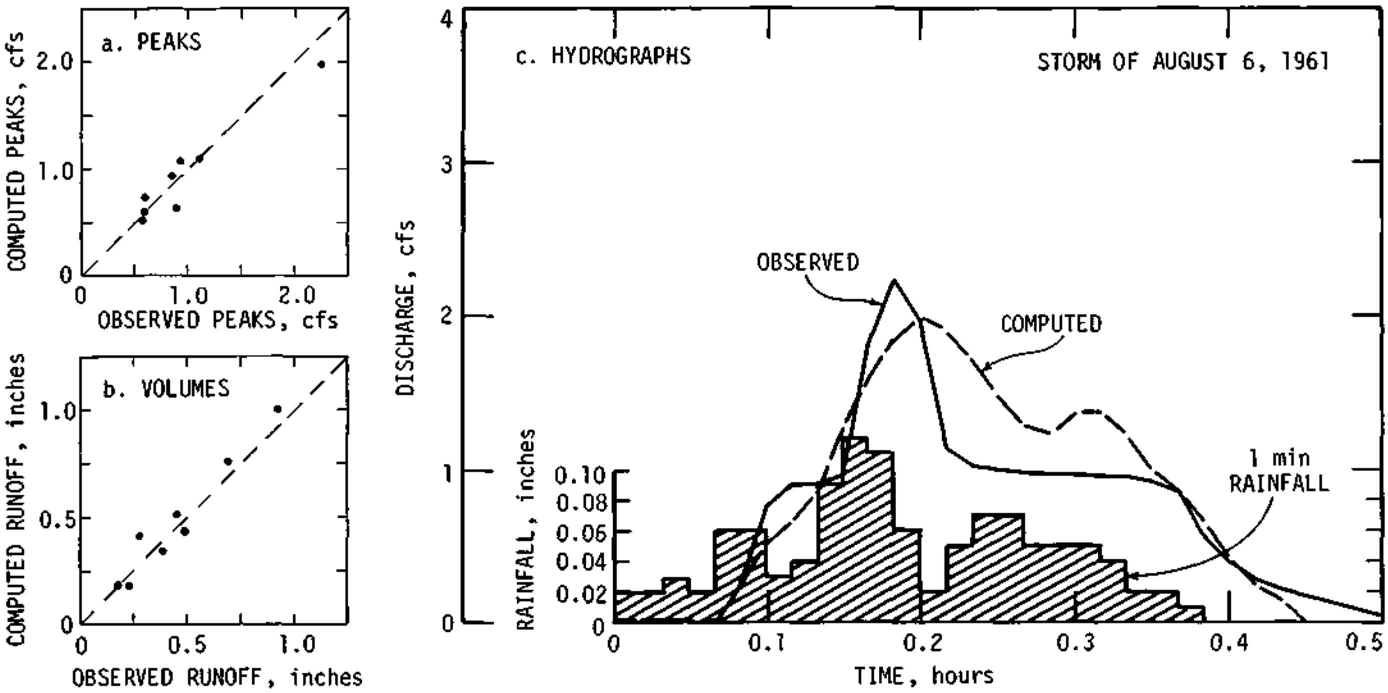


Figure 56. ILLUDAS results for South Parking Lot No. 1 basin

Table 20. Storm Data and Results for South Parking Lot No. 1 Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
0.39 acres	0.39 acres	0.39 acres	0 acres	0 acres

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	61460		0.54	1.11	0.49	0.92	0.44	0	-11.9	1.11	0.23
2	62460		0.29	0.57	0.23	0.81	0.19	0	-19.8	0.51	-9.8
3	81060		0.29	0.59	0.18	0.63	0.19	0	1.7	0.60	2.1
4	81360		0.87	0.93	0.69	0.80	0.76	0	10.2	1.07	15.5
5	81061		0.52	0.60	0.28	0.55	0.42	0	46.1	0.74	23.3
6	90960		0.62	0.85	0.46	0.74	0.52	0	12.7	0.94	10.1
7	80661		1.11	2.22	0.92	0.83	1.00	0	8.4	1.97	-11.4
8	70263		0.46	0.89	0.39	0.84	0.35	0	-8.4	0.63	-29.7
Mean values						0.76			14.9		12.8

Computed peaks were high in 5 cases, average + error = 10.2 percent
Computed peaks were low in 3 cases, average — error = 17.0 percent
Computed runoff volumes were high in 5 cases, average + error = 15.8 percent
Computed runoff volumes were low in 3 cases, average — error = 13.4 percent

Mt. Washington Basin, Cincinnati, Ohio

This basin is located in southeastern suburban Cincinnati. The data were collected in support of investigations conducted by the Robert A. Taft Sanitary Engineering Center of the federal Water Pollution Control Administration. The project is described by Weibel et al. (1964). The basin is residential and light commercial as shown in figure 57. Subsurface separate storm drainage is provided throughout the basin. Excellent storm drainage is provided for off-street parking lots and alleys. The general slope of the ground is 2 to 3 percent, but there are some very steep slopes in the vicinity of the outlet channel. Soils in the basin are of both Rossmoyne and Avonburg series which are classified as hydrologic group C and D, respectively. Rainfall was measured by a weighing-bucket type raingage located just outside the basin. Runoff was determined from stage measurements above a 4-foot rectangular weir.

Figure 57. View of Mt. Washington basin

Results

Figure 58c shows the excellent timing of the peak discharges achieved by ILLUDAS on this basin. Table 21 and figure 58 a and b indicate clearly that the computed peaks and runoff volumes are too low. The uniformity of the error seems to suggest that the contributing area of the basin was underestimated during the original analysis. There was also some question as to the watershed boundary along the commercially developed Beechmont Avenue (figure 59). Considering the questions in the basin data, ILLUDAS does an acceptable job on this basin.

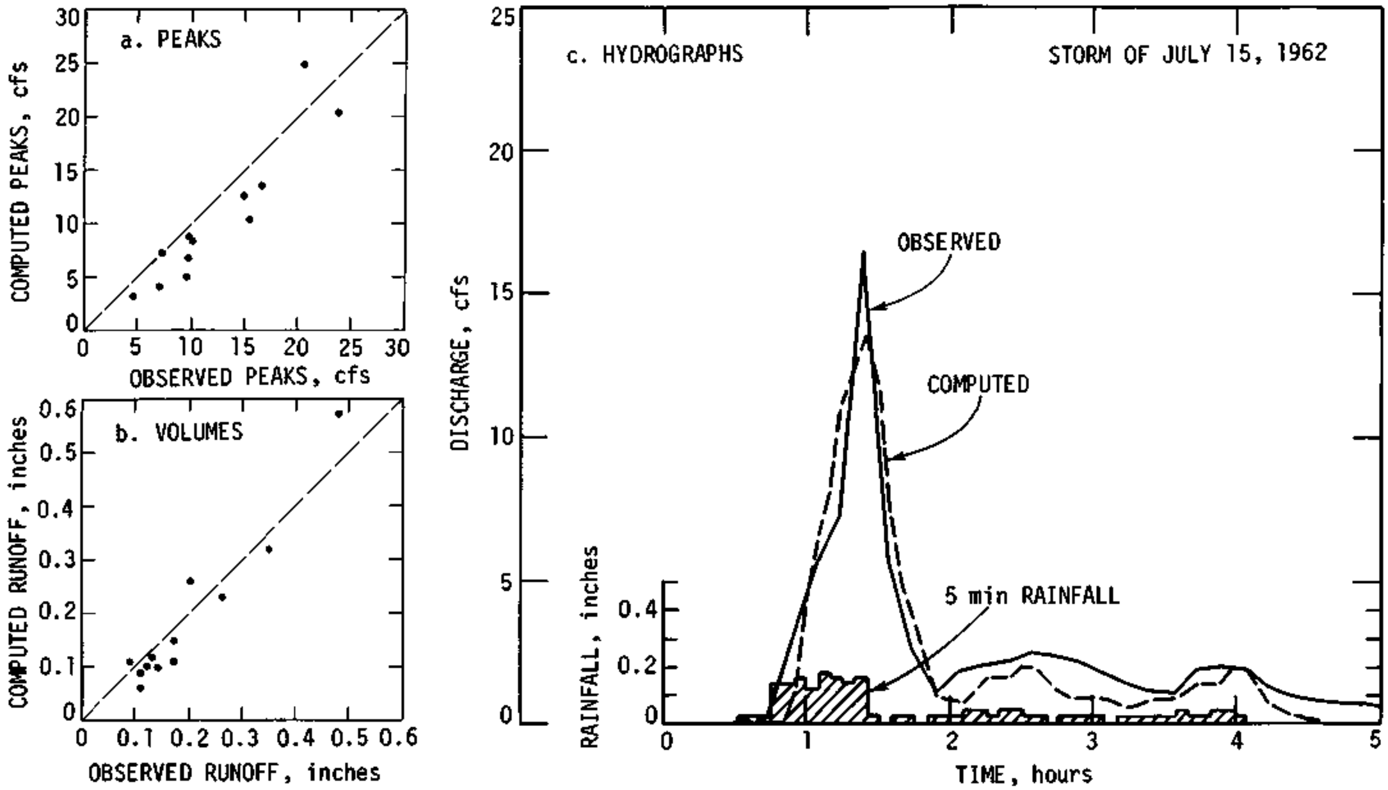


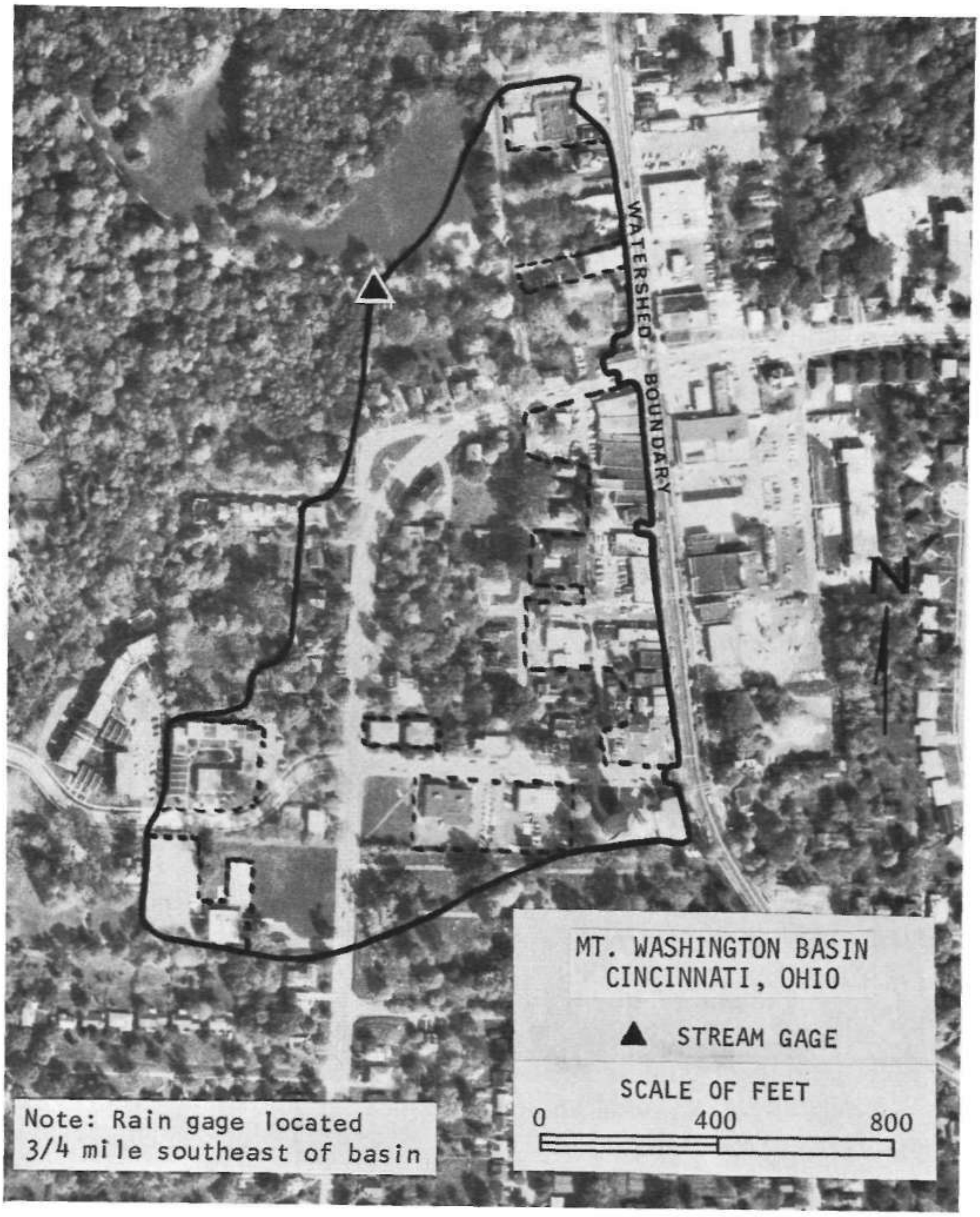
Figure 58. ILLUDAS results on Mt. Washington basin

Table 21. Storm Data and Results for Mt. Washington Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
30.7 acres	16.2 acres	14.0 acres	2.2 acres	14.5 acres
	52.8 percent	45.6 percent	7.2 percent	47.2 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent*
1	70862	3	0.43	14.9	0.11	0.25	0.09	23	-13.8	12.5	-16.0
2	71562	4	0.98	16.5	0.35	0.36	0.32	33	-6.9	13.6	-17.5
3	100262	1	0.65	4.6	0.09	0.14	0.11	0	25.9	3.1	-32.5
4	41963	2	0.67	9.5	0.13	0.20	0.12	0	-14.2	5.1	-46.6
5	42263	3	0.72	7.2	0.17	0.23	0.15	16	-8.8	7.3	1.9
6	51363	2	2.02	23.7	0.48	0.24	0.57	25	17.2	20.4	-13.9
7	72263	4	0.33	7.1	0.11	0.33	0.06	18	-45.7	4.0	-43.2
8	72363	4	0.65	20.5	0.20	0.31	0.26	39	28.9	24.9	21.7
9	81963	2	1.15	9.6	0.26	0.23	0.23	8	-11.7	8.8	-9.1
10	82863	2	0.60	15.3	0.14	0.23	0.10	5	-23.0	10.4	-31.9
11	31464	4	0.50	9.6	0.17	0.35	0.11	22	-36.3	6.8	-29.6
12	42064	4	0.38	10.0	0.12	0.31	0.10	33	-13.3	8.5	-14.7
Mean values						0.26			20.5		23.2

Computed peaks were high in 2 cases, average + error = 11.8 percent
Computed peaks were low in 10 cases, average — error = 25.5 percent
Computed runoff volumes were high in 3 cases, average + error = 24.0 percent
Computed runoff volumes were low in 9 cases, average — error = 19.3 percent



(USDA photo, 1968)

Figure 59. Aerial photo of Mt. Washington basin

Turtle Creek Basin, Dallas, Texas

This basin of 7.98 square miles is one of the two large basins to be considered in this report. The basin is located in Dallas and is upstream from a continuous-record stream gaging station operated by the U. S. Geological Survey. Figure 62 shows the residential nature of the basin. The large open areas in the vicinity of the main channel are illustrated in figure 60. Turtle Creek is characterized by a series of low dams and resultant small impoundments. The Austin-Dalco series soils in the basin are gently sloping clayey soils over chalk and are predominantly hydrologic group C. Although large open channels characterize Turtle Creek basin, 150 of the 185 subbasins used in the analysis were drained by underground storm drains. The slope of the storm drains ranges from 0.5 to 2.5 percent. Of the 1077-acre directly connected paved area (table 22), nearly 500 acres is made up of streets. The remaining 577 acres comes from rooftops in the commercial and University areas and a small percentage of residential roofs. The 480 acres of supplemental paved area comes from residential roofs and sidewalks. A strip of grass equivalent to the width of the front yards and the park areas adjacent to the main channel account for the contributing grassed area.

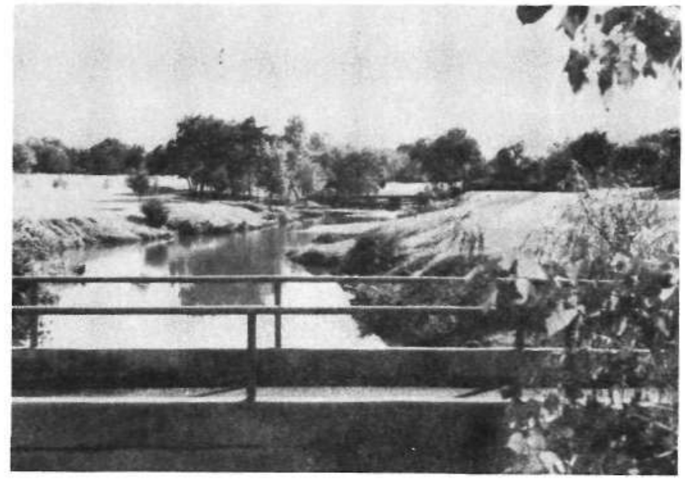


Figure 60. Ponded channel on the main stem of Turtle Creek basin

Results

Although there is considerable scatter apparent in figure 61 a and b, this might be expected from a basin of this size. Large portions of the runoff come from the grassed area as indicated in table 22. Overall, the performance of ILLUDAS seems acceptable for this basin.

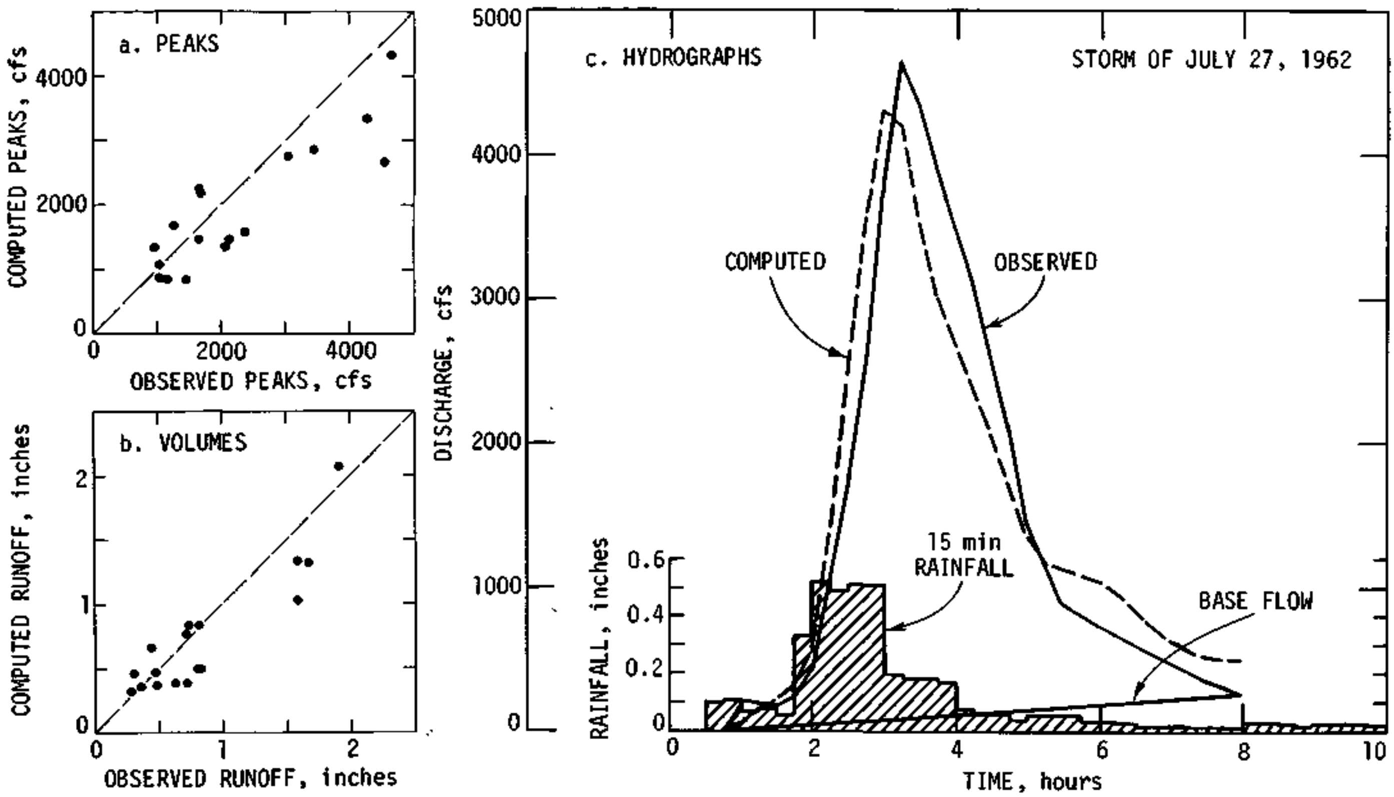


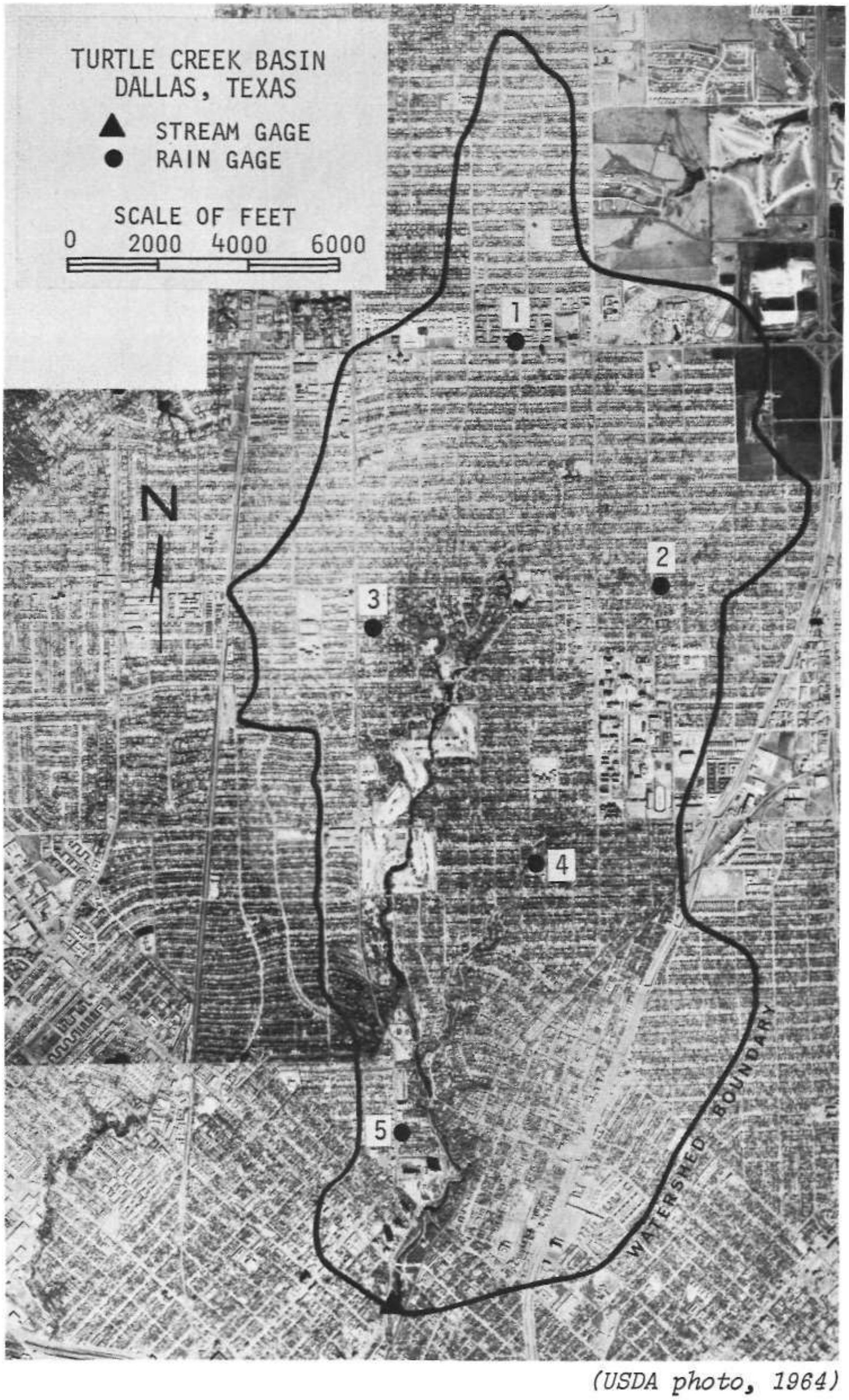
Figure 61. ILLUDAS results for Turtle Creek basin

Table 22. Storm Data and Results for Turtle Creek Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
5107 acres	1857 acres	1077 acres	480 acres	1225 acres
	36.4 percent	21.1 percent	9.4 percent	24.0 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	112261	2	1.62	1640	0.49	0.30	0.36	12	-26.1	1476	-10.0
2	43062	4	1.72	3050	0.82	0.48	0.84	60	2.3	2767	-9.3
3	72762	4	4.37	4640	1.91	0.44	2.07	57	8.5	4322	-6.8
4	90862	4	1.78	1690	0.71	0.40	0.77	54	7.9	2155	27.5
5	100862	1	3.48	3450	1.59	0.46	1.34	46	-15.7	2831	-17.9
6	42763	4	1.39	1660	0.44	0.32	0.66	56	48.9	2235	34.7
7	42863	4	2.73	4290	1.66	0.61	1.31	57	-21.2	3334	-22.3
8	61663	1	1.55	1160	0.29	0.19	0.32	4	9.8	806	-30.5
9	63063	2	1.34	1040	0.36	0.27	0.36	28	0.0	890	-14.4
10	111864	3	1.41	1030	0.47	0.34	0.47	42	0.6	1044	1.4
11	51065	3	2.32	4520	1.59	0.68	1.02	54	-35.4	2650	-41.4
12	20965	4	2.19	1280	0.74	0.34	0.83	50	11.5	1698	32.7
13	42966	4	1.17	2380	0.80	0.69	0.50	55	-37.4	1589	-33.2
14	42966	4	0.99	2080	0.72	0.73	0.38	51	-47.0	1355	-34.8
15	43066	4	1.05	1460	0.63	0.60	0.38	48	-39.9	846	-42.0
16	50166	4	1.21	2120	0.82	0.67	0.50	50	-38.7	1445	-31.8
17	61766	4	1.16	968	0.32	0.28	0.47	53	47.4	1328	37.2
Mean values						0.46			23.4		25.1

Computed peaks were high in 5 cases, average + error = 26.7 percent
Computed peaks were low in 12 cases, average — error = —24.5 percent
Computed runoff volumes were high in 8 cases, average + error =17.1 percent
Computed runoff volumes were low in 9 cases, average — error = 29.0 percent



(USDA photo, 1964)

Figure 62. Aerial photo of Turtle Creek basin

Hunting Bayou Basin, Cavalcade Street, Houston, Texas

Hunting Bayou in Houston, Texas, is gaged at two locations, at Cavalcade Street and farther downstream at Falls Street. The Cavalcade Street tributary is the smaller of the two basins and is completely contained in the Falls Street basin.

The aerial view in figure 65 shows that the eastern half of the basin at Cavalcade Street is residential and the western portion consists more of commercial, industrial, and railroad areas. Open channels make up much of the storm drainage system (figure 63). The slope of the main channel is about 10 feet per mile. The U. S. Geological Survey operates this gage and publishes storm events annually. Instrumentation consists of a Type-SR gage located on the downstream side of the bridge at Cavalcade Street. Rainfall data are supplemented by three recording gages within about 2 miles of the basin. Surface slopes in the basin are very flat with elevations ranging from 50 to 53 feet mean sea level. The soils are of the Addicks, Clodine, and Gessner series, all classified in hydrologic group D.

Figure 63. Vegetated channel in Hunting Bayou basin above Cavalcade Street

Results

ILLUDAS does a marginally acceptable job on this basin. The cause of the scatter in figure 64 a and b is not clear, but the very large rainfalls involved might be a factor. Table 23 shows that the rainfall varies from 1.60 to 4.29 inches for the storms studied. Further trouble is caused by large variations in the runoff ratio, 0.18 to 0.62, and these variations do not seem to be directly related to the antecedent moisture condition. Changing vegetation in the channel is also a factor in fitting this basin.

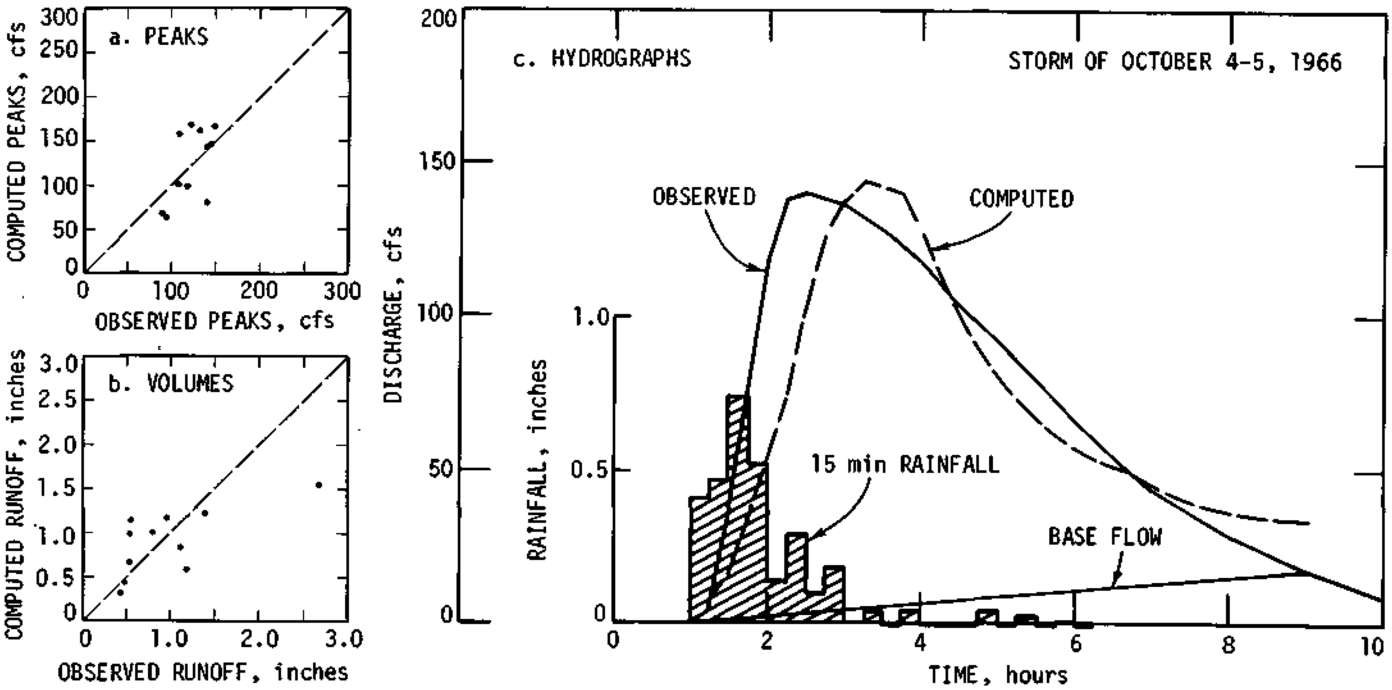


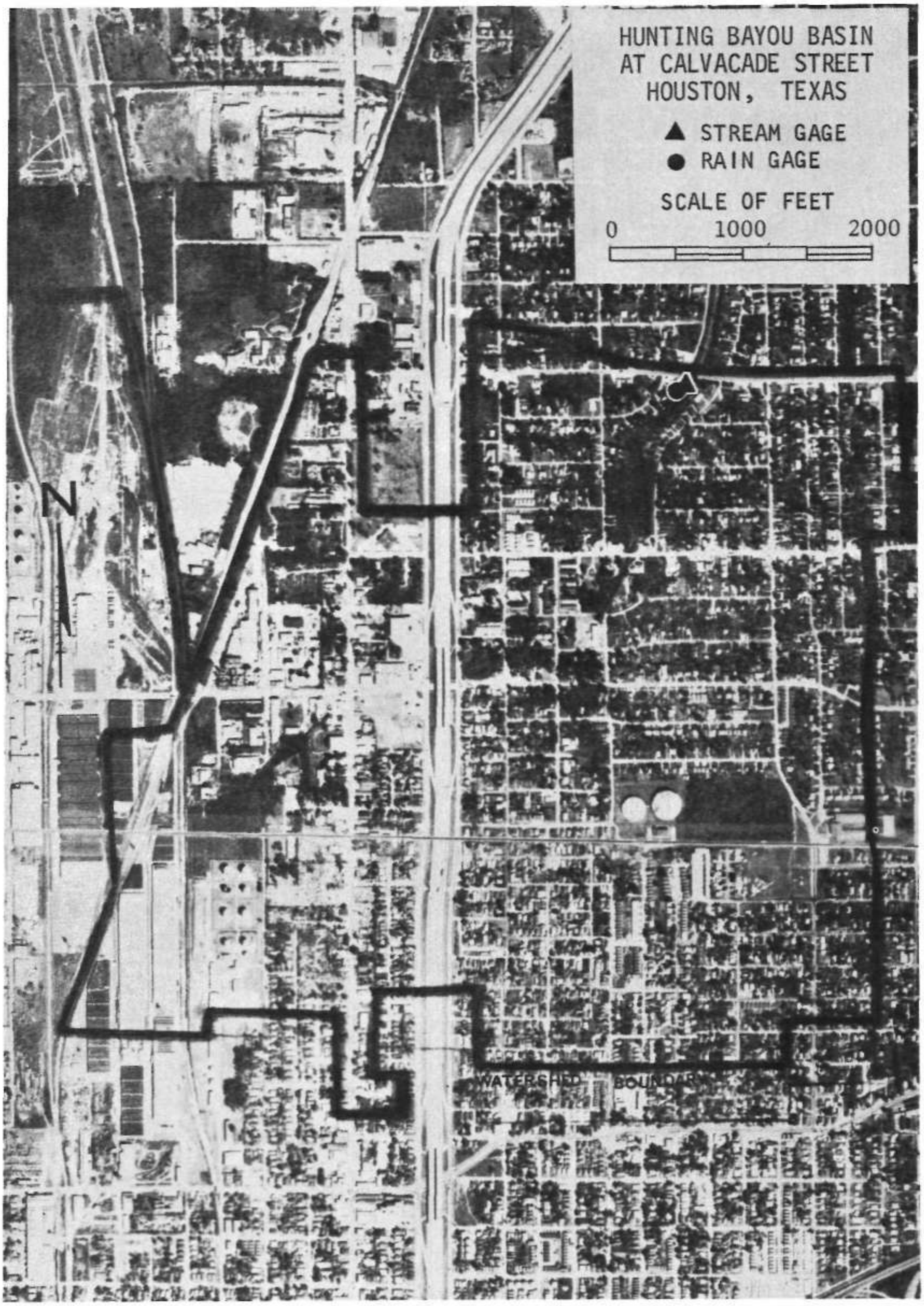
Figure 64. ILLUDAS results for Hunting Bayou basin, Cavalcade Street

Table 23. Storm Data and Results for Hunting Bayou Basin, Cavalcade Street

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
659 acres	204 acres	174 acres	15 acres	127 acres
	31.0 percent	26.4 percent	2.3 percent	19.3 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	92265	3	3.10	109	0.55	0.18	1.14	31	107.0	158	44.6
2	32866	2	1.65	90	0.43	0.26	0.32	13	-26.5	69	-23.0
3	41466	2	3.90	119	1.38	0.35	1.21	18	-11.8	100	-16.3
4	100466	2	3.07	140	0.80	0.26	1.00	21	24.0	144	2.8
5	52967	2	1.60	95	0.48	0.30	0.43	9	-9.2	66	-30.0
6	92167	2	3.50	132	0.96	0.27	1.16	23	21.4	162	22.6
7	51068	3	4.29	149	2.67	0.62	1.55	29	-42.0	167	12.3
8	90868	4	2.30	121	0.54	0.23	0.98	41	82.2	169	40.1
9	91768	4	1.95	144	1.11	0.57	0.82	40	-26.1	146	1.5
10	110568	2	2.15	111	0.53	0.24	0.66	18	24.8	102	-8.2
11	11669	1	2.33	140	1.18	0.50	0.58	0	-50.2	82	-41.6
Mean values						0.34			38.6		22.1

Computed peaks were high in 6 cases, average + error = 20.6 percent
Computed peaks were low in 5 cases, average — error = 23.8 percent
Computed runoff volumes were high in 5 cases, average + error = 51.9 percent
Computed runoff volumes were low in 6 cases, average — error = 27.6 percent



(USDA photo, 1964)

Figure 65. Aerial photo of Hunting Bayou basin, Cavalcade Street

Hunting Bayou Basin, Falls Street, Houston, Texas

The Falls Street gage is located downstream from the Cavalcade Street tributary gage. The gaging techniques and publication of data are the same as those for the Cavalcade gage. This basin tends to be more residential than the smaller Cavalcade Street basin (figure 68). Open channels form most of the storm drainage system (figure 66).

The effective roughness of these channels is evidently higher than what would normally be estimated for use in Manning's equation. On the basis of flow measurement data, values of Manning's 'n' were computed for both Hunting Bayou gages. A value of 0.09 was determined for the Cavalcade tributary channel and 0.7 for the Falls Street main channel. The main channel at Falls Street has a slope of 6 feet per mile. Soils are of the Addicks, Clodine, and Gessner series, all classified as hydrologic group D.

Figure 66. Vegetated channel in Hunting Bayou basin above Falls Street

Results

ILLUDAS seems to do a slightly better job on this basin than on the Cavalcade Street tributary, but the factors influencing them are essentially the same. The variable channel roughness due to vegetation is again evident (figure 66), as is the large variation in the runoff ratio shown in table 24. Figure 67 a and b shows that the computed peaks and runoff volumes are in the right range, but the scatter is too great.

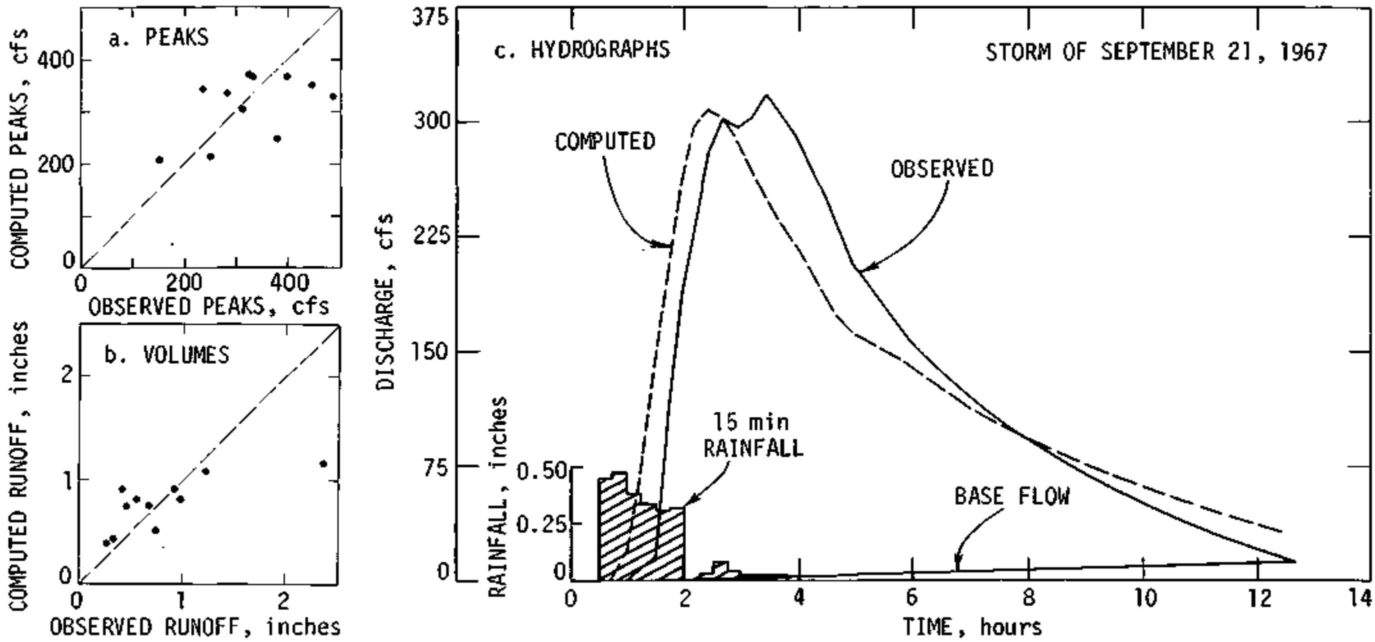


Figure 67. ILLUDAS results for Hunting Bayou basin, Falls Street

Table 24. Storm Data and Results for Hunting Bayou Basin, Falls Street

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
2189 acres	640 acres	5 39 acres	51 acres	493 acres
	29.2 percent	24.6 percent	2.3 percent	22.5 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	92265	3	2.67	236	0.42	0.16	0.91	31	119.6	342	45.1
2	32866	2	1.65	250	0.25	0.15	0.39	2	53.2	214	-14.5
3	41466	2	3.94	485	1.21	0.31	1.18	20	-2.4	328	-32.3
4	100466	2	2.93	399	0.90	0.31	0.91	24	0.9	368	-7.8
5	52967	2	1.74	154	0.31	0.18	0.44	8	39.7	209	35.6
6	92167	2	2.65	315	0.67	0.25	0.75	16	11.6	305	-3.3
7	51068	3	3.73	445	2.35	0.63	1.26	30	-46.5	350	-21.3
8	90868	4	1.94	325	0.55	0.28	0.82	45	49.9	370	14.0
9	91768	4	1.91	333	0.97	0.51	0.81	45	-16.8	366	9.8
10	110568	2	2.38	284	0.45	0.19	0.75	25	67.2	333	17.3
11	11669	1	2.24	380	0.74	0.33	0.52	0	-29.8	246	-35.1
Mean values						0.30			39.8		21.5

Computed peaks were high in 5 cases, average + error = 24.4 percent
Computed peaks were low in 6 cases, average - error = 19.0 percent
Computed runoff volumes were high in 7 cases, average + error = 48.9 percent
Computed runoff volumes were low in 4 cases, average — error = 23.9 percent

(USDA photo, 1964)

Figure 68. Aerial photo of Hunting Bayou basin, Falls Street

Boneyard Creek Basin, Champaign-Urbana, Illinois

The Boneyard Creek watershed has been gaged by the USGS continuously since 1948. The watershed area was reduced from 4.7 to 3.58 square miles in 1960 by a diversion. Only storms that have occurred since the diversion are presented here. An earlier report by Terstriep and Stall (1969) represented applications of an early version of the RRL method to the Boneyard Creek basin. Tucker (1970) published storm data for the same storms used in this report. The stream gaging site is equipped with a concrete control and a water stage recorder. A Stevens chart recorder was replaced in 1964 with an automatic digital recorder reading 5-minute intervals. Precipitation data are collected by weighing-bucket type rainfall recorders. The Water Survey operated four such gages within the watershed and six more in the general vicinity from 1949 through 1965. The basin contains portions of Urbana, Champaign, and the University of Illinois campus. The basin is entirely urban containing old and new residential areas and a sizable commercial area (figure *69).* Storm drainage is provided by an underground lateral system leading into about 3 miles of open channel. The soils are predominantly Flanigan silt loam, hydrologic group B. An aerial view is shown in figure 71.

Figure 69. Street scene in the commercial area of Boneyard Creek basin

Results

ILLUDAS does an excellent job of reproducing runoff hydrographs on Boneyard Creek. Table 25 and figure 70 show that the runoff predictions are consistent throughout the range of flows studied. There is a tendency for the peaks to be overpredicted for the larger storms. This is to be expected on a basin such as Boneyard Creek because of the large amount of surface ponding that takes place throughout the basin. Much of the ponding is caused by poor inlets, badly deteriorated pipes, and partially plugged pipes that are not accounted for by ILLUDAS.

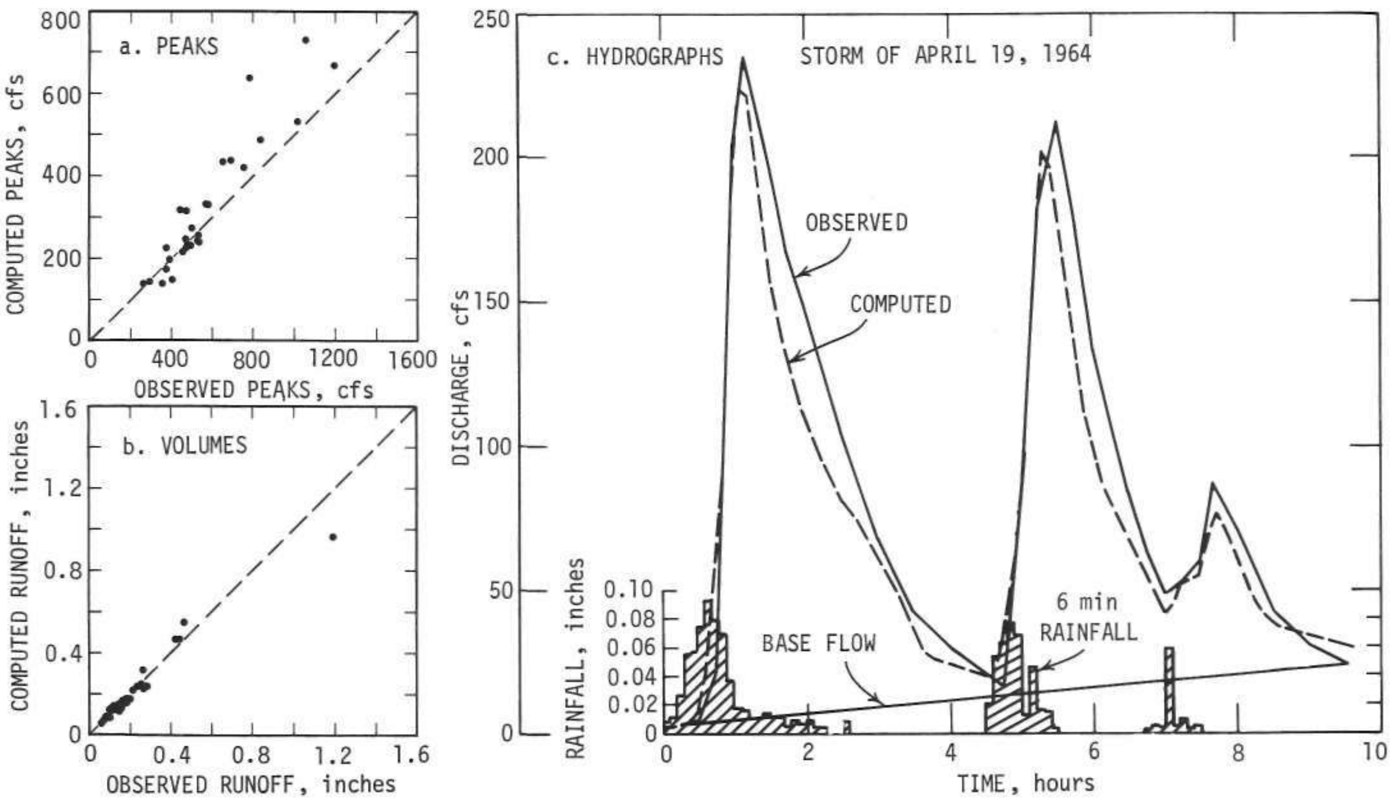


Figure 70. ILLUDAS results for Boneyard Creek basin

Table 25. Storm Data and Results for Boneyard Creek Basin

Total Basin Area	Total Paved Area	Directly Connected Paved Area	Supplemental Paved Area	Contributing Grassed Area
2290 acres	1010 acres	534 acres	299 acres	332 acres
	44.1 percent	23.3 percent	13.1 percent	14.5 percent

			Observed Storm Data				Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	102660	2	0.65	185	0.10	0.16	0.13	0	23.2	223	20.7
2	111560	1	0.84	223	0.16	0.19	0.17	0	7.8	213	-4.3
3	112860	1	0.36	143	0.06	0.17	0.06	0	-4.0	141	-1.5
4	30461	2	0.69	235	0.12	0.18	0.13	0	3.8	232	-1.5
5	60661	1	1.96	390	0.42	0.21	0.47	9	13.6	637	63.3
6	92361	2	0.39	175	0.07	0.18	0.07	0	-6.3	139	-20.6
7	51062	4	0.65	247	0.16	0.24	0.15	18	-1.6	271	9.7
8	52662	2	0.47	185	0.10	0.21	0.09	1	-13.5	171	-7.6
9	52762	3	0.47	128	0.08	0.17	0.09	0	8.5	139	8.9
10	71162	2	0.65	192	0.11	0.17	0.13	0	14.4	193	0.7
11	71362	3	0.90	247	0.20	0.23	0.18	2	-11.5	229	-7.3
12	82162	1	0.71	233	0.12	0.17	0.14	0	20.1	313	34.3
13	90362	2	0.63	219	0.11	0.17	0.12	0	15.8	315	43.7
14	61063	2	0.86	289	0.18	0.21	0.18	0	-3.4	328	13.5
15	71963	1	1.12	326	0.23	0.20	0.24	1	5.9	431	32.2
16	82863	2	1.08	342	0.26	0.24	0.23	2	-8.6	437	27.8
17	30864	2	0.65	263	0.15	0.23	0.13	0	-12.9	252	-4.2
18	41964	2	1.16	234	0.28	0.25	0.24	0	-13.8	223	-4.8
19	41964	2	0.57	264	0.14	0.25	0.11	0	-23.3	235	-11.0
20	42064	4	3.08	507	1.19	0.39	0.96	28	-20.0	529	4.3
21	61464	2	0.40	200	0.07	0.18	0.07	0	-2.2	148	-25.9
22	52565	2	1.04	376	0.21	0.21	0.22	2	4.5	419	11.4
23	70265	2	1.91	579	0.46	0.24	0.55	23	18.2	727	25.6
24	82565	2	1.86	596	0.44	0.24	0.47	16	6.2	664	11.4
25	91465	2	0.78	262	0.18	0.23	0.16	0	-11.2	239	-8.8
26	42066	3	1.15	288	0.25	0.21	0.25	3	3.0	330	14.6
27	62766	2	0.74	231	0.14	0.18	0.15	0	9.1	247	7.0
28	81866	1	1.43	416	0.26	0.18	0.31	1	21.8	483	16.1
Mean values						0.21			11.0		15.8

Computed peaks were high in 17 cases, average + error = 20.3 percent
Computed peaks were low in 11 cases, average — error = 8.9 percent
Computed runoff volumes were high in 15 cases, average + error = 11.7 percent
Computed runoff volumes were low in 13 cases, average — error = 10.2 percent

(USDA photo, 1966)

Figure 71. Aerial photo of Boneyard Creek basin

Watershed W-1, Stillwater, Oklahoma

Grassed Rural Basins

The following two basins are included in an attempt to isolate the grassed-area functions built into ILLUDAS. Although ILLUDAS was intended for and should be used on urban areas, grass lawns and parks are a part of nearly all modern urban areas. These pastured, natural grass basins provide data that are as close as the authors could come to measured rainfall and runoff from a residential lawn. ILLUDAS should not be expected to provide exact reproductions of the grassed-area hydrographs measured on these basins, but rather it should demonstrate its ability to simulate, in general, runoff from grassed areas.

The Stillwater W-1 basin is entirely in native grass pasture with average surface slopes of about 5 percent. It is part of a cooperative research project of the Agricultural Research Service of USDA and the Oklahoma Agricultural Experiment Station. Data for selected events are published annually by USDA (1964). Detailed soils information was not available for this site, but it is assumed to be similar to Lucien which is a common northern Oklahoma soil in hydrologic group C. Figure 72 is a topographic map of the basin showing the location of instrumentation.

Results

Computed results in table 26 show rather large errors in the reproduction of peaks and runoff volumes. However, figure 73 a and b indicates that although the peaks are consistently too low, the runoff volumes are in an acceptable range. The results seem especially good considering how high the runoff ratios (table 26) are for this rural basin. When we consider the increased sensitivity of the antecedent moisture condition when all runoff comes from the grassed area, and the number of factors involved in a complete analysis of grassed-area-runoff not included in ILLUDAS, the results are acceptable.

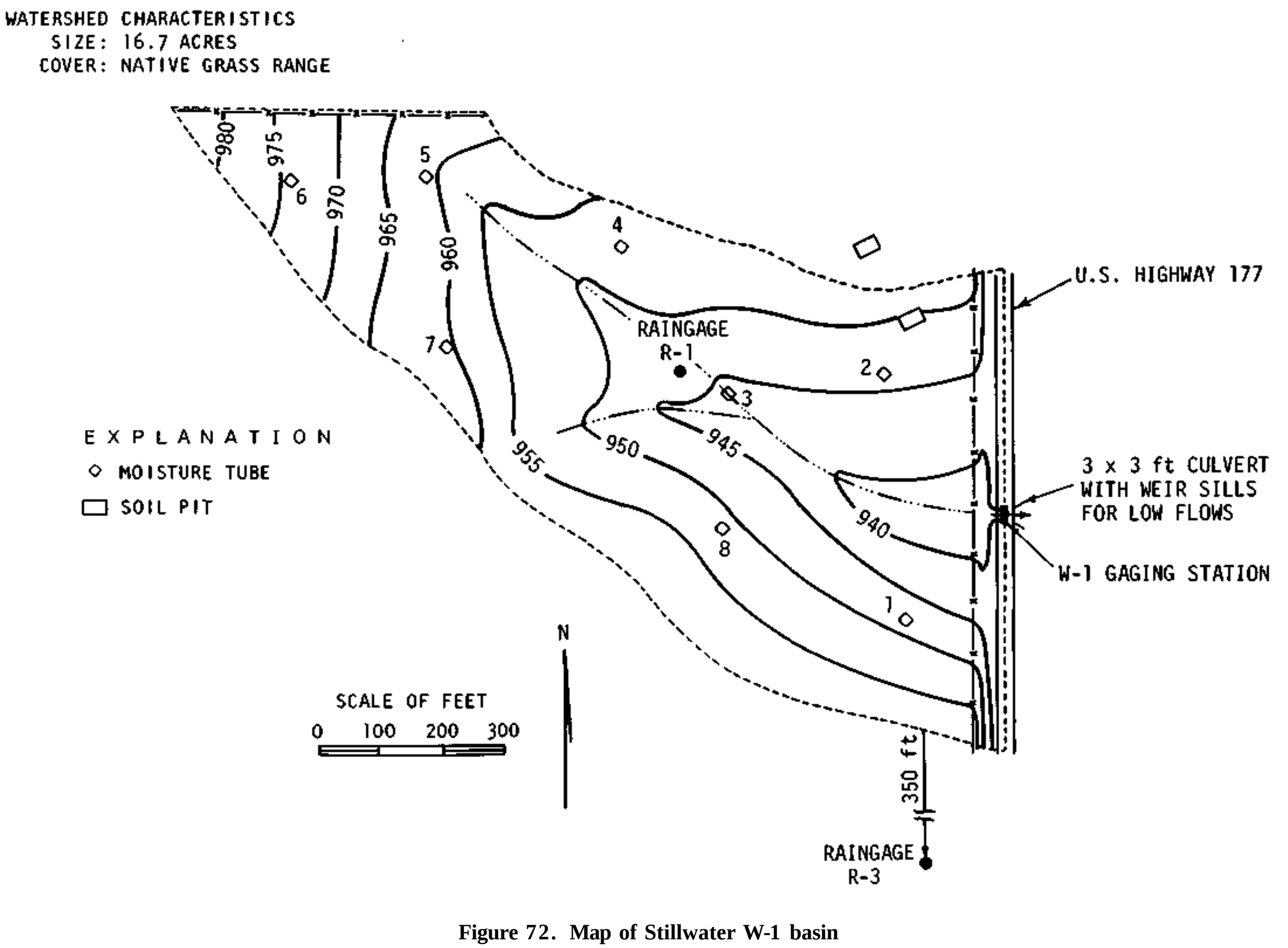


Figure 72. Map of Stillwater W-1 basin

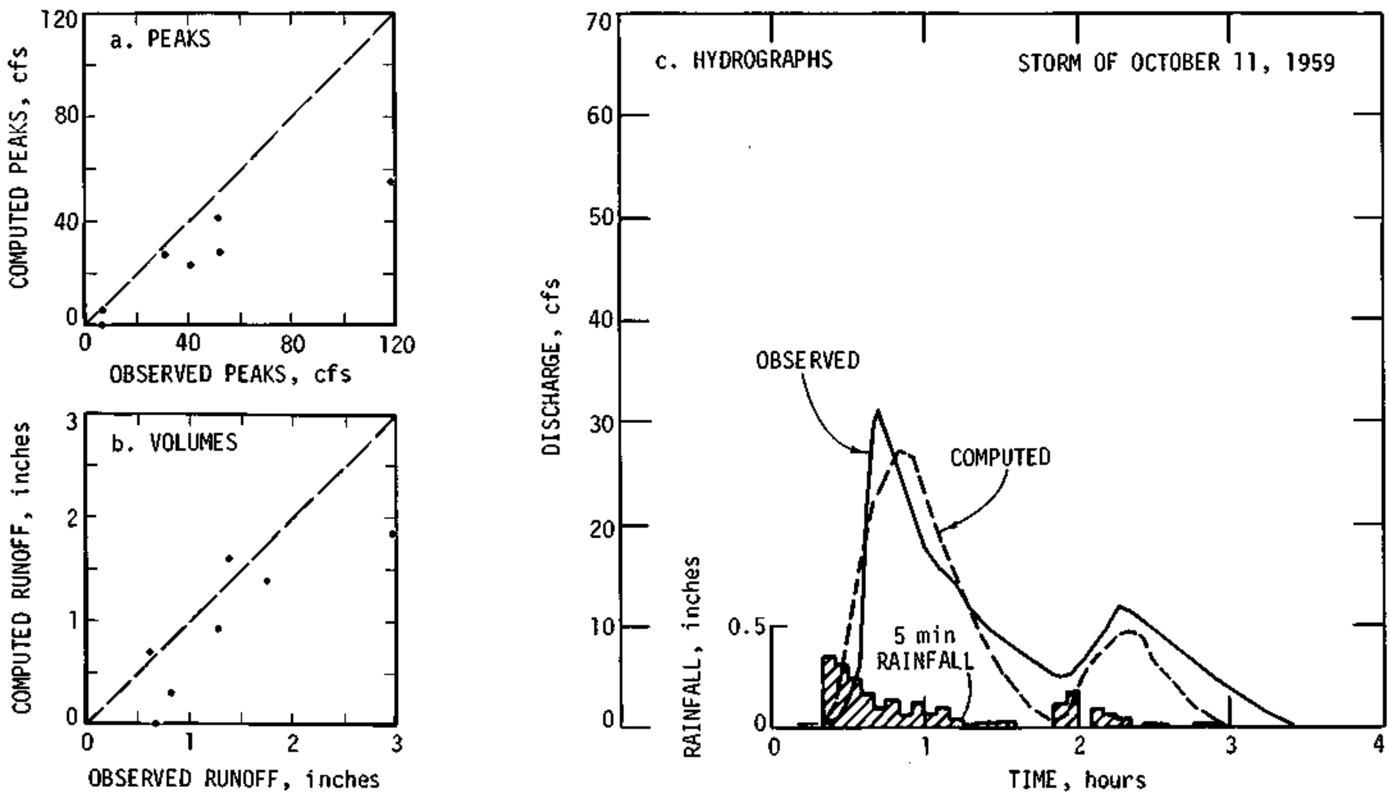


Figure 73. ILLUDAS results for Stillwater W-1 basin

Table 26. Storm Data and Results for Stillwater W-l Basin

Total Basin Area	Contributing Grassed Area
16.7 acres	16.7 acres
	100 percent

			Observed Storm Data				**Computed Results**				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff runoff *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	41857	2	3.91	118	2.97	0.76	1.85	100	-37.9	55	-53.6
2	62757	4	1.01	41	0.62	0.62	0.70	100	12.9	23	-45.4
3	100259	4	2.20	31	1.75	0.79	1.39	100	-20.2	27	-12.5
4	52860	4	2.50	51	1.37	0.55	1.60	100	16.9	41	-20.2
5	60762	3	2.01	52	1.28	0.64	0.92	100	-28.6	28	-46.4
6	90463	2	2.86	7	0.83	0.29	0.30	100	-64.3	6	-14.4
7	51064	3	1.18	7	0.59	0.50	0	100		0	
Mean values						0.59			30.1		32.1

Watershed No. 4, Mooref ield. West Virginia

This basin is a pasture planted to Kentucky Bluegrass and several native species. Land slopes are fairly steep, 10 to 12 percent being most common. The basin is gaged as a cooperative project of USDA, the Potomac Valley Soil Conservation District, and the West Virginia Agricultural Experiment Station. Selected events are published annually by USDA (1964). The predominant soil in the basin is Litz, a shaley silt classified in hydrologic group C. Figure 74 is a topographic map of the basin also showing the location of instruments. The berm around the divide of this basin provides excellent definition of the total contributing area. Uniform surface slopes also help make this an exceptionally good experimental basin.

Results

ILLUDAS does what it is expected to do on this basin. The reproduction of individual hydrographs is poor as illustrated in figure 75c, but the overall reproduction of peaks and runoff volumes in figure 75 a and b is acceptable. In table 27 where zero runoff volumes are predicted by ILLUDAS for storms 2, 4, and 5, only 0.02, 0.02, and 0.01 inch of runoff occurred. An average runoff ratio of 0.08 (table 27) compared with 0.59 for the previous Oklahoma basin illustrates the flexibility of ILLUDAS. It also demonstrates the importance of the antecedent moisture condition since both of these basins are hydrologic group C soils.

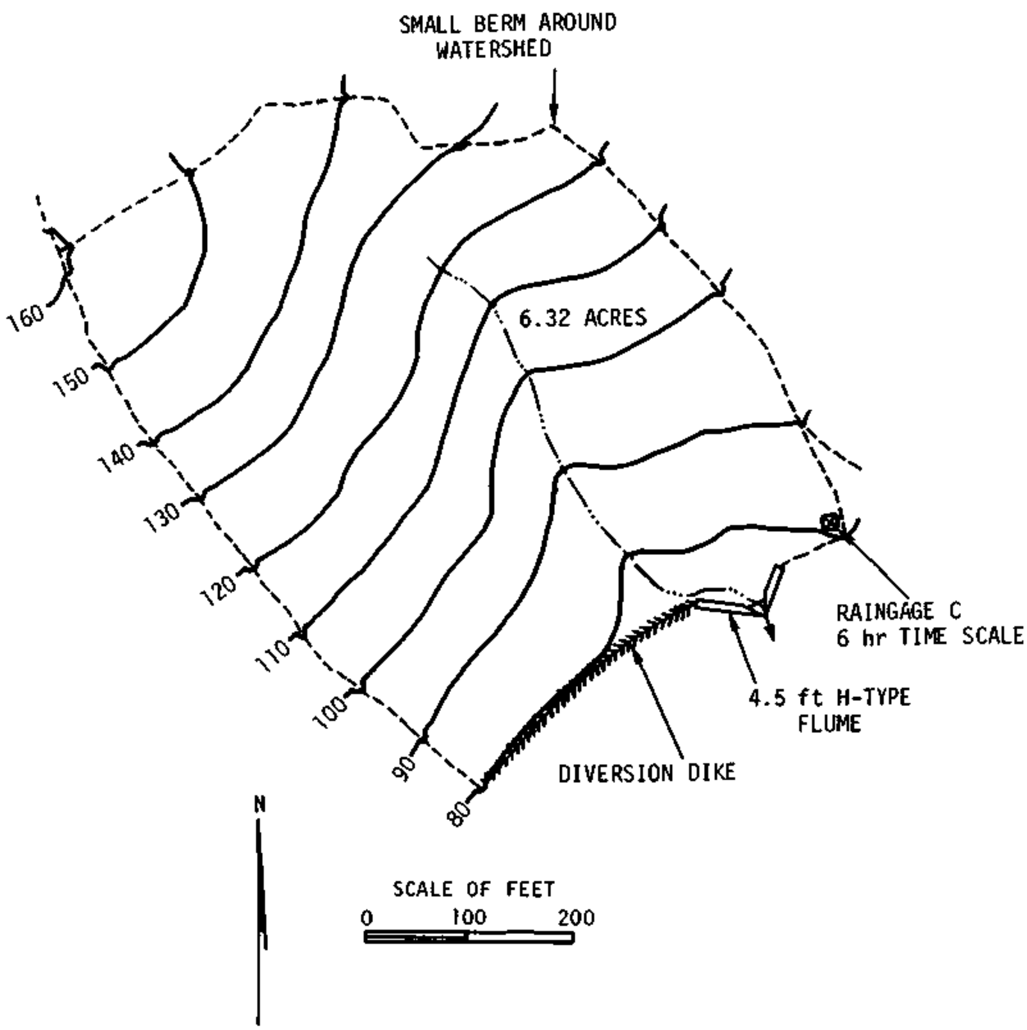


Figure 74. Map of Moorefield No. 4 basin

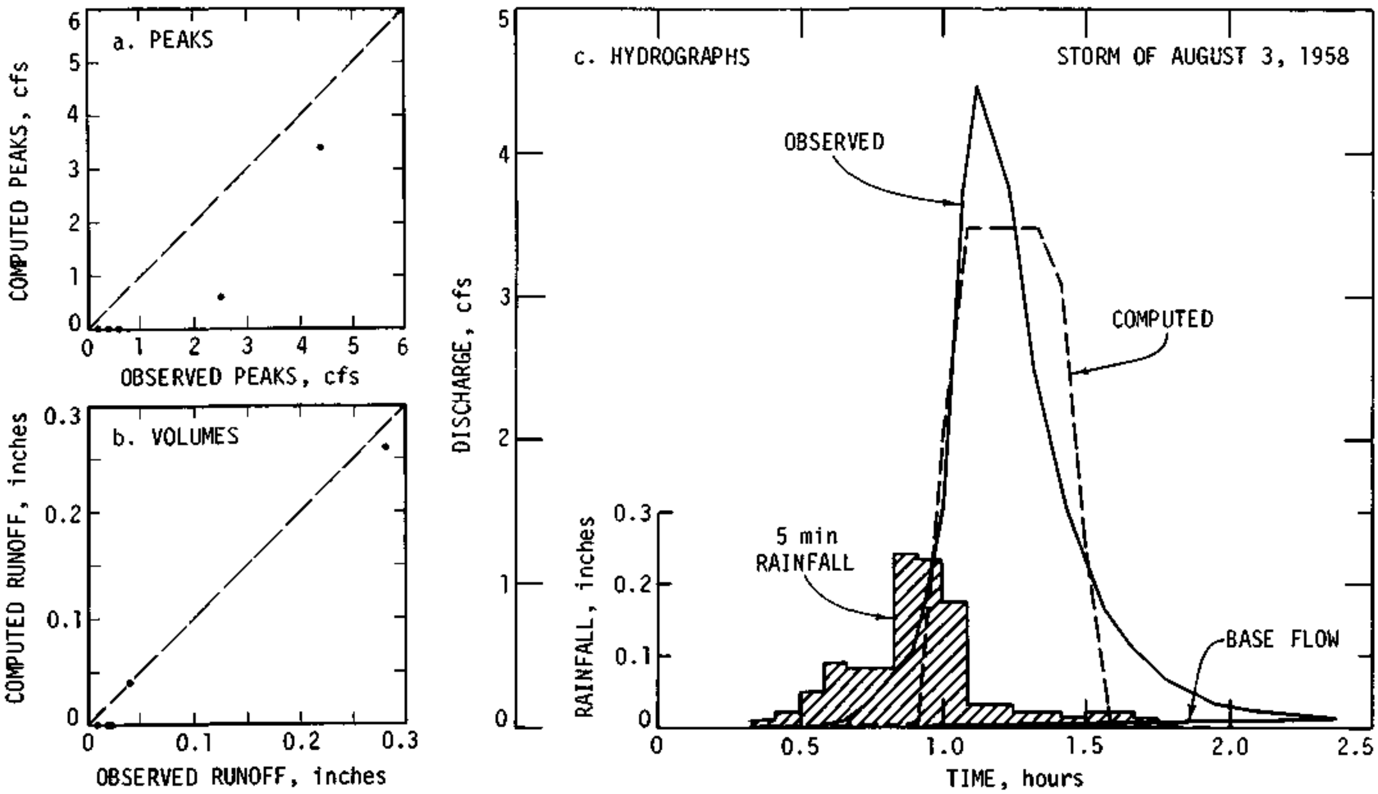


Figure 75. ILLUDAS results for Moorefield No. 4 basin

Table 27. Storm Data and Results for Moorefield No. 4 Basin

Total Basin Area	Contributing Grassed Area
6.32 acres	6.32 acres
	100 percent

		Observed Storm Data					Computed Results				
Storm	Date	AMC	Rain *(inches)*	Peak flow *(cfs)*	Runoff volume *(inches)*	Runoff ratio	Runoff volume *(inches)*	Grassed runoff *(percent)*	Error *(percent)*	Peak flow *(cfs)*	Error *(percent)*
1	80358	3	1.13	4.4	0.28	0.25	0.26	100	-7.8	3.4	-21.9
2	80961	1	0.63	0.6	0.02	0.04	0	100		0	
3	60764	2	0.78	1.5	0.04	0.06	0.04	100	-3.0	0.6	-62.6
4	81765	1	0.74	0.4	0.02	0.02	0	100		0	
5	82166	2	0.60	0.2	0.01	0.02	0	100		0	
Mean values						0.08			5.4		42.2

Summary of Verification Results

The verification of a digital model by applying it to actual basins if full of pitfalls. The first problem is the reliability of the observed data. The collection of rainfall-runoff data on urban areas is still in a development stage. Methods of stream gaging and rainfall collection that have been successfully applied to rural basins for years are not adequate for most urban basins. The time frame for an urban basin is entirely different from that of a rural basin.

Rainfall increments must be at shorter time intervals than normally considered. The 5-minute interval should not be exceeded for any urban basin, and this may not be adequate for small or steep basins. One raingage on a basin is not enough to define the storm properly. Variability in the runoff ratio from storm to storm is often the result of nonuniform rainfall over the basin. Stage recording devices must be able to react quickly both up and down with virtually no time lag. The location of a gaging station in an urban area is often compromised by the lack of really good sites. Even if a good site is available, flow measurements at high stages are nearly impossible because of the short duration of these flows. Anyone using a digital model on actual basins will inevitably find basins on which he is certain that the model is better than the data.

Verification with actual basins is further complicated by a number of conditions that are nearly impossible to evaluate in a model application. Old pipes, for example, are sometimes partially collapsed and often partially filled with debris. The actual location, slope, or diameter of existing pipes may not be known. Inadequately designed or damaged inlets can cause surface ponding that severely affects the shape of the downstream hydrograph. Many of these problems have serious effects on the evaluation of the model but are not involved when using the model to design a new drainage system.

Because of these uncertainties, it is difficult to establish an entirely objective means of assessing the performance of the model on a particular basin. It would seem that some simple criterion, such as the computed peaks being within ±20 percent of the observed values, could be used to determine the value of the model. On basins where many of the above problems exist, however, the data can easily be off by more than 20 percent. Rather than throw out such basins, it seemed more appropriate to include all of the results in this report and exercise some judgment on whether or not the model does an acceptable job on a particular basin.

The summarized verification results for all 23 basins are presented in table 28. The number of times that computed volumes and peaks were overestimated and underestimated are included along with the mean absolute error expressed as a percent of the observed value. The final column represents the conclusion of the authors as to the overall results of the ILLUDAS application.

In the authors judgment ILLUDAS produced acceptable results on 14 of the 23 basins studied. Three other basins were considered marginal, three were indeterminate, and three were not acceptable. In all cases the results could have been improved by making adjustments in the measured basin parameters such as the contributing areas and inlet times. With minor exceptions the data originally measured on topographic maps or obtained from the gaging agency were used throughout the study. The channel roughness was increased from original estimates on Hunting Bayou and Boneyard Creek.

Table 28. Summary of Results from ILLUDAS Applications on All Basins

Basin	Basin area (acres)	Total paved area (acres)	Total paved area (percent)	Directly connected paved area (acres)	Directly connected paved area (percent)	Hydro-logic soil group	Basin slope	Computed runoff No. high	Computed runoff No. low	Computed runoff Mean absolute error (percent)	Computed peaks No. high	Computed peaks No. low	Computed peaks Mean absolute error (percent)	Fit*
Woodoak Drive	14.7	4.9	33.9	2.8	19.4	B	flat	5	5	17.5	5	5	27.0	A
Sewer District No. 8	206.0	43.0	21.0	37.5	18.2	C-D	flat	10	0	60.6	9	1	55.3	NA
Echo Park Avenue	252.0	136.0	53.8	97.7	38.8	B-C	steep	6	12	31.3	2	16	36.1	NA
Crane Creek	273.0	65.5	23.9	39.7	14.5	C-D	mod	3	14	33.8	6	11	34.4	A
Tripps Run Tributary	322.0	100.0	31.0	56.9	17.7	B-C	mod	4	6	41.2	4	6	40.1	I
Tar Branch	384.0	227.0	59.0	195.0	51.0	B	mod	12	5	50.7	10	7	33.1	M
Third Fork	1075.0	397.0	37.0	293.0	27.0	B-D	mod	1	14	32.8	6	9	22.0	A
Dry Creek	1882.0	583.0	31.0	365.0	19.0	C	flar	4	4	51.6	3	5	49.1	I
Wingohocking	5326.0	3246.0	61.0	3246.0	61.0	B-D	mod	13	3	72.5	14	2	50.0	NA
First Street	61.2	49.3	80.5	39.6	64.7	B-C	flat	0	2	7.5	0	2	15.9	1
Seventeenth Street	141.0	118.3	83.7	58.4	41.4	B	flat	7	3	26.9	7	3	11.4	A
Northwestern	1213.0	498.0	41.0	353.0	29.0	C	flat	1	10	23.3	4	7	19.6	A
Montebello No. 4	0.54	0.39	72.2	0.34	63.0	C	mod	2	1	1.9		12	14.4	A
Northwood	47.4	32.5	68.6	22.7	47.9	B	mod	3	0	30.2	2	1	11.1	A
Gray Haven	23.3	12.1	52.0	10.3	44.0	B	mod	7	2	13.2	2	7	14.7	A
South Parking Lot No. 1	0.39	0.39	100	0.39	100		mod	5	3	14.9	5	3	12.8	A
Mt. Washington	30.7	16.2	52.8	14.0	45.6	C-D	mod	3	9	20.5	2	10	23.2	A
Turtle Creek	5107	1857	36.4	1077	21.1	C	flat	8	9	23.4	5	12	25.1	A
Hunting Bayou — Cavalcade	659	204	31.0	174	26.4	D	flat	5	6	38.6	6	5	22.1	M
Hunting Bayou — Falls Street	2189	640	29.2	539	24.6	D	flat	7	4	39.8	5	6	21.5	M
Boneyard Creek	2290	1010	44.1	534	23.3	B	flat	15	13	11.0	17	11	15.8	A
Stillwater W-1	16.7	0		0		C	mod	2	5	30.1	0	7	32.1	A
Moorefield No. 4	6.32	0		0		C	steep	0	5	5.4	0	5	42.2	A

*The fit of ILLUDAS is designated as follows:

A = Acceptable
NA= Not acceptable
I = Indeterminate
M = Marginal

Applying ILLUDAS

ILLUDAS may be used for the hydrologic design of a new drainage system or for the evaluation of an existing system. In either case a map of the basin is required on which the drainage boundary and the existing or proposed drainage system have been delineated. Figure 76 illustrates such a map. Points A through G may represent an inlet, a group of inlets, or a manhole, and hence each is a design point in the system. Additional design points may be added at changes in grade or to break up a long reach. For an existing system, design points should also be located at changes in pipe size or channel cross section.

Considerable topographic information must be available to determine the sub-basin area contributing to each design point. For example, in figure 76 sub-basin I contributes to point A, sub-basin III contributes to point C, sub-basin V contributes to point E, etc. Determination of sub-basin boundaries is often complicated by the existence of structures or drainage divides. On new designs, final grade plans are required. For the evaluation of existing systems it is often difficult to obtain enough information on old laterals to define the drainage network properly. After the sub-basins have been defined, they must be further sub-divided into directly connected paved areas, supplemental paved areas, and contributing grassed areas.

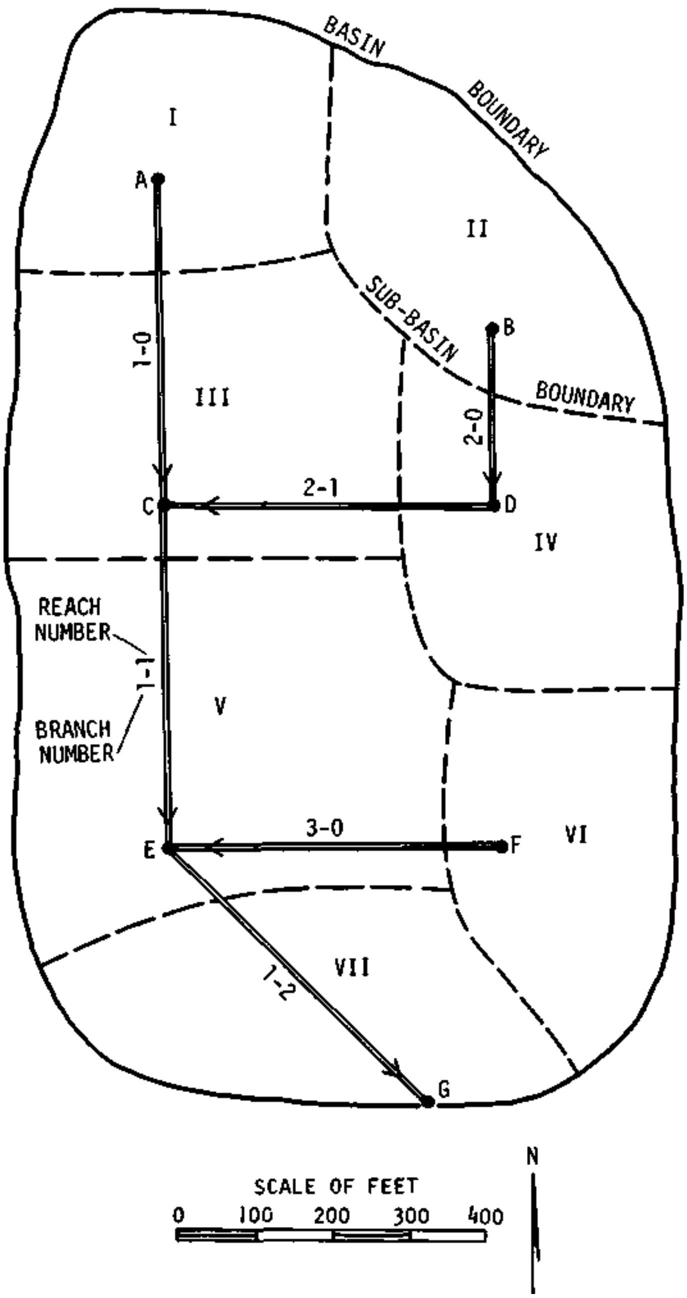


Figure 76. Sample basin showing sub-basins and reach numbering system

The design points are of course connected by lengths of underground pipe or open channel. When evaluating an existing system, the actual reaches may be described as closed circular or rectangular sections, or as open trapezoidal sections. The length, slope, and roughness of each reach are required. If the problem is a new design, ILLUDAS will use circular pipe sections exclusively. The method of assigning the reach and branch numbers indicated on figure 76 is described in the section on input data. The actual order in which ILLUDAS examines and combines the various sub-basins is determined by the order in which the cards appear in the data deck.

Table 29 indicates the proper order and some of the basic data for the basin shown in figure 76. The logic of ILLUDAS through the first part of table 29 would include the following steps:

1) Compute the surface runoff hydrograph from sub-basin I
2) Route the sub-basin I hydrograph through reach 1-0 to point C
3) Compute the surface runoff hydrograph from sub-basin II
4) Route the sub-basin II hydrograph through reach 2-0 to point D
5) Compute the surface runoff hydrograph from sub-basin IV
6) Combine the sub-basin IV hydrograph with the routed hydrograph from sub-basin II
7) Route the combined hydrograph through reach 2-1 to point C
8) Compute the surface runoff hydrograph from sub-basin III
9) Combine the sub-basin III hydrograph with the routed hydrographs from steps 2 and 7
10) Route the combined hydrograph from step 9 through reach 1-1 to point E.

In table 29 the sub-basin associated with a given reach is the sub-basin that contributes to the upstream end of that reach.

Table 29. Basic Data for Sample Basin

REACH				SUB-BASINS *(acres)*				
Branch and reach number	Length *(ft)*	Slope *(percent)*	Diameter *(inches)*	Sub-basin number	Area	Directly connected paved area	Supple-mental paved area	Contrib-uting grassed area
1-0	420	1.0	15	I	3.0	1.1	0.2	1.0
2-0	220	1.0	12	II	3.0	1.4	0.3	1.0
2-1	420	0.8	15	IV	3.0	0.8	0.5	1.2
CONFLUENCE								
1-1	440	1.2	24	III	4.0	1.9	0.2	1.4
3-0	430	0.7	15	VI	3.0	0.8	0.3	1.0
CONFLUENCE								
1-2	470	1.0	24	V	5.0	2.0	0.5	1.9
			Total		21.0	8.0	2.0	7.5
			Percent of Basin			38.1	9.5	35.7

Thus sub-basin VII will not enter the computations until the reach downstream from point G is considered. The method of telling ILLUDAS that a confluence has been reached will be described under input data. All reaches *above* a confluence must be completed before the reach *below* the confluence can be considered.

ILLUDAS provides the user with several options relating to detention storage. He must first determine if his application of ILLUDAS is an evaluation of an existing system or the design of a new system. If it is an evaluation, the output will show the flow capacity of each reach. If any reaches are incapable of carrying the design flow, the overflow will be accumulated and shown on the output as detention storage.

If the application is a new design, the output from ILLUDAS will show the design flow and the required pipe size for each reach. In the new design mode the user may specify a volume of allowable storage for one or more reaches in the system. ILLUDAS will utilize that storage and show the required outlet pipe size on the output. If the user prefers, he may specify the maximum discharge to be allowed for a given reach in the system and ILLUDAS will show the volume of detention storage accumulated at that reach.

The selection of the mode of the application, i.e., evaluation or new design, can vary from reach to reach. This option is useful in correcting problems in existing drainage systems. After an evaluation run has been made for the entire basin, the reaches where problems exist can be switched to the new design mode and an appropriate design determined. The procedure for selecting the mode of the run will be discussed under input data, cards II and VI.

The Design Storm

The user must determine the total rainfall in the design storm and also the temporal distribution of that rainfall. Table 30 and figure 77 provide the best source of point rainfall in

Table 30. Average Rainfall Frequency for Storm Periods from 15 Minutes to 3 Hours

Average return period *(years)*	Depth *(inches)* for given storm period *(hours)* 0.25	0.50	1.0	2.0	3.0
Northwest Section					
2	0.70	0.95	1.25	1.4	1.5
5	0.90	1.25	1.65	2.0	2.2
10	1.10	1.50	2.00	2.4	2.7
25	1.40	1.95	2.60	3.1	3.4
50	1.65	2.35	3.15	3.8	4.2
North Central Section					
2	0.70	0.95	1.24	1.4	1.5
5	0.85	1.25	1.55	1.8	2.0
10	1.00	1.45	1.95	2.2	2.5
25	1.25	1.75	2.35	2.7	3.1
50	1.45	2.00	2.65	3.2	3.5
South Central Section					
2	0.70	0.95	1.25	1.5	1.7
5	0.90	1.25	1.65	1.9	2.2
10	1.05	1.50	1.95	2.3	2.6
25	1.35	1.95	2.55	2.9	3.3
50	1.65	2.35	3.10	3.7	4.1
Southeast Section					
2	0.75	1.05	1.35	1.7	1.9
5	0.95	1.35	1.80	2.1	2.4
10	1.15	1.60	2.15	2.6	2.9
25	1.45	2.05	2.65	3.3	3.6
50	1.75	2.50	3.25	3.9	4.3

From Illinois State Water Survey Technical Letter 13, 1970.

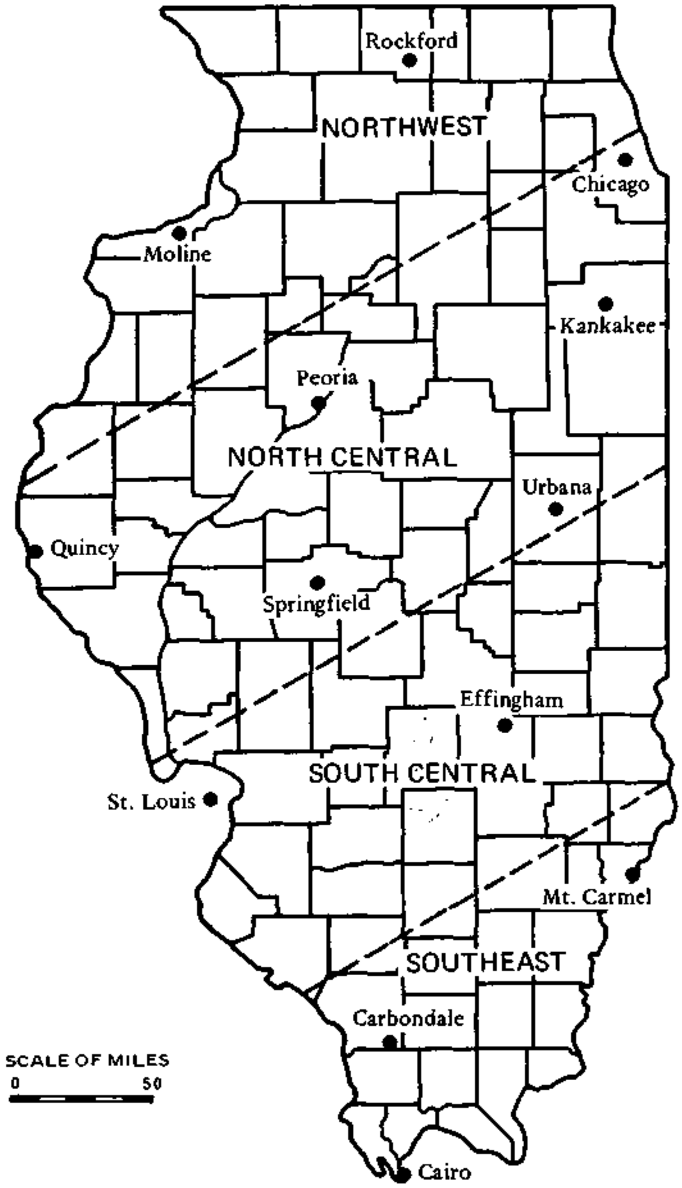


Figure 77. Sections of state according to rainfall frequencies

Illinois for various combinations of return period and duration. If the user is not working in Illinois, similar data are available for the entire United States in Technical Paper 40, U. S. Weather Bureau (1961). The return period to be used is usually fixed by ordinance or custom in each locality. Return periods of 2 to 5 years are common in residential areas. Commercial and industrial areas are often protected by designs for 10- to 25-year storms.

The proper duration to use in conjunction with table 30 will be referred to as the critical duration. Since the predicted peak flow from a basin will vary with the duration of rainfall, the critical duration will be that which causes the greatest peak flow. Historically the time of concentration for a basin, i.e., the travel time from the most remote point in the basin to the outlet, has been used as the critical duration of that basin. In an urban basin there is some question as to whether the time of travel over grassed areas should be included in the computation of critical duration. If runoff from paved areas is far in excess of runoff from grassed areas, the critical duration may well be less than the time of concentration from the total basin.

Studies using ILLUDAS have shown that 1 hour may be used as the critical duration for a wide range of basin sizes. Figure 78 illustrates the effect of duration on the peak discharge from four basins ranging in size from 0.45 square miles to 8.3 square miles. The locations of these basins are shown in figure 9. The 5-year storm rainfalls, table 31, used to compute

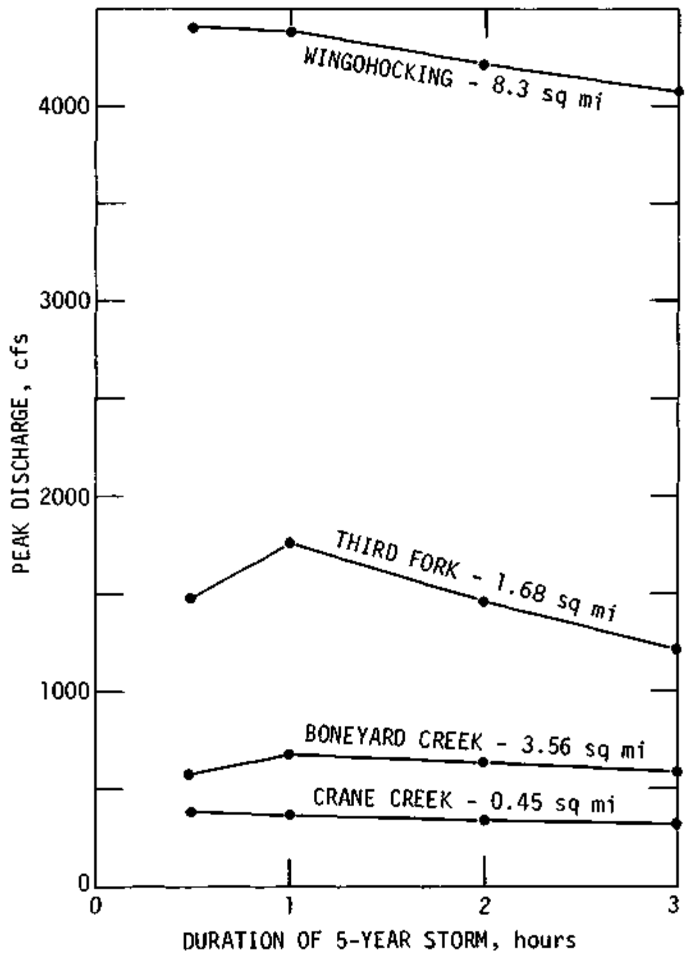


Figure 78. Effect of rainfall duration on the peak discharge from various basins

Table 31. Five-Year Storm Rainfall for Four Basins

	Total rainfall *(inches)* for given duration *(hours)*			
Basin	0.5	1.0	2.0	3.0
Wingohocking Basin, Philadelphia, Pa.	1.54	1.91	2.31	2.60
Third Fork Basin, Durham, N. C.	1.75	2.17	2.64	2.94
Boneyard Creek Basin, Champaign, Illinois	1.45	1.84	2.20	2.43
Crane Creek Basin, Jackson, Mississippi	1.95	2.48	3.06	3.41

these peaks were obtained from Technical Paper 40 for the appropriate areas of the country and at the durations shown.

In figure 78 the 1-hour duration is obviously the critical duration for the Boneyard Creek and Third Fork basins. For the Crane Creek and Wingohocking basins the 30-minute peak is higherthan the 1-hour peak but the difference is insignificant.

The time distribution of rainfall developed by Huff (1967) is recommended for use in Illinois and is a built-in feature of ILLUDAS, called the 'standard distribution.' Huff used heavy storms for point rainfall and for mean rainfall on areas of 50, 100, 200, and 400 square miles. The 261 storms on the east-central Illinois dense raingage network during the 11 years 1955-1966 were grouped according to the quartile in which rainfall was heaviest to provide four basic types of distribution. At a point and for small areas, first-quartile storms were the most frequent; consequently, the median distribution for a first-quartile storm was selected for use with ILLUDAS. This distribution is provided for point rainfall in table 32 and figure 79.

Table 32. Time Distribution of Storm Rainfall for Use in Illinois, First-Quartile Median Curve for Point Rainfall

Cumulative storm time		Cumulative storm rainfall
(minutes)	*(percent)*	*(percent)*
5	8.3	21
10	16.7	44
15	25.0	59
20	33.3	68
25	41.7	75
30	50.0	80
35	58.3	84
40	66.7	87
45	75.0	90
50	83.3	94
55	91.7	97
60	100	100

The user may specify any rainfall pattern or observed storm by including the rainfall increments at equal time intervals throughout the duration of the storm. The exact procedure for doing this is described in the section on input data.

Differences between values given in table 30 and the U. S. Weather Bureau Technical Paper 40 result from several causes and are to be expected in view of the great space and time variability in rainfall (Stall and Huff, 1971). Basic differences between the two sets of frequency relations include method of curve fitting, length of sampling period, stations employed in the analyses, and use of annual maxima as opposed to partial duration series in preparation of the published results.

The Weather Bureau used a combination of freehand curves and the Gumbel curve-fitting procedure in developing their frequency relations. The Water Survey used the Frechet curve-fitting procedure which appeared to be slightly superior to Gumbel and several others tested. When the same set of rainfall frequency data is fitted by the Gumbel and Frechet

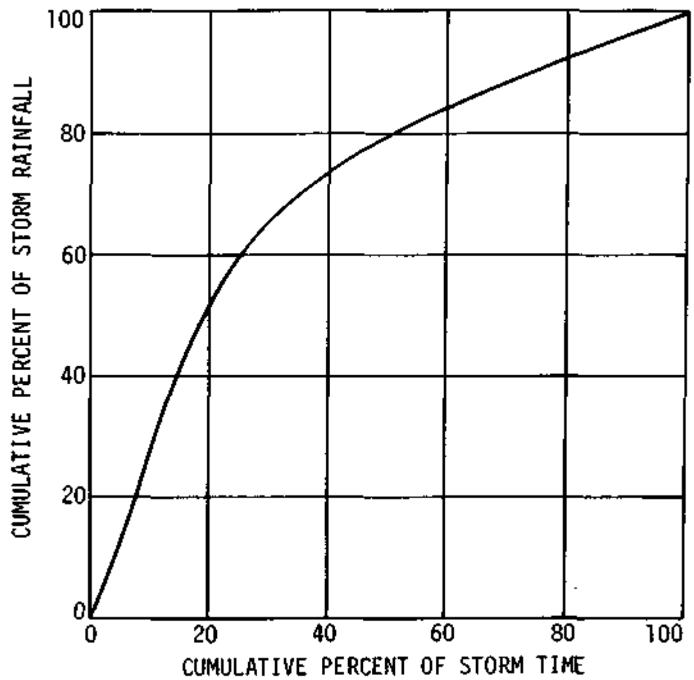


Figure 79. Time distribution of storm rainfall, median curve for point rainfall (Huff, 1967)

methods a cross-over takes place near the 10-year recurrence interval, with the Frechet values smaller at shorter intervals and larger at longer intervals. The Water Survey employed a 40-year sampling period (1916-1955) for 39 stations in their analyses and computed regional average relationships. The Weather Bureau made use of all stations with 5 years or longer records, but placed primary emphasis on long-term Weather Bureau stations, and prepared isohyetal maps from point frequency data. Also, the Water Survey results are presented in terms of annual maxima, whereas the Weather Bureau used the partial duration series in preparation of the maps in Technical Paper 40.

The Program

ILLUDAS is available from the Illinois State Water Survey in the form of a Fortran IV deck of some 700 cards. To date the program has been run only on the University of Illinois IBM 360 model 75 system. No attempt has been made to break the program down for use on smaller systems but subroutines provided in ILLUDAS could be used to facilitate this task. In its present form ILLUDAS requires 220,000 bites of core. A typical solution on a basin containing 50 pipe lengths requires about 1 minute of central processor time. Printed output requires an 88 space line. The major functions of the program are indicated by the flow diagram in figure 80.

Input Data

A complete input deck in its proper order is illustrated in figure 81. The actual content and format for each of these card types is shown on figure 82. Large scale coding sheets similar

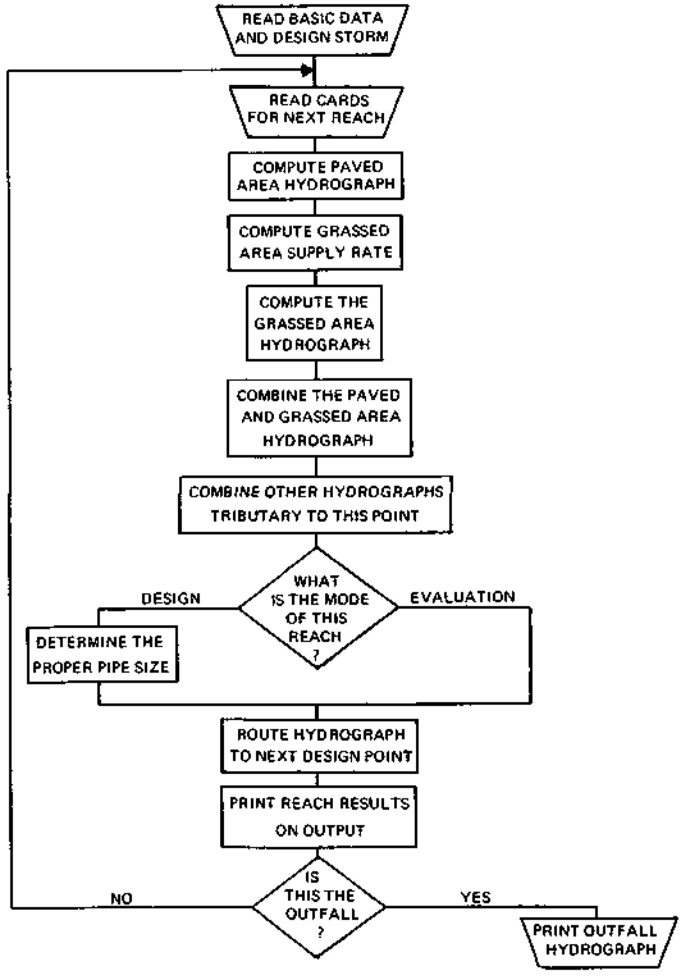


Figure 80. Flow chart for ILLUDAS

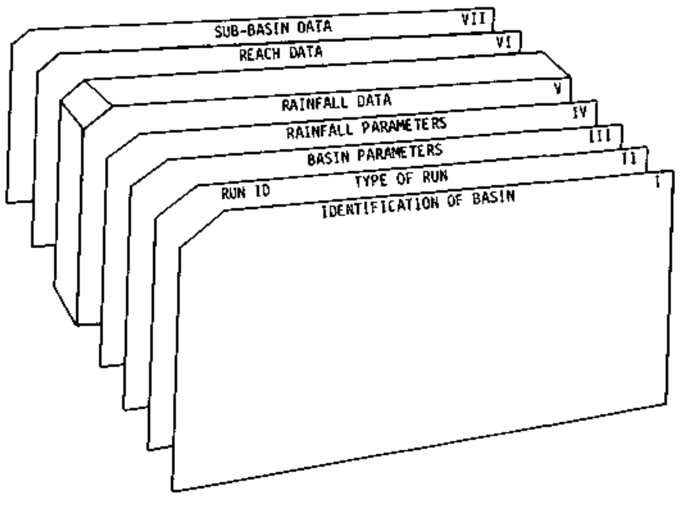


Figure 81. ILLUDAS data deck sequence

to figure 82 are available from the State Water Survey.

A few simple rules should be used when filling out a coding sheet. All numbers must be right justified, i.e., placed as far to the right in the available field as possible. Decimal points are not necessary unless fractional numbers are being used. The number 14.0, for example, may be entered on the coding sheet as 14 in the right side of the available field. When a decimal point is used it must occupy a location in the field just as an integer would. Thus the number 14.72 would require 5 spaces in a field. If more than one card of a particular type will be included in a deck, the location of the decimal points should remain the same from card to card to facilitate key punching.

The following discussion of the required input data follows item by item the same order as the input deck.

CARD I - IDENTIFICATION

The user may enter any alpha numeric information he wishes on columns 2-70 of two identification cards. The information will be printed on the first two lines of the computer output to serve as identification. Two cards should be included in the deck even if one is blank.

CARD II - TYPE OF RUN

Item 1. Run Number — May contain any numeric information suitable to identify the particular set of data. This information will appear on the printed output as the run number.

Items 2 and 3. New Design and Evaluation — These locations should contain a positive integer and a blank if this is a new design or if it is desired to know the proper pipe size for all reaches in the basin. If, on the other hand, the pipe sizes are all known and the user wishes to evaluate the system and locate problem areas, items 2 and 3 should contain blank and a positive integer, respectively.

CARD III — BASIN PARAMETERS

Item 1. Basin Area — Contains the total area of the basin in acres.

Item 2. Paved Area Abstraction — An initial abstraction to be made from rain falling on the paved portions of the basin to account for surface wetting and depression storage. This should be 0.1 inch unless the user has a specific reason to change the amount.

Item 3. Grassed Area Abstraction — An initial abstraction to be made from rain falling on grassed portions of the basin to account for depression storage. This should be 0.2 inch unless the user has a specific reason to change the amount.

Item 4. Predominant Soil Group — The most common hydrologic soil group (as defined by the U. S. Soil Conservation Service and classified by Chow, 1964) in the basin. The user should enter a 1, 2, 3, or 4 corresponding to hydrologic soil

CARD NUMBER

I

IDENTIFICATION OF BASIN
20 A4

II

1	2	3
RUN NUMBER	NEW DESIGN	EVALUATION
F10.0	F10.0	F10.0

III

1	2	3	4	5	6
BASIN AREA	PAVED ABSTRT	GRASSED ABSTRT	SOIL GROUP	MINIMUM DIAMETER	NEW PIPE "n"
F10.0	F10.0	F10.0	I10	F10.0	F10.0

IV

1	2	3	4	5	6	7	8
RAINFALL PROVIDED	NO. OF INCREMENTS	Δt	STANDARD DISTRIBUTION	DURATION	RETURN PERIOD	TOTAL RAIN	AMC
F10.0	F10.0	F10.0	F10.0	F10.0	F10.0	F10.0	F10.0

V

RAINFALL DATA (OPTIONAL)
10F8.0

VI

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
BR	RCH	TERM BR	CON BR	OPTION	L	S	"n"	SEC	DIAM	H	W	LS	QM	RATIO	STORE	END	HYD
F3.0	F3.0	F3.0	F3.0	I3	F5.0	F5.0	F5.0	I1	F4.0	F5.0	F5.0	F5.0	F5.0	F5.0	F5.0	A3	I2

VII

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
BR	RCH	AREA	DCPA	% DCPA	SPA	% SPA	PVD ENT	PVD L	PVD S	GA	% GA	GA ENT	GA L	GA S	SOIL
F3.0	F3.0	F9.0	F5.0	F3.0	F5.0	F3.0	F5.0	F5.0	F5.0	F5.0	F3.0	F5.0	F5.0	F5.0	I2

Figure 82. Code worksheet used for ILLUDAS input data

groups A, B, C, or D where A indicates a low runoff potential and D indicates a high runoff potential.

Item 5. Minimum Diameter — The user will specify the smallest pipe diameter in inches to be considered in the system design. This should be a commonly available pipe size such as 12 inches from which the program will increment the pipe size in 3-inch amounts.

Item 6. New Pipe 'n' — The Manning's 'n' value to be assigned to any new pipe size determined by ILLUDAS. Values of 'n' in common use are 0.013 for concrete pipe and 0.015 for clay pipe.

CARD IV — RAINFALL PARAMETERS

Item 1. Rainfall Provided — A positive integer in this location indicates that rainfall increments will be provided by the user. If the user provides the rainfall data, only items 1, 2, 3, and 8 on card IV need be completed.

Item 2. Number of Rainfall Increments — If the user is providing the rainfall increments, the number of increments should be entered here. As in item 1, this is left blank if the standard rainfall distribution is used.

Item 3. Time Increment — The rainfall input to ILLUDAS must be in equal increments of time from the beginning to the end of rainfall regardless of the source of the data. The user specifies the time interval here in minutes. In general the time interval should be as short as the quality of the rainfall data will allow. Another guide to the selection of an adequate interval is the average inlet time in the sub-basins. The time interval should not greatly exceed this inlet time and ideally should be 1/2 to 1/3 of the average inlet time.

Item 4. Standard Distribution — A positive integer in this location indicates that the rainfall distribution provided in ILLUDAS will be used. If this option is used, items 3, 5, 6, 7, and 8 on card IV must also be specified.

Item 5. Duration — A duration of 60 minutes is a good approximation of the critical duration, as discussed under 'design storm' in this report. Since this time can only be approximated for a complex urban basin, two or more storms having the same return period, and different durations, might be tried and the one causing the greatest runoff peak be used in design.

Item 6. Return Period — This return period in years will be printed on the computer output to aid in identification of the results. The return period as well as the duration of rainfall will be needed to determine the total rainfall from table 30. The return period to be used in a design is often regulated by ordinance of the municipality being served. If the choice is up to the designer, he will probably want to run two or more different return periods and compare costs of the various systems.

Item 7. Total Rainfall — The total rainfall in inches in the design storm as specified here will be checked against the data provided in card V if that option is used, or it will be distributed over the specified duration if rainfall increments are not provided by the user. The total rainfall for a given duration and return period can be determined for various parts of Illinois from table 30 or for other states from U. S. Weather Bureau Technical Paper 40.

Item 8. Antecedent Moisture Condition — The user must enter a 1, 2, 3, or 4 in this location. No other entries will be recognized by ILLUDAS. The antecedent moisture content is dependent on the total rainfall occurring on the five days immediately preceding the design storm. Table 4 describes these four conditions and the 5-day antecedent rainfall necessary to create them.

CARD V — RAINFALL DATA

Rainfall Pattern — This card contains the incremental rainfall in inches. The time interval, as specified in item 3 on card IV, must be uniform throughout the storm. The first rainfall increment should be zero and the increments must be 10 per card. Up to 500 rainfall increments may be used to define the storm pattern. The data on these cards will not be considered by ILLUDAS unless there is a positive integer in item 1 on card IV.

CARD VI — REACH DATA

Item 1. Branch — The branch is the first part of an identifying number assigned to each length of channel or pipe between design points. Normally the main channel of the basin would be branch number 1 from the fringe of the basin all the way to the outfall or lowermost design point. Branch number 2 would then be assigned to a major tributary from the fringe of the basin to the point where it intersects branch number 1 (figure 76). The numerical value of the branch numbers does not influence the order in which ILLUDAS is applied to the basin since this is determined by the order in which the reach cards appear in the input deck.

Item 2. Reach — The second portion of the identifying number assigned to each length of channel is the reach number. The uppermost length of channel in each branch must be assigned a reach number of zero. This number is increased by 1 for each consecutive downstream reach in a particular branch (figure 76).

Items 3 and 4. Terminating Branch and Continuing Branch — The intersection of two branches will be referred to as a confluence. A special form of card VI known as a confluence card will be used to tell ILLUDAS that a confluence has been reached. The confluence card contains only items 3 and 4. Item 3 is the number of the branch that terminates at the confluence. Item 4 is the number of the branch that continues through the confluence. The confluence card is not used until all reaches upstream from the confluence have been completed.

Item 5. Option — This item is used to define the mode (evaluation or new design) to be used for this reach. It may contain a blank which returns control of the mode to items 2 and 3 on card II or it may contain a 1 or a 2. A 1 calls the new design mode in which ILLUDAS will select a large enough pipe to pass the design hydrograph. A 2 calls the evaluation mode in which ILLUDAS will route the hydrograph through the existing pipe and print out the accumulated storage if the pipe should be undersize.

Item 6. Reach Length — This is the length in feet of this particular section of open channel or pipe.

Item 7. Slope — This is the average bed slope in percent, that is, feet drop per 100 feet, for this reach.

Item 8. Manning's 'n' — This item gives the roughness coefficient of the pipe or open channel if this reach is part of an existing system. Suggested values of 'n' are available in Chow (1964) and most other engineering handbooks. If the run is a new design, ILLUDAS will use the value specified in item 6 on card III.

Item 9. Section — This location will contain a 1, 2, or 3 indicating whether the existing cross section of the reach is circular, rectangular, or trapezoidal, respectively. For a new system design, this location would be blank since ILLUDAS uses only circular sections for new designs.

Item 10. Diameter — If this reach is an existing circular section, the user will enter the diameter in inches in this location. The user must reduce other odd-shaped sections such as oval, horseshoe, or egg-shaped to equivalent circular sections and indicate these by a 1 in item 9 and the equivalent diameter in item 10.

Item 11. Height — This location should contain the height in feet of a rectangular section. If the section should be trapezoidal this item will indicate the bank-full depth in feet.

Item 12. Width — As in item 11, this location serves two mutually exclusive functions. It may contain the width in feet of a rectangular section, or the bottom-width in feet of a trapezoidal section.

Item 13. Lateral Slope — This location is used only for trapezoidal sections and contains the lateral or side-slope of the trapezoid expressed as the feet of rise per foot of run.

Item 14. Allowable Discharge — The user may limit the flow in a particular reach by specifying here the maximum allowable discharge (QM) for the reach in cfs.

Item 15. Rainfall Ratio — The user may change the total rainfall being applied to this particular sub-basin by entering here the desired total rainfall divided by the total rainfall specified in item 7 on card IV.

Item 16. Available Storage — If the user wishes to incorporate detention storage into his design, he can specify the amount of storage in 1000 cubic feet to be provided at the entrance to this particular reach.

Item 1 7. End Test — The word END should appear in this location on the last reach in the basin.

Item 18. Print Hydrograph — The ordinates of the design hydrograph entering any reach can be printed in tabular form on the computer output by entering a positive integer in this location.

CARD VII — SUB-BASIN DATA

This card is actually a continuation of card VI. The branch and reach from card VI are repeated here to identify the sub-basin and to aid in deck assembly. Each sub-basin drains into the upstream end of the reach having the same number.

Items 1 and 2. Branch and Reach — These items are repeated from card VI for identification of the sub-basin.

Item 3. Sub-Basin Area — This is the total area of the sub-basin in acres.

Item 4. Directly Connected Paved Area — This is the paved area in acres within the sub-basin that is directly connected to the drainage system without passing over grassed surfaces. This area usually consists of the streets and alleys, most of the driveways, parking lots, and roof tops that drain onto paved areas or are piped directly to the drainage system. Item 5 can be substituted for item 4.

Item 5. Percent Directly Connected Paved Area — The user may specify the percent of the sub-basin that is directly connected and paved rather than the actual area in item 4. Either item 4 or item 5 should be used, not both.

Item 6. Supplemental Paved Area — Supplemental paved area is the paved area in acres within the sub-basin from which runoff flows onto grassed areas before reaching the drainage system. This area usually consists of sidewalks, portions of driveways, tennis courts, and roof tops which drain onto grassed areas. Item 7 can be substituted for item 6.

Item 7. Percent Supplemental Paved Area — This is the area described in item 6 expressed as a percent. Either item 6 or item 7 but not both should be used.

Item 8. Paved Area Entry Time — This is the time in minutes for rain falling on the most remote point in the directly connected paved area of the sub-basin to flow to the design point or upstream end of the pipe reach. The user may compute this time by any method of his own choosing or allow ILLUDAS to determine the entry time by specifying the next two items.

Item 9. Paved Area Flow Path — This is the length in feet of the longest probable flow path runoff might be expected to take from the most remote point on the directly connected paved area of the basin to the design point. This item will be needed by ILLUDAS to compute the paved area entry time and should be left blank if item 8 is specified by the user.

Item 10. Paved Area Slope — This is a representative slope in percent along the flow path described in item 9. This item should be left blank if the user specifies the paved area entry time in item 8.

Item 11. Contributing Grassed Area — This item should include the grassed area in acres within the sub-basin that would, if subjected to heavy rainfall, contribute runoff to the storm drainage system. Depending on the final grading and the drainage network in a residential area, this could vary from a strip of grass along the front of a row of houses to virtually all grassed area in the basin. Parks and other large grassed areas must be judged by their runoff potential. If a playground area, for example, is virtually flat and tends to pond water, it should not be included as contributing grassed area. Item 12 can be used instead of item 11.

Item 12. Percent Contributing Grassed Area — The user may specify the grassed area described in item 11 as a percent of the sub-basin in this location. Either item 11 or item 12 should be used, not both.

Item 13. Grassed Area Entry Time — This is the time in minutes for rain falling on the most remote portion of the contributing grassed area to flow to a point on the contributing paved area. The user may determinc this value by any method of his choosing or allow ILLUDAS to perform the calculation by specifying items 14 and 15 below.

Item 14. Grassed Area Flow Path — This is the length in feet of a probable flow path from the most remote portion of the grassed area to a point on the contributing paved area. If the user entered the grassed area entry time in item 13, this location should be left blank.

Item 15. Grassed Flow Path Slope — This is a representative slope in percent from the flow path described in item 14. Both item 14 and item 15 are used by ILLUDAS to compute the grassed area entry time and should be left blank if the user specifies the entry time in item 13.

Item 16. Hydrologic Soil Group — This item is the same as the soil group described in item 4 on card III and should be left blank unless the soil in this particular sub-basin differs from the predominant soil shown on card III.

Output

Two basic types of printed output are available from ILLUDAS, one for a new design and the other for an evaluation of an existing system. The output resulting from a new design is presented in table 3 3. These results were obtained by applying a 2-inch 1-hour storm distributed according to table 32 to the sample basin shown in figure 76. The first two lines contain the identification from card I of the input data. The following line of data, as indicated by the headings, contains a run identification number specified in the input data, the total area of the basin, the time increment used in the design storm, and the dominant hydrologic soil group in the basin.

The fourth line contains parameters related to the design storm as described by the headings. The next line of numerical data contains the branch and reach numbers, the length of the reach in feet, the slope of the reach in percent, and Manning's 'n.' The remainder of the line contains blanks or zeros since there are no existing pipes in a new design.

The next line, beginning with REQUIRED PIPE, contains information about the pipe selected by ILLUDAS to carry the design flow. The program then suggests a 15-inch pipe having a capacity of 6.99 cfs and a velocity of 5.7 fps. Continuing on the same line, the following two numbers are the peak of the design hydrograph at the entrance of the reach and the peak of the surface runoff hydrograph for this particular sub-basin. These two numbers are the same since this reach is the uppermost reach of a branch; that is, there are no upstream pipes contributing flow to this reach. The last number in the line, detention, is zero since the reach was designed to carry the peak of the design hydrograph. Two corresponding lines of information follow for each reach in the basin. Following the last reach are the ordinates of the outlet hydrograph at intervals equal to the time increment of the design storm.

Table 33. Printed Output Resulting from the New Design of Storm Drainage Pipes for a Basin

SAMPLE BASIN SHOWN IN FIGURE 76
10 YEAR 1 HOUR RAINFALL FROM NW ILLINOIS

RUN NUMBER	BASIN AREA ACRES	TIME INCREMENT MINUTES	SOIL GROUP 1234=ABCD
100	24.0	5.0	3

TOTAL RAIN INCHES	FREQUENCY YEARS	DURATION MINUTES	AMC	PAVED ABS. INCHES	GRASS ABS. INCHES
2.00	25	60.0	3	0.10	0.20

B	R	LENG FT	SLP PCT	N	HT FT	BW FT	V/H	DIA INS	CAPAC CFS	VEL FPS	DESIGN Q-CFS	INLET Q-CFS	DETENTION CUBIC FT
1.	0.	420.	1.00	0.0	0.0	0.0	0.0	0.	0.0	0.0	0.0		0.0
		REQUIRED PIPE =						15.	6.99	5.70	4.82	4.82	0.0
2.	0.	220.	1.00	0.0	0.0	0.0	0.0	0.	0.0	0.0	0.0		0.0
		REQUIRED PIPE =						15.	6.99	5.70	6.14	6.14	0.0
2.	1.	420.	0.80	0.0	0.0	0.0	0.0	0.	0.0	0.0	0.0		0.0
		REQUIRED PIPE =						21.	15.34	6.38	11.10	4.96	0.0
1.	1.	440.	1.20	0.0	0.0	0.0	0.0	0.	0.0	0.0	0.0		0.0
		REQUIRED PIPE =						24.	26.82	8.54	23.08	7.61	0.0
3.	0.	430.	0.70	0.0	0.0	0.0	0.0	0.	0.0	0.0	0.0		0.0
		REQUIRED PIPE =						15.	5.85	4.77	4.39	4.39	0.0
1.	2.	470.	1.00	0.0	0.0	0.0	0.0	0.	0.0	0.0	0.0		0.0
		REQUIRED PIPE =						30.	44.40	9.05	35.60	8.46	0.0

OUTLET HYDROGRAPH ORDINATES - CFS

1.83	5.28	24.02	34.73	35.40	33.21	27.77	23.72	23.45	24.76
21.51	17.35	14.29	9.46	5.28	2.81	1.65	0.72	0.13	0.0

Table 34 shows the results obtained from the analysis of an existing drainage system. The sample basin shown in figure 76 was again used. Undersize pipes were specified for three of the reaches to demonstrate generation of detention storage. The format of this *evaluation* output is the same as that for the *new design* output described above. Beginning with the line of data for branch 1 reach 0 the length, slope, and roughness have not changed. The diameter, capacity, and velocity now appear on this line rather than the following line indicating that this reach is an existing pipe. Had this reach been a rectangular pipe the height and width in feet would have appeared under headings HT and BW, respectively. Had it been an open channel the base width in feet and lateral slope as feet of rise per foot of run would have appeared under headings BW and V/H, respectively.

The second line of information relating to branch 1 reach 0 does not contain the required pipe designation or any reference to a replacement pipe. The design hydrograph peak and the surface runoff peak for the sub-basin are presented on this line exactly as they were in table 33. Because the capacity of this

Table 34. Printed Output Resulting from the Evaluation of an Existing Storm Drainage System

SAMPLE BASIN SHOWN IN FIGURE 76
10 YEAR 1 HOUR RAINFALL FROM NW ILLINOIS

RUN NUMBER	BASIN AREA ACRES	TIME INCREMENT MINUTES	SOIL GROUP 1234=ABCD
100	24.0	5.0	3

TOTAL RAIN INCHES	FREQUENCY YEARS	DURATION MINUTES	AMC	PAVED ABS. INCHES	GRASS ABS. INCHES
2.00	25	60.0	3	0.10	0.20

B	R	LENG FT	SLP PCT	N	HT FT	BW FT	V/H	DIA INS	CAPAC CFS	VEL FPS	DESIGN Q-CFS	INLET Q-CFS	DETENTION CUBIC FT
1.	0.	420.	1.00	0.012	0.0	0.0	0.0	15.	6.99	5.70			0.0
											4.82	4.82	0.0
2.	0.	220.	1.00	0.012	0.0	0.0	0.0	12.	3.85	4.91			0.0
											6.14	6.14	2259.81
2.	1.	420.	0.80	0.012	0.0	0.0	0.0	15.	6.25	5.10			0.0
											8.81	4.96	4092.61
1.	1.	440.	1.20	0.012	0.0	0.0	0.0	24.	26.81	8.54	0.0		0.0
											18.45	7.61	0.0
3.	0.	430.	0.70	0.012	0.0	0.0	0.0	15.	5.85	4.77	0.0		0.0
											4.39	4.39	0.0
1.	2.	470.	1.00	0.012	0.0	0.0	0.0	24.	24.47	7.80	0.0		0.0
											31.02	8.46	5226.75

OUTLET HYDROGRAPH

1.83	5.28	22.48	24.49	24.49	24.49	24.49	24.49	24.49	24.49
24.49	24.49	20.95	9.73	10.93	6.81	7.64	3.10	0.44	0.0
0.0									

15-inch pipe (6.99 cfs) is greater than the design peak (4.82 cfs) detention storage is zero. The following two lines show that the capacity of the 12-inch pipe in branch 2 reach 0 is 3.85 cfs and that the design peak is 6.14 cfs. The pipe is not adequate and generates a detention storage of 2259 cubic feet as indicated at the end of the second line. Comparison of the outfall hydrographs in tables 33 and 34 and the evaluation results for branch 1 reach 2 shows that the restricted capacity of the outfall pipe has reduced the peak of the outfall hydrograph from 35.4 cfs to 24.5 cfs and generated 5226 cubic feet of detention storage.

It should be pointed out that table 33 was produced completely in the new design mode and that table 34 was produced completely in the evaluation mode. In practice the mode may vary from reach to reach. For example, detention could have been introduced into branch 1 reach 1 in table 33 by specifying a 12-inch pipe for that reach. The required pipe subsequently computed for branch 1 reach 2 would then have been considerably smaller than the 30 inches indicated in table 33.

REFERENCES

American Public Works Association. 1966. *Urban drainage practices, procedures, and needs.* Chicago, Special Report 13.

American Society of Civil Engineers. 1969. *Water and metropolitan man.* New York.

Antoine, Louis H., Jr. 1964. *Drainage and best use of urban land.* Public Works, February, p. 88-90.

Ardis, Colby V., Kenneth J. Ducker, and Arno T. Lenz. 1969. *Storm drainage practices of thirty-two cities.* ASCE Journal of the Hydraulics Division, v.95(HYl).

Burgess and Niple, Ltd. 1969. *Stream pollution and abatement from combined sewer overflows, Bucyrus, Ohio.* Federal Environmental Protection Agency Water Pollution Control Series 11024 FKN.

Chow, Ven Te. 1964. *Handbook of applied hydrology.* McGraw-Hill Book Co., Inc., New York, p. 14-17.

Crawford, N. H. 1971. *Studies in the application of digital simulation to urban hydrology.* U. S. Office of Water Resources Research, Washington, D. C. Contract Report 14-31-0001-3375, 100 p.

Hicks, W. I. 1944. *A method of computing urban runoff.* American Society of Civil Engineers Transactions, v. 109: 1217-1253.

Holtan, H. N., and G. W. Musgrave. 1947. *Soil water and its disposal under corn and under bluegrass.* USDA Soil Conservation Service TP-68.

Holtan, H. N. 1961. *A concept for infiltration estimates in watershed engineering.* USDA Agricultural Research Service, ARS 41-51, October.

Holtan, H. N., C. B. England, and V. O. Shanholtz. 1967. *Concepts in hydrologic soil grouping.* American Society of Civil Engineers Transactions, v.l0(3):407-410.

Huff, F. A. 1967. *Time distribution of rainfall in heavy storms.* Water Resources Research, v.3(4): 1007-1019.

Hydrologic data for experimental agricultural watersheds in the United States. 1964. USDA, Agricultural Research Service.

Illinois State Water Survey. 1970. *Rainfall frequencies.* Technical Letter 13.

Izzard, C. F. 1946. *Hydraulics of runoff from developed surfaces.* Proceedings 26th Annual Meeting Highway Research Board, v.26:129-146.

Jens, Stifel W., and M. B. McPherson. 1964. *Hydrology of urban areas.* (Section 20) *In* Handbook of Applied Hydrology, Ven Te Chow [Ed.], McGraw-Hill Book Co., Inc., New York.

Knapp, J. W., J. C. Schaake, Jr., and W. Viessman, Jr. 1963. *Measuring rainfall and runoff at storm water inlets.* ASCE Journal of the Hydraulics Division, v.89(HY5):99.

Linsley, Ray K. 1971. *A critical review of currently available hydrologic models for analysis of urban stormwater runoff.* U. S. Office of Water Resources Research, Washington, D. C.

Metropolitan Sanitary District of Greater Chicago. 1970. *Manual of procedures for the administration of the sewer permit ordinance.*

Rice, Leonard B. 1971. *Reduction of urban runoff peak flows by ponding.* ASCE Journal of the Irrigation and Drainage Division, v.97(IR3):469-482.

Seaburn, G. E. 1970. *Preliminary results of hydrologic studies at two recharge basins on Long Island, New York.* U. S. Geological Survey Professional Paper 627-C, 17 p.

Stall, John B., and Floyd A. Huff. 1971. *The structure of storm rainfall.* Illinois State Water Survey Reprint 165.

Stall, John B., Michael L. Terstriep, and F. A. Huff. 1970. *Some effects of urbanization on floods.* Illinois State Water Survey Reprint 133.

Stall, John B., and Michael L. Terstriep. 1972. *Storm sewer design — an evaluation of the RRL method.* U. S. Environmental Protection Agency Technical Series, EPA-R2-72-068, U. S. Government Printing Office, Washington, D. C.

Terstriep, Michael L., and John B. Stall. 1969. *Urban runoff by the road research laboratory method.* ASCE Journal of the Hydraulics Division, v.95(HY6): 1809-1834.

Tucker, L. Scott. 1968. *Northwood gaging installation, Baltimore — instrumentation and data.* ASCE Urban Water Resources Research Program, Technical Memorandum 1.

Tucker, L. Scott. 1969. *Availability of rainfall-runoff data for sewered catchments.* ASCE Urban Water Resources Research Program, Technical Memorandum 8, 43 p.

Tucker, L. Scott. 1970. *Availability of rainfall-runoff data for partly sewered urban drainage catchments.* ASCE Urban Water Resources Research Program, Technical Memorandum 13.

U. S. Weather Bureau. 1961. *Rainfall frequency atlas of the United States.* Technical Paper No. 40.

U. S. Department of Agriculture. 1968. *Moisture-tension data for selected soils on experimental watersheds.* Agricultural Research Service, ARS 41-144, 609 p.

Watkins, L. H. 1962. *The design of urban sewer systems.* Department of Scientific and Industrial Research, London, Her Majesty's Stationery Office, Road Research Technical Paper 55.

Weibel, S. R., R. J. Anderson, and R. L. Woodward. 1964. *Urban land runoff as a factor in stream pollution.* Journal Water Pollution Control Federation, v.36(7).

Wilson, K. V. 1968. *A preliminary study of the effect of urbanization on floods in Jackson, Mississippi.* U. S. Geological Survey Professional Paper 5750, p. 259-261.