Urban Storm-Runoff Modeling--
Madison, Wisconsin

R. S. GRANT AND GERALD GODDARD

U. S. GEOLOGICAL SURVEY
Water-Resources Investigations
Open-File Report 79-1273

Prepared in cooperation with the
Dane County Regional Planning Commision

November 1979

UNITED STATES DEPARTMENT OF THE INTERIOR

CECIL D. ANDRUS, *SECRETARY*

GEOLOGICAL SURVEY

H. William Menard, *Director*

For additional information write to:

U. S. Geological Survey
1815 University Avenue
Madison, Wisconsin 53706

CONTENTS

ILLUSTRATIONS

TABLES

Urban Storm-Runoff Modeling--
Madison, Wisconsin

R. S. GRANT AND GERALD GODDARD

ABSTRACT

The Illinois Urban Drainage Area Simulator was used to analyze the effects that (1) physical changes to storm-sewer conduits, and (2) increased runoff detention and infiltration would have on storm runoff in four urban basins in Madison, Wisconsin. The model was calibrated using monitoring data for the four basins collected over a 1-year period. A brief evaluation was made of a modified version of the model that simulates quality of urban runoff. Additional monitoring and computer analysis are necessary to calibrate the water-quality portion of the model before it can be used as a management tool in Madison. This study was done in cooperation with the Dane County Regional Planning Commission (DCRPC).

Tables presenting results of various storm-water-management options are included. Some notable simulation results were that a 25 percent storm-sewer slope reduction yielded only a 3 percent peak-discharge reduction, and increasing storm-sewer roughness by increasing Manning's "n" from 0.013 to 0.040 decreased peak discharge about 10 to 20 percent. Detention of 10 percent of runoff throughout each basin yielded peak-discharge reductions of about 10 to 20 percent. Infiltration of all parking-lot runoff reduced peak discharges 5 to 24 percent. Peak discharges were reduced by 71 to 88 percent by substituting porous pavement for conventional pavement. Draining 90 percent of the residential rooftops onto lawns instead of driveways reduced peak discharge from 7 to 31 percent. Runoff-volume reduction was similarly reduced for the induced infiltration simulations.

Storage requirements for hypothetical storm-water-treatment plants ranged from 2.6 to 29 acre-feet for the smallest and largest basins, respectively, with a treatment capacity of 25 cubic feet per second.

A brief inconclusive evaluation of the water-quality subroutines of the model was made. Close agreement was noted between observed and simulated loads for nitrates, organic nitrogen, total phosphate, and total solids. Ammonia nitrogen and orthophosphate computed by the model ranged 7 to 11 times greater than the observed loads. Observed loads are doubtful because of the sparsity of water-quality data.

INTRODUCTION

The Dane County Regional Planning Commission (DCRPC) was required to develop the "Dane County Water Quality Plan" for compliance with Section 208 of the Federal Water Pollution Control Act amendments. To develop the water-quality plan, it was necessary to evaluate the effects of urban runoff on the water quality of the receiving lakes near Madison. The U.S. Geological Survey in cooperation with DCRPC established a monitoring network to assess quantity and quality of urban runoff and to provide data for calibration of an urban-runoff computer model.

The purpose of this study was to simulate various storm-water-management options to determine their effects on storm-runoff quantity and quality using an urban storm-water-runoff computer model. Methods for reduction of peak discharge and runoff volume were evaluated using the model. The model also was used to compute quality of urban runoff. The observed and simulated storm-runoff quality were compared to see if further simulations would be feasible.

The urban-runoff monitoring stations were all within Madison (figs. 1-5). Flow data were collected by the U.S. Geological Survey. The DCRPC provided the water-quality data and information on chemical composition and loading rates of materials swept from streets.

For readers who prefer SI metric units, data in this report may be converted by the following factors:

Multiply	By	To obtain
inch (in.)	25.40	millimeter (mm)
foot (ft)	304.8	millimeter (mm)
foot (ft)	0.3048	meter (m)
foot per second (ft/s)	0.3048	meter per second (m/s)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)
acre	0.4047	hectare (ha)
acre-foot (acre-ft)	1,233.5	cubic meter (m^3)
cubic foot per second (ft^3/s)	0.0283	cubic meter per second (m^3/s)
pound (lb)	453.5	gram (g)
part per million (ppm)	1.000	milligram per liter (mg/L)

DESCRIPTION OF SIMULATION MODEL

For flow simulations, the model used in this study was the Illinois Urban Drainage Area Simulator (ILLUDAS) developed by the Illinois State Water Survey (Terstriep and Stall, 1974). "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." (Terstriep and Stall, p. 1, 1974). ILLUDAS also can apply different rainfall volumes to each subbasin if rainfall volume is not uniform throughout the basin. ILLUDAS also will simulate water storage when conduit capacity is exceeded. The volume stored is printed out so that detention-storage requirements in the basin can be evaluated.

For water-quality simulations, the model used in this study was the QUAL-ILLUDAS program, which is an ILLUDAS program with water-quality subroutines added by the engineering firm Howard, Needles, Tammen, and Bergendoff. The "QUAL" part of the model reportedly is nearly the same as the Storm Water Management Model (Terstriep, written commun., April 1977) developed by the U.S. Environmental Protection Agency (1975).

STORM-RUNOFF DATA COLLECTION

Data used for computer-model calibration were collected from April through October 1976. All gaging stations were equipped with flow-control structures that were calibrated by current-meter measurements of discharge along with theoretical determinations. A continual record of stage and precipitation at 5-minute intervals was provided by digital recorders at each station. Additional rain gages, recording and nonrecording, within the study area (figs. 2-5) were used to evaluate time and space distribution of rainfall for storms used in the modeling study.

The DCRPC manually collected water samples for chemical analyses during some storm periods. Manual sampling was used throughout the data-collection period and later supplemented by automatic sampling. The samples were analyzed for ortho-phosphate, total phosphate, ammonia nitrogen, nitrate nitrogen, organic nitrogen, and total solids for use in the water-quality simulations.

Data on streets cleaned and cleaning dates were supplied by Madison for use as input for the quality simulations. Information on street-loading rates and chemical characteristics of street debris were provided by the Dane County Regional Planning Commission. The street debris was collected for analysis in late September 1976.

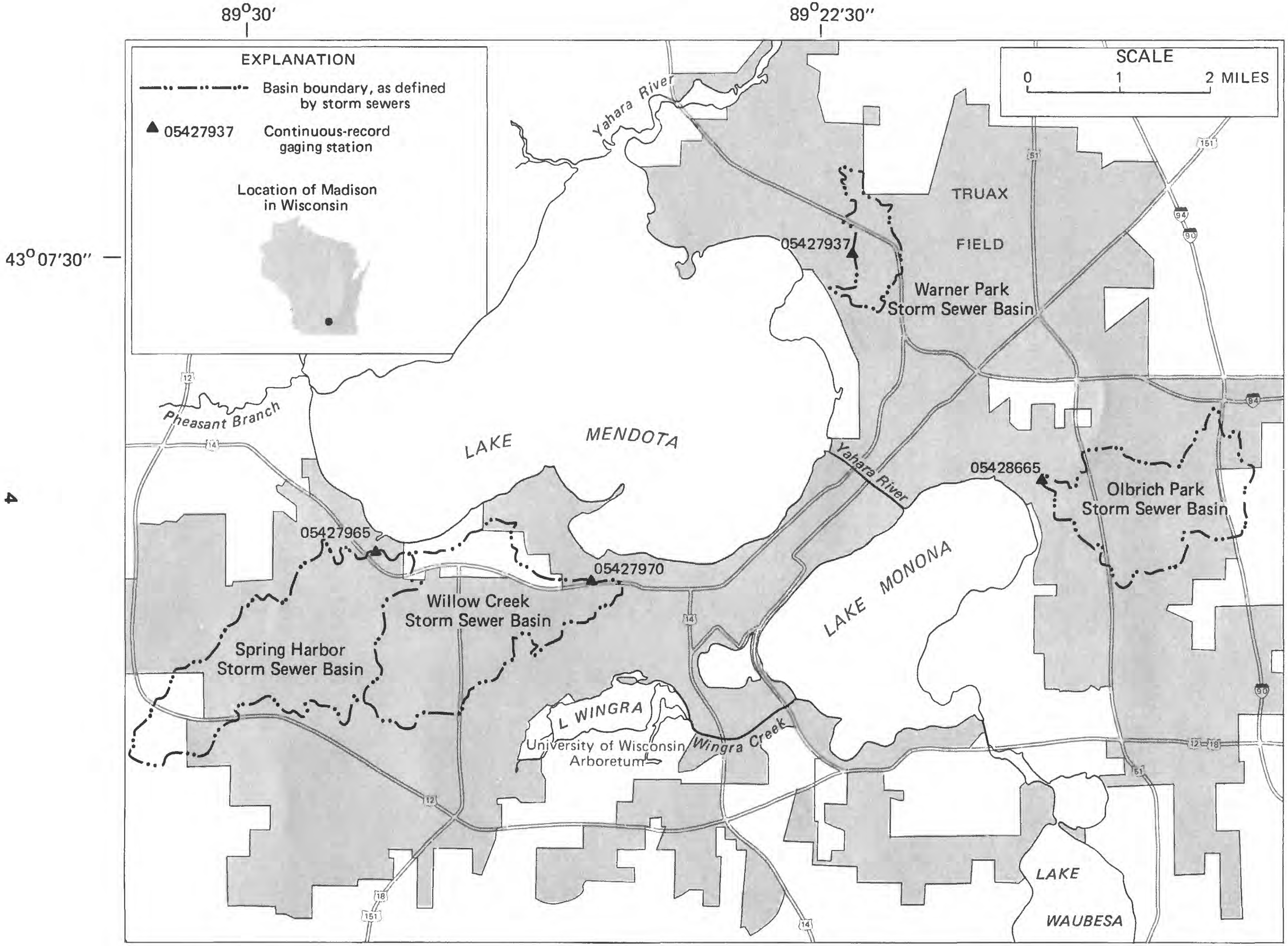
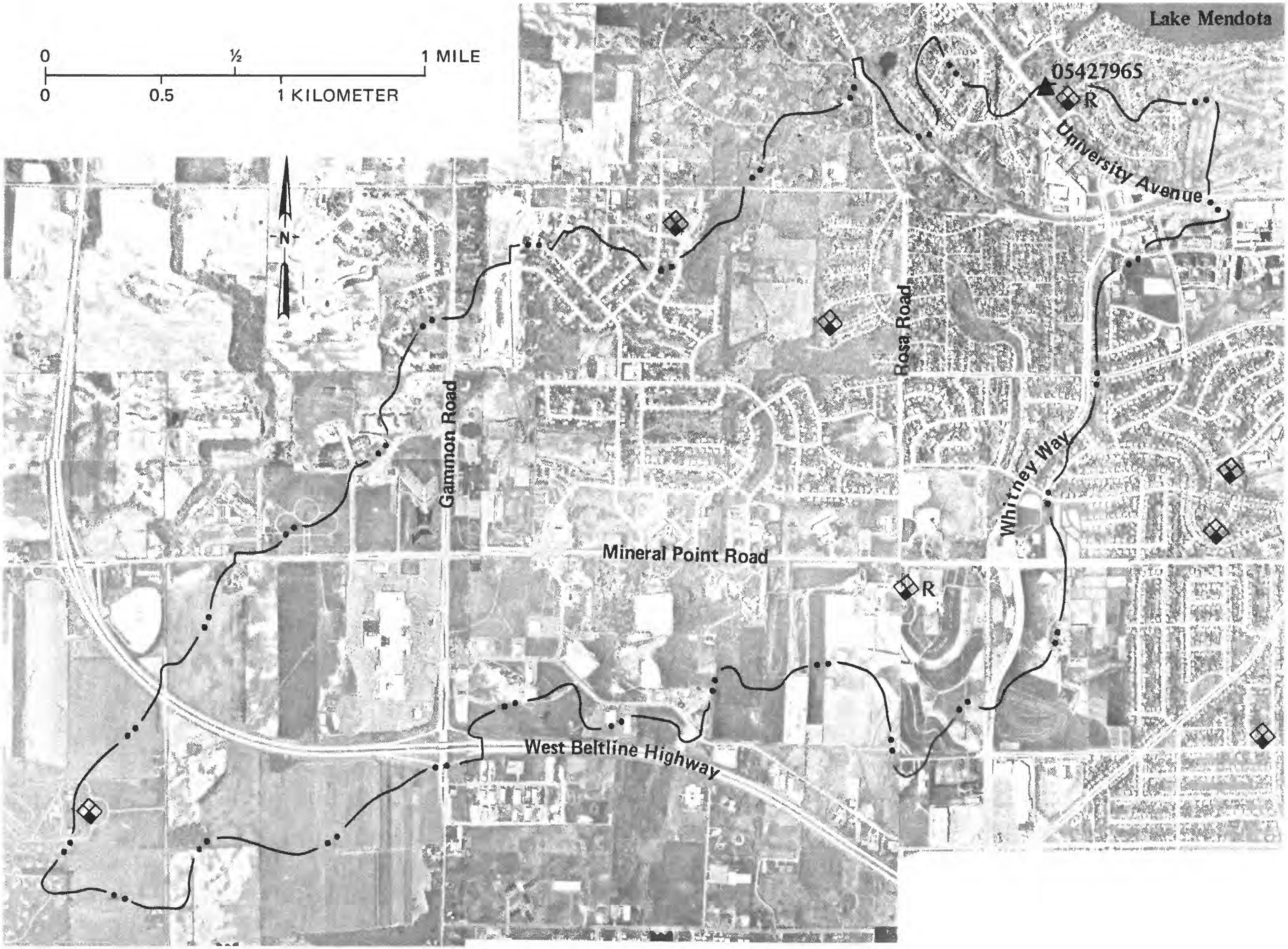


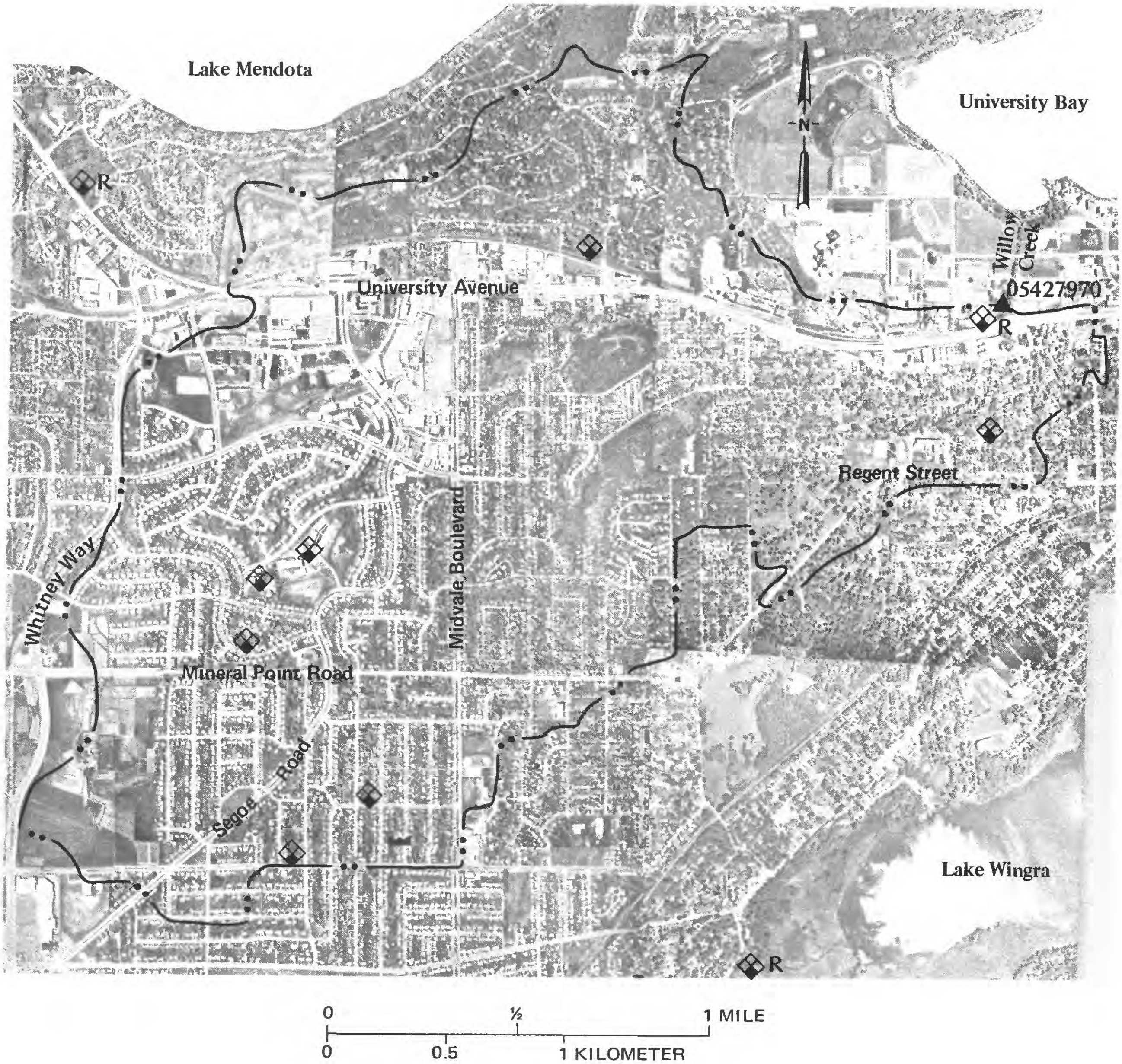
Figure 1. Location of storm-sewer basins and gaging stations used in study.



EXPLANATION

—— •• ——	Basin boundary, as defined by storm sewers
▲ 05427965	Continuous record gaging station and station number
◈	Precipitation gage
	R *indicates precipitation gage equipped with a recorder*

Figure 2. Aerial view of Spring Harbor storm-sewer basin.



EXPLANATION

——•• ——	Basin boundary, as defined by storm sewers
▲ 05427970	Continuous record gaging station and station number
◈	Precipitation gage
	R *indicates precipitation gage equipped with a recorder*

Figure 3. Aerial view of Willow Creek storm-sewer basin.

EXPLANATION

—••————••— Basin boundary, as defined by storm sewers

▲ 05428665 Continuous record gaging station and station number

◈ Precipitation gage

R *indicates precipitation gage equipped with a recorder*

Figure 4. Aerial view of Olbrich Park storm-sewer basin.

Figure 5. Aerial view of Warner Park storm-sewer basin.

MODEL CALIBRATION

ILLUDAS discharge calibration was made using data collected at the four urban gaging stations (figs. 2-5) from April through October 1976. A summary of the simulated and observed hydrologic data for each basin is presented in tables 1-4. The antecedent-moisture condition (AMC) code number presented in tables 1-4 is defined by Terstriep and Stall (1974) as follows:

AMC	Description	Total rainfall during 5 days preceding storm (in.)
1	Bone dry	0
2	Rather dry	0 to 0.5
3	Rather wet	0.5 to 1
4	Saturated	Over 1

The runoff ratio is the monitored runoff volume divided by the rainfall over the whole basin. Observed and simulated storm runoff for the four basins are compared graphically in figures 6-9. Comparisons between observed and simulated hydrographs for what are considered good and poor agreements for the Olbrich Park basin are presented in figures 10 and 11. Agreements were considered good if there was a relatively low percentage error between observed and simulated peak discharge and runoff volume, and if the shapes of the observed and simulated hydrographs were similar. The lack of simulated grassed runoff appears to cause the sharp recessions of the simulated hydrographs. Even though there were some poor agreements between observed and simulated hydrographs, the ILLUDAS models of each of the four basins were adequate for the purposes of this investigation because of generally good overall agreement between observed and simulated discharges.

Some of the large differences between simulated and observed flows may be partly attributed to unrecorded time or space variations in rainfall. The observed stage-discharge relations also may vary, especially at higher flows. Although the Willow Creek gaging station has been in operation since October 1973, the stage-discharge relation at higher flows has not been verified adequately. The other three gaging stations were installed just before this study began and have fairly well defined stage-discharge relationships except for the very high flows at the Spring Harbor gage. The high discharges at the Spring Harbor gage were determined by theoretical computations. The theoretical relationship agreed well with current-meter measurements made at low to medium-high flows.

Better model calibration may have been achieved if the stations had been operated for a longer time. This would have provided more storms for calibration with a wider variety of storm duration, intensity, and antecedent-moisture conditions, especially because precipitation from April through October 1976 was significantly below normal. Additional storms also would allow better definition of the stage-discharge relation.

Table 1.--Hydrologic data summary for Spring Harbor basin

Basin characteristics (acres)		Percent of total basin area
Total basin area	2,106	100.0
Total paved area	434	20.6
Directly connected paved area	329	15.6
Supplemental paved area	105	5.0
Contributing grassed area	830	39.4
Noncontributing area	408	19.4

Storm number	Date of storm	Antecedent-moisture condition (AMC)	Rainfall (in.)	Runoff ratio[1]	Peak discharge (ft^3/s)		Error (percent)	Runoff (in.)		Error (percent)	Simulated grassed runoff (percent of total runoff)
					Observed	Simulated		Observed	Simulated		
1	June 13, 1976	2	0.31	0.10	24.0	23	-4.2	0.03	0.02	-33.3	0
2	June 23, 1976	1	.63	.13	69.0	54	-21.7	.08	.07	-12.5	.2
3	July 28, 1976	2	.48	.10	58.0	56	-3.4	.05	.05	0	0
4	Aug. 13-14, 1976	1	1.11	.11	144.0	241	67.4	.12	.15	25.0	1.7
5	Aug. 25, 1976	1	.70	.10	64.0	69	7.8	.07	.05	-28.6	0
6	Aug. 27-28, 1976	3	.45	.13	47.0	35	-25.5	.06	.03	-50.0	1.3
7	Sept. 19, 1976	1	.37	.05	11.0	20	81.8	.02	.03	50.0	0

[1]Monitored runoff volume divided by rainfall over whole basin.

Table 2.--Hydrologic data summary for Willow Creek basin

Basin characteristics (acres)		Percent of total basin area
Total basin area	2,021	100.0
Total paved area	593	29.3
Directly connected paved area	494	24.4
Supplemental paved area	99	4.9
Contributing grassed area	534	26.4
Noncontributing area	302	15.0

Storm number	Date of storm	Antecedent-moisture condition (AMC)	Rainfall (in.)	Runoff ratio[1]	Peak discharge (ft^3/s)		Error (percent)	Runoff (in.)		Error (percent)	Simulated grassed runoff (percent of total runoff)
					Observed	Simulated		Observed	Simulated		
1	June 12-13, 1976	1	0.22	0.14	53.0	81	52.8	0.03	0.03	0	0
2	June 13-14, 1976	3	.39	.13	36.0	87	142	.05	.07	40.0	0
3	June 23, 1976	2	.25	.08	28.0	95	239	.02	.04	50.0	0
4	June 23, 1976	3	.57	.21	182	163	-10.4	.12	.11	-8.3	0
5	July 30, 1976	3	.24	.13	39.0	50	28.2	.03	.03	0	0
6	Aug. 14, 1976	2	.82	.23	768.0	625	-18.6	.20	.20	0	.6
7	Aug. 25, 1976	1	.42	.14	132.0	151	14.4	.06	.08	33.3	0
8	Aug. 27-28, 1976	2	.51	.14	104.0	172	65.4	.07	.10	42.9	0
9	Sept. 19, 1976	1	.43	.09	21.0	54	157	.04	.08	100.0	0

[1]Monitored runoff volume divided by rainfall over whole basin.

Table 3.--Hydrologic data summary for Olbrich Park basin

Basin characteristics (acres)		Percent of total basin area
Total basin area	1,509	100.0
Total paved area	259	17.2
Directly connected paved area	136	9.0
Supplemental paved area	123	8.2
Contributing grassed area	568	37.6
Noncontributing area	423	28.0

Storm number	Date of storm	Antecedent-moisture condition (AMC)	Rainfall (in.)	Runoff ratio[1]	Peak discharge (ft^3/s)		Error (percent)	Runoff (in.)		Error (percent)	Simulated grassed runoff (percent of total runoff)
					Observed	Simulated		Observed	Simulated		
1	May 15, 1976	2	0.92	0.10	49.0	40	-18.4	0.09	0.06	-33.3	0
2	June 13-14, 1976	1	.38	.03	2.8	24	757	.01	.02	100.0	0
3	June 23, 1976	2	.46	.07	59.0	43	-27.1	.03	.04	33.3	0
4	July 28, 1976	2	.54	.06	49.0	67	36.7	.03	.04	33.3	0
5	July 30, 1976	3	.43	.07	41.0	36	-12.2	.03	.02	-33.3	5.8
6	Aug. 14, 1976	1	.54	.07	101.0	124	22.8	.04	.07	75.0	2.4
7	Aug. 25, 1976	1	.38	.05	12.0	7	-41.7	.02	.01	-50.0	0
8	Aug. 27-28, 1976	2	.31	.06	10.0	15	50	.02	.02	0	0
9	Sept. 19, 1976	1	.37	.05	9.6	21	118	.02	.02	0	0
10	Oct. 5, 1976	1	.51	[2]	8.0	9	12.5	.03	----	-----	0

[1]Monitored runoff volume divided by rainfall over whole basin.

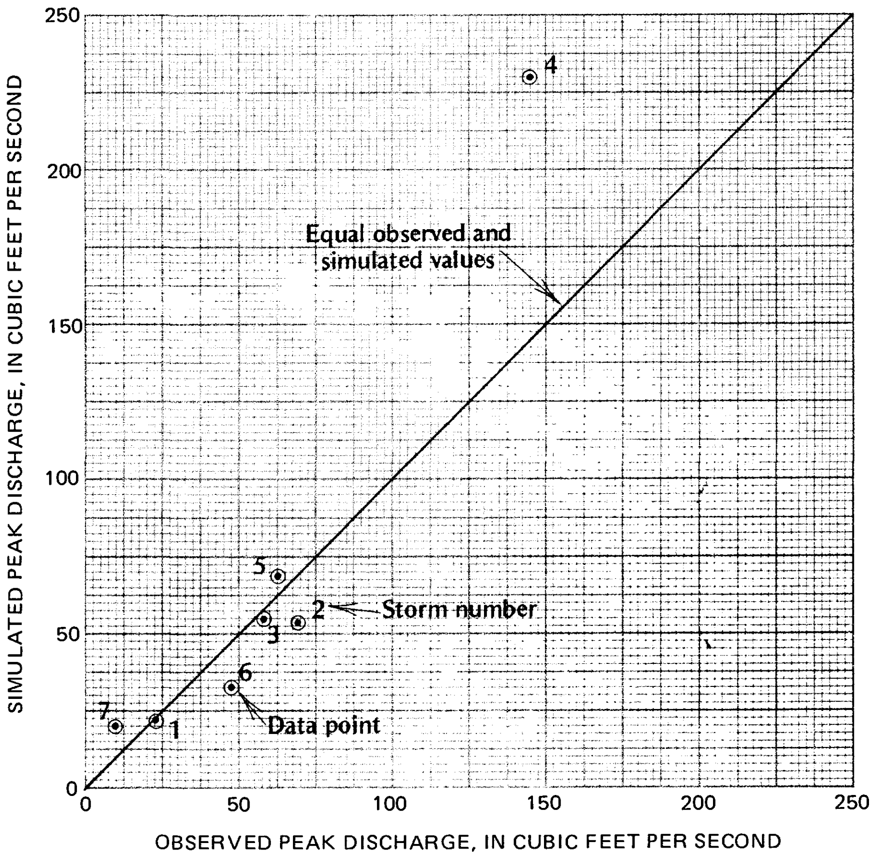
[2]Observed record incomplete.

Table 4.--Hydrologic data summary for Warner Park basin

Basin characteristics (acres)		Percent of total basin area
Total basin area	370	100.0
Total paved area	105	28.4
Directly connected paved area	95.1	25.7
Supplemental paved area	9.6	2.6
Contributing grassed area	104	28.1
Noncontributing area	56.3	15.2

Storm number	Date of storm	Antecedent-moisture condition (AMC)	Rainfall (in.)	Runoff ratio[1]	Peak discharge (ft^3/s)		Error (percent)	Runoff (in.)		Error (percent)	Simulated grassed runoff (percent of total runoff)
					Observed	Simulated		Observed	Simulated		
1	June 13-14, 1976	1	0.33	0.18	16.0	19	18.8	0.06	0.06	0	0
2	June 23, 1976	1	.20	.20	19.0	9	-52.6	.04	.03	-25.0	0
3	July 28, 1976	1	.53	.19	55.0	71	29.1	.10	.11	10.0	0
4	July 28, 1976	3	.30	.33	91.0	43	-52.7	.10	.05	-50.0	0
5	July 30, 1976	3	.25	.24	47.0	15	-68.1	.06	.04	-33.3	0
6	Aug. 14, 1976	1	.88	.23	175.0	149	-14.9	.20	.20	0	0
7	Aug. 25, 1976	1	.50	.22	50.0	29	-42.0	.11	.10	-9.0	0
8	Aug. 27-28, 1976	3	.35	.26	56.0	22	-60.7	.09	.06	-33.3	0
9	Sept. 19, 1976	1	.38	.18	26.0	29	10.3	.07	.07	0	0

[1]Monitored runoff volume divided by rainfall over whole basin.



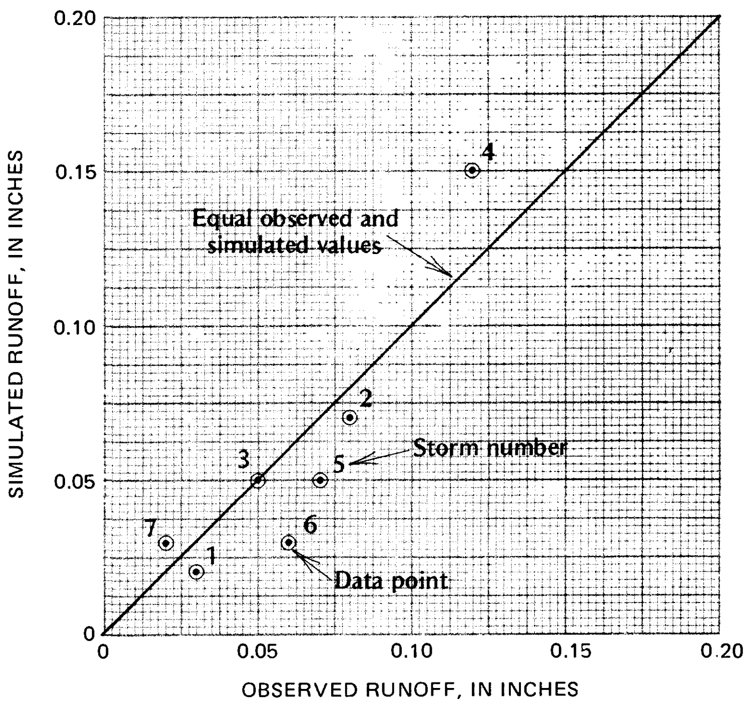
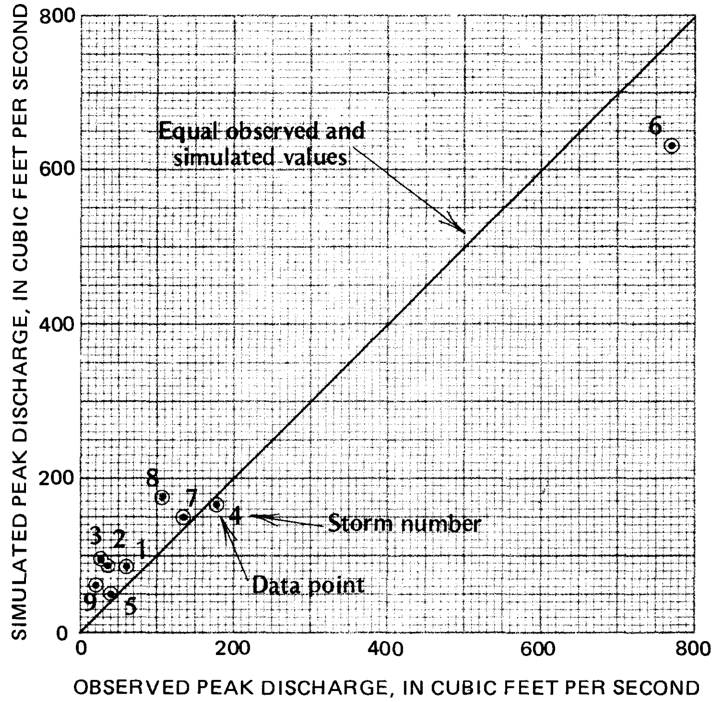


Figure 6. Observed and simulated runoff and peak discharges for Spring Harbor basin.



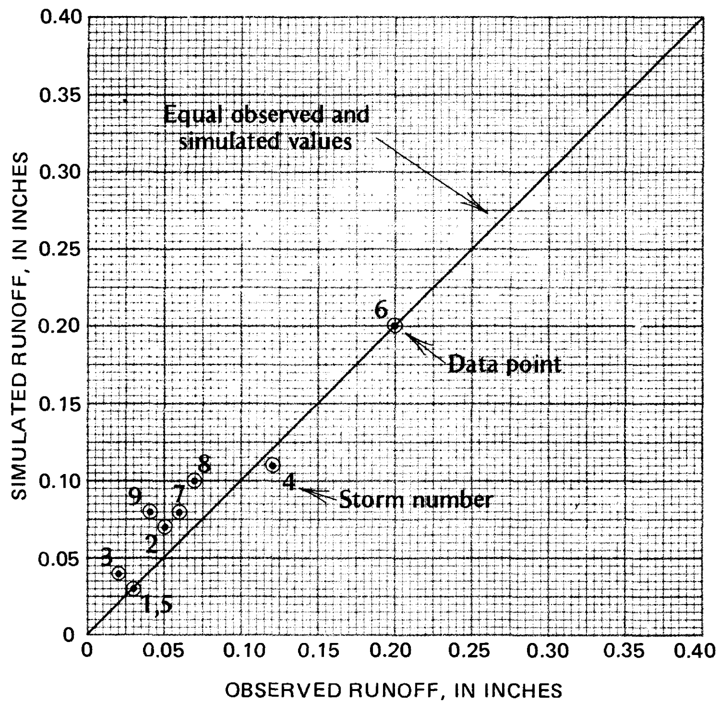
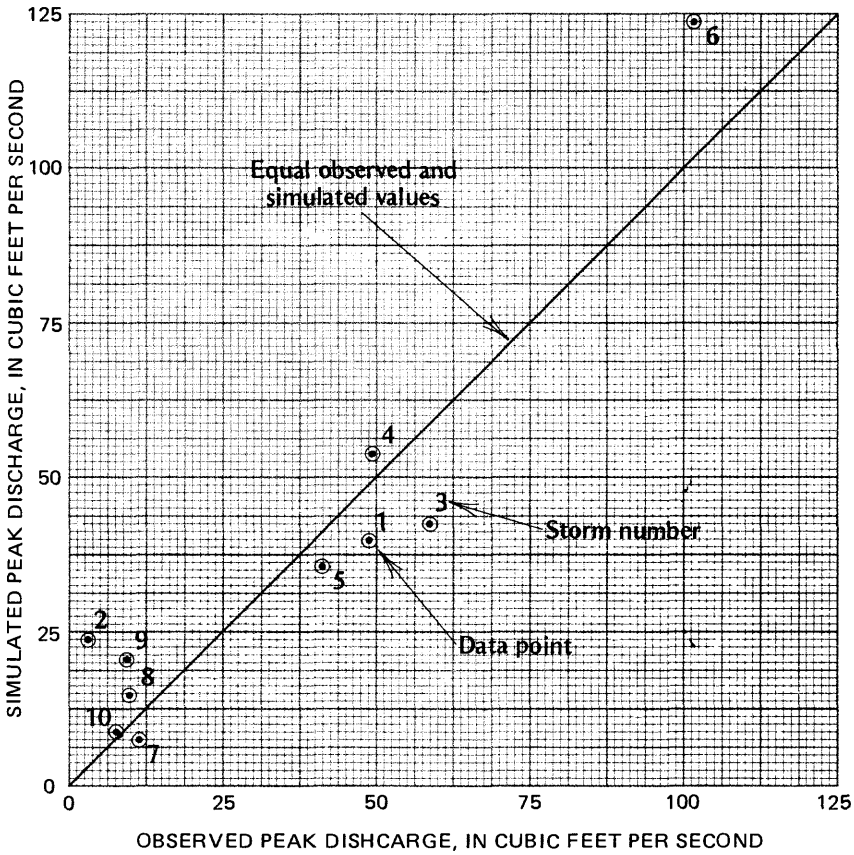


Figure 7. Observed and simulated runoff and peak discharges for Willow Creek basin.



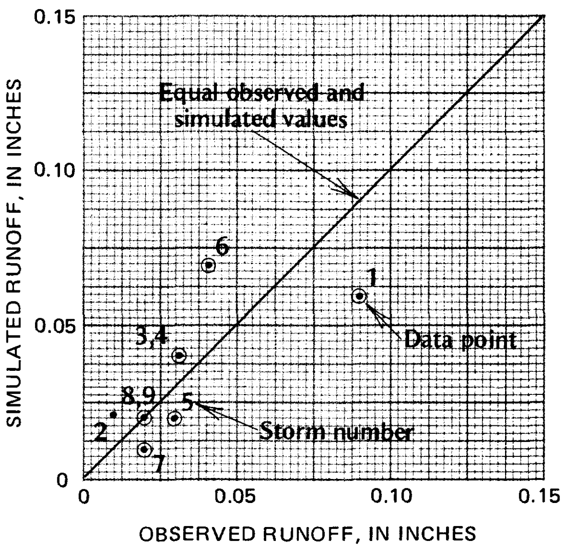
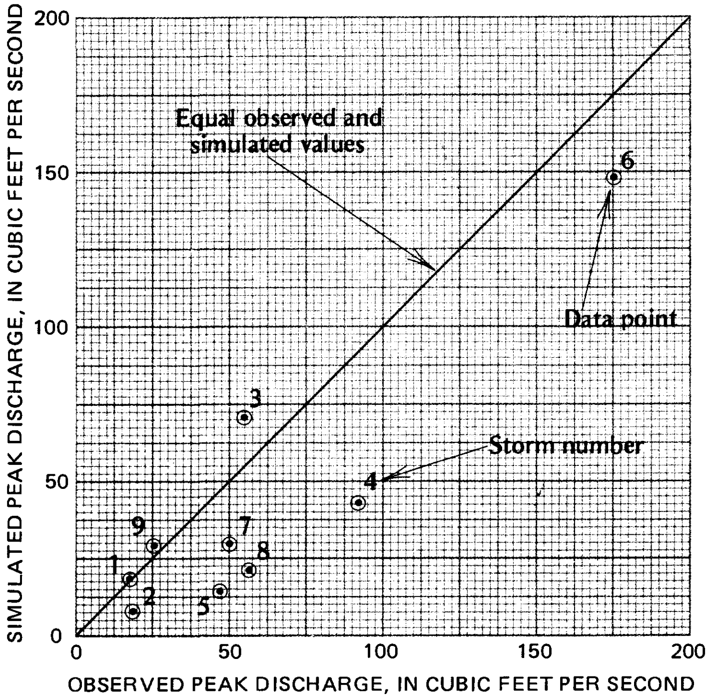


Figure 8. Observed and simulated runoff and peak discharges for Olbrich Park basin.



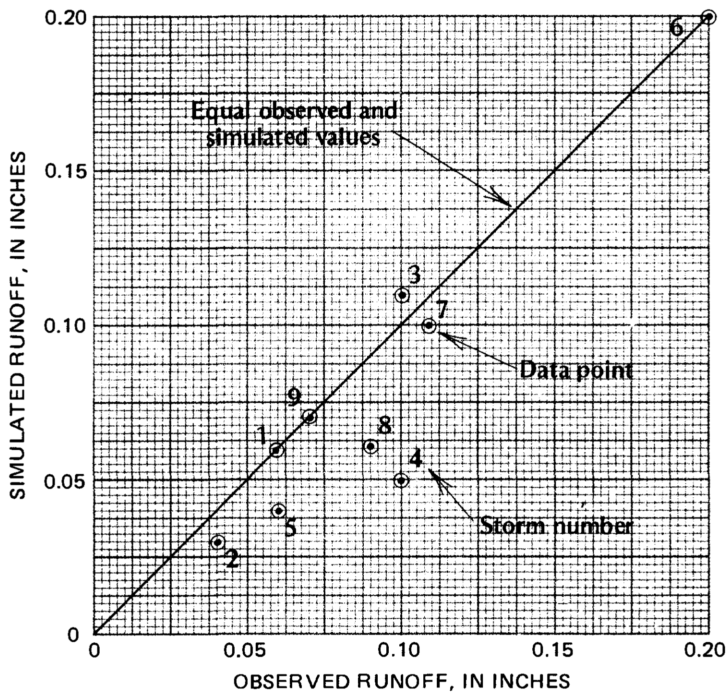


Figure 9. Observed and simulated runoff and peak discharges for Warner Park basin.

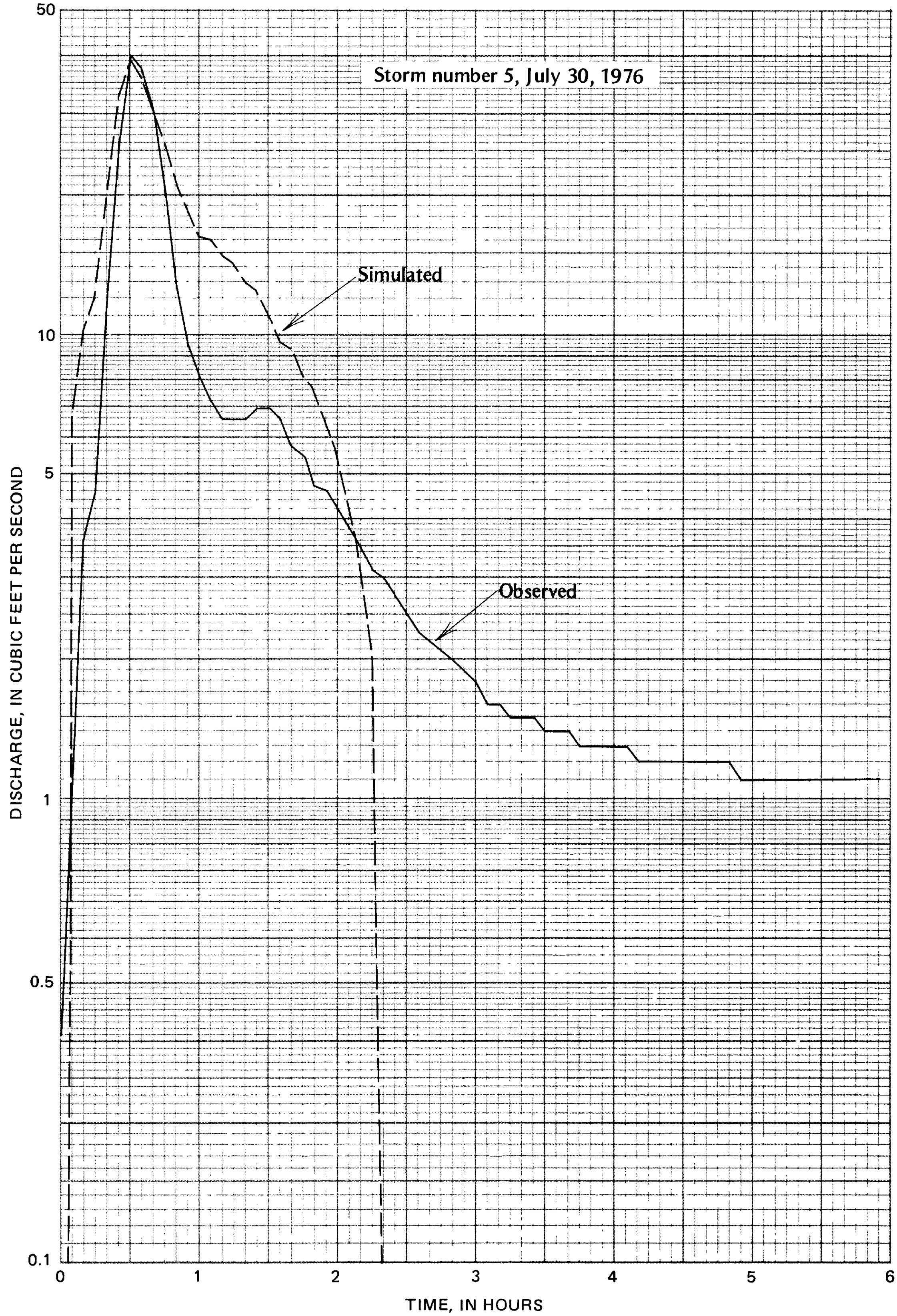


Figure 10. Comparison of observed and simulated hydrographs showing good agreement for Warner Park basin.

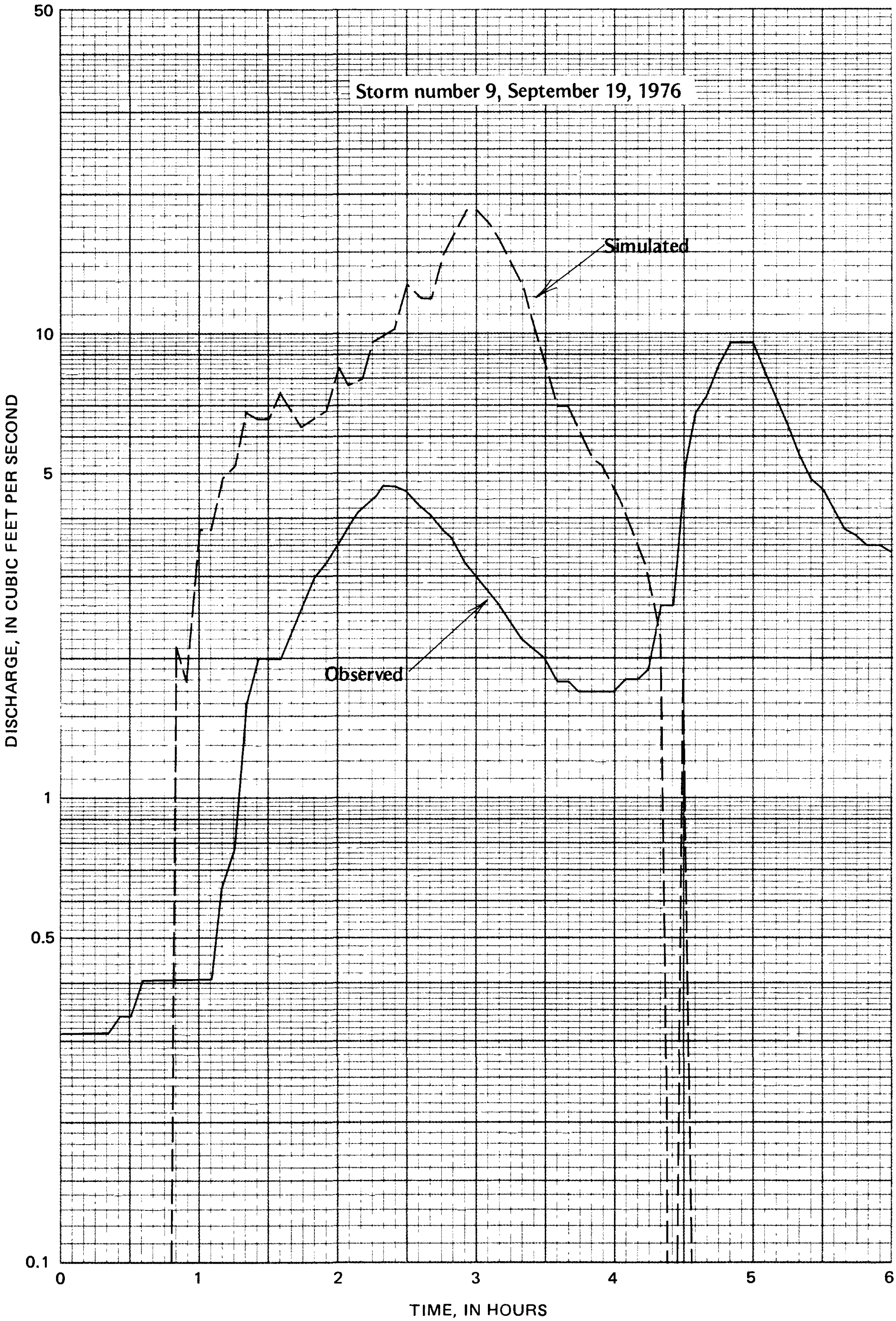


Figure 11. Comparison of observed and simulated hydrographs showing poor agreement for Olbrich Park basin.

A brief evaluation was made of the QUAL-ILLUDAS program's accuracy of simulating storm-runoff quality. One storm was simulated using the Spring Harbor basin model. Table 5 summarizes the results comparing simulated and observed water-quality constituents for the storm of June 23, 1976. Attempts were not made to adjust the water-quality models for a better agreement with observed data.

TYPICAL RAINFALL DETERMINATION

An identical precipitation pattern for all basins was necessary to compare the effects that physical changes to the storm-sewer systems would have on peak discharges and runoff volumes. A "typical" or average year of rainfall volume and storm distribution needed to be determined for making model projections. Runoff volume for the "typical" year would be computed for a range of physical conditions in each basin. Large storms during the year would be used in peak-discharge attenuation studies.

Precipitation records for calendar years 1940-75 for Truax Field in Madison were analyzed to find a "typical" year of rainfall. The mean annual precipitation for this period was 30.6 in. During this period there were 10 "near normal" years when annual precipitation was within 7.5 percent of the mean annual.

Records for the 10 "near normal" years were compared on a seasonal basis. Mean precipitation for 1940-75 for the seasons March-May, June-August, and September-November was compared to that for the 10 "near normal" years to reduce the field of typical year candidates. Calendar years 1968 and 1972 had the closest to normal seasonal precipitation.

Table 5.--Ratio of simulated to observed water-quality constituents for storm of June 23, 1976, Spring Harbor basin[1]

Run	Ortho-phosphate	Total phosphate	Nitrate nitrogen	Ammonia nitrogen	Organic nitrogen	Total solids
A[2]	11	1.8	0.66	9.2	1.2	1.5
B[3]	8.7	1.4	.50	7.0	.90	1.2

[1]Used street-loading rates for period September 29-October 4 determined by Dane County Regional Planning Commission.

[2]Used street-sweeping dates supplied by the city of Madison. These data indicated most streets had not been swept since last significant rainfall (38 days).

[3]Assumed all streets in basin had been swept 7 days before storm.

To determine which of these 2 years was more "typical", the number of storms per season and the average time between storms was compared to the normal for the period 1940-75. A storm was defined as any rainfall of 0.2 in. or more in a 24-hour period. Data from this comparison are presented in table 6. These data indicate that both years are roughly similar from June-November; March-May 1972 is normal.

Based on the preceding analyses, calendar year 1972 was the most "near normal" year for the period 1940-75. Five-minute rainfall data for calendar year 1972 were available from a U.S. Geological Survey rain gage located near the west end of Lake Wingra (fig. 3) and were used in the model projections made in this study.

Table 6.--Number of storms and average time between storms for calendar years 1968 and 1972

	1968	1972	Normal	Percent different than normal	
				1968	1972
			March-May		
Number of storms	10	13	13.0	-23.1	0.0
Number of days between storms	9.2	7.1	7.1	29.6	0.0
			June-August		
Number of storms	13	13	14.1	-7.8	-7.8
Number of days between storms	7.1	7.1	6.5	9.2	9.2
			September-November		
Number of storms	9	13	10.8	-16.7	20.3
Number of days between storms	10.1	7.0	8.4	20.2	-16.7
	Average percent difference			1.9	0.8

MODEL PROJECTIONS

The purpose of model projections is to analyze the effects that (1) physical changes to storm-sewer conduits, and (2) increased runoff detention and infiltration would have on peak discharge or runoff-volume attenuation. The purposes of peak-discharge and runoff-volume attenuation are (1) to minimize the treatment-flow capacities and the costs of hypothetical storm-runoff treatment facilities; (2) to induce sedimentation in detention areas rather than in Lakes Mendota and Monona; and (3) to induce infiltration of rainfall where natural removal of contaminants might occur along with reduction of runoff volume to treatment facilities.

The storm of August 23, 1972, was used in the simulations because it produced the highest peak discharge for this "typical" year (in the Willow Creek model). The storm yielded 1.0 in. of rainfall and nearly had the intensity of a 2-year, 30-minute rainfall (U.S. Weather Bureau, 1961) during the first half hour of the storm. It was assumed there were no time and space distribution differences in rainfall over the basins.

A range of physical changes in the storm-drainage systems was analyzed for determination of effects on peak discharges and storage requirements for hypothetical storm-water treatment facilities. The intent of this analysis was to determine if it might be feasible to treat storm water for the more common storms, allowing some untreated storm water to bypass the plant during larger, less frequent events. Sensitivity of this range in physical changes to a range of storm types was not evaluated. This type of analysis would be required in a more detailed storm-water management study to determine frequency of bypassing and effects of bypassed storm water on the receiving lakes.

PEAK-DISCHARGE ATTENUATION

Simulated peak discharges in the four basins were reduced by the following changes in each model:

1. Reducing storm-sewer slope (table 7);
2. increasing storm-sewer roughness from "n" = 0.013 to "n" = 0.040 (table 8);
3. reducing storm-sewer slope and increasing roughness from "n" = 0.013 to "n" = 0.040 (table 9); and
4. detaining 10 percent of runoff from each subbasin (table 10).

Some notable results from the above simulations were that major reductions of storm-sewer slope in the model yielded minor reductions in peak discharge (for example, 25 percent slope reduction yielded about 3 percent peak-discharge reduction). However, increasing storm-sewer roughness by increasing Manning's "n" from 0.013 to 0.040 decreased peak discharge about 10 to 20 percent. Increasing storm-sewer roughness from "n" = 0.013 to "n" = 0.040 and also reducing storm-sewer slope by 25 percent resulted in peak-discharge reductions of about 20 to 60 percent. Detention of 10 percent of runoff from each subbasin yielded peak-discharge reductions of about 10 to 20 percent.

Peak discharges also can be reduced by induced infiltration of rainfall. Major reductions in peak discharge were found by infiltrating all parking-lot runoff (5 to 24 percent reduction), by substituting porous pavement for conventional pavement (71 to 88 percent reduction), and by draining 90 percent of residential rooftops onto lawns instead of driveways (7 to 31 percent reduction). Runoff-volume reduction was similarly reduced for the induced infiltration runs.

The errors in percent associated with observed versus simulated peak discharges were in most instances many times the reduction in percent attributed to effects of reduction in slope and increase in roughness. The models, however, were assumed to accurately simulate the basins for the purposes of future storm-water management planning and to reveal in general the relative effects of certain types of alteration on storm-water runoff. Further calibration and more detailed study would be required before any of these alternatives could be implemented.

Storm-sewer slope and roughness changes in the models were made only on the largest buried conduits and on the largest open channels in the basins. The buried conduits were conceivably large enough to allow construction activities and were usually in the downstream part of the basin. Existing conduit slopes were obtained from maps and data supplied by the city of Madison and the DCRPC. These storm-sewer slopes were then reduced in the model by 10, 25, and 50 percent. Conduit slopes could effectively be reduced by construction of a series of small check dams throughout existing conduits.

A Manning's "n" of 0.013 was used for all existing concrete pipes and box culverts while the "n" for the open channels ranged from 0.018 to 0.090 and was estimated based on field observations. The existing pipes and box culverts were assumed to be free of rocks and debris and in good condition.

Table 7.--Summary of peak-discharge reduction by reducing storm-sewer slope for storm of August 23, 1972

Percent slope reduction	Percent peak-discharge reduction			
	Olbrich Park	Spring Harbor	Warner Park	Willow Creek
10	0	2	1	2
25	1	3	3	3
50	4	10	8	15

Roughness of the storm sewers could be increased from 0.013 to the model simulation value of 0.040 by lining the conduit bottoms with the appropriate rock. Such changes would increase channel storage and increase time of concentration. However, storm-sewer flow capacities would be reduced and that could cause flooding problems during large storms.

Detention of runoff in each basin could be achieved by construction of small detention areas in each subbasin or by parking-lot or rooftop storage.

Table 8.--Summary of peak-discharge reduction for storm of August 23, 1972, by increasing storm-sewer roughness from "n" = 0.013 to "n" = 0.040

Effect	Percent peak-discharge reduction			
	Olbrich Park	Spring Harbor	Warner Park	Willow Creek
In downstream branch[1]	13	23	16	42
In open conduits only	5	11	16	--

[1]Main storm-sewer branch in downstream part of basin that is conceivably large enough to allow construction activities. Can include large pipes, box culverts, or open conduits.

Table 9.--Summary of peak-discharge reduction for storm of August 23, 1972, by reducing storm-sewer slope and increasing roughness from "n" = 0.013 to "n" = 0.040

Percent slope reduction	Percent peak-discharge reduction			
	Olbrich Park	Spring Harbor	Warner Park	Willow Creek
10	15	26	17	62
25	18	31	18	65
50	25	40	23	72

Table 10.--Summary of peak-discharge reduction for storm of August 23, 1972, by detaining 10 percent of runoff in each subbasin

Effect	Storm-sewer basin			
	Olbrich Park	Spring Harbor	Warner Park	Willow Creek
Percent peak-discharge reduction	11	12	21	14

RUNOFF-VOLUME ATTENUATION

Simulated runoff volumes and peak discharges in the four modeled basins were reduced by the following changes in each model:

(1) infiltration of parking-lot runoff (table 11),
(2) substitution of porous pavement (table 12), and
(3) infiltration of residential rooftop runoff (table 13).

Infiltration of parking-lot runoff was simulated by replacing parking-lot paved area in the model with grassed area. In reality, parking-lot runoff can be intercepted and infiltrated by strategically located planting strips in and around parking lots (Aron and Borrelli, 1975). Estimating infiltration using porous pavement also was done by replacing street areas with grassed area in the model. This should roughly indicate the value of porous pavement assuming the infiltration rates of each are equal. Infiltration of residential rooftop runoff was simulated assuming 90 percent of the downspouts drained onto grassed area and not onto driveways. In the model, this was done by removal of 90 percent of the rooftop areas which previously were assumed to drain onto driveways and into the streets.

Standard infiltration curves in the ILLUDAS program are used to compute grassed-area runoff. The infiltration curve used depends upon the antecedent-moisture condition (p. 9) and the hydrologic soil group. Soils in the Madison area are in hydrologic soil group B which have moderate infiltration rates and are moderately well drained (Terstriep and Stall, p. 9, 1974). The computation of grassed-area runoff is highly sensitive to the hydrologic soil group.

TREATMENT-PLANT STORAGE REQUIREMENTS

Using the "typical" storm of August 23, 1972, storage requirements were computed by ILLUDAS for a range of treatment capacities at hypothetical storm-water-treatment plants at the downstream end of each monitored basin (table 14). Time for all runoff to pass through the treatment plants from the start of the storm also was computed because time between storms needs

Table 11.--Summary of runoff volume and peak-discharge reduction for storm of August 23, 1972, by infiltration of parking-lot runoff

Effect	Storm-sewer basin			
	Olbrich Park	Spring Harbor	Warner Park	Willow Creek
Percent runoff volume reduction	12	26	21	20
Percent peak-discharge reduction	5	16	24	23

Table 12.--Summary of runoff volume and peak-discharge reduction for storm of August 23, 1972, by use of porous pavement on streets and parking lots

Effect	Storm-sewer basin			
	Olbrich Park	Spring Harbor	Warner Park	Willow Creek
Percent runoff volume reduction	75	50	74	78
Percent peak-discharge reduction	71	88	74	79

Table 13.--Summary of runoff volume and peak-discharge reduction for storm of August 23, 1972, by infiltration of residential rooftop runoff

Effect	Storm-sewer basin			
	Olbrich Park	Spring Harbor	Warner Park	Willow Creek
Percent runoff volume reduction	--	11	7	22
Percent peak-discharge reduction	--	31	7	26

Table 14.--Required storage volume and holding time for hypothetical storm-water-treatment plants for storm of August 23, 1972

Treatment-plant capacity (ft^3/s)	Required storage volume (acre-ft)/ holding time (hours)			
	Olbrich Park	Spring Harbor	Warner Park	Willow Creek
5	8.0/22	13.7/40	5.8/17	34.1/42
10	7.2/11	11.7/30	4.8/8.8	32.7/42
25	5.3/4.7	7.9/26	2.6/3.8	28.8/18
50	3.2/2.6	4.0/26	1.4/3.5	23.6/9.3
100	.3/2.6	.6/26	.2/3.5	14.9/5.0

to be considered when determining storage and flow-capacity requirements. A plant with low treatment capacity may have to bypass runoff from a second storm if it occurs soon after a first storm.

Storage requirements for hypothetical treatment plants ranged from 2.6 to 29 acre-feet for the smallest and largest basins, respectively, for a treatment capacity of 25 ft^3/s.

SUMMARY AND CONCLUSIONS

Four urbanized drainage basins in Madison, Wis., were monitored for flow and water quality as part of a 208 water-quality planning study conducted by the Dane County Regional Planning Commission. The flow data were used by the U.S. Geological Survey to calibrate an urban runoff computer model which was then used to simulate hypothetical changes on the physical drainage systems for storm-water management.

The model used was the Illinois Urban Drainage Area Simulator (ILLUDAS) developed by Terstriep and Stall (1974) of the Illinois State Water Survey. A cursory evaluation was made of a version of ILLUDAS modified by the consulting firm Howard, Needles, Tammen, and Bergendoff to simulate quality of urban runoff (QUAL-ILLUDAS) using data from only one storm.

The ILLUDAS models were calibrated accurately enough for the purposes of this investigation. A design storm was developed for use in modeling hypothetical situations to evaluate resultant effects on urban runoff by (1) reducing effective storm-sewer slope, (2) increasing storm-sewer roughness,

(3) combining "1" and "2", (4) using detention ponds, (5) infiltrating parking-lot runoff, (6) using porous pavement on streets, and (7) infiltrating residential rooftop runoff. ILLUDAS also was used to compute storage requirements for storm-water-treatment plants for a range in treatment flow capacities.

The results of these simulations indicate that the use of porous pavement on streets and parking lots may be the most effective alternative in reducing runoff volume and peak discharge. The other simulated alternatives also were relatively effective with reduction of storm-sewer slope and infiltration of residential rooftop runoff being the least effective alternatives. Some of these alternatives may not be economically feasible or even practical over an entire storm-sewer basin. However certain alternatives such as the use of porous pavement or detention ponds could be more easily implemented in newly developing areas than they could be in other established neighborhoods. This would tend to lessen the effects of further urbanization. Additional hypothetical changes or combinations of changes in the basins could have been modeled. Some alternatives may need to be evaluated in more detail. The purpose of this study, however, was to reveal in general what the relative effects of certain types of alterations would have on storm-water runoff.

The QUAL-ILLUDAS simmulations were not conclusive because only a cursory evaluation was made. One storm was modeled and yielded computed loads close to observed loads for nitrate and organic nitrogen, total phosphate, and total solids. Ammonia nitrogen and orthophosphate loads computed by the model were about 7 to 11 times greater than the observed loads. The observed loads are doubtful, however, because of a sparsity of water-quality data for the observed storm.

Because the design storm had no more than a 2-year, 30-minute rainfall intensity, additional historic and hypothetical storms need to be input to the models before a detailed storm-water management plan is developed. Fairly common magnitude storms could surcharge storm sewers in some locations causing flooding if existing conduit slope or roughness is altered, for example. A detailed modeling study would reveal locations where problems could exist and could reveal solutions to the problems. A solution for one location could cause problems at other locations, upstream or downstream. A detailed model study would show the effects changes in a basin would have in other parts of the basin and is thus a requirement for preparation of a good storm-water-management plan.

Additional water-quality monitoring will be required for calibration of a useful water-quality model. Because the monitored basins are so large, smaller areas within the basins also need to be monitored. Data from small basins with fairly homogeneous land use are necessary to quantify street-loading rates and chemical characteristics of street dirt. These characteristics could change seasonally and would have to be quantified seasonally. Effects of construction activities in the basins on sediment discharge in the storm sewers need to be evaluated to see if controls on construction procedures are necessary.

SELECTED REFERENCES

Aron, Gert, and Borrelli, John, 1975, Practical alternatives in storm runoff control, in National Symposium on Urban Hydrology and Sediment Control Proceedings: University of Kentucky, Bulletin UKY BU 109, p. 13-16.

Bradford, W. L., 1977, Urban stormwater pollutant loadings: a statistical summary through 1972: Journal of the Water Pollution Control Federation, v. 49, no. 4, p. 613-622.

Huber, W. C., Heaney, J. P., Medina, M. A., Peltz, W. A., Sheikh, Hasan, and Smith, G. F., 1975, Storm water management model, user's manual, version II: U.S. Environmental Protection Agency Report EPA-670/2-75-017, 350 p.

National Oceanic and Atmospheric Administration, 1975, Local climatological data, Madison, Wisconsin, annual summary, 1975: U.S. Department of Commerce, Environmental Data Service.

Terstriep, M. L., and Stall, J. B., 1974, The Illinois urban drainage area simulator, Illudas: Illinois State Water Survey Bulletin 58, 90 p.

U.S. Environmental Protection Agency, 1974, Water quality management planning for urban runoff: U.S. Environmental Protection Agency Report EPA 440/9-75-004, 226 p.

U.S. Weather Bureau, 1961, Rainfall frequency atlas of the United States: U.S. Weather Bureau Technical Paper No. 40, 61 p.

University of Kentucky, 1974, Proceedings of the National symposium on urban rainfall and runoff and sediment control: University of Kentucky, Bulletin UKY BU 106, 246 p.

_______1975, Proceedings of the National symposium on urban hydrology and sediment control: University of Kentucky, Bulletin UKY BU 109, 314 p.

_______1976, Proceedings of the National symposium on urban hydrology, hydraulics, and sediment control: University of Kentucky, Bulletin UKY BU 111, 385 p.