EPA-600/2-76-175a
August 1976

Environmental Protection Technology Series

ASSESSMENT OF MATHEMATICAL MODELS FOR STORM AND COMBINED SEWER MANAGEMENT

Municipal Environmental Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

EPA-600/2-76-175a
August 1976

ASSESSMENT OF MATHEMATICAL MODELS FOR STORM AND COMBINED SEWER MANAGEMENT

By

Albin Brandstetter
Battelle Pacific Northwest Laboratories
Water and Land Resources Department
Richland, Washington 99352

Contract No. 68-03-0251

Project Officer

Chi-Yuan Fan
Storm and Combined Sewer Section
Municipal Environmental Research Laboratory (Cincinnati)
Edison, New Jersey 08817

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

DISCLAIMER

The Municipal Environmental Research Laboratory has reviewed this report and approved its publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

The objective of the study described herein was the evaluation of comprehensive mathematical models for the nonsteady simulation of urban storm and combined wastewater flow and quality. It is hoped that the application of appropriate models will aid in the cost-effective design and improvement of urban wastewater collection and treatment systems.

Francis T. Mayo
Director
Municipal Environmental Research Laboratory

ABSTRACT

Mathematical models for the nonsteady simulation of urban runoff were evaluated to determine their suitability for the engineering assessment, planning, design and control of storm and combined sewerage systems. The models were evaluated on the basis of information published by the model builders and model users. Seven models were also tested by computer runs using both hypothetical and real catchment data. Most of the models evaluated include the nonsteady simulation of the rainfall-runoff process and flow routing in sewers; a few include also the simulation of wastewater quality, options for dimensioning sewerage system components, and features for real-time control of overflows during rainstorms.

This report was submitted in fulfillment of Contract No. 68-03-0251 by the Water and Land Resources Department, Battelle-Pacific Northwest Laboratories, under sponsorship of the Office of Research and Development, U. S. Environmental Protection Agency. Work was completed in August 1975.

CONTENTS

		Page
Abstract		iv
List of Figures		vi
List of Tables		xiv
Acknowledgments		xix
Sections		
I	Conclusions	1
II	Recommendations	36
III	Introduction	43
IV	Model Selection	45
V	Model Reviews	49
VI	Test Data	130
VII	Test Results	242
VIII	Costs for Computer Application	346
IX	Discussion	357
X	References	362
XI	Glossary	377
XII	Appendices	384

FIGURES (Continued)

No.		Page
	Runoff for Two-Hour Triangular Rainstorm - Small Hypothetical Catchment, 96 m (316 ft) Long	
31	0.1% Slope, 0% Imperviousness.	256
32	" " 50% Imperviousness.	257
33	" " 100% Imperviousness.	258
34	10% Slope, 0% Imperviousness.	259
35	" " 50% Imperviousness.	260
36	" " 100% Imperviousness.	261
	Runoff for Two-Hour Triangular Rainstorm - Large Hypothetical Catchment, 305 m (1000 ft) Long	
37	0.1% Slope, 0% Imperviousness	262
38	" " 50% Imperviousness	263
39	" " 100% Imperviousness	264
40	10% Slope, 0% Imperviousness	265
41	" " 50% Imperviousness	266
42	" " 100% Imperviousness	267
	Outflows for Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow	
43	0.05% Slope.	281
44	0.5% Slope	282
45	5% Slope	283
	Large Hypothetical Pipe	
46	0.05% Slope.	284
47	0.5% Slope	285
48	5% Slope	286

FIGURES (Continued)

No.		Page
	Outflow Concentrations for Constant Inflow Concentrations With Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow	
49	0.05% Slope	287
50	0.5% Slope	287
51	5% Slope	288
	Large Hypothetical Pipe	
52	0.05% Slope	288
53	0.5% Slope	289
54	5% Slope	289
	Outflow Concentrations for Triangular Inflow Concentrations With Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow	
55	0.05% Slope	290
56	0.5% Slope	290
57	5% Slope	291
	Large Hypothetical Pipe	
58	0.05% Slope	291
59	0.5% Slope	292
60	5% Slope	292
	Outflow Concentrations for Inverted Triangular Inflow Concentrations With Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow	
61	0.05% Slope	293
62	0.5% Slope	293

FIGURES (Continued)

No.		Page
63	5% Slope	294
	Large Hypothetical Pipe	
64	0.05% Slope.	294
65	0.5% Slope	295
66	5% Slope	295
	Comparison of Measured and Computed Runoff for the Oakdale Avenue Catchment	
	May 19, 1959 Storm	
67	EPA Stormwater Management Model.	305
68	Battelle Urban Wastewater Management Model	305
69	Chicago Flow Simulation Program.	306
70	Dorsch Hydrograph Volume Method.	306
	July 2, 1960 Storm	
71	EPA Stormwater Management Model.	307
72	Battelle Urban Wastewater Management Model	308
73	Chicago Flow Simulation Program.	309
74	Dorsch Hydrograph Method	310
	July 26, 1960 Storm	
75	EPA Stormwater Management Model.	311
76	Battelle Urban Wastewater Management Model	312
77	Chicago Flow Simulation Program.	314
78	Dorsch Hydrograph Method	315

FIGURES (Continued)

No.		Page
	August 2, 1963 Storm	
79	EPA Stormwater Management Model.	316
80	Battelle Urban Wastewater Management Model	317
81	Chicago Flow Simulation Program.	317
82	Dorsch Hydrograph Volume Method.	318
	Comparison of Measured and Computed Runoff for Bloody Run Catchment Storm of November 9, 1970	
	Bank #1 Station	
83	EPA Stormwater Management Model.	325
84	Battelle Urban Wastewater Management Model	326
85	Chicago Flow Simulation Program.	326
	Bank #2 Station	
86	EPA Stormwater Management Model.	327
87	Battelle Urban Wastewater Management Model	327
88	Chicago Flow Simulation Program.	328
	Longview #1 Station	
89	EPA Stormwater Management Model.	328
90	Battelle Urban Wastewater Management Model	329
91	Chicago Flow Simulation Program.	329
	Longview #2 Station	
92	EPA Stormwater Management Model.	330
93	Battelle Urban Wastewater Management Model	330
94	Chicago Flow Simulation Program.	331

TABLES

No.		Page
1	Comparison of Major Model Categories	6
2	Comparison of Hydrologic Features of Models	7
3	Comparison of Hydraulic Features of Models	11
4	Comparison of Water Quality Features of Models	15
5	Comparison of Miscellaneous Features of Models	19
6	Hypothetical Subcatchment Dimensions	133
7	Hypothetical Subcatchment Characteristics	134
8	Infiltration Rates and Retention Storage Capacities of Hypothetical Subcatchments	135
9	Hypothetical Catchment Data Combinations	137
10	Summary of Rainstorms for Hypothetical Catchments	140
11	Two-Hour Constant Rainstorm for Hypothetical Catchments	142
12	One-Hour Triangular Rainstorm for Hypothetical Catchments	142
13	Two-Hour Triangular Rainstorm for Hypothetical Catchments	143
14	Four-Hour Triangular Rainstorm for Hypothetical Catchments	144
15	Hypothetical Pipe Dimensions	145
16	Hypothetical Pipe Boundary Conditions	146

TABLES (Continued)

No.		Page
17	Hypothetical Pipe Capacities	151
18	Hypothetical Pipe Data Combinations	152
19	Inflow Hydrograph Shapes for Hypothetical Pipes	153
20	Inflow Concentration Shapes for Hypothetical Pipes	155
21	Continuous Inflow for Hypothetical Pipes	156
22	One-Hour Triangular Inflow for Hypothetical Pipes	157
23	Two-Hour Triangular Inflow for Hypothetical Pipes	158
24	Four-Hour Triangular Inflow for Hypothetical Pipes	159
25	Inflow Concentrations for Continuous and Two-Hour Triangular Inflow Hydrographs for Hypothetical Pipes	161
26	Inflow Concentrations for One-Hour Triangular Inflow Hydrographs for Hypothetical Pipes	162
27	Inflow Concentrations for Four-Hour Triangular Inflow Hydrographs for Hypothetical Pipes	163
28	Breakdown of Pervious and Impervious Surfaces in the Oakdale Avenue Catchment	166
29	Physical Characteristics of Subcatchments of the Oakdale Avenue Catchment	167
30	Estimate of Soil Characteristics for the Oakdale Avenue Catchment	170
31	Physical Characteristics of the Sewer Elements of the Oakdale Avenue Catchment	171

TABLES (Continued)

No.		Page
32	Reliable Rainfall and Runoff Records for the Oakdale Avenue Catchment	172
	Rainfall and Runoff Data for Oakdale Avenue Catchment	
33	Storm of May 19, 1959	177
34	Storm of July 2, 1960	180
35	Storm of July 26, 1960	185
36	Storm of August 2, 1963	191
37	Division of Bloody Run Catchment into Different Land Uses	195
38	Physical Characteristics of Subcatchments of the Bloody Run Catchment	198
39	Estimate of Soil Characteristics for the Bloody Run Catchment	199
40	Physical Characteristics of the Sewer Elements of the Bloody Run Catchment	202
41	Summary of Available Runoff Records for Rainstorms of Bloody Run Catchment Selected for Model Testing	205
	Storm of November 9, 1970 - Bloody Run Catchment	
42	Rainfall Data	218
43	Runoff Data	219
44	Suspended Solids Concentrations	220
45	Biochemical Oxygen Demand Concentrations	220
	Storm of November 14, 1970 - Bloody Run Catchment	
46	Rainfall Data	221

TABLES (Continued)

No.		Page
47	Runoff Data.	225
48	Suspended Solids Concentrations.	227
49	Biochemical Oxygen Demand Concentrations	227
	Storm of May 13, 1971 - Bloody Run Catchment	
50	Rainfall Data.	228
51	Runoff Data.	231
52	Suspended Solids Concentrations.	233
53	Biochemical Oxygen Demand Concentrations	233
	Storm of August 25, 1971 - Bloody Run Catchment	
54	Rainfall Data.	234
55	Runoff Data.	238
56	Suspended Solids Concentrations.	240
57	Biochemical Oxygen Demand Concentrations	241
	Peak Runoff From Hypothetical Catchments	
58	Two-Hour Constant Rainstorm.	245
59	One-Hour Triangular Rainstorm.	247
60	Two-Hour Triangular Rainstorm.	249
61	Four-Hour Triangular Rainstorm	251
	Peak Runoff Times From Hypothetical Catchments	
62	One-Hour Triangular Rainstorm.	253
63	Two-Hour Triangular Rainstorm.	254
64	Four-Hour Triangular Rainstorm	255
65	Peak Outflows From Hypothetical Pipes.	273

TABLES (Continued)

No.		Page
66	Peak Weir Overflows From Hypothetical Pipes.	275
67	Times of Peak Outflows From Hypothetical Pipes . . .	277
68	Extreme Outflow Concentrations of Small Hypothetical Pipes	278
69	Extreme Outflow Concentrations of Large Hypothetical Pipes	279
70	Times of Extreme Outflow Concentrations of Small Hypothetical Pipes	280
71	Times of Extreme Outflow Concentrations of Large Hypothetical Pipes	280
72	Runoff Comparisons for Oakdale Storms.	302
73	Runoff Comparisons for Bloody Run Storms	321
74	Summary of Hypothetical Data Tests	351
75	Computer Processing Times for Hypothetical Data Tests	352
76	Costs of Computer Runs for Hypothetical Data Tests	356

ACKNOWLEDGMENTS

The work performed for this study was conducted under contract No. 68-03-0251 of the U.S. Environmental Protection Agency with valuable guidance from Messrs. Chi-Yuan Fan (project officer), Richard Field, and Harry C. Torno. Battelle staff members contributing significantly include Mr. Larry V. Kimmel, who performed much of the model and data collection, Mr. Stacy E. Wise and Ms. Annette S. Myhres, who performed the computer program conversions and data analyses, Ms. Leila A. Counts, who edited the report, and Ms. Jan Greenwell, who contributed significantly to the final report typing and assembly.

The cooperation of model developers and users is greatly appreciated.

SECTION I

CONCLUSIONS

CONTENTS

	Page
Summary	3
Battelle Urban Wastewater Management Model	24
British Road Research Laboratory Model	25
Chicago Flow Simulation Program	25
Chicago Hydrograph Method	26
Colorado State University Urban Runoff Modeling	26
Corps of Engineers STORM Model	27
Dorsch Consult Hydrograph-Volume Method	28
Environmental Protection Agency Stormwater Management Model	28
Hydrocomp Simulation Program	29
Massachusetts Institute of Technology Urban Watershed Model	30
Minneapolis-St. Paul Urban Runoff Model	30
Seattle Computer Augmented Treatment and Disposal System	31
SOGREAH Looped Sewer Model	32
University of Cincinnati Urban Runoff Model	32

CONTENTS (Continued)

Page

University of Illinois Storm Sewer System Simulation Model 33

University of Massachusetts Combined Sewer Control Simulation Model 33

Water Resources Engineers Stormwater Management Model . .34

Wilsey and Ham Urban Watershed System 35

SUMMARY

The reviewed mathematical models are suitable for the simulation of storm and combined sewerage systems or for incorporation in comprehensive simulation models. Considerable differences exist from model to model, however, in the types of phenomena considered and in the mathematical formulations for each phenomenon. The model reviews summarize the objectives, advantages, and limitations of each model.

For some applications, models are available with considerable simplifications in their mathematical detail to reduce input data requirements, computer storage requirements, and computer running time. Some models include unnecessary approximations considering the present state-of-the-art of hydrologic modeling and computer capability. Some of the simplifications, however, are needed for applications to real-time control of overflows which require repeated simulations within fixed time constraints on a small process computer with slow execution times.

The following eleven models were evaluated on the basis of published information and personal communication with the model builders and model users:

1. British Road Research Laboratory Model
2. Chicago Hydrograph Method
3. Colorado State University Urban Runoff Modeling
4. Corps of Engineers Hydrologic Engineering Center STORM Model
5. Hydrocomp Simulation Program
6. Minneapolis-St. Paul Urban Runoff Model
7. Municipality of Metropolitan Seattle Computer Augmented Treatment and Disposal System
8. University of Cincinnati Urban Runoff Model
9. University of Illinois Storm Sewer System Simulation Model
10. University of Massachusetts Urban Runoff Model
11. Wilsey & Ham Urban Watershed Model

The following seven models were also tested by computer runs using both hypothetical and real catchment data:

1. Battelle Urban Wastewater Management Model

2. Dorsch Hydrograph-Volume Method

3. Environmental Protection Agency Stormwater Management Model

4. Massachusetts Institute of Technology Urban Watershed Model

5. Metropolitan Sanitary District of Greater Chicago Flow Simulation Program

6. SOGREAH Looped Sewer Model

7. Water Resources Engineers Stormwater Management Model

The following seven models are also listed but documentation was received too late for detailed reviews:

1. Chicago Runoff and Pollution Model

2. CH2M-Hill Wastewater Collection System Analysis Model

3. Dorsch Consult Quantity-Quality Simulation Program

4. Illinois State Water Survey Urban Drainage Area Simulator

5. Norwegian Institute for Water Research Sewerage System Models

6. Queen's University Urban Runoff Model

7. University of Nebraska Urban Hydrologic Simulator

The tables presented in the following pages provide a quick overview of the features of each model. Table 1 lists major model features and indicates whether a certain phenomenon is being modeled or considered by a model. The table consequently indicates the comprehensiveness of each model. Tables 2 through 5 expand on Table 1 by listing additional categories, brief statements indicating the mathematical formulations for particular phenomena, model limitations, and other details which would be helpful in assessing the applicability of a model for a particular purpose.

Drafts of these tables were submitted to the model developers and their comments were incorporated in the final versions. Not all tables were returned, however, leaving some uncertainties with respect to model formulations of a few models. These are indicated in the tables.

Tables 1 to 5 can be used to select the most useful model for a particular application based on features needed for simulating specific storm and combined sewerage system conditions. Usually the simplest model which simulates the desired phenomena with adequately accurate mathematical formulations should be selected. Input data requirements and computer running times generally decrease with decreasing complexity of the model. Some models include options to suppress portions of the simulation if only selected phenomena are of interest. Although this feature is not listed in the tables, it should be considered in the model selection.

Model testing with hypothetical data showed that computer running time is governed more by efficient formulations of the overall model logic than by the basic equations used for specific phenomena. For instance, no consistent pattern in computer running time was evident between the use of the kinematic and dynamic wave equation. Consequently, since the dynamic wave equation can be solved to simulate downstream flow control, backwater, flow reversal, surcharging, and pressure flow (none of which can be simulated by the kinematic wave equation) the application of models using the dynamic wave equation is recommended, provided the selected model includes an efficient numerical algorithm for its solution. Some models require only the input of typical subcatchment elements and perform hydrologic computations only for these typical subcatchments, but then consider the actual locations of all subcatchments for the overland and sewer flow routing computations. This can save considerable input preparation and computer running time.

Various models stand out due to their completeness of hydrologic and hydraulic formulations, the ease of input data preparation, the efficiency of computational algorithms, and the adequacy of the program output. Other models, although deficient in some of these respects, merit consideration due to special features which are not included in the more comprehensive models but may be required for specific applications.

The following models are consequently recommended for routine applications:

1. Battelle Urban Wastewater Management Model for real-time control and/or design optimization considering

Table 1. COMPARISON OF MAJOR MODEL CATEGORIES

	CATCHMENT HYDROLOGY							SEWER HYDRAULICS								WASTEWATER QUALITY									MISCELLANEOUS					
	MULTIPLE CATCHMENT INFLOWS	DRY-WEATHER FLOW	INPUT OF SEVERAL HYETOGRAPHS	SNOWMELT	RUNOFF FROM IMPERVIOUS AREAS	RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS	FLOW ROUTING IN SEWERS	UPSTR AND DOWNSTR FLOW CONTROL	SURCHARGING AND PRESSURE FLOW	DIVERSIONS	PUMPING STATIONS	STORAGE	PRINTS STAGE	PRINTS VELOCITIES	DRY-WEATHER QUALITY	STORMWATER QUALITY	QUALITY ROUTING	SEDIMENTATION AND SCOUR	QUALITY REACTIONS	WASTEWATER TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION	CONTINUOUS SIMULATION	CAN CHOOSE TIME INTERVAL	DESIGN COMPUTATIONS	REAL-TIME CONTROL	APPLIED TO REAL PROBLEMS	COMPUTER PROGRAM AVAILABLE
BATTELLE NORTHWEST	●	●	●		●	●		●		●	●		●	●	●	●	●	●			●					●	●	●	●	●
BRITISH ROAD RESEARCH LAB	●	●	●		●			●															●			●			●	●
CHICAGO FLOW SIMULATION	●	●	●	●	●	●	●	●					●	●									●		●	●			●	●
CHICAGO HYDRO-GRAPH METHOD	●	●	●		●	●	●	●							●								●			●	●		●	●
COLORADO STATE UNIVERSITY	●	●			●			●	●					●	●															●
CORPS OF ENGINEERS				●	●	●	●				●		●				●			●		●			●				●	●
DORSCH CONSULT	●	●	●		●	●	●	●	●	●	●		●	●	●	●	●	●				●	●			●			●	
ENV PROTECTION AGENCY	●	●	●		●	●		●	●	●	●	●	●			●	●	●	●	●	●		●	●		●	●		●	●
HYDROCOMP	●	●	●	●	●	●	●	●	●		●		●	●	●	●	●	●		●		●	●	●	●	●		●	●	
MASSACHUSSETTS INST OF TECHN	●	●	●	●	●	●		●		●	●		●	●	●								●			●			●	
MINNEAPOLIS-ST. PAUL	●	●	●		●	●		●			●															●		●	●	●
SEATTLE	●	●	●		●	●	●	●	●	●	●	●	●	●												●		●	●	●
SOGREAH	●	●			●	●		●	●	●	●	●	●	●	●			●					●	●		●			●	
UNIVERSITY OF CINCINNATI	●				●	●		●															●			●				●
UNIVERSITY OF ILLINOIS	●	●		●	●	●		●	●		●	●	●	●	●											●	●			●
UNIVERSITY OF MASSACHUSETTS	●	●			●		●	●	●					●	●										●					●
WATER RESOURCES ENGINEERS	●	●	●		●	●		●	●	●	●	●	●	●	●	●	●	●		●			●	●		●			●	
WILSEY AND HAM	●				●	●		●																		●	●		●	

Table 2a. COMPARISON OF HYDROLOGIC FEATURES OF MODELS

	MULTIPLE CATCHMENT INFLOWS	DRY-WEATHER FLOW	SUBCATCHMENT PRECIPITATION	EVAPORATION	SNOW ACCUMULATION AND MELT	GROUNDWATER SIMULATION	GUTTER FLOW
BATTELLE NORTHWEST	YES	HOURLY, DAILY, AND SEASONAL PATTERNS*	WEIGHTED AVG. OF SEVERAL HYETOGRAPHS* FOR EACH SUBCATCHMENT	NO	NO	NO	NO
BRITISH ROAD RESEARCH LABORATORY	YES	STRAIGHT LINE*	WEIGHTED AVG. OF SEVERAL HYETOGRAPHS* FOR EACH SUBCATCHMENT	NO	NO	NO	AVG. TRAVEL TIME FROM MANNING EQU.
CHICAGO FLOW SIMULATION	YES	HOURLY FLOW*	NOT MORE THAN ONE HYETOGRAPH* PER SUBCATCHMENT	NONLINEAR FUNCTION OF SOIL MOISTURE AND DAILY AVG. AIR TEMPERATURE	BASED ON BUREAU OF RECLAMATION ENGINEERING MONOGRAPH 35 METHOD	NO	NO
CHICAGO HYDROGRAPH METHODS	YES	HYDROGRAPHS*	WEIGHTED AVG. OF SEVERAL HYETOGRAPHS* FOR EACH SUBCATCHMENT	FIXED FUNCTION, INDEPENDENT OF METEOROLOGICAL VARIATIONS	NO	NO	KINEMATIC WAVE EQU. WITH MANNING EQU.
COLORADO STATE UNIVERSITY	YES, BUT ONLY INTO SINGLE PIPE STRING	HYDROGRAPHS*	NO	NO	NO	NO	NO
CORPS OF ENGINEERS	FOR ONE CATCHMENT ONLY	NO	ONE HYETOGRAPH* FOR SINGLE CATCHMENT	CONSTANT RATE* BETWEEN RAINSTORMS FOR EACH MONTH	DEGREE-DAY METHOD	NO	NO
DORSCH CONSULT	YES	DIURNAL PATTERN COMPUTED FROM LAND USE	ONE HYETOGRAPH* PER SUBCATCHMENT, SPECIAL STATISTICAL ANALYSES	NONLINEAR FUNCTION OF AIR TEMPERATURE	NO	NO	COMPUTED AS SURFACE RUNOFF OR CHANNEL ROUTING
ENVIRONMENTAL PROTECTION AGENCY	YES	HOURLY, DAILY, AND SEASONAL PATTERNS* OR COMPUTED FROM LAND USE	ONE HYETOGRAPH* PER SUBCATCHMENT	NO	NO	NO	KINEMATIC WAVE EQU. WITH MANNING EQU.
HYDROCOMP	YES	HYDROGRAPHS*	PERCENT OF GAGE PRECIP., ONE HYETOGRAPH* PER SUBCATCHMENT	BASED ON MEAS. POT. EVAPORATION AND AVAILABLE MOISTURE	BASED ON CORPS OF ENGINEERS METHOD	NONLINEAR FUNCTIONS FOR CONTRIBUTIONS TO SURFACE FLOW	NO

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 2b. COMPARISON OF HYDROLOGIC FEATURES OF MODELS

	MULTIPLE CATCHMENT INFLOWS	DRY-WEATHER FLOW	SUBCATCHMENT PRECIPITATION	EVAPORATION	SNOW ACCUMULATION AND MELT	GROUNDWATER SIMULATION	GUTTER FLOW
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	YES	HYDROGRAPHS*	ONE HYETOGRAPH* PER SUBCATCHMENT, STORM CAN BE MOVED ACROSS CATCHMENT	PENMAN EQU., BUT GENERALLY NOT USED	BASED ON CORPS OF ENGINEERS METHOD	NO	KINEMATIC WAVE EQU. WITH MANNING EQU.
MINNEAPOLIS-ST. PAUL	YES	DIURNAL PATTERN*	WEIGHTED AVG. OF SEVERAL HYETOGRAPHS* FOR EACH SUBCATCHMENT	NO	NO	NO	NO
SEATTLE	EACH CATCHMENT REPRESENTED INDEPENDENTLY	DIURNAL PATTERN*	SIX HYETOGRAPHS*	NO	NO	NO	NO
SOGREAH	YES	HYDROGRAPHS*	SEVERAL HYETOGRAPHS OR SYNTHETIC DESIGN HYETOGRAPH	NO	NO	NO	NO
UNIVERSITY OF CINCINNATI	YES	NO	ONE HYETOGRAPH* FOR ENTIRE SEWERAGE SYSTEM	NO	NO	NO	OUTFLOW EQUALS INFLOW DURING SAME TIME INTERVAL
UNIVERSITY OF ILLINOIS	YES	HYDROGRAPHS*	YES**	NO	BASED ON CORPS OF ENGINEERS METHOD	NO	KINEMATIC WAVE EQU. WITH MANNING EQU.
UNIVERSITY OF MASSACHUSETTS	YES, BUT ONLY INTO SINGLE PIPE STRING	CONSTANT FLOW*	ONE HYETOGRAPH* FOR ENTIRE SEWERAGE SYSTEM, GENERATION OF SYN. HYETOGRAPH	NO	NO	NO	NO
WATER RESOURCES ENGINEERS	YES	AVG. FLOW* TRANSFORMED INTO HOURLY AND DAILY PATTERNS BASED ON LAND USE	UP TO THREE HYETOGRAPHS* PER SUBCATCHMENT	NO	NO	NO	KINEMATIC WAVE EQU. WITH MANNING EQU.
WILSEY AND HAM	YES	NO	ONE HYETOGRAPH* FOR ENTIRE SEWERAGE SYSTEM	NO	NO	NO	KINEMATIC WAVE EQU. WITH MANNING EQU.

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
*PROVIDED AS INPUT DATA
**MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 2c. COMPARISON OF HYDROLOGIC FEATURES OF MODELS

	INFILTRATION ON IMPERVIOUS AREAS	OTHER RAINFALL LOSSES ON IMPERVIOUS AREAS	STORM RUNOFF FROM IMPERVIOUS AREAS	INFILTRATION ON PERVIOUS AREAS	OTHER RAINFALL LOSSES ON PERVIOUS AREAS	STORM RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS
BATTELLE NORTHWEST	EXPONENTIAL DECAY FUNCTION, DEPENDENT ON CATCHMENT MOISTURE	INITIAL LOSS* BEFORE RUNOFF BEGINS	UNIT HYDROGRAPH	MODIFIED HOLTAN EQU., DEPENDENT ON SOIL MOISTURE	INITIAL LOSS* BEFORE RUNOFF BEGINS	UNIT HYDROGRAPH	NO
BRITISH ROAD RESEARCH LABORATORY	NO	LOSS AS FUNCTION OF TIME*	RAINFALL EXCESS ROUTED USING TIME-AREA CURVE	NO	NO	NO	NO
CHICAGO FLOW SIMULATION	NO	NO	CONSTANT FRACTION OF OVERLAND FLOW STORAGE	POLYNOMIAL REGRESSION EQU., DEPENDENT ON SOIL MOISTURE	NO	LINEAR STORAGE ROUTING OF OVERLAND FLOW STORAGE	ACCOUNTING OF CATCHMENT MOISTURE, OVERLAND AND CHANNEL STORAGE
CHICAGO HYDROGRAPH METHOD	NO	INITIAL LOSS* BEFORE RUNOFF BEGINS, RECOVERY BETWEEN STORMS	IZZARD'S EQU.	MODIFIED HORTON'S EQU., DEPENDENT ON SOIL MOISTURE	FUNCTION OF MOISTURE CONDITIONS, RECOVERY BETWEEN STORMS	IZZARD'S EQU.	ACCOUNTING OF CATCHMENT MOISTURE, OVERLAND AND CHANNEL STORAGE
COLORADO STATE UNIVERSITY	NO	NO	HYDROGRAPHS*	NO	NO	HYDROGRAPHS*	NO
CORPS OF ENGINEERS	CONSTANT FRACTION OF RAINFALL MINUS DEPRESSION STORAGE	CONSTANT LOSS RATE* MODIFIED BY EVAPORATION	CONSTANT FRACTION OF RAINFALL MINUS DEPRESSION STORAGE	CONSTANT FRACTION OF RAINFALL MINUS DEPRESSION STORAGE	CONSTANT LOSS RATE* MODIFIED BY EVAPORATION	CONSTANT FRACTION OF RAINFALL MINUS DEPRESSION STORAGE	ACCOUNTING OF CATCHMENT MOISTURE AND RESERVOIR STORAGE
DORSCH CONSULT	NO	INITIAL LOSS* BEFORE RUNOFF BEGINS, MODIFIED BY EVAPORATION	KINEMATIC WAVE EQU.	OPTION TO USE HORTON OR HOLTAN EQU. DEPENDENT ON SOIL MOISTURE	INITIAL LOSS BEFORE RUNOFF BEGINS, MODIFIED BY EVAPORATION	KINEMATIC WAVE EQU.	ACCOUNTING OF CATCHMENT MOISTURE AND DEPRESSION STORAGE
ENVIRONMENTAL PROTECTION AGENCY	NO	INITIAL LOSS* BEFORE RUNOFF BEGINS	KINEMATIC WAVE EQU. WITH MANNING EQU.	HORTON EQU., INDEPENDENT OF SOIL MOISTURE	INITIAL LOSS* BEFORE RUNOFF BEGINS	KINEMATIC WAVE EQU. WITH MANNING EQU.	NO
HYDROCOMP	NO	INITIAL LOSS* BEFORE RUNOFF BEGINS, MODIFIED BY EVAPORATION	KINEMATIC WAVE EQU. WITH MANNING EQU.	EMPIRICAL EQU., DEPENDENT ON SOIL MOISTURE	INITIAL LOSS* BEFORE RUNOFF BEGINS, MODIFIED BY EVAPORATION	KINEMATIC WAVE EQU. WITH MANNING EQU.	ACCOUNTING OF CATCHMENT MOISTURE, GROUND AND SURFACE WATER STORAGE

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 2d. COMPARISON OF HYDROLOGIC FEATURES OF MODELS

	INFILTRATION ON IMPERVIOUS AREAS	OTHER RAINFALL LOSSES ON IMPERVIOUS AREAS	STORM RUNOFF FROM IMPERVIOUS AREAS	INFILTRATION ON PERVIOUS AREAS	OTHER RAINFALL LOSSES ON PERVIOUS AREAS	STORM RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	NO	INITIAL LOSS* BEFORE RUNOFF BEGINS	KINEMATIC WAVE EQU. WITH MANNING EQU.	OPTION TO USE HORTON EQU., HOLTAN EQU., SCS METHOD, RUNOFF COEFF. METHOD	INITIAL LOSS* BEFORE RUNOFF BEGINS	KINEMATIC WAVE EQU. WITH MANNING EQU.	NO
MINNEAPOLIS-ST. PAUL	EXPONENTIAL DECAY FUNCTION	INITIAL LOSS* BEFORE RUNOFF BEGINS	UNIT HYDROGRAPH	MODIFIED HOLTAN EQU., DEPENDENT ON SOIL MOISTURE	INITIAL LOSS* BEFORE FUNOFF BEGINS	UNIT HYDROGRAPH	NO
SEATTLE	EMPIRICAL FUNCTIONS**	NO	UNIT HYDROGRAPH	EMPIRICAL FUNCTIONS**	NO	UNIT HYDROGRAPH	RUNOFF ADJUSTED BASED ON PREVIOUS STORM**
SOGREAH	SYNTHETIC RAINFALL EXCESS	SYNTHETIC RAINFALL EXCESS	MUSKINGUM METHOD	SYNTHETIC RAINFALL EXCESS	SYNTHETIC RAINFALL EXCESS	MUSKINGUM METHOD	NO
UNIVERSITY OF CINCINNATI	NO	DEPRESSION LOSS FUNCTION OF RAINFALL	KINEMATIC WAVE EQU. WITH MANNING EQU.	HORTON EQU., WITH TIME OFFSET BASED ON RAINFALL, INDEP. OF SOIL MOISTURE	DEPRESSION LOSS FUNCTION OF RAINFALL AND INFILTRATION	STORAGE ROUTING WITH MANNING EQU.	NO
UNIVERSITY OF ILLINOIS	HORTON EQU.	YES **	KINEMATIC WAVE EQU. WITH MANNING EQU.	HORTON EQU.	YES**	KINEMATIC WAVE EQU. WITH MANNING EQU.	NO
UNIVERSITY OF MASSACHUSETTS	NO	INITIAL LOSS** BEFORE RUNOFF BEGINS	CONSTANT FRACTION OF RAINFALL MINUS INITIAL LOSS	NO	NO	NO	RECOVERY OF DEPRESSION STORAGE
WATER RESOURCES ENGINEERS	CONSTANT RATE*	INITIAL LOSS* BEFORE RUNOFF BEGINS	KINEMATIC WAVE EQU. WITH MANNING EQU.	HORTON EQU.	INITIAL LOSS* BEFORE RUNOFF BEGINS	KINEMATIC WAVE EQU. WITH MANNING EQU.	NO
WILSEY AND HAM		YES**	KINEMATIC WAVE EQU. WITH MANNING EQU.	YES**	YES**	KINEMATIC WAVE EQU. WITH MANNING EQU.	NO

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 3a. COMPARISON OF HYDRAULIC FEATURES OF MODELS

	OPEN CHANNEL NETWORK	FREE SURFACE FLOW	BACKWATER EFFECTS	FLOW REVERSAL	SURCHARGING AND PRESSURE FLOW	DIFFERENT PIPE CROSS-SECTIONS	INFILTRATION INTO SEWERS OR OPEN CHANNELS
BATTELLE NORTHWEST	CONVERGING BRANCH NETWORK WITH DIVERSIONS	KINEMATIC WAVE EQU., CHARACTERISTIC SOLUTION, MANNING EQU.	NO	NO	CONSIDERED INDEPENDENTLY IN EACH PIPE	CIRCULAR PIPE	NO
BRITISH ROAD RESEARCH LABORATORY	CONVERGING BRANCH NETWORK	STORAGE ROUTING, MANNING EQU.	NO	NO	NO	CIRCULAR AND RECTANGULAR PIPE, TRAPEZOIDAL CHANNEL	NO
CHICAGO FLOW SIMULATION	CONVERGING BRANCH NETWORK	STORAGE ROUTING, MANNING EQU.	NO	NO	NO	CIRCULAR PIPE, TRAPEZOIDAL CHANNEL WITH FLOOD PLAIN	NO
CHICAGO HYDROGRAPH METHOD	CONVERGING BRANCH NETWORK	STORAGE ROUTING, MANNING EQU.	NO	NO	NO	CIRCULAR PIPE, TRAPEZOIDAL CHANNEL	NO
COLORADO STATE UNIVERSITY	SINGLE STRING OF PIPES	DYNAMIC WAVE EQU., VARIOUS SOLUTIONS, MANNING OR DARCY-WEISBACH EQU.	DYNAMIC WAVE EQU.	DYNAMIC WAVE EQU.	NO	CIRCULAR PIPE	NO
CORPS OF ENGINEERS	NO FLOW ROUTING	NO	NO	NO	NO	NO	NO
DORSCH CONSULT	CONV. AND DIV. BRANCH AND LOOP NETWORK WITHOUT FLOW REVERSAL	DYNAMIC WAVE EQU., IMPLICIT FINITE DIFF. SOL., MANNING OR PRANDTL EQU.	DYNAMIC WAVE EQU.	NO	DYNAMIC WAVE EQU.	VARIOUS STANDARD SHAPES PLUS ARBITRARY SHAPES	CONSTANT RATE *
ENVIRONMENTAL PROTECTION AGENCY	CONVERGING BRANCH NETWORKS WITH DIVERSIONS	KINEMATIC WAVE EQU., EXPLICIT FINITE DIFF. SOL., MANNING EQU.	FOR KNOWN DEPTH VS STORAGE RELATION AT NOT MORE THAN TWO LOCATIONS	NO	CONSIDERED INDEPENDENTLY IN EACH PIPE	THIRTEEN STANDARD SHAPES PLUS TWO ARBITRARY SHAPES	CONSTANT RATE *
HYDROCOMP	CONVERGING BRANCH NETWORK WITH DIVERSIONS	KINEMATIC WAVE EQU., WITH DIFFUSION TERM,** MANNING EQU.	DIFFUSION TERM IN KINEMATIC WAVE EQU.	NO	NO	CIRCULAR PIPE, TRAPEZOIDAL CHANNEL WITH FLOOD PLAIN	NONLINEAR FUNCTION DEPENDENT ON SOIL MOISTURE

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
*PROVIDED AS INPUT DATA
**MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 3b. COMPARISON OF HYDRAULIC FEATURES OF MODELS

	OPEN CHANNEL NETWORK	FREE SURFACE FLOW	BACKWATER EFFECTS	FLOW REVERSAL	SURCHARGING AND PRESSURE FLOW	DIFFERENT PIPE CROSS-SECTIONS	INFILTRATION INTO SEWERS OR OPEN CHANNELS
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	CONVERGING AND DIVERGING BRANCH AND LOOP NETWORK	KINEMATIC WAVE EQU., EXPLICIT OR IMPLICIT FINITE DIFF. SOL., MANNING EQU.	FOR KNOWN DEPTH VS DISCHARGE RELATIONSHIP	NO	CONSIDERED INDEPENDENTLY IN EACH PIPE	VARIOUS STANDARD SHAPES PLUS ARBITRARY SHAPES	NO
MINNEAPOLIS-ST. PAUL	CONVERGING BRANCH NETWORK WITH DIVERSIONS	PROGRESSIVE AVG. LAG METHOD	NO	NO	NO	CIRCULAR PIPE	NO
SEATTLE	NO FLOW ROUTING	NO	CONSIDERED ONLY FOR TRUNK SEWER STORAGE UPSTREAM OF REGULATORS	NO	CONSIDERED ONLY FOR TRUNK SEWER STORAGE UPSTREAM OF REGULATORS	CIRCULAR AND HORSESHOE PIPE	DECAY FUNCTION OF RAINFALL
SOGREAH	CONVERGING AND DIVERGING BRANCH AND LOOP NETWORK	DYNAMIC WAVE EQU., IMPLICIT FINITE DIFF. SOL., MANNING EQU.	DYNAMIC WAVE EQU.	DYNAMIC WAVE EQU.	YES	CIRCULAR AND EGG-SHAPED PIPE, TRAPEZ. CHANNEL ARBITRARY SHAPES	HYDROGRAPHS *
UNIVERSITY OF CINCINNATI	CONVERGING BRANCH NETWORK	TRANSLATION OF HYDRO GRAPH BY AVG. FLOW TIME OF HYDROGRAPH, MANNING EQU.	NO	NO	NO	CIRCULAR PIPE AND RECTANGULAR CHANNEL	NO
UNIVERSITY OF ILLINOIS	CONVERGING BRANCH NETWORK	DYNAMIC WAVE EQU., EXPLICIT FINITE DIFF. SOL., DARCY-WEISBACH EQU.	DYNAMIC WAVE EQU.	DYNAMIC WAVE EQU.	NO	CIRCULAR PIPE	NO
UNIVERSITY OF MASSACHUSETTS	SINGLE STRING OF PIPES	DYNAMIC WAVE EQU., IMPLICIT FINITE DIFF. SOL., MANNING EQU.	YES		NO	CIRCULAR PIPE	NO
WATER RESOURCES ENGINEERS	CONVERGING AND DIVERGING BRANCH AND LOOP NETWORK	DYNAMIC WAVE EQU., EXPLICIT FINITE DIFF. SOL., MANNING EQU.	DYNAMIC WAVE EQU.	DYNAMIC WAVE EQU.	CONSIDERED INDEPENDENTLY AT EACH JUNCTION **	FIVE PIPE SHAPES, TRAPEZOIDAL CHANNEL	CONSTANT RATE *
WILSEY AND HAM	CONVERGING BRANCH NETWORK	KINEMATIC WAVE EQU.,** MANNING EQU.	NO	NO	NO		NO

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
*PROVIDED AS INPUT DATA
**MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 3c. COMPARISON OF HYDRAULIC FEATURES OF MODELS

	DIFFERENT DIVERSION STRUCTURES	DIVERSION COMPUTATION	PUMPING STATIONS	STORAGE FACILITIES	STORAGE COMPUTATION	COMPUTES AND PRINTS STAGE HYDROGRAPHS	COMPUTES AND PRINTS FLOW VELOCITIES
BATTELLE NORTHWEST	FOUR COMMON TYPES OF ORIFICE/ WEIR COMBINATIONS	ORIFICE AND WEIR EQU. NEGLECTING DOWNSTREAM CONDITIONS	NO	FOR UNDEFINED SHAPE, ONLY INFLOW VOLUME COMPUTED	COMPUTES INFLOW AND VOLUME OF STORMWATER OVER-FLOW	FOR EACH PIPE REACH	FOR EACH PIPE REACH
BRITISH ROAD RESEARCH LABORATORY	NO	NO	NO	NO	NO	NO	NO
CHICAGO FLOW SIMULATION	NO	NO	NO	FOR UNDEFINED SHAPE, NO DEPTH COMPUTATIONS	STORES INFLOW EXCEEDING SPECIFIED MAX. OUTFLOW	FOR EACH PIPE AND CHANNEL REACH	NO
CHICAGO HYDROGRAPH METHOD	NO	NO	NO	NO	NO	NO	FOR EACH SUBCATCHMENT
COLORADO STATE UNIVERSITY	NO	NO	NO	NO	NO	AT ANY POINT	AT ANY POINT
CORPS OF ENGINEERS	UNDEFINED SHAPE	EXCESS OVER STORAGE AND TREATMENT CAPACITY OVERFLOWS	NO	EXCESS OVER TREATMENT CAPACITY IS STORED	STORED WATER RETURNS TO TREATMENT PLANT	NO	NO
DORSCH CONSULT	WEIR AND ORIFICE	WEIR AND ORIFICE EQU. CONSIDERING UPSTREAM AND DOWNSTREAM CONDITIONS	NO	YES	FUNCTION OF SHAPE, INFLOW AND OUTFLOW	FOR EACH SEWER SYSTEM ELEMENT	COMPUTED FOR EACH SEWER SYSTEM ELEMENT, BUT PRINTS ONLY SELECTED VALUES
ENVIRONMENTAL PROTECTION AGENCY	TWO TYPES: UNSPECIFIED SHAPE OR WEIR	CONSTANT DIVERSION OR WEIR EQU. NEGLECTING DOWN-STREAM CONDITIONS	CONSTANT DISCHARGE IF WET WELL EXCEEDS SPECIFIED DEPTH	TWO ONLY, MODELS FOUR TYPES OF OUTLET STRUCTURES	FUNCTION OF SHAPE, INFLOW AND OUTFLOW	COMPUTED, BUT NOT PRINTED	COMPUTED, BUT NOT PRINTED
HYDROCOMP	DIVERSION HYDROGRAPHS*	DIVERSION HYDROGRAPHS*	PUMPING SCHEDULE*	YES, CONSIDERS BOTH SPILLWAY AND POWER TURBINE FLOW	RULE CURVE OPERA-TION OR PRESET OUTFLOWS, INCLUDING POWER FLOWS	FOR EACH CHANNEL REACH AND STORAGE	FOR EACH PIPE AND CHANNEL REACH AND STORAGE

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 3d. COMPARISON OF HYDRAULIC FEATURES OF MODELS

	DIFFERENT DIVERSION STRUCTURES	DIVERSION COMPUTATION	PUMPING STATIONS	STORAGE FACILITIES	STORAGE COMPUTATION	COMPUTES AND PRINTS STAGE HYDROGRAPHS	COMPUTES AND PRINTS FLOW VELOCITIES
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	WEIR AND ORIFICE	WEIR AND ORIFICE EQU., NEGLECTING DOWNSTR. COND., OR DEPTH VS DISCHARGE	NO	YES, MODELS WEIR AND ORIFICE OUTLET	FUNCTION OF SHAPE, INFLOW AND OUTFLOW	FOR EACH PIPE AND CHANNEL REACH	FOR EACH PIPE AND CHANNEL REACH
MINNEAPOLIS-ST. PAUL	THREE TYPES OF COMMON ORIFICE/WEIR COMB., THREE SPECIAL STRUCTURES	ORIFICE AND WEIR EQU. NEGLECTING DOWNSTREAM CONDITIONS	NO	NO	NO	NO	NO
SEATTLE	SIX TYPES OF GATE CONTROL STURCTURES	ORIFICE AND WEIR EQU.'S CONSIDERING UPSTR. AND DOWNSTR. CONDITIONS	TWO TYPES: PROPORTIONAL AND CONSTANT RATE	STORAGE IN TRUNK SEWERS UPSTREAM OF REGULATORS	FUNCTION OF SHAPE, INFLOW AND OUTFLOW	FOR EACH INFLOW POINT AND JUNCTION	FOR EACH PIPE REACH
SOGREAH	RECTANGULAR WEIR AND ORIFICE INCL. TIME VARYING GATE SETTINGS	ORIFICE AND WEIR EQU. CONS. UPSTR. AND DOWNSTR. COND. AND FLOW REVERSAL	YES**	YES	YES**	FOR EACH SEWER SYSTEM ELEMENT AND COMPUTATIONAL POINT	FOR EACH SEWER SYSTEM ELEMENT AND COMPUTATIONAL POINT
UNIVERSITY OF CINCINNATI	NO	NO	NO	NO	NO	COMPUTED FOR EACH PIPE REACH, BUT NOT PRINTED	COMPUTED FOR EACH PIPE REACH, BUT NOT PRINTED
UNIVERSITY OF ILLINOIS	WEIR OR GATE FLOW CONTROL AND DIVERSION AT OUTLET	WEIR EQU. NEGLECTING DOWNSTR. CONDITION, OR DEPTH VS DISCHARGE	PUMPING RATE CAN BE OUTLET BOUNDARY CONDITION	YES	FUNCTION OF SHAPE, INFLOW AND OUTFLOW	FOR EACH SEWER SYSTEM ELEMENT AND AT ANY DESIRED POINTS	FOR EACH SEWER SYSTEM ELEMENT AND AT ANY DESIRED POINTS
UNIVERSITY OF MASSACHUSETTS	NO	NO	NO	NO	NO	FOR EACH PIPE REACH	FOR EACH PIPE REACH
WATER RESOURCES ENGINEERS	WEIRS, GATES OR ORIFICES, WITH OR WITHOUT TIDE GATES	ORIFICE AND WEIR EQU. CONS. UPSTR. AND DOWNSTR. COND. AND FLOW REVERSAL	UP TO 3-STAGE PUMPING STATIONS, RULE CURVE OPERATION	YES	FUNCTION OF SHAPE, INFLOW AND OUTFLOW	FOR EACH SEWER JUNCTION	FOR EACH PIPE AND CHANNEL REACH
WILSEY AND HAM	NO	NO	NO	NO	NO	YES	YES

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 4a. COMPARISON OF WATER QUALITY FEATURES OF MODELS

	QUALITY CONSTITUENTS	DRY-WEATHER QUALITY	STORM RUNOFF QUALITY	QUALITY INTERACTIONS ON CATCHMENTS	QUALITY ROUTING IN CHANNELS	SEDIMENTATION AND SCOUR IN CHANNELS	QUALITY REACTIONS IN CHANNELS
BATTELLE NORTHWEST	7 ARBITRARY CONSERVATIVE CONSTITUENTS	HOURLY, DAILY AND SEASONAL PATTERNS	LINEAR FUNCTION OF STORM DISCHARGE AND VOLUME	NO	ADVECTION WITH MIXING BETWEEN SUCCESSIVE TIME STEPS	NO	NO
BRITISH ROAD RESEARCH LABORATORY	NO	NO	NO	NO	NO	NO	NO
CHICAGO FLOW SIMULATION	NO	NO	NO	NO	NO	NO	NO
CHICAGO HYDROGRAPH METHOD	NO	NO	NO	NO	NO	NO	NO
COLORADO STATE UNIVERSITY	NO	NO	NO	NO	NO	NO	NO
CORPS OF ENGINEERS	SUSPENDED AND SETTLEABLE SOLIDS, BOD, NITROGEN AND PHOSPHATE	NO	NONLINEAR FUNCT. OF CATCHMENT CHAR., POLL. ACCUMULATION AND RUNOFF	BOD, NITROGEN AND PHOSPHATE DEP. ON SUSPENDED AND SETTLEABLE SOLIDS	NO	NO, BUT LAND SURFACE EROSION BY UNIVERSAL SOIL LOSS EQU.	NO
DORSCH CONSULT	SEPARATE MODEL, 4 ARBITRARY CONSERVATIVE CONSTITUENTS	DIURNAL PATTERNS CALCULATED FROM LAND USE	NONLINEAR FUNCT. OF CATCHM. CHAR., POLL. ACCUMUL. AND RUNOFF	NO	ADVECTION WITH PARTIAL MIXING BETWEEN SUCCESSIVE TIME STEPS	NO	NO
ENVIRONMENTAL PROTECTION AGENCY	SUSP. AND SETTL. SOLIDS, BOD, COD, COLIFORM, N, PO_4, OIL AND GREASE	HOURLY, DAILY AND SEASONAL PATTERNS* OR COMPUTED FROM LAND USE	NONLINEAR FUNCTION OF CATCHMENT CHAR., POLL. ACCUMULATION, AND RUNOFF	BOD, NITROGEN AND PHOSPHATE DEP. ON SUSPENDED AND SETTLEABLE SOLIDS	ADVECTION WITH MIXING BETWEEN SUCCESSIVE TIME STEPS	SUSPENDED SOLIDS, CONSIDERING PARTICLE SIZE DISTRIBUTION	FIRST-ORDER BOD DECAY WITHOUT INTERACTIONS
HYDROCOMP	17 CONSTITUENTS INCLUDING WATER TEMPERATURE	INFLOW CONCENTRATIONS* OR NONLINEAR FUNCTION OF INFLOW HYDROGRAPH	NONLINEAR FUNCTION OF CATCHMENT CHAR., POLL. ACCUMULATION AND RUNOFF	NO	ADVECTION WITH WEIGHTED MIXING BETWEEN SUCCESSIVE TIME STEPS	NO	VARIOUS REACTIONS AND INTERACTIONS

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 4b. COMPARISON OF WATER QUALITY FEATURES OF MODELS

	QUALITY CONSTITUENTS	DRY-WEATHER QUALITY	STORM RUNOFF QUALITY	QUALITY INTERACTIONS ON CATCHMENTS	QUALITY ROUTING IN CHANNELS	SEDIMENTATION AND SCOUR IN CHANNELS	QUALITY REACTIONS IN CHANNELS
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	NO	NO	NO	NO	NO	NO	NO
MINNEAPOLIS-ST. PAUL	NO	NO	NO	NO	NO	NO	NO
SEATTLE	NO	NO	NO	NO	NO	NO	NO
SOGREAH	CONSERVATIVE CONSTITUENTS AND 1ST-ORDER DECAY	NO	YES*	NO	ADVECTION	NO	NO
UNIVERSITY OF CINCINNATI	NO	NO	NO	NO	NO	NO	NO
UNIVERSITY OF ILLINOIS	NO	NO	NO	NO	NO	NO	NO
UNIVERSITY OF MASSACHUSETTS	NO	NO	NO	NO	NO	NO	NO
WATER RESOURCES ENGINEERS	SUSP. AND SETTL. SOLIDS, BOD, N, PO_4, OIL AND GREASE, 17 CONS. CONST.	AVG. CONCENTRATIONS TRANSFORMED INTO DIURNAL PATTERNS BASED ON LAND USE	NONLINEAR FUNCT. OF CATCHMENT CHAR., POLL. ACCUMULATION AND RUNOFF	ALL CONSTITUENTS DEPENDENT ON SUSPENDED AND SETTLEABLE SOLIDS	ADVECTION	NO	NO
WILSEY AND HAM	NO	NO	NO	NO	NO	NO	NO

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 4c. COMPARISON OF WATER QUALITY FEATURES OF MODELS

	QUALITY ROUTING THROUGH STORAGE	QUALITY REACTIONS IN STORAGE	TREATMENT FACILITIES	QUALITY INTERACTIONS DURING TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION
BATTELLE NORTHWEST	NO	NO	TABULAR FUNCTION OF FLOW AND CONCENTRATION	NO	NO	NO	NO
BRITISH ROAD RESEARCH LABORATORY	NO	NO	NO	NO	NO	YES	NO
CHICAGO FLOW SIMULATION	NO	NO	NO	NO	NO	YES	NO
CHICAGO HYDROGRAPH METHOD	NO	NO	NO	NO	NO	YES	NO
COLORADO STATE UNIVERSITY	NO	NO	NO	NO	NO	NO	NO
CORPS OF ENGINEERS	PLUG FLOW	NO	COMPUTES ONLY INFLUENT FLOW AND POLLUTANTS	NO	FUNCTION OF POLLUTANT ACCUMULATION AND STREET SWEEPING	NO	NO
DORSCH CONSULT	PLUG FLOW	NO	TABULAR FUNCTION OF FLOW AND CONCENTRATION	NO	FUNCTION OF POLLUTANT ACCUMULATION AND STREET SWEEPING	YES	YES, BUT NO QUALITY REACTIONS AND INTERACTIONS
ENVIRONMENTAL PROTECTION AGENCY	PLUG FLOW OR INSTANTANEOUS MIXING	NO	ONE FACILITY ONLY, EQU.'S FOR NINE PHYSICAL/CHEMICAL PROCESSES	COLIFORM REMOVAL DEPENDENT ON SUSPENDED SOLIDS	NO	FOR ONE AND TWO-DIMENSIONAL REPRESENTATION	FOR ONE AND TWO-DIMENSIONAL REPRESENTATION
HYDROCOMP	WEIGHTED MIXING BETW. ONE TO NINE LAYERS, OUTFLOW FROM ANY LAYER	VARIOUS REACTIONS AND INTERACTIONS**	NO	NO	FUNCTION OF POLLUTANT ACCUMULATION AND STREET SWEEPING	YES	YES

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 4d. COMPARISON OF WATER QUALITY FEATURES OF MODELS

	QUALITY ROUTING THROUGH STORAGE	QUALITY REACTIONS IN STORAGE	TREATMENT FACILITIES	QUALITY INTERACTIONS DURING TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	NO	NO	NO	NO	NO	NO	NO
MINNEAPOLIS-ST. PAUL	NO	NO	NO	NO	NO	NO	NO
SEATTLE	NO	NO	NO	NO	NO	NO	NO
SOGREAH	INSTANTANEOUS MIXING 1ST-ORDER DECAY POSSIBLE	NO	NO	NO	NO	YES	NO
UNIVERSITY OF CINCINNATI	NO	NO	NO	NO	NO	YES	NO
UNIVERSITY OF ILLINOIS	NO	NO	NO	NO	NO	NO	NO
UNIVERSITY OF MASSACHUSETTS	NO	NO	NO	NO	NO	NO	NO
WATER RESOURCES ENGINEERS	EXPONENTIAL RELEASE FUNCTION FOR CATCHBASINS	NO	NO	NO	NO	YES	YES
WILSEY AND HAM	NO	NO	NO	NO	NO	NO	NO

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 5a. COMPARISON OF MISCELLANEOUS FEATURES OF MODELS

	CONTINUOUS SIMULATION	TIME INTERVAL	MIN. AND MAX. TIME PERIOD FOR SIMULATION	ALLOWS INPUT OF INITIAL CONDITIONS	DESIGN COMPUTATIONS	REAL-TIME CONTROL	TESTED ON URBAN DATA AND APPLIED TO REAL PROBLEMS
BATTELLE NORTHWEST	SINGLE STORM PERIOD, NO WATER AND QUALITY BALANCE BETWEEN STORMS	CONSTANT INTERVALS IN MINUTES (MIN. = 2, MAX. = 60)	MIN. 1 HR, MAX. 6 HRS OR 56 TIME STEPS	CATCHMENT MOISTURE, NOT NEEDED FOR ROUTING	LEAST-COST SIZES OF SEWERS, STORAGE AND TREATMENT FACILITIES	DYNAMIC PROGRAMMING FOR REGULATOR OPERATION	LIMITED TESTING AND APPLICATIONS
BRITISH ROAD RESEARCH LABORATORY	SINGLE STORM PERIOD, NO WATER BALANCE BETWEEN STORMS	CONSTANT INTERVALS IN MINUTES	NO APPARENT MIN., MAX. 500 TIME STEPS	CATCHMENT MOISTURE, NOT FOR CHANNELS	NO	NO	EXTENSIVE TESTING AND APPLICATIONS
CHICAGO FLOW SIMULATION	CONTINUOUS	CONSTANT INTERVALS IN MINUTES (MIN. = 5, MAX. = 60)	MIN. 24 HOURS, NO APPARENT MAX.	NO	NO	NO	LIMITED TESTING AND APPLICATIONS
CHICAGO HYDROGRAPH METHOD	SINGLE STORM PERIOD, WATER BALANCE BETWEEN STORMS	CONSTANT INTERVALS IN MINUTES (1 MIN. REQ'D FOR RAIN, CAN BE MORE FOR RUNOFF)	NO APPARENT MIN., MAX. 500 MINUTES	CATCHMENT MOISTURE, NOT FOR CHANNELS	SIZES PIPES FOR PEAK FLOW, NO COST COMPUTATIONS	NO	LIMITED TESTING, EXTENSIVE APPLICATIONS
COLORADO STATE UNIVERSITY	CONTINUOUS FLOW ROUTING IN SINGLE STRING OF PIPES	DEPENDENT ON NUMERICAL STABILITY CONDITION	NO APPARENT MIN. OR MAX.	CONSTANT DISCHARGE IN ENTIRE PIPE STRING	NO	DIFFERENT SCHEMES WERE INVESTIGATED	TESTED ON EXPERIMENTAL PIPE STRING, NO APPLICATIONS
CORPS OF ENGINEERS	CONTINUOUS FOR SEVERAL YEARS	MUST BE 1 HOUR	MIN. 24 HOURS, NO APPARENT MAX.	CATCHMENT MOISTURE AND QUALITY, NOT FOR CHANNELS AND STORAGE	NO	NO	LIMITED TESTING AND APPLICATIONS
DORSCH CONSULT	SINGLE STORM PERIOD, WATER BALANCE BETWEEN STORMS	CONSTANT INTERVALS IN MINUTES	NO APPARENT MIN., MAX. 500 TIME STEPS	CATCHMENT MOISTURE, NOT NEEDED FOR ROUTING	NO	NO	MODERATE TESTING, EXTENSIVE APPLICATIONS
ENVIRONMENTAL PROTECTION AGENCY	SINGLE STORM PERIOD, NO WATER AND QUALITY BALANCE BETWEEN STORMS	CONSTANT INTERVALS IN MINUTES	NO APPARENT MIN., MAX. 150 TIME STEPS	CATCHMENT MOISTURE AND QUALITY, CONSTANT FLOW AND QUALITY IN CHANNELS	COSTS OF STORAGE AND TREATMENT, SIZES PIPES TO ELIMINATE SURCHARGING	NO	MODERATE TESTING, EXTENSIVE APPLICATIONS
HYDROCOMP	CONTINUOUS FOR SEVERAL YEARS	MINUTES, HOURLY, BI-DAILY, DAILY OR SEMI-MO.; COMPU. USE SM. INTERVALS	NO APPARENT MIN. OR MAX.	FOR ALL SEWER SYSTEM ELEMENTS	NO	FLOW FORECASTING AND SYSTEM OPERATION WITHOUT OPTIMIZATION	MODERATE URBAN, EXTENSIVE NONURBAN TESTING AND APPLICATIONS

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
*PROVIDED AS INPUT DATA
**MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 5b. COMPARISON OF MISCELLANEOUS FEATURES OF MODELS

	CONTINUOUS SIMULATION	TIME INTERVAL	MIN. AND MAX. TIME PERIOD FOR SIMULATION	ALLOWS INPUT OF INITIAL CONDITIONS	DESIGN COMPUTATIONS	REAL-TIME CONTROL	TESTED ON URBAN DATA AND APPLIED TO REAL PROBLEMS
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	SINGLE STORM PERIOD	CONSTANT INTERVAL IN MINUTES, SET INTERNALLY FOR EACH ELEMENT	NO MIN. OR MAX.	CATCHMENT MOISTURE, CHANNEL DISCHARGES	LEAST-COST SIZES OF PIPES, STORAGE AND TREATMENT IN SEPARATE PROGRAM	NO	EXTENSIVE TESTING AND APPLICATIONS
MINNEAPOLIS-ST. PAUL	SINGLE STORM PERIOD, NO WATER BALANCE BETWEEN STORMS	CONSTANT INTERVAL IN SECONDS	NO APPARENT MIN. OR MAX.	CATCHMENT MOISTURE, NOT FOR CHANNELS	NO	ITERATIVE RUNS WITH TRIAL REGULATOR SETTINGS	LIMITED TESTING AND APPLICATIONS
SEATTLE	SINGLE STORM PERIOD, LIMITED WATER BALANCE BETWEEN STORMS	CONSTANT INTERVAL IN MINUTES	MIN. 3 HOURS, MAX. 24 HOURS	NO	NO	AUTOM. REGULATOR OPERATION USING RULE CURVES	LIMITED TESTING, ROUTINE APPLICATIONS FOR REAL-TIME CONTROL
SOGREAH	SINGLE STORM PERIOD, NO WATER BALANCE BETWEEN STORMS	CONSTANT INTERVAL IN MINUTES CAN BE DIFF. FOR EACH HOUR	MIN. 1 HOUR, NO APPARENT MAX.	FOR EACH SEWER SYSTEM ELEMENT	NO	NO	LIMITED TESTING, MODERATE APPLICATIONS
UNIVERSITY OF CINCINNATI	SINGLE STORM PERIOD, NO WATER BALANCE BETWEEN STORMS	CONSTANT INTERVAL IN MINUTES	NO APPARENT MIN., MAX. 120 TIME STEPS	CATCHMENT MOISTURE, NOT FOR CHANNELS	NO	NO	LIMITED TESTING, NO APPLICATIONS
UNIVERSITY OF ILLINOIS	SINGLE STORM PERIOD UNLESS CONTINUOUS HYDROGRAPHS ARE INPUT	CONSTANT INTERVAL IN SECONDS, RESTRICTED BY NUMERICAL STABILITY CONDITION	NO APPARENT MIN. OR MAX.	CONSTANT FLOW	SIZES OF PIPES FOR PEAK FLOW OR MAX. DEPTH, NO COST COMPUTATIONS	NO	LIMITED TESTING, NO APPLICATIONS
UNIVERSITY OF MASSACHUSETTS	CONTINUOUS EXCEPT FOR SNOW	MUST BE 1 HOUR	MIN. 1 HOUR, MAX. 1 MONTH		NO	NO	NO
WATER RESOURCES ENGINEERS	SINGLE STORM PERIOD, NO WATER AND QUALITY BALANCE BETWEEN STORMS	CONSTANT INTERVAL, RESTRICTED BY NUMERICAL STABILITY CONDITION	NO MIN. OR MAX.	FOR ALL SEWER SYSTEM ELEMENTS	NO	NO	MODERATE TESTING AND APPLICATIONS
WILSEY AND HAM	SINGLE STORM PERIOD, NO WATER BALANCE BETWEEN STORMS			CATCHMENT MOISTURE, NOT FOR CHANNELS	SIZES OF PIPES FOR PEAK FLOW	NO	

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 5c. COMPARISON OF MISCELLANEOUS FEATURES OF MODELS

	ERROR MESSAGES	PRINCIPAL PRINTED OUTPUT	PRINCIPAL GRAPHIC OUTPUT	UNITS OF MEASUREMENT	COMPUTER VERSIONS AND CORE STORAGE REQUIREMENT	COMPUTER LANGUAGE	COMPUTER PROGRAM AVAILABLE
BATTELLE NORTHWEST	LIMITED	RAIN, DEPTH, VEL., DISCH., STOR., QUAL., COSTS AND SIZES FOR DESIGN	NETWORK DIAGRAM, RAIN, DISCHARGE, STORAGE, QUALITY	ENGLISH, CONCEN-TRATIONS METRIC	DEC PDP-9, 32K BYTES; UNIVAC 1108, 64K BYTES	SIMULATION: FORTRAN IV, 5000 STATEMENTS; OPTIMIZ.: FORTRAN V, 2500 STATEMENTS	YES, DRAFT DOCUMENTATION
BRITISH ROAD RESEARCH LABORATORY	LIMITED	DISCHARGE	DISCHARGE	ENGLISH	IBM 360	FORTRAN IV, 1200 STATEMENTS	YES
CHICAGO FLOW SIMULATION	LIMITED	RAIN, SNOW, DEPTH, DISCHARGE, STORAGE,	DEPTH, DISCHARGE, STORAGE	ENGLISH	IBM 360, CDC 6400, 32K BYTES	FORTRAN IV, 650 STATEMENTS	YES
CHICAGO HYDROGRAPH METHOD	LIMITED	RAIN, VELOCITIES DISCHARGE, SEWER DIAMETERS	DISCHARGE	ENGLISH	IBM 1130 8K BYTES	FORTRAN IV	YES, DRAFT USER'S MANUAL
COLORADO STATE UNIVERSITY	LIMITED	DEPTH, VELOCITY, DISCHARGE	NO	ENGLISH	CDC 6400 AND 6600	FORTRAN IV	YES
CORPS OF ENGINEERS	MODERATE	RAIN, DISCHARGE, STORAGE, QUALITY, LAND EROSION	STORAGE UTILIZATION	ENGLISH, CONCEN-TRATIONS METRIC	UNIVAC 1108, IBM 360/50, CDC 6600, 70K BYTES	FORTRAN IV, 4400 STATEMENTS	YES
DORSCH CONSULT	EXTENSIVE, PLUS SEPARATE DATA CHECKING PROGRAM (250 MESSAGES)	DEPTH, DISCHARGE (SEPARATELY FOR SANITARY AND STORM WATER)	DEPTH, DISCHARGE	METRIC AND ENGLISH	UNIVAC 1108, CDC 6600	FORTRAN IV, 7000 STATEMENTS	UNDER USE AGREEMENT
ENVIRONMENTAL PROTECTION AGENCY	MODERATE	RAIN, DISCH., QUAL., SEWER SIZES, COSTS OF STORAGE AND TREATMENT	RAIN, DISCHARGE, QUALITY	ENGLISH, CONCENTRATIONS METRIC	IBM 360 AND 370, CDC 6400, UNIVAC 1108, 350K BYTES	FORTRAN IV, MORE THAN 10,000 STATEMENTS	YES, BEING EXPANDED WITH NEW MODEL FEATURES
HYDROCOMP	EXTENSIVE	RAIN, SNOW, SOIL MOISTURE, DEPTH, DISCHARGE, STORAGE, QUALITY	DISCHARGE, QUALITY	METRIC AND ENGLISH	IBM 360 AND 370, 240K BYTES	PL/1	UNDER USE AGREEMENT

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

Table 5d. COMPARISON OF MISCELLANEOUS FEATURES OF MODELS

	ERROR MESSAGES	PRINCIPAL PRINTED OUTPUT	PRINCIPAL GRAPHIC OUTPUT	UNITS OF MEASUREMENT	COMPUTER VERSIONS AND CORE STORAGE REQUIREMENT	COMPUTER LANGUAGE	COMPUTER PROGRAM AVAILABLE
MASSACHUSETTS INSTITUTE OF TECHNOLOGY	MODERATE	RAIN, SNOW, DEPTH, DISCHARGE, VOLUMES	NETWORK DIAGRAM, RAIN, DISCHARGE	ENGLISH SOME METRIC CONVERSIONS	IBM 360/67, IBM 370/155, IBM 370/195	FORTRAN IV, MORE THAN 10,000 STATEMENTS	UNDER USE AGREEMENT
MINNEAPOLIS-ST. PAUL	LIMITED	RAIN, DISCHARGE, CATCHMENT MOISTURE CONDITIONS, REGULATOR STATUS	NO	ENGLISH	DEC PDP-9, 32K BYTES	FORTRAN IV	YES
SEATTLE	ADEQUATE FOR REAL-TIME OVERFLOW CONTROL	RAIN, REGULATOR FLOW DEPTH AND CONTROL STATUS	NO	ENGLISH	XEROX SIGMA 2, 52K BYTES	FORTRAN IV	YES, BUT REAL-TIME CONTROL FUNCTIONS ARE SYSTEM SPECIFIC
SOGREAH	MODERATE	DEPTH, VELOCITY, DISCHARGE, PRESSURE, STREET FLOODING DEPTH	DEPTH, VELOCITY, DISCHARGE	METRIC	IBM 360/65, 65K BYTES FOR 1000 COMPUTA-TIONAL POINTS	FORTRAN IV, 3000 STATEMENTS	UNDER USE AGREEMENT
UNIVERSITY OF CINCINNATI	NONE	DISCHARGE	NO	ENGLISH	IBM 360/65	FORTRAN IV, 850 STATEMENTS	YES
UNIVERSITY OF ILLINOIS	MODERATE	DEPTH, VELOCITY, DISCHARGE, SEWER DIAMETERS	DEPTH, DISCHARGE	ENGLISH	IBM 360/75, 100 TO 300 K BYTES	PL/1 AND ASSEMBLER LANGUAGE, 3000 STATEMENTS	YES
UNIVERSITY OF MASSACHUSETTS	LIMITED	DEPTH, DISCHARGE	NO	ENGLISH	CDC 3600	FORTRAN IV	YES
WATER RESOURCES ENGINEERS	MODERATE	DEPTH, VELOCITY, DISCHARGE, QUALITY	DEPTH, DISCHARGE	METRIC AND ENGLISH	UNIVAC 1108, CDC 6600, IBM 360/50	FORTRAN IV	YES
WILSEY AND HAM		DISCHARGE, SEWER DIAMETERS		ENGLISH	CDC 6600, TYM-SHARE XDS940		UNDER USE AGREEMENT

NOTES: BLANK SPACE INDICATES THAT RELEVANT INFORMATION WAS NOT AVAILABLE
* PROVIDED AS INPUT DATA
** MATHEMATICAL FORMULATION AND/OR METHOD OF SOLUTION NOT AVAILABLE

hydraulic, water quality and cost constraints, provided the hydrologic and hydraulic model assumptions are adequate for particular applications (lumping of many small subcatchments into few large catchments, neglect of downstream flow control, backwater, flow reversal, surcharging, and pressure flow).

2. Corps of Engineers STORM Model for preliminary planning of required storage and treatment capacity for storm runoff from single major catchments, considering both the quantity and quality of the surface runoff and untreated overflows.

3. Dorsch Consult Hydrograph Volume Method for single-event flow analysis considering most important hydraulic phenomena (except flow reversal). A Quantity-Quality Simulation Program for continuous wastewater flow and quality analysis is now available, but the model was completed too late for evaluation.

4. Environmental Protection Agency Stormwater Management Model for single-event wastewater flow and quality analysis provided the hydraulic limitations of the model are acceptable (neglect of downstream flow control and flow reversal, inadequate backwater, surcharging, and pressure flow formulation). A new version patterned after the Corps of Engineers STORM Model is now available for continuous simulation, but this version was completed too late for evaluation.

5. Hydrocomp Simulation Program for single-event and continuous wastewater flow and quality analysis provided the hydraulic limitations of the model are acceptable (approximate backwater and downstream flow control formulation, neglect of flow reversal, surcharging, and pressure flow).

6. Massachusetts Institute of Technology Urban Watershed Model for single-event flow analysis provided the hydraulic limitations of the model (neglect of backwater, downstream flow control, backwater, flow reversal, surcharging, and pressure flow), or the use of a separate model for these phenomena is acceptable.

7. Seattle Computer Augmented Treatment and Disposal System as an example of an operating real-time control system to reduce untreated overflows. A more comprehensive computer model simulating both wastewater flow and quality and including mathematical optimization should be considered, however, for new systems.

8. SOGREAH Looped Sewer Model for single-event wastewater flow and quality analysis considering all important hydraulic phenomena.

9. Water Resources Engineers Stormwater Management Model for single-event wastewater flow and quality analysis considering most important hydraulic phenomena.

The remaining reviewed models do not appear to provide sufficient special features which are not included in the models mentioned above. Their use may be advantageous, nevertheless, for certain applications where model assumptions are adequate, and especially where assistance from the model developers is easily available.

Following are brief descriptions of each model to give a quick overview of the available models. Detailed reviews and numerical test results are presented in Sections V and VII through IX.

BATTELLE URBAN WASTEWATER MANAGEMENT MODEL

The Battelle Urban Wastewater Management Model is intended primarily for the simulation of large urban catchments. It simulates the time-varying runoff and water quality in combined sewerage systems consisting of several catchments and a converging branch sewer network. Up to seven conservative water quality constituents can be modeled. The model can be used for a real-time control of overflows during rainstorms and for least-cost design of sewerage system modifications. The model is limited to the simulation of single runoff events.

For real-time control of overflows during rainstorms, diversions at controllable regulators are computed using dynamic programming to maximize the utilization of available sewer, storage and treatment capacities and minimize pollutant overflows. For design studies, sizes of sewers, overflow storage facilities, treatment plants, and overflow treatment facilities are computed which will minimize costs for specified constraints on the quality of overflows and treatment plant effluents.

The model includes special provisions which considerably reduce the input data requirements and computer running time. Data need to be defined only for typical urban subcatchment elements, rather than all elements, and the appropriate hydrologic computations are performed only for these typical elements. The flow routing, however, considers the actual location of the elements.

The model can be used in different modes, using various combinations of flow simulation, water quality simulation,

real-time overflow control, and least-cost design optimization. Program improvements are necessary for the efficient implementation of the real-time control optimization.

The model has been verified on very limited data. The numerical testing indicated that the model is satisfactory for conditions of slowly varying runoff (where time steps of not less than 5-minutes are adequate), and where downstream flow control, backwater, flow reversal, surcharging, and pressure flow are insignificant.

Comprehensive error diagnostics and the simulation of additional sewerage system components were programmed in versions being used by Watermation, Inc., a consulting firm in St. Paul, Minnesota.

BRITISH ROAD RESEARCH LABORATORY MODEL

The British Road Research Laboratory Model simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a converging branch sewer and open channel network. The model computes surface runoff only from impervious areas and neglects the contribution of pervious areas. The model is limited to the simulation of single runoff events. Water quality, real-time control and design features are not included.

The model has been tested extensively with urban hydrologic data. The model appears to produce satisfactory results for drainage areas of less than 13 km^2 (5 mi^2) if the impervious areas directly connected to the storm drainage system comprise more than 15 percent of the drainage area, and if the frequency of the storm is less than 20 years. The model requires a minimum of input data, is easy to use, and provides a fairly accurate means of computing runoff from the paved areas of urban catchments. Downstream flow control, backwater, flow reversal, surcharging, pressure flow, and diversion structures are not modeled.

The Illinois State Water Survey developed a new model based on the British Road Research method which considers also the runoff contribution from pervious areas and includes a design option to size retention basins or circular sewers (Illinois State Water Survey Urban Drainage Area Simulator).

CHICAGO FLOW SIMULATION PROGRAM

The Chicago Flow Simulation Program is intended primarily for the continuous simulation of large catchments consisting of both sewered and nonsewered areas. It simulates the time-varying runoff in combined sewerage systems and nonurban

drainage basins consisting of several catchments and a converging sewer and natural channel network. The flow routing formulation for natural channels includes provisions for flow and storage in floodplains. The model can be used for continuous simulation using hourly or smaller time steps. Water quality, real-time control, and design features are not included.

The model is very easy to use, but limited in its applicability by the assumptions inherent in the precipitation-runoff computations which neglect catchment shape, slope and surface roughness for sewered areas and impervious areas of nonsewered areas and assume a constant surface roughness for all pervious areas of nonsewered areas. Runoff and routing constants are internal to the program and may have to be changed for application of the model in different areas. Downstream flow control, backwater, flow reversal, surcharging, pressure flow, and diversions are not modeled. Model testing on mostly nonurban areas ranging in size from 12 to 264 km^2 (4.7 to 102 mi^2) produced generally satisfactory results.

CHICAGO HYDROGRAPH METHOD

The Chicago Hydrograph Method simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a converging sewer and open channel network. The model computes diameters of circular pipes for peak flows. Although it includes formulations for continuous catchment moisture accounting, it can be used only for the simulation of single runoff events due to computer program limitations. Water quality and real-time control are not included.

The model includes considerable simplifications in the routing of overland, gutter, sewer, and channel flow which would appear to limit its accuracy and are not necessary considering present state-of-the-art of flow routing techniques. Downstream flow control, backwater, flow reversal, surcharging, pressure flow, and diversions are not modeled. The model is useful primarily for the design of circular sewer pipes. No model testing using measured urban runoff data is reported by the model developers.

Separate versions have recently been completed for the continuous simulation of both wastewater flow and quality using one-hour time steps (Chicago Runoff and Pollution Model).

COLORADO STATE UNIVERSITY URBAN RUNOFF MODELING

The Colorado State University urban runoff modeling efforts are included in this review because special research activities are expected to provide new and improved techniques for the

control of overflows and the simulation of nonsteady flow in pipes. Two parallel efforts are being conducted. One effort is concerned with the development of concepts for the real-time control of overflows and the other with physical and numerical modeling of nonsteady flow in circular pipes.

No computer programs have been developed under the real-time control research effort which can be used directly for real-time applications. Valuable concepts evolved under this program, however, related to the future direction of urban wastewater management and the development of automated real-time wastewater control systems.

Various explicit finite difference solutions of the dynamic wave equations for gradually varied nonsteady open channel flow were investigated. The programs are not set up for pipe networks and diversion structures but merit consideration for incorporation into comprehensive urban runoff simulation models.

CORPS OF ENGINEERS STORM MODEL

The Storage, Treatment and Overflow Model (STORM) of the Corps of Engineers Hydrologic Engineering Center is intended primarily for the evaluation of stormwater storage and treatment capacity required to reduce untreated overflows below specified values. The model can simulate hourly stormwater runoff and quality for a single catchment for several years. Five water quality constituents are computed for different land uses: suspended and settleable solids, biochemical oxygen demand, nitrogen, and phosphorus.

The model does not route the stormwater runoff and quality in a sewer or channel network. Computations of treatment, storage and overflow proceed on an hourly basis by simple runoff volume and pollutant mass balance for the entire catchment. If the hourly runoff exceeds the treatment capacity, the excess runoff becomes untreated overflow. If the runoff is less than the treatment capacity and water is in storage, then the excess treatment capacity is utilized to diminish the storage volume.

The model appears useful primarily for preliminary planning studies to estimate approximate magnitudes of untreated stormwater overflows for various combinations of storage and treatment capacities. The model does not consider costs, however, and repeated runs with different capacities are required to determine an optimal combination meeting constraints on overflows. The model is limited in its application to stormwater drainage systems since it does not consider dry-weather flow and quality.

DORSCH CONSULT HYDROGRAPH-VOLUME METHOD

The Dorsch Consult Hydrograph-Volume Method simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a sewer and open channel network, including loops and converging and diverging branches. The flow routing is based on the dynamic wave equations and simulates backwater, surcharging and pressure flow but not flow reversal. The model also simulates retention basins and diversion structures. The model is limited to the simulation of single runoff events. Water quality, real-time control and design features are not included. A separate model for continuous simulation of wastewater flow and quality has just been completed (Quantity-Quality Simulation Program).

This is one of the most complete models for the computation of runoff from urban catchments and the routing of flows in sewer networks. The implicit solution of the dynamic wave equations provides an accurate means of computing flow routing considering both upstream and downstream boundary conditions and special hydraulic structures.

This is a proprietary model of Dorsch Consult of Munich, Germany, which maintains a North American office in Toronto, Canada. The model has been applied extensively, and model testing has shown good agreement between measured and computed runoff.

ENVIRONMENTAL PROTECTION AGENCY STORMWATER MANAGEMENT MODEL

The Stormwater Management Model of the U.S. Environmental Protection Agency is one of the most comprehensive mathematical models for the simulation of storm and combined sewerage systems. It computes the combined storm and sanitary runoff from several catchments and routes the flows through a converging branch sewer network. It can model two types of flow diversion structures, three storage basins, and one overflow treatment plant.

Suspended and settleable solids, biochemical and carbonaceous oxygen demand, coliform bacteria, nitrogen, phosphorus, and oil and grease are modeled, and the performance and cost of nine unit treatment processes can be computed. A receiving water segment of the model computes the flows and water quality impact of sewerage system effluents in receiving waters. The model does not include real-time control. The model is limited to the simulation of single runoff events. The model includes an option to size sewers to eliminate surcharging, but cannot adequately simulate downstream flow control, backwater, surcharging, pressure flow and flow reversal.

Segments of the model have been tested on different sets of data since adequate data of rainfall, runoff and water quality were not available for a single catchment to test the complete model. Testing of the runoff flow computations on catchments ranging in size from 5 to 2200 ha (13 to 5400 acres) showed good agreement with measured values. The accuracy of the water quality computations, particularly the formulations relating water quality with land use, has not been sufficiently established to be used with confidence for prediction purposes.

The computer program is very complex and requires a major effort for its implementation. The input data are arranged in logical groups, but improvements in the user's manual and documentation of the model's theoretical bases is needed to understand the meaning and use of some data. Program improvements, addition of new model features, and a new user's manual are in progress at the University of Florida. Special workshops are being held periodically to instruct potential users in model use, and assistance is available from the Environmental Protection Agency and private consulting firms for model implementation.

HYDROCOMP SIMULATION PROGRAM

The Hydrocomp Simulation Program is an improved version of the Stanford Watershed Model which was the first comprehensive mathematical model of catchment hydrology. Recently, a separate program was developed for the simulation of water quality in river basins and interfaced with the hydrologic program. The program is formulated for the continuous simulation of both water flow and quality from several catchments and the routing in a converging branch sewer and open channel network. Although originally developed for nonurban areas, modifications give the program the capability to simulate both sewered and nonsewered areas. The water quality model simulates 17 water quality constituents, including their reactions and interactions in natural water bodies. The model does not include wastewater treatment, real-time control, design, and cost computations.

The addition of special sewerage system phenomena and elements is needed to provide the capability to simulate comprehensive networks. The hydrology formulation may be more complicated than needed for normal application to urban catchments considering the lack of available data for calibration purposes.

A major advantage of the model is the capability for continuous simulation of both water flow and quality in complex networks since it considers both catchment moisture and water

quality accounting between runoff events and allows time intervals from a few minutes to bi-monthly. The hydrologic model has been tested and applied extensively in the United States and abroad in both nonurban and urban catchments. Testing of the water quality model is in progress.

This is a proprietary model of Hydrocomp International, Inc. of Palo Alto, California. User's manuals are available, and user's workshops are held periodically.

MASSACHUSETTS INSTITUTE OF TECHNOLOGY URBAN WATERSHED MODEL

The Massachusetts Institute of Technology (MIT) Urban Watershed Model simulates the time-varying runoff of several catchments and a sewer and open channel network including loops and converging and diverging branches. The model is limited to the simulation of single runoff events. Water quality and real-time control features are not included.

A separate model computes the sizes and costs of sewers, storage and treatment facilities which will result in the least-cost combination of alternatives for the elimination of untreated overflows and the reduction of flooding and surcharging. A method based on filter theory can be used to compute the infiltration coefficients from measured rainfall and runoff. The original models were developed at MIT under various projects for the U.S. Office of Water Resources Research but the models have been modified by Resource Analysis, Inc., for routine applications. The user has to contract with Resource Analysis, Inc. of Cambridge, Massachusetts, for routine application of the complete and revised program package.

The model includes special provisions which considerably reduce the input data requirements and computer running time. Data need to be defined only for typical urban subcatchment elements, rather than all elements, and the appropriate hydrologic computations are performed only for these typical elements. The flow routing, however, considers the actual location of the elements.

Testing on 9 ha (23 acres) to 120 km^2 (46 mi^2) catchments produced good agreements between measured and computed runoff values. The model is being used extensively for engineering assessments.

MINNEAPOLIS-ST. PAUL URBAN RUNOFF MODEL

The Minneapolis-St. Paul Urban Runoff Model was developed for real-time forecasting of flows in the major trunk and

interceptor sewers of the Minneapolis-St. Paul combined sewerage system. Its purpose was to compute regulator settings which would reduce untreated overflows to the Mississippi River during rainstorms. The model computes the runoff of several large catchments, diverts the flows at controllable regulating structures, and routes them through a converging branch sewerage network to the treatment plant. The model is specifically designed for real-time control operation on a small computer and cannot be used easily for the assessment of existing or the design of new sewerage systems since it uses a highly simplified flow routing procedure whose coefficients have to be calibrated with measured data or derived from more sophisticated models.

Although the model was designed for continuous simulation by incorporating provisions for the accounting of catchment moisture between rainstorms, the necessary formulations were never added, and the program can be used only for the simulation of single runoff events. The model is no longer used for real-time control, however, since the required trial and error procedure with estimates of regulator settings is too time-consuming, and operator experience with the system appears to be more efficient for the control of the regulators. The model does not include water quality, design and cost computations.

SEATTLE COMPUTER AUGMENTED TREATMENT AND DISPOSAL SYSTEM

The Computer Augmented Treatment and Disposal System (CATAD) of the Municipality of Metropolitan Seattle (Metro) is an operating system for real-time control of untreated overflows from the main trunk and interceptor sewer regulators of the metropolitan Seattle, Washington, combined sewerage system. The system does not include a comprehensive mathematical model for the simulation of runoff from several catchments and the routing of flows in a sewerage network. Present system operation is based on computer control of regulators and pumping stations using real-time data acquisition during rainstorms and rule curve operation. Water quality is monitored but not utilized by the scheme. Manual override is possible for alarm conditions.

Evaluation of the control scheme has shown significant reductions in untreated overflows compared to uncontrolled conditions. The scheme's effectiveness is impaired, however, since comprehensive optimization is not used to maximize utilization of available sewer flow and storage capacity and to consider the effect of the quality of the overflowed wastewaters on the receiving water quality.

SOGREAH LOOPED SEWER MODEL

The Looped Sewer Model of the French consulting firm Societe Grenobloise d'Etudes et d'Application Hydrauliques (SOGREAH) simulates the time-varying runoff of combined sewerage systems consisting of several catchments and a sewer and open channel network including loops and converging and diverging branches. The model includes formulations for most hydraulic phenomena encountered in closed conduit and open channel networks. The flow routing solves the dynamic wave equations coupled with equations for special sewer system facilities, such as diversion structures, pumping stations, inverted siphons, and retention basins. The solution considers both upstream and downstream boundary conditions, backwater, flow reversal, surcharging, and pressure flow. The model has been expanded recently to include the advective transport of pollutants. The model does not include real-time control, design and cost computations.

The model routing scheme is based on a river basin model developed by SOGREAH earlier. The firm did not report verification of the sewer model with urban hydrologic data since the river basin model verification apparently produced satisfactory results. This is a proprietary model of SOGREAH of Grenoble, France. An office is maintained in New York City. The firm is also represented by Lasalle Hydraulic Laboratory in Canada.

UNIVERSITY OF CINCINNATI URBAN RUNOFF MODEL

The University of Cincinnati Urban Runoff Model simulates the time-varying runoff of storm sewerage systems consisting of several catchments and a converging branch sewer and open channel network. The model does not include provisions for dry-weather flow, water quality, real-time control, and design. It is limited to the simulation of single runoff events.

The model includes several simplifications which appear to reduce the model's accuracy and applicability. These include the assumption of areal uniformity of rainfall, the neglect of dry-weather or base flows, the computation of infiltration from rainfall rather than overland flow depth, the neglect of catchment moisture conditions in the infiltration computations, the steady-state formulation for gutter flow routing, and the neglect of dynamic effects in the sewer flow routing.

Although some encouraging results are reported by the model developers on a 5.2 ha (12.9 acre) and 964 ha (2380 acre) catchment, the tests were restricted to fairly simple rainfall events and required considerable calibration of the infiltration rate coefficients. Testing by others on a 70 ha (173 acre)

and 500 ha (1240 acre) catchment in Australia using more complex storm patterns showed considerable differences between measured and computed runoff values.

UNIVERSITY OF ILLINOIS STORM SEWER SYSTEM SIMULATION MODEL

The University of Illinois Storm Sewer System Simulation Model computes nonsteady flows in a converging sewerage network based on a solution of the dynamic wave equations. The solution considers upstream and downstream flow controls, backwater, and flow reversal in circular sewers and in-line storage at sewer junctions. The model does not compute dry-weather flows from land use or runoff from precipitation but requires inflow hydrographs to the sewers as input data. Separate general hydrologic catchment models have been developed, however, and research for the development of an urban hydrologic model is in progress for potential interfacing with the flow routing model. The routing model includes a feature for the sizing of circular pipes to accommodate peak flows. Costs are not considered. The model does not include water quality and real-time control features. A separate model exists for sewer design using dynamic programming with cost and risk considerations.

The model has not been verified with real sewerage network data or applied to real systems. Testing of the routing scheme with experimental pipe data indicates, however, that high accuracy can be expected.

UNIVERSITY OF MASSACHUSETTS COMBINED SEWER CONTROL SIMULATION MODEL

The University of Massachusetts Combined Sewer Control Simulation Model simulates the time-varying runoff of several catchments and a single string of circular sewers. The model computes runoff from impervious areas only using hourly rainfall data. This restricts the model to the simulation of runoff contributions from pervious areas. A separate computer program is available to compute synthetic hourly rainfall from recorded rainfall using a Markov chain model.

The flow routing is accomplished with an implicit solution of the dynamic wave equations which considers upstream and downstream flow control, backwater, and flow reversal. Special sewerage system facilities, such as diversion and flow control structures and storage facilities, are not modeled. The model is formulated for the continuous simulation of runoff for periods of up to one month, but neglects catchment moisture balance between runoff events and snow accumulation and melt. Water quality, real-time control, and design features are not included.

The model has not been tested with real catchment data. Testing with hypothetical data indicated that the combination of hourly runoff computation from rainfall with the implicit flow routing scheme may be a practical combination for the simulation of large catchments and trunk and interceptor sewers where short-duration peak discharges may be negligible. Considerable additions would be needed to the program to make it generally applicable, such as the computation of runoff from pervious areas, the consideration of a time lag between rainfall and runoff, the simulation of evapotranspiration and snow accumulation and melt, and the expansion of the routing scheme to sewer networks and special sewerage system facilities.

WATER RESOURCES ENGINEERS STORMWATER MANAGEMENT MODEL

The Stormwater Management Model of Water Resources Engineers, Inc. (WRE) is a modified version of the Stormwater Management Model of the Environmental Protection Agency. The model simulates the time-varying combined storm and dry-weather runoff and wastewater quality of several catchments and a sewer and open channel network, including loops and converging and diverg ing branches. The flow routing is based on the dynamic wave equations and considers both upstream and downstream flow control, backwater, and flow reversal. The solution is coupled with hydraulic equations for flow control and diversion structures and pumping stations. A special formulation computes surcharging and pressure flow independently for each junction.

Both dry-weather and stormwater quality are computed for 23 constituents: suspended and settleable solids, biochemical oxygen demand, nitrogen, phosphorus, oil and grease, and 17 arbitrary conservative constituents. The pollutants are routed through the sewerage system, but treatment processes are not modeled. The model does not include real-time control, design, and cost computations. It is limited to the simulation of single runoff events. A separate model for receiving water flow and quality is available but it is not interfaced with the WRE Stormwater Management Model.

The model includes special provisions which considerably reduce the input data requirements and computer running time. Data need to be defined only for typical urban subcatchment elements, rather than all elements, and the appropriate hydrologic computations are performed only for these typical elements. The flow routing, however, considers the actual location of the elements.

The model represents a very comprehensive formulation of sewerage system flow phenomena coupled with water quality computations. The explicit solution of the dynamic wave

equations requires very small time steps and is time-consuming on the computer. The model would be needed where the simulation of most modeled hydraulic phenomena is important and water quality routing is desired. The computation of water quality from land use characteristics has not been sufficiently verified.

The model was released to the City of San Francisco and is now in the public domain. A European office is maintained by Water Resources Engineers in Hamburg, Germany.

WILSEY AND HAM URBAN WATERSHED SYSTEM

The Wilsey and Ham Urban Watershed System computes the time-varying stormwater runoff of several catchments and a converging branch sewerage network. A design option sizes circular pipes for peak flows. The model does not include provisions for dry-weather flow, water quality, and real-time control. A special feature routes catchbasin inflow exceeding the free flow capacity of the sewer in the gutter to the next downstream catchbasin. Downstream flow control, backwater, flow reversal, surcharging, and pressure flow are not modeled. The model does not simulate flow control and diversion structures. The model is limited to the simulation of single runoff events.

The model includes special provisions which considerably reduce the input data requirements and computer running time. Data need to be defined only for typical urban subcatchment elements, rather than all elements, and the appropriate hydrologic computations are performed only for these typical elements. The flow routing, however, considers the actual location of the elements.

Although sufficient details are not available on the mathematical formulations of the model, it appears to be an efficient model for the evaluation and design of small storm sewerage systems for which the limitations of the flow routing scheme are acceptable. The program has been applied extensively but testing with real catchment data to evaluate model accuracy has not been reported.

This is a proprietary model of Wilsey and Ham, Inc. of Foster City, California.

SECTION II

RECOMMENDATIONS

CONTENTS

	Page
Introduction	37
Historical Data Collection	37
Future Data Measurements	37
Model Development	38
Model Formulations	39
Hydrologic Features	39
Hydraulic Features	40
Water Quality Features	41
Miscellaneous Features	42

INTRODUCTION

Various recommendations for urban hydrologic data collection and mathematical model development can be formulated from the model reviews and numerical testing. The recommendations are intended to guide future development, testing and application of mathematical models for the engineering assessment, planning, design and control of storm and combined sewerage systems.

HISTORICAL DATA COLLECTION

The scarcity of reliable urban hydrologic data considerably limited the testing of model accuracy. To provide a better data base, the University of Florida is currently collecting urban catchment characteristics, rainfall, runoff, and water quality. The data base will make it easier for developers and the urban planning and engineering community to exercise and verify existing or new urban hydrologic models. The program includes indentification of data sources, establishment of data reliability and accuracy, data collection, arrangement of the data in common formats, and development of a mechanism for updating and disseminating the data.

FUTURE DATA MEASUREMENTS

The difficulties encountered in obtaining reliable comprehensive data for the testing of the models reviewed in this study indicate the need for future data collection programs to:

- establish a new data measurement program for selected urban catchments;
- include urban experimental catchments comprising all land uses of interest;
- locate these catchments in different climatic zones;
- select catchments where the runoff and water quality can be identified and measured separately for each major land use category;
- measure all physical characteristics of a catchment which influence storm and combined sewer runoff and water quality;
- measure all data categories required by the most sophisticated mathematical model that may be needed for storm and combined sewerage system assessments;

- measure time-varying data at sufficiently short intervals to identify significant variations in weather, rainfall, runoff, and water quality;

- measure not only storm runoff, but also dry-weather flow and snowmelt (including water quality); and

- establish a rigid quality control program to assure consistent data accuracy.

MODEL DEVELOPMENT

The wide variety of model formulations for similar purposes indicates the need for future model development to:

- identify major use categories and establish model criteria for each use (i.e., engineering assessment, planning, design, and control);

- identify model criteria as a function of sewerage system size and spatial and temporal variability of phenomena to be modeled;

- develop a standard set of models for each major use category;

- develop a model for long-term (over many years) planning purposes;

- develop an efficient, economical design model for simultaneous sizing of sewers, open channels, retention basins, overflow regulators, and treatment plants;

- disseminate information about existing efficient steady-state models for street sewer design of small subsystems;

- develop an efficient real-time control model for reducing combined sewer overflows which will consider both the flow and quality of the overflows and run on a small process computer;

- develop a set of models which does not include water quality to reduce model complexity for applications where water quality computations are not needed;

- develop criteria for mathematical formulations of each modeled phenomenon;

- develop model testing criteria to form a common basis for future model comparisons;

- interface models with current sophisticated receiving water quality models which consider important interacting water quality constituents for river basins, estuaries, and deep stratified lakes and impoundments;

- interface urban hydrologic models that include water quality simulations with sewage treatment plant models;

- establish more reliable correlations or functional relationships between specific land uses and the quality of storm and sanitary runoff;

- investigate water quality changes in sewers and retention basins and formulate appropriate mathematical equations for significant physical/chemical/biological reactions and interactions;

- investigate the effect of numerical dispersion of the selected mass transport scheme and provide guidelines to relate the numerical dispersion to actual hydrodynamic dispersion and molecular diffusion; and

- develop consistent guidelines for model documentation and testing to facilitate model comparisons by interested model users.

MODEL FORMULATIONS

Implementation of the following recommendations would make the models more generally applicable and easier to evaluate objectively. It would also eliminate many simplistic approaches which were developed prior to the common availability of computers and are no longer justified in computer models.

Hydrologic Features

Model formulations should:

- include options to read in either cumulative rainfall or rain intensities at irregular time intervals to eliminate time-consuming hand conversions of commonly available data;

- provide the capability in models designed for large catchments to read in data from more than one weather station and to compute a weighted average for the catchment or any subcatchment;

- include formulations for catchment moisture accounting during periods of no rainfall, considering depletion of soil moisture and depression storage;

- formulate infiltration as a function of actual soil moisture to make the model applicable to all rainfall conditions, not only to events for which the rainfall exceeds the potential infiltration capacity;

- compute infiltration from overland flow depth rather than rainfall intensity;

- consider the actual distance and time of overland flow rather than the catchment length (which may be different) to compute infiltration;

- compute rainfall losses and overland flow routing separately for pervious and impervious areas; and

- consider the locations of impermeable areas in relation to permeable areas in the computation of overland flow, if overland flow is not routed separately for each.

Hydraulic Features

Model formulations should:

- include formulations for backwater, flow reversal, downstream flow control, surcharging, pressure flow, converging and diverging branches, loops, and special sewer elements such as regulators, pumping stations, retention basins, and tide gates in flow routing schemes for complex combined sewerage systems;

- replace linear storage routing by the kinematic wave equation for conditions where backwater, flow reversal, downstream hydraulic control, and surcharging are insignificant; the approximations of linear storage routing schemes are not justified for computer models;

- print warning statements in models which do not simulate backwater, flow reversal, downstream hydraulic control, and surcharging if these conditions occur;

- formulate models which solve the dynamic wave equation to simulate converging, diverging and looped networks in any combination;

- compare explicit and implicit solutions of the dynamic wave equations more thoroughly for large systems with respect to accuracy and efficienty of computation for flow routing where backwater, flow reversal, downstream hydraulic control, and surcharging need to be modeled;

- simulate the propagation of surcharging in the conduit network instead of computing it for each conduit independently;

- include formulations in comprehensive models for standard conduit and open channel cross sections and the option to specify any arbitrary cross section by appropriate input data;

- base the performance of standard diversion structures (including overflow regulators) on the appropriate hydraulic equations or tables derived from them;

- include provisions in comprehensive programs to add subroutines for various types of pumping stations, diversion structures, retention basins, and retention basin outlet structures; and

- include provisions to read in operating rules for controllable regulator, pumping station, and storage basin operation.

Water Quality Features

Model formulations should:

- consider hourly, daily and seasonal variations in dry-weather flow and water quality formulations;

- consider seasonal variations in stormwater quality formulations;

- include formulations for catchment pollutant accounting during periods of no precipitation (for continuous models with water quality simulation);

- simulate scour and sedimentation in conduits, open channels, and storage basins in models simulating pollutant transport; and

- simulate reactions and interactions of pollutants in storage basins.

Miscellaneous Features

Model formulations should:

- include input of initial conditions in continuous simulation models rather than assuming them to be zero or some other fixed values;

- incorporate a scheme defining typical catchment and sewerage system elements to reduce duplication of input data and runoff simulation;

- provide the option to obtain output of water flow, depth, velocity, and water quality concentrations (if simulated) for any simulated sewerage system element;

- list dimension limitations for all arrays to indicate the number of sewer elements, time steps, etc., that can be simulated;

- provide test data deck and output listing for all model options;

- separate model documentation into theoretical description, model testing, user's manual, and programmer's manual to facilitate use of the model by individuals with different interests; the user's manual should be concise in order to instruct technicians in setting up data and interpreting output; it should only refer to the model's theoretical bases when necessary to clarify use instructions; and

- provide a mathematical expression, or its equivalent defining computer running time as a function of modeled sewerage system size and length of simulated time period.

SECTION III

INTRODUCTION

In the past, the most common methods for analyzing and designing urban storm and combined sewerage systems have been based on the rational formula for rainfall-runoff computations and steady-state flow equations for sewer network flow analyses. Recognizing the limitations of these methods for highly complex systems under dynamic flow conditions, mathematical models have been developed in recent years to provide better tools for the analysis of existing and planned sewerage systems and system improvements.

This study evaluates mathematical models which simulate dynamic wastewater flow and quality conditions for engineering assessment, control, planning and design of storm and combined sewerage systems. The most promising models for practical application were tested during the study using hypothetical and real urban catchment data. Evaluations were made on the bases of accuracy, cost of use, computer requirements, data requirements, input data preparation requirements and output options available to the user. The purpose of the evaluations is to help the practicing engineer decide which of the models will meet his requirements.

Mathematical models of catchment hydrology are available which handle all or any combination of the following phenomena:

- Catchment runoff
- Flow routing
- Water quality
- Costs

Some of the models, however, have been developed for nonurban catchments and are not directly applicable to urban catchments without major revisions to account for special urban hydrologic

phenomena such as runoff from pervious and impervious areas, flow routing in closed conduits, and wastewater quality.

Eighteen models were identified which were specifically developed or adapted for the dynamic simulation of urban catchments. All but two of these models include the nonsteady simulation of the rainfall-runoff process and most models include flow routing in the sewers. Several models include also the continuous simulation of wastewater quality. Some models include options for dimensioning sewer pipes and two of them use mathematical optimization schemes for least-cost design of new sewerage system components. Three models have provisions for the real-time control of overflows during rainstorms.

A brief review of these models indicates a tremendous diversity in scope and purpose, mathematical detail, system elements and hydrologic phenomena being modeled, size of the system that can be handled, data input requirements, and computer output. This diversity, of course, is a result of the varying conditions and objectives which govern the design and evaluation of individual sewerage systems.

The state of development of these models also varies significantly. Some models have been developed and verified extensively, others have been developed but not verified, while some have been conceptualized but not carried to the point of application. In addition, because no standards exist for evaluating and comparing models, differing criteria have been used in the evaluations. The rigor with which the models were tested varies greatly, from intuitive judgments to graphical comparisons and more demanding statistical analyses.

These considerations have hindered both the technology transfer to interested municipalities and consulting engineers and the routine use of these new methodologies. In fact, due to a lack of information on data input requirements, model limitations, and program output options combined with often-overstated claims of capabilities, use of this advanced methodology has received some criticism.

The objective of this state-of-the-art survey is therefore to provide the engineer and planner with a readily available reference containing brief but precise descriptions and evaluations of all the models available, thus facilitating his selection of the appropriate model for a particular application.

SECTION IV

MODEL SELECTION

CONTENTS

	Page
Introduction.	46
Criteria for Model Selection.	46
Selected Models	47

INTRODUCTION

This study was limited to a review of nonsteady simulation models which can be used for the engineering assessment, planning, design, and control of storm and combined sewerage systems. Engineering assessment involves the evaluation of system performance under various hydrologic and wasteload conditions to determine problem areas and needs for system improvement. This may include the determination of undesirable surcharging, backwater and flooding conditions, the volume and quality of overflows during rainstorms, and the performance of storage facilities and treatment plants. For planning purposes, the effects of both existing and potential hydrologic conditions and land use decisions on existing and planned sewerage systems and system components would be studied.

Design involves the computation of sizes of sewers, flow control and diversion structures and storage and treatment facilities which will meet specified performance criteria. Further considerations of sewerage system design may include elimination of undesirable flooding, reduction of untreated overflows, and better use of existing and planned facilities.

Control involves the regulation of flows to reduce overflows and improve system efficiency. It may include the regulation of controllable diversion structures and the operation of storage reservoirs and treatment plants during rainstorms to maximize system capacity and minimize untreated discharges to receiving waters. The need for potential system improvements and implementation of real-time control schemes for manual or fully automatic control of wastewater flows, storage and treatment may also be considered in this phase.

CRITERIA FOR MODEL SELECTION

To serve the purposes described above, mathematical models must consider the spatial nonuniformity of rainfall; the time-varying runoff resulting from rainstorms of different intensities and durations; spatial and temporal variations in dry-weather flows; the effect on hydrograph shapes of different flow travel times from various catchments; the attenuation of flood peaks during overland, gutter, and sewer conduit flow routing; and the operation of flow diversion structures and storage facilities under dynamic wasteload conditions. The models should be able to combine the runoff from several catchments and route the wastewaters within the sewer networks.

Most of the eighteen selected models meet these requirements. The few exceptions included in the study are models with special provisions or advantages for the evaluation of important

aspects of urban catchments and sewerage systems, or with features which could improve existing models or provide new modeling capability.

All eighteen models were evaluated on the basis of published information and communication with model developers. In addition, seven of the more comprehensive models were selected for testing by computer runs using hypothetical and real catchment data.

In selecting these models, a minimum requirement was the capability to consider several raingages, to compute runoff from several catchments, and to route flows in a converging branch sewer network. Models which rely heavily on mathematical formulations which cannot be derived readily from catchment and sewer physical characteristics were not tested, nor were models whose oversimplification restricts their use unnecessarily, considering present computer capabilities and the state-of-the-art of hydrologic modeling. Among the proprietary models selected, one firm did not respond to the request to participate in the numerical testing.

SELECTED MODELS

The eleven models which were evaluated solely on the basis of published information in reports by model builders and model users are:

1. British Road Research Laboratory Model
2. Chicago Hydrograph Method
3. Colorado State University Urban Runoff Modeling
4. Corps of Engineers Hydrologic Engineering Center STORM Model
5. Hydrocomp Simulation Program
6. Minneapolis-St. Paul Urban Runoff Model
7. Seattle Computer Augmented Treatment and Disposal System
8. University of Cincinnati Urban Runoff Model
9. University of Illinois Storm Sewer System Simulation Model
10. University of Massachusetts Urban Runoff Model
11. Wilsey & Ham Urban Watershed Model

The seven models which were also tested by computer runs are:

1. Battelle Urban Wastewater Management Model

2. Dorsch Consult Hydrograph-Volume Method

3. Environmental Protection Agency Stormwater Management Model

4. Massachusetts Institute of Technology Urban Watershed Model

5. Metropolitan Sanitary District of Greater Chicago Flow Simulation Program

6. SOGREAH Looped Sewer Model

7. Water Resources Engineers Stormwater Management Model

A review of all eighteen models is presented in Section V of the report, which includes a summary description of each model, a list of the phenomena considered, brief outlines of the mathematical formulations with comments on their advantages and shortcomings, and comments on the computer program and output options. The theoretical background equations of the seven models selected for numerical testing are presented in Appendix C. The hypothetical and real catchment data used for the numerical testing are given in Section VI and the numerical results and evaluations in Sections VII to IX.

SECTION V

MODEL REVIEWS

CONTENTS

	Page
Introduction	53
Battelle Urban Wastewater Management Model	53
Summary	53
Methods	55
Computer Program	59
Evaluation	60
British Road Research Laboratory Model	60
Summary	60
Methods	61
Computer	62
Evaluation	63
Chicago Flow Simulation Program	63
Summary	63
Methods	64
Computer Program	66
Evaluation	66
Chicago Hydrograph Method	66
Summary	66
Methods	68
Computer Program	69
Evaluation	70

CONTENTS (Continued)

	Page
Colorado State University Urban Runoff Modeling	70
Summary.	70
Methods.	71
Computer Programs.	72
Evaluation	72
Corps of Engineers Storm Model.	73
Summary.	73
Methods.	74
Computer Program	75
Evaluation	76
Dorsch Consult Hydrograph-Volume Method	76
Summary.	76
Methods.	78
Computer Program	79
Evaluation	80
Environmental Protection Agency Stormwater Management Model.	81
Summary.	81
Methods.	83
Computer Program	87
Evaluation	88
Hydrocomp Simulation Program.	89
Summary.	89
Methods.	92
Computer Program	94
Evaluation	94
Massachusetts Institute of Technology Urban Watershed Model	95
Summary.	95
Methods.	97
Computer Program	98
Evaluation	98

CONTENTS (Continued)

	Page
Minneapolis-Saint Paul Urban Runoff Model	98
Summary	98
Methods	100
Computer Program	101
Evaluation	102
Seattle Computer Augmented Treatment and Disposal System	102
Summary	102
Methods	104
Computer Program	105
Evaluation	105
SOGREAH Looped Sewer Model	105
Summary	105
Methods	107
Computer Program	108
Evaluation	109
University of Cincinnati Urban Runoff Model	110
Summary	110
Methods	111
Computer Program	112
Evaluation	112
University of Illinois Storm Sewer System Simulation Model	112
Summary	112
Methods	114
Computer Program	114
Evaluation	115
University of Massachusetts Combined Sewer Control Simulation Model	116
Summary	116
Methods	117

CONTENTS (Continued)

Page

Computer Program 118

Evaluation 118

Water Resources Engineers Stormwater Management Model 119

Summary. 119

Methods. 120

Computer Program 122

Evaluation 123

Wilsey and Ham Urban Watershed System 123

Summary. 123

Methods. 125

Computer Program 125

Evaluation 125

Other Models. 126

Chicago Runoff and Pollution Model 126

CH2M-Hill Wastewater Collection System Analysis Model 126

Dorsch Consult Quantity-Quality Simulation Program. 127

Illinois State Water Survey Urban Drainage Area Simulator 127

Norwegian Institute for Water Research Sewerage System Models 128

Queen's University Urban Runoff Model. 128

University of Nebraska Urban Hydrologic Simulator. 129

INTRODUCTION

Descriptions and reviews of principal model features are presented in this section to guide the potential model user in model selection. The general applicability of each model is summarized, the principal physical phenomena modeled are listed, the mathematical methods and computer programs are described, past model testing and applications are referenced, and brief evaluations are presented. Model equations are not included in this section to facilitate quick scanning of all model descriptions. Equations for the seven models tested with computer runs are included in Appendix C.

It is impossible to describe all the features of each model in a state-of-the-art review and to discuss every model advantage and limitation based solely on information provided by the model builders and selected numerical tests. This would be possible perhaps for evaluations of mathematical models of individual phenomena, but it becomes a tremendous task when the interactions of all the submodels of the comprehensive models in this study must be considered. Only extensive use and experimentation with a particular model will reveal all its advantages and shortcomings. It is hoped, however, that this review will provide sufficient information to the potential user to aid him in selecting the appropriate model for his needs. Once the user has decided that a particular model may be of interest, careful study of the documents referenced for the model is recommended before its computer implementation is begun. This would reveal model details which, although important, cannot be described sufficiently in this review.

The model comparisons emphasize evaluation of the complete model, rather than detailed critiques of submodels of individual phenomena. Formulations of individual phenomena are discussed only in the context of their use in the complete model and no attempt is made to discuss all available models for each phenomenon. As an example, a tremendous variety of models exist for simulating the rainfall-runoff process alone. No attempt was made to describe all of them. Rather, only those used in the comprehensive urban hydrologic models were reviewed. This type of evaluation provides the engineer with the information he needs to select a comprehensive model but does not require him to be familiar with the many available variations for modeling each urban hydrologic phenomenon.

BATTELLE URBAN WASTEWATER MANAGEMENT MODEL

Summary

The Battelle Urban Wastewater Management Model (Brandstetter et al., 1973) is intended primarily for the simulation of major

sewer system components, such as trunk and interceptor sewers, regulators, overflow storage facilities, and treatment plants. It provides a means of evaluating the time-varying performance of a planned or existing sewerage system under a variety of rainfall conditions, considering both the time and spacial varitions of rainfall rates without requiring the simulation of every small sewer, manhole, etc. in the system. Stormwater and sanitary runoff and quality of major catchment areas are computed using hydrologic lumping techniques to arrive at hydrographs and quality graphs at major regulators and at junctions of major trunks and interceptors. The runoff is then routed through trunk and interceptor sewers to the treatment plant. The model is limited to the simulation of single runoff events.

The model can simulate up to seven conservative wastewater quality parameters. Quality reactions, with the exception of changes at treatment facilities, are not modeled. The model determines the required diversion at control regulators during real-time rainstorm events in order to minimize wastewater discharges to receiving waters. The model can also be used for design and planning studies. It computes sizes and costs of structural sewer system modifications such as sewers, storage and treatment facilities which will result in the least-cost combination of alternatives for improving system performance.

The model has been improved and programmed for an IBM 360 computer by Watermation, Inc., a consulting firm in Saint Paul, Minnesota. Improvements include formulations for additional sewerage system elements, such as pumping stations, and the addition of new error diagnostics. The firm is using the model for engineering assessments and design purposes.

The Battelle Urban Wastewater Management Model includes the following features:

1. dry-weather flow and quality of several catchments;

2. several rainfall records;

3. stormwater runoff and quality from pervious and impervious areas of several catchments;

4. routing of combined wastewater flow and quality in a converging branch network;

5. circular closed conduits;

6. surcharging and pressure flow;

7. diversion of wastewater flow and quality at overflow regulators;

8. filling of overflow storage facilities;

9. wastewater quality improvement at treatment plants and overflow treatment facilities;

10. regulator diversions for optimal utilization of sewerage system capacities to minimize untreated overflows; and

11. least-cost sizes of sewers, overflow storage and treatment facilities, and main treatment plants which meet constraints on effluent quality.

The model does not include the following features:

1. dry-weather flow and quality and stormwater quality from land use characteristics;

2. evapotranspiration;

3. snow accumulation and melt;

4. catchment moisture and water quality balance during periods of no precipitation;

5. flow and quality routing in gutters;

6. flow and quality routing in loops and diverging branches;

7. noncircular closed conduits and open channels;

8. downstream flow control, backwater, and flow reversal;

9. emptying of overflow storage facilities;

10. inline storage facilities;

11. sedimentation and scour;

12. wastewater quality decay, reactions and interactions; and

13. receiving water flow and quality.

Methods

Dry-weather flow and quality is provided as input data in the form of average values for each catchment and adjustment factors to account for diurnal (hourly), weekday, and seasonal adjustments. The model does not include provisions to compute either

dry-weather flow or quality from land use. Storm runoff is computed from the weighted average rainfall of several raingages for each catchment. The record of each raingage is provided as input in the form of cumulative values and can be given at irregular intervals. This eliminates the need to compute intensities prior to running the program and to read in zeros for periods of no rain.

Losses are subtracted from the rainfall separately for the pervious and impervious areas of each catchment to obtain the excess precipitation contributing to surface runoff. Losses from impervious areas, such as wetting of surfaces and retention by storage in depressions, are assumed to decay exponentially with time as a function of catchment moisture conditions after an initial loss is satisfied. Losses from pervious areas include infiltration and the losses considered for impervious areas. They are approximated by a modified Holtan infiltration equation which considers an initial loss that must be satisfied and treats infiltration as a function of soil moisture content. Horton's equation, in contrast, if not modified, considers infiltration only as a function of time, regardless of changes in soil moisture conditions during the rainstorm. A disadvantage of the Battelle model formulation, however, is that infiltration is computed from rainfall rather than the depth of water on the catchment. Consequently, the coefficients of the infiltration equation must be calibrated to compensate for this approximation.

The excess precipitation is then convoluted with a unit hydrograph to determine the storm runoff from each catchment. The unit hydrographs are derived for ungaged catchments by a method developed by the Soil Conservation Service. They are computed using the physical catchment characteristics which include drainage area, length, slope, soil and vegetation. Refinement of these unit hydrographs is recommended if concurrent records of rainfall and runoff are available. For a given catchment, the same unit hydrograph is used for runoff from both pervious and impervious areas, which may lead to inaccuracies and calibration problems.

The advantage of the unit hydrograph method over more hydrodynamic methods of overland flow routing, such as the use of the kinematic wave equations, is that large drainage areas of up to several hundred acres can be treated as a single catchment for simulation purposes. This allows the simulation of large metropolitan sewerage systems without the need for defining each small sewer. The model is therefore particularly suited for the simulation of major trunk and interceptor sewers. The disadvantage of the unit hydrograph method is its approximation of the basically nonlinear runoff phenomenon by a linear model and the lack of verification for urban catchments. This is not unique to this method, however, and other more hydrodynamic methods

suffer from the same deficiency due to the shortage of reliable runoff data from urban catchments.

The quality of the storm runoff is computed using linear multiple regression equations which relate the concentration of each pollutant to the storm runoff rate, cumulative runoff during the storm, and initial conditions. The regression coefficients have to be provided as input data together with seasonal adjustment factors. There are no provisions in the model to compute stormwater quality from land use characteristics such as population density, type of urban development, street sweeping practices, etc.

The dry-weather flow and quality are combined with the stormwater flow and quality for each catchment and routed from each inlet point through the sewers. This includes the combining of wastewaters and pollutants at sewer junctions. If a regulator is encountered during the routing, the flow is diverted according to the hydraulic performance of the regulator.

The flow routing is accomplished by the kinematic wave formulation of the equations of motion. The equations consist of the nonsteady equation of continuity, Manning's equation to define the energy gradient, and relationships based on the geometry of circular pipes.

The kinematic wave equations are solved in Lagrangian coordinates (characteristic solution) in order to reduce the need for knowing initial conditions everywhere in the modeled system and to facilitate the optimization under transient conditions. This formulation does not consider downstream hydraulic control, backwater effects and flow reversal, and approximates surcharging and pressure flow independently for each pipe.

Convective transport of pollutants is computed using the kinematic wave celerity instead of the flow velocity to facilitate the consideration of both wastewater flows and quality in the real-time control optimization. The magnitude of the resulting error in the pollutant travel times has not been evaluated. The model does not consider hydrodynamic dispersion, scour and deposition, decay, reactions and interactions between the pollutants.

At each regulator the quantity of the flow diverted into the system and that portion overflowed is computed using the appropriate weir and orifice discharge equations for each type of structure. At present, the model simulates four types of fixed (noncontrollable) regulators: perpendicular weir overflow with orifice dry-weather outlet (weir may have other angles to flow direction); sidespill weir with orifice dry-weather outlet;

leaping weir overflow; and orifice overflow (no weir). The regulator equations consider only upstream flow conditions and neglect the effect of downstream conditions on the regulator discharge.

In addition, the control optimization determines the required flow diversion at any type of controllable regulator, such as inflatable rubber dams for the storm overflow outlets in combination with hydraulically operated gates for diversion of dry-weather flows. The present version of the model does not, however, compute the weir height or gate setting which would produce the required flow diversion.

At regulators and treatment facilities, the cumulative wastewater volume and pollutant mass of the effluent is computed. If the overflow is diverted to a storage facility, the cumulative values represent the time history of cumulative inflow to the storage facility. Diversions to each storage facility are determined by the control optimization which evaluates the optimum utilization of all storage facilities for the duration of the storm runoff. It is assumed that the stored wastewaters are returned to the system during dry-weather flow periods. This is not computed, however, in the present version of the model.

Both overflow and in-line treatment are handled by the model. The main treatment plant is considered as in-line treatment. The rate of pollutant removal is determined from tables which relate flow rate and concentrations with treatment efficiency for each type of treatment and pollutant. These tables are provided as input data. The model does not include mathematical formulations of treatment processes which relate physical characteristics of the facility with treatment efficiency.

The optimization consists of a two-phase scheme combining design optimization with optimum control of the sewer system. For simulation purposes alone the optimization is not activated. For control purposes alone only the control optimization is used. For design and planning purposes, the design optimization is run in conjunction with the control optimization to incorporate cost savings resulting from the optimum system operation into the total design.

For real-time operation of a system, the control optimization computes at regulators the required diversion of flows and pollutant masses to receiving waters, interceptors, storage facilities, and treatment plants. Dynamic programming is used to allocate available sewer, storage facility, and treatment plant capacities so that the total mass of pollutants entering the receiving waters through storm runoff is minimized. In

addition, constraints can be imposed at each regulator on either the maximum pollutant mass discharge rate or concentration of the wastewaters overflowed.

For planning and design studies the model can simulate the performance of a sewer system, permitting consideration of potential alternatives for system improvements. This may include new sewer lines, regulators, and storage and treatment facilities. The design optimization determines the most economical combination of new facilities to perform a specified task: reducing waste discharges to receiving waters below a specified level or minimizing waste discharges for the amount of investment available. It provides required sizes and costs of sewer pipes, regulators, treatment plants, and storage facilities. A modified gradient technique is employed for the design optimization. This technique considers both the dynamic response of the planned system as determined by the hydraulic and water quality computations and the optimum operation of the planned system as determined by the control optimization.

Computer Program

The program is available to any user. Its documentation, however, is only in the draft stage. The program is written in Fortran IV. The Battelle version consists of two main segments: the hydraulic and water quality computations are programmed for a DEC PDP-9 computer and the optimization for a Univac 1108 computer. The total program is operated in batch mode using an interface between the two computers. Programming modifications in the interface between the two segments of the Battelle versions are required before the model can be implemented on different computers. An IBM computer version of the complete program is being used by the Control Systems Group of the Department of Public Utilities of the City of Cleveland (Pew et al., 1972). IBM versions are also being used by Watermation, Inc., of Saint Paul, Minnesota.

Most program output is on a hydrostatic printer and includes tables and plots of rainfall intensities for each catchment, junction, regulator, storage and treatment facility; water level, discharge, velocity, wave celerity and Froude number for each pipe reach; cumulative storage volume for each storage facility; and size and cost of sewers, storage and treatment facilities for design optimization. In addition, schematic plots of the sewerage system showing the sewer layout and locations of inlets, regulators, junctions, storage and treatment facilities can be obtained. Options are available to suppress any or all printed and plotted output and to choose only cathode ray tube displays of selected tables and plots.

Evaluation

At present, this is the only model which includes mathematical optimization techniques for both real-time control applications and for least-cost design purposes. It is the only model which considers the costs of sewers, storage and treatment facilities in the design optimization. It is also the only model which considers wastewater quality objectives in the real-time and design optimization.

Most limitations resulting from various approximations and simplifications in the wastewater flow and quality formulations are described above. The principal limitations which may restrict the model's general applicability are the neglect of downstream hydraulic controls in the flow routing and the neglect of inline storage facilities and storage in the sewers. Consideration of these aspects would require major model revisions. Other modifications, such as the addition of formulations for new types of regulators, other pipe cross sections, pumping stations, and inverted syphons, can be accomplished by the addition of appropriate subroutines.

The model can be used only for the simulation of individual runoff events since it does not include provisions for either catchment moisture or water quality accounting between rainstorms. The model has been tested on very limited urban hydrologic data and its accuracy has not been sufficiently established. The Watermation version of the model is being used for engineering assessments and design purposes.

BRITISH ROAD RESEARCH LABORATORY MODEL

Summary

The British Road Research Laboratory Model simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a converging branch sewer and open channel network (Watkins, 1962; Terstriep and Stall, 1969). The model computes surface runoff only from impervious areas which are directly connected to the storm drainage system (flow does not pass pervious areas). The model is limited to the simulation of single runoff events. Water quality, real-time control and design features are not included.

The Illinois State Water Survey Urban Drainage Area Simulator is an extension of this model which simulates also the runoff from pervious areas and includes an option to either size retention basins or circular sewers (Terstriep and Stall, 1974). Model documentation was received too late for a detailed review.

The British Road Research Laboratory Model includes the following features:

1. dry-weather flow of several catchments;
2. several rainfall records
3. storm runoff from impervious areas directly connected to the storm drainage system for several catchments;
4. flow routing in gutters;
5. routing of combined wastewater flow in a converging branch network; and
6. circular and rectangular closed conduits and trapezoidal open channels.

The model does not include the following features:

1. dry-weather flow from land use characteristics;
2. evapotranspiration;
3. snow accumulation and melt;
4. catchment moisture balance between storms;
5. flow routing in loops and diverging branches;
6. downstream flow control, backwater, surcharging, pressure flow, and flow reversal;
7. flow control and diversion structures;
8. storage facilities;
9. water quality;
10. real-time control;
11. design; and
12. costs.

Methods

Dry-weather or base flow is provided as input data in the form of a linear function of time. This is probably satisfactory

for runoff periods of a few hours. The model does not include provisions to compute dry-weather flow from land use.

Storm runoff is computed from the weighted average rainfall of several raingages. The rainfall has to be provided in the form of cumulative values at constant time intervals. Losses are subtracted from the rainfall according to a function of time which is defined as input data. Since the model does not consider runoff from pervious areas, which requires the consideration of infiltration, the subtraction of losses from rainfall without considering the overland flow depth is probably adequate.

The rainfall excess is routed to sewer inlets using average travel times computed for each hydrograph from Manning's equation. This appears to be an adequate approximation for storms of low intensity. For storms of high intensity, however, the effect of different travel times for different runoff rates is expected to introduce significant errors into the runoff computation.

The combined flow is routed from each inlet through the sewers with a storage routing technique which uses average travel times computed for each inlet hydrograph from Manning's equation. The error introduced by this approximation becomes larger as the flow variations and range of flows increase. The model can be used for open channels since it includes routing formulations for trapezoidal open channels in addition to circular and rectangular closed conduits.

The formulation does not consider downstream flow control, backwater effects, surcharging, pressure flow, and flow reversal. The model does not simulate flow control and diversion structures and storage facilities.

The model can be used only for the simulation of individual runoff events since it does not include provisions for catchment moisture accounting between rainstorms. The model does not include design features.

Computer Program

The program is available from the Storm and Combined Sewer Section, Advanced Waste Treatment Research Laboratory of the U.S. Environmental Protection Agency. A user's manual is not available; the user has to rely on limited information documented in the references. The program was written in Fortran IV for the IBM 360/75 computer and Calcomp plotter of the University of Illinois.

Program output includes printed and plotted rainfall intensities for each raingage and discharges at sewer junctions and the

downstream sewer outlet. Water levels and velocities are not computed.

Evaluation

Model applicability is severely limited since it neglects the runoff from pervious catchment areas such as gardens, lawns, and natural areas. This restricts the model's application to catchments with large portions of paved or impervious areas. In addition, the approximations in the overland and sewer flow routing schemes introduce unacceptable errors for flow resulting from high intensity storms.

The model was tested by Stall and Terstriep (1972) using data from ten urban catchments ranging in size from 6 ha (14 acres) to 21 km^2 (8 sq mi). The results indicate that model accuracy appears to be satisfactory for catchments of less than 13 km^2 (5 sq mi) if the impervious areas directly connected to the storm drainage system comprise more than 15 percent of the total drainage area and if the frequency of the storm is less than 20 years.

CHICAGO FLOW SIMULATION PROGRAM

Summary

The Chicago Flow Simulation Program is intended primarily for the simulation of large catchments consisting of both sewered and nonsewered areas (Lanyon and Jackson, 1972 and 1974). It simulates the time-varying runoff in combined sewerage systems and nonurban drainage basins consisting of several catchments and a converging brach sewer and open channel network. The flow routing formulation for natural channels includes provisions for flow and storage in flood plains. The model can be used for continuous simulation using hourly or shorter time steps. Water quality, real-time control and design features are not included.

The Chicago Flow Simulation Program includes the following features:

1. dry-weather flow of several catchments;

2. rainfall, snowfall, air temperature, and wind velocity at several weather stations;

3. evapotranspiration;

4. snow accumulation and melt;

5. stormwater runoff from pervious and impervious areas of several catchments;

6. catchment moisture accounting during periods of no precipitation;

7. routing of combined wastewater flow in a converging branch sewer and natural channel network;

8. circular closed conduits and trapezoidal open channels and flood plains; and

9. in-line storage reservoirs with preset operating rule.

The model does not include the following features:

1. dry-weather flow from land use characteristics;

2. flow routing in gutters;

3. flow routing in loops and diverging branches;

4. noncircular closed conduits;

5. downstream flow control, backwater, surcharging, pressure flow, and flow reversal;

6. flow control and diversion structures;

7. water quality;

8. real-time control;

9. design; and

10. costs.

Methods

Dry-weather flow is computed for each catchment from population and dry-weather flow per capita for each hour provided as input data. The model does not include provisions to compute dry-weather flow from land use.

Storm runoff is computed from rainfall or snowmelt. One weather station can be specified for each subcatchment. The snowmelt computations are based on a method of the U.S. Bureau of Reclamation (1966) which requires as input data precipitation at constant time steps, daily average air temperature and daily

average wind velocity. The precipitation values have to be provided as the amount of precipitation which fell in each time interval.

Originally, the program was formulated for 1-hour time steps and required at least 1 day of hourly precipitation data as input. The version furnished to Battelle was modified to accept shorter time increments, but still requires a minimum of 1 day's data. This is a considerable inconvenience for small time steps, especially since a separate data card is used for each time step.

Different formulations are used for the computation of runoff from rainfall for sewered and nonsewered areas. If the area is sewered, it is assumed that the rainfall-runoff relationship is linear and that the water stored on the catchment during a time step flows into the sewer during the same time step. It is assumed that all rain falling on impervious areas becomes runoff and a constant fraction of the rain falling on pervious areas becomes runoff. This is a serious restriction since it neglects variations in catchment characteristics such as shape, slope, and surface roughness and assumes a constant rate of rainfall loss. Considerable errors are therefore introduced if much of the catchment is sewered.

For nonsewered areas, it is assumed that all rain falling on impervious areas becomes runoff. A time delay is introduced, however, by specifying that a constant fraction of the water stored on the impervious areas becomes runoff during each time step. This again neglects the effect of variations in catchment shape, slope and surface roughness on overland flow. For pervious areas, rainfall losses are computed using an empirical function relating the losses to rainfall, catchment shape, and soil moisture. Continuous accounting of soil moisture is accomplished with an empirical equation which considers deep percolation and evaporation. Evaporation is defined as a function of soil moisture at each time step and average daily air temperature. Linear storage routing is used to compute overland flow from the pervious areas, considering overland flow storage, catchment shape and slope but assuming a constant surface roughness.

Flow routing in both circular sewers and trapezoidal open channels is accomplished by a storage routing technique using Manning's equation. The equations for the circular pipes are simplified by assuming a linear relationship between depth, cross-sectional flow area, and discharge. The effects of this approximation on flow routing accuracy have not been determined. A special provision is incorporated into the model for the consideration of water storage and flow routing in flood plains if the flow exceeds open channel capacity. The flow routing procedure does not consider downstream flow control, backwater

effects, surcharging, pressure flow, and flow reversal.

Reservoirs can be simulated with a fixed type of operation which assumes that all inflow exceeding a specified maximum outflow is stored. The formulation does not require data on reservoir and outlet geometry. The model does not simulate other flow control facilities and diversion structures.

Computer Program

An IBM version of the program is available from the developers at the Metropolitan Sanitary District of Greater Chicago and a CDC 6400 version is available from Battelle-Northwest. Both versions are written in Fortran IV. A complete user's manual is not available but sufficient information on model formulation and input data requirements is given by Lanyon and Jackson (1974) and there is a supplementary manuscript describing the input data formats.

Program output includes input rainfall data, daily rainfall totals, daily drainage from snowmelt, and end of day snow depths at each raingage; daily average discharge at selected points; maximum stage and discharge for each reach; and stage, discharge and storage for each time step at selected reaches. Velocities are not computed. Line printer plots can also be obtained for stage, discharge and storage as functions of time for selected reaches if the computational time step is one hour.

Evaluation

Model applicability appears limited to large catchments (larger than 5 km^2 = 2 mi^2) as a result of the simplifying assumptions for runoff from pervious and impervious areas of both sewered and nonsewered areas. The assumptions appear to be adequate for local conditions of major Chicago drainage basins using continuous simulations with 1-hr time steps. The model could be made more generally applicable by defining the physical meaning of the runoff coefficients and defining them as input data rather than as internal values in the program.

Testing reported by Lanyon and Jackson (1974) for ten monthly rural catchments with drainage areas ranging from 12 to 264 km^2 (4.7 to 102 sq mi) and 3 to 45 percent imperviousness indicated satisfactory comparisons with measured streamflows for most tested storms.

CHICAGO HYDROGRAPH METHOD

Summary

The Chicago Hydrograph Method simulates the time-varying runoff in combined sewerage systems consisting of several catchments

and a converging branch sewer or open channel network (Tholin and Keifer, 1959; Keifer et al., 1970). The model computes diameters of circular sewers for peak flows. Costs are not considered. The model is limited to the simulation of single runoff events. Water quality and real-time control are not included.

The Chicago Runoff and Pollution Model is an extension of the model for the continuous simulation of both wastewater flow and quality (City of Chicago, 1972). Model documentation was received too late, however, for a detailed review.

The Chicago Hydrograph Method includes the following features:

1. dry-weather flow of several catchments;
2. several rainfall records;
3. evapotranspiration;
4. storm runoff from pervious and impervious areas of several catchments;
5. catchment moisture accounting during periods of no rainfall;
6. flow routing in gutters;
7. routing of combined wastewater flow in a converging branch network;
8. circular closed conduits and trapezoidal open channels; and
9. sizing of circular pipes.

The model does not include the following features:

1. dry-weather flow from land use characteristics;
2. snow accumulation and melt;
3. flow routing in loops and diverging branches;
4. noncircular closed conduits;
5. downstream flow control, backwater, surcharging, pressure flow, and flow reversal;
6. flow control and diversion structures;

7. storage facilities;

8. water quality;

9. real-time control; and

10. costs.

Methods

Dry-weather flow is provided as input data in the form of hydrographs at constant time intervals for each catchment. The model does not include provisions for computing dry-weather flow from land use.

Storm runoff is computed from the weighted average rainfall of several raingages. Rainfall intensities have to be provided at 1-minute time intervals, a serious limitation. Longer time intervals can be specified, however, for the runoff and routing calculations. The program includes an option to compute design storms of 1 to 100-year frequencies by providing the coefficients for the design storm equations as input data.

Infiltration losses on pervious areas are computed from Horton's equation which has been modified to account for actual infiltration. This would appear to produce a more accurate loss computation than does the more common use of Horton's equation in other models which neglect the effect of actual infiltration or soil moisture on the potential infiltration rate. The Chicago Hydrograph Method, however, computes actual infiltration from rainfall rather than overland storage depth, thus introducing a different kind of approximation. After subtracting infiltration from rainfall, the remaining moisture is reduced by an exponential function of available depression storage and cumulative rainfall excess to account for losses due to depression storage. This formulation is based on the assumption that smaller depressions fill sooner than larger depressions. The remaining moisture then becomes the overland flow contribution from the pervious area.

It is assumed that runoff occurs from impervious areas until all available depression storage is filled.

During dry periods, the potential infiltration is assumed to recover by a constant rate of 76 mm/hr (3 in./hr). Depression storage on pervious areas is assumed to recover as a function of potential infiltration rates. Depression storage on imperviou areas is assumed to recover fully after 6 hours of dry weather. The recovery of catchment moisture conditions implicitly considers evaporation but can be considered only a rough approxima-

tion since it neglects the influence of meteorological variables other than rainfall.

These formulations appear to give the method continuous simulation capability. Nevertheless it can be used only for the simulation of single runoff events since the program does not provide for the input of initial catchment moisture and channel flow conditions, does not print out final catchment moisture conditions, and is restricted to simulation periods of less than 500 minutes.

Overland flows from pervious and impervious areas are routed separately by means of a storage routing technique which uses empirical functions and is based on a method by Izzard. The routed overland flows from both pervious and impervious areas are then combined and routed through the gutters using a storage routing technique. Routing coefficients relating storage with discharge must be provided as input data for the overland flow routing for both pervious and impervious areas and for the gutter routing. Equations are provided in the user's manual for estimating the coefficients from catchment and gutter physical characteristics. The method seems unnecessarily complicated since more exact and simpler formulations based on the kinematic wave equations are available.

Flow routing is formulated for circular pipes and trapezoidal open channels again using a storage routing technique. Manning's equation is solved for the peak flow of each hydrograph to define a linear relationship between storage and discharge. The flow routing procedure does not consider downstream flow control, backwater effects, surcharging, pressure flow, and flow reversal.

If the diameter is not given for a circular pipe, then the program computes the diameter as a function of the peak flow at the downstream end of the pipe. The program does not compute the sizes of open channels.

Computer Program

An IBM 1130 version of the computer program written in Fortran IV and a user's manual (in draft form) is available from the Department of Public Works, Bureau of Engineering of the City of Chicago.

Computer output includes rainfall intensities for each raingage at 1-minute time intervals and the time intervals selected for the routing; discharge at ten selected times or all time intervals for each subcatchment and reach, the combined flow at each junction, and the downstream outlet; peak discharge and time of peak discharge for each subcatchment; peak discharge, peak

velocity and time of peak discharge for each reach; and the diameter of circular pipes if the diameter was entered as zero. In addition, the discharge hydrographs can be plotted on an IBM 1627/II plotter. Water levels and velocities other than peak velocities are not computed.

Evaluation

The model is suitable for the simulation of nonsteady runoff from urban and nonurban catchments. It includes considerable simplifications in the overland, gutter and flow routing which would appear to limit the model's accuracy and which are not necessary considering the present state-of-the-art of flow routing techniques. Routing constants have to be provided as input data for both pervious and impervious overland flow routing and the gutter routing. Equations are provided in the manual for external estimation of the constants, which is an inconvenience. The routing coefficients for the sewer and open channel routing are computed by the program.

Although the model includes provisions for catchment moisture accounting during periods of no rainfall, it cannot be used for continuous simulations due to the computer program limitations described above. A special feature of the model is its design capability for the sizing of circular sewer pipes, which could make it attractive in spite of the other limitations.

The model developers do not report any testing of the method with measured urban runoff data which would allow judgment on model accuracy.

COLORADO STATE UNIVERSITY URBAN RUNOFF MODELING

Summary

The Colorado State University's urban runoff modeling efforts are included in this review because these special research activities are expected to provide new and improved techniques for the control of overflows and the simulation of nonsteady flow in pipes.

Two parallel research efforts are being conducted. One is directed toward the development of automatic control of sewer systems. The second study is concerned with the physical modeling of runoff from rainfall and flow routing in pipes and the verification of mathematical models using the data derived from the physical models. Both studies are concerned only with the estimation of urban runoff and the design of storm sewers and control based on wastewater flow alone. Wastewater quality and treatment are not considered.

Methods

Considerable research is being conducted into the feasibility and potential techniques for automated real-time control of overflows in storm and combined sewerage systems. A series of reports has been published covering various aspects of these studies, ranging from conceptual development of mathematical control schemes to the consideration of social and political aspects of urban wastewater management (Smith et al., 1972; Grigg et al., 1973 and nine other supporting technical reports). Although an operational mathematical model for real-time control has not been developed to date, these efforts are significant in developing system concepts and the future direction of urban wastewater management.

Separate research is being conducted at Colorado State University to investigate various flow routing techniques in circular pipes (Yevjevich and Barnes, 1970). The investigations have covered both experimental and numerical studies. A circular conduit 0.91 m (3 ft) in diameter and 250 m (822 ft) long was used to measure the propagation of flood waves of various configurations. The experiments provided data for the numerical testing of various finite difference solutions of the dynamic wave equations for gradually varied free-surface unsteady flow (the Saint-Venant equations). Various explicit finite-difference solutions of the partial differential equations were investigated, including the unstable, diffusing, upstream-differencing, leap-frog, and Lax-Wendroff schemes. In addition, both first-order and second-order interpolation schemes of the characteristic differential equations were studied. It was concluded that the specified interval scheme of the characteristic equations using second-order interpolation was the most accurate of all the methods studied and that the second-order interpolation Lax-Wendorff scheme was the most accurate of all the explicit methods studied in solving the partial differential equations.

The diffusing scheme, Lax-Wendroff scheme, and specified interval scheme of the method of characteristics were computer programmed for a single string of pipes. The schemes consider constant flow initial conditions and permit input hydrographs at various points along the pipe string. The formulations consider both upstream and downstream hydraulic control, backwater, and flow reversal. Surcharging and pressure flow are not considered. The formulations do not provide for junctions and diversions. Nevertheless, the programs can be viewed as significant steps toward the formulation of more general flow routing schemes in circular pipes than the more common and restrictive storage routing and kinematic wave routing schemes.

The investigations did not consider implicit finite difference schemes of either the partial differential equations or the characteristic differential equations. Implicit methods are becoming more popular as computer capabilities increase since they may require less computer time. Explicit methods become numerically unstable if the selected time step exceeds a maximum value which depends on the selected finite difference scheme and the node spacing. Implicit methods allow much longer time steps with generally very little sacrifice in accuracy. Implicit methods are more difficult to program, however, than explicit methods, particularly for large networks with complex boundary conditions including junctions, diversion structures, pumping plants, and storage facilities.

Computer Programs

No computer programs have been developed under the real-time control research effort which are sufficiently operational to form either a real-time control scheme or significant parts of such a scheme.

However, computer programs are available for three nonsteady routing schemes from the Office of Research, Federal Highway Administration, U.S. Department of Transportation. The programs are written in Fortran IV for a CDC 6400 and 6600. The programs compute and print water levels, velocities, and discharges for any desired point along a pipe string.

Evaluation

Research at Colorado State University related to real-time control of storm and combined sewerage systems has been limited to the development of concepts. No operational simulation programs have been developed, but the concepts will be useful in evaluating the future direction of urban wastewater management and the development of a real-time wastewater control systems.

The explicit solutions and computer programs of the Saint-Venant equations for gradually varied unsteady flow in circular pipes constitute a valuable contribution to the simulation of general flow conditions in sewers. The methods merit consideration for incorporation into comprehensive sewerage system simulation models since they consider upstream and downstream hydraulic control, backwater, and flow reversal which are not modeled by the more common formulations based on storage routing and the kinematic wave equations.

CORPS OF ENGINEERS STORM MODEL

Summary

The Storage, Treatment and Overflow Model (STORM) of the Corps of Engineers Hydrologic Engineering Center is intended primarily for evaluating of the stormwater storage and treatment capacity required to reduce untreated overflows below specified values (Corps of Engineers, 1973). The model can simulate hourly stormwater runoff and quality for a single catchment for several years. Five water quality constituents are computed for different land uses: suspended and settleable solids, biochemical oxygen demand, nitrogen, and phosphorus.

The model does not route flows and quality through a sewer network and does not have real-time and design capability. Its main purpose is the assessment of future needs for treatment and storage of stormwater under varying land use conditions. Dry-weather flows and quality are not simulated; consequently the model is not applicable to combined sewerage systems.

The STORM model includes the following features:

1. rainfall, snowfall, air temperature, and evaporation at one weather station;
2. evapotranspiration;
3. snow accumulation and melt;
4. stormwater quality for different land uses;
5. suspended and settleable solids, biochemical oxygen demand, nitrogen, and phosphorus;
6. stormwater runoff and quality from pervious and impervious areas of a single catchment;
7. catchment moisture accounting during periods of no precipitation;
8. capacity of storage facilities;
9. hydraulic capacity of treatment facilities; and
10. volume and quality of overflows.

The model does not include the following features:

1. dry-weather flow and quality;

2. consideration of nonuniform catchment precipitation distribution;

3. stormwater runoff and quality from more than one catchment;

4. flow and quality routing in gutters, sewers and open channels;

5. wastewater quality decay, reactions and interactions;

6. wastewater quality improvement by treatment;

7. real-time control;

8. design; and

9. costs.

Methods

Hourly stormwater runoff is defined as the product of a runoff coefficient and hourly rainfall excess. Only one precipitation record can be used. The runoff coefficient is the weighted average of empirical runoff coefficients for the pervious and impervious areas and represents the fraction of rainfall excess lost to infiltration. The rainfall excess is defined as the difference between hourly rainfall and losses to depression storage. The depression storage at the beginning of a rainstorm is defined as the available depression storage at the end of the previous rainstorm plus a linear recovery to account for evaporation during the period of no precipitation. A different evaporation rate can be specified for each month of the year. Snowmelt is computed by the degree-day method which requires mean daily air temperature as input data.

The runoff coefficients, available depression storage, and depression storage recovery factor have to be derived by calibration with observed data. Default values are given in the program but no instructions are provided for estimating adequate values for ungaged catchments. Dry-weather flow is not simulated.

The computations for the stormwater runoff quality are based on formulations first used in the EPA Stormwater Management Model, which simulated only suspended solids, biochemical oxygen demand, and coliform bacteria. The STORM model has been expanded to simulate suspended and settleable solids, biochemical oxygen demand, nitrogen, and phosphorus. Empirical equations

considering land use, street sweeping practices, and days between rainstorms define the amount of each pollutant on the ground at the beginning of a rainstorm. An exponential function of the runoff rate determines the rate of each pollutant being washed off the catchment during each hour. The rates of runoff of BOD, N, and PO_4 are assumed to be dependent on the rate of runoff of suspended and settleable solids.

The runoff quality formulations depend on a large number of empirical coefficients which have been derived from very limited urban stormwater runoff and quality data. The coefficients are internal to the program and do not account for variations in land use and catchment characteristics. Application of the model in different areas may therefore require programming changes to modify the coefficients. Input data which may be difficult to obtain include the daily rate of dust and dirt accumulation and the percent of each pollutant contained in the dust and dirt for different land uses. Default values are provided by the program. Dry-weather quality is not considered by the model.

Computations of treatment, storage and overflow proceed on an hourly basis by simple runoff volume and pollutant mass balance. If the hourly runoff exceeds the treatment capacity, the excess runoff is put into storage. If the storage capacity is also exceeded, the excess runoff becomes untreated overflow. If the runoff is less than the treatment capacity and water is in storage, then the excess treatment capacity is utilized to diminish the storage volume.

Plug flow is assumed for the routing of pollutants through storage. The water quality is not modified in storage. For treatment, only the hydraulic capacity of treatment facilities is considered. Stormwater quality improvement by treatment is not modeled.

Computer Program

The computer program is written in Fortran IV for a UNIVAC 1108 computer and is available from the Hydrologic Engineering Center of the Corps of Engineers. The simulations require 1-hour time steps and a minimum of 24 hours of simulation. Several years of data can be simulated in a single run; the maximum duration depends only on computer running costs and the objective of the analysis.

Program output includes hourly precipitation for a single raingage and various summaries of the stormwater runoff and quality analysis for selected storm events. These include the duration and amount of rainfall, the time and amount of

treatment, the amount of runoff, the utilization and age of storage, the amount of overflow to receiving waters, and averages for all selected events. Runoff amounts are defined in inches and pollutant quality in pounds. In addition, tables and line printer plots are available indicating the utilization of storage.

Evaluation

The model is suitable for the continuous simulation of stormwater runoff and quality from a single urban catchment. Addition of features for the simulation of dry-weather flow and quality would be desirable to extend model applicability to combined sewerage systems.

Limitations on accuracy of the runoff computations are imposed by the simplified rainfall-runoff formulation, particularly the assumptions of a constant infiltration loss rate during rainstorms, a constant evaporation rate between rainstorms, and immediate runoff of the hourly rainfall excess. The last approximation would reduce model accuracy as the catchment size increases and the time of concentration of the runoff becomes longer than one hour.

The stormwater quality relationships are based on empirical formulations which have been tested on very limited data and whose accuracy has not been sufficiently established. It is therefore not possible to estimate their accuracy for application to areas where no records of concurrent urban stormwater runoff and quality are available for calibration purposes.

The model appears to be useful, however, for general planning purposes to estimate the relative magnitudes of required storage and treatment capacities to reduce stormwater overflows below desired levels. The model computes only the quality of the overflows to the receiving waters. Addition of the computation of the quality of the treatment plant effluents would be desirable to obtain the total pollutant load on the receiving water.

Verification of the complete model is in progress. (Roesner, et.al., 1974).

DORSCH CONSULT HYDROGRAPH-VOLUME METHOD

Summary

The Dorsch Consult Hydrograph-Volume Method simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a closed conduit and open channel network including loops and converging and diverging branches

(Ritter and Warg, 1971; Koniger, 1972; Koniger and Klym, 1972; Koniger et al., 1972; Klym et al., 1972; Mevius, 1973; Dorsch Consult, 1973; and Klym, 1975). The flow routing is based on a simplified solution of the full dynamic wave equations and simulates backwater, surcharging, and pressure flow, but not flow reversal. The model simulates diversion structures and storage basins. The model is limited to the simulation of single runoff events. Water quality, real-time control and design features are not included.

A separate Quantity-Quality Simulation Program has been developed for the continuous simulation of wastewater flow and quality (Geiger, 1975). Model documentation was received too late for a detailed review. A brief description and hypothetical test results furnished by Dorsch Consult are included in Appendix E.

The Dorsch Consult Hydrograph-Volume Method includes the following features:

1. dry-weather flow of several catchments;
2. several rainfall records;
3. special statistical analyses of historical rainfall records to compute design hyetographs (separate program);
4. stormwater runoff from pervious and impervious areas of several catchments;
5. evapotranspiration;
6. catchment moisture accounting during periods of no precipitation;
7. routing of combined wastewater flow in a network of loops and converging and diverging branches;
8. various specified closed conduit and open channel cross sections and any arbitrary shape;
9. backwater, upstream and downstream flow control, flow reversal, surcharging and pressure flow;
10. flow control and diversion structures; and
11. retention storage basins.

The model does not include the following features:

1. dry-weather flow from land use characteristics;
2. snow accumulation and melt;
3. water quality;
4. real-time control;
5. design; and
6. costs.

Methods

Sanitary flow is computed from input data on the average per capita contribution and the number of residents. A constant groundwater infiltration rate to the sewers is also provided as input data. In addition, input data defining an average base flow, a ratio of the average to the peak hourly flow and a ratio of the average to the night flow are used to compute a diurnal pattern of dry-weather flows. At every manhole, a constant rate of industrial wastewater flow may be provided as input data. The model does not include provisions to compute dry-weather flow from land use.

A separate rainfall record can be provided for each subcatchment to compute surface runoff. A separate program performs special statistical analyses to compute design storms of specified frequencies from recorded rainfall events.

Runoff from impervious areas is computed separately from roofs and streets. No rainfall losses are assumed for impervious areas without depression storage. An initial loss has to be satisfied before runoff begins from impervious areas with depression storage and from pervious areas. In addition, infiltration losses are computed for pervious areas using Horton's equation. The statistical distribution of surface depressions is considered in the runoff and infiltration computations to account for earlier runoff from shallow depressions and for reduced infiltration when only a part of the area is covered with water and the rainfall intensity is less than the infiltration capacity.

Infiltration calculations continue during periods of no rainfall as long as water is on the pervious areas. Puddles on impervious areas are reduced by evaporation. Runoff from snowmelt is not computed.

Infiltration is computed from overland flow depth, which is more accurate than computing it directly from rainfall as some model formulations do. Horton's equation was modified to account for soil moisture changes during and between rainfall periods.

Overland flow is computed with the kinematic wave equation using Manning's equation. The overland flow length is assumed to be the average width of streets, roofs or lawns within a subcatchment (e.g., flow length on pervious areas = pervious area/sewer length). A data edit program provides these mean values. This eliminates the necessity of providing input data for each urban lot or block. Flow routing in gutters may be treated like that in sewers.

An implicit finite difference solution of the dynamic wave equations (the Saint-Venant equations) is formulated for the routing of nonsteady flows through sewers, open channels, and retention basins. The formulations consider drop structures and are coupled with equations computing overflow at diversion structures. The Manning or Prandtl-Colebrook equation is used to define the energy losses for the flow routing, and the Poleni equation computes the diverted flow and overflow at diversion structures.

An iterative procedure is used to compute depths and discharges at each time step, starting with estimated values in the first iteration. First, the new discharges are computed in the downstream direction and then the new depths are computed in the upstream direction. Normally 30 iterations are required for each time step to obtain satisfactory convergence. This type of numerical solution prevents the computation of flow reversal although the basic equations include this phenomenon. Flow reversal can be simulated by inserting hypothetical sewer segments where flow reversal is expected, and specifying flow in the direction opposite that in the parallel original segment. The solution technique, however, considers upstream and downstream flow control and backwater. The dynamic wave equation is solved in such a way that it also simulates surcharging and pressure flow. The program allows input of both sewer invert, basement and ground surface elevations and computes flooding resulting from backwater or surcharging conditions.

The model does not include water quality, real-time control, design and cost computations.

Computer Program

This is a proprietary model developed by the engineering consulting firm Dorsch Consult Ingenieurgesellschaft mbH of

Munich, Germany. A North American office is maintained in Toronto, Canada. The firm indicated that the model may be released under certain use and distribution restrictions. The program is written in Fortran IV and can be run on a Univac 1108 and CDC 6600.

The program can be run in either British or metric units. Program output includes water surface elevations and discharges for every desired point in the system. Cumulative inflow and outflow are printed for diversion structures and retention basins. The stored volume is also printed for retention basins.

Evaluation

This is one of the most complete and possibly accurate models for the computation of runoff from urban catchments and the routing of flows in sewer networks. Extreme detail with respect to subcatchment discretization is required, however, to calculate accurate overland flow from rainfall. Simplifications are probably possible to allow larger subcatchment areas without significant loss in accuracy.

The implicit solution of the Saint-Venant equations provides an accurate means of computing the flow routing in the sewers coupled with routing through diversion structures and retention basins. The consideration of both upstream and downstream boundary conditions and the computation of backwater is part of the basic equations and does not have to rely on approximate methods such as those included in some kinematic wave or storage routing techniques. The implicit solution of the Saint-Venant equations coupled with diversion equations is complicated however, and time consuming on the computer as compared to more approximate methods. To simplify the solution somewhat an iterative scheme is used which cannot consider flow reversal (although it is contained in the basic equations).

Although the model formulations are considerably more precise than most tested models, the increased accuracy demands a sacrifice in computer time. The model would be needed primarily where backwater, downstream flow control, diversion structures, retention basins, and surcharging are important features of the sewerage system assessment. If these features are not present or are considered insignificant, simpler models requiring less computer time can be used.

The model has been applied extensively in Germany and Switzerland, and moderately in Canada, the U.S.A. and other countries. Model comparisons with measured runoff data are limited. Simulations of several storms on the Oakdale Avenue catchment in Chicago, Illinois, with a drainage area of 19 ha (47 acres)

resulted in good comparisons between measured and computed runoff.

ENVIRONMENTAL PROTECTION AGENCY STORMWATER MANAGEMENT MODEL

Summary

The U.S. Environmental Protection Agency's Stormwater Management Model is one of the most comprehensive mathematical models available for the simulation of storm and combined sewerage systems. It computes the combined storm and sanitary runoff from several catchments and routes the flows through a converging branch sewer network (Metcalf & Eddy et al., 1971). Flow diversion structures can be modeled and storage can be simulated for both inline and overflow retention basins. An additional feature is a receiving water model which includes nonsteady formulations of hydrodynamics and mass transport for two-dimensional (vertically mixed) water bodies receiving sewerage system effluents.

Both dry-weather and stormwater quality for suspended and settleable solids, biochemical and chemical oxygen demand, coliform bacteria, phosphorus, nitrogen, and oil and grease are computed for each modeled catchment and routed through the sewerage system. Mathematical formulations which simulate various combinations of overflow treatment processes for one treatment facility are included to evaluate the effectiveness of overflow treatment. The model does not include real-time control features. The model is limited to the simulation of single runoff events and inline treatment cannot be simulated. Cost functions are built into the program to compute the cost of overflow storage and treatment.

The Stormwater Management Model includes the following features:

1. dry-weather flow and quality of several catchments from land use;

2. several rainfall records;

3. stormwater runoff and quality for pervious and impervious areas of several catchments from land use;

4. light water quality constituents: suspended and settleable solids, biochemical and chemical oxygen demand, coliform bacteria, phosphorus, nitrogen, oil and grease;

5. flow routing in gutters;

6. routing of combined wastewater flow and quality in a converging branch network;

7. twelve specified closed conduit cross sections, a trapezoidal section, and two arbitrary shapes;

8. backwater, surcharging and pressure flow;

9. two types of diversion structures;

10. pumping stations;

11. one overflow and two inline storage facilities with four types of outlet facilities;

12. sedimentation and scour of suspended solids;

13. first-order water quality decay of BOD;

14. treatment of coliform dependent on suspended solids;

15. one overflow treatment plant with arbitrary combinations of nine unit treatment processes;

16. costs of storage and treatment;

17. receiving water flow and quality, and

18. sizing of circular pipes.

The model does not include the following features:

1. evapotranspiration;

2. snow accumulation and melt;

3. catchment moisture accounting during periods of no precipitation;

4. sewer flow and quality routing in loops and diverging branches;

5. downstream flow control and flow reversal;

6. water quality reactions and interactions in the sewers and in storage; and

7. real-time control.

Methods

Dry-weather flow and quality of eight constituents can either be provided as average values or computed from land use characteristics such as total population, population density, land use, residential income and home valuation of each subcatchment. Adjustment factors can be read in for diurnal (hourly) variations in flow and quality. Weekday adjustment factors can be read in for all but coliform bacteria. Industrial or commercial process flows and quality can be input separately.

Rainfall intensities of several raingages can be provided as input data to compute storm runoff from several catchments. Only one raingage record can be assigned to a particular subcatchment. A more accurate method of representing spatially nonuniform rainfall would be to compute a weighted average of the records of all raingages surrounding a subcatchment.

Losses are subtracted separately from the rainfall falling on pervious and impervious areas. Runoff occurs only when all depression storage is filled. No other losses are computed for impervious areas. Evaporative losses are not computed.

For pervious areas, the potential infiltration is computed with Horton's equation and the actual infiltration depends on the available overland flow depth. This is a more accurate computation than basing the infiltration losses on the amount of rainfall. Horton's equation, on the other hand, computes the potential infiltration as a function of time only and does not account for the change in potential infiltration with changes in soil moisture. The equation is satisfactory as long as the available moisture is greater than the potential infiltration, but errors are introduced during low intensity and intermittent storms when the available moisture is less than the computed potential infiltration. As a consequence, the infiltration coefficients of the equation have to be adjusted for different storms although theoretically they should be based on catchment soil characteristics only, independent of the storm patterns. This makes it difficult to use one set of coefficients with confidence for prediction purposes.

Overland flow is computed separately for pervious and impervious areas using a kinematic wave formulation with Manning's equation. A similar formulation is used for the gutter flow routing. The model formulation assumes that overland flow length and catchment length are equal. This is satisfactory for small catchments, such as individual lots and possibly city blocks, but errors are introduced if the difference between overland flow and catchment length increases (as occurs when several city blocks are lumped into a single

subcatchment). The model assumes that infiltration occurs throughout the time required for the overland flow to run the entire length of the catchment. Actual infiltration, however, may occur only during the time it takes the overland flow to run the length of a single lot. As a consequence, the computed infiltration would be greater than the actual infiltration and the computed runoff hydrograph would be underestimated. Users of the model apparently compensate for this error by reducing the infiltration coefficients. A better solution would be to reformulate the overland flow computation to account for the fact that infiltration may occur over a shorter distance than overland flow.

Infiltration from the ground into the sewers accounts for dry-weather infiltration, wet-weather infiltration, melting residual ice and snow infiltration, and groundwater infiltration. Average infiltration values for the entire modeled drainage basin have to be provided as input data. The entire catchment infiltration is then apportioned to individual sewers on the basis of the conduit perimeter and number of joints in each conduit. A degree-day method is used to compute the infiltrations from snowmelt, but the accumulation of snow and surface runoff from snowmelt is not modeled. The effect of the infiltrating water on the quality of the sewage is considered negligible and not modeled.

Flow routing in the sewers is accomplished with the kinematic wave equation using Manning's equation. The basic formulation neglects downstream flow control, backwater, surcharging, pressure flow and flow reversal. Special formulations are incorporated in the model, however, to approximate backwater, surcharging and pressure flow. General downstream control and flow reversal are not modeled.

Backwater is modeled by formulating a special backwater element with an inline storage element of fixed length and simulating it with a storage routing technique. This requires input data on flow depth versus storage volume which is difficult to compute for special sewer shapes (such as horseshoe shapes, etc.) with a nonzero invert slope. Also, the assumption of a constant backwater length appears to make the formulation highly approximate. Since the program permits only two inline storage elements, backwater can be considered at only two locations in the entire sewerage system. Also definition of a backwater element reduces the number of actual storage facilities that can be modeled.

If surcharging occurs, the model assumes that all water in excess of full pipe flow capacity is stored in the next upstream manhole. The formulation neglects the storage volume

of the manhole and the actual propagation of the surcharging farther upstream. The formulation consequently serves only to conserve volume and to warn the user that surcharging conditions exist, without adequately computing the phenomenon. Conduits can be sized by the program to avoid surcharging.

The flow routing scheme is formulated for 12 specified closed conduit cross-sections and a trapezoidal section. In addition, two arbitrary conduit shapes can be specified by providing input data relating the dimensions of each shape to flow area, hydraulic radius, and discharge.

Two types of flow diversion structures can be modeled. The first type assumes that all inflow exceeding a maximum value is overflowed. No data are needed to describe the geometry of this type. The second type assumes that no overflow occurs until the inflow exceeds a specified value. In this case, the depth of flow is found from a linear relationship between flow and depth and the overflow computed from this depth with a weir discharge equation. A more accurate formulation would be possible based on the weir and orifice equations.

Pumping stations can also be modeled. The model assumes that the pumps begin to operate at a constant pumping rate when the volume in the wet well reaches a maximum value and continue to pump until the wet well is pumped dry.

The model can simulate the performance of one overflow and up to two inline storage facilities. The model computes depth and volume of storage as a function of the inflow and outflow. The outflow is computed based on the hydraulic performance of the following four options of outflow conditions: gravity with orifice centerline at zero storage tank depth; gravity with fixed weir; gravity with both weir and orifice; and fixed rate pumps. Either regularly or irregularly shaped reservoirs can be specified by different input data. If a reservoir overflows, program execution terminates.

Stormwater quality is computed as a nonlinear function of stormwater runoff rate (which includes an exponential decay with time to account for the higher rates of pollutants being washed off during the beginning of a storm). Coefficients in equations for each of the three modeled pollutants account for different land uses. The equations consider street sweeping practices and days between rainstorms in defining the amount of each pollutant on the ground at the beginning of a rainstorm. The formulations account also for the contribution of accumulated BOD in catchbasins. Many of the coefficients in the formulations are internal to the program and have been derived from very limited data of measured stormwater runoff and

quality. Programming changes may therefore be required to apply the model to different areas and land use characteristics.

Stormwater and dry-weather qualities are combined at inlet manholes and routed through the sewers according to the flow velocity of the sewage. Dispersion is not modeled directly, but by averaging qualities between successive time steps. The equations consider first-order decay of BOD and sedimentation and scour of suspended solids. Reactions and interactions between the pollutants during the routing are not considered.

Plug flow or complete mixing can be specified for the routing of pollutants through storage facilities. Deposition of suspended solids can be computed, but not resuspension. Reactions and interactions in storage are not modeled.

Nine overflow treatment processes can be modeled and arranged by the user in any series or combination. The modeled unit processes are: bar racks, fine screens, sedimentation, dissolved air flotation, dissolved air flotation preceded by fine screens, microstrainers, high rate filters, effluent screens, and chlorination. Mathematical formulations for each process relate hydraulic capacity to removal efficiency. The user does not have the option of varying the process efficiencies without changing coefficients internal to the program. For some processes, however, the amount of chemicals used is read in to account for their effect on treatment performance.

BOD removal and suspended solids removal are considered independently. Coliform removal, however, is defined as a function of suspended solids removal.

Cost functions are built into the model which compute the cost of overflow treatment. Separate functions are defined for each unit treatment process and for storage and pumping if part of the treatment facility. Annual capital costs, land costs, and operation and maintenance costs are related to hydraulic capacity by power functions. The shapes of the cost functions are defined internally by the program. Costs can be adjusted by reading in the Engineering News Record Index for regional adjustments and future expected changes.

The Stormwater Management Model includes also a receiving water flow and quality segment which is based on the original EPA dynamic estuary model. Although review of this model segment is not within the planned scope of this study, it is noted because it adds considerably to the comprehensiveness of the Stormwater Management Model. It gives the model the capability not only of modeling flow and quality in sewerage systems, but

of assessing the impact of the sewerage system effluents on the receiving water quality.

The flow computations of the receiving water models are based on a simplified explicit finite difference solution of the one-dimensional dynamic wave equations for nonsteady gradually varied open channel flow (Saint-Venant equations).

Although the equations are written for one-dimensional flow only, a special solution technique allows the simulation of two-dimensional flow fields represented by an irregular grid of links and nodes. The continuity equation is solved at the nodes (junctions of the links) and the momentum equation is solved along the links (channels between the nodes). The model is very costly to use since very short time steps are required for numerical stability. The formulation becomes unstable if the discharges in adjacent channels and storage volumes of adjacent nodes are not of the same order of magnitude.

The model includes the transport of several conservative and nonconservative water quality constituents, considering first-order decay. Transport is modeled by convection only and chemical and biological interactions between different water quality constituents are not considered. Other receiving water flow and quality models developed recently for the Environmental Protection Agency for both steady and nonsteady state water quality predictions in river basins (including streams, impoundments and estuaries) consider several water quality constituents, including chemical and biological reactions and interactions.

Computer Program

The original version of the computer program, which is available from the U.S. Environmental Protection Agency, is written in Fortran IV for an IBM 360/65 computer and is also compatible for a UNIVAC 1108 computer. There are five main programs which, depending on computer core storage capacity, can be either loaded together or in sequence depending on the user's needs. The program was converted by Battelle-Northwest for a CDC 6400 computer as part of this study. This version allows the loading of the main programs in any sequence specified by the user.

Fairly complete documentation of the model was published by the EPA, including a summary report, user's manual, verification and testing report, and program listings. Unfortunately, no one of these reports presents a complete description of the theoretical bases and mathematical formulations of the model.

The equations for some modeled phenomena are described in the user's manual, and some are not described at all. The reader must compare the two reports to obtain a fair understanding of the capabilities and limitations of the complete model and the meaning of the input data. Also, the user's manual includes much discussion of model verification and testing which adds to the report's bulkiness and makes it more difficult to find essential information for the preparation of input data.

Program output includes tables and line printer plots of rainfall intensities for each raingage, combined runoff and quality for each subcatchment, and routed discharges and water quality at selected points of the sewerage system. Summaries of treatment effectiveness and costs are also available. Water levels and flow velocities in the sewers are not computed.

Evaluation

The EPA Stormwater Management Model is one of the most complete mathematical models available for the assessment and planning of storm and combined sewerage systems. Consideration of both wastewater flows and qualities is an important aspect of evaluating needed treatment facilities and the impact of sewage effluents on receiving waters. This is also one of the few models which include cost computations. Although the model does not consider costs in the sizing of sewers, the computation of the costs of overflow storage and treatment should be a valuable aid to the engineer.

Most model limitations which result from various approximations and simplifications in the wastewater flow and quality formulations are described above. Program limitations which may restrict the model's general applicability include the neglect of downstream hydraulic controls (with the exception of very rough approximations of backwater and surcharging conditions) and the absence of formulations for inline treatment and main treatment plants. Better formulations which consider both upstream and downstream flow conditions are also needed for diversion structures.

The model can be used only for the simulation of individual runoff events since it does not include provisions for either catchment moisture or water quality accounting between rainstorms.

The new version of the program includes an option to suppress the water quality computations and perform the flow simulations alone. This can save considerable computer time if water quality computations are not needed.

The model is very complex and a major effort is required to implement it. The poorly organized documentation makes it very difficult for the user to implement the model without many false starts resulting from misinterpretations of the theoretical bases of the model and the meaning of some input data.

The model has been tested to some extent on urban hydrologic data. Due to the lack of available comprehensive measured data of concurrent rainfall, runoff, and water quality, different parts of the model were tested with different sets of data. Numerical testing of the flow computations with data from catchments ranging in size between 5 and 2200 ha (13 and 5400 acres) showed satisfactory accuracy. The accuracy of water quality predictions can be expected to be only of the right order of magnitude. Considerable model improvement, particularly of the formulations relating water quality with land use, are needed before the water quality model can be used with confidence.

Improvements are needed in the output formats. The arrangement of the catchment runoff table makes it difficult to abstract the hydrograph of a particular subcatchment. Complete output of routed flow and quality can be obtained at only twenty selected locations. This is adequate for the evaluation of a few important locations, such as major outfalls. It is a serious limitation, however, for the evaluation of an entire sewerage system since repeated runs of the same data are required to obtain sufficient information on the adequacy, performance, and utilization of all modeled sewer system elements.

Model improvements are in progress at the University of Florida under an EPA contract to add snowmelt, to include more accurate flow and water quality routing schemes, to add new water quality parameters and unit treatment processes, and to simulate inline (main) treatment processes (Heaney et al., 1973). A new and improved user's manual is being prepared for the revised model (Huber et al., 1974). The model is also being revised to add continuous simulation capability for planning purposes (Smith, 1975). The model is being used by many consulting firms which can assist cities in model implementation, and a special program for model dissemination and user assistance was conducted by the University City Science Center of Philadelphia (Hagarman and Dressler, 1975).

HYDROCOMP SIMULATION PROGRAM

Summary

The Hydrocomp Simulation Program is one of the most comprehensive mathematical models for the simulation of both rural and

urban catchments. The water flow computations are based on the Stanford Watershed Model which was the first comprehensive mathematical model of catchment hydrology (Crawford and Linsley, 1966). The Hydrocomp Simulation Program, however, is a considerable improvement over the original Stanford Watershed Model, both with respect to mathematical formulations and data handling capability (Hydrocomp, 1972). Recently, a separate program was developed for the simulation of water quality in river basins which can be interfaced with the hydrologic and flow routing program (Hydrocomp, 1973).

The Hydrocomp Simulation Program is formulated for the continuous simulation of water flow and quality from several catchments and routing in converging branch sewer and open channel networks. Catchment moisture and water quality are accounted for during periods of no precipitation, so the model can be used for continuous simulation of several years. Special features are included for the simulation of impoundments and diversions, including the flow of water over spillways and through hydroelectric turbines. Real-time control and design and cost features are not included.

Seventeen water quality constituents and their reactions and interactions in natural water bodies can be simulated. The applicability of the formulations to combined sewage has not been established.

The Hydrocomp Simulation Program includes the following features:

1. dry-weather flow and quality of several catchments;

2. rainfall, snowfall, pan evaporation, air temperature, dewpoint, solar radiation, and wind velocity of several weather stations;

3. evapotranspiration;

4. snow accumulation and melt;

5. stormwater runoff and quality from pervious and impervious areas of several catchments from land use;

6. catchment moisture and water quality accounting during periods of no precipitation;

7. groundwater infiltration into sewers and open channels;

8. seventeen water quality constituents with reactions and interactions in natural (receiving) water bodies;

9. routing of combined wastewater flow and quality in a converging branch network;

10. circular closed conduits and trapezoidal open channels with trapezoidal flood plains;

11. upstream and downstream flow control and backwater;

12. diversion hydrographs;

13. in-line storage reservoirs with rule curve operation; and

14. water quality decay, reactions, and interactions in natural (receiving) water bodies.

The model does not include the following features:

1. dry-weather flow and quality from land use;

2. flow reversal, surcharging and pressure flow;

3. sedimentation and scour;

4. water quality decay, reactions and interactions in sewers;

5. flow and quality routing in loops and diverging branches;

6. noncircular closed conduits;

7. sewer flow control and diversion structures;

8. pumping stations;

9. wastewater treatment;

10. real-time control;

11. design; and

12. costs.

Methods

The Hydrocomp Simulation Program includes the most complete formulation of catchment hydrology of all the models reviewed. The model computes runoff from pervious and impervious areas separately and considers evapotranspiration, snowmelt, and soil moisture accounting.

Precipitation (rain or snow) data is input at constant time intervals for one or more raingages. Other meteorological data needed for the evapotranspiration and snowmelt computations are provided on a daily basis. Not more than one precipitation record can be assigned to a single subcatchment.

Snow accumulation and melt are computed by a method developed by the U.S. Corps of Engineers. The equations require data on precipitation, air temperature, dew point, solar radiation, and wind velocity. Programming defaults are used if some of these data are unavailable. Only precipitation and air temperature are essential.

Dry-weather flow and quality data are input at constant time intervals. Dry-weather quality can also be defined by a power function of dry-weather flow.

The potential infiltration rate is computed from an empirical function of soil moisture. The actual infiltration depends on the rainfall excess after subtracting interception losses rather than on the overland flow depth. The infiltrated moisture is divided into upper zone and lower zone storage. Upper zone storage includes depression storage. Part of the upper zone storage percolates into the lower zone storage and the rest is divided into overland flow and interflow, which become channel (or sewer) inflow. The lower zone storage is divided into a channel inflow contribution and into deep groundwater storage which does not contribute to surface runoff.

Interception, upper zone, lower zone, and groundwater storage are depleted by evaporation or evapotranspiration computed as functions of potential evapotranspiration and available moisture.

Overland flow is routed with a modified kinematic wave formulation using Manning's equation. Several empirical coefficients internal to the program relate surface detention storage to overland flow discharge.

Although the model's representation of infiltration, groundwater percolation and storage, overland flow, groundwater contribution to surface flow, and evaporation from all moisture sources

is based on physical concepts, the mathematical formulations are based largely on empirical relationships. They require several empirical coefficients, some of which are defined as soil moisture capacities. The model appears sufficiently complex that its use for prediction purposes without initial calibration with measured data may not be very reliable. Simpler models with fewer empirical coefficients seem adequate to model infiltration and surface flow contributions in urban catchments.

Flow routing in sewers and open channels is accomplished using the kinematic wave equation with Manning's equation. The solution considers the geometry of circular pipes and of trapezoidal open channels with trapezoidal flood plains. A diffusion term in the kinematic wave equations approximates downstream flow control and backwater conditions. Surcharging and pressure flow in sewers is not modeled but a warning is printed if it occurs. The time step for the routing computations is computed internally to maintain numerical stability.

Reservoirs and channels can be simulated in the channel network. Storage capacity has to be defined as a function of depth and reservoir discharge has to be defined by rule curves relating discharge with time. Diversions are modeled by requiring diversion hydrographs as input data. Flow control, diversions, and reservoir outlet structures are apparently not modeled using appropriate hydraulic equations for weir, gate, and orifice discharge.

The computation of stormwater quality is based on formulations first used by the EPA Stormwater Management Model and then expanded in the Corps of Engineers STORM Model and Water Resources Engineers Stormwater Management Model. The concentration of each pollutant washed off a catchment is computed as a nonlinear function of runoff. Special empirical functions are built in for water quality balance between runoff events to account for dirt and dust accumulation, natural decay, street cleaning practices, and different land uses. The pollutant accumulation between runoff events can vary with the calendar month. Different values can be specified for pervious and impervious areas. The equations and coefficients have not been sufficiently tested for reliable predictions without calibration with measured data.

Pure advection is used to route pollutants through the sewer and open channel network. Dispersion is approximated by a weighted average of concentration values at successive time steps. For receiving water bodies, chemical and biological reactions and interactions among the seven modeled water quality constituents are computed. Only one-dimensional flow and water

quality routing is formulated. Vertical water quality interchange can be approximated among three horizontal layers in impoundments by providing empirical mixing coefficients. The model documentation does not indicate whether reactions are computed for sewage. The model does not include formulations for wastewater treatment, real-time control, or design and cost computations.

Computer Program

The Hydrocomp Simulation Program is a proprietary model developed by the consulting firm Hydrocomp International, Inc., of Palo Alto, California. Separate user's manuals are available for the flow and water quality computations of the model. These contain sufficient detail covering the mathematical bases of the modeled phenomena and the required input data. The user must contract with Hydrocomp for routine application of the program. The firm conducts periodic workshops to instruct potential users in hydrologic simulation methods and application of the Hydrocomp Simulation Program.

The computer program consists of four main programs and is compatible with IBM 360 and 370 computers with a minimum of 240K bytes of core memory. Information is not published which would allow estimates of computer execution times as a function of problem size.

Computer output includes precipitation; soil moisture status; water stages, velocities, discharges, and water quality concentrations for all channels and storage; and volume of storage. Data input and output can be in metric or British units.

Evaluation

The Hydrocomp Simulation Program is the only available hydrologic model which can be applied for the continuous simulation of both water flow and quality in urban and rural watersheds and channel networks. It was originally developed for the hydrologic simulation of rural catchments, and recently water quality computations for natural rivers and impoundments were added.

Modifications for urban applications include the addition of separate runoff computations for pervious and impervious areas and flow and water quality routing in circular closed conduits. Other model additions, however, would make the model more generally applicable to urban sewerage systems: for instance, geometries of different closed conduit cross-sections, hydraulic equations for different flow control and diversion structures,

formulations for surcharging and pressure flow, and simulation of treatment processes.

This hydrologic model has been tested and applied extensively in many nonurban watersheds in the United States and abroad. Reported testing on urban data is limited to four catchments ranging in size from 100 to 1068 ha (248 to 2640 acres) with less than 22 percent imperviousness (Crawford, 1971). Comparisons between measured and computed runoff for selected storm events have produced generally good agreement, with some exceptions attributed to potentially unreliable measured data. Good agreement between measured and computed runoff are also reported for four stream gages in the partially urbanized North Branch of the Chicago River drainage basin, which has a drainage area of 264 km^2 (102 mi^2) and 13 percent imperviousness (Hydrocomp, 1970). Testing and application of the water quality model is in progress.

MASSACHUSETTS INSTITUTE OF TECHNOLOGY URBAN WATERSHED MODEL

Summary

The Massachusetts Institute of Technology (MIT) Urban Watershed Model simulates the time-varying runoff of several catchments and a sewer and open channel network including loops and converging and diverging branches (Harley et al., 1970). The model is limited to the simulation of single runoff events. Water quality and real-time control features are not included.

A separate model includes design features to compute the sizes and costs of sewers, storage and treatment facilities which will result in the least-cost combination of alternatives for the elimination of untreated overflows and the reduction of flooding and surcharging (Kirshen et al., 1972).

The original model was developed at MIT for the U.S. Office of Water Resources Research, but the model has been modified by Resource Analysis, Inc., for its routine application (Schaake et al., 1973).

The MIT urban Watershed Model includes the following features:

1. dry-weather flow of several catchments;
2. air temperature at one weather station;
3. several rainfall records;
4. special statistical analyses to compute design hyetographs (separate program);

5. evapotranspiration;

6. snow accumulation and melt;

7. option to choose one of four different infiltration equations;

8. stormwater runoff from pervious and impervious areas of several catchments;

9. catchment moisture accounting during periods of no precipitation;

10. flow routing in gutters;

11. routing of combined wastewater flow in a converging network of loops and converging and diverging branches;

12. various standard closed conduits and open channel cross-sections and arbitrary shapes;

13. downstream flow control, backwater, surcharging and pressure flow;

14. flow control and diversion structures;

15. storage facilities;

16. hydraulic capacities of treatment facilities; and

17. least-cost sizes of sewers, overflow storage and treatment plants meeting constraints on surcharging and untreated effluent volume (separate program).

The model does not include the following features:

1. dry-weather flow from land use characteristics;

2. catchment moisture accounting during periods of no precipitation;

3. flow reversal;

4. water quality; and

5. real-time control.

Methods

Dry-weather flow data for each inlet is input in the form of hydrograph values at constant time intervals. The model does not include provisions to compute dry-weather flow from land use.

Several rainfall intensity records can be provided as input data. Not more than one rainfall record can be used to compute runoff from a subcatchment. Special provisions are built into the model to move a rainstorm across a catchment by specifying its direction and velocity of movement. A separate model is available which computes design hyetographs of specified frequencies from measured rainfall data.

Initial losses to fill depression storage on pervious and impervious areas are subtracted before surface runoff begins. The Resource Analysis version has four options to compute infiltration on the pervious areas: Horton's equation, Holtan's equation, a U.S. Soil Conservation Service method, and a runoff coefficient method. Infiltration is subtracted from rainfall if the last two methods are used, but computed from overland flow depth if Horton's or Holtan's equation is used. A method based on filter theory can be used to estimate the infiltration parameters from measured rainfall and runoff (Leclerc and Schaake, 1973). Snow melt is computed by the Corps of Engineers degree-day method which requires mean daily air temperature as input data. The Penman equation is programmed to compute evaporation but generally not used for single runoff event simulation.

Flow routing is accomplished with the kinematic wave equation. The equations are solved by a finite difference scheme for overland flow, flow in gutters, and flow in open channels and for various standard cross-sections and any arbitrary shape closed conduits. Downstream flow control and backwater can be considered if the stage-discharge relationship is known. Surcharging and pressure flow is computed separately for each pipe reach. Flow reversal is not modeled. Weir and orifice flow control and diversion structures can be modeled by their hydraulic equations.

The design model computes the sizes and costs of circular sewers, the volumes of overflow storage facilities, and the hydraulic capacities of treatment plants needed to reduce undesirable flooding and surcharging and to eliminate untreated overflows. Linear programming is used to determine the least-cost combination of these facilities for a selected design storm event. The optimization is based on needed hydraulic

capacities alone and does not consider water quality. It requires catchment runoff hydrographs as input data and uses a simplified flow routing scheme.

Computer Program

The MIT Urban Watershed Model was developed under a series of projects for the U.S. Office of Water Resources Research. Each project covered a different aspect of the overall model development. The computer programs were written for an IBM 360/67 in Fortran IV. The design program utilizes the linear programming package of the 1971 IBM Mathematical Programming System Extended (MPSX). The model's core storage requirement is not documented. The potential user has to contract with Resources Analysis, Inc., of Cambridge, Massachusetts, for routine applications of the model.

Computer output of the model includes tables of rainfall intensities and overland, catchment and channel depth, velocity and discharge. Samples of program output are not included in the documentation for the design option but output for this option includes the volume and duration of flooding for each sewer and the required sizes and costs of sewer overflow storage facilities and treatment plants.

Evaluation

The MIT model is a useful and efficient tool for the simulation of urban and nonurban catchments, including both sewer and natural stream networks where backwater, downstream flow control and surcharging are not important. Testing of the model on catchments with drainage areas up to 120 km^2 (46 mi^2) shows good agreement between computed and measured runoff (Leclerc and Schaake, 1973; Resource Analysis, 1974). The model is being used extensively for practical engineering assessments.

MINNEAPOLIS-SAINT PAUL URBAN RUNOFF MODEL

Summary

The Minneapolis-Saint Paul Urban Runoff Model was developed for real-time forecasting of flows in the major trunk and interceptor sewers of the Minneapolis-Saint Paul combined sewerage system to reduce untreated overflows to the Mississippi River during rainstorms (Minneapolis-Saint Paul Sanitary District, 1971). The model computes the runoff of several large catchments, diverts the flows at controllable regulating structures, and routes them through a converging branch sewerage network to the treatment plant. It is specifically designed

for real-time control operation on a small computer and cannot be used for the assessment of existing or the design of new sewerage systems since it uses a highly simplified flow routing procedure whose coefficients have to be calibrated with measured data or derived from more sophisticated models.

Although the model was designed for continuous simulation by incorporating provisions for the accounting of catchment moisture between rainstorms, the necessary formulations were never added and the program can be used only for the simulation of single runoff events. The real-time control operation, which is not based on mathematical optimization techniques, requires repeated trial and error runs with estimates of regulator settings. The model does not include water quality, design, and cost computations.

The Minneapolis-Saint Paul Urban Runoff Model includes the following features:

1. dry-weather flow of several catchments;
2. several rainfall records;
3. stormwater runoff from pervious and impervious areas of several catchments;
4. routing of combined wastewater flow in a converging network of unspecified sewer cross-sections;
5. six types of diversion structures;
6. telemetry system for real-time acquisition of rainfall, flow level, and regulator status data; and
7. remote control of diversion gates and weirs.

The model does not include the following features:

1. dry-weather and stormwater flow from land use;
2. evapotranspiration;
3. snow accumulation and melt;
4. catchment moisture accounting during periods of no precipitation;
5. flow routing in gutters;

6. flow routing in loops and diverging branches;

7. noncircular closed conduits and open channels;

8. downstream flow control, backwater, flow reversal, surcharging, and pressure flow;

9. pumping stations;

10. storage facilities;

11. water quality;

12. design; and

13. costs.

Methods

Dry-weather flow data is input in the form of average values for each catchment and adjustment factors to account for diurnal (hourly) variations. The model does not include provisions to compute dry-weather flow from land use. Storm runoff is computed for each catchment from the weighted average rainfall of several raingages. The raingage record is input in the form of cumulative values and can be given at irregular intervals. This is a distinct advantage over the common requirement of most models that rainfall intensities be furnished at constant time intervals, since it eliminates the need to compute intensities prior to running the program and to read in zeros for periods of no rain.

The rainfall excess contribution of each catchment is computed separately for pervious and impervious areas. Runoff does not begin until all depression storage is filled. Other losses are computed for impervious areas by an exponential function and by a modified Holtan's equation for pervious areas. Both formulations are functions of cumulative losses and consequently account for catchment moisture conditions. This is a better formulation than others which treat potential infiltration as a function of time only without regard to the actual infiltration. The Minneapolis-Saint Paul Model, on the other hand, subtracts all losses from rainfall, which is less accurate than basing them on overland flow and depth.

The excess precipitation is then convoluted with unit hydrographs to determine the storm runoff from each catchment. The unit hydrographs are computed from catchment characteristics (including drainage area, length, slope, soil, and vegetation) by a method of the U.S. Soil Conservation Service. The same

unit hydrograph is applied to runoff from both the pervious and impervious portion of each catchment, which may lead to inaccuracies and calibration problems.

The unit hydrograph method was selected because of its rapid computer implementation for the computation of runoff from large catchments, an important factor in real-time control of major trunk sewer regulators. It allows the simulation of the entire Minneapolis-Saint Paul combined sewerage system by only 15 catchments.

The progressive average lag method is used to route the flows in the sewers. The method requires two empirical coefficients (an average travel time and the number of upstream flow values to be averaged to obtain a downstream routed flow value) which must be obtained by calibration with measured data or by derivation with a different routing method. Since sufficient data are not available in a sewerage system, the coefficients were obtained for the Minneapolis-Saint Paul system through solution of the characteristic form of the dynamic wave equations for a single pipe. Noncircular pipes were approximated by circular pipes of equal cross-sectional area. The progressive average lag method does not require any information on the geometry of the modeled pipes or open channels once the routing coefficients are known. It was chosen since it is extremely fast and was considered sufficiently accurate for real-time control purposes. It cannot be used for the assessment or design of sewerage systems where relationships between the geometry of the conduits and channels and the hydraulic performance of the system are required.

Diversions are computed at regulators using combinations of orifice and weir equations which depend on the geometric configuration of each structure. The program includes formulations for the following three common types of structures: perpendicular weir overflow with orifice dry-weather outlet (weir may have other angles to flow direction), orifice or gate overflow (without weir), and a structure which overflows when flow exceeds a preset maximum. In addition, three special regulators in the Minneapolis-Saint Paul trunk sewers (which probably would not occur in other systems) were programmed.

Computer Program

The computer program is written in Fortran IV for a DEC PDP-9 computer with 16K words of core memory. The program consists of seven main programs which are run sequentially. The program is interfaced with a real-time data acquisition system for rainfall, water level and regulator status data. Remote control of regulator weir and gate positions is accomplished manually.

Program output includes rainfall intensities and catchment moisture conditions for each catchment and discharges at regulators and selected interceptor sewer locations. Water depths and velocities are not computed.

Evaluation

The model is intended primarily for the real-time forecasting of flows in major trunk and interceptor sewers. It is formulated to run on a small process computer within time constraints imposed by the real-time forecasting requirement. As a consequence the mathematical formulations include several simplifications, particularly in the catchment runoff and sewer routing portions, to stay within the time and core storage constraints of the small computer. These simplifications make the model unsuitable for assessment of existing sewerage systems or the design of new sewerage systems. They merit consideration, however, for schemes involving real-time data acquisition and control of overflows during rainstorms.

The model does not include mathematical optimization techniques to determine optimal regulator settings which would minimize overflows during rainstorms subject to capacity constraints of the existing sewerage system and the treatment plant. In its present form, it has to be run repeatedly with trial values of regulator settings until the desired regulator operation is found which would maximize the utilization of system capacity and minimize overflows. Since this type of operation is time consuming and fairly inefficient, the model is no longer used for real-time control. Instead, regulator settings are determined during a rainstorm by the operators on the basis of their experience using the telemetered raingage and water level readings.

SEATTLE COMPUTER AUGMENTED TREATMENT AND DISPOSAL SYSTEM

Summary

The Computer Augmented Treatment and Disposal System (CATAD) of the Municipality of Metropolitan Seattle (Metro) is an operating system for the real-time control of untreated overflows from the main trunk and interceptor sewer regulators of the metropolitan Seattle, Washington, combined sewerage system (Municipality of Metropolitan Seattle, 1971; Gibbs and Poole, 1972; Gibbs et al., 1972; Mallory and Leiser, 1973; Leiser, 1974). Although it does not include a comprehensive mathematical model for the simulation of combined wastewater runoff at this time, the system is reviewed in this study because of its capability for automatic computer control of diversion structures and pumping stations.

The system includes a small process computer, real-time data acquisition of rainfall, water level and quality data, and remote automatic computer control of regulators and pumping stations. The control scheme uses rule curves relating trunk sewer storage upstream of regulators with time and appropriate hydraulic equations for each regulator and pumping station to maximize storage in the sewerage system and minimize untreated overflows. Each control structure is controlled independently and potentially undesirable effects on overall system performance are handled by manual overrides. Real-time wastewater quality data are not utilized as criteria for determining overflow rates and locations.

The Computer Augmented Treatment and Disposal System includes the following features:

1. rainfall at six raingages (more being added);

2. rule curves for each major trunk and interceptor sewer regulator relating storage with time for typical rainfall patterns;

3. hydraulic equations for each diversion structure to compute discharge considering both upstream and downstream hydraulic conditions;

4. hydraulic equations for each diversion structure to compute gate settings required to obtain desirable regulator discharge;

5. two types of pumping stations;

6. inverted siphons;

7. automatic real-time data acquisition for rainfall, water levels and quality, and regulator and pumping station status; and

8. remote automatic computer control of controllable regulators and pumping stations.

The system does not include the following features;

1. precipitation-runoff computations;

2. flow routing in gutters, sewers and open channels;

3. storage reservoirs;

4. water quality within the system;

5. design; and

6. costs.

Methods

The Computer Augmented Treatment and Disposal System is operational to the extent of providing automatic control of sixteen regulators and two pumping stations. The remote control is determined using real-time data acquisition of rainfall from six raingages and water levels upstream and downstream of controllable flow regulating structures and pumping stations. Rule curves are used to determine the operation of the regulators and pumping stations which will maximize available sewer storage capacity and minimize untreated overflows.

The rule curves represent sewer storage capacities as functions of time for the sewer upstream of each controlled regulator and pumping station. Each curve was developed from runoff patterns of representative historical rainstorm patterns and represents the desirable filling schedule for a sewer based on its storage capacity. The rule curves may be adjusted as storm patterns vary and as storage requirements change.

Hydraulic equations were programmed in two modes for each type of regulator. The first mode is used to evaluate regulator performance under different flow conditions and regulator settings. It computes the regulator discharges for given inflows and regulator settings. The second (control) mode is used during the real-time control operation to determine the regulator settings (such as gate heights) which will result in the desired regulator discharges as determined by the rule curves for the predicted runoff pattern.

The present control scheme uses simulation at 1/2-hour intervals to compute the required regulator and pumping station operation for the following 1/2 hour. Regulator and pumping station controls are then reset at 5- or 10-minute intervals, depending on the rate of change of the runoff. All flow level gages at the regulator and pumping stations are scanned every minute and all raingages every 5 or 10 minutes to provide the real-time data for the control scheme. The system is therefore operating in full computer control mode. Manual override is possible for alarm conditions.

At present, comprehensive optimization is not used to maximize utilization of available sewerage system flow and storage capacity. The effect of flow routing is considered by the control

system, however, by automatically adjusting flow control structures if excessive flow conditions occur in the sewers. Different approaches to the development of a comprehensive sewerage system simulation and control model are being investigated. Separate submodels have been programmed, such as the computation of runoff from rainfall using the unit hydrograph method and flow routing in the sewers using the dynamic wave equations. These submodels have not been combined with the regulator equations into a comprehensive simulation model for the entire trunk and interceptor sewerage system.

Computer Program

The computer programs are written in Fortran for a Xerox Sigma 2 computer with 44K words of core memory, of which 31K are used for the data acquisition and remote control operation (foreground operation) while the remaining 13K are available for other purposes (background operation). The programs are system specific and not transferable for general application elsewhere.

Evaluation

This is an operational system for real-time automatic computer control of untreated overflows during rainstorms. The system has been shown to be very effective in reducing untreated combined overflows to the receiving waters. The system's effectiveness could be improved by replacing the rule curve operation with systematic optimization techniques which maximize the utilization of available sewer flow and storage capacity and consider constraints on the quality of the overflows.

SOGREAH LOOPED SEWER MODEL

Summary

The Looped Sewer Model of the French consulting firm Societe Grenobloise d'Etudes et d'Applications Hydrauliques (SOGREAH) simulates the time-varying runoff of combined sewerage systems consisting of several catchments and a sewer and open channel network including loops and converging and diverging branches (SOGREAH, 1973 - 5 reports). The model includes formulations for most hydraulic phenomena encountered in closed conduit and open channel networks. The flow routing solves the dynamic wave equations coupled with equations for special sewer system facilities, such as diversion structures, pumping stations, inverted siphons, and retention basins. The solution considers both upstream and downstream boundary conditions, backwater, surcharging, pressure flow, and flow reversal. Conservative water quality constituents can be routed through the sewerage network.

The model appears to be limited to the simulation of single runoff events. Real-time control and design features are not included. This is a proprietary model and not all details of the model formulations are available. The firm has North American branch offices in New York City and Lasalle, Quebec, Canada.

The SOGREAH Looped Sewer Model includes the following features:

1. dry-weather flow of several catchments;
2. design hyetographs and hydrographs from catchment characteristics or several rainfall records;
3. stormwater runoff from pervious and impervious areas of several catchments;
4. routing of combined wastewater flow in a network of loops and converging and diverging branches;
5. circular and egg-shaped closed conduits, trapezoidal open channels, and any arbitrary shapes;
6. backwater, upstream and downstream flow control, surcharging, pressure flow, and flow reversal;
7. flow control and diversion structures considering both upstream and downstream flow conditions;
8. inverted siphons;
9. pumping stations;
10. overflow and in-line storage facilities;
11. receiving water flow (separate program); and
12. routing of conservative and first-order decaying water quality constituents.

The model does not include the following features:

1. dry-weather flow and quality and stormwater quality from land use characteristics;
2. input of recorded rainfall data;
3. evapotranspiration;
4. snow accumulation and melt;

5. catchment moisture accounting during periods of no precipitation;

6. sedimentation and scour;

7. wastewater quality decay, reactions and interactions;

8. real-time control;

9. design; and

10. costs.

Methods

Dry-weather flow data for each inlet is input in the form of hydrograph values at constant time intervals. The model does not include provisions for computing dry-weather flow from land use.

Options exist to read in several rainfall records, move a rainstorm over a catchment in a specified direction, or compute a synthetic design rainfall. The synthetic rainfall is defined by a maximum design rainfall of specified frequency for each catchment using a formula developed by Caquot. The formula considers catchment characteristics such as slope, percent imperviousness, and drainage area and contains eight empirical coefficients. The maximum design rainfall is transformed into a design rainfall hyetograph by a second formula which requires two additional empirical coefficients. The rational formula or the Horton equation can be used to compute the rainfall excess hyetograph from either measured or synthetic hyetographs.

The rainfall excess hyetographs are then routed over each catchment to the sewer inlets using the Muskingum flood routing method. This requires two additional empirical coefficients, one of which is estimated and the other computed with a formula suggested by Schaake and Geyer. The formula considers the percent imperviousness and the overland flow length and slope and contains four additional empirical coefficients. The two Muskingum coefficients are adjusted from runoff measurements if available.

The formulations for the catchment runoff do not consider evapotranspiration, snow accumulation and melt, and catchment moisture accounting during periods of no precipitation. Consequently, the model is not suited for continuous simulation of catchment runoff.

Flow routing is accomplished with the dynamic wave equations for nonsteady gradually varied open channel flow (Saint-Venant equations). Strickler's equation (same as Mannings's equation) is used to compute the energy gradient. An implicit finite difference solution is employed which considers various upstream and downstream boundary conditions, including the performance of various types of special sewerage system facilities. The solution technique computes backwater and flow reversal and special formulations are included to handle surcharging, pressure flow, and flooding at sewer inlets. This gives the model the capability to simulate nonsteady flows not only in a converging branch network but also in diverging branches and loops. Equations for circular and egg-shaped closed conduits and for trapezoidal open channels are built into the program. The user can add subroutines for other cross sections and also read in geometric data for any arbitrary shapes.

Equations are built into the program to compute the discharge over weirs and through gates and orifices. Regulator gate positions can be read in as functions of time. Inverted siphons and two types of pumping stations can also be simulated. The equations for these special facilities consider both upstream and downstream flow conditions and their solutions are coupled with the solution of the dynamic wave equations for the flow routing. The model appears to simulate storage facilities but details on relevant formulations were not available.

The flow routing portion of the model is set up for continuous simulation. The user can read in initial conditions for every sewer system element, stop the computations after any number of simulated hours, and restart the simulation using the final conditions of the previous run as the new initial conditions.

The model does not include real-time control and design and cost computations. A water quality program is being developed and the formulations for routing of conservative and first-order decaying substances through the sewerage network are operational. Pollutant transport is computed by an explicit finite difference solution of the advection equation.

Computer Program

This is a proprietary model developed by the French consulting firm Societe Grenobloise d'Etudes et d'Applications Hydrauliques of Grenoble, France. North American representatives are in New York City and Lasalle, Quebec, Canada. A user's manual is available in French, but supporting documentation on model features and formulation is inadequate to fully assess the model's capabilities and limitations.

The program consists of five main programs which are loaded in sequence and coupled through common input/output data files. The program was written in Fortran IV for an IBM 360/65. The complete program has about 3,000 Fortran statements.

For program operation, only one of the five main programs is loaded into core at any one time. The program uses dynamic core allocation. A core storage requirement of 65K bytes is estimated for a simulation of 100 sewerage system nodes with 1000 computational points (where stages and discharges are computed). The execution time for this system is estimated at 0.25 seconds for the generation of each 3-hour inlet hydrograph and 2.7 seconds per time step for the routing computations.

Program output includes tables and plots of water depths (or pressures), velocities and discharges for each computational point. Program input and output are in metric units.

Evaluation

The SOGREAH Looped Sewer Model includes very comprehensive formulations for the routing of flow in sewerage networks. It can simulate all nonsteady gradually varied flow phenomena, surcharging and pressure flow, and hydraulic performance of special facilities for complex sewerage systems, including loops and converging and diverging branches. The implicit solution of the Saint-Venant equations coupled with equations for special facilities is complicated, however, and longer computer execution times can be expected than for simpler models. The length of the time steps can be changed during the simulation to save computer time. For example, short time steps can be used during rapidly varying flow conditions and longer time steps during slowly varying flow conditions without loss of accuracy. Numerical testing indicated that the implicit solution is computationally as fast as some solutions of the simpler kinematic wave equation.

The model would be useful primarily where the consideration of backwater, downstream flow control, diversion structures, retention basins, surcharging, and flow reversal are important features of the sewerage system assessment. If these features are not present or are considered insignificant, simpler models requiring less computer time can be used.

The model is based on a mathematical model for flows in river basins developed by SOGREAH earlier. Due to satisfactory verification results with the river basin model, SOGREAH did not report verification of the sewer model with measured urban rainfall and runoff data.

UNIVERSITY OF CINCINNATI URBAN RUNOFF MODEL

Summary

The University of Cincinnati Urban Runoff Model simulates the time-varying runoff of storm sewerage systems consisting of several catchments and a converging branch sewer and open channel network (University of Cincinnati, 1970; Papadakis and Pruel, 1972). The model does not include provisions for dry-weather flow, water quality, real-time control and design. It is limited to the simulation of single runoff events.

The University of Cincinnati Urban Runoff Model includes the following features:

1. one rainfall record;
2. stormwater runoff from pervious and impervious areas of several catchments;
3. flow routing in gutters;
4. routing of stormwater runoff in a converging branch network; and
5. circular closed conduits and rectangular open channels.

The model does not include the following features:

1. dry-weather flow;
2. consideration of nonuniform catchment rainfall distribution;
3. evapotranspiration;
4. snow accumulation and melt;
5. catchment moisture accounting during periods of no precipitation;
6. flow routing in loops and diverging branches;
7. noncircular closed conduits;
8. downstream flow control, backwater, flow reversal, surcharging, and pressure flow;
9. flow control and diversion structures;

10. pumping stations;

11. storage facilities;

12. water quality;

13. real-time control;

14. design; and

15. costs.

Methods

The University of Cincinnati Urban Runoff Model accepts the input of only one hyetograph for the entire drainage basin. Although this may be a weighted average of several raingage records, it neglects the areal nonuniformity of rainfall over the catchment. Losses to depression storage for both pervious and impervious areas are subtracted as a function of available depression storage capacity and cumulative rainfall and losses. Infiltration on pervious areas is computed with Horton's equation, with its time origin offset to balance cumulative rainfall and potential infiltration. This could improve the computation or infiltration compared to its unmodified use; other inaccuracies are introduced, however, since all losses are computed from rainfall without considering the overland flow depth and storage, and Horton's equation does not consider soil moisture conditions.

Overland flow is computed by a storage routing technique similar to the method used by the Stanford Watershed Model and Hydrocomp Simulation Program. The method uses Manning's equation and includes several empirical coefficients internal to the program to relate surface detention storage with overland flow.

The gutter flow routing uses a steady state approach, assuming that the gutter outflow equals its inflow during the same time interval.

Flow routing in the sewers is accomplished by a simple translation of the upstream hydrograph by its average travel time computed with Manning's equation. The formulation is extremely approximate, since it neglects the effects of varying travel times as functions of the discharge. The formulation does not consider downstream flow control, backwater, flow reversal, surcharging, and pressure flow. Special sewer system elements, such as flow control and diversion structures, are not modeled.

Computer Program

The computer program is written in Fortran IV for an IBM 360 and is available from the University of Cincinnati. The program has about 800 statements. Its core storage and running time are not published.

Program output primarily includes discharges at selected points. Water depths and velocities are not printed.

Evaluation

The University of Cincinnati Urban Runoff Model contains several simplifications, as described above, which appear to severely limit its accuracy. The assumption of uniform rainfall distribution, the neglect of dry-weather flows, the computations of infiltration from rainfall rather than available moisture on the catchment, the neglect of catchment moisture conditions on potential infiltration rates, the steady state flow routing in gutters, and the sewer flow routing technique neglecting dynamic flow phenomena impose severe constraints on the model's applicability for complex combined sewerage systems.

Although some encouraging comparisons between measured and computed results have been reported for the 5.2 ha (12.9 acre) Oakdale catchment in Chicago, Illinois and the 964 ha (2,380 acre) Bloody Run catchment in Cincinnati, Illinois (Papadakis and Pruel, 1973), they were restricted to single rainfall events and required considerable calibration, particularly with respect to the selection of infiltration values. Other testing on the 70 ha (173 acre) Vine Street catchment in Melbourne, Australia and on the 500 ha (1,240 acre) Yarralumla Creek catchment in Canberra, Australia using more complex storm patterns showed considerable differences between measured and computed runoff values (Heeps and Mein, 1974).

UNIVERSITY OF ILLINOIS STORM SEWER SYSTEM SIMULATION MODEL

Summary

Considerable research is in progress at the University of Illinois on the development of hydrologic models in general and urban runoff models in particular. A comprehensive mathematical model combining rainfall-runoff computations for urban catchments and sewer flow routing in a single computer program has not been developed.

A model is available, however, for the nonsteady routing of flows in converging branch sewer networks. It is based on the solution of the dynamic wave equations which consider upstream and downstream flow control, backwater, and flow reversal

(Sevuk, 1973; Sevuk et al., 1973). The model requires hydrographs as input data but could become a component in a comprehensive urban hydrologic model including rainfall-runoff computations. The model does not include water quality and real-time control features. A design feature determines sizes of circular pipes for peak flows. Costs are not considered. Separate hydrologic catchment models have been developed (Chow and Kulandaiswamy, 1971; Chow and Ben-Zvi, 1973) and research for the development of an urban catchment model is in progress for potential interfacing with the routing model.

The University of Illinois Storm Sewer System Simulation Model includes the following features:

1. dry-weather and storm runoff hydrographs of several catchments have to be provided as input data;

2. routing of combined wastewater flow in a converging sewer network;

3. circular closed conduits;

4. upstream and downstream flow control, backwater, and flow reversal;

5. weir, gate or pumping station flow controls without diversions at the outlet;

6. in-line storage facilities; and

7. pipe sizes for peak flow.

The model does not include the following features:

1. dry-weather flow from land use;

2. precipitation-runoff computations;

3. flow routing in gutters;

4. flow routing in loops and diverging branches;

5. noncircular closed conduits and open channels;

6. surcharging and pressure flow;

7. diversion structures;

8. in-line pumping stations;

9. water quality;

10. real-time control; and

11. costs.

Methods

The University of Illinois Storm Sewer System Simulation Model is strictly a flow routing model and does not include the computation of dry-weather flows from land use or the computation of runoff from precipitation. Inflows to the modeled sewer network have to be provided in the form of hydrographs as input data. The flows are routed through a converging network of circular sewers with the equations for nonsteady gradually varied open channel flow (Saint-Venant equations). The characteristic partial differential equations are solved by a first-order explicit finite difference formulation. The Darcy-Weisbach equation is used to define the energy gradient. Weir and gate controls and pumping rates can be specified as downstream boundary conditions, but diversions are not computed. The formulation considers upstream and downstream flow control, backwater, flow reversal, and in-line storage. Surcharging and pressure flow are not computed.

The model includes a design option for computing the sizes of circular sewer pipes using an iterative approach. The diameters of the upstream inlet pipes are first computed for the peak inflow. The diameters of the remaining pipes of a branch are then computed based on a kinematic wave routing procedure. The resulting diameters are then corrected based on the more exact flow routing scheme using the dynamic wave equations. This approach will result in a hydraulically sound design but the solution does not consider costs.

Computer Program

The computer program is written in PL/1 and assembler language for an IBM 360/75 computer and consists of approximately 3000 statements. The program and a user's manual are available from the University of Illinois. The program is written specifically for IBM 360 and 370 systems and would require considerable reprogramming for use on other computers. The program is run with 220k bytes of core memory, but could be run with as little as 100k for small sewerage networks. A memory of at least 300k bytes is recommended by the authors, however, for efficient applications.

Program output includes tables of depths, velocities and discharges, Calcomp plots of depths and discharges at selected points of the network, and line printer plots of the inflow hydrographs at the sewer entrances. Pipe sizes are printed for the design option.

Evaluation

The University of Illinois Storm System Simulation Model includes a fairly comprehensive formulation of free-surface flow in circular pipes. It can simulate all nonsteady gradually varied flow phenomena but does not include provisions for the simulation of surcharging and pressure flow. It cannot simulate loops and diverging branches and diversion structures.

The explicit numerical solution of the Saint-Venant equations for special junction and downstream outlet facilities is fairly straightforward, but can be expected to require long computation times as a result of the short computational time steps required to maintain numerical stability. The solution technique includes a special overlapping feature to model large sewerage systems in smaller segments, thus increasing the size of a network that can be simulated with the available computer core storage capacity. A disadvantage of the method is that two iterations are required to solve the dynamic wave equations for the entire network. The resulting increase in computer time is offset somewhat, however, by allowing different time steps in different segments of the network.

A special feature of the model is its capability to dimension circular sewers. The method does not result in a least-cost solution. It is the only design method among the reviewed models, however, which considers the complete dynamic wave equations in the design procedure. Consequently, it can be expected to produce a more accurate sizing of pipe diameters than other design methods based on more approximate routing methods, provided satisfactory design inflows can be defined. Although the model has the capability for continuous simulation, the cost of its application for design purposes using several years of records would be very high.

No application or verification of the model with real urban catchment and sewerage system data is reported. Testing of the model with data from laboratory pipe flow experiments and sensitivity analyses with hypothetical data indicate, however, that high accuracy can be expected for the routing of flows through converging networks of circular pipes.

UNIVERSITY OF MASSACHUSETTS COMBINED SEWER CONTROL SIMULATION MODEL

Summary

The University of Massachusetts Combined Sewer Control Simulation Model simulates the time-varying runoff of several catchments and a single string of circular sewers (Ray, 1972). The model computes surface runoff from impervious areas only using hourly rainfall data which may be computed by a separate Markov chain model. The flow routing is accomplished with an implicit solution of the dynamic wave equations which consider upstream and downstream flow control, backwater and flow reversal. Special sewerage system facilities, such as diversion structures and storage facilities, are not modeled. The model is formulated for continuous simulation but neglects evapotranspiration and snow accumulation and melt. Water quality, real-time control, and design features are not included.

The University of Massachusetts Combined Sewer Control Simulation Model includes the following features:

1. one rainfall record;
2. generation of synthetic hourly rainfall;
3. stormwater runoff from impervious areas of several catchments;
4. routing of combined wastewater flow through a single string of circular pipes; and
5. upstream and downstream flow control, backwater, and flow reversal.

The model does not include the following features:

1. dry-weather flow from land use characteristics;
2. consideration of nonuniform distribution of catchment precipitation;
3. evapotranspiration;
4. snow accumulation and melt;
5. catchment moisture balance during periods of no precipitation;
6. stormwater runoff from pervious areas;

7. flow routing in gutters;

8. flow routing in loops, converging and diverging branches;

9. noncircular closed conduits and open channels;

10. surcharging and pressure flow;

11. flow control and diversion structures;

12. pumping stations;

13. storage facilities;

14. water quality;

15. real-time control;

16. design; and

17. costs.

Methods

Dry-weather flow data can be input as a constant flow value for each catchment. One rainfall hyetograph can be input for the entire catchment. Hourly rainfall can be computed from a Markov chain model using recorded rainfall data to generate synthetic rainfall. A sixth-order Markov chain determines the lengths of dry and wet periods and a first-order Markov chain computes hourly precipitation values during the wet periods. The Markov chain model allows the evaluation of potential rainfall events which have the same statistical properties as the recorded events but which occur in different sequences of wet and dry periods.

Surface runoff from pervious areas is neglected. For impervious areas, it is assumed that all rain falling during an hour becomes surface runoff during the same hour after an initial loss has been satisfied. Moisture recovery during dry periods (such as evapotranspiration) is not considered. The model is formulated for continuous simulation of periods up to one month but does not consider snow accumulation and melt.

The combined wastewater flow is routed through a single string of circular pipes with multiple inlets for the runoff from the modeled subcatchments. The flow routing is accomplished by an implicit finite difference solution of the dynamic wave equations for nonsteady gradually varied open channel flow (Saint-Venant

equations). The friction slope is defined by Manning's equation. The solution considers upstream and downstream flow control, backwater, and flow reversal. The model, however, appears to assume free outfall at the downstream end of the pipe string and does not simulate special flow control and diversion structures.

Computer Program

The computer program is written for a CDC 3600 and consists of two main programs. The first is used to compute synthetic hourly rainfall for each subcatchment from recorded rainfall and the second routes the combined wastewater flow from several subcatchments through a single string of pipes. Details of the program, such as computer language, number of statements, and estimates of computer running times, are not published. The computer program is available but a user's manual has not been written.

Computer output includes rainfall intensities for each subcatchment and discharges and depths at selected points of the pipe string.

Evaluation

The University of Massachusetts Combined Sewer Control Simulation Model is of interest since it includes the continuous generation of hourly synthetic rainfall and the only implicit solution of the Saint-Venant equations for circular pipes among the generally available models. These two features merit consideration for incorporation into more comprehensive sewerage system models.

The implicit flow routing scheme would require extensive modification and expansion, however, to include special boundary conditions not considered in the present version. To be more generally applicable, a model would have to simulate a network of sewers, diversion and flow control structures, and storage facilities.

The computation of runoff using hourly values restricts the model to the evaluation of large systems where rainfall-runoff response times are accordingly long. The addition of a runoff model for pervious areas, the consideration of a time lag between the occurrence of rainfall and its resulting runoff and the simulation of evapotranspiration and snow accumulation and melt would be necessary to make the model a truly continuous simulation model.

The model has not been tested with real catchment data. Testing with hypothetical and experimental pipe data was inadequate to indicate model accuracy. The testing indicated, however, that the matching of hourly runoff computations with the implicit dynamic flow routing technique is practical, since the routing

technique allows equally long time steps provided they do not obscure significant runoff peaks of shorter duration. This limitation would make the model most useful for the simulation of large catchments and sewer pipes, such as the lumping of smaller subcatchments to compute inflows and routing only in major trunk and interceptor sewers without the need to simulate every block and lateral sewer.

WATER RESOURCES ENGINEERS STORMWATER MANAGEMENT MODEL

Summary

The Stormwater Management Model of Water Resources Engineers, Inc. (WRE) is a modified version of the Stormwater Management Model of the U.S. Environmental Protection Agency. It simulates the time-varying combined storm and sanitary runoff and wastewater quality of several catchments and a sewer and open channel network, including loops and converging and diverging branches (Shubinski and Roesner, 1973).

The flow routing procedure is based on the dynamic wave equations which consider backwater, flow reversal and both upstream and downstream flow control. A special formulation is built in for the solution of surcharging and pressure flow. The solution is coupled with the hydraulic equations for flow control and diversion structures considering both upstream and downstream flow conditions. The model simulates overflow storage basins but not in-line storage facilities.

Both dry-weather and stormwater quality are computed for six constituents: suspended solids, settleable solids, biochemical oxygen demand, nitrogen, phosphorus, and oil and grease. The pollutants are routed through the sewer system but treatment processes are not modeled. A separate model for simulating both flow and quality in receiving waters is available (Chen and Orlob, 1972).

The model does not include real-time control and design features and is limited to the simulation of single runoff events. The model has been released to the City of San Francisco and is in the public domain.

The WRE Stormwater Management Model includes the following features:

1. dry-weather flow and quality of several catchments from land use;

2. several rainfall records;

3. stormwater runoff and quality for pervious and impervious areas of several catchments;

4. six water quality constituents: suspended solids, settleable solids, biochemical oxygen demand, nitrogen, phosphorus, and oil and grease;

5. BOD concentrations of stormwater runoff dependent on suspended and settleable solids runoff;

6. contribution of catchbasins to stormwater quality;

7. routing of combined wastewater flow and quality in a network of loops and converging and diverging branches;

8. five closed conduit cross-sections and trapezoidal open channel;

9. backwater, upstream and downstream flow control, flow reversal, surcharging, and pressure flow;

10. weir, gate, and orifice control and diversion structures considering both upstream and downstream flow conditions;

11. pumping stations; and

12. overflow storage basins.

The model does not include the following features:

1. evapotranspiration;

2. snow accumulation and melt;

3. catchment moisture balance during periods of no precipitation;

4. flow routing in gutters;

5. water quality decay, reactions and interactions in the sewers and in storage;

6. real-time control;

7. design; and

8. costs.

Methods

Average values of dry-weather flow and concentrations of the six modeled water quality constituents are provided as input data

and varied to account for time and land use differences by internal functions. Five different land uses are considered.

Rainfall intensitites for several raingages can be provided to compute runoff from several catchments. Only one raingage can be assigned to a particular subcatchment. Runoff is computed separately for pervious and impervious areas and begins after all depression storage is filled. Horton's equation is used to compute the potential infiltration and the actual infiltration depends on the depth of overland flow. No adjustments are made in the potential infiltration rates to account for changing soil moisture conditions. Other losses, such as evapotranspiration, are not considered.

Overland flow is computed separately for pervious and impervious areas using the kinematic wave equation with Manning's equation. Special formulations are built into the program to eliminate the need to provide input data for each urban lot or block. Larger subcatchments can be defined and data are needed only to describe the geometry and characteristics of an average block in each subcatchment, the number of blocks per subcatchment, and the average length of each subcatchment. The formulation not only simplifies data preparation, but distinguishes between the overland flow length of lots or blocks (which influences infiltration) and the subcatchment length (which may not influence infiltration).

Flow routing in the sewers is accomplished with the dynamic wave equations for nonsteady gradually varied open channel flow (Saint-Venant equation). Manning's equation defines the energy gradient. A special explicit finite difference solution is used which is similar to the method first developed by WRE for the EPA Dynamic Estuary Model (Shubinski et al., 1965). The link-node approach solves the continuity equation at nodes (junctions or sewer connections) and the momentum equation along links (sewer reaches).

The solution technique considers both upstream and downstream flow control, backwater, and flow reversal and is linked with hydraulic equations for special sewerage system elements. This gives the model the capability to simulate flows in converging branch networks, diverging branches, and loops. The flow routing is formulated for circular, rectangular, horseshoe, basket-handle, and egg-shaped closed conduits and trapezoidal open channels.

Explicit solutions of the dynamic wave equations are simpler to program than implicit solutions but must use very short computational time steps to remain numerically stable. As a result, explicit solutions are faster per time step, but require more

time steps than implicit solutions to obtain the same level of accuracy.

Special formulations are built into the model to simulate surcharging and pressure flow. If surcharging occurs, the Hardy-Cross method is used to compute the extent of surcharging and pressure flow in the sewerage network.

Hydraulic equations are built into the model and linked with the flow routing formulation to simulate flow control and diversion structures consisting of weirs, gates, and orifices and considering both upstream and downstream flow control. Up to three-stage pumping stations can be simulated with the pumping rates specified by rule curves. Overflow storage basins can be simulated but details on their mathematical representation were not available. Inline storage basins cannot be simulated.

The stormwater quality formulations are similar to the EPA model. Stormwater runoff concentrations are computed for each modeled constituent as a function of the runoff rate and account for the rate of dirt and dust accumulation between rainstorms and for street cleaning practices. A special formulation is included to compute the pollutant contribution of catchbasins. The equations contain many empirical coefficients which have been derived from very limited data and may not be generally applicable. Many of the coefficients are internal to the program and programming changes are necessary to calibrate them with measured data.

Stormwater and dry-weather quality are combined at sewer inlets and routed through the sewers according to the flow velocity of the sewage. Dispersion, sedimentation and scour, water quality decay, reactions and interactions are not modeled. Numerical dispersion may be expected as a result of the finite difference solution of the transport equation. The model does not compute wastewater treatment.

Real-time control, design, and cost computations are not included in the model. Although a separate receiving water flow and quality model is used by WRE, it is not interfaced with the Stormwater Management Model.

Computer Program

The computer program consists of three main programs which are run sequentially. Model documentation does not describe details of the computer program. The program is available.

Program output includes tables of discharges and water quality for the modeled subcatchments, tables of stage, velocities,

discharges, concentrations and mass flow rates for selected sewer system elements, and line printer plots of stage, discharge and concentrations.

Evaluation

The Stormwater Management Model is the only model which includes a very complete formulation of sewerage system hydraulics and the simulation of wastewater quality. It can simulate all non-steady gradually varied flow phenomena, surcharging and pressure flow, and the hydraulic performance of special sewerage system facilities for complex sewerage systems including loops and converging and diverging branches. The explicit solution of the dynamic wave equation requires longer computer times than simpler routing schemes, however, and is limited to severe stability conditions with respect to the length of the computational time step.

The model would be useful primarily where the consideration of backwater, downstream flow control, flow reversal, surcharging and flow control and diversion structures are important features of the sewerage system assessment. If these features are not present or are insignificant, simpler models requiring less computer time can be used.

The model can be used only for the simulation of single runoff events since it does not include provisions for either catchment moisture or water quality accounting between rainstorms. A model limitation is the absence of formulations for treatment plants to assess the impact of sewerage system effluents on receiving water quality.

The ability to read in typical urban block data, rather than detailed data, for each modeled block is a major advantage. This, coupled with the distinction between pervious area length and catchment length, is expected to give better accuracy for coarser catchment discretizations than is provided by models which do not make this distinction.

Model verification with data from the 19 ha (47 acres) Northwood catchment in Baltimore, Maryland and the 1540 ha (3800 acres) Selby Street catchment in San Francisco shows good agreement between measured and computed runoff. Water quality predictions for the Selby Street catchment are of the right order of magnitude.

WILSEY AND HAM URBAN WATERSHED SYSTEM

Summary

The Wilsey and Ham Urban Watershed System computes the time-

varying stormwater runoff of several catchments and a converging branch sewerage network (Amorocho, 1972). The model does not include provisions for dry-weather flow, water quality, and real-time control. A design option sizes circular sewer pipes for peak flows. The model is limited to the simulation of single runoff events. This is a proprietary model and not all details of the model formulations are available.

The Wilsey and Ham Urban Watershed System includes the following features:

1. one rainfall record;
2. stormwater runoff from impervious and pervious areas of several catchments;
3. flow routing in gutters;
4. routing of stormwater runoff in a converging network of circular sewers; and
5. design of circular sewers.

The model does not include the following features:

1. dry-weather flow;
2. consideration of nonuniform distribution of catchment precipitation;
3. evapotranspiration;
4. snow accumulation and melt;
5. catchment moisture balance during periods of no precipitation;
6. flow routing in loops and diverging branches;
7. noncircular closed conduits and open channels;
8. downstream flow control, backwater, flow reversal, surcharging and pressure flow;
9. flow control and diversion structures;
10. pumping stations;
11. storage facilities;
12. water quality;

13. real-time control; and

14. costs.

Methods

The model does not consider dry-weather flow and computes only stormwater runoff from one rainfall record for the entire catchment. Losses are subtracted to account for interception and infiltration from rainfall on the impervious and pervious areas but details of the loss computations are not available. The model includes a special feature which allows the definition of typical urban subcatchment elements by appropriate data describing their physical characteristics. This greatly reduces the amount of data preparation since it eliminates repetition of data for subcatchment elements of similar characteristics.

Overland flow is routed to the gutters by an approximation of the kinematic wave equation. A kinematic wave formulation is also used for flow routing in standard shapes of gutters. Computer time is saved by performing these computations only for the basic catchment elements, rather than all catchment elements, and combining the resulting runoff hydrographs in the proper sequences as defined by their occurrence in the actual catchment.

Flow routing in a converging system of circular sewers is accomplished by the kinematic wave equation with Manning's equation. A special feature routes catchbasin inflow exceeding the free flow capacity of the sewer in the gutter to the next downstream catchbasin. In the design mode, the sewer pipe diameter is increased to accomodate the entire inflow under free flow conditions. The model does not compute backwater, flow reversal, surcharging and pressure flow. Costs are not considered in the design.

Computer Program

Details of the computer program and estimates of running times are not available. The program is written in two versions to run on a CDC 6600 computer and on Tymshare's XDS 940 system. The program is available under a special use agreement.

Computer output includes discharges at selected points of the network and pipe diameters for the design option.

Evaluation

Although sufficient details are not available on the mathematical formulations of the model, it appears to be an efficient model for the evaluation and design of small storm sewerage networks

for which the limitations of the flow routing scheme are acceptable. The program can be expected to be very efficient both with respect to input data preparation and computer running times due to its special features for the standardization of typical subcatchment elements.

The model has been applied extensively by Wilsey and Ham for storm sewerage system evaluations and design. Testing of the model with real catchment data to evaluate its accuracy has not been reported.

OTHER MODELS

Documentation was received too late for a detailed review of several models. A brief description of each model is given below, model documentation is listed in Section IX - References, and addresses of the model developers are given in Appendix A - List of Sources of Computer Programs.

Chicago Runoff and Pollution Model

The Chicago Runoff and Pollution Model was developed by the City of Chicago Department of Public Works for the continuous simulation of the time-varying runoff and water quality in combined sewerage systems consisting of several catchments and a converging branch sewer and open channel network (City of Chicago, 1972). The model can simulate periods of several years using 1-hour time steps. The model can simulate surcharging and diversion structures. Water quality simulations include the computation of biochemical oxygen demand and suspended solids as functions of storm runoff, the consideration of dry-weather flow and quality (provided as input data), the computation of water quality improvement by treatment (as tabular function of flow), and catchment water quality balance between rainstorms. The model does not include real-time control, design, and cost computations. The rainfall-runoff and flow routing portions of the model use methods included in the computer version of the Chicago Hydrograph method. A separate model is available for nonsteady receiving water flow and quality simulation.

The computer program is written in Fortran IV for IBM 1130 and IBM 370/158 computers. The program and a draft user's manual is available. The program has been tested and applied.

CH2M-Hill Wastewater Collection System Analysis Model

The CH2M-Hill Wastewater Collection System Analysis Model simulates the time-varying runoff in a combined sewerage system consisting of several catchments and a converging branch

sewer and open channel network with special structures such as diversions and pumping stations (O'Neel et al., 1974). The model is based on formulations of the U.S. Environmental Protection Agency Stormwater Management Model, but was adapted to run on small computers. Present versions operate on an IBM 1130 and a DEC PDP-10. Water quality and receiving water simulation is not included at present. The model is being tested with data from the City of Portland, Oregon.

Dorsch Consult Quantity-Quality Simulation Program

The Dorsch Consult Quantity-Quality Simulation Program is intended for single event or continuous simulation of the time-varying runoff and water quality in combined sewerage systems consisting of several catchments and a closed conduit and open channel network including loops and converging and diverging branches (Geiger, 1975). Both runoff and water quality concentrations from catchment areas are calculated by a unit hydrograph method, considering different land uses including residential, commercial, industrial, and mixed. The flow routing through the sewerage network is based on the dynamic wave equations. Statistical analyses are incorporated to provide monthly and annual flow and pollutant duration curves for any node in the network. Four conservative water quality constituents can be routed.

Catchment runoff quality formulations are currently for biochemical oxygen demand and settleable solids. Formulations are also planned for carbonaceous oxygen demand, suspended solids, coliform bacteria, chloride, and nutrients. The model does not include real-time control, design and cost computations.

The computer program is written in Fortran IV for a UNIVAC 1108 computer and accepts both English and metric units of measurements. The program has been tested and applied. This is a proprietary program of Dorsch Consult Ingenieurgesellschaft mbH of Munich, Germany. A North American office is maintained in Toronto, Canada.

A more detailed description provided by Dorsch Consult is included in Appendix E - Additional Model Test Results. It also includes water quality routing results for the hypothetical pipe data described in Section VI - Test Data.

Illinois State Water Survey Urban Drainage Area Simulator

The Illinois State Water Survey Urban Drainage Area Simulator was developed by the Illinois State Water Survey for single-event simulation of time-varying runoff in combined sewerage

systems consisting of several catchments and a converging sewer and open channel network (Terstriep and Stall, 1974). The model is based on a computer version of the British Road Research Laboratory Model, but computes also the runoff from pervious areas and includes the option to either size circular sewers or retention basins. The design option is based on hydraulic considerations alone and does not consider costs. The model does not include real-time control and water quality computations.

The computer program is written in Fortran IV for an IBM 360/75 computer. It has been tested extensively. The program and a user's manual are available.

Norwegian Institute for Water Research Sewerage System Models

The Norwegian Institute for Water Research developed four computer models for the evaluation of flows and water quality in combined sewerage systems: a sewer network, treatment plant, sludge treatment, and economic model (Lindholm, 1972 and 1974).

The sewer network model computes the single-event or continuous (to one year) runoff and biochemical oxygen demand from several catchments and routes them through a converging branch sewerage system with diversions, pumping stations and storage facilities The program computes also the sizes of sewers required to prevent surcharging, and the cost of the entire sewerage network.

The sewage treatment plant model computes the removal of BOD and phosphorus for five unit processes, which can be arranged in different configurations, using empirical relationships. The model is run in conjunction with the sewer network model to determine optimal combinations of storage and treatment using a trial-and-error procedure.

Capital and operating costs are computed in the sewer network and treatment plant model and the separate cost-analysis model. Detailed descriptions of the sludge treatment and cost-analysis model and information on model testing with measured data were not available. The computer programs are written in Fortran IV for a UNIVAC 1108 computer.

Queen's University Urban Runoff Model

The Queen's University Urban Runoff Model simulates the time-varying runoff in a combined sewerage system consisting of several catchments and a converging sewerage network (Watt, 1975). Runoff is computed from rainfall with the unit-hydrograph method, and routed through the sewers with a simple time-offset method using Manning's equation. Special sewerage

system structures and water quality are not simulated. The model does not include real-time control, design, and cost features. The program has been tested with real catchment data.

University of Nebraska Urban Hydrologic Simulator

The University of Nebraska Urban Hydrologic Simulator computes the time-varying runoff from several catchments and routes them through a converging closed conduit network (Surkan, 1973; and Surkan and Kelton, 1974). The model assumes that storm runoff is a constant fraction of the rainfall, and that pipe flow velocities are a linear function of distance only. Special sewerage network structures, water quality, real-time control, design, and cost features are not included. The model has been tested with real catchment data. The program is written in Fortran IV. The program and user's instructions are available.

SECTION VI

TEST DATA

CONTENTS

	Page
Introduction.	131
Hypothetical Data	132
Hypothetical Catchment Data.	133
Hypothetical Pipe Data	145
Real Catchment Data	164
Description of the Oakdale Avenue Catchment.	164
Monitoring System of the Oakdale Avenue Catchment.	169
Rainfall and Runoff Data of the Oakdale Avenue Catchment	169
Description of the Bloody Run Catchment.	194
Monitoring System of the Bloody Run Catchment.	201
Rainfall and Runoff Data of the Bloody Run Catchment.	201

INTRODUCTION

For purposes of comparing model performance, numerical testing of the seven most comprehensive models was conducted in this study using both hypothetical and real catchment data. The purpose of the testing was to show that different models with different mathematical formulations for the same phenomena will produce differing results given the same input data. The testing consequently provides useful information to the model user by indicating ranges of applicability for each model. Most of the models reviewed were tested and verified by the model developers and subsequent model users on very limited data because sufficiently complete and reliable urban catchment, precipitation, runoff, and water quality data were unavailable. Quite commonly, only portions of a model were tested on measured data. Sometimes different portions of a model were tested with data from different catchments since comprehensive data were not available from a single catchment, especially when both flow and quality data were required, and only in rare instances were two or more models tested on the same data. Testing with the hypothetical data shows the sensitivity of each model to model parameter variations, while testing with real catchment data provides information on model accuracy as well.

The reviewed models require widely varying detail with respect to the spatial discretization of data describing the catchment and sewerage system. Some models require extremely detailed information (such as sizes and slopes of individual roofs, driveways, lawns, gutters, etc.) while others can lump areas of several hundred acres into single subcatchments. Great differences exist also between the models with respect to the required time discretization for precipitation, runoff and water quality data and simulation time steps. Some models require time steps of less than one minute to satisfy numerical stability conditions, while others can be run with hourly or daily data.

The model testing with real catchment data required the collection of available data on urban catchment and sewer system characteristics, precipitation, runoff, and wastewater quality. A large number of data sources were investigated and data for several U.S. cities were collected. Selected data were digitized in the computer formats required by the models being tested. It became rather difficult, however, to find real catchment data which provided sufficient detail to satisfy the input data requirements of all models. Additional difficulties which were encountered included missing information on physical characteristics of the subcatchments and sewerage systems, uncertainties in watershed infiltration characteristics and

moisture conditions, insufficient documentation of the instrumentation and data analysis techniques used during the measurement periods, and uncertainties in the accuracy of the measured rainfall, runoff, and water quality data.

Great reluctance was expressed by model developers, therefore, to run real catchment data whose accuracy was uncertain and over whose collection they had no control. The use of hypothetical data eliminated these uncertainties. Consequently, the model testing concentrated on hypothetical data. All seven models were tested with the same set of hypothetical data but only the Battelle, Chicago Flow Simulation, EPA Stormwater Management and Dorsch models were also tested on the same set of real catchment data.

The numerical testing concentrated on the most common features of the selected models, the rainfall-runoff simulations and the flow and quality routing. Special features such as snowmelt, the simulation of treatment processes, real-time control and design aspects were not tested. The hypothetical data testing was applied separately to the rainfall-runoff formulations and the routing procedures to examine the response of the formulations of each phenomenon to physical parameter variations. Testing with the real catchment data served the objectives of testing the interactions of the runoff computations for several catchments and the routing of flows and quality through a sewer network.

This section lists the selected test data. The results of testing seven models by computer runs with these data are described in the following section.

HYPOTHETICAL DATA

The hypothetical data runs were not intended to portray accuracy but to determine the sensitivity of the models to parameter variations. The data were also selected with the objective of determining limits of applicability of the models, such as the ability to handle surcharging and backwater conditions. In addition, the hypothetical data runs provided information on the cost of running each model.

Two independent sets of hypothetical data were used. One set tests the rainfall-runoff relationship for various catchment conditions. The second set tests the sewer flow routing (and water quality routing, if applicable) for various combinations of pipe parameters.

Hypothetical Catchment Data

The data selected for the hypothetical catchment tests represent two different catchment sizes, two orientations of each catchment, ten combinations of physical catchment characteristics, two different antecedent conditions and four different rainstorms. This makes a total of 320 data combinations.

The selected catchment dimensions are given in Table 6. The drainage areas of 0.465 and 4.65 ha (1.115 and 11.5 acres) represent a range of common sizes generally used by the tested models to simulate urban subcatchments. Although some models use smaller discretizations and others can simulate several hundred acres as a single subcatchment, these sizes are representative of most model requirements. Rectangular shapes were selected since most models require this approximation for the overland flow routing on irregularly shaped catchments.

Length-to-width ratios of 1:2 and 2:1, respectively, were selected for the catchments. These represented two orientations of the flow (as shown in Figure 1) to test the effect of the catchment shape approximation on the shape of the runoff hydrograph. The dimensions were 48.2 x 96.4 m (158 x 316 ft) for the small catchments and 152.4 x 304.8 m (500 x 1000 ft) for the large catchments.

The physical catchment characteristics selected are listed in Table 7. Five different ratios of pervious and impervious areas were chosen, ranging from completely pervious to completely impervious (0, 25, 50, 75, and 100 percent imperviousness), to test the effect of correctly estimating this catchment property on the runoff hydrographs and to test the relative importance of the contribution of runoff from pervious and

Table 6. HYPOTHETICAL SUBCATCHMENT DIMENSIONS

Shape number	Length		Width		Area	
	m	ft	m	ft	ha	acres
1	96.4	316	48.2	158	0.465	1.15
2	48.2	158	96.4	316	0.465	1.15
3	304.8	1000	152.4	500	4.645	11.48
4	152.4	500	304.8	1000	4.645	11.48

Note: Length is parallel, width is perpendicular to direction of overland flow.

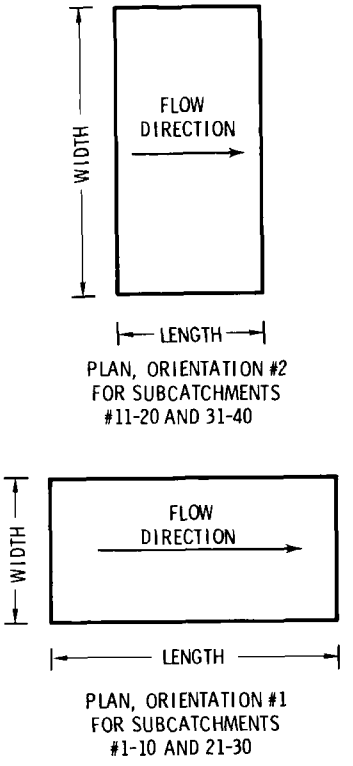


Figure 1. Sketch of hypothetical subcatchment orientations

Table 7. HYPOTHETICAL SUBCATCHMENT CHARACTERISTICS

Characteristics	Values
Subcatchment imperviousness, percent	0, 25, 50, 75 and 100
Subcatchment slope (in direction of flow), percent	0.1 and 10
Manning's n of pervious area	0.25
Manning's n of impervious area	0.025
Infiltration decay rate of pervious area, $seconds^{-1}$	0.001

Note: The infiltration decay rate is defined by Horton's infiltration equation.

impervious areas to the total catchment runoff. Catchment slopes of 0.1 and 10 percent, representing the common range of catchment slopes, were chosen to test the effect of overland flow routing formulations on the runoff hydrograph. Average values of 0.025 and 0.25 for Manning's n were used for the impervious and pervious areas, respectively. The influence of variations in catchment surface roughness was not tested.

Two initial conditions, representing initially dry and wet catchment moisture conditions, were tested. Infiltration rates and surface retention (depression) storage values are given in Tables 7 and 8. The values correspond to data requirements for Horton's infiltration equation, which is used by most models, and the definition of surface water storage in depressions which

Table 8. INFILTRATION RATES AND RETENTION STORAGE CAPACITIES OF HYPOTHETICAL CATCHMENTS

	Dry initial condition		Saturated initial condition	
Units	mm/hr	in./hr	mm/hr	in./hr
Maximum infiltration rate of pervious areas	50.8	2.0	12.7	0.5
Minimum infiltration rate of pervious areas	12.7	0.5	12.7	0.5
Maximum infiltration rate of impervious areas	0.0	0.0	0.0	0.0
Minimum infiltration rate of impervious areas	0.0	0.0	0.0	0.0
Units	mm	in.	mm	in.
Retention storage capacity of pervious areas	5.1	0.20	0.0	0.0
Retention storage capacity of impervious areas	1.3	0.05	0.0	0.0

Notes: The infiltration rates are defined by Horton's infiltration equation. The retention storage capacities represent the amount of rain which has to fall before runoff begins.

does not become surface runoff. The selected potential infiltration rates for pervious areas are 50.8 mm/hr (2.0 in./hr) for dry conditions and 12.7 mm/hr (0.5 in./hr) for saturated conditions. An infiltration decay rate of 0.001 sec^{-1} was chosen. Zero infiltration was assumed for impervious areas. The retention storage capacity is 5.1 mm (0.20 in.) for pervious and 1.3 mm (0.05 in.) for impervious areas under dry conditions.

A difficulty arises in defining the retention storage capacity for saturated conditions as a result of model limitations. Ideally, the models should define the potential storage capacity independent of moisture conditions, but allow input data defining the depth of moisture in the depressions at the beginning of the simulation. Most models, however, do not make this distinction and a zero retention storage capacity has to be defined for initially saturated conditions. This, however, introduces errors into the infiltration and overland flow computations in those models which consider the actual depth of water in the depressions for these computations.

The variations in slope, imperviousness and initial conditions result in 20 subcatchments for each size and orientation. The data combinations for each set of 20 subcatchments are given in Table 9. Each subcatchment was assumed to drain into an independent manhole where the input hydrograph of each subcatchment was obtained. Gutter routing was not considered. Figure 2 shows two arrangements of the sets of 20 subcatchments for simulation purposes. Deviations from this arrangement were required, however, to meet specific model requirements and limitations.

Four hypothetical rainstorms were used to test the response of the selected models to different durations and intensities of rainfall. One rainstorm represented a constant rainfall of 25.4 mm/hr (1.0 in./hr) intensity and 2 hr duration; the remaining three were triangular rainstorms of 1, 2, and 4 hr duration and 101.6, 50.8, and 25.4 mm/hr (4, 2, and 1 in./hr) peak intensity, respectively. The four rainstorms had a total rainfall of 50.8 mm (2 in.). A summary of the rainfall data is presented in Table 10 and the rainfall intensity and cumulative rainfall hyetographs are shown in Figures 3 and 4. A time step of 5 minutes was suggested for the simulation runs and both intensities and cumulative values are tabulated in Tables 11 to 14 in response to different input requirements by different models. The effect of different time discretizations was not tested.

Table 9. HYPOTHETICAL CATCHMENT DATA COMBINATIONS

Subcatchment number	Length		Width		Area		Slope, percent	Imperviousness, percent
	m	ft	m	ft	ha	acres		
1	152.4	500	304.8	1000	4.645	11.48	0.1	0
2	152.4	500	304.8	1000	4.645	11.48	0.1	25
3	152.4	500	304.8	1000	4.645	11.48	0.1	50
4	152.4	500	304.8	1000	4.645	11.48	0.1	75
5	152.4	500	304.8	1000	4.645	11.48	0.1	100
6	152.4	500	304.8	1000	4.645	11.48	10.0	0
7	152.4	500	304.8	1000	4.645	11.48	10.0	25
8	152.4	500	304.8	1000	4.645	11.48	10.0	50
9	152.4	500	304.8	1000	4.645	11.48	10.0	75
10	152.4	500	304.8	1000	4.645	11.48	10.0	100
11	304.8	1000	152.4	500	4.645	11.48	0.1	0
12	304.8	1000	152.4	500	4.645	11.48	0.1	25
13	304.8	1000	152.4	500	4.645	11.48	0.1	50
14	304.8	1000	152.4	500	4.645	11.48	0.1	75
15	304.8	1000	152.4	500	4.645	11.48	0.1	100
16	304.8	1000	152.4	500	4.645	11.48	10.0	0
17	304.8	1000	152.4	500	4.645	11.48	10.0	25
18	304.8	1000	152.4	500	4.645	11.48	10.0	50
19	304.8	1000	152.4	500	4.645	11.48	10.0	75
20	304.8	1000	152.4	500	4.645	11.48	10.0	100

Table 9 (Continued). HYPOTHETICAL CATCHMENT DATA COMBINATIONS

Subcatchment number	Length		Width		Area		Slope, percent	Imperviousness, percent
	m	ft	m	ft	ha	acres		
21	48.2	158	96.4	316	0.465	1.15	0.1	0
22	48.2	158	96.4	316	0.465	1.15	0.1	25
23	48.2	158	96.4	316	0.465	1.15	0.1	50
24	48.2	158	96.4	316	0.465	1.15	0.1	75
25	48.2	158	96.4	316	0.465	1.15	0.1	100
26	48.2	158	96.4	316	0.465	1.15	10.0	0
27	48.2	158	96.4	316	0.465	1.15	10.0	25
28	48.2	158	96.4	316	0.465	1.15	10.0	50
29	48.2	158	96.4	316	0.465	1.15	10.0	75
30	48.2	158	96.4	316	0.465	1.15	10.0	100
31	96.4	316	48.2	158	0.465	1.15	0.1	0
32	96.4	316	48.2	158	0.465	1.15	0.1	25
33	96.4	316	48.2	158	0.465	1.15	0.1	50
34	96.4	316	48.2	158	0.465	1.15	0.1	75
35	96.4	316	48.2	158	0.465	1.15	0.1	100
36	96.4	316	48.2	158	0.465	1.15	10.0	0
37	96.4	316	48.2	158	0.465	1.15	10.0	25
38	96.4	316	48.2	158	0.465	1.15	10.0	50
39	96.4	316	48.2	158	0.465	1.15	10.0	75
40	96.4	316	48.2	158	0.465	1.15	10.0	100

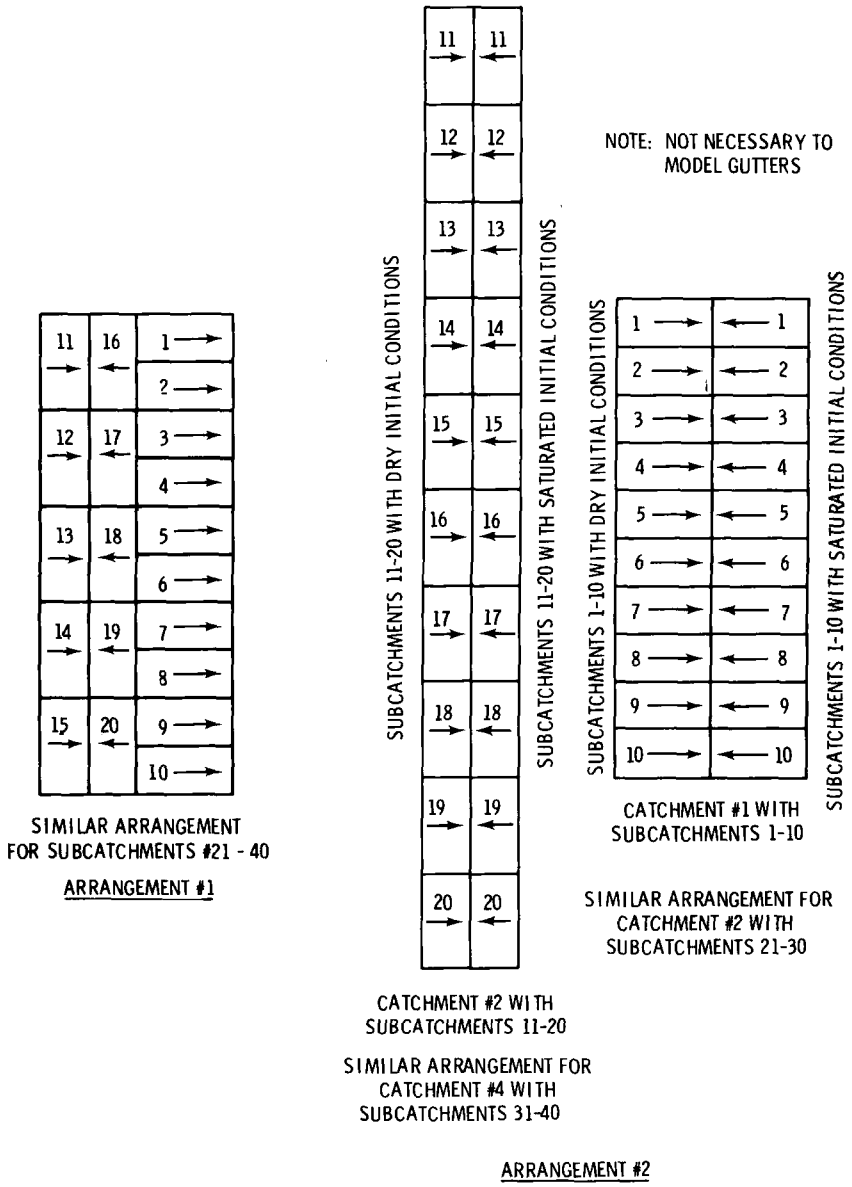


Figure 2. Suggested arrangements of hypothetical subcatchments for computer runs

Table 10. SUMMARY OF RAINSTORMS FOR HYPOTHETICAL CATCHMENTS

Storm number	Shape	Rainfall duration, hours	Maximum intensity		Total rainfall	
			mm/hr	in./hr	mm	in.
1	Constant	2	25.4	1.0	50.8	2.0
2	Triangle	1	101.6	4.0	50.8	2.0
3	Triangle	2	50.8	2.0	50.8	2.0
4	Triangle	3	25.4	1.0	50.8	2.0

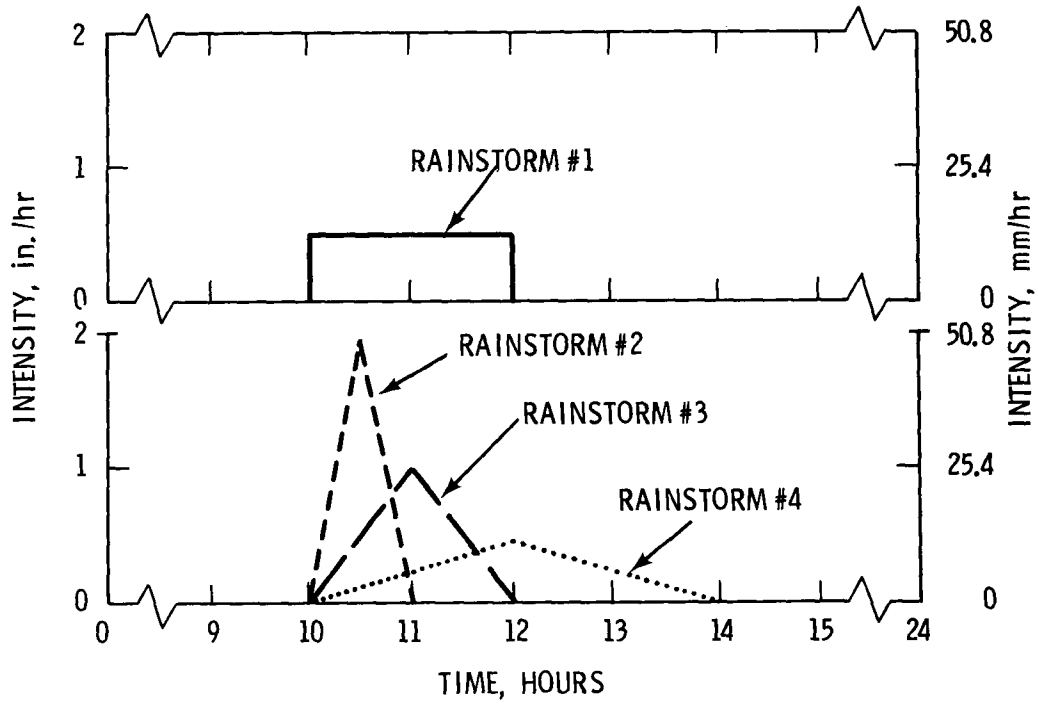


Figure 3. Hypothetical catchment hyetographs (rain intensities)

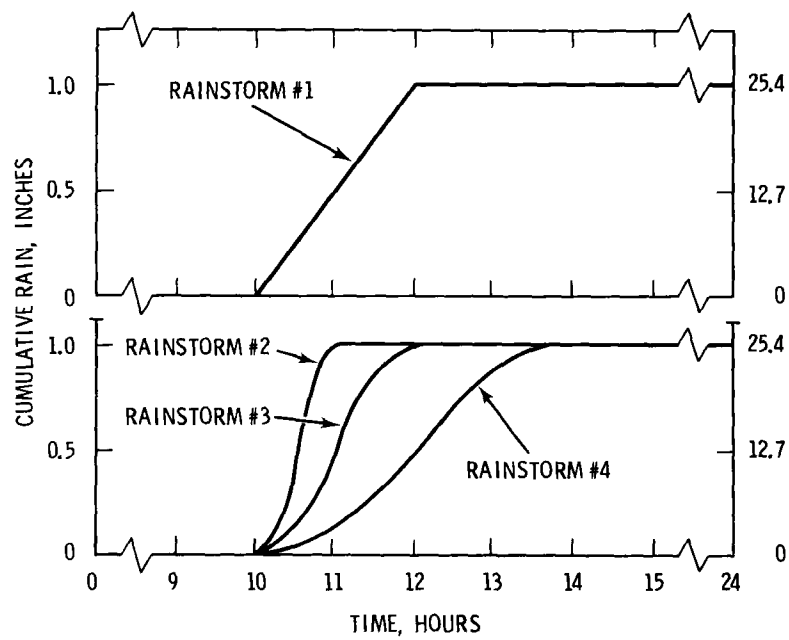


Figure 4. Hypothetical catchment hyetographs (cumulative rain)

Table 11. TWO-HOUR CONSTANT RAINSTORM FOR HYPOTHETICAL CATCHMENTS

Time step	Clock time hr:min	Rain intensities mm/hr	Rain intensities in./hr	Cumulative rain mm	Cumulative rain in.
1	10:00	0.0	0.00	0.0	0.00
2	10:05	25.4	1.00	2.1	0.08
3	10:10	25.4	1.00	4.2	0.17
4	10:15	25.4	1.00	6.4	0.25
5	10:20	25.4	1.00	8.5	0.33
6	10:25	25.4	1.00	10.6	0.42
7	10:30	25.4	1.00	12.7	0.50
8	10:35	25.4	1.00	14.8	0.58
9	10:40	25.4	1.00	16.9	0.67
10	10:45	25.4	1.00	19.1	0.75
11	10:50	25.4	1.00	21.2	0.83
12	10:55	25.4	1.00	23.3	0.92
13	11:00	25.4	1.00	25.4	1.00
14	11:05	25.4	1.00	27.5	1.08
15	11:10	25.4	1.00	29.6	1.17
16	11:15	25.4	1.00	31.8	1.25
17	11:20	25.4	1.00	33.9	1.33
18	11:25	25.4	1.00	36.0	1.42
19	11:30	25.4	1.00	38.1	1.50
20	11:35	25.4	1.00	40.2	1.58
21	11:40	25.4	1.00	42.3	1.67
22	11:45	25.4	1.00	44.5	1.75
23	11:50	25.4	1.00	46.6	1.83
24	11:55	25.4	1.00	48.7	1.92
25	12:00	25.4	1.00	50.8	2.00
26	12:05	0.0	0.00	50.8	2.00

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 12. ONE-HOUR TRIANGULAR RAINSTORM FOR HYPOTHETICAL CATCHMENTS

Time step	Clock time hr:min	Rain intensities mm/hr	Rain intensities in./hr	Cumulative rain mm	Cumulative rain in.
1	10:00	0.0	0.00	0.0	0.00
2	10:05	8.5	0.33	0.7	0.03
3	10:10	25.4	1.00	2.8	0.11
4	10:15	42.3	1.67	6.4	0.25
5	10:20	59.3	2.33	11.3	0.45
6	10:25	76.2	3.00	17.6	0.70
7	10:30	93.1	3.67	25.4	1.00
8	10:35	93.1	3.67	33.2	1.31
9	10:40	76.2	3.00	39.5	1.56
10	10:45	59.3	2.33	44.5	1.75
11	10:50	42.3	1.67	48.0	1.89
12	10:55	25.4	1.00	50.1	1.97
13	11:00	8.5	0.33	50.8	2.00
14	11:05	0.0	0.00	50.8	2.00

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 13. TWO-HOUR TRIANGULAR RAINSTORM FOR HYPOTHETICAL CATCHMENTS

Time step	Clock time hr:min	Rain intensities mm/hr	Rain intensities in./hr	Cumulative rain mm	Cumulative rain in.
1	10:00	0.0	0.00	0.0	0.00
2	10:05	2.1	0.08	0.2	0.01
3	10:10	6.4	0.25	0.7	0.03
4	10:15	10.6	0.42	1.6	0.06
5	10:20	14.8	0.58	2.8	0.11
6	10:25	19.1	0.75	4.4	0.17
7	10:30	23.3	0.92	6.4	0.25
8	10:35	27.5	1.08	8.6	0.34
9	10:40	31.7	1.25	11.3	0.44
10	10:45	36.0	1.42	14.3	0.56
11	10:50	40.2	1.58	17.6	0.69
12	10:55	44.4	1.75	21.3	0.84
13	11:00	48.7	1.92	25.4	1.00
14	11:05	48.7	1.92	29.5	1.16
15	11:10	44.4	1.75	33.2	1.31
16	11:15	40.2	1.58	36.5	1.44
17	11:20	36.0	1.42	39.5	1.56
18	11:25	31.7	1.25	42.2	1.66
19	11:30	27.5	1.08	44.5	1.75
20	11:35	23.3	0.92	46.4	1.83
21	11:40	19.1	0.75	48.0	1.89
22	11:45	14.8	0.58	49.2	1.94
23	11:50	10.6	0.42	50.1	1.97
24	11:55	6.4	0.25	50.6	1.99
25	12:00	2.1	0.08	50.8	2.00
26	12:05	0.0	0.00	50.8	2.00

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 14. FOUR-HOUR TRIANGULAR RAINSTORM FOR HYPOTHETICAL CATCHMENTS

Time step	Clock time hr:min	Rain intensities mm/hr	Rain intensities in./hr	Cumulative rain mm	Cumulative rain in.
1	10:00	0.0	0.00	0.0	0.00
2	10:05	0.5	0.02	0.0	0.00
3	10:10	1.6	0.06	0.2	0.01
4	10:15	2.6	0.10	0.4	0.02
5	10:20	3.7	0.15	0.7	0.03
6	10:25	4.8	0.19	1.1	0.04
7	10:30	5.8	0.23	1.6	0.06
8	10:35	6.9	0.27	2.2	0.09
9	10:40	7.9	0.31	2.8	0.11
10	10:45	9.0	0.35	3.6	0.14
11	10:50	10.1	0.40	4.4	0.17
12	10:55	11.1	0.44	5.3	0.21
13	11:00	12.2	0.48	6.4	0.25
14	11:05	13.2	0.52	7.5	0.29
15	11:10	14.3	0.56	8.6	0.34
16	11:15	15.3	0.60	9.9	0.39
17	11:20	16.4	0.65	11.3	0.44
18	11:25	17.5	0.69	12.7	0.50
19	11:30	18.5	0.73	14.3	0.56
20	11:35	19.6	0.77	15.9	0.63
21	11:40	20.6	0.81	17.6	0.70
22	11:45	21.7	0.85	19.4	0.77
23	11:50	22.8	0.90	21.3	0.84
24	11:55	23.8	0.94	23.3	0.92
25	12:00	24.9	0.98	25.4	1.00
26	12:05	24.9	0.98	27.5	1.08
27	12:10	23.8	0.94	29.5	1.16
28	12:15	22.8	0.90	31.4	1.23
29	12:20	21.7	0.85	33.2	1.31
30	12:25	20.6	0.81	34.9	1.37
31	12:30	19.6	0.77	36.5	1.44
32	12:35	18.5	0.73	38.1	1.50
33	12:40	17.5	0.69	39.5	1.56
34	12:45	16.4	0.65	40.9	1.61
35	12:50	15.3	0.60	42.2	1.66
36	12:55	14.3	0.56	43.3	1.71
37	13:00	13.2	0.52	44.5	1.75
38	13:05	12.2	0.48	45.5	1.79
39	13:10	11.1	0.44	46.4	1.83
40	13:15	10.2	0.40	47.2	1.86
41	13:20	9.0	0.35	48.0	1.89
42	13:25	7.9	0.31	48.6	1.91
43	13:30	6.9	0.27	49.2	1.94
44	13:35	5.8	0.23	49.7	1.96
45	13:40	4.8	0.19	50.1	1.97
46	13.45	3.7	0.15	50.4	1.98
47	13:50	2.6	0.11	50.6	1.99
48	13:55	1.6	0.06	50.8	2.00
49	14:00	0.5	0.02	50.8	2.00
50	14:05	0.0	0.00	50.8	2.00

Note: Intensities represent averages for time interval preceding indicated clock time.

Hypothetical Pipe Data

The data selected for the hypothetical pipe tests represent two different pipe diameters and three invert slopes. In addition, two types of upstream and downstream boundary conditions were specified. The first assumes free inflow into the upstream end of the pipes and free outfall at the downstream end. The second assumes a storage tank at the upstream end of the pipe and a diversion structure at the downstream end. Four inflow hydrographs, each with three inflow quality constituents, were specified for the flow and water quality routing. This makes a total of 48 data combinations.

The hypothetical pipe dimensions are given in Table 15. The pipe diameters of 0.61 and 3.66 m (2 and 12 ft) represent circular pipe sizes normally encountered in sewerage networks which may be analyzed by dynamic mathematical models. Three slopes (0.05, 0.5 and 5 percent) were selected to represent average and extreme values. Only one value of surface roughness represented by a Manning's n of 0.01 was tested. The pipe is 3048 m (10,000 ft) long to provide sufficient length to adequately show the effect of different routing schemes on the hydrograph shapes.

The selected pipe boundary conditions are summarized in Table 16 and a schematic for the second boundary condition (upstream storage tank and downstream regulator) is shown in Figure 5. The axis of the pipe is at the elevation of the bottom of the storage tank; that is, the bottom half of the pipe continues through the storage tank. The diameter of the storage tank is twice that of the pipe. The depth of the storage tank is 3.35 m (11 ft) for the small pipe and 7.92 m (26 ft) for the large pipe. The top of the tank is assumed to be at ground level.

The diversion structure represents a typical overflow regulator. It consists of an overflow weir with a height equal to the pipe radius and a dry-weather orifice outlet with a diameter equal to the pipe radius. The elevation of the orifice invert is at

Table 15. HYPOTHETICAL PIPE DIMENSIONS

Characteristic	Dimensions
Diameter	0.61 and 3.66 m (2 and 12 ft)
Length	3048 m (10,000 ft)
Slope	0.05, 0.5 and 5 percent
Manning's n	0.01

Table 16. HYPOTHETICAL PIPE BOUNDARY CONDITIONS

Boundary Condition #1

Upstream boundary: free inflow without time delay or energy loss.

Downstream boundary: free outfall.

Boundary Condition #2

Upstream boundary: open circular storage tank with the following characteristics:

Tank diameter	=	twice pipe diameter
Tank height	=	3.35 m for 0.61 m pipe (11 ft for 2 ft pipe)
Tank height	=	7.92 m for 3.66 m pipe (26 ft for 12 ft pipe)
Elevation of tank bottom	=	elevation of pipe axis (bottom half of pipe continues through storage tank)
Elevation of tank top	=	ground elevation

Assume orifice discharge from tank into pipe according to orifice equation (or equivalent):

$$Q = C A (2g H)^{1/2} \qquad (1)$$

where

Q = orifice discharge in m^3/sec or cfs

C = 0.60 = orifice discharge coefficient (non-dimensional)

A = orifice flow area (cross sectional area of flow through orifice) in m^2 or ft^2

g = acceleration of gravity = 9.81 m/sec^2 (32.2 ft/sec^2)

H = depth of water in tank in m or ft above centroid of orifice flow area

Table 16 (Continued). HYPOTHETICAL PIPE BOUNDARY CONDITIONS

Downstream boundary: regulator consisting of weir and circular orifice immediately upstream of weir, both with free outfall with the following characteristics:

Weir height = 1/2 pipe diameter
Orifice diameter = 1/2 pipe diameter
Elevation of
orifice invert = elevation of pipe invert

Assume orifice discharge according to orifice equation (or equivalent) as defined by equation (1) above.

Assume weir discharge according to weir equation (or equivalent):

$$Q = C A H^{1/2} \tag{2}$$

where

Q = weir discharge in m^3/sec or cfs

C = weir discharge coefficient = 1.66 $m^{1/2}$/sec (3.00 $ft^{1/2}$/sec)

A = cross-sectional area of flow over weir in m^2 or ft^2

H = depth of flow above weir in m or ft

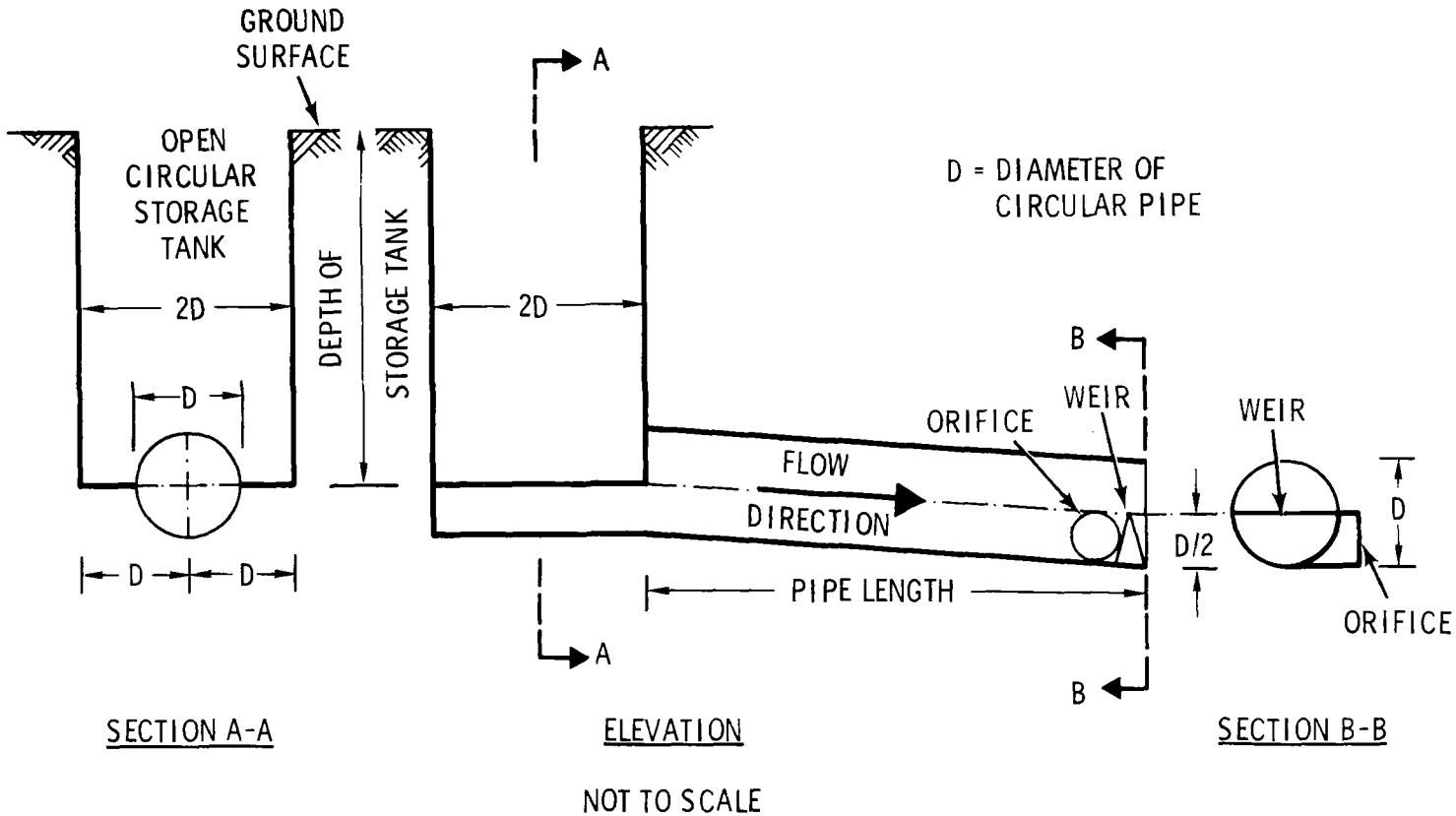


Figure 5. Sketch of hypothetical pipe for boundary condition #2 (sketch shows upstream storage tank and downstream regulator as specified for boundary condition #2; free inflow into and outflow from pipe is assumed for boundary condition #1)

the elevation of the pipe invert. The orifice is located directly upstream of the weir. Free outfall is assumed for the flow passing the orifice and weir. Provided the tested models included the computation of diversions and the appropriate hydraulic equations, the orifice discharge was defined by the common orifice discharge equation (1) and the weir discharge by the common weir equation (2) as given in Table 16. The orifice discharge coefficient was assumed to be 0.60 and the weir discharge coefficient 1.66 $m^{1/2}$/sec (3.00 $ft^{1/2}$/sec). The flow area in equation (2) is defined as the actual cross-sectional area of flow above the weir crest given by the flow depth and the circular shape of the pipe. Most models, however, approximate the flow area by a rectangle of width equal to the width of the weir crest.

The dimensions of the storage tanks and diversion structures were selected in a manner which would produce complete filling of the tank and overflow at the top of the tank for certain inflow hydrographs. If overflow occurred, it was assumed that the excess flow was lost and did not return to the pipe after recession of the water level below the tank top.

The variations in pipe diameter, slope, and boundary conditions result in 12 different pipe combinations which were arranged for simulation purposes as shown in Figure 6. Deviations from this arrangement were required, however, to meet specific model requirements and limitations. A summary of hypothetical pipe capacities for each combination, including maximum, full, 90 percent and 9 percent of full pipe flow, is given in Table 17. The discharge values were computed with Manning's equation. The data combinations for each pipe are summarized in Table 18.

Four inflow hydrograph shapes were selected to test the various model routing schemes. They represent a continuous constant inflow equal to 90 percent of full pipe flow and three triangular inflow hydrographs of 1, 2, and 4 hr durations. The triangular inflow hydrographs have a peak flow of 90 percent of full pipe flow and a constant flow before and after the triangular inflow equal to 10 percent of peak flow (9 percent of full flow). These inflow hydrographs are summarized in Table 19 and the hydrograph shapes are shown in Figure 7. The beginning time of the triangular inflow hydrographs was set at 10:00 a.m. for models which require the input of clock times.

Three conservative inflow water quality constituents were specified for each inflow hydrograph to test the water quality routing schemes of the models. The testing of the water quality routing assumed pure convection. Hydrodynamic dispersion, decay, reactions and interactions were not tested. The inflow qualities were assumed to represent a continuous constant concentration of 100 mg/l, a triangular shape with a peak concentration of 100 mg/l and zero concentration before and after

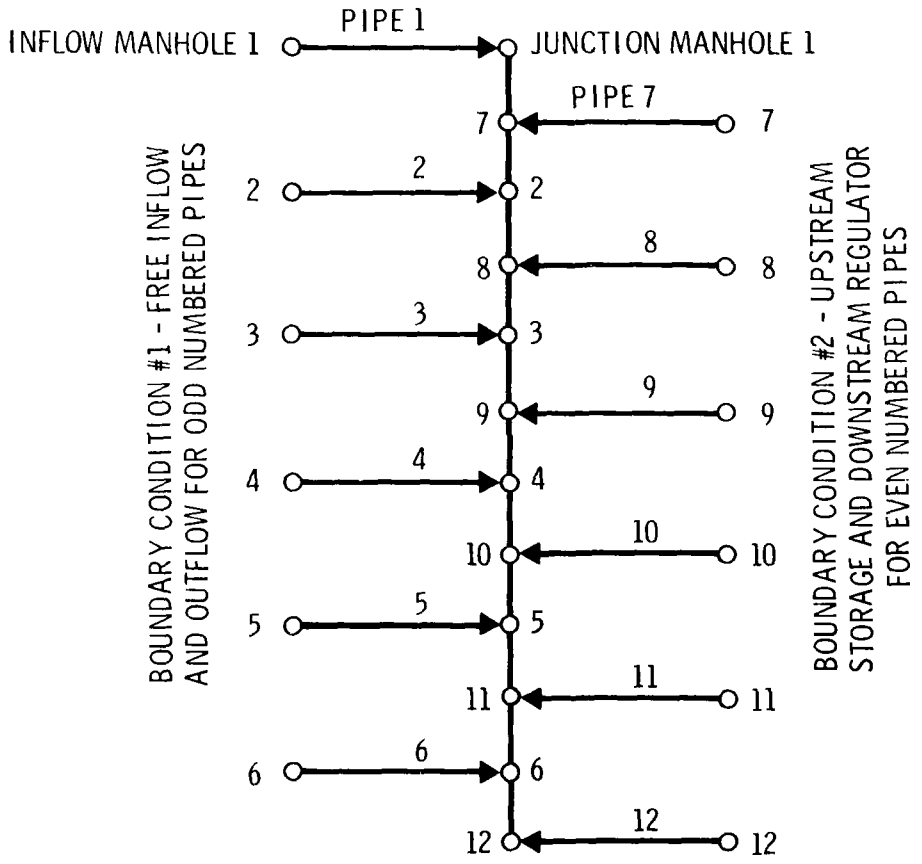


Figure 6. Suggested arrangement of hypothetical pipes for computer runs (it may not be necessary to model connecting pipes)

Table 17. HYPOTHETICAL PIPE CAPACITIES

Pipe number	Diameter m	Diameter ft	Slope, percent	Discharge: Maximum m^3/sec	Discharge: Maximum cfs	Discharge: Full m^3/sec	Discharge: Full cfs	Discharge: Q_1 = 90% of full m^3/sec	Discharge: Q_1 = 90% of full cfs	Discharge: Q_2 = 9% of full m^3/sec	Discharge: Q_2 = 9% of full cfs
1,2	0.81	2	0.05	0.202	7.12	0.187	6.59	0.168	5.94	0.017	0.59
3,4	0.81	2	0.50	0.638	22.52	0.590	20.85	0.532	18.77	0.053	1.88
5,6	0.81	2	5.00	2.017	71.21	1.867	65.94	1.681	59.35	0.168	5.94
7,8	4.86	12	0.05	23.973	846.50	22.197	783.80	19.977	705.41	1.998	70.54
9,10	4.86	12	0.50	75.477	2665.15	70.194	2478.59	63.174	2230.73	6.317	223.07
11,12	4.86	12	5.00	239.730	8465.03	221.972	7837.99	199.775	7054.19	19.978	705.42

Notes: Odd numbered pipes for boundary condition #1, even numbered pipes for boundary condition #2. Discharges computed with Manning's equation using n = 0.01.

Table 18. HYPOTHETICAL PIPE DATA COMBINATIONS

Pipe number	Diameter		Slope, percent	Upstream storage and downstream regulator	Maximum inflow		Minimum inflow	
	m	ft			m^3/sec	cfs	m^3/sec	cfs
1	0.81	2	0.05	No	0.168	5.94	0.017	0.59
2	0.81	2	0.05	Yes	0.168	5.94	0.017	0.59
3	0.81	2	0.50	No	0.532	18.77	0.053	1.88
4	0.81	2	0.50	Yes	0.532	18.77	0.053	1.88
5	0.81	2	5.00	No	1.681	59.35	0.168	5.94
6	0.81	2	5.00	Yes	1.681	59.35	0.168	5.94
7	4.86	12	0.05	No	19.977	705.41	1.998	70.54
8	4.86	12	0.05	Yes	19.977	705.41	1.998	70.54
9	4.86	12	0.50	No	63.174	2230.73	6.317	223.07
10	4.86	12	0.50	Yes	63.174	2230.73	6.317	223.07
11	4.86	12	5.00	No	199.775	7054.19	19.978	705.42
12	4.86	12	5.00	Yes	199.775	7054.19	19.978	705.42

Note: Minimum inflows apply only to triangular inflows.

Table 19. INFLOW HYDROGRAPH SHAPES FOR HYPOTHETICAL PIPES

Shape number	Shape	Runoff duration, hours	Maxium inflow	Minimum inflow
1	Constant	Continuous	90% of full pipe flow	9% of full pipe flow
2	Triangle	1		
3	Triangle	2		
4	Triangle	4		

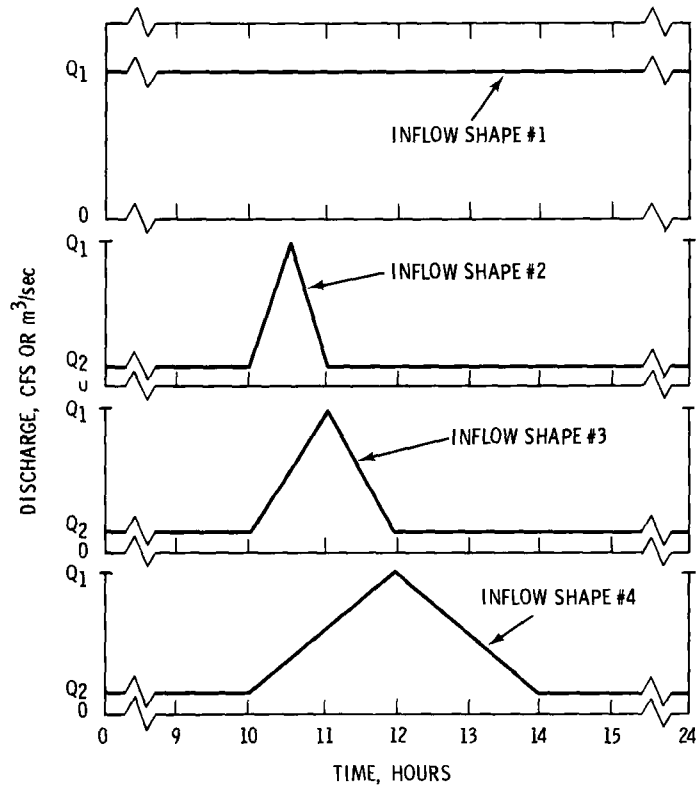


Figure 7. Hypothetical pipe inflow hydrograph shapes

the triangular shape, and an inverted triangular shape with a minimum of 0 mg/l and a constant concentration of 100 mg/l before and after the triangular shape. The durations of the triangular concentration shapes are identical to the triangular inflow hydrographs. A 2-hr duration was specified for the continuous inflow hydrographs. These inflow concentration shapes are summarized in Table 20 and shown in Figure 8. A time step of 5 minutes was suggested for the simulation runs. The hypothetical inflow hydrograph values for each pipe are tabulated at these time intervals in Tables 21 to 24 and the hypothetical inflow concentrations in Tables 25 to 27. The effect of different time discretizations was not tested.

It should be noted that some models allow the input of hourly values of dry-weather flow and quality and provide for linear interpolation at specified constant time intervals. This greatly simplifies the input of the hypothetical inflow hydrographs and concentrations.

Table 20. INFLOW CONCENTRATION SHAPES FOR HYPOTHETICAL PIPES

Shape number	Shape	Concentration duration, hours	Maximum concentration mg/ℓ	Minimum concentration mg/ℓ
1	Constant	Continuous	100	100
2	Triangle	See note	100	0
3	Inverted triangle		100	0

Note: Assume a base of two hours for the triangular concentrations when used with the continuous flow hydrographs, and a base equal to the base of the triangular inflow hydrographs when used with the triangular inflow hydrographs.

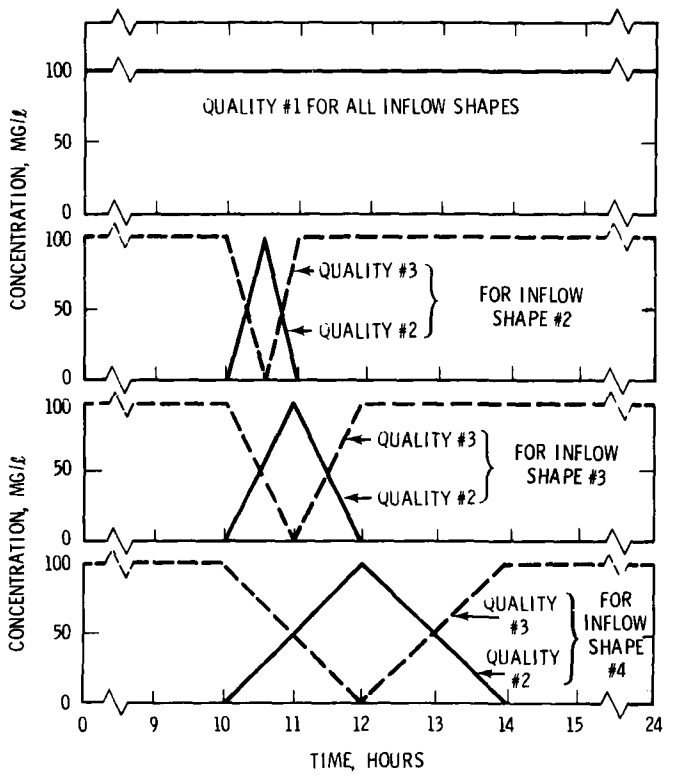


Figure 8. Hypothetical pipe inflow concentrations

Table 21. CONTINUOUS INFLOW FOR HYPOTHETICAL PIPES

Time step	Clock time, hr:min	Inflow											
		Pipe #1 & #2		Pipe #3 & #4		Pipe #5 & #6		Pipe #7 & #8		Pipe #9 & #10		Pipe #11 & #12	
		m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
1	10:10	0.168	5.94	0.532	18.77	1.681	59.35	19.977	705.41	63.174	2230.73	199.773	7054.19
2	10:05	↑	↑	↑	↑	↑	↑	↑	↑	↑	↑	↑	↑
3	10:10												
4	10:15												
5	10:20												
6	10:25												
7	10:30												
8	10:35												
9	10:40												
10	10:45												
11	10:50												
12	10:55	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
13	11:00	0.168	5.94	0.532	18.77	1.681	59.35	19.977	705.41	63.174	2230.73	199.773	7054.19
14	11:05	↑	↑	↑	↑	↑	↑	↑	↑	↑	↑	↑	↑
15	11:10												
16	11:15												
17	11:20												
18	11:25												
19	11:30												
20	11:35												
21	11:40												
22	11:45												
23	11:50												
24	11:55	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
25	12:00	0.168	5.94	0.532	18.77	1.681	59.35	19.977	705.41	63.174	2230.73	199.773	7054.19

Note: Assume constant inflows equal to the initial and final tabulated value before and after the tabulated time steps.

Table 22. ONE-HOUR TRIANGULAR INFLOW FOR HYPOTHETICAL PIPES

Time step	Clock time, hr:min	Inflow											
		Pipe #1 & #2		Pipe #3 & #4		Pipe #5 & #6		Pipe #7 & #8		Pipe #9 & #10		Pipe #11 & #12	
		m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
1	10:00	0.017	0.59	0.053	1.88	0.168	5.94	1.998	70.54	6.317	223.07	19.977	705.42
2	10:05	0.042	1.48	0.133	4.70	0.420	14.83	4.995	176.35	15.793	557.68	49.943	1763.55
3	10:10	0.067	2.37	0.213	7.52	0.672	23.73	7.991	282.16	25.269	892.28	79.910	2821.68
4	10:15	0.092	3.26	0.292	10.34	0.925	32.63	10.988	387.97	34.745	1226.89	109.876	3879.81
5	10:20	0.118	4.15	0.372	13.16	1.177	41.53	13.984	493.78	44.221	1561.49	139.842	4937.94
6	10:25	0.143	5.04	0.452	15.98	1.429	50.43	16.981	599.59	53.697	1896.10	169.808	5996.07
7	10:30	0.168	5.94	0.532	18.77	1.681	59.35	19.977	705.41	63.173	2230.70	199.775	7054.20
8	10:35	0.143	5.04	0.452	15.98	1.429	50.43	16.981	599.59	53.697	1896.10	169.808	5996.07
9	10:40	0.118	4.15	0.372	13.16	1.177	41.53	13.984	493.78	44.221	1561.49	139.842	4937.94
10	10:45	0.092	3.26	0.292	10.34	0.925	32.63	10.988	387.97	34.745	1226.89	109.876	3879.81
11	10:50	0.067	2.37	0.213	7.52	0.672	23.73	7.991	282.16	25.269	892.29	79.910	2821.68
12	10:55	0.042	1.48	0.133	4.70	0.420	14.83	4.995	176.35	15.793	557.68	49.943	1763.55
13	11:00	0.017	0.59	0.053	1.88	0.168	5.94	1.998	70.54	6.317	223.07	19.977	705.42

Note: Assume constant inflows equal to the initial and final tabulated value before and after the tabulated time steps.

Table 23. TWO-HOUR TRIANGULAR INFLOW FOR HYPOTHETICAL PIPES

Time step	Clock time, hr:min	Inflow											
		Pipe #1 & #2		Pipe #3 & #4		Pipe #5 & #6		Pipe #7 & #8		Pipe #9 & #10		Pipe #11 & #12	
		m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
1	10:00	0.017	0.59	0.053	1.88	0.168	5.94	1.998	70.54	6.317	223.07	19.977	705.42
2	10:05	0.029	1.04	0.093	3.28	0.294	10.39	3.496	123.45	11.055	390.37	34.960	1234.48
3	10:10	0.042	1.48	0.133	4.69	0.420	14.84	4.994	176.35	15.793	557.68	49.944	1763.55
4	10:15	0.055	1.93	0.173	6.10	0.546	19.29	6.493	229.26	20.531	724.98	64.927	2292.61
5	10:20	0.067	2.37	0.213	7.51	0.672	23.74	7.991	282.16	25.269	892.28	79.910	2821.68
6	10:25	0.080	2.82	0.253	8.92	0.798	28.19	9.489	335.07	30.007	1059.58	94.893	3350.74
7	10:30	0.092	3.26	0.293	10.32	0.925	32.64	10.987	387.98	34.745	1226.89	109.876	3879.81
8	10:35	0.105	3.71	0.333	11.73	1.051	37.09	12.486	440.88	39.483	1394.19	124.859	4408.87
9	10:40	0.118	4.15	0.372	13.14	1.177	41.55	13.984	493.79	44.221	1561.49	139.842	4937.94
10	10:45	0.130	4.60	0.412	14.55	1.303	46.00	15.482	546.69	48.959	1728.79	154.826	5467.01
11	10:50	0.143	5.04	0.452	15.95	1.429	50.45	16.980	599.60	53.697	1896.10	169.809	5996.07
12	10:55	0.155	5.49	0.492	17.36	1.555	54.90	18.479	652.50	58.435	2063.40	184.792	6525.13
13	11:00	0.168	5.94	0.532	18.77	1.681	59.35	19.977	705.41	63.173	2230.70	199.775	7054.20
14	11:05	0.155	5.49	0.492	17.36	1.555	54.90	18.479	652.50	58.435	2063.40	184.792	6525.13
15	11:10	0.143	5.04	0.452	15.95	1.429	50.45	16.980	599.60	53.697	1896.10	169.809	5996.07
16	11:15	0.130	4.60	0.412	14.55	1.303	46.00	15.482	546.69	48.959	1728.79	154.826	5467.01
17	11:20	0.118	4.15	0.372	13.14	1.177	41.55	13.984	493.79	44.221	1561.49	139.842	4937.94
18	11:25	0.105	3.71	0.333	11.73	1.051	37.09	12.486	440.88	39.483	1394.19	124.859	4408.87
19	11:30	0.092	3.26	0.293	10.32	0.925	32.64	10.987	387.98	34.745	1226.89	109.876	3879.81
20	11:35	0.080	2.82	0.253	8.92	0.798	28.19	9.489	335.07	30.007	1059.58	94.893	3350.74
21	11:40	0.067	2.37	0.213	7.51	0.672	23.74	7.991	282.16	25.269	892.28	79.910	2821.68
22	11:45	0.055	1.93	0.173	6.10	0.546	19.29	6.493	229.26	20.531	724.98	64.927	2292.61
23	11:50	0.042	1.48	0.133	4.69	0.420	14.84	4.994	176.35	15.793	557.68	49.944	1763.55
24	11:55	0.029	1.04	0.093	3.28	0.294	10.39	3.496	123.45	11.055	390.37	34.960	1234.48
25	12:00	0.017	0.59	0.053	1.88	0.168	5.94	1.998	70.54	6.317	223.07	19.977	705.42

Note: Assume constant inflows equal to the initial and final tabulated value before and after the tabulated time steps.

Table 24. FOUR-HOUR TRIANGULAR INFLOW FOR HYPOTHETICAL PIPES

Time step	Clock time, hr:min	Inflow											
		Pipe #1 & #2		Pipe #3 & #4		Pipe #5 & #6		Pipe #7 & #8		Pipe #9 & #10		Pipe #11 & #12	
		m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
1	10:00	0.017	0.59	0.053	1.88	0.168	5.94	1.998	70.54	6.317	223.07	19.977	705.42
2	10:05	0.023	0.81	0.073	2.58	0.231	8.16	2.747	96.99	8.686	306.72	27.469	969.95
3	10:10	0.029	1.04	0.093	3.29	0.294	10.38	3.496	123.45	11.055	390.37	34.960	1234.49
4	10:15	0.036	1.26	0.113	3.99	0.357	12.61	4.245	149.90	13.424	474.02	42.452	1499.02
5	10:20	0.042	1.48	0.133	4.70	0.420	14.83	4.995	176.35	15.793	557.68	49.943	1763.55
6	10:25	0.048	1.71	0.153	5.40	0.483	17.06	5.744	202.81	18.162	641.33	57.435	2028.08
7	10:30	0.055	1.93	0.173	6.10	0.546	19.28	6.493	229.26	20.531	724.98	64.927	2292.62
8	10:35	0.061	2.15	0.193	6.81	0.609	21.51	7.242	255.71	22.900	808.63	72.418	2557.15
9	10:40	0.067	2.37	0.213	7.51	0.672	23.74	7.991	282.16	25.269	892.28	79.910	2821.68
10	10:45	0.074	2.60	0.233	8.21	0.735	25.96	8.740	308.62	27.638	975.93	87.401	3086.21
11	10:50	0.080	2.82	0.253	8.92	0.798	28.19	9.489	335.07	30.007	1059.58	94.893	3350.75
12	10:55	0.086	3.04	0.273	9.62	0.861	30.41	10.238	361.52	32.376	1143.23	102.384	3615.28
13	11:00	0.092	3.27	0.293	10.33	0.925	32.64	10.988	387.98	34.745	1226.89	109.876	3879.81
14	11:05	0.098	3.49	0.313	11.03	0.988	34.87	11.737	414.43	37.114	1310.54	117.368	4144.34
15	11:10	0.105	3.71	0.333	11.73	1.051	37.09	12.486	440.88	39.483	1394.19	124.859	4408.88
16	11:15	0.111	3.93	0.353	12.44	1.114	39.32	13.235	467.33	41.852	1477.84	132.351	4673.41
17	11:20	0.118	4.15	0.372	13.14	1.177	41.54	13.984	493.79	44.221	1561.49	139.842	4937.94
18	11:25	0.124	4.38	0.392	13.84	1.240	43.77	14.733	520.24	46.590	1645.14	147.334	5202.47
19	11:30	0.130	4.60	0.412	14.55	1.303	46.00	15.482	546.69	48.959	1728.79	154.825	5467.01
20	11:35	0.137	4.83	0.432	15.25	1.366	48.22	16.232	573.15	51.328	1812.44	162.317	5731.54
21	11:40	0.143	5.05	0.452	15.96	1.429	50.45	16.981	599.60	53.697	1896.10	169.809	5996.07
22	11:45	0.149	5.27	0.472	16.66	1.492	52.67	17.730	626.05	56.066	1979.75	177.300	6260.60
23	11:50	0.155	5.49	0.492	17.36	1.555	54.90	18.479	652.50	58.435	2063.40	184.792	6525.14
24	11:55	0.162	5.72	0.512	18.07	1.618	57.12	19.228	678.96	60.804	2147.05	192.283	6789.67
25	12:00	0.168	5.94	0.532	18.77	1.681	59.35	19.977	705.41	63.173	2230.70	199.775	7054.20

Table 24 (Continued). FOUR-HOUR TRIANGULAR INFLOW FOR HYPOTHETICAL PIPES

Time step	Clock time, hr:min	Inflow											
		Pipe #1 & #2		Pipe #3 & #4		Pipe #5 & #6		Pipe #7 & #8		Pipe #9 & #10		Pipe #11 & #12	
		m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
26	12:05	0.162	5.72	0.512	18.07	1.618	57.12	19.228	678.96	60.804	2147.05	192.283	6789.67
27	12:10	0.155	5.49	0.492	17.36	1.555	54.90	18.479	652.50	58.435	2063.40	184.792	6525.14
28	12:15	0.149	5.27	0.472	16.66	1.492	52.67	17.730	626.05	56.066	1979.75	177.300	6260.60
29	12:20	0.143	5.05	0.452	15.96	1.429	50.45	16.981	599.60	53.697	1896.10	169.809	5996.07
30	12:25	0.137	4.83	0.432	15.25	1.366	48.22	16.232	573.15	51.328	1812.44	162.317	5731.54
31	12:30	0.130	4.60	0.412	14.55	1.303	46.00	15.482	546.69	48.959	1728.79	154.825	5467.01
32	12:35	0.124	4.38	0.392	13.84	1.240	43.77	14.733	520.24	46.590	1645.14	147.334	5202.47
33	12:40	0.118	4.15	0.372	13.14	1.177	41.54	13.984	493.79	44.221	1561.49	139.842	4937.94
34	12:45	0.111	3.93	0.353	12.44	1.114	39.32	13.235	467.33	41.852	1477.84	132.351	4673.41
35	12:50	0.105	3.71	0.333	11.73	1.051	37.09	12.486	440.88	39.483	1394.19	124.859	4408.88
36	12:55	0.099	3.49	0.313	11.03	0.988	34.87	11.737	414.43	37.114	1310.54	117.368	4144.34
37	13:00	0.092	3.27	0.293	10.33	0.925	32.64	10.988	387.98	34.745	1226.89	109.876	3879.81
38	13:05	0.086	3.04	0.273	9.62	0.861	30.41	10.238	361.52	32.376	1143.23	102.384	3615.28
39	13:10	0.080	2.82	0.253	8.92	0.798	28.19	9.489	335.07	30.007	1059.58	94.893	3350.75
40	13:15	0.074	2.60	0.233	8.21	0.735	25.96	8.740	308.62	27.638	975.93	87.401	3086.21
41	13:20	0.067	2.37	0.213	7.51	0.672	23.74	7.991	282.16	25.269	892.28	79.910	2821.68
42	13:25	0.061	2.15	0.193	6.81	0.609	21.51	7.242	255.71	22.900	808.63	72.418	2557.15
43	13:30	0.055	1.93	0.173	6.10	0.546	19.28	6.493	229.26	20.531	724.98	64.927	2292.62
44	13:35	0.048	1.71	0.153	5.40	0.483	17.06	5.744	202.81	18.162	641.33	57.435	2028.08
45	13:40	0.042	1.48	0.133	4.70	0.420	14.83	4.995	176.35	15.793	557.68	49.943	1763.55
46	13:45	0.036	1.26	0.113	3.99	0.357	12.61	4.245	149.90	13.424	474.02	42.452	1499.02
47	13:50	0.029	1.04	0.093	3.29	0.294	10.38	3.496	123.45	11.055	390.37	34.960	1234.49
48	13:55	0.023	0.81	0.073	2.58	0.231	8.16	2.747	96.99	8.686	306.72	27.469	969.95
49	14:00	0.017	0.59	0.053	1.88	0.168	5.94	1.998	70.54	6.317	223.07	19.977	705.42

Note: Assume constant inflows equal to the intitial and final value before and after the tabulated time steps.

Table 25. INFLOW CONCENTRATIONS FOR CONTINUOUS AND TWO-HOUR TRIANGULAR INFLOW HYDROGRAPHS FOR HYPOTHETICAL PIPES

Time step	Clock time, hr:min	Concentrations		
		Quality #1 mg/ℓ	Quality #2 mg/ℓ	Quality #3 mg/ℓ
1	10:00	100.00	0.00	100.00
2	10:05	100.00	8.33	91.67
3	10:10	100.00	16.67	83.33
4	10:15	100.00	25.00	75.00
5	10:20	100.00	33.33	66.67
6	10:25	100.00	41.67	58.33
7	10:30	100.00	50.00	50.00
8	10:35	100.00	58.33	41.67
9	10:40	100.00	66.67	33.33
10	10:45	100.00	75.00	25.00
11	10:50	100.00	83.33	16.67
12	10:55	100.00	91.67	8.33
13	11:00	100.00	100.00	0.00
14	11:05	100.00	91.67	8.33
15	11:10	100.00	83.33	16.67
16	11:15	100.00	75.00	25.00
17	11:20	100.00	66.67	33.33
18	11:25	100.00	58.33	41.67
19	11:30	100.00	50.00	50.00
20	11:35	100.00	41.67	58.33
21	11:40	100.00	33.33	66.67
22	11:45	100.00	25.00	75.00
23	11:50	100.00	16.67	83.33
24	11:55	100.00	8.33	91.67
25	12:00	100.00	0.00	100.00
26	12:05	100.00	0.00	100.00

Note: Assume constant concentrations equal to the initial and final tabulated value before and after the tabulated time steps.

Table 26. INFLOW CONCENTRATIONS FOR ONE-HOUR TRIANGULAR INFLOW HYDROGRAPHS FOR HYPOTHETICAL PIPES

Time step	Clock time, hr:min	Concentrations		
		Quality #1 mg/ℓ	Quality #2 mg/ℓ	Quality #3 mg/ℓ
1	10:00	100.00	0.00	100.00
2	10:05	100.00	16.67	83.33
3	10:10	100.00	33.33	66.67
4	10:15	100.00	50.00	50.00
5	10:20	100.00	66.67	33.33
6	10:25	100.00	83.33	16.67
7	10:30	100.00	100.00	0.00
8	10:35	100.00	83.33	16.67
9	10:40	100.00	66.67	33.33
10	10:45	100.00	50.00	50.00
11	10:50	100.00	33.33	66.67
12	10:55	100.00	16.67	83.33
13	11:00	100.00	0.00	100.00
14	11:05	100.00	0.00	100.00

Note: Assume constant concentrations equal to the initial and final tabulated value before and after the tabulated time steps.

Table 27. INFLOW CONCENTRATIONS FOR FOUR-HOUR TRIANGULAR INFLOW HYDROGRAPHS FOR HYPOTHETICAL PIPES

Time step	Clock time, hr:min	Concentrations Quality #1 mg/ℓ	Quality #2 mg/ℓ	Quality #3 mg/ℓ
1	10:00	100.00	0.00	100.00
2	10:05	100.00	4.17	95.83
3	10:10	100.00	8.33	91.67
4	10:15	100.00	12.50	87.50
5	10:20	100.00	16.67	83.33
6	10:25	100.00	20.83	79.17
7	10:30	100.00	25.00	75.00
8	10:35	100.00	29.17	70.83
9	10:40	100.00	33.33	66.67
10	10:45	100.00	37.50	62.50
11	10:50	100.00	41.67	58.33
12	10:55	100.00	45.83	54.17
13	11:00	100.00	50.00	50.00
14	11:05	100.00	54.17	45.83
15	11:10	100.00	58.33	41.67
16	11:15	100.00	62.50	37.50
17	11:20	100.00	66.67	33.33
18	11:25	100.00	70.83	29.17
19	11:30	100.00	75.00	25.00
20	11:35	100.00	79.17	20.83
21	11:40	100.00	83.33	16.67
22	11:45	100.00	87.50	12.50
23	11:50	100.00	91.67	8.33
24	11:55	100.00	95.83	4.17
25	12:00	100.00	100.00	0.00
26	12:05	100.00	95.83	4.17
27	12:10	100.00	91.67	8.33
28	12:15	100.00	87.50	12.50
29	12:20	100.00	83.33	16.67
30	12:25	100.00	79.17	20.83
31	12:30	100.00	75.00	25.00
32	12:35	100.00	70.83	29.17
33	12:40	100.00	66.67	33.33
34	12:45	100.00	62.50	37.50
35	12:50	100.00	58.33	41.67
36	12:55	100.00	54.17	45.83
37	13:00	100.00	50.00	50.00
38	13:05	100.00	45.83	54.17
39	13:10	100.00	41.67	58.33
40	13:15	100.00	37.50	62.50
41	13:20	100.00	33.33	66.67
42	13:25	100.00	29.17	70.83
43	13:30	100.00	25.00	75.00
44	13:35	100.00	20.83	79.17
45	13:40	100.00	16.67	83.33
46	13:45	100.00	12.50	87.50
47	13:50	100.00	8.33	91.67
48	13:55	100.00	4.17	95.83
49	14:00	100.00	0.00	100.00
50	14:05	100.00	0.00	100.00

Note: Assume constant concentrations equal to the initial and final tabulated value before and after the tabulated time steps.

REAL CATCHMENT DATA

An attempt was made to select three real catchments with drainage areas covering three orders of magnitude which had reliable concurrent rainfall, runoff, and water quality data. Since no small catchment was found which also had quality data, the Oakdale Avenue catchment in Chicago, Illinois, with a drainage area of 5.22 ha (12.9 acres) was selected because it had reliable rainfall and runoff data. An intermediate size catchment was selected which had rainfall, runoff, and water quality data, but it was subsequently discovered that the data were unreliable. It was then too late in the study to find substitute data. The Bloody Run catchment in Cincinnati, Ohio, with a drainage area of 964 ha (2380 acres) was selected to represent a large catchment. It has rainfall, runoff, and water quality data. Difficulties were encountered, however, in ascertaining the reliability of the data, most of which had to be scaled from plotted hyetographs, hydrographs, and water quality graphs since only small portions were available in tabular form.

Description of the Oakdale Avenue Catchment

The Oakdale Avenue catchment is located in an urban area about 6 miles northwest of downtown Chicago, Illinois, (Tucker, 1968). This catchment is 5.22 ha (12.9 acres) in size (approximately 2-1/2 blocks long by one block wide) and consists entirely of residential lots and adjoining streets (Figure 9). A breakdown of the extent of pervious and impervious surfaces in the drainage area is given in Table 28.

The backbone of the drainage system consists of a 76-cm (30-in.) diameter reinforced concrete combined sewer that drains east along Oakdale Avenue for about two blocks. The existing 76-cm (30-in.) combined sewer was installed in 1958 and replaced a smaller combined sewer of inadequate stormwater capacity. The 76-cm (30-in.) combined sewer drains into a 3.20 m x 3.20 m (10.5 ft x 10.5 ft) concrete combined trunk sewer that drains north toward the North Branch of the Chicago River.

As shown in Figure 10, the catchment was divided into 13 subcatchments for the runoff simulations. Each of the 13 subcatchments has its individual inlet manhole. In Figure 10, the elements of the sewer system (subcatchments, pipes, and manholes) are numbered for identification. Table 29 gives physical characteristics of the subcatchments, which vary in size from 0.33 ha (0.82 acres) to 0.65 ha (1.60 acres). The imperviousness of the subcatchments varies from 39.5 to 56.5 percent. Ground slopes vary from 0.37 to 0.90 percent. Manning's roughness coefficient was assumed to be 0.012 or 0.013 for the impervious areas and 0.350 for the pervious areas.

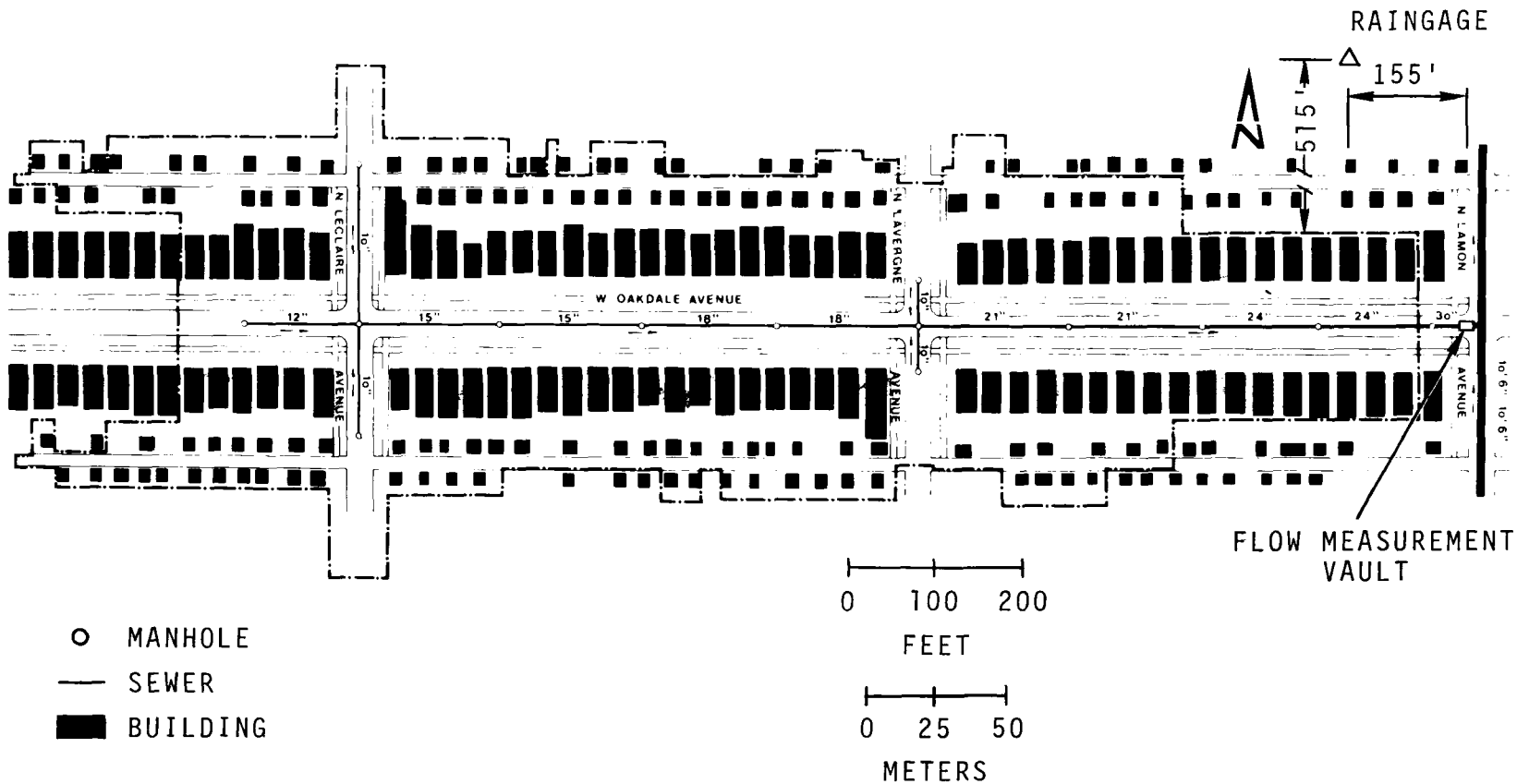


Figure 9. Map of Oakdale Avenue catchment, Chicago, Illinois, showing land use and sewer layout

Table 28. BREAKDOWN OF PERVIOUS AND IMPERVIOUS SURFACES IN THE OAKDALE AVENUE CATCHMENT

Land use	Drainage area ha	Drainage area acres
Impervious area:		
Draining directly to combined sewer:		
Houses	1.02	2.52
Streets	0.64	1.58
Alleys	0.23	0.58
Garages	0.11	0.27
Sidewalks		
to street	0.04	0.11
to alley	0.04	0.09
Subtotals	2.08	5.15
Draining indirectly to combined sewer:		
Public walks	0.23	0.57
Private walks	0.06	0.15
Subtotals	0.29	0.72
Total impervious area:	2.37	5.87
Pervious area: (grassed, etc.)		
Total pervious area:	2.85	7.05
Total drainage area:	5.22	12.92

Table 29. PHYSICAL CHARACTERISTICS OF SUBCATCHMENTS OF THE OAKDALE AVENUE CATCHMENT

Subcatchment number	Inlet manhole number	Length		Width		Drainage area		Imperviousness, percent	Ground slope, percent	Manning's n	
		m	ft	m	ft	ha	acres			imperv. area	pervious area
1	1	212.4	697	30.5	100	0.65	1.60	44.0	0.60	0.013	0.350
2	2	74.4	244	45.7	150	0.34	0.84	44.0	0.90	0.012	0.350
3	3	212.4	697	24.4	80	0.52	1.28	51.7	0.60	0.013	0.350
4	4	166.1	545	12.2	40	0.20	0.50	55.4	0.90	0.013	0.350
5	5	78.9	259	48.8	160	0.38	0.95	39.5	0.90	0.012	0.350
6	6	78.3	257	61.0	200	0.48	1.18	41.7	0.90	0.012	0.350
7	7	75.6	248	48.8	160	0.37	0.91	40.5	0.90	0.012	0.350
8	8	147.8	485	24.4	80	0.36	0.89	54.0	0.37	0.013	0.350
9	9	195.0	643	12.2	40	0.24	0.59	56.5	0.90	0.013	0.350
10	10	174.0	571	27.4	90	0.48	1.18	47.4	0.37	0.013	0.350
11	11	78.9	259	48.8	160	0.38	0.95	40.6	0.90	0.012	0.350
12	12	80.5	264	61.0	200	0.49	1.21	40.6	0.90	0.012	0.350
13	13	68.0	223	48.8	160	0.33	0.82	43.1	0.90	0.012	0.350

Note: Length is in the direction of flow, width is perpendicular to direction of flow, for a rectangular area of equal size as actual subcatchment area.

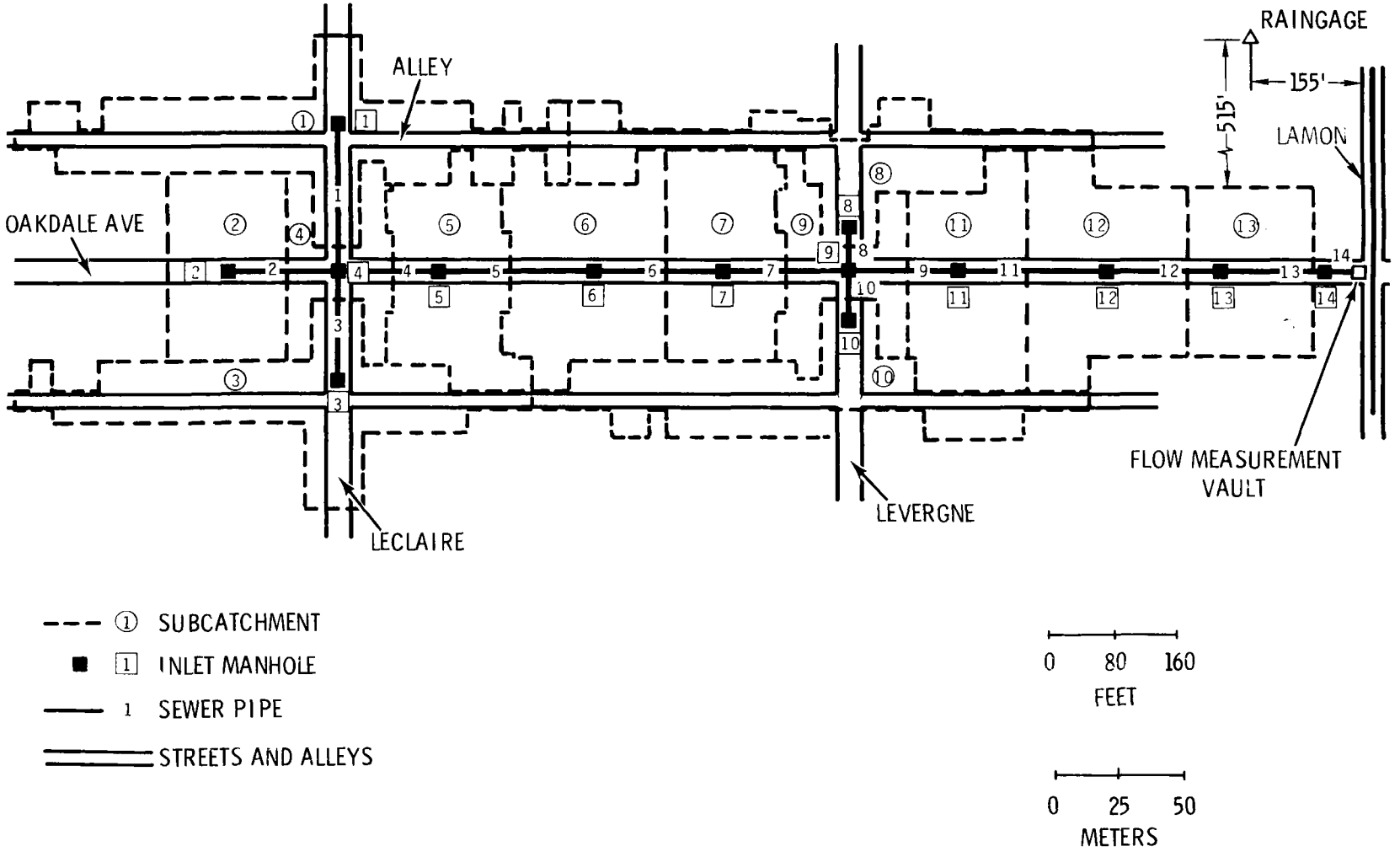


Figure 10. Map of Oakdale Avenue catchment showing subcatchments and sewer system elements

As shown in Table 30, the maximum infiltration, the minimum infiltration, and the decay rate of infiltration were assumed to be 63.5 mm/hr (2.50 in./hr), 11.4 mm/hr (0.45 in./hr), and 0.00115 sec^{-1}, respectively. The overall volumes of retention storage on the pervious and the impervious areas were assumed to be 5.08 mm (0.20 in.) and 2.03 mm (0.08 in.), respectively.

As shown in Table 31, the diameters of the sewer pipes vary from 25 cm (10 in.) to 76 cm (30 in.). The pipe slopes vary from 0.30 to 4.20 percent. The Manning coefficients of these pipes were assumed to be 0.012.

Monitoring System of the Oakdale Avenue Catchment

The rainfall-runoff measuring and recording systems were installed in conjunction with the new 76-cm (30-in.) combined sewer operation in 1959. Runoff has been measured with a Simplex 76-cm (30-in.) type "S" parabolic flume housed in a vault located on the corner of Lamon and Oakdale Avenues. Rainfall measurements have been conducted using a tipping bucket raingage which is located about one block north of the drainage area on top of Falconer Elementary School. No runoff water quality measurements were conducted.

The raingage and a transmitter in the flow measuring vault are connected to remote recorders through leased telephone lines. The data instrumentation, transmission, and recording systems operate only during periods of rainfall. The system is put into operation when the first 0.25 mm (0.01 in.) of rain tips the bucket in the raingage.

Rainfall and Runoff Data of the Oakdale Avenue Catchment

Rainfall and runoff in the Oakdale Avenue catchment have been periodically measured and recorded since 1957 by the Chicago Department of Public Works, Bureau of Engineering. Storm data considered reasonably reliable by the Bureau of Engineering are identified in Table 32. Detailed rainfall and runoff data for these storms have been reported by Tucker (1968).

In the present study, the rainstorms on the following four dates were selected to test several methods of computing urban runoff quantity: May 19, 1959; July 2, 1960; July 26, 1960; and August 2, 1963. These rainstorms were selected since they represent a typical range of possible combinations of rainfall intensities and durations.

Table 30. ESTIMATE OF SOIL CHARACTERISTICS FOR THE OAKDALE AVENUE CATCHMENT

Item	Units	
	Metric	British
Pervious areas:		
Retention storage capacity	5.1 mm	0.20 in.
Maximum infiltration rate	63.5 mm/hr	2.50 in./hr
Minimum infiltration rate	11.4 mm/hr	0.45 in./hr
Infiltration decay rate	0.00115 sec^{-1}	0.00115 sec^{-1}
Impervious areas:		
Retention storage capacity	2.0 mm	0.08 in.
Maximum infiltration rate	0.0 mm/hr	0.00 in./hr
Minimum infiltration rate	0.0 mm/hr	0.00 in./hr
Infiltration decay rate	0.0 sec^{-1}	0.00 sec^{-1}

Notes: The infiltration rates are defined by Horton's infiltration equation. The retention storage capacities represent the amount of rain which has to fall before surface runoff begins.

Table 31. PHYSICAL CHARACTERISTICS OF SEWER ELEMENTS OF THE OAKDALE AVENUE CATCHMENT

Pipe number	Downstream junction number	Diameter		Length		Invert slope, percent	Manning's n
		m	ft	m	ft		
1	4	0.25	0.83	51.8	170	0.71	0.012
2	4	0.30	1.00	39.6	130	0.72	0.012
3	4	0.25	0.83	32.0	105	1.08	0.012
4	5	0.38	1.25	48.8	160	0.45	0.012
5	6	0.38	1.25	53.3	175	0.45	0.012
6	7	0.46	1.50	48.5	159	0.40	0.012
7	9	0.46	1.50	53.0	174	0.40	0.012
8	9	0.25	0.83	15.2	50	3.78	0.012
9	11	0.53	1.75	50.6	166	0.35	0.012
10	9	0.25	0.83	13.7	45	4.20	0.012
11	12	0.53	1.75	51.8	170	0.35	0.012
12	13	0.61	2.00	41.8	137	0.30	0.012
13	14	0.61	2.00	40.2	132	0.30	0.012
14	--	0.76	2.50	13.4	44	0.30	0.012

Table 32. RELIABLE RAINFALL AND RUNOFF RECORDS FOR THE OAKDALE AVENUE CATCHMENT

Date of storm	Remarks (no. of flow peaks)
May 19, 1959	Good peak flow (1)
July 29, 1959	Good peak flow (1)
October 6, 1959	2 small peaks, long period of rainfall
July 2, 1960	3 peaks, 3rd peak - flume was flooded
July 22, 1960	Small peak flow (1)
July 26, 1960	Medium peak flow (3)
August 20, 1960	2 peaks, 1st peak unreliable
September 18, 1960	Medium peak flow (1)
October 14, 1960	Small peak flow (1)
April 24, 1961	Small peak flow (2)
August 31, 1961	2 peaks, 1st peak unreliable
July 2, 1962	4 peaks, 4th peak - flume was flooded
April 17, 1963	Medium peak flow (2)
April 19, 1963	Good peak flow (1)
April 29 & 30, 1963	Medium peak flow (2)
May 4, 1963	2 small peaks, 1st peak unreliable
June 19, 1963	Medium peak flow (1)
July 13, 1963	Small peak flow (5)
August 2, 1963	Medium peak flow (2)
August 24, 1963	Small peak flow (2)
September 22, 1964	Small peak flow (2)

The storm of May 19, 1959, (Figure 11 and Table 33) had high intensity rainfall of a short duration followed by low intensity rainfall of a long duration. This storm generated runoff with one good peak. The storm of July 2, 1960, (Figure 12 and Table 34) had lighter long-duration rainfall followed by heavier short-duration rainfall. Runoff produced by this storm had three peaks and the last peak was the largest. Since the flume was flooded at the flow measuring location during the last high-intensity rainfall, the recession curve of the last peak was not recorded.

The storm of July 26, 1960, (Figure 13 and Table 35) had relatively light long-duration rainfall which resulted in successive small peaks of runoff. The storm of August 2, 1963, (Figure 14 and Table 36) had two high intensity, short duration rainfalls which produced two successive medium peaks of runoff.

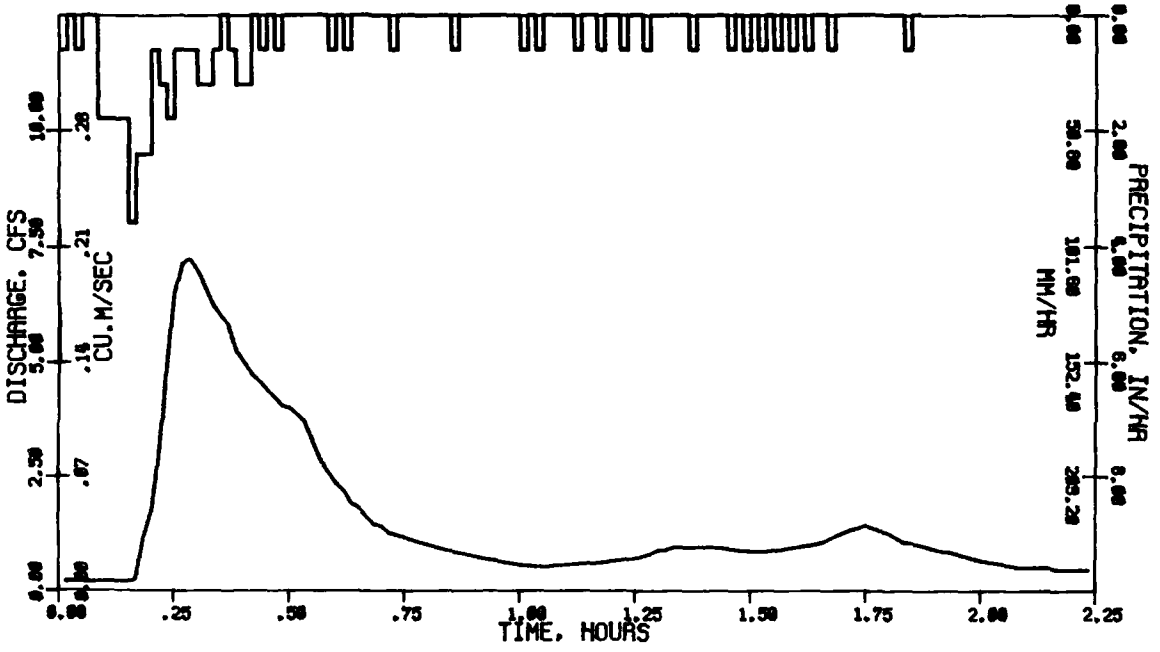


Figure 11. Rainfall Hyetograph and Runoff Hydrograph for the Storm of May 19, 1959 - Oakdale Avenue Catchment

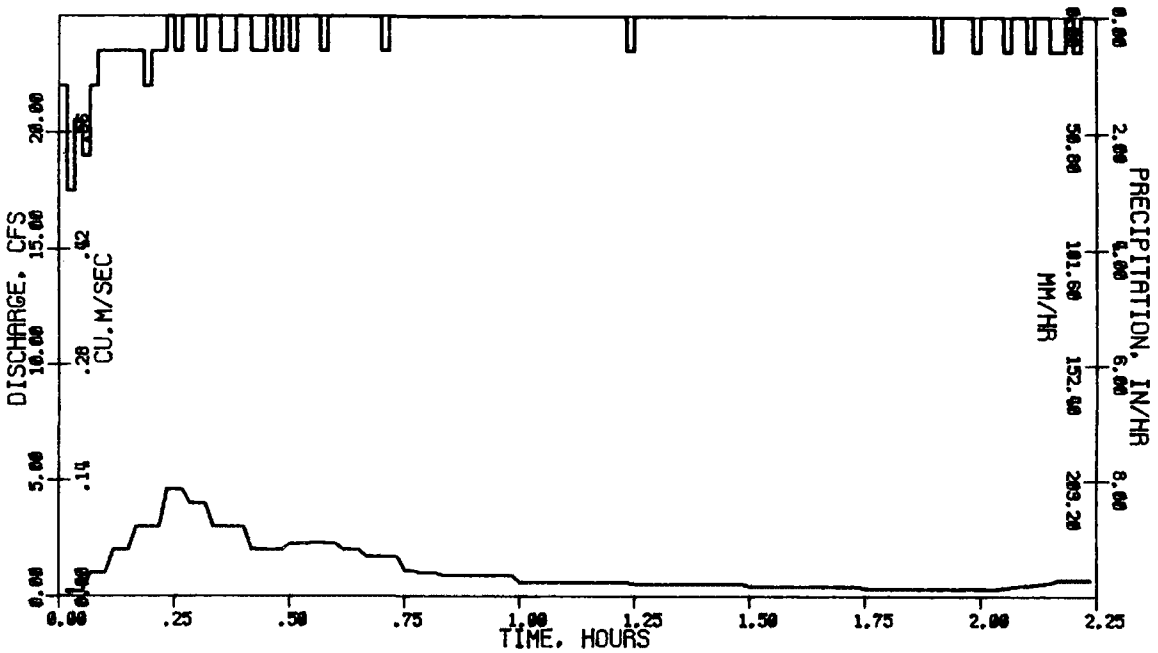


Figure 12a. Rainfall Hyetograph and Runoff Hydrograph for the Storm of July 2, 1960 -Oakdale Avenue Catchment

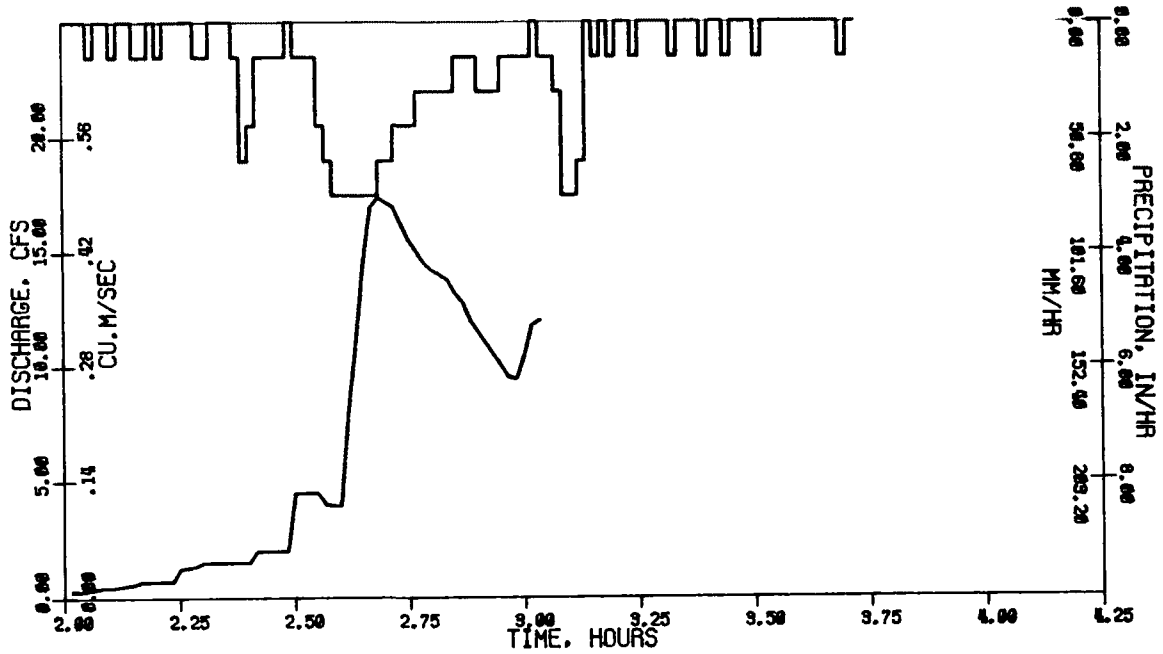


Figure 12b. Rainfall Hyetograph and Runoff Hydrograph for the Storm of July 2, 1960 -Oakdale Avenue Catchment

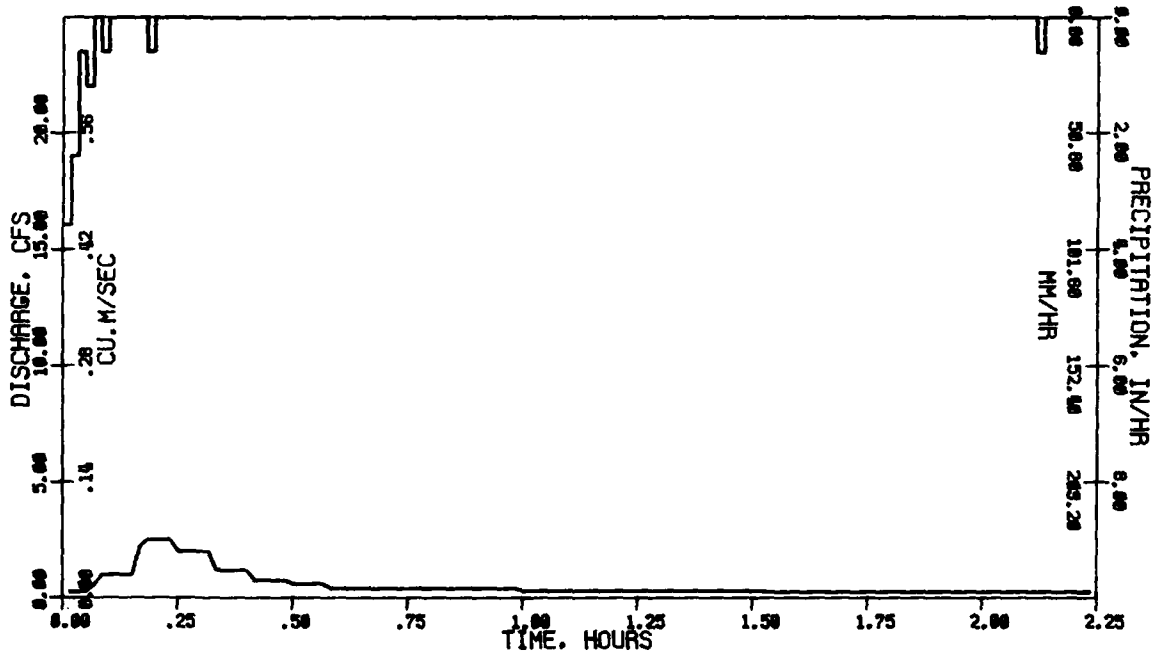


Figure 13a. Rainfall Hyetograph and Runoff Hydrograph for the Storm of July 26, 1960 -Oakdale Avenue Catchment

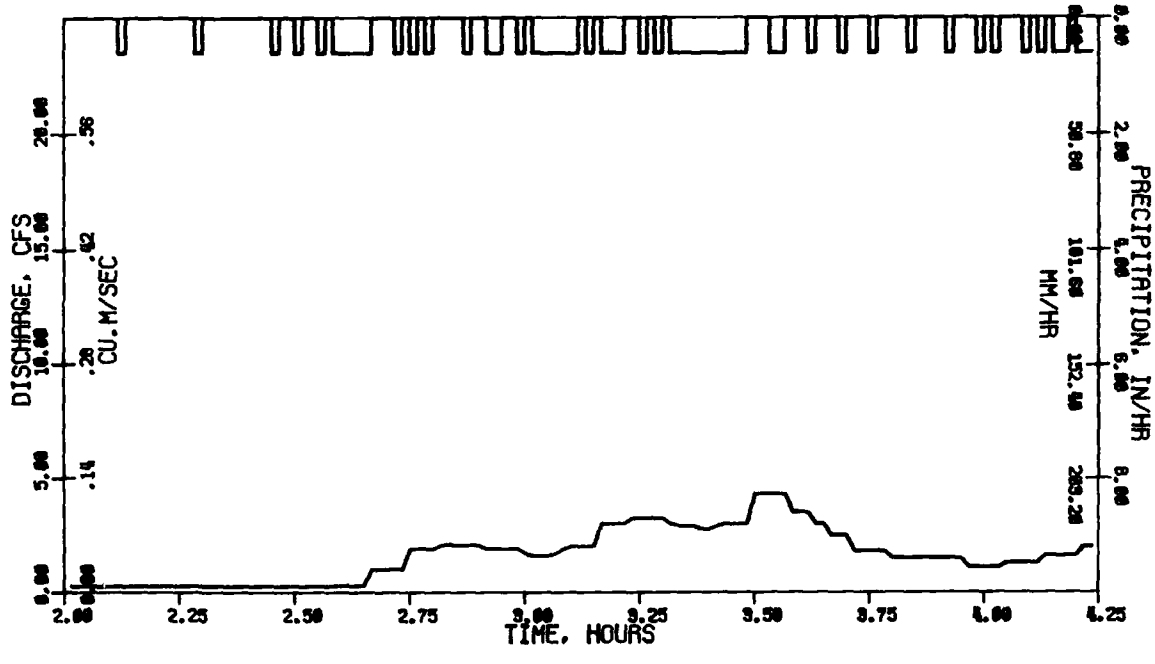


Figure 13b. Rainfall Hyetograph and Runoff Hydrograph for the Storm of July 26, 1960 -Oakdale Avenue Catchment

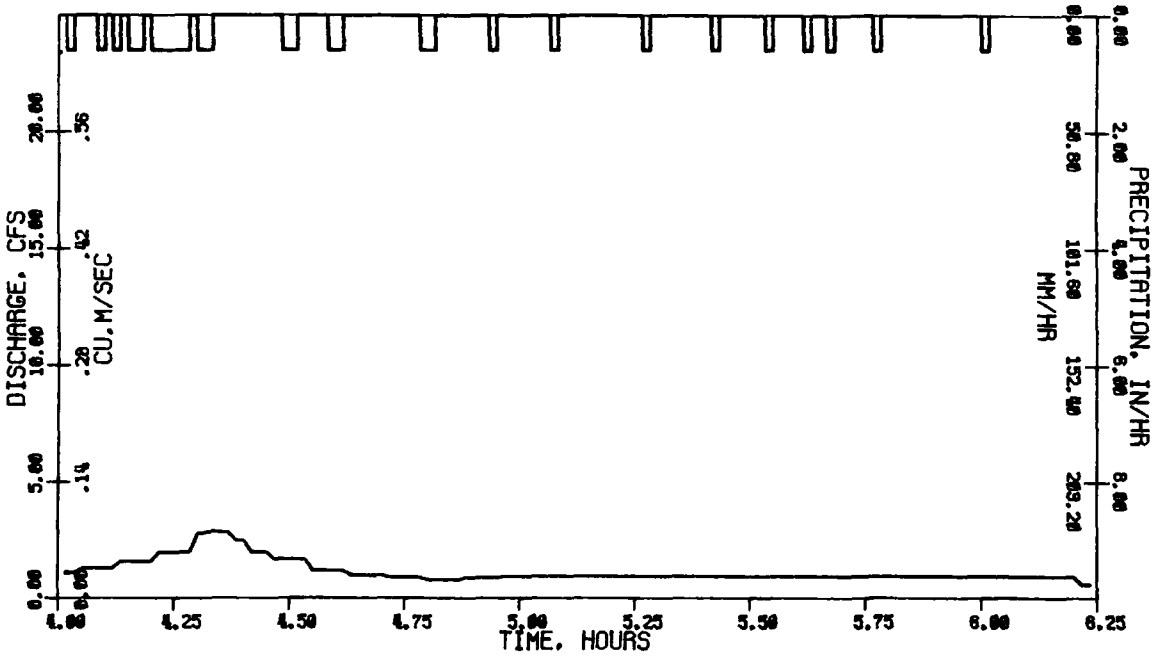


Figure 13c. Rainfall Hyetograph and Runoff Hydrograph for the Storm of July 26, 1960 -Oakdale Avenue Catchment

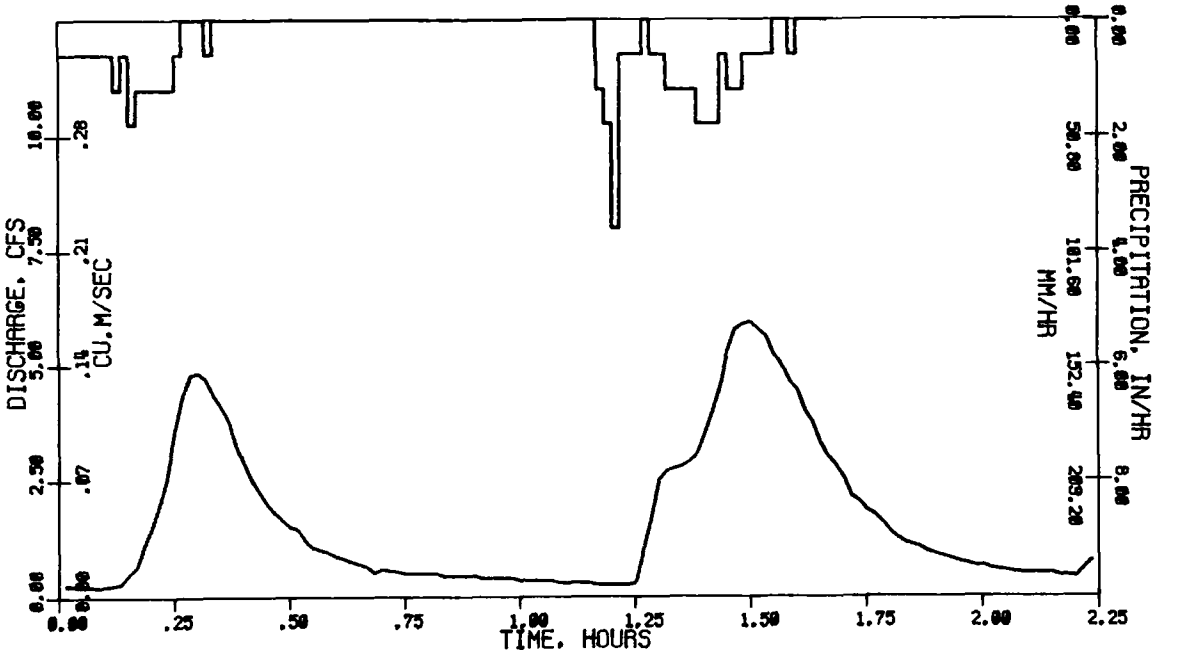


Figure 14. Rainfall Hyetograph and Runoff Hydrograph for the Storm of August 2, 1963 - Oakdale Avenue Catchment

Table 33. RAINFALL AND RUNOFF DATA FOR THE STORM OF MAY 19, 1959 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
1	15.2	0.60	0.3	0.01	0.006	0.20
2	0.0	0.00	0.3	0.01	0.006	0.20
3	15.2	0.60	0.5	0.02	0.006	0.20
4	0.0	0.00	0.5	0.02	0.006	0.20
5	0.0	0.00	0.5	0.02	0.006	0.20
6	45.5	1.80	1.3	0.05	0.006	0.20
7	45.5	1.80	2.0	0.08	0.006	0.20
8	45.5	1.80	2.8	0.11	0.006	0.20
9	45.5	1.80	3.6	0.14	0.006	0.20
10	91.2	3.60	5.1	0.20	0.007	0.25
11	61.0	2.40	6.1	0.24	0.033	1.15
12	61.0	2.40	7.1	0.28	0.048	1.70
13	15.2	0.60	7.4	0.29	0.085	3.00
14	30.5	1.20	7.9	0.31	0.142	5.00
15	45.5	1.80	8.6	0.34	0.184	6.50
16	15.2	0.60	8.9	0.35	0.202	7.15
17	15.2	0.60	9.1	0.36	0.205	7.25
18	15.2	0.60	9.4	0.37	0.198	7.00
19	30.5	1.20	9.9	0.39	0.188	6.65
20	30.5	1.20	10.4	0.41	0.177	6.25
21	15.2	0.60	10.7	0.42	0.170	6.00
22	0.0	0.00	10.7	0.42	0.164	5.80
23	15.2	0.60	10.9	0.43	0.149	5.25
24	30.5	1.20	11.4	0.45	0.142	5.00
25	30.5	1.20	11.9	0.45	0.135	4.75
26	0.0	0.00	11.9	0.47	0.130	4.60
27	15.2	0.60	12.2	0.48	0.125	4.40
28	0.0	0.00	12.2	0.48	0.120	4.25
29	15.2	0.60	12.4	0.49	0.115	4.05
30	0.0	0.00	12.4	0.49	0.113	4.00
31	0.0	0.00	12.4	0.49	0.109	3.85
32	0.0	0.00	12.4	0.49	0.105	3.70
33	0.0	0.00	12.4	0.49	0.092	3.25
34	0.0	0.00	12.4	0.49	0.081	2.85
35	0.0	0.00	12.4	0.49	0.074	2.60
36	15.2	0.60	12.7	0.50	0.067	2.35
37	0.0	0.00	12.7	0.50	0.062	2.20
38	15.2	0.60	13.0	0.51	0.054	1.90
39	0.0	0.00	13.0	0.51	0.051	1.80
40	0.0	0.00	13.0	0.51	0.045	1.60
41	0.0	0.00	13.0	0.51	0.041	1.45
42	0.0	0.00	13.0	0.51	0.040	1.40
43	0.0	0.00	13.0	0.51	0.035	1.25
44	15.2	0.60	13.2	0.52	0.034	1.20
45	0.0	0.00	13.2	0.52	0.033	1.15
46	0.0	0.00	13.2	0.52	0.031	1.10
47	0.0	0.00	13.2	0.52	0.030	1.05
48	0.0	0.00	13.2	0.52	0.028	1.00
49	0.0	0.00	13.2	0.52	0.027	0.95
50	0.0	0.00	13.2	0.52	0.025	0.90

Table 33 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF MAY 19, 1959 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
51	0.0	0.00	13.2	0.52	0.024	0.86
52	15.2	0.60	13.5	0.53	0.023	0.82
53	0.0	0.00	13.5	0.53	0.022	0.78
54	0.0	0.00	13.5	0.53	0.021	0.74
55	0.0	0.00	13.5	0.53	0.020	0.70
56	0.0	0.00	13.5	0.53	0.019	0.67
57	0.0	0.00	13.5	0.53	0.018	0.64
58	0.0	0.00	13.5	0.53	0.017	0.61
59	0.0	0.00	13.5	0.53	0.016	0.58
60	0.0	0.00	13.5	0.53	0.016	0.55
61	15.2	0.60	13.7	0.54	0.203	0.53
62	0.0	0.00	13.7	0.54	0.199	0.52
63	15.2	0.60	14.0	0.55	0.195	0.51
64	0.0	0.00	14.0	0.55	0.203	0.53
65	0.0	0.00	14.0	0.55	0.210	0.55
66	0.0	0.00	14.0	0.55	0.214	0.56
67	0.0	0.00	14.0	0.55	0.218	0.57
68	15.2	0.60	14.2	0.56	0.222	0.58
69	0.0	0.00	14.2	0.56	0.226	0.59
70	0.0	0.00	14.2	0.56	0.229	0.60
71	15.2	0.60	14.5	0.57	0.237	0.62
72	0.0	0.00	14.5	0.57	0.245	0.64
73	0.0	0.00	14.5	0.57	0.252	0.66
74	15.2	0.60	14.7	0.58	0.260	0.68
75	0.0	0.00	14.7	0.58	0.268	0.70
76	0.0	0.00	14.7	0.58	0.287	0.75
77	15.2	0.60	15.0	0.59	0.306	0.80
78	0.0	0.00	15.0	0.59	0.336	0.88
79	0.0	0.00	15.0	0.59	0.344	0.90
80	0.0	0.00	15.0	0.59	0.363	0.95
81	0.0	0.00	15.0	0.59	0.363	0.95
82	0.0	0.00	15.0	0.59	0.363	0.95
83	15.2	0.60	15.2	0.60	0.363	0.95
84	0.0	0.00	15.2	0.60	0.363	0.95
85	0.0	0.00	15.2	0.60	0.363	0.95
86	0.0	0.00	15.2	0.60	0.356	0.93
87	0.0	0.00	15.2	0.60	0.348	0.91
88	15.2	0.60	15.5	0.61	0.340	0.89
89	0.0	0.00	15.5	0.61	0.333	0.87
90	15.2	0.60	15.7	0.62	0.325	0.85
91	0.0	0.00	15.7	0.62	0.024	0.85
92	15.2	0.60	16.0	0.63	0.024	0.85
93	0.0	0.00	16.0	0.63	0.025	0.88
94	15.2	0.60	16.3	0.64	0.026	0.90
95	0.0	0.00	16.3	0.64	0.026	0.93
96	15.2	0.60	16.5	0.65	0.027	0.95
97	0.0	0.00	16.5	0.65	0.028	0.98
98	15.2	0.60	16.8	0.66	0.028	1.00
99	0.0	0.00	16.8	0.66	0.029	1.03
100	0.0	0.00	16.8	0.66	0.031	1.10

Table 33 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF MAY 19, 1959 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
101	15.2	0.60	17.0	0.67	0.033	1.18
102	0.0	0.00	17.0	0.67	0.035	1.25
103	0.0	0.00	17.0	0.67	0.037	1.32
104	0.0	0.00	17.0	0.67	0.039	1.36
105	0.0	0.00	17.0	0.67	0.040	1.42
106	0.0	0.00	17.0	0.67	0.039	1.36
107	0.0	0.00	17.0	0.67	0.037	1.30
108	0.0	0.00	17.0	0.67	0.035	1.25
109	0.0	0.00	17.0	0.67	0.033	1.15
110	0.0	0.00	17.0	0.67	0.030	1.05
111	15.2	0.60	17.3	0.68	0.029	1.03
112	0.0	0.00	17.3	0.68	0.028	1.00
113					0.027	0.95
114					0.026	0.90
115					0.024	0.85
116					0.024	0.83
117					0.022	0.78
118					0.021	0.75
119					0.020	0.70
120					0.018	0.65
121					0.018	0.62
122					0.017	0.59
123					0.016	0.56
124					0.015	0.53
125					0.014	0.50
126					0.014	0.50
127					0.014	0.50
128					0.014	0.50
129					0.014	0.50
130					0.013	0.45
131					0.013	0.45
132					0.013	0.45
133					0.013	0.45
134					0.013	0.45
135					0.011	0.40
136					0.011	0.40
137					0.011	0.40
138					0.011	0.37
139					0.011	0.37
140					0.010	0.35
141					0.010	0.35
142					0.009	0.33
143					0.009	0.33
144					0.009	0.31
145					0.009	0.31
146					0.008	0.29
147					0.008	0.29
148					0.008	0.27
149					0.008	0.27
150					0.007	0.25

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 34. RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 2, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
1	30.5	1.20	0.5	0.02	0.006	0.20
2	76.2	3.00	1.8	0.07	0.006	0.20
3	45.5	1.79	2.5	0.10	0.006	0.20
4	61.0	2.40	3.6	0.14	0.028	1.00
5	30.5	1.20	4.1	0.16	0.028	1.00
6	15.2	0.60	4.3	0.17	0.028	1.00
7	15.2	0.60	4.6	0.18	0.057	2.00
8	15.2	0.60	4.8	0.19	0.057	2.00
9	15.2	0.60	5.1	0.20	0.057	2.00
10	15.2	0.60	5.3	0.21	0.085	3.00
11	15.2	0.60	5.6	0.22	0.085	3.00
12	30.5	1.20	6.1	0.24	0.085	3.00
13	15.2	0.60	6.4	0.25	0.085	3.00
14	15.2	0.60	6.6	0.26	0.130	4.60
15	0.0	0.00	6.6	0.26	0.130	4.60
16	15.2	0.60	6.9	0.27	0.130	4.60
17	0.0	0.00	6.9	0.27	0.113	4.00
18	0.0	0.00	6.9	0.27	0.113	4.00
19	15.2	0.60	7.1	0.28	0.113	4.00
20	0.0	0.00	7.1	0.28	0.085	3.00
21	0.0	0.00	7.1	0.28	0.085	3.00
22	15.2	0.60	7.4	0.29	0.085	3.00
23	15.2	0.60	7.6	0.30	0.085	3.00
24	0.0	0.00	7.6	0.30	0.085	3.00
25	0.0	0.00	7.6	0.30	0.057	2.00
26	15.2	0.60	7.9	0.31	0.057	2.00
27	15.2	0.60	8.1	0.32	0.057	2.00
28	0.0	0.00	8.1	0.32	0.057	2.00
29	15.2	0.60	8.4	0.33	0.057	2.00
30	0.0	0.00	8.4	0.33	0.057	2.00
31	15.2	0.60	8.6	0.34	0.064	2.25
32	0.0	0.00	8.6	0.34	0.064	2.25
33	0.0	0.00	8.6	0.34	0.065	2.30
34	0.0	0.00	8.6	0.34	0.065	2.30
35	15.2	0.60	8.9	0.35	0.064	2.25
36	0.0	0.00	8.9	0.35	0.064	2.25
37	0.0	0.00	8.9	0.35	0.057	2.00
38	0.0	0.00	8.9	0.35	0.057	2.00
39	0.0	0.00	8.9	0.35	0.057	2.00
40	0.0	0.00	8.9	0.35	0.048	1.70
41	0.0	0.00	8.9	0.35	0.048	1.70
42	0.0	0.00	8.9	0.35	0.048	1.70
43	15.2	0.60	9.1	0.36	0.048	1.70
44	0.0	0.00	9.1	0.36	0.048	1.70
45	0.0	0.00	9.1	0.36	0.031	1.10
46	0.0	0.00	9.1	0.36	0.031	1.10
47	0.0	0.00	9.1	0.36	0.028	1.00
48	0.0	0.00	9.1	0.36	0.028	1.00
49	0.0	0.00	9.1	0.36	0.028	1.00
50	0.0	0.00	9.1	0.36	0.026	0.90

Table 34 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 2, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
51	0.0	0.00	9.1	0.36	0.026	0.90
52	0.0	0.00	9.1	0.36	0.026	0.90
53	0.0	0.00	9.1	0.36	0.026	0.90
54	0.0	0.00	9.1	0.36	0.026	0.90
55	0.0	0.00	9.1	0.36	0.026	0.90
56	0.0	0.00	9.1	0.36	0.026	0.90
57	0.0	0.00	9.1	0.36	0.026	0.90
58	0.0	0.00	9.1	0.36	0.026	0.90
59	0.0	0.00	9.1	0.36	0.026	0.90
60	0.0	0.00	9.1	0.36	0.017	0.60
61	0.0	0.00	9.1	0.36	0.017	0.60
62	0.0	0.00	9.1	0.36	0.017	0.60
63	0.0	0.00	9.1	0.36	0.017	0.60
64	0.0	0.00	9.1	0.36	0.017	0.60
65	0.0	0.00	9.1	0.36	0.017	0.60
66	0.0	0.00	9.1	0.36	0.017	0.60
67	0.0	0.00	9.1	0.36	0.017	0.60
68	0.0	0.00	9.1	0.36	0.017	0.60
69	0.0	0.00	9.1	0.36	0.017	0.60
70	0.0	0.00	9.1	0.36	0.017	0.60
71	0.0	0.00	9.1	0.36	0.017	0.60
72	0.0	0.00	9.1	0.36	0.017	0.60
73	0.0	0.00	9.1	0.36	0.017	0.60
74	0.0	0.00	9.1	0.36	0.017	0.60
75	15.2	0.60	9.4	0.37	0.014	0.50
76	0.0	0.00	9.4	0.37	0.014	0.50
77	0.0	0.00	9.4	0.37	0.014	0.50
78	0.0	0.00	9.4	0.37	0.014	0.50
79	0.0	0.00	9.4	0.37	0.014	0.50
80	0.0	0.00	9.4	0.37	0.014	0.50
81	0.0	0.00	9.4	0.37	0.014	0.50
82	0.0	0.00	9.4	0.37	0.014	0.50
83	0.0	0.00	9.4	0.37	0.014	0.50
84	0.0	0.00	9.4	0.37	0.014	0.50
85	0.0	0.00	9.4	0.37	0.014	0.50
86	0.0	0.00	9.4	0.37	0.014	0.50
87	0.0	0.00	9.4	0.37	0.014	0.50
88	0.0	0.00	9.4	0.37	0.014	0.50
89	0.0	0.00	9.4	0.37	0.014	0.50
90	0.0	0.00	9.4	0.37	0.014	0.50
91	0.0	0.00	9.4	0.37	0.011	0.40
92	0.0	0.00	9.4	0.37	0.011	0.40
93	0.0	0.00	9.4	0.37	0.011	0.40
94	0.0	0.00	9.4	0.37	0.011	0.40
95	0.0	0.00	9.4	0.37	0.011	0.40
96	0.0	0.00	9.4	0.37	0.011	0.40
97	0.0	0.00	9.4	0.37	0.011	0.40
98	0.0	0.00	9.4	0.37	0.011	0.40
99	0.0	0.00	9.4	0.37	0.011	0.40
100	0.0	0.00	9.4	0.37	0.011	0.40

Table 34 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 2, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
101	0.0	0.00	9.4	0.37	0.011	0.40
102	0.0	0.00	9.4	0.37	0.011	0.40
103	0.0	0.00	9.4	0.37	0.011	0.40
104	0.0	0.00	9.4	0.37	0.011	0.40
105	0.0	0.00	9.4	0.37	0.009	0.30
106	0.0	0.00	9.4	0.37	0.009	0.30
107	0.0	0.00	9.4	0.37	0.009	0.30
108	0.0	0.00	9.4	0.37	0.009	0.30
109	0.0	0.00	9.4	0.37	0.009	0.30
110	0.0	0.00	9.4	0.37	0.009	0.30
111	0.0	0.00	9.4	0.37	0.009	0.30
112	0.0	0.00	9.4	0.37	0.009	0.30
113	0.0	0.00	9.4	0.37	0.009	0.30
114	0.0	0.00	9.4	0.37	0.009	0.30
115	15.2	0.60	9.7	0.38	0.009	0.30
116	0.0	0.00	9.7	0.38	0.009	0.30
117	0.0	0.00	9.7	0.38	0.009	0.30
118	0.0	0.00	9.7	0.38	0.009	0.30
119	0.0	0.00	9.7	0.38	0.009	0.30
120	15.2	0.60	9.9	0.39	0.009	0.30
121	0.0	0.00	10.2	0.40	0.009	0.30
122	0.0	0.00	10.2	0.40	0.009	0.30
123	0.0	0.00	10.2	0.40	0.010	0.35
124	15.2	0.60	10.4	0.41	0.011	0.40
125	0.0	0.00	10.4	0.41	0.013	0.45
126	0.0	0.00	10.4	0.41	0.013	0.45
127	15.2	0.60	10.7	0.42	0.014	0.50
128	0.0	0.00	10.7	0.42	0.016	0.55
129	0.0	0.00	10.7	0.42	0.017	0.60
130	15.2	0.60	10.9	0.43	0.020	0.70
131	15.2	0.60	11.2	0.44	0.020	0.70
132	0.0	0.00	11.2	0.44	0.020	0.70
133	15.2	0.60	11.4	0.45	0.020	0.70
134	0.0	0.00	11.4	0.45	0.020	0.70
135	0.0	0.00	11.4	0.45	0.035	1.25
136	0.0	0.00	11.4	0.45	0.037	1.30
137	0.0	0.00	11.4	0.45	0.038	1.35
138	15.2	0.60	11.7	0.46	0.043	1.50
139	15.2	0.60	11.9	0.47	0.043	1.50
140	0.0	0.00	11.9	0.47	0.043	1.50
141	0.0	0.00	11.9	0.47	0.043	1.50
142	0.0	0.00	11.9	0.47	0.043	1.50
143	15.2	0.60	12.2	0.48	0.043	1.50
144	61.0	2.40	13.2	0.52	0.043	1.50
145	45.5	1.80	14.0	0.55	0.057	2.00
146	15.2	0.60	14.2	0.56	0.057	2.00
147	15.2	0.60	14.5	0.57	0.057	2.00
148	15.2	0.60	14.7	0.58	0.057	2.00
149	15.2	0.60	15.0	0.59	0.057	2.00
150	0.0	0.00	15.0	0.59	0.127	4.50

Table 34 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 2, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
151	15.2	0.60	15.2	0.60	0.127	4.50
152	15.2	0.60	15.5	0.61	0.127	4.50
153	15.2	0.60	15.8	0.62	0.127	4.50
154	45.5	1.80	16.5	0.65	0.113	4.00
155	61.0	2.40	17.5	0.69	0.112	3.95
156	76.2	3.00	18.8	0.74	0.112	3.95
157	76.2	3.00	20.1	0.79	0.227	8.00
158	76.2	3.00	21.3	0.84	0.312	11.00
159	76.2	3.00	22.6	0.89	0.411	14.50
160	76.2	3.00	23.9	0.94	0.481	17.00
161	76.2	3.00	25.2	0.99	0.493	17.40
162	61.0	2.40	26.2	1.03	0.487	17.20
163	61.0	2.40	27.2	1.07	0.481	17.00
164	45.5	1.80	27.9	1.10	0.459	16.20
165	45.5	1.80	28.7	1.13	0.439	15.50
166	45.5	1.80	29.5	1.16	0.425	15.00
167	30.5	1.20	30.0	1.18	0.411	14.50
168	30.5	1.20	30.5	1.20	0.402	14.20
169	30.5	1.20	31.0	1.22	0.397	14.00
170	30.5	1.20	31.5	1.24	0.391	13.80
171	30.5	1.20	32.0	1.26	0.374	13.20
172	15.2	0.60	32.3	1.27	0.363	12.80
173	15.2	0.60	32.5	1.28	0.340	12.00
174	15.2	0.60	32.8	1.29	0.326	11.50
175	30.5	1.20	33.3	1.31	0.312	11.00
176	30.5	1.20	33.8	1.33	0.297	10.50
177	30.5	1.20	34.3	1.35	0.283	10.00
178	15.2	0.60	34.5	1.36	0.269	9.50
179	15.2	0.60	34.8	1.37	0.268	9.45
180	15.2	0.60	35.1	1.38	0.297	10.50
181	15.2	0.60	35.3	1.39	0.334	11.80
182	0.0	0.00	35.3	1.39	0.340	12.00
183	15.2	0.60	35.6	1.40	Measuring	
184	15.2	0.60	35.8	1.41	flume flooded;	
185	30.5	1.20	36.3	1.43	no runoff	
186	76.2	3.00	37.6	1.48	record for	
187	76.2	3.00	38.9	1.53	remainder of	
188	61.0	2.40	39.9	1.57	storm.	
189	0.0	0.00	39.9	1.57		
190	15.2	0.60	40.1	1.58		
191	0.0	0.00	40.1	1.58		
192	15.2	0.60	40.4	1.59		
193	0.0	0.00	40.4	1.59		
194	0.0	0.00	40.4	1.59		
195	15.2	0.60	40.6	1.60		
196	0.0	0.00	40.6	1.60		
197	0.0	0.00	40.6	1.60		
198	0.0	0.00	40.6	1.60		
199	0.0	0.00	40.6	1.60		
200	15.2	0.60	40.9	1.61		

Table 34 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 2, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
201	0.0	0.00	40.9	1.61		
202	0.0	0.00	40.9	1.61		
203	0.0	0.00	40.9	1.61		
204	15.2	0.60	41.2	1.62		
205	0.0	0.00	41.2	1.62		
206	0.0	0.00	41.2	1.62		
207	15.2	0.60	41.4	1.63		
208	0.0	0.00	41.4	1.63		
209	0.0	0.00	41.4	1.63		
210	0.0	0.00	41.4	1.63		
211	15.2	0.60	41.6	1.64		
212	0.0	0.00	41.6	1.64		
213	0.0	0.00	41.6	1.64		
214	0.0	0.00	41.6	1.64		
215	0.0	0.00	41.6	1.64		
216	0.0	0.00	41.6	1.64		
217	0.0	0.00	41.6	1.64		
218	0.0	0.00	41.6	1.64		
219	0.0	0.00	41.6	1.64		
220	0.0	0.00	41.6	1.64		
221	0.0	0.00	41.6	1.64		
222	15.2	0.60	41.8	1.65		
223	0.0	0.00	41.8	1.65		

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 35. RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 26, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
1	91.2	3.60	1.5	0.06	0.007	0.25
2	61.0	2.40	2.5	0.10	0.007	0.25
3	15.2	0.60	2.8	0.11	0.007	0.25
4	30.5	1.20	3.3	0.13	0.014	0.50
5	0.0	0.00	3.3	0.13	0.028	1.00
6	15.2	0.60	3.6	0.14	0.028	1.00
7	0.0	0.00	3.6	0.14	0.028	1.00
8	0.0	0.00	3.6	0.14	0.028	1.00
9	0.0	0.00	3.6	0.14	0.028	1.00
10	0.0	0.00	3.6	0.14	0.062	2.20
11	0.0	0.00	3.6	0.14	0.071	2.50
12	15.2	0.60	3.8	0.15	0.071	2.50
13	0.0	0.00	3.8	0.15	0.071	2.50
14	0.0	0.00	3.8	0.15	0.071	2.50
15	0.0	0.00	3.8	0.15	0.057	2.00
16	0.0	0.00	3.8	0.15	0.057	2.00
17	0.0	0.00	3.8	0.15	0.057	2.00
18	0.0	0.00	3.8	0.15	0.057	2.00
19	0.0	0.00	3.8	0.15	0.057	2.00
20	0.0	0.00	3.8	0.15	0.034	1.20
21	0.0	0.00	3.8	0.15	0.034	1.20
22	0.0	0.00	3.8	0.15	0.034	1.20
23	0.0	0.00	3.8	0.15	0.034	1.20
24	0.0	0.00	3.8	0.15	0.034	1.20
25	0.0	0.00	3.8	0.15	0.021	0.75
26	0.0	0.00	3.8	0.15	0.021	0.75
27	0.0	0.00	3.8	0.15	0.021	0.75
28	0.0	0.00	3.8	0.15	0.021	0.75
29	0.0	0.00	3.8	0.15	0.021	0.75
30	0.0	0.00	3.8	0.15	0.017	0.60
31	0.0	0.00	3.8	0.15	0.017	0.60
32	0.0	0.00	3.8	0.15	0.017	0.60
33	0.0	0.00	3.8	0.15	0.017	0.60
34	0.0	0.00	3.8	0.15	0.017	0.60
35	0.0	0.00	3.8	0.15	0.011	0.40
36	0.0	0.00	3.8	0.15	0.011	0.40
37	0.0	0.00	3.8	0.15	0.011	0.40
38	0.0	0.00	3.8	0.15	0.011	0.40
39	0.0	0.00	3.8	0.15	0.011	0.40
40	0.0	0.00	3.8	0.15	0.011	0.40
41	0.0	0.00	3.8	0.15	0.011	0.40
42	0.0	0.00	3.8	0.15	0.011	0.40
43	0.0	0.00	3.8	0.15	0.011	0.40
44	0.0	0.00	3.8	0.15	0.011	0.40
45	0.0	0.00	3.8	0.15	0.011	0.40
46	0.0	0.00	3.8	0.15	0.011	0.40
47	0.0	0.00	3.8	0.15	0.011	0.40
48	0.0	0.00	3.8	0.15	0.011	0.40
49	0.0	0.00	3.8	0.15	0.011	0.40
50	0.0	0.00	3.8	0.15	0.011	0.40

Table 35 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 26, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
51	0.0	0.00	3.8	0.15	0.011	0.40
52	0.0	0.00	3.8	0.15	0.011	0.40
53	0.0	0.00	3.8	0.15	0.011	0.40
54	0.0	0.00	3.8	0.15	0.011	0.40
55	0.0	0.00	3.8	0.15	0.011	0.40
56	0.0	0.00	3.8	0.15	0.011	0.40
57	0.0	0.00	3.8	0.15	0.011	0.40
58	0.0	0.00	3.8	0.15	0.011	0.40
59	0.0	0.00	3.8	0.15	0.011	0.40
60	0.0	0.00	3.8	0.15	0.009	0.30
61	0.0	0.00	3.8	0.15	0.009	0.30
62	0.0	0.00	3.8	0.15	0.009	0.30
63	0.0	0.00	3.8	0.15	0.009	0.30
64	0.0	0.00	3.8	0.15	0.009	0.30
65	0.0	0.00	3.8	0.15	0.009	0.30
66	0.0	0.00	3.8	0.15	0.009	0.30
67	0.0	0.00	3.8	0.15	0.009	0.30
68	0.0	0.00	3.8	0.15	0.009	0.30
69	0.0	0.00	3.8	0.15	0.009	0.30
70	0.0	0.00	3.8	0.15	0.009	0.30
71	0.0	0.00	3.8	0.15	0.009	0.30
72	0.0	0.00	3.8	0.15	0.009	0.30
73	0.0	0.00	3.8	0.15	0.009	0.30
74	0.0	0.00	3.8	0.15	0.009	0.30
75	0.0	0.00	3.8	0.15	0.009	0.30
76	0.0	0.00	3.8	0.15	0.009	0.30
77	0.0	0.00	3.8	0.15	0.009	0.30
78	0.0	0.00	3.8	0.15	0.009	0.30
79	0.0	0.00	3.8	0.15	0.009	0.30
80	0.0	0.00	3.8	0.15	0.009	0.30
81	0.0	0.00	3.8	0.15	0.009	0.30
82	0.0	0.00	3.8	0.15	0.009	0.30
83	0.0	0.00	3.8	0.15	0.009	0.30
84	0.0	0.00	3.8	0.15	0.009	0.30
85	0.0	0.00	3.8	0.15	0.009	0.30
86	0.0	0.00	3.8	0.15	0.009	0.30
87	0.0	0.00	3.8	0.15	0.009	0.30
88	0.0	0.00	3.8	0.15	0.009	0.30
89	0.0	0.00	3.8	0.15	0.009	0.30
90	0.0	0.00	3.8	0.15	0.007	0.25
91	0.0	0.00	3.8	0.15	0.007	0.25
92	0.0	0.00	3.8	0.15	0.007	0.25
93	0.0	0.00	3.8	0.15	0.007	0.25
94	0.0	0.00	3.8	0.15	0.007	0.25
95	0.0	0.00	3.8	0.15	0.007	0.25
96	0.0	0.00	3.8	0.15	0.007	0.25
97	0.0	0.00	3.8	0.15	0.007	0.25
98	0.0	0.00	3.8	0.15	0.007	0.25
99	0.0	0.00	3.8	0.15	0.007	0.25
100	0.0	0.00	3.8	0.15	0.007	0.25

Table 35 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 26, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
101	0.0	0.00	3.8	0.15	0.007	0.25
102	0.0	0.00	3.8	0.15	0.007	0.25
103	0.0	0.00	3.8	0.15	0.007	0.25
104	0.0	0.00	3.8	0.15	0.007	0.25
105	0.0	0.00	3.8	0.15	0.007	0.25
106	0.0	0.00	3.8	0.15	0.007	0.25
107	0.0	0.00	3.8	0.15	0.007	0.25
108	0.0	0.00	3.8	0.15	0.007	0.25
109	0.0	0.00	3.8	0.15	0.007	0.25
110	0.0	0.00	3.8	0.15	0.007	0.25
111	0.0	0.00	3.8	0.15	0.007	0.25
112	0.0	0.00	3.8	0.15	0.007	0.25
113	0.0	0.00	3.8	0.15	0.007	0.25
114	0.0	0.00	3.8	0.15	0.007	0.25
115	0.0	0.00	3.8	0.15	0.007	0.25
116	0.0	0.00	3.8	0.15	0.007	0.25
117	0.0	0.00	3.8	0.15	0.007	0.25
118	0.0	0.00	3.8	0.15	0.007	0.25
119	0.0	0.00	3.8	0.15	0.007	0.25
120	0.0	0.00	3.8	0.15	0.007	0.25
121	0.0	0.00	3.8	0.15	0.007	0.25
122	0.0	0.00	3.8	0.15	0.007	0.25
123	0.0	0.00	3.8	0.15	0.007	0.25
124	0.0	0.00	3.8	0.15	0.007	0.25
125	0.0	0.00	3.8	0.15	0.007	0.25
126	0.0	0.00	3.8	0.15	0.007	0.25
127	0.0	0.00	3.8	0.15	0.007	0.25
128	15.2	0.60	4.1	0.16	0.007	0.25
129	0.0	0.00	4.1	0.16	0.007	0.25
130	0.0	0.00	4.1	0.16	0.007	0.25
131	0.0	0.00	4.1	0.16	0.007	0.25
132	0.0	0.00	4.1	0.16	0.007	0.25
133	0.0	0.00	4.1	0.16	0.007	0.25
134	0.0	0.00	4.1	0.16	0.007	0.25
135	0.0	0.00	4.1	0.16	0.007	0.25
136	0.0	0.00	4.1	0.16	0.007	0.25
137	0.0	0.00	4.1	0.16	0.007	0.25
138	0.0	0.00	4.1	0.16	0.007	0.25
139	0.0	0.00	4.1	0.16	0.007	0.25
140	0.0	0.00	4.1	0.16	0.007	0.25
141	0.0	0.00	4.1	0.16	0.007	0.25
142	0.0	0.00	4.1	0.16	0.007	0.25
143	0.0	0.00	4.1	0.16	0.007	0.25
144	0.0	0.00	4.1	0.16	0.007	0.25
145	0.0	0.00	4.1	0.16	0.007	0.25
146	0.0	0.00	4.1	0.16	0.007	0.25
147	0.0	0.00	4.1	0.16	0.007	0.25
148	15.2	0.60	4.3	0.17	0.007	0.25
149	0.0	0.00	4.3	0.17	0.007	0.25
150	0.0	0.00	4.3	0.17	0.007	0.25

Table 35 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 26, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
151	15.2	0.60	4.6	0.18	0.007	0.25
152	15.2	0.00	4.6	0.18	0.007	0.25
153	15.2	0.00	4.6	0.18	0.007	0.25
154	15.2	0.60	4.8	0.19	0.007	0.25
155	15.2	0.00	4.8	0.19	0.009	0.30
156	15.2	0.60	5.1	0.20	0.009	0.30
157	0.0	0.60	5.3	0.21	0.009	0.30
158	15.2	0.60	5.6	0.22	0.009	0.30
159	0.0	0.60	5.8	0.23	0.009	0.30
160	15.2	0.60	6.1	0.24	0.028	1.00
161	15.2	0.00	6.1	0.24	0.028	1.00
162	15.2	0.00	6.1	0.24	0.028	1.00
163	0.0	0.00	6.1	0.24	0.028	1.00
164	0.0	0.60	6.4	0.25	0.028	1.00
165	15.2	0.00	6.4	0.25	0.054	1.90
166	0.0	0.60	6.6	0.26	0.054	1.90
167	15.2	0.00	6.6	0.26	0.054	1.90
168	0.0	0.60	6.9	0.27	0.054	1.90
169	15.2	0.00	6.9	0.27	0.058	2.05
170	15.2	0.00	6.9	0.27	0.600	2.10
171	0.0	0.00	6.9	0.27	0.058	2.05
172	0.0	0.00	6.9	0.27	0.058	2.05
173	15.2	0.60	7.1	0.28	0.058	2.05
174	0.0	0.00	7.1	0.28	0.058	2.05
175	15.2	0.00	7.1	0.28	0.054	1.90
176	15.2	0.60	7.4	0.29	0.054	1.90
177	15.2	0.60	7.6	0.30	0.054	1.90
178	15.2	0.00	7.6	0.30	0.054	1.90
179	15.2	0.00	7.6	0.30	0.054	1.90
180	0.0	0.60	7.9	0.31	0.048	1.70
181	0.0	0.00	7.9	0.31	0.045	1.60
182	0.0	0.60	8.1	0.32	0.045	1.60
183	15.2	0.60	8.4	0.33	0.045	1.60
184	0.0	0.60	8.6	0.34	0.048	1.70
185	15.2	0.60	8.9	0.35	0.054	1.90
186	0.0	0.60	9.1	0.36	0.057	2.00
187	15.2	0.60	9.4	0.37	0.057	2.00
188	0.0	0.00	9.4	0.37	0.057	2.00
189	0.0	0.60	9.7	0.38	0.057	2.00
190	0.0	0.00	9.7	0.38	0.057	3.00
191	0.0	0.60	9.9	0.39	0.085	3.00
192	15.2	0.60	10.2	0.40	0.085	3.00
193	0.0	0.60	10.4	0.41	0.085	3.00
194	0.0	0.00	10.4	0.41	0.092	3.25
195	15.2	0.00	10.4	0.41	0.092	3.25
196	15.2	0.60	10.7	0.42	0.092	3.25
197	0.0	0.00	10.7	0.42	0.092	3.25
198	0.0	0.60	10.9	0.43	0.092	3.25
199	15.2	0.00	10.9	0.43	0.085	3.00
200	0.0	0.60	11.2	0.44	0.082	2.90

Table 35 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 26, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
201	0.0	0.60	11.4	0.45	0.082	2.90
202	0.0	0.60	11.7	0.46	0.082	2.90
203	0.0	0.60	11.9	0.47	0.078	2.75
204	0.0	0.60	12.2	0.48	0.078	2.75
205	15.2	0.60	12.5	0.49	0.082	2.90
206	0.0	0.60	12.7	0.50	0.085	3.00
207	0.0	0.60	13.0	0.51	0.085	3.00
208	0.0	0.60	13.2	0.52	0.085	3.00
209	0.0	0.60	13.5	0.53	0.085	3.00
210	15.2	0.00	13.5	0.53	0.122	4.30
211	0.0	0.00	13.5	0.53	0.122	4.30
212	0.0	0.00	13.5	0.53	0.122	4.30
213	0.0	0.60	13.7	0.54	0.122	4.30
214	15.2	0.60	14.0	0.55	0.122	4.30
215	0.0	0.00	14.0	0.55	0.099	3.50
216	15.2	0.00	14.0	0.55	0.099	3.50
217	0.0	0.00	14.0	0.55	0.099	3.50
218	0.0	0.60	14.2	0.56	0.085	3.00
219	0.0	0.00	14.2	0.56	0.085	3.00
220	15.2	0.00	14.2	0.56	0.071	2.50
221	0.0	0.00	14.2	0.56	0.071	2.50
222	15.2	0.60	14.5	0.57	0.071	2.50
223	0.0	0.00	14.5	0.57	0.071	2.50
224	15.2	0.00	14.5	0.57	0.071	2.50
225	15.2	0.00	14.5	0.57	0.051	1.80
226	15.2	0.60	14.7	0.58	0.051	1.80
227	15.2	0.00	14.7	0.58	0.051	1.80
228	15.2	0.00	14.7	0.58	0.051	1.80
229	15.2	0.00	14.7	0.58	0.051	1.80
230	15.2	0.00	14.7	0.58	0.043	1.50
231	15.2	0.60	15.0	0.59	0.043	1.50
232	15.2	0.00	15.0	0.59	0.043	1.50
233	15.2	0.00	15.0	0.59	0.043	1.50
234	0.0	0.00	15.0	0.59	0.043	1.50
235	0.0	0.00	15.0	0.59	0.043	1.50
236	0.0	0.60	15.2	0.60	0.043	1.50
237	15.2	0.00	15.2	0.60	0.043	1.50
238	15.2	0.00	15.2	0.60	0.043	1.50
239	0.0	0.00	15.2	0.60	0.043	1.50
240	0.0	0.60	15.5	0.61	0.031	1.10
241	0.0	0.00	15.5	0.61	0.031	1.10
242	15.2	0.60	15.8	0.62	0.031	1.10
243	0.0	0.00	15.8	0.62	0.031	1.10
244	0.0	0.00	15.8	0.62	0.031	1.10
245	0.0	0.00	15.8	0.62	0.037	1.30
246	15.2	0.60	16.0	0.63	0.037	1.30
247	0.0	0.00	16.0	0.63	0.037	1.30
248	0.0	0.60	16.3	0.64	0.037	1.30
249	0.0	0.00	16.3	0.64	0.037	1.30
250	15.2	0.60	16.5	0.65	0.045	1.60

Table 35 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF JULY 26, 1960 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
251	15.2	0.60	16.8	0.66	0.045	1.60
252	0.0	0.00	16.8	0.66	0.045	1.60
253	15.2	0.60	17.0	0.67	0.045	1.60
254	15.2	0.60	17.3	0.68	0.045	1.60
255	15.2	0.60	17.5	0.69	0.057	2.00
256	15.2	0.60	17.8	0.70	0.057	2.00
257	15.2	0.60	18.0	0.71	0.057	2.00
258	0.0	0.00	18.0	0.71	0.057	2.00
259	15.2	0.60	18.3	0.72	0.057	2.00
260	15.2	0.60	18.5	0.73	0.079	2.80
261	0.0	0.00	18.5	0.73	0.081	2.85
262	0.0	0.00	18.5	0.73	0.082	2.90
263	0.0	0.00	18.5	0.73	0.081	2.85
264	0.0	0.00	18.5	0.73	0.081	2.85
265	0.0	0.00	18.5	0.73	0.071	2.50
266	0.0	0.00	18.5	0.73	0.071	2.50
267	0.0	0.00	18.5	0.73	0.057	2.00
268	0.0	0.00	18.5	0.73	0.057	2.00
269	0.0	0.00	18.5	0.73	0.057	2.00
270	15.2	0.60	18.8	0.74	0.048	1.70
271	15.2	0.60	19.1	0.75	0.048	1.70
272	0.0	0.00	19.1	0.75	0.048	1.70
273	0.0	0.00	19.1	0.75	0.048	1.70
274	0.0	0.00	19.1	0.75	0.048	1.70
275	0.0	0.00	19.1	0.75	0.034	1.20
276	15.2	0.60	19.3	0.76	0.034	1.20
277	15.2	0.60	19.6	0.77	0.034	1.20
278	0.0	0.00	19.6	0.77	0.034	1.20
279	0.0	0.00	19.6	0.77	0.034	1.20
280	0.0	0.00	19.6	0.77	0.028	1.00
281	0.0	0.00	19.6	0.77	0.028	1.00
282	0.0	0.00	19.6	0.77	0.028	1.00
283	0.0	0.00	19.6	0.77	0.028	1.00
284	0.0	0.00	19.6	0.77	0.028	1.00
285	0.0	0.00	19.6	0.77	0.026	0.90
286	0.0	0.00	19.6	0.77	0.026	0.90
287	0.0	0.00	19.6	0.77	0.026	0.90
288	15.2	0.60	19.8	0.78	0.026	0.90
289	15.2	0.60	20.1	0.79	0.026	0.90
290	0.0	0.00	20.1	0.79	0.023	0.80
291	0.0	0.00	20.1	0.79	0.023	0.80
292	0.0	0.00	20.1	0.79	0.023	0.80
293	0.0	0.00	20.1	0.79	0.023	0.80
294	0.0	0.00	20.1	0.79	0.023	0.80
295	0.0	0.00	20.1	0.79	0.026	0.90
296	0.0	0.00	20.1	0.79	0.026	0.90
297	15.2	0.60	20.3	0.80	0.026	0.90
298	0.0	0.00	20.3	0.80	0.026	0.90
299	0.0	0.00	20.3	0.80	0.026	0.90
300	0.0	0.00	20.3	0.80	0.027	0.95

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 36. RAINFALL AND RUNOFF DATA FOR THE STORM OF AUGUST 2, 1963 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
1	15.2	0.60	0.3	0.01	0.007	0.25
2	15.2	0.60	0.5	0.02	0.007	0.24
3	15.2	0.60	0.8	0.03	0.007	0.23
4	15.2	0.60	1.0	0.04	0.006	0.22
5	15.2	0.60	1.3	0.05	0.006	0.20
6	15.2	0.60	1.5	0.06	0.007	0.23
7	15.2	0.60	1.8	0.07	0.007	0.26
8	30.5	1.20	2.3	0.09	0.009	0.30
9	15.2	0.60	2.5	0.10	0.014	0.50
10	45.5	1.80	3.3	0.13	0.018	0.65
11	30.5	1.20	3.8	0.15	0.031	1.10
12	30.5	1.20	4.3	0.17	0.043	1.50
13	30.5	1.20	4.8	0.19	0.057	2.00
14	30.5	1.20	5.3	0.21	0.074	2.60
15	30.5	1.20	5.8	0.23	0.102	3.60
16	15.2	0.60	6.1	0.24	0.123	4.35
17	0.0	0.00	6.1	0.24	0.136	4.80
18	0.0	0.00	6.1	0.24	0.137	4.85
19	0.0	0.00	6.1	0.24	0.133	4.70
20	15.2	0.60	6.4	0.25	0.123	4.35
21	0.0	0.00	6.4	0.25	0.116	4.10
22	0.0	0.00	6.4	0.25	0.108	3.80
23	0.0	0.00	6.4	0.25	0.092	3.25
24	0.0	0.00	6.4	0.25	0.081	2.85
25	0.0	0.00	6.4	0.25	0.071	2.50
26	0.0	0.00	6.4	0.25	0.064	2.25
27	0.0	0.00	6.4	0.25	0.057	2.00
28	0.0	0.00	6.4	0.25	0.051	1.80
29	0.0	0.00	6.4	0.25	0.047	1.65
30	0.0	0.00	6.4	0.25	0.043	1.50
31	0.0	0.00	6.4	0.25	0.041	1.45
32	0.0	0.00	6.4	0.25	0.034	1.20
33	0.0	0.00	6.4	0.25	0.030	1.05
34	0.0	0.00	6.4	0.25	0.028	1.00
35	0.0	0.00	6.4	0.25	0.027	0.95
36	0.0	0.00	6.4	0.25	0.025	0.87
37	0.0	0.00	6.4	0.25	0.023	0.82
38	0.0	0.00	6.4	0.25	0.021	0.75
39	0.0	0.00	6.4	0.25	0.020	0.70
40	0.0	0.00	6.4	0.25	0.018	0.65
41	0.0	0.00	6.4	0.25	0.015	0.52
42	0.0	0.00	6.4	0.25	0.017	0.59
43	0.0	0.00	6.4	0.25	0.016	0.56
44	0.0	0.00	6.4	0.25	0.015	0.53
45	0.0	0.00	6.4	0.25	0.014	0.50
46	0.0	0.00	6.4	0.25	0.014	0.50
47	0.0	0.00	6.4	0.25	0.014	0.50
48	0.0	0.00	6.4	0.25	0.014	0.50
49	0.0	0.00	6.4	0.25	0.014	0.50
50	0.0	0.00	6.4	0.25	0.013	0.45

Table 36 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF AUGUST 2, 1963 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities		Cumulative rain		Runoff	
	mm/hr	in./hr	mm	in.	m^3/sec	cfs
51	0.0	0.00	6.4	0.25	0.013	0.45
52	0.0	0.00	6.4	0.25	0.013	0.45
53	0.0	0.00	6.4	0.25	0.013	0.45
54	0.0	0.00	6.4	0.25	0.013	0.45
55	0.0	0.00	6.4	0.25	0.011	0.40
56	0.0	0.00	6.4	0.25	0.011	0.40
57	0.0	0.00	6.4	0.25	0.011	0.40
58	0.0	0.00	6.4	0.25	0.011	0.40
59	0.0	0.00	6.4	0.25	0.011	0.40
60	0.0	0.00	6.4	0.25	0.010	0.35
61	0.0	0.00	6.4	0.25	0.010	0.35
62	0.0	0.00	6.4	0.25	0.010	0.35
63	0.0	0.00	6.4	0.25	0.010	0.35
64	0.0	0.00	6.4	0.25	0.010	0.35
65	0.0	0.00	6.4	0.25	0.009	0.30
66	0.0	0.00	6.4	0.25	0.009	0.30
67	0.0	0.00	6.4	0.25	0.009	0.30
68	0.0	0.00	6.4	0.25	0.009	0.30
69	0.0	0.00	6.4	0.25	0.009	0.30
70	0.0	0.00	6.4	0.25	0.007	0.25
71	30.5	1.20	6.9	0.27	0.007	0.25
72	45.5	1.80	7.6	0.30	0.007	0.25
73	91.2	3.60	9.1	0.36	0.007	0.25
74	15.2	0.60	9.4	0.37	0.007	0.25
75	15.2	0.60	9.7	0.38	0.009	0.30
76	15.2	0.60	9.9	0.39	0.028	1.00
77	0.0	0.00	9.9	0.39	0.047	1.65
78	15.2	0.60	10.2	0.40	0.071	2.50
79	15.2	0.60	10.4	0.41	0.077	2.70
80	30.5	1.20	10.9	0.43	0.078	2.75
81	30.5	1.20	11.4	0.45	0.079	2.80
82	30.5	1.20	11.9	0.47	0.082	2.90
83	30.5	1.20	12.5	0.49	0.086	3.05
84	45.5	1.80	13.2	0.52	0.099	3.50
85	45.5	1.80	14.0	0.55	0.113	4.00
86	45.5	1.80	14.7	0.58	0.129	4.55
87	15.2	0.60	15.0	0.59	0.150	5.30
88	30.5	1.20	15.5	0.61	0.164	5.80
89	30.5	1.20	16.0	0.63	0.167	5.90
90	15.2	0.60	16.3	0.64	0.169	5.95
91	15.2	0.60	16.5	0.65	0.164	5.80
92	15.2	0.60	16.8	0.66	0.160	5.65
93	15.2	0.60	17.0	0.67	0.149	5.25
94	0.0	0.00	17.0	0.67	0.142	5.00
95	0.0	0.00	17.0	0.67	0.132	4.65
96	15.2	0.60	17.3	0.68	0.126	4.45
97	0.0	0.00	17.3	0.68	0.113	4.00
98					0.106	3.75
99					0.094	3.30
100					0.085	3.00

Table 36 (Continued). RAINFALL AND RUNOFF DATA FOR THE STORM OF AUGUST 2, 1963 - OAKDALE AVENUE CATCHMENT

Time from beginning of storm, minutes	Rain intensities mm/hr	Rain intensities in./hr	Cumulative rain mm	Cumulative rain in.	Runoff m^3/sec	Runoff cfs
101					0.079	2.80
102					0.072	2.55
103					0.061	2.15
104					0.058	2.05
105					0.052	1.85
106					0.050	1.75
107					0.045	1.60
108					0.040	1.40
109					0.035	1.25
110					0.033	1.15
111					0.031	1.10
112					0.030	1.05
113					0.027	0.95
114					0.026	0.90
115					0.024	0.85
116					0.023	0.80
117					0.021	0.75
118					0.020	0.70
119					0.018	0.65
120					0.019	0.67
121					0.017	0.60
122					0.016	0.57
123					0.016	0.55
124					0.015	0.52
125					0.014	0.50
126					0.014	0.50
127					0.014	0.50
128					0.014	0.50
129					0.014	0.50
130					0.013	0.45
131					0.013	0.44
132					0.012	0.42
133					0.017	0.60
134					0.021	0.75
135					0.023	0.80
136					0.021	0.75
137					0.020	0.72
138					0.018	0.65
139					0.017	0.60
140					0.016	0.55
141					0.014	0.51
142					0.014	0.48
143					0.013	0.45
144					0.012	0.43
145					0.011	0.40
146					0.011	0.40
147					0.011	0.39
148					0.011	0.39
149					0.011	0.39
150					0.011	0.38

Note: Intensities represent averages for time interval preceding indicated clock time.

Description of the Bloody Run Catchment

The Bloody Run catchment is located in the northwest section of Cincinnati, Ohio. The catchment is an urban drainage area comprised of 964 ha (2380 acres) of rolling terrain and diversified land use, drained by a combined sewerage system designed for a 10-year frequency (University of Cincinnati, 1970). Topographically, the area is characterized by two main valleys running approximately east and west. Most of the commercial and industrial sections are located in these valleys. The residential housing is found on the ridges. This watershed has an average population of about 30 persons/ha (12 persons/acre) and about 45 percent of its area is pervious. Approximately 55 percent of the area is residential; 17 percent commercial; 5 percent industrial; and 22 percent open land and parks. A more detailed division of the basin into different land uses is presented in Table 37 and Figure 15.

Sewer, topographical and zoning maps were used to divide the Bloody Run catchment into 37 subcatchments for the runoff simulations. It was intended that each area should include one major type of land use and incorporate an individual inlet manhole (University of Cincinnati, 1970). The resulting division, together with the sewer system, is shown in Figure 16. The subcatchments and the elements of the sewer system (pipes and manholes) are numbered for identification. Physical characteristics of these subcatchments are presented in Table 38. The subcatchments varied in size from 7.25 ha (17.9 acres) to 101.3 ha (250.2 acres); the imperviousness varied from 4.9 to 81.4 percent, and the ground slopes from 1.2 to 11.3 percent. Manning's roughness coefficients were assumed to be 0.013 for impervious areas and 0.250 for the pervious areas.

Table 39 indicates estimates of soil characteristics for this catchment. The maximum infiltration, the minimum infiltration and the decay rate of infiltration were assumed to be 72.2 mm/hr (3.00 in./hr), 13.2 mm/hr (0.52 in./hr) and 0.00113 sec^{-1}. The depths of retention storage were assumed to be 4.67 mm (0.184 in.) for pervious areas and 1.57 mm (0.062 in.) for impervious areas.

The Bloody Run sewer network is a combined system with a main trunk line that splits into three branches following the valleys of the drainage basin, as shown in Figure 17. The map shows the layout of all sewers with dimensions greater than 69 cm (27 in.). The sewer outfall is located at the southwestern tip of the catchment and discharges into an interceptor leading to the Mill Creek Wastwater Treatment Plant. Overflows from storms are discharged directly into Mill Creek through an open channel. Manholes are located wherever there is a significant change in pipe size, direction or slope. Each of the 37 subcatchments has its individual inlet manhole.

Table 37. DIVISION OF BLOODY RUN CATCHMENT INTO DIFFERENT LAND USES

Area number	Area ha	Area acres	Land use
0-1	15.6	38.5	
0-2	7.6	18.8	
0-3	111.3	275.0	
0-4	3.7	9.2	
0-5	14.0	34.5	Open land and parks
0-6	14.0	34.5	
0-7	55.9	138.0	
0-8	13.0	32.2	
RS-1	11.2	27.6	
RS-2	45.7	113.0	
RS-3	64.8	160.0	Residential,
RS-4	160.3	396.0	single-family housing
RS-5	87.4	216.0	
RS-6	31.6	78.0	
RS-7	55.7	137.5	
RM-1	11.2	27.6	
RM-2	34.6	85.5	Residential,
RM-3	23.3	57.5	multi-family housing
RM-4	4.7	11.5	
C-1	15.9	39.2	
C-2	15.4	38.0	
C-3	64.8	160.0	Commercial
C-4	18.6	46.0	
I-1	3.7	9.2	Industrial
I-2	32.2	79.5	
S-1	14.0	34.5	Schools
S-2	16.2	40.0	
H	15.8	39.0	Hospital
	962.2	2376.3	Total area

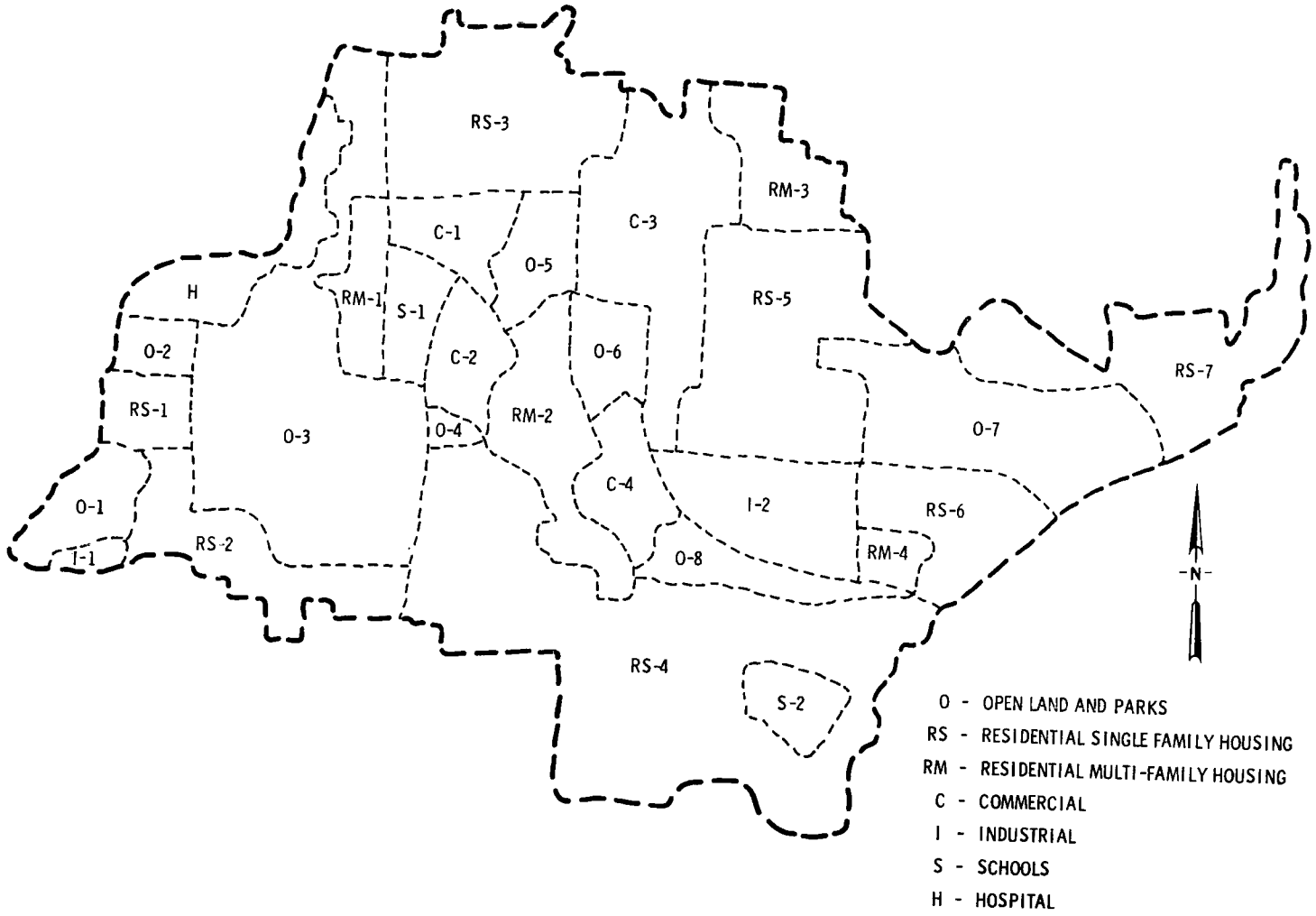


Figure 15. Map of Bloody Run catchment, Cincinnati, Ohio, showing different land uses

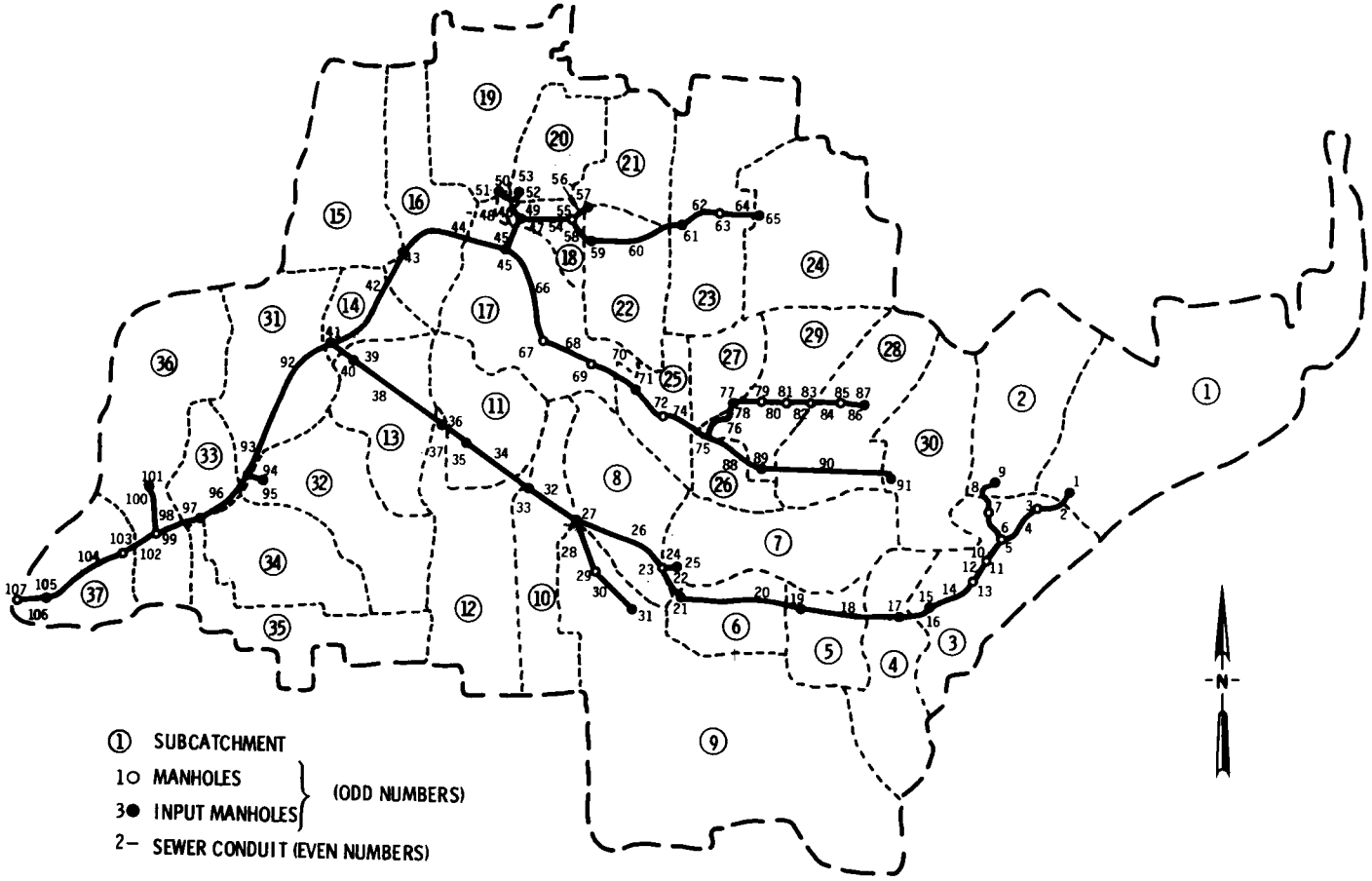


Figure 16. Map of Bloody Run catchment showing subcatchments and sewer system elements

Table 38. PHYSICAL CHARACTERISTICS OF SUBCATCHMENTS OF THE BLOODY RUN CATCHMENT

Subcatchment number	Inlet manhole number	Length m	Length ft	Width m	Width ft	Drainage area ha	Drainage area acres	Imperviousness, percent	Ground slope, percent	Manning's n imperv. area	Manning's n pervious area
1	1	1527.4	5011	466.3	1530	71.2	176.0	33.7	3.1	0.013	0.250
2	9	800.4	2626	346.9	1138	27.8	68.6	28.7	3.8	0.013	0.250
3	15	770.2	2527	384.7	1262	29.6	73.2	53.5	4.4	0.013	0.250
4	17	190.5	625	1253.3	4112	23.9	59.0	34.7	3.0	0.013	0.250
5	19	353.7	1062	474.9	1558	15.4	38.0	36.8	11.2	0.013	0.250
6	21	457.2	1500	295.7	970	13.5	33.4	38.9	10.5	0.013	0.250
7	25	934.8	3067	298.7	980	27.9	69.0	73.3	3.2	0.013	0.250
8	27	419.1	1375	518.5	1701	21.7	53.7	56.8	9.0	0.013	0.250
9	31	1390.5	4562	728.2	2389	101.3	250.2	51.9	4.0	0.013	0.250
10	33	228.6	750	800.1	2625	18.3	45.2	58.5	2.0	0.013	0.250
11	35	342.9	1125	453.2	1487	15.5	38.4	81.4	4.3	0.013	0.250
12	37	419.1	1375	789.7	2591	33.1	81.8	54.5	3.2	0.013	0.250
13	39	419.1	1375	464.2	1523	20.2	49.8	10.8	5.0	0.013	0.250
14	41	705.9	2316	115.8	380	8.2	20.2	34.7	3.5	0.013	0.250
15	43	819.3	2688	355.7	1167	29.1	72.0	20.7	3.5	0.013	0.250
16	43	438.9	1440	1069.9	3510	46.9	116.0	45.6	3.7	0.013	0.250
17	45	757.7	2486	487.1	1598	36.9	91.2	64.5	2.3	0.013	0.250
18	47	342.9	1125	211.2	693	7.2	17.9	24.5	4.0	0.013	0.250
19	51	381.0	1250	625.8	2053	23.8	58.9	39.8	2.2	0.013	0.250
20	53	342.9	1125	501.7	1646	17.2	42.5	39.7	1.5	0.013	0.250
21	57	266.7	875	548.0	1798	14.6	36.1	42.3	4.2	0.013	0.250
22	59	342.9	1125	470.9	1545	16.1	39.9	46.4	2.5	0.013	0.250
23	61	324.0	1063	1088.4	3571	35.3	87.1	60.8	5.7	0.013	0.250
24	65	476.1	1562	750.4	2462	35.7	88.3	38.7	6.3	0.013	0.250
25	71	476.4	1563	255.7	839	12.2	30.1	44.6	8.3	0.013	0.250
26	75	303.0	994	399.3	1310	12.1	29.9	71.7	6.4	0.013	0.250
27	77	228.6	750	387.1	1270	8.9	21.9	56.4	5.9	0.013	0.250
28	87	342.9	1125	711.7	2335	24.4	60.3	48.7	4.3	0.013	0.250
29	89	304.8	1000	637.3	2091	19.4	48.0	39.9	3.6	0.013	0.250
30	91	343.8	1128	595.6	1954	20.5	50.6	29.7	6.6	0.013	0.250
31	93	378.9	1243	705.0	2313	26.7	66.0	5.0	4.9	0.013	0.250
32	95	275.5	904	793.1	2602	21.9	54.0	4.9	2.5	0.013	0.250
33	97	254.8	836	396.2	1300	10.1	25.0	16.8	2.7	0.013	0.250
34	97	253.0	830	800.1	2625	20.3	50.1	10.9	2.3	0.013	0.250
35	97	255.4	838	990.3	3249	25.3	62.5	39.5	1.2	0.013	0.250
36	101	2316.5	7600	216.4	710	49.4	122.0	49.7	6.6	0.013	0.250
37	105	742.5	2436	266.7	875	19.8	49.0	33.8	7.1	0.013	0.250

Note: Length is in direction of flow, width is perpendicular to direction of flow, for a rectangular area of equal size as the actual subcatchment area.

Table 39. ESTIMATE OF SOIL CHARACTERISTICS FOR THE BLOODY RUN CATCHMENT

Item	Units	
	Metric	British
Pervious areas:		
Retention storage capacity	4.7 mm	0.184 in.
Maximum infiltration rate	76.2 mm/hr	3.00 in./hr
Minimum infiltration rate	13.2 mm/hr	0.52 in./hr
Infiltration decay rate	0.00115 sec^{-1}	0.00113 sec^{-1}
Impervious areas:		
Retention storage capacity	1.6 mm	0.62 in.
Maximum infiltration rate	0.0 mm/hr	0.00 in./hr
Minimum infiltration rate	0.0 mm/hr	0.00 in./hr
Infiltration decay rate	0.0 sec^{-1}	0.00 sec^{-1}

Notes: The infiltration rates are defined by Horton's infiltration equation. The retention storage capacities represent the amount of rain which has to fall before surface runoff begins.

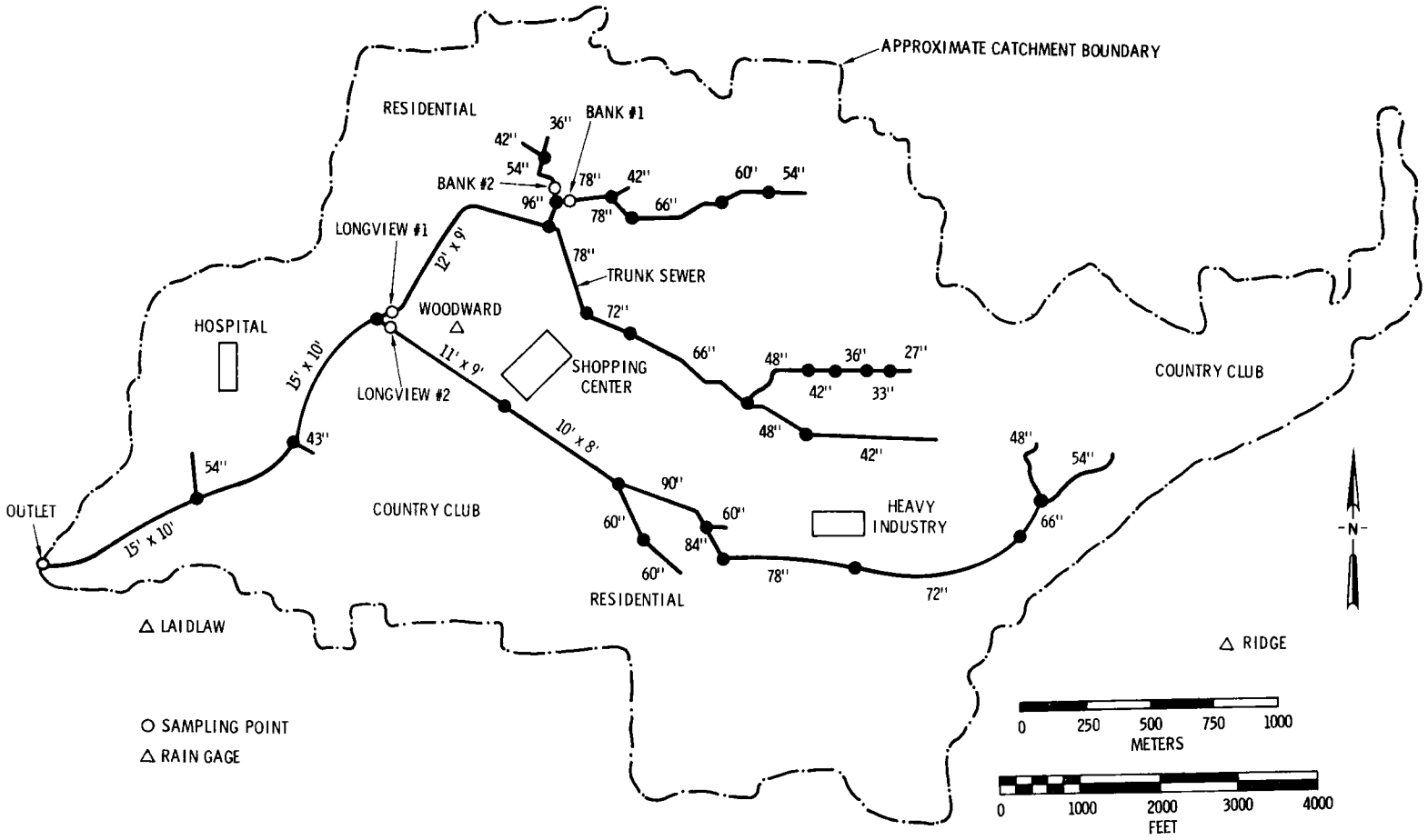


Figure 17. Map of Bloody Run catchment showing sewer layout and raingage and runoff sampling locations

The sizes, slopes and lengths of the sewer pipes are tabulated in Table 40. This sewer system utilizes circular pipes with 69 cm (27 in.) to 229 cm (90 in.) diameters, semi-circular pipes with widths and heights, respectively, of 3.05 m (10 ft) by 2.44 m (8 ft), 3.35 m (11 ft) by 2.74 m (9 ft) and 3.66 m (12 ft) by 2.74 m (9 ft), and a rectangular pipe 4.57 m (15 ft) wide by 3.05 m (10 ft) high. The pipe slopes vary from 0.28 to 5.70 percent. Manning's roughness coefficients of these pipes were assumed to be 0.013. Cross sections were available for three of the noncircular sewers (Figure 18).

Monitoring System of the Bloody Run Catchment

Three raingages manufactured by the Belfort Instrument Company were used to measure and record automatically the accumulative mass curve of each rain. The raingages consisted of a weighing mechanism in a prebalanced collection system which caused a pen to trace changes on a chart. The charts recorded up to 152 mm (6 in.) of total rainfall. Figure 17 shows the locations of the three raingages: the Laidlaw and Ridge raingages are located immediately southwest and southeast of the study area, respectively, and the Woodward raingage is located in the center of the western half of the Bloody Run catchment.

Runoff for the Bloody Run catchment was measured at five stations along the sewer network, as shown in Figure 17. These stations, which are named the Outlet, Longview #1, Longview #2, Bank #1, and Bank #2 monitoring stations, recorded runoff over a 3-year period from 1970 through 1972. Samples for water quality analysis were taken during the stormwater runoff. The flow-measuring apparatus consisted of a compressor, a manometer, and a Taylor pressure type recorder. The recorder measured the pressure due to the depth and velocity of water flowing in the sewer by bubbling air through a long tube inserted into the water. Manning's equation was used to compute discharge from the measured pressure.

Rainfall, Runoff, and Wastewater Quality Data of the Bloody Run Catchment

Four rainstorms were selected for model testing from the records of rainfall, runoff, and wastewater quality for approximately 50 rainstorms. Detailed data for these storms have been reported by the University of Cincinnati (1970 and 1972). The four storms were selected on the basis of three major criteria: 1) existence of raingage records for all three raingages; 2) reliability of the collected data as indicated by the University of Cincinnati Urban Runoff Project; and 3) variation in hydrograph shape (intensity and duration of runoff quantity) at the Outlet monitoring station.

Table 40. PHYSICAL CHARACTERISTICS OF THE SEWER ELEMENTS OF THE BLOODY RUN CATCHMENT

Pipe number	Upstream manhole number	Diameter or height m	Diameter or height ft	Width m	Width ft	Length m	Length ft	Invert slope, percent	Manning's n
2	1	1.4	4.50			120.1	394.0	2.60	0.013
4	3	1.4	4.50			210.1	689.3	2.30	0.013
6	7	1.2	4.00			115.9	380.4	2.20	0.013
8	9	1.2	4.00			142.3	467.0	2.60	0.013
10	5	1.7	5.50			97.4	319.4	1.40	0.013
12	11	1.8	6.00			105.2	345.3	0.90	0.013
14	13	1.9	6.25			175.9	577.0	1.00	0.013
16	15	1.8	6.00			141.0	462.7	1.40	0.013
18	17	1.8	6.00			435.4	1428.5	1.80	0.013
20	19	2.0	6.50			436.5	1432.0	1.25	0.013
22	21	2.1	7.00			202.4	664.0	1.10	0.013
24	25	1.5	5.00			45.6	149.7	1.00	0.013
26	23	2.3	7.50			377.5	1238.5	1.30	0.013
28	29	1.5	5.00			228.6	750.0	1.20	0.013
30	31	1.5	5.00			213.4	700.0	1.00	0.013
32	27	2.4	8.00	3.05	10.00	213.4	700.0	0.70	0.013
34	33	2.4	8.00	3.05	10.00	274.3	900.0	0.70	0.013
36	35	2.4	8.00	3.05	10.00	133.6	438.3	0.70	0.013
38	37	2.7	9.00	3.35	11.00	449.2	1473.6	0.40	0.013
40	39	2.7	9.00	3.35	11.00	106.7	350.0	0.40	0.013
42	43	2.7	9.00	3.66	12.00	510.0	1673.1	0.30	0.013
44	45	2.7	9.00	3.66	12.00	436.9	1433.4	0.28	0.013
46	47	2.4	8.00			181.5	595.4	0.60	0.013
48	49	1.4	4.50			88.8	291.2	1.30	0.013
50	51	1.1	3.50			70.5	231.2	2.40	0.013
52	53	0.9	3.00			23.2	76.0	3.40	0.013
54	55	2.0	6.50			213.0	698.7	0.90	0.013
56	57	1.1	3.50			63.8	209.3	2.10	0.013
58	59	2.0	6.50			135.0	442.8	0.80	0.013
60	61	1.7	5.50			331.0	1086.0	1.30	0.013
62	63	1.5	5.00			141.2	463.4	1.50	0.013
64	65	1.4	4.50			144.9	475.5	1.50	0.013
66	67	2.0	6.50			381.6	1251.9	0.86	0.013
68	69	1.8	6.00			283.6	930.4	1.00	0.013
70	71	1.7	5.50			111.3	365.0	1.90	0.013
72	73	1.7	5.50			123.4	404.7	2.30	0.013
74	75	1.7	5.50			243.9	800.2	1.10	0.013
76	77	1.2	4.00			146.6	481.0	1.57	0.013
78	79	1.2	4.00			108.0	354.4	2.18	0.013
80	81	1.1	3.50			77.3	253.6	5.70	0.013
82	83	0.9	3.00			118.7	389.4	1.42	0.013
84	85	0.8	2.75			104.2	342.0	1.67	0.013
86	87	0.7	2.25			99.3	325.9	1.65	0.013
88	89	1.2	4.00			261.5	858.0	2.10	0.013
90	91	1.1	3.50			518.2	1700.0	1.50	0.013
92	41	3.0	10.00	4.6	15.00	667.5	2190.0	0.28	0.013
94	95	1.2	4.00			51.8	170.0	0.90	0.013
96	93	3.0	10.00	4.6	15.00	236.2	775.0	0.32	0.013
98	97	3.0	10.00	4.6	15.00	150.9	495.0	0.32	0.013
100	101	1.4	4.50			215.3	706.5	0.50	0.013
102	99	3.0	10.00	4.6	15.00	152.4	500.0	0.32	0.013
104	103	3.0	10.00	4.6	15.00	474.3	1556.0	0.34	0.013
106	105	3.0	10.00	4.6	15.00	101.8	334.0	0.34	0.013

Note: Diameters are for circular pipes, heights and widths for non-circular pipes.

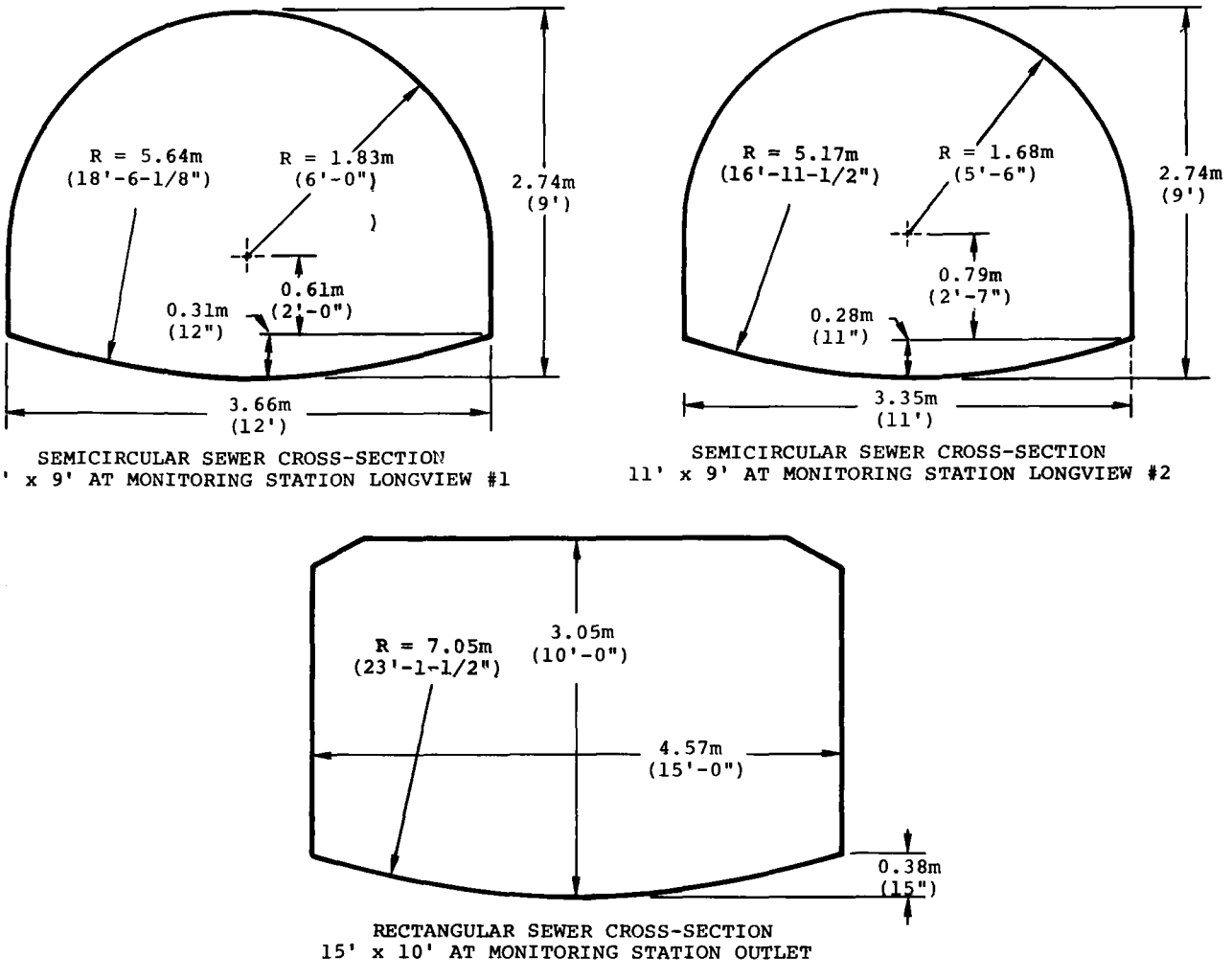


Figure 18. Cross-sections of noncircular sewers of Bloody Run catchment

The dates of the selected rainstorms are November 9, 1970; November 14, 1970; May 13, 1971; and August 25 and 26, 1971. Table 41 lists the numbers of tables and figures which present the measured rainfall, runoff and wastewater quality data for the selected storms. As indicated by the blank spaces in Table 41, not all parameters were measured at all stations for all four storms. Communication with the University of Cincinnati indicated, however, that these storms represent a fair compromise between completeness of data, reliability of data, and variety of rainfall and runoff shapes and durations.

The storm of November 9, 1970, (Figures 19-21 and Tables 42-45) had two light, short-duration rainfalls interspersed by a long dry period. The first rainfall was relatively larger than the second one, but the runoff produced by this storm shows two small peaks of about equal magnitude. Suspended solids and BOD were measured for about 1 hour during the early part of the storm. Water quality data are not available for this storm at the Outlet monitoring station.

The storm of November 14, 1970, (Figures 22-24 and Tables 46-49) had relatively light and constant rainfall over a long duration. The runoff at the Outlet monitoring station had three large peaks, with irregular forms in the early half of the storm. The three peaks were followed by lower and fairly constant runoff which gradually decreased to the dry weather discharge. Suspended solids and BOD were measured for about 1 hour during the middle of the storm period.

The storm of May 13, 1971, (Figures 25-27 and Tables 50-53) shows two rainfall periods, the first heavier than the second. The rainfall pattern is similar to the storm of November 9, 1970, but the May 13, 1971 rainfall lasted longer and produced higher intensities. The time interval between the two rainfall periods for this storm was also shorter than for the November 9, 1970 storm. The resulting runoff at the Outlet monitoring station shows one large peak during the early part of the storm and a small peak near the end of the storm. Suspended solids and BOD were measured for about 1-1/2 hours, starting approximately 1 hour after the beginning of the rainfall.

The storm of August 25 and 26, 1971, (Figures 28-30 and Tables 54-57) had short duration but very intensive rainfalls which occurred intermittently over a 10-hour period. The runoff was measured only during the first 3 or 4 hours of the storm, reflecting only the first and second rainfall bursts. The runoff at the Outlet monitoring station shows two large peaks with relatively short intervals between them. Suspended solids and BOD were measured only during the early stage of the storm.

Table 41. SUMMARY OF AVAILABLE RUNOFF RECORDS FOR RAINSTORMS OF BLOODY RUN CATCHMENT SELECTED FOR MODEL TESTING

Date of Storm	Recorded Data	Monitoring station									
		Outlet		Longview #1		Longview #2		Bank #1		Bank #2	
		Table #	Figure #	Table #	Figure #	Table #	Figure #	Table #	Figure #	Table #	Figure #
November 9, 1970	Runoff	43	19	43	19	43	19	43	19		
	Suspended solids							44	20	44	20
	BOD			45	21	45	21	45	21	45	21
November 14, 1970	Runoff	47	22	47	22	47	22	47	22		
	Suspended solids							48	23	48	23
	BOD			49	24	49	24	49	24	49	24
May 13, 1971	Runoff	51	25	51	25	51	25	51	25	51	25
	Suspended solids	52	26								
	BOD	53	27								
August 25 & 26, 1971	Runoff	55	28			55	28	55	28	55	28
	Suspended solids	56	29								
	BOD	57	30								

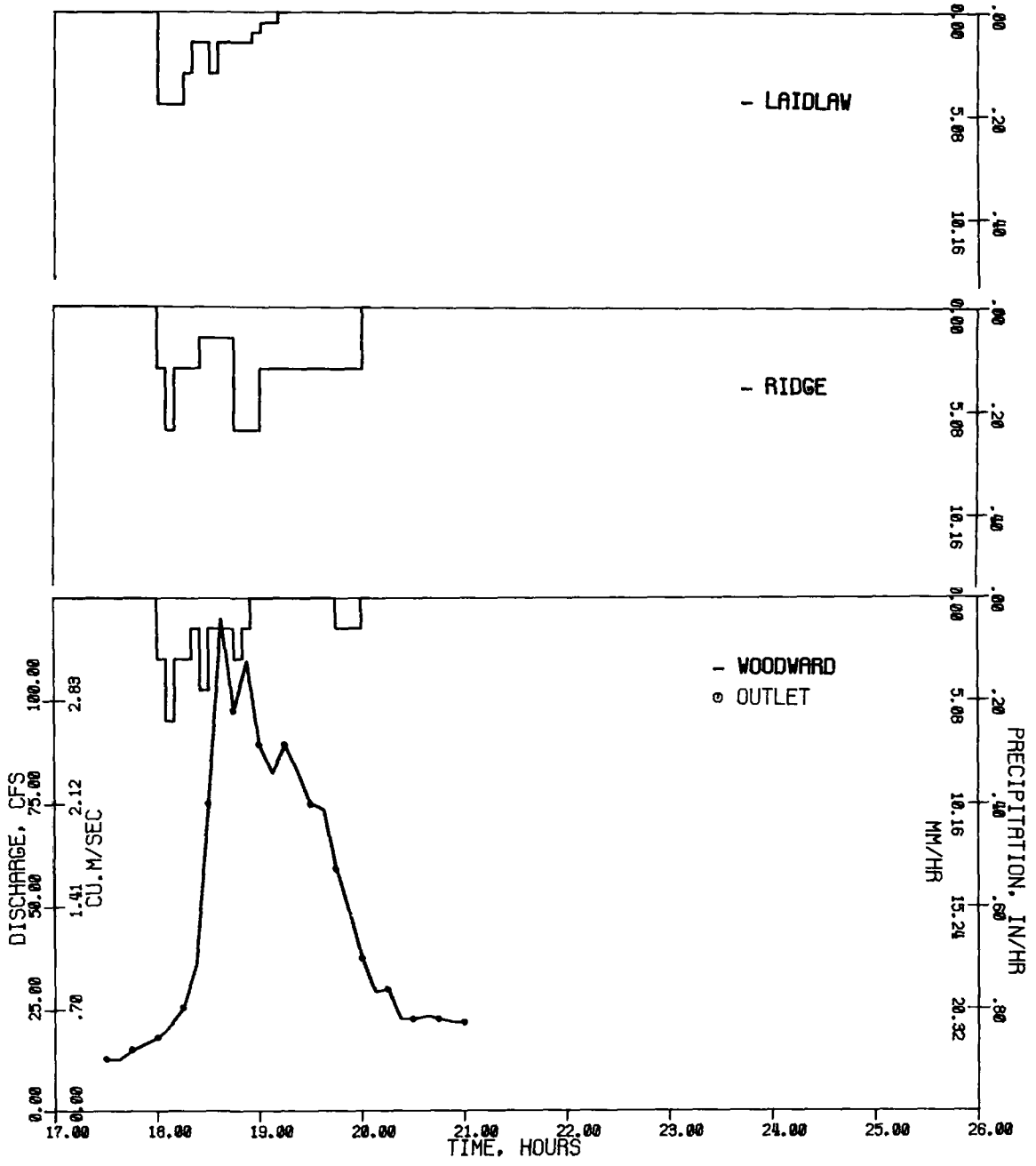


Figure 19a. Rainfall Hyetographs and Runoff Hydrographs for the Storm of November 9, 1970.- Bloody Run Catchment

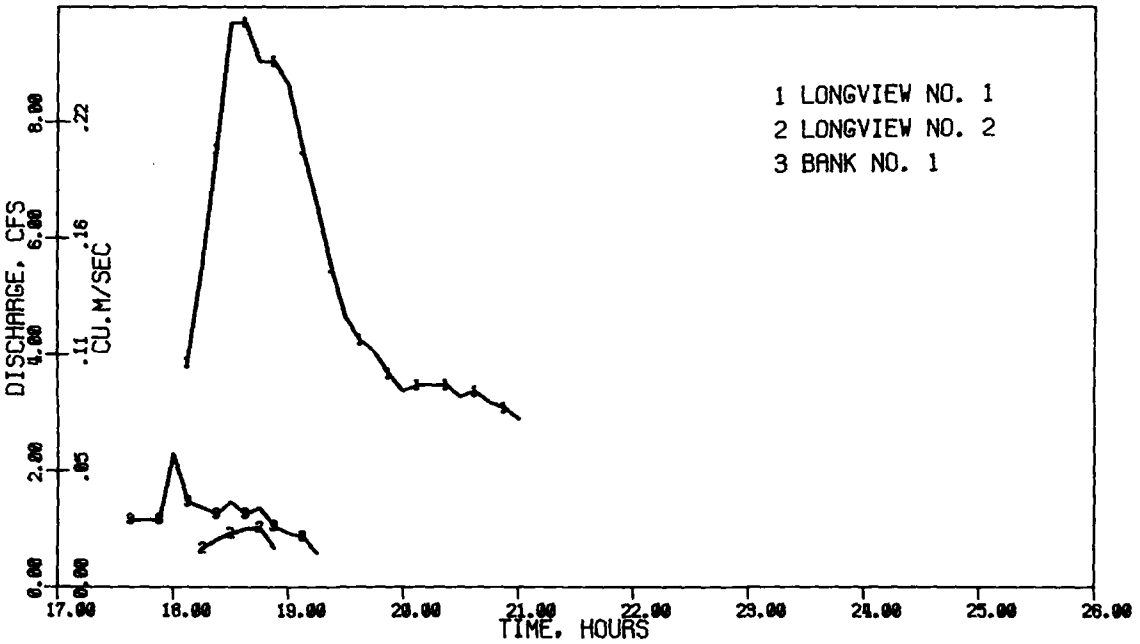


Figure 19b. Runoff Hydrographs for the Storm of November 9, 1970 - Bloody Run Catchment

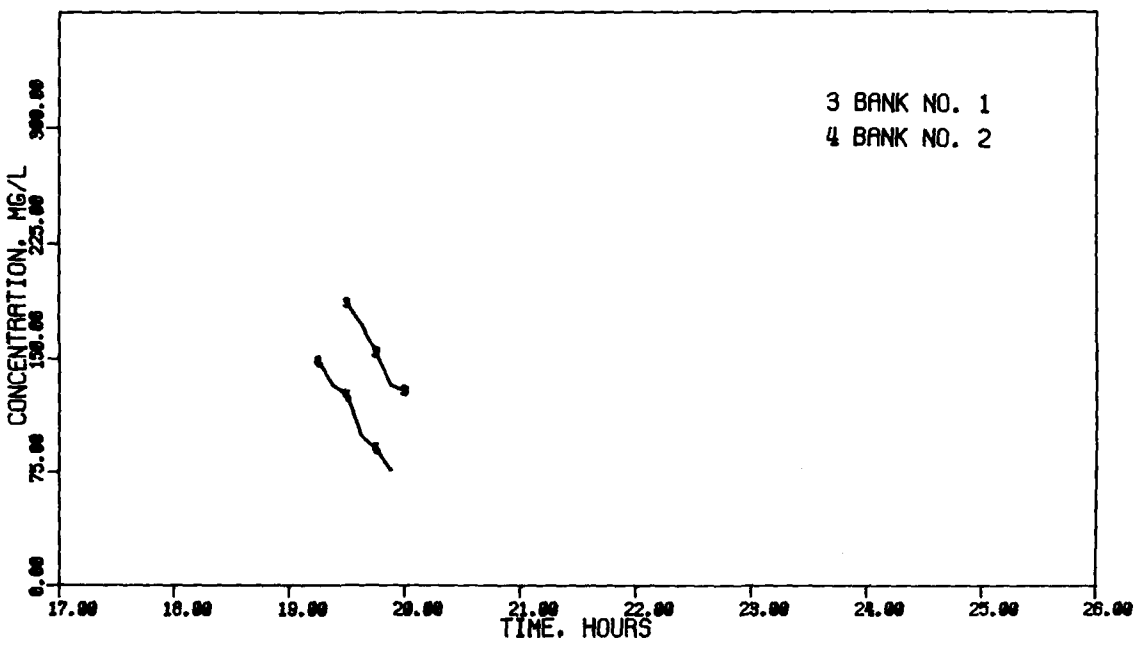


Figure 20. Suspended Solids Concentrations for the Storm Runoff of November 9, 1970 - Bloody Run Catchment

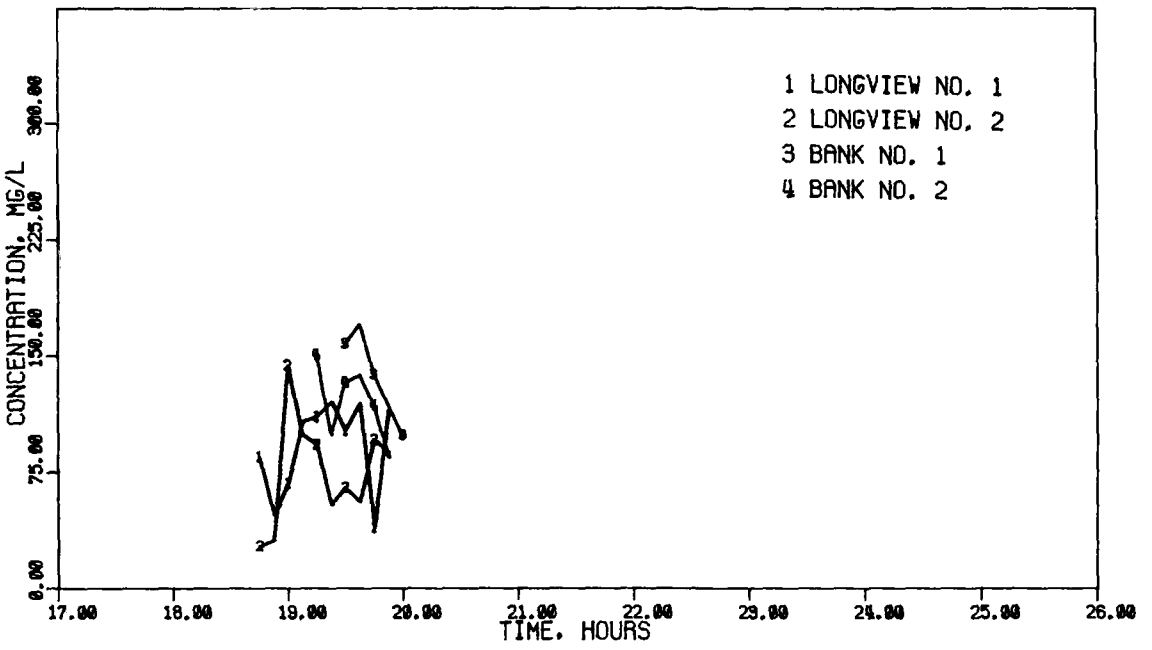


Figure 21. Biochemical Oxygen Demand Concentrations for the Storm Runoff of November 9, 1970 - Bloody Run Catchment

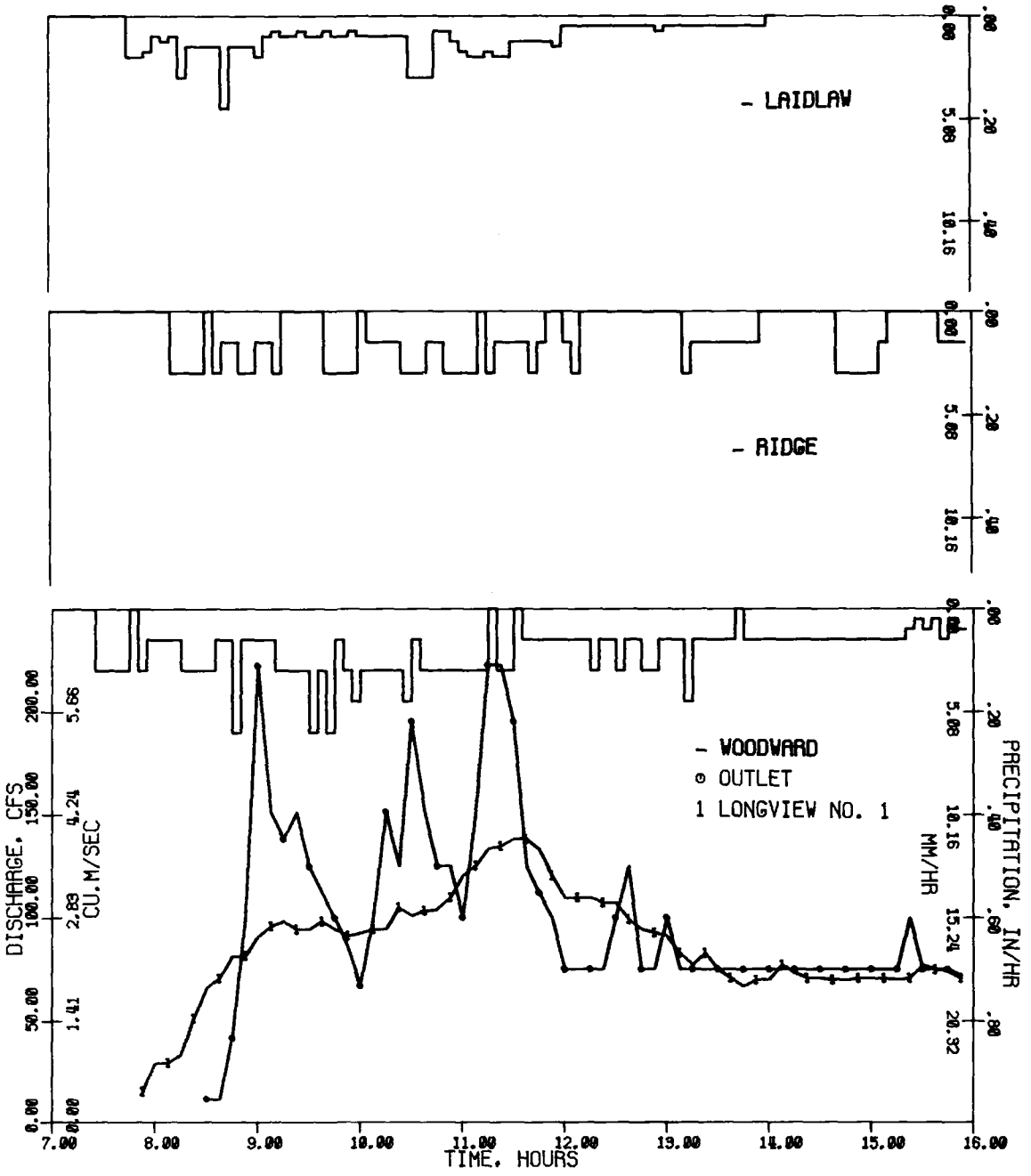


Figure 22a. Rainfall Hyetographs and Runoff Hydrographs for the Storm of November 14, 1970 - Bloody Run Catchment

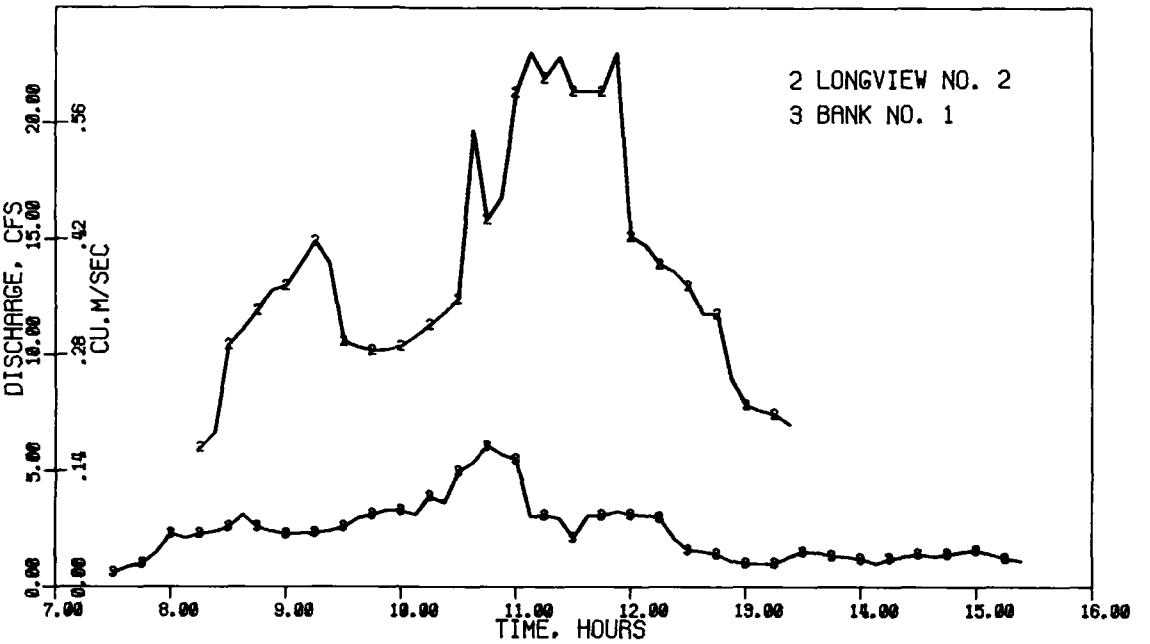


Figure 22b. Runoff Hydrographs for the Storm of November 14, 1970 - Bloody Run Catchment

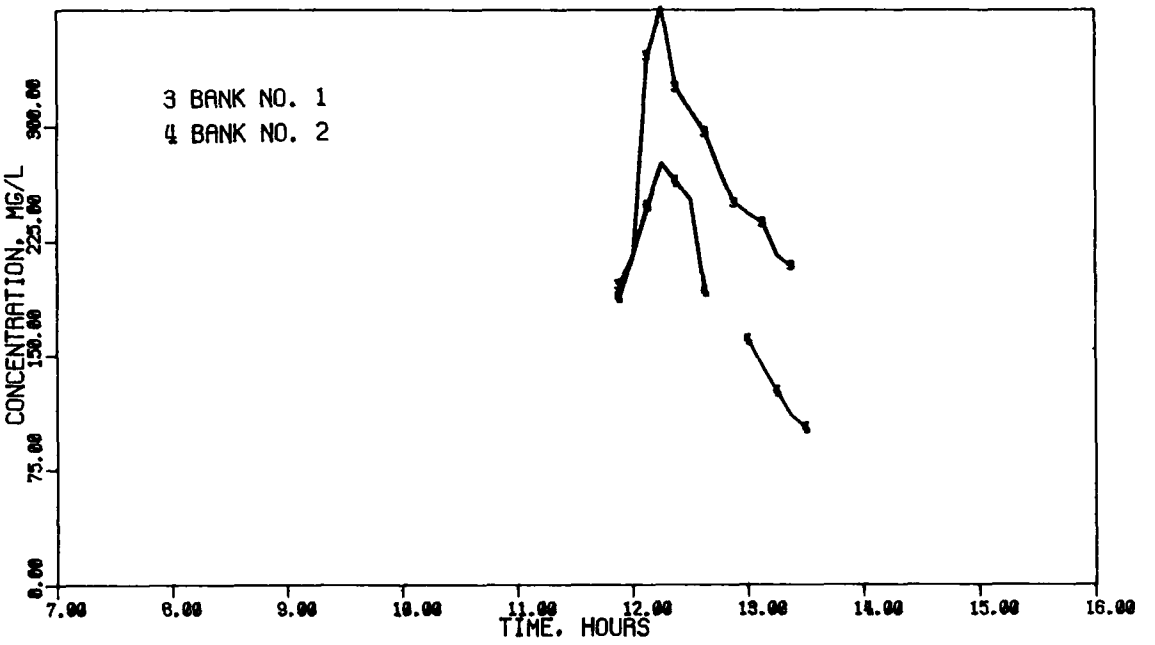


Figure 23. Suspended Solids Concentrations for the Storm Runoff of November 14, 1970 - Bloody Run Catchment

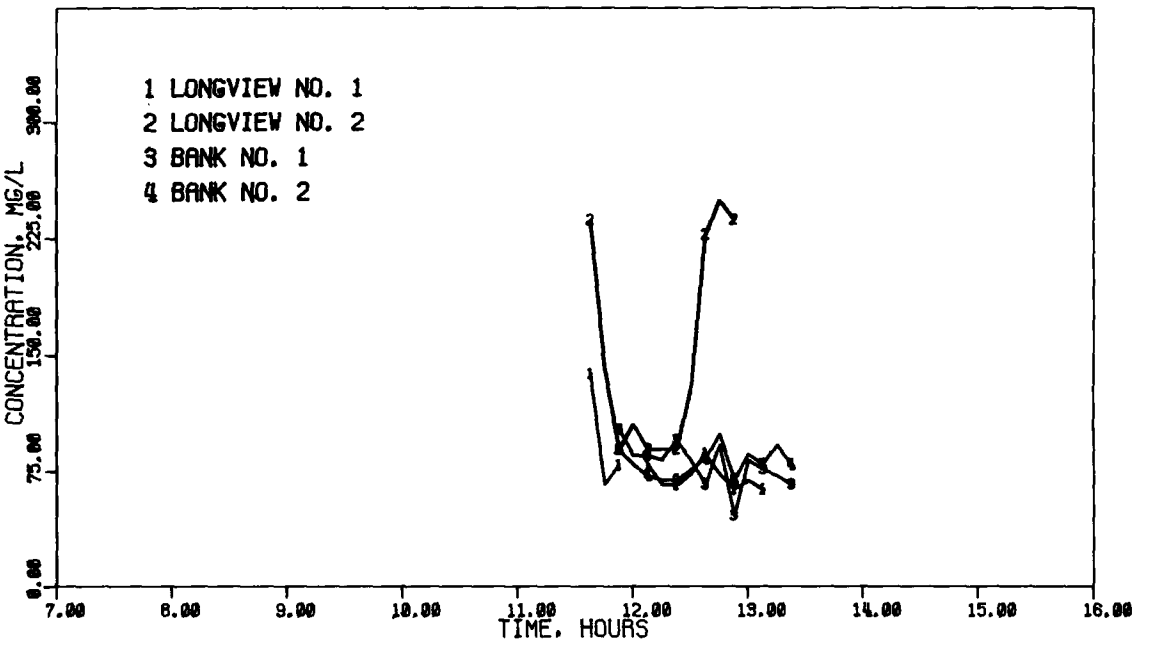


Figure 24. Biochemical Oxygen Demand Concentrations for the Storm Runoff of November 14, 1970 - Bloody Run Catchment

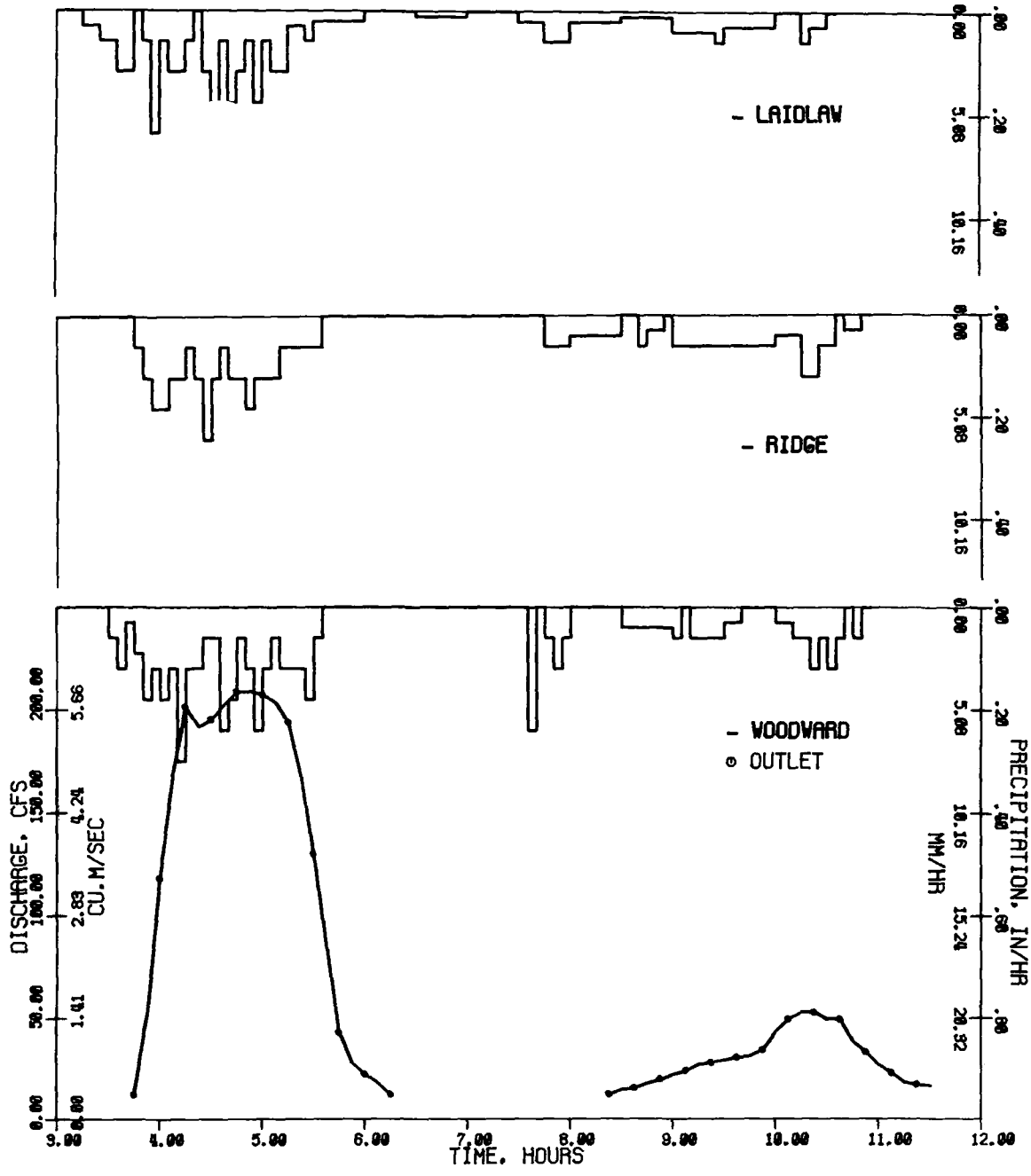


Figure 25a. Rainfall Hyetographs and Runoff Hydrographs for the Storm of May 13, 1971 - Bloody Run Catchment

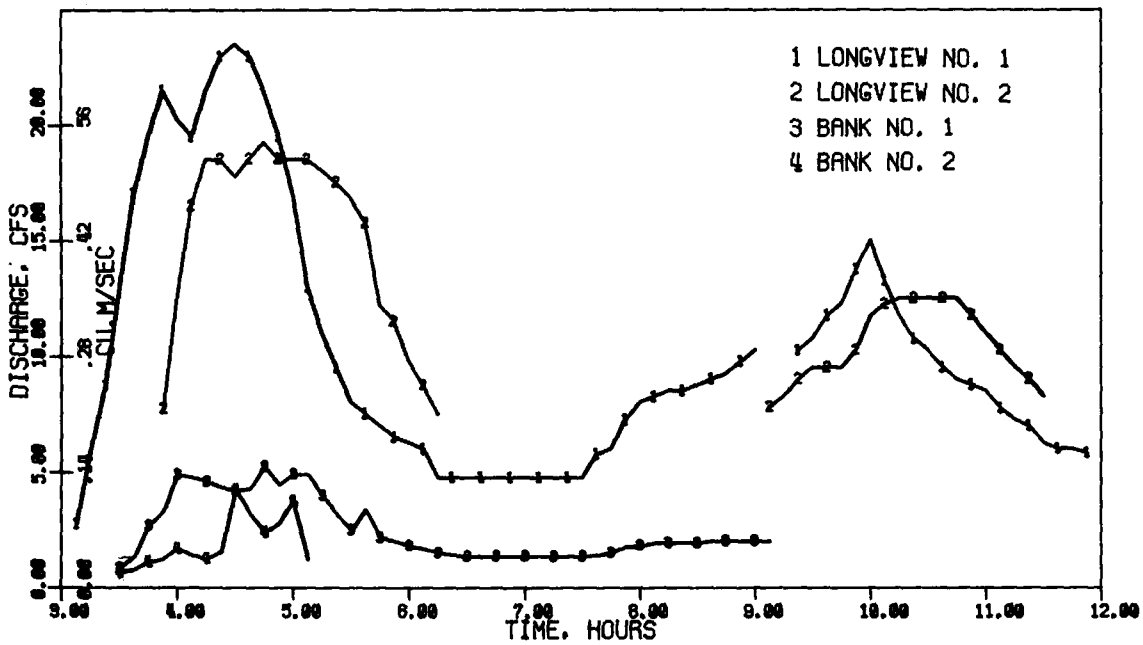


Figure 25b. Runoff Hydrographs for the Storm of May 13, 1971 - Bloody Run Catchment

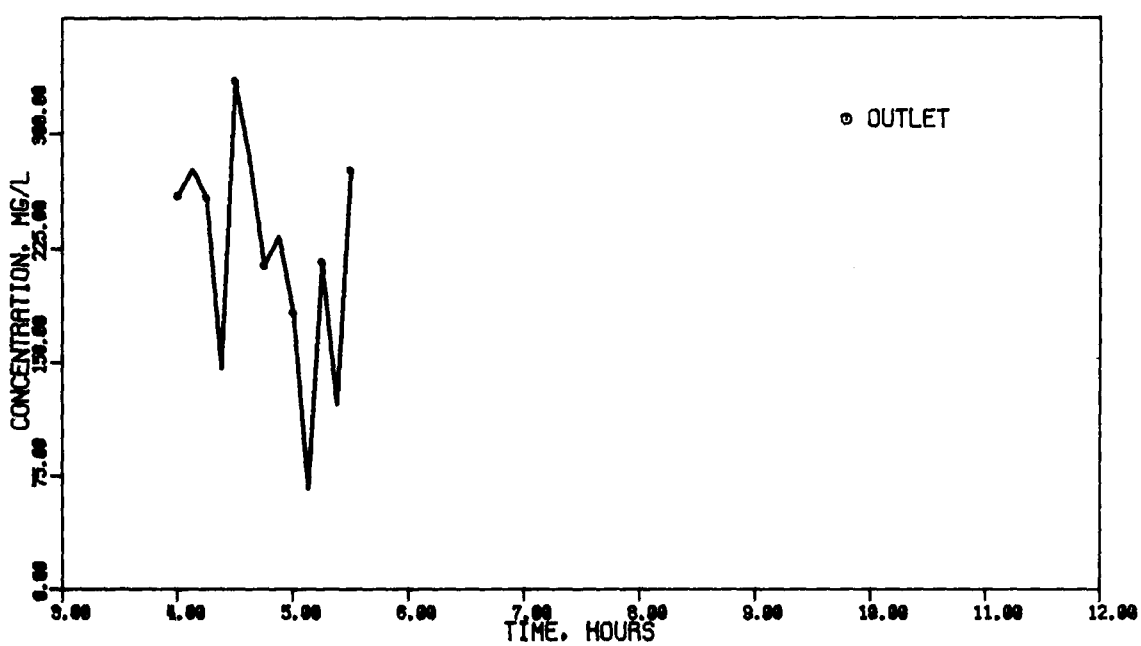


Figure 26. Suspended Solids Concentrations for the Storm Runoff of May 13, 1971 - Bloody Run Catchment

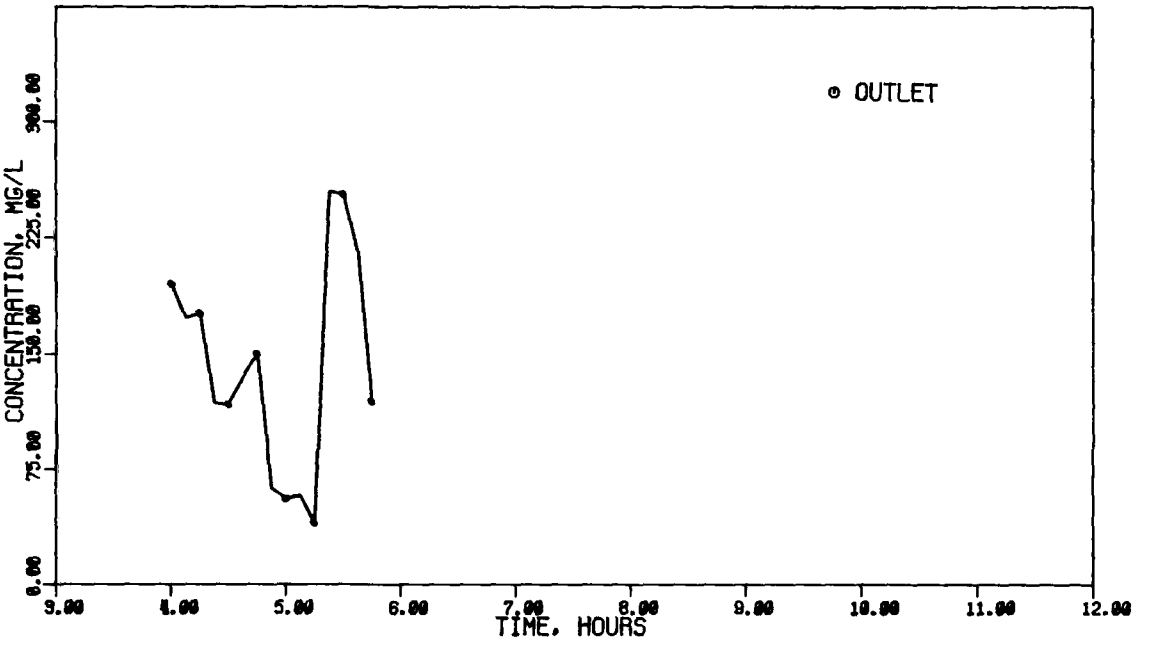


Figure 27. Biochemical Oxygen Demand Concentrations for the Storm Runoff of May 13, 1971 - Bloody Run Catchment

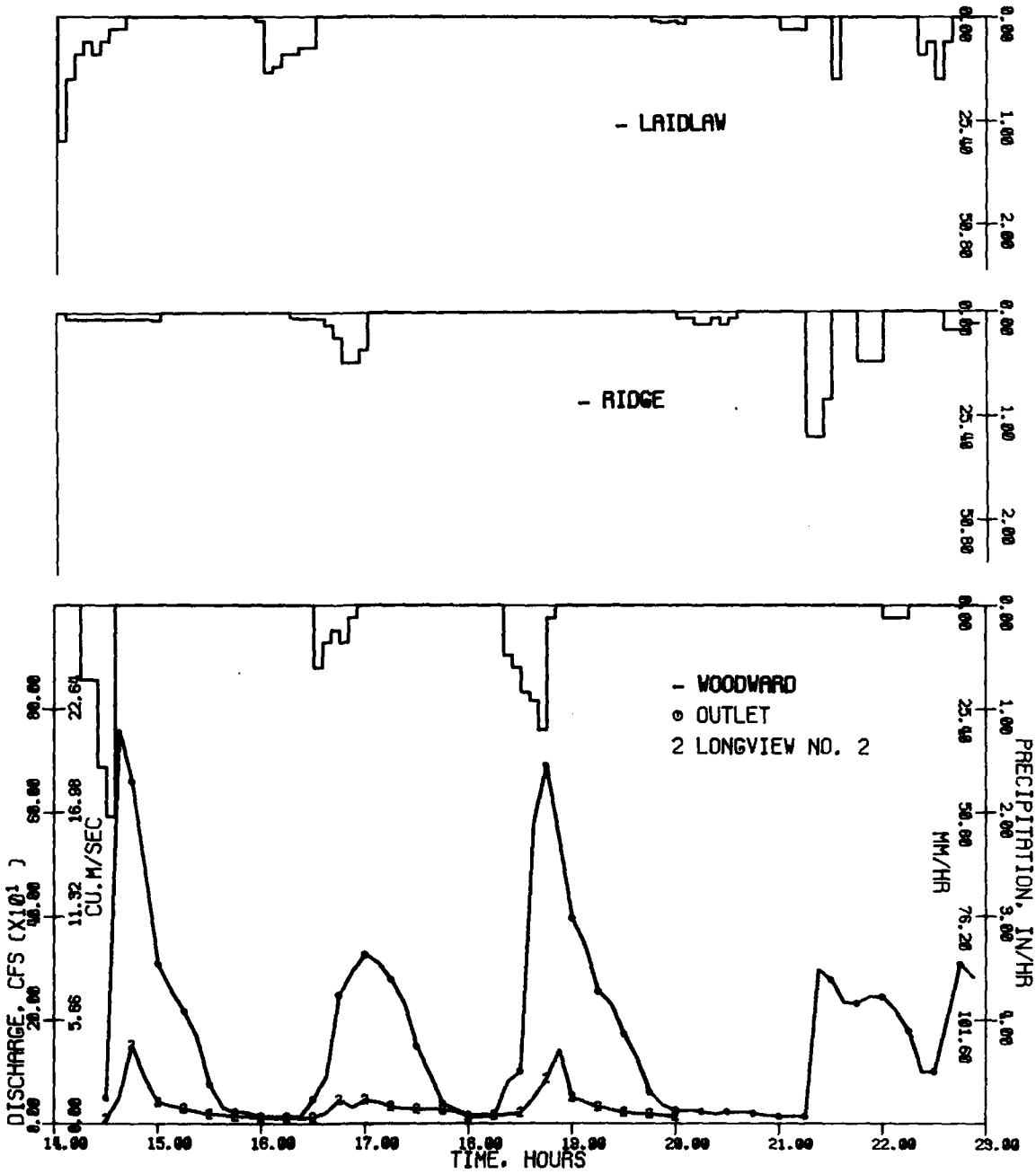


Figure 28a. Rainfall Hyetographs and Runoff Hydrographs for the Storm of August 25, 1971 - Bloody Run Catchment

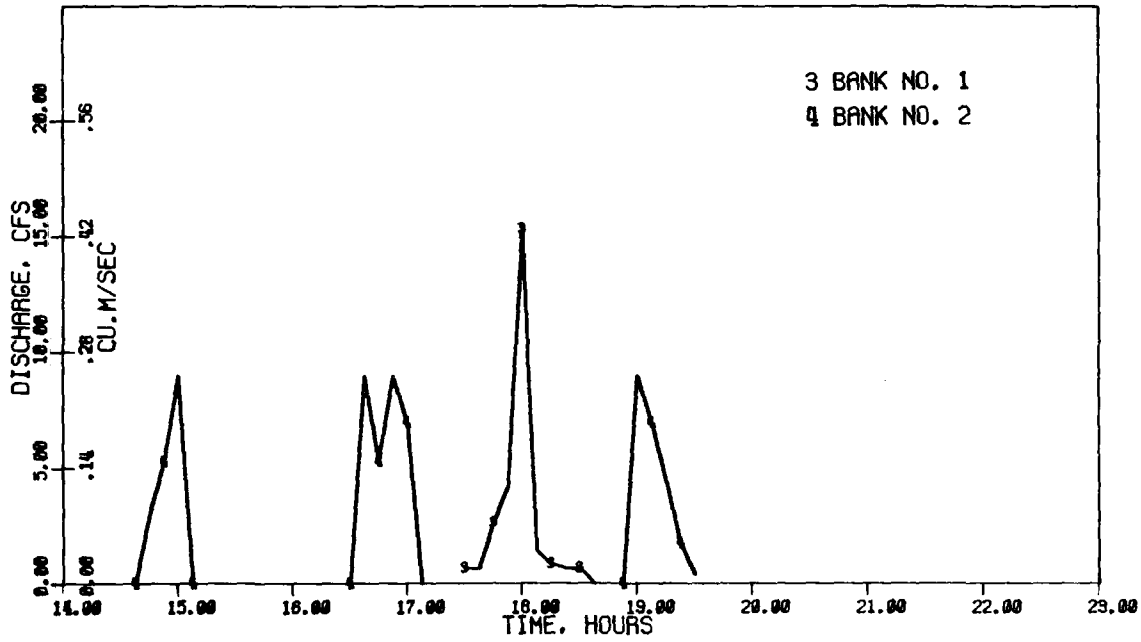


Figure 28b. Runoff Hydrographs for the Storm of August 25, 1971 - Bloody Run Catchment

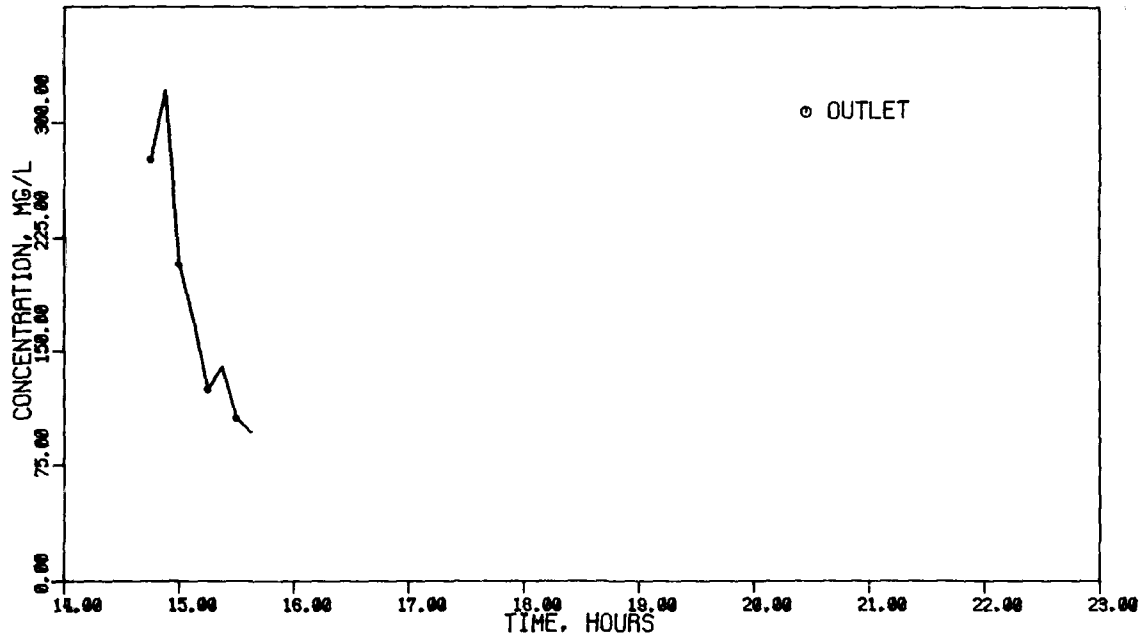


Figure 29. Suspended Solids Concentrations for the Storm Runoff of August 25, 1971 - Bloody Run Catchment

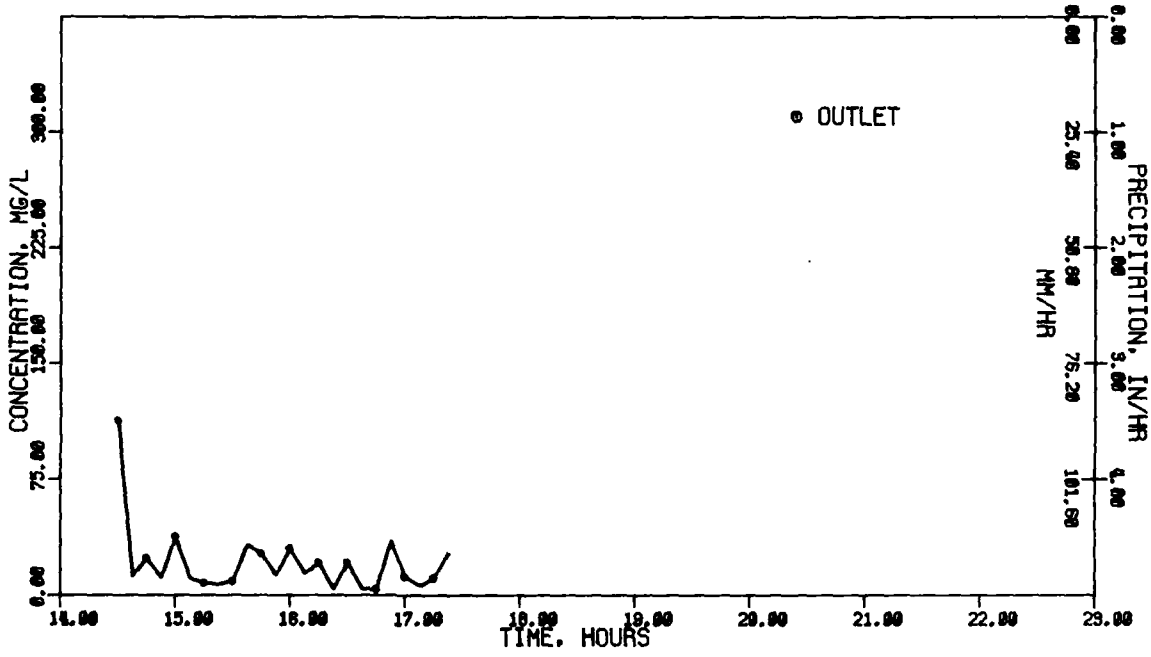


Figure 30. Biochemical Oxygen Demand Concentrations for the Storm Runoff of August 25, 1971 - Bloody Run Catchment

Table 42. RAINFALL DATA OF THE STORM OF NOVEMBER 9, 1970 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
18:00	0.0	0.00	0.0	0.00	0.0	0.00	0.0	0.00	0.0	0.00	0.0	0.00
18:05	3.0	0.12	3.0	0.12	4.6	0.18	0.3	0.01	0.3	0.01	0.4	0.02
18:10	6.1	0.24	6.1	0.24	4.6	0.18	0.8	0.03	0.8	0.03	0.8	0.03
18:15	3.0	0.12	3.0	0.12	4.6	0.18	1.0	0.04	1.0	0.04	1.1	0.05
18:20	3.0	0.12	3.0	0.12	3.0	0.12	1.3	0.05	1.3	0.05	1.4	0.06
18:25	1.5	0.06	3.0	0.12	1.5	0.06	1.4	0.06	1.5	0.06	1.5	0.06
18:30	4.6	0.18	1.5	0.06	1.5	0.06	1.8	0.07	1.7	0.07	1.7	0.07
18:35	1.5	0.06	1.5	0.06	3.0	0.12	1.9	0.08	1.8	0.07	1.9	0.08
18:40	1.5	0.06	1.5	0.06	1.5	0.06	2.0	0.08	1.9	0.08	2.0	0.08
18:45	1.5	0.06	1.5	0.06	1.5	0.06	2.2	0.09	2.0	0.08	2.2	0.09
18:50	3.0	0.12	6.1	0.24	1.5	0.06	2.4	0.10	2.5	0.10	2.3	0.09
18:55	1.5	0.06	6.1	0.24	1.5	0.06	2.5	0.10	3.0	0.12	2.4	0.10
19:00	0.0	0.00	6.1	0.24	1.0	0.04	2.5	0.10	3.6	0.14	2.5	0.10
19:05	0.0	0.00	3.0	0.12	1.0	0.02	2.5	0.10	3.8	0.15	2.5	0.10
19:10	0.0	0.00	3.0	0.12	0.5	0.02	2.5	0.10	4.1	0.16	2.6	0.10
19:15	0.0	0.00	3.0	0.12	0.0	0.00	2.5	0.10	4.3	0.17	2.6	0.10
19:20	0.0	0.00	3.0	0.12			2.5	0.10	4.6	0.18		
19:25	0.0	0.00	3.0	0.12			2.5	0.10	4.8	0.19		
19:30	0.0	0.00	3.0	0.12			2.5	0.10	5.1	0.20		
19:35	0.0	0.00	3.0	0.12			2.5	0.10	5.3	0.21		
19:40	0.0	0.00	3.0	0.12			2.5	0.10	5.6	0.22		
19:45	0.0	0.00	3.0	0.12			2.5	0.10	5.8	0.23		
19:50	1.5	0.06	3.0	0.12			2.7	0.11	6.1	0.24		
19:55	1.5	0.06	3.0	0.12			2.8	0.11	6.4	0.25		
20:00	1.5	0.06	3.0	0.12			2.9	0.12	6.6	0.26		
20:05	0.0	0.00	0.0	0.00			2.9	0.12	6.6	0.26		
20:10								0.12				

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 43. RUNOFF DATA OF THE STORM OF NOVEMBER 9, 1970 - BLOODY RUN CATCHMENT

Time	Runoff									
	Outlet		Longview #1		Longview #2		Bank #1		Bank #2	
hr:min:sec	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
17:30:00	0.361	12.75					0.033	1.15		
17:37:30	0.361	12.75					0.033	1.15		
17:45:00	0.425	15.00					0.033	1.15		
17:52:30	0.467	16.50					0.065	2.29		
18:00:00	0.510	18.00					0.041	1.46		
18:07:30	0.595	21.00	0.109	3.85			0.039	1.36		
18:15:00	0.722	25.50	0.156	5.49	0.019	0.66	0.035	1.25		
18:22:30	1.020	36.00	0.213	7.51	0.023	0.82	0.041	1.46		
18:30:00	2.124	75.00	0.275	9.72	0.026	0.92	0.035	1.25		
18:37:30	3.398	120.00	0.275	9.72	0.028	0.99	0.039	1.36		
18:45:00	2.761	97.50	0.256	9.05	0.029	1.02	0.030	1.04		
18:52:30	3.101	109.50	0.256	9.05	0.019	0.67	0.026	0.92		
19:00:00	2.528	89.25	0.245	8.66			0.024	0.86		
19:07:30	2.336	82.50	0.213	7.51			0.016	0.58		
19:15:00	2.528	89.25	0.186	6.55						
19:22:30	2.336	82.50	0.156	5.49						
19:30:00	2.124	75.00	0.131	4.62						
19:37:30	2.082	73.50	0.120	4.24						
19:45:00	1.678	59.25	0.114	4.04						
19:52:30	1.381	48.75	0.104	3.66						
20:00:00	1.062	37.50	0.095	3.37						
20:07:30	0.828	29.25	0.097	3.47						
20:15:00	0.850	30.00	0.097	3.47						
20:22:30	0.637	22.50	0.097	3.47						
20:30:00	0.637	22.50	0.093	3.27						
20:37:30	0.658	23.25	0.095	3.37						
20:45:00	0.637	22.50	0.090	3.18						
20:52:50	0.616	21.75	0.087	3.08						
21:00:00	0.616	21.75	0.082	2.89						

Note: Runoff values were scaled from figures published by the University of Cincinnati (1972). Blank spaces indicate no measurements.

Table 44. SUSPENDED SOLIDS CONCENTRATIONS OF THE STORM OF NOVEMBER 9, 1970 - BLOODY RUN CATCHMENT

	Concentrations, mg/l			
Time, hr:min:sec	Outlet	Longview #1	Longview #2	Bank #1	Bank #2
19:15:00					148
19:22:30					132
19:30:00				186	125
19:37:30				172	98
19:45:00				154	90
19:52:30				132	76
20:00:00				128	

Note: Blank spaces indicate no measurements.

Table 45. BIOCHEMICAL OXYGEN DEMAND CONCENTRATIONS OF THE STORM RUNOFF OF NOVEMBER 9, 1970 - BLOODY RUN CATCHMENT

	Concentrations, mg/l			
Time, hr:min:sec	Outlet	Longview #1	Longview #2	Bank #1	Bank #2
18:50:00		85	27		
18:57:30		47	31		
19:05:00		68	144		
19:12:30		108	99		
19:15:30					151
19:20:00		111	93		
19:22:30					99
19:27:30		121	54		
19:30:00				158	133
19:35:00		102	65		
19:37:30				171	138
19:42:30		120	56		
19:45:00				138	118
19:50:00		39	96		
19:52:30				118	85
19:57:30		115	88		
20:00:00				99	

Note: Blank spaces indicate no measurements.

Table 46. RAINFALL DATA OF THE STORM OF NOVEMBER 14, 1970 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
07:25	0.0	0.00					0.0	0.00				
07:30	3.0	0.12					0.3	0.01				
07:35	3.0	0.12					0.5	0.02				
07:40	3.0	0.12					0.8	0.03				
07:45	3.0	0.12			0.0	0.00	1.0	0.04			0.0	0.00
07:50	0.0	0.00			2.0	0.08	1.0	0.04			0.2	0.01
07:55	3.0	0.12			2.0	0.08	1.3	0.05			0.3	0.01
08:00	1.5	0.06			1.8	0.07	1.4	0.06			0.5	0.02
08:05	1.5	0.06			1.0	0.04	1.5	0.06			0.6	0.02
08:10	1.5	0.06	0.0	0.00	1.3	0.05	1.7	0.07	0.0	0.00	0.7	0.03
08:15	1.5	0.06	3.0	0.12	1.0	0.04	1.8	0.07	0.3	0.01	0.8	0.03
08:20	3.0	0.12	3.0	0.12	3.0	0.12	2.0	0.08	0.5	0.02	1.0	0.04
08:25	3.0	0.12	3.0	0.12	1.5	0.06	2.3	0.09	0.8	0.03	1.1	0.05
08:30	3.0	0.12	3.0	0.12	1.5	0.06	2.5	0.10	1.0	0.04	1.3	0.05
08:35	3.0	0.12	0.0	0.00	1.5	0.06	2.8	0.11	1.0	0.04	1.4	0.06
08:40	1.5	0.06	3.0	0.12	1.5	0.06	2.9	0.12	1.3	0.05	1.5	0.06
08:45	1.5	0.06	1.5	0.06	4.6	0.18	3.0	0.12	1.4	0.06	1.9	0.08
08:50	6.1	0.24	1.5	0.06	1.5	0.06	3.6	0.14	1.5	0.06	2.0	0.08
08:55	1.5	0.06	3.0	0.12	1.5	0.06	3.7	0.15	1.8	0.07	2.2	0.09
09:00	1.5	0.06	3.0	0.12	1.5	0.06	3.8	0.15	2.0	0.08	2.3	0.09
09:05	1.5	0.06	1.5	0.06	2.0	0.08	3.9	0.16	2.2	0.09	2.5	0.10
09:10	1.5	0.06	1.5	0.06	1.0	0.04	4.1	0.16	2.3	0.09	2.5	0.10
09:15	3.0	0.12	3.0	0.12	0.8	0.03	4.3	0.17	2.5	0.10	2.6	0.10
09:20	3.0	0.12	0.0	0.00	1.0	0.04	4.6	0.18	2.5	0.10	2.7	0.11
09:25	3.0	0.12	0.0	0.00	1.0	0.04	4.8	0.19	2.5	0.10	2.8	0.11
09:30	3.0	0.12	0.0	0.00	0.8	0.03	5.1	0.20	2.5	0.10	2.8	0.11
09:35	6.1	0.24	0.0	0.00	1.0	0.04	5.6	0.22	2.5	0.10	2.9	0.11
09:40	3.0	0.12	0.0	0.00	1.0	0.04	5.8	0.23	2.5	0.10	3.0	0.12
09:45	6.1	0.24	3.0	0.12	0.8	0.03	6.4	0.25	2.8	0.11	3.1	0.12
09:50	1.5	0.06	3.0	0.12	1.0	0.04	6.5	0.26	3.0	0.12	3.2	0.12

Table 46 (Continued). RAINFALL DATA OF THE STORM OF NOVEMBER 14, 1970 - BLOODY RUN CATCHMENT

Time,	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
hr:min	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
09:55	3.0	0.12	3.0	0.12	1.0	0.04	6.7	0.27	3.3	0.13	3.2	0.13
10:00	4.6	0.18	3.0	0.12	0.8	0.03	7.1	0.28	3.6	0.14	3.3	0.13
10:05	3.0	0.12	0.0	0.00	1.0	0.04	7.4	0.29	3.6	0.14	3.4	0.13
10:10	3.0	0.12	1.5	0.06	1.0	0.04	7.6	0.30	3.7	0.15	3.5	0.14
10:15	3.0	0.12	1.5	0.06	1.0	0.04	7.9	0.31	3.8	0.15	3.6	0.14
10:20	3.0	0.12	1.5	0.06	1.0	0.04	8.1	0.32	3.9	0.16	3.6	0.14
10:25	3.0	0.12	1.5	0.06	1.0	0.04	8.4	0.33	4.1	0.16	3.7	0.15
10:30	4.6	0.18	3.0	0.12	1.0	0.04	8.8	0.35	4.3	0.17	3.8	0.15
10:35	1.5	0.06	3.0	0.12	3.0	0.12	8.9	0.35	4.6	0.18	4.1	0.16
10:40	3.0	0.12	3.0	0.12	3.0	0.12	9.1	0.36	4.8	0.19	4.3	0.17
10:45	3.0	0.12	1.5	0.06	3.0	0.12	9.4	0.37	5.0	0.20	4.6	0.18
10:50	3.0	0.12	1.5	0.06	0.8	0.03	9.7	0.38	5.1	0.20	4.6	0.18
10:55	3.0	0.12	3.0	0.12	0.8	0.03	9.9	0.39	5.3	0.21	4.7	0.19
11:00	3.0	0.12	3.0	0.12	1.3	0.05	10.2	0.40	5.6	0.22	4.8	0.19
11:05	3.0	0.12	3.0	0.12	1.8	0.07	10.4	0.41	5.8	0.23	5.0	0.20
11:10	3.0	0.12	3.0	0.12	2.0	0.08	10.7	0.42	6.1	0.24	5.1	0.20
11:15	3.0	0.12	0.0	0.00	2.0	0.08	10.9	0.43	6.1	0.24	5.3	0.21
11:20	0.0	0.00	3.0	0.12	1.8	0.07	10.9	0.43	6.4	0.25	5.4	0.21
11:25	3.0	0.12	1.5	0.06	2.0	0.08	11.2	0.44	6.5	0.26	5.6	0.22
11:30	3.0	0.12	1.5	0.06	2.0	0.08	11.4	0.45	6.6	0.26	5.8	0.23
11:35	0.0	0.00	1.5	0.06	1.3	0.05	11.4	0.45	6.7	0.27	5.9	0.23
11:40	1.5	0.06	1.5	0.06	1.3	0.05	11.6	0.46	6.9	0.27	6.0	0.24
11:45	1.5	0.06	3.0	0.12	1.3	0.05	11.7	0.46	7.1	0.28	6.1	0.24
11:50	1.5	0.06	1.5	0.06	1.3	0.05	11.8	0.47	7.2	0.29	6.2	0.24
11:55	1.5	0.06	0.0	0.00	1.3	0.05	11.9	0.47	7.4	0.29	6.3	0.25
12:00	1.5	0.06	0.0	0.00	1.5	0.06	12.1	0.48	7.4	0.29	6.4	0.25
12:05	1.5	0.06	1.5	0.06	0.5	0.02	12.2	0.48	7.4	0.29	6.5	0.26
12:10	1.5	0.06	3.0	0.12	0.5	0.02	12.3	0.49	7.6	0.30	6.5	0.26
12:15	1.5	0.06	0.0	0.00	0.5	0.02	12.4	0.49	7.6	0.30	6.6	0.26
12:20	3.0	0.12	0.0	0.00	0.5	0.02	12.7	0.50	7.6	0.30	6.6	0.26

Table 46 (Continued). RAINFALL DATA OF THE STORM OF NOVEMBER 14, 1970 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
12:25	1.5	0.06	0.0	0.00	0.5	0.02	12.8	0.51	7.6	0.30	6.7	0.26
12:30	1.5	0.06	0.0	0.00	0.5	0.02	13.0	0.51	7.6	0.30	6.7	0.26
12:35	3.0	0.12	0.0	0.00	0.5	0.02	13.2	0.52	7.6	0.30	6.7	0.27
12:40	1.5	0.06	0.0	0.00	0.5	0.02	13.3	0.53	7.6	0.30	6.8	0.27
12:45	1.5	0.06	0.0	0.00	0.5	0.02	13.5	0.53	7.6	0.30	6.8	0.27
12:50	3.0	0.12	0.0	0.00	0.5	0.02	13.7	0.54	7.6	0.30	6.9	0.27
12:55	3.0	0.12	0.0	0.00	0.5	0.02	13.7	0.54	7.6	0.30	6.9	0.27
13:00	1.5	0.06	0.0	0.00	0.8	0.03	13.8	0.55	7.6	0.30	7.0	0.27
13:05	1.5	0.06	0.0	0.00	0.5	0.02	14.0	0.55	7.6	0.30	7.0	0.28
13:10	1.5	0.06	0.0	0.00	0.5	0.02	14.1	0.56	7.6	0.30	7.1	0.28
13:15	4.6	0.18	3.0	0.12	0.5	0.02	14.5	0.57	7.9	0.31	7.1	0.28
13:20	1.5	0.06	1.5	0.06	0.5	0.02	14.6	0.58	8.0	0.32	7.1	0.28
13:25	1.5	0.06	1.5	0.06	0.5	0.02	14.7	0.58	8.1	0.32	7.2	0.28
13:30	1.5	0.06	1.5	0.06	0.5	0.02	14.9	0.59	8.3	0.33	7.2	0.28
13:35	1.5	0.06	1.5	0.06	0.5	0.02	15.0	0.59	8.4	0.33	7.3	0.29
13:40	1.5	0.06	1.5	0.06	0.5	0.02	15.1	0.60	8.5	0.34	7.3	0.29
13:45	0.0	0.00	1.5	0.06	0.5	0.02	15.2	0.60	8.6	0.34	7.3	0.29
13:50	1.5	0.06	1.5	0.06	0.5	0.02	15.4	0.61	8.8	0.35	7.4	0.29
13:55	1.5	0.06	1.5	0.06	0.5	0.02	15.5	0.61	8.9	0.35	7.4	0.29
14:00	1.5	0.06	0.0	0.00	0.5	0.02	15.6	0.62	8.9	0.35	7.5	0.29
14:05	1.5	0.06	0.0	0.00	0.0	0.00	15.7	0.62	8.9	0.35	7.5	0.29
14:10	1.5	0.06	0.0	0.00			15.9	0.63	8.9	0.35		
14:15	1.5	0.06	0.0	0.00			16.0	0.63	8.9	0.35		
14:20	1.5	0.06	0.0	0.00			16.1	0.64	8.9	0.35		
14:25	1.5	0.06	0.0	0.00			16.3	0.64	8.9	0.35		
14:30	1.5	0.06	0.0	0.00			16.4	0.65	8.9	0.35		
14:35	1.5	0.06	0.0	0.00			16.5	0.65	8.9	0.35		
14:40	1.5	0.06	0.0	0.00			16.6	0.66	8.9	0.35		
14:45	1.5	0.06	3.0	0.12			16.8	0.66	9.1	0.36		
14:50	1.5	0.06	3.0	0.12			16.9	0.67	9.4	0.37		

Table 46 (Continued). RAINFALL DATA OF THE STORM OF NOVEMBER 14, 1970 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
14:55	1.5	0.06	3.0	0.12			17.0	0.67	9.7	0.38		
15:00	1.5	0.06	3.0	0.12			17.1	0.68	9.9	0.39		
15.05	1.5	0.06	3.0	0.12			17.3	0.68	10.2	0.40		
15:10	1.5	0.06	1.5	0.06			17.4	0.69	10.3	0.41		
15:15	1.5	0.06	0.0	0.00			17.5	0.69	10.4	0.41		
15:20	1.5	0.06	0.0	0.00			17.7	0.70	10.4	0.41		
15:25	1.0	0.04	0.0	0.00			17.8	0.70	10.4	0.41		
15:30	0.5	0.02	0.0	0.00			17.8	0.70	10.4	0.41		
15:35	1.0	0.04	0.0	0.00			17.8	0.70	10.4	0.41		
15:40	0.5	0.02	0.0	0.00			17.9	0.71	10.4	0.41		
15:45	1.5	0.06	1.5	0.06			18.0	0.71	10.4	0.41		
15:50	0.5	0.02	1.5	0.06			18.1	0.71	10.5	0.42		
15:55	1.0	0.04	1.5	0.06			18.2	0.72	10.7	0.42		
16:00	1.5	0.06	1.5	0.06			18.3	0.72	10.8	0.43		
16:05	1.5	0.06	1.5	0.06			18.4	0.73	10.9	0.43		
16:10	1.5	0.06	1.5	0.06			18.5	0.73	11.0	0.44		
16:15	1.5	0.06	1.5	0.06			18.7	0.74	11.2	0.44		
16:20	1.5	0.06	1.5	0.06			18.8	0.74	11.3	0.45		
16:25	1.3	0.05	1.5	0.06			18.9	0.74	11.4	0.45		
16:30	0.5	0.02	1.5	0.06			18.9	0.75	11.6	0.46		
16:35	0.3	0.01	0.0	0.00			19.0	0.75	11.6	0.46		
16:40	0.3	0.01					19.0	0.75				
16:45	0.3	0.01					19.0	0.75				
16:50	0.3	0.01					19.0	0.75				
16:55	0.0	0.00					19.0	0.75				

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 47. RUNOFF DATA OF THE STORM OF NOVEMBER 14, 1970 - BLOODY RUN CATCHMENT

Time	Runoff									
	Outlet		Longview #1		Longview #2		Bank #1		Bank #2	
hr:min:sec	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
07:30:00							0.017	0.61		
07:37:30							0.025	0.88		
07:45:00							0.028	1.00		
07:52:30			0.437	15.44			0.042	1.49		
08:00:00			0.828	29.25			0.065	2.29		
08:07:30			0.828	29.25			0.059	2.10		
08:15:00			0.943	33.31	0.170	6.00	0.065	2.29		
08:22:30			1.450	51.19	0.187	6.62	0.068	2.39		
08:30:00	0.332	11.71	1.864	65.81	0.295	10.42	0.073	2.59		
08:37:30	0.332	11.71	2.002	70.69	0.315	11.14	0.088	3.11		
08:45:00	1.174	41.46	2.301	81.25	0.337	11.90	0.073	2.59		
08:52:30	2.832	100.00	2.301	81.25	0.362	12.77	0.068	2.39		
09:00:00	6.302	222.56	2.577	91.00	0.367	12.96	0.064	2.26		
09:07:30	4.282	151.22	2.715	95.88	0.394	13.92	0.066	2.32		
09:15:00	3.919	138.41	2.784	98.31	0.421	14.88	0.066	2.32		
09:22:30	4.282	151.22	2.669	94.25	0.394	13.92	0.068	2.41		
09:30:00	3.540	125.00	2.669	94.25	0.299	10.56	0.073	2.59		
09:37:30	3.177	112.20	2.715	98.31	0.292	10.32	0.085	2.99		
09:45:00	2.832	100.00	2.669	94.25	0.288	10.18	0.088	3.11		
09:52:30	2.435	85.98	2.577	91.00	0.289	10.22	0.094	3.32		
10:00:00	1.899	67.07	2.623	92.62	0.294	10.37	0.093	3.29		
10:07:30	2.832	100.00	2.669	94.25	0.306	10.80	0.088	3.11		
10:15:00	4.282	151.22	2.669	94.25	0.319	11.28	0.110	3.90		
10:22:30	3.540	125.00	2.968	104.81	0.334	11.81	0.102	3.61		
10:30:00	5.525	195.12	2.853	100.75	0.351	12.38	0.141	4.99		
10:37:30	4.282	151.22	2.922	103.19	0.557	19.68	0.152	5.37		
10:45:00	3.540	125.00	2.945	104.00	0.449	15.84	0.172	6.07		
10:52:30	3.540	125.00	3.106	109.69	0.476	16.80	0.161	5.68		
11:00:00	2.832	100.00	3.405	120.25	0.603	21.31	0.155	5.48		
11:07:30	4.282	151.22	3.543	125.12	0.652	23.04	0.086	3.05		
11:15:00	6.302	222.56	3.773	133.25	0.621	21.94	0.088	3.09		
11:22:30	6.302	222.56	3.819	134.88	0.647	22.85	0.085	2.99		
11:30:00	5.525	195.12	3.911	138.12	0.605	21.36	0.059	2.10		
11:37:30	3.540	125.00	3.911	138.12	0.605	21.36	0.088	3.09		
11:45:00	3.177	112.20	3.773	133.25	0.605	21.36	0.088	3.09		
11:52:30	2.832	100.00	3.405	120.25	0.652	23.04	0.093	3.27		
12:00:00	2.124	75.00	3.106	109.69	0.428	15.12	0.088	3.11		
12:07:30	2.124	75.00	3.106	109.69	0.417	14.74	0.088	3.09		
12:15:00	2.124	75.00	3.106	109.69	0.394	13.92	0.085	2.99		
12:22:00	2.124	75.00	3.037	107.25	0.384	13.58	0.059	2.09		
12:30:00	2.832	100.00	3.037	107.25	0.367	12.96	0.045	1.59		
12:37:30	3.540	125.00	2.807	99.12	0.333	11.76	0.043	1.51		
12:45:00	2.124	75.00	2.669	94.25	0.333	11.76	0.040	1.40		
12:52:30	2.124	75.00	2.623	92.62	0.254	8.98	0.031	1.10		
13:00:00	2.832	100.00	2.577	91.00	0.221	7.82	0.028	1.00		
13:07:30	2.124	75.00	2.347	82.88	0.215	7.58	0.028	1.00		
13:15:00	2.124	75.00	2.186	77.19	0.211	7.44	0.028	1.00		
13:22:30	2.124	75.00	2.347	82.88	0.199	7.01	0.037	1.30		
13:30:00	2.124	75.00	2.117	74.75			0.043	1.51		
13:37:30	2.124	75.00	2.002	70.69			0.041	1.46		
13:45:00	2.124	75.00	1.886	66.62			0.037	1.31		
13:52:30	2.124	75.00	1.979	69.89			0.036	1.28		
14:00:00	2.124	75.00	1.979	69.89			0.034	1.19		
14:07:30	2.124	75.00	2.186	77.19			0.028	1.00		
14:15:00	2.124	75.00	2.070	73.12			0.034	1.21		
14:22:30	2.124	75.00	2.002	70.69			0.037	1.31		
14:30:00	2.124	75.00	2.002	70.69			0.040	1.40		
14:37:30	2.124	75.00	1.979	69.89			0.037	1.30		
14:45:00	2.124	75.00	1.979	69.89			0.040	1.40		
14:52:00	2.124	75.00	2.002	70.69			0.043	1.52		

Table 47 (Continued). RUNOFF DATA OF THE STORM OF NOVEMBER 14, 1970 - BLOODY RUN CATCHMENT

Time	Runoff									
	Outlet		Longview #1		Longview #2		Bank #1		Bank #2	
hr:min:sec	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
15:00:00	2.124	75.00	2.002	70.69			0.045	1.59		
15:07:30	2.124	75.00	2.002	70.69			0.040	1.40		
15:15:00	2.124	75.00	1.979	69.89			0.034	1.21		
15:22:30	2.832	100.00	2.002	70.69			0.031	1.11		
15:30:00	2.124	75.00	2.186	77.19	0.028	1.00				
15:37:30	2.124	75.00	2.117	74.75	0.028	1.00				
15:45:00	2.124	75.00	2.094	73.94	0.031	1.10				
15:52:30	2.037	71.95	2.002	70.69	0.042	1.49				
16:00:00	2.037	71.95	1.979	69.89	0.043	1.52				
16:07:30	2.003	70.73	1.933	68.25	0.037	1.31				
16:15:00	1.899	67.07	2.070	73.12	0.028	1.00				
16:22:30	1.675	59.15	2.117	74.75	0.028	1.00				
16:30:00	1.675	59.15	2.347	82.88	0.028	1.00				
16:37:30	1.675	59.15	2.324	82.06	0.028	1.00				
16:45:00	1.589	56.10	2.646	93.44	0.028	1.00				
16:52:30	1.364	48.17	2.301	81.25	0.028	1.00				
17:00:00	1.312	46.34	2.301	81.25	0.028	1.00				
17:07:30	1.247	44.02	1.864	65.81	0.054	1.91				
17:15:00	0.898	31.71	1.864	65.81	0.054	1.91				
17:22:30	0.742	26.22	1.864	65.81	0.054	1.91				
17:30:00	0.742	26.22	1.841	65.00	0.054	1.91				
17:37:30	0.691	24.39	1.815	64.19	0.023	0.80				
17:45:00	0.622	21.95	1.815	64.19	0.023	0.80				
17:52:30	0.622	21.95	1.772	62.56	0.023	0.80				
18:00:00	0.604	21.34	1.749	61.75	0.023	0.80				
18:07:30			1.702	60.12						
18:15:00			1.702	60.12						
18:22:30			1.657	58.50						
18:30:00			1.611	56.88						
18:37:30			1.565	55.25						
18:45:00			1.518	53.62						
18:52:30			1.472	52.00						
19:00:00			1.450	51.19						

Note: Runoff values were scaled from figures published by the University of Cincinnati (1972). Blank spaces indicate no measurements.

Table 48. SUSPENDED SOLIDS CONCENTRATIONS OF THE STORM RUNOFF OF NOVEMBER 14, 1970 - BLOODY RUN CATCHMENT

Time, hr:min:sec	Concentrations, mg/l Outlet	Longview #1	Longview #2	Bank #1	Bank #2
11:55:00				196	188
12:02:30				216	216
12:10:00				345	248
12:17:30				377	276
12:25:00				325	264
12:32:30				310	252
12:40:00				296	192
12:47:30				272	
12:55:00				250	160
13:02:30				243	142
13:10:00				237	126
13:17:30				216	110
13:25:00				209	102

Note: Blank spaces indicate no measurements.

Table 49. BIOCHEMICAL OXYGEN DEMAND CONCENTRATIONS OF THE STORM RUNOFF OF NOVEMBER 14, 1970 - BLOODY RUN CATCHMENT

Time, hr:min:sec	Concentrations, mg/l Outlet	Longview #1	Longview #2	Bank #1	Bank #2
11:40:00		138	237		
11:47:30		66	142		
11:55:00		79	89	102	89
12:02:30			105	85	79
12:10:00		79	89	85	72
12:17:30		66	89	82	69
12:25:00		66	89	95	69
12:32:30		73	128	82	76
12:40:00		86	227	66	83
12:47:30		73	250	92	99
12:55:00		63	237	46	69
13:02:30		69		82	86
13:10:00		63		76	79
13:17:30				72	92
13:25:00				66	79

Note: Blank spaces indicate no measurements.

Table 50. RAINFALL DATA OF THE STORM OF MAY 13, 1971 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
03:15					0.0	0.00					0.0	0.00
03:20					0.8	0.03					0.1	0.00
03:25					0.8	0.03					0.1	0.01
03:30	0.0	0.00			1.5	0.06	0.0	0.00			0.3	0.01
03:35	1.5	0.06			1.5	0.06	0.1	0.01			0.4	0.02
03:40	3.0	0.12			3.0	0.12	0.4	0.02			0.6	0.03
03:45	0.8	0.03	0.0	0.00	3.0	0.12	0.4	0.02	0.0	0.00	0.9	0.04
03:50	2.3	0.09	1.5	0.06	0.0	0.00	0.6	0.03	0.1	0.01	0.9	0.04
03:55	4.6	0.18	3.0	0.12	1.5	0.06	1.0	0.04	0.4	0.02	1.0	0.04
04:00	3.0	0.12	4.6	0.18	6.1	0.24	1.3	0.05	0.8	0.03	1.5	0.06
04:05	4.6	0.18	4.6	0.18	1.5	0.06	1.7	0.07	1.1	0.05	1.7	0.07
04:10	3.0	0.12	3.0	0.12	3.0	0.12	1.9	0.08	1.4	0.06	1.9	0.08
04:15	7.6	0.30	3.0	0.12	3.0	0.12	2.5	0.10	1.7	0.07	2.2	0.09
04:20	3.0	0.12	1.5	0.06	1.5	0.06	2.8	0.11	1.8	0.07	2.3	0.09
04:25	3.0	0.12	3.0	0.12	0.0	0.00	3.0	0.12	2.0	0.08	2.3	0.09
04:30	1.5	0.06	6.1	0.24	3.0	0.12	3.2	0.13	2.5	0.10	2.5	0.10
04:35	1.5	0.06	3.0	0.12	4.6	0.18	3.3	0.13	2.8	0.11	2.9	0.12
04:40	6.1	0.24	1.5	0.06	1.5	0.06	3.8	0.15	2.9	0.12	3.0	0.12
04:45	4.6	0.18	3.0	0.12	4.6	0.18	4.2	0.17	3.2	0.13	3.4	0.14
04:50	1.5	0.06	3.0	0.12	3.0	0.12	4.3	0.17	3.4	0.14	3.7	0.15
04:55	3.0	0.12	4.6	0.18	1.5	0.06	4.6	0.18	3.8	0.15	3.8	0.15
05:00	6.1	0.24	3.0	0.12	4.6	0.18	5.1	0.20	4.1	0.16	4.2	0.17
05:05	3.0	0.12	3.0	0.12	1.5	0.06	5.3	0.21	4.3	0.17	4.3	0.17
05:10	1.5	0.06	3.0	0.12	3.0	0.12	5.5	0.22	4.6	0.18	4.6	0.18
05:15	3.0	0.12	1.5	0.06	3.0	0.12	5.7	0.23	4.7	0.19	4.6	0.18
05:20	3.0	0.12	1.5	0.06	0.8	0.03	6.0	0.24	4.8	0.19	4.6	0.18
05:25	3.0	0.12	1.5	0.06	0.8	0.03	6.2	0.25	5.0	0.20	4.7	0.19
05:30	4.6	0.18	1.5	0.06	1.5	0.06	6.6	0.26	5.1	0.20	4.8	0.19
05:35	1.5	0.06	1.5	0.06	0.5	0.02	6.7	0.27	5.2	0.21	4.9	0.19
05:40	0.0	0.00	0.0	0.00	0.5	0.02	6.9	0.27	5.2	0.21	4.9	0.19
05:45	0.0	0.00	0.0	0.00	0.5	0.02	6.9	0.27	5.2	0.21	5.0	0.20
05:50	0.0	0.00	0.0	0.00	0.5	0.02	6.7	0.27	5.2	0.21	5.0	0.20

Table 50 (Continued). RAINFALL DATA OF THE STORM OF MAY 13, 1971 - BLOODY RUN CATCHMENT

Time,	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
hr:min	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
05:55	0.0	0.00	0.0	0.00	0.5	0.02	6.7	0.27	5.2	0.21	5.0	0.20
06:00	0.0	0.00	0.0	0.00	0.5	0.02	6.7	0.27	5.2	0.21	5.1	0.20
06:05	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.1	0.20
06:10	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.1	0.20
06:15	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.1	0.20
06:20	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.1	0.20
06:25	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.1	0.20
06:30	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.1	0.20
06:35	0.0	0.00	0.0	0.00	0.3	0.01	6.7	0.27	5.2	0.21	5.1	0.20
06:40	0.0	0.00	0.0	0.00	0.3	0.01	6.7	0.27	5.2	0.21	5.1	0.20
06:45	0.0	0.00	0.0	0.00	0.3	0.01	6.7	0.27	5.2	0.21	5.1	0.20
06:50	0.0	0.00	0.0	0.00	0.3	0.01	6.7	0.27	5.2	0.21	5.2	0.20
06:55	0.0	0.00	0.0	0.00	0.3	0.01	6.7	0.27	5.2	0.21	5.2	0.20
07:00	0.0	0.00	0.0	0.00	0.3	0.01	6.7	0.27	5.2	0.21	5.2	0.21
07:05	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.2	0.21
07:10	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.2	0.21
07:15	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.2	0.21
07:20	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.2	0.21
07:25	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.2	0.21
07:30	0.0	0.00	0.0	0.00	0.0	0.00	6.7	0.27	5.2	0.21	5.2	0.21
07:35	0.0	0.00	0.0	0.00	0.5	0.02	6.7	0.27	5.2	0.21	5.3	0.21
07:40	6.1	0.24	0.0	0.00	0.5	0.02	7.2	0.29	5.2	0.21	5.3	0.21
07:45	0.0	0.00	0.0	0.00	0.5	0.02	7.2	0.29	5.2	0.21	5.3	0.21
07:50	1.5	0.06	1.5	0.06	1.5	0.06	7.4	0.29	5.3	0.21	5.5	0.22
07:55	3.0	0.12	1.5	0.06	1.5	0.06	7.6	0.30	5.5	0.22	5.6	0.22
08:00	1.5	0.06	1.5	0.06	1.5	0.06	7.7	0.31	5.6	0.22	5.7	0.23
08:05	0.0	0.00	1.0	0.04	0.5	0.02	7.7	0.31	5.7	0.22	5.8	0.23
08:10	0.0	0.00	1.0	0.04	0.5	0.02	7.7	0.31	5.8	0.23	5.8	0.23
08:15	0.0	0.00	1.0	0.04	0.5	0.02	7.7	0.31	5.8	0.23	5.8	0.23
08:20	0.0	0.00	1.0	0.04	0.5	0.02	7.7	0.31	5.9	0.23	5.9	0.23
08:25	0.0	0.00	1.0	0.04	0.5	0.02	7.7	0.31	6.0	0.24	5.9	0.23
08:30	0.0	0.00	1.0	0.04	0.5	0.02	7.7	0.31	6.1	0.24	6.0	0.24

Table 50 (Continued). RAINFALL DATA OF THE STORM OF MAY 13, 1971 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
08:35	1.0	0.04	0.0	0.00	0.3	0.01	7.8	0.31	6.1	0.24	6.0	0.24
08:40	1.0	0.04	0.0	0.00	0.3	0.01	7.9	0.31	6.1	0.24	6.0	0.24
08:45	1.0	0.04	1.5	0.06	0.3	0.01	8.0	0.32	6.2	0.25	6.0	0.24
08:50	1.0	0.04	0.8	0.03	0.3	0.01	8.1	0.32	6.3	0.25	6.0	0.24
08:55	1.0	0.04	0.8	0.03	0.3	0.01	8.2	0.32	6.4	0.25	6.1	0.24
09:00	1.0	0.04	0.0	0.00	0.3	0.01	8.3	0.33	6.4	0.25	6.1	0.24
09:05	1.5	0.06	1.5	0.06	1.0	0.04	8.4	0.33	6.5	0.26	6.2	0.24
09:10	0.0	0.00	1.5	0.06	1.0	0.04	8.4	0.33	6.6	0.26	6.3	0.25
09:15	1.5	0.06	1.5	0.06	1.0	0.04	8.5	0.34	6.7	0.27	6.4	0.25
09:20	1.5	0.06	1.5	0.06	1.0	0.04	8.6	0.34	6.9	0.27	6.4	0.25
09:25	1.5	0.06	1.5	0.06	1.0	0.04	8.8	0.35	7.0	0.28	6.5	0.26
09:30	1.5	0.06	1.5	0.06	1.5	0.06	8.9	0.35	7.1	0.28	6.6	0.26
09:35	0.8	0.03	1.5	0.06	0.8	0.03	9.0	0.35	7.2	0.29	6.7	0.26
09:40	0.8	0.03	1.5	0.06	0.8	0.03	9.0	0.36	7.4	0.29	6.8	0.27
09:45	0.0	0.00	1.5	0.06	0.8	0.03	9.0	0.36	7.5	0.30	6.8	0.27
09:50	0.0	0.00	1.5	0.06	0.8	0.03	9.0	0.36	7.6	0.30	6.9	0.27
09:55	0.0	0.00	1.5	0.06	0.8	0.03	9.0	0.36	7.7	0.31	7.0	0.27
10:00	0.0	0.00	1.5	0.06	0.8	0.03	9.0	0.36	7.9	0.31	7.0	0.28
10:05	0.8	0.03	1.0	0.04	0.0	0.00	9.1	0.36	8.0	0.31	7.1	0.28
10:10	0.8	0.03	1.0	0.04	0.0	0.00	9.1	0.36	8.0	0.32	7.1	0.28
10:15	1.5	0.06	1.0	0.04	0.0	0.00	9.3	0.37	8.1	0.32	7.1	0.28
10:20	1.5	0.06	3.0	0.12	1.5	0.06	9.4	0.37	8.4	0.33	7.2	0.28
10:25	3.0	0.12	3.0	0.12	0.8	0.03	9.7	0.38	8.6	0.34	7.2	0.28
10:30	1.5	0.06	1.5	0.06	0.8	0.03	9.8	0.39	8.8	0.35	7.3	0.29
10:35	3.0	0.12	1.5	0.06	0.0	0.00	10.0	0.40	8.9	0.35	7.3	0.29
10:40	1.5	0.06	0.0	0.00			10.2	0.40	8.9	0.35		
10:45	0.0	0.00	0.8	0.03			10.2	0.40	9.0	0.35		
10:50	1.5	0.06	0.8	0.03			10.3	0.41	9.0	0.36		
10:55	0.0	0.00	0.0	0.00			10.3	0.41	9.0	0.36		

Note: Intensities represent averages for time interval preceding indicated clock time.

Table 51. RUNOFF DATA OF THE STORM OF MAY 13, 1971 - BLOODY RUN CATCHMENT

Time	Runoff									
	Outlet		Longview #1		Longview #2		Bank #1		Bank #2	
hr:min:sec	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
03:07:30			0.078	2.75						
03:15:00			0.170	6.00						
03:22:30			0.248	8.75						
03:30:00			0.368	13.00			0.024	0.83	0.018	0.65
03:37:30			0.481	17.00			0.038	1.33	0.023	0.80
03:45:00	0.345	12.20	0.552	19.50			0.076	2.67	0.031	1.10
03:52:30	1.485	52.44	0.609	21.50	0.219	7.75	0.093	3.30	0.035	1.25
04:00:00	3.350	118.29	0.573	20.25	0.354	12.50	0.139	4.90	0.048	1.70
04:07:30	4.748	167.68	0.552	19.50	0.467	16.50	0.135	4.77	0.040	1.40
04:15:00	5.705	201.46	0.609	21.50	0.524	18.50	0.130	4.60	0.035	1.25
04:22:30	5.429	191.71	0.651	23.00	0.524	18.50	0.123	4.35	0.044	1.55
04:30:00	5.525	195.12	0.665	23.50	0.503	17.75	0.119	4.20	0.120	4.25
04:37:30	5.732	202.44	0.651	23.00	0.524	18.50	0.121	4.28	0.091	3.20
04:45:00	5.912	208.78	0.609	21.50	0.545	19.25	0.148	5.23	0.068	2.40
04:52:30	5.912	208.78	0.552	19.50	0.524	18.50	0.126	4.45	0.079	2.80
05:00:00	5.871	207.32	0.481	17.00	0.524	18.50	0.139	4.90	0.106	3.75
05:07:30	5.767	203.66	0.368	13.00	0.524	18.50	0.139	4.90	0.035	1.23
05:15:00	5.491	193.90	0.311	11.00	0.510	18.00	0.113	4.00		
05:22:30	4.765	168.29	0.269	9.50	0.496	17.50	0.089	3.15		
05:30:00	3.674	129.76	0.227	8.00	0.476	16.80	0.071	2.49		
05:37:30	2.383	84.15	0.212	7.50	0.446	15.75	0.096	3.38		
05:45:00	1.209	42.68	0.198	7.00	0.345	12.20	0.061	2.15		
05:52:30	0.777	27.44	0.184	6.50	0.326	11.50	0.057	2.00		
06:00:00	0.622	21.95	0.177	6.25	0.278	9.80	0.051	1.80		
06:07:30	0.501	17.68	0.170	6.00	0.248	8.75	0.047	1.65		
06:15:00	0.345	12.20	0.135	4.75	0.212	7.50	0.042	1.50		
06:22:30			0.135	4.75			0.040	1.40		
06:30:00			0.135	4.75			0.038	1.33		
06:37:30			0.135	4.75			0.038	1.33		
06:45:00			0.135	4.75			0.038	1.33		
06:52:30			0.135	4.75			0.038	1.33		
07:00:00			0.135	4.75			0.038	1.33		
07:07:30			0.135	4.75			0.038	1.33		
07:15:00			0.135	4.75			0.038	1.33		
07:22:30			0.135	4.75			0.038	1.33		
07:30:00			0.135	4.75			0.038	1.33		
07:37:30			0.163	5.75			0.040	1.40		
07:45:00			0.170	6.00			0.042	1.48		
07:52:30			0.205	7.25			0.050	1.75		
08:00:00			0.227	8.00			0.051	1.80		
08:07:30			0.234	8.25			0.054	1.92		
08:15:00			0.241	8.50			0.054	1.92		
08:22:30	0.345	12.20	0.241	8.50			0.054	1.92		
08:30:00	0.414	14.63	0.248	8.75			0.054	1.92		
08:37:30	0.432	15.24	0.255	9.00			0.057	2.00		
08:45:00	0.501	17.68	0.262	9.25			0.057	2.00		
08:52:30	0.546	19.27	0.276	9.75			0.057	2.00		
09:00:00	0.622	21.95	0.290	10.25			0.057	2.00		
09:07:30	0.673	23.78			0.221	7.80	0.057	2.00		
09:15:00	0.767	27.07			0.234	8.25				
09:22:30	0.794	28.05	0.290	10.25	0.255	9.00				
09:30:00	0.822	29.02	0.304	10.75	0.269	9.50				
09:37:30	0.863	30.49	0.333	11.75	0.269	9.50				
09:45:00	0.881	31.10	0.347	12.25	0.269	9.50				
09:52:30	0.967	34.15	0.389	13.75	0.290	10.25				
10:00:00	1.209	42.68	0.425	15.00	0.333	11.75				
10:07:30	1.399	49.39	0.375	13.25	0.347	12.25				
10:15:00	1.485	52.44	0.333	11.75	0.354	12.50				
10:22 30	1.485	52.44	0.304	10.75	0.354	12.50				
10:30:00	1.392	49.15	0.290	10.25	0.354	12.50				

Table 51 (Continued). RUNOFF DATA OF THE STORM OF MAY 13, 1971 - BLOODY RUN CATCHMENT

Time	Runoff									
	Outlet		Longview #1		Longview #2		Bank #1		Bank #2	
hr:min:sec	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
10:37:30	1.392	49.15	0.269	9.50	0.354	12.50				
10:45:00	1.088	38.41	0.255	9.00	0.354	12.50				
10:52:30	0.932	32.93	0.248	8.75	0.333	11.75				
11:00:00	0.760	26.83	0.241	8.50	0.311	11.00				
11:07:30	0.639	22.56	0.219	7.75	0.290	10.25				
11:15:00	0.515	18.17	0.205	7.25	0.269	9.50				
11:22:30	0.483	17.07	0.198	7.00	0.255	9.00				
11:30:00	0.466	16.46	0.177	6.25	0.234	8.25				
11:37:30			0.170	6.00						
11:45:00			0.170	6.00						
11:52:30			0.164	5.80						
12:00:00			0.164	5.80						
12:07:30			0.163	5.75						
12:15:00			0.149	5.25						
12:22:30			0.142	5.00						
12:30:00			0.135	4.75						

Note: Runoff values were scaled from figures published by the University of Cincinnati (1972). Blank spaces indicate no measurements.

Table 52. SUSPENDED SOLIDS CONCENTRATIONS OF THE STORM RUNOFF OF MAY 13, 1970 - BLOODY RUN CATCHMENT

	Concentrations, mg/l				
Time, hr:min:sec	Outlet	Longview #1	Longview #2	Bank #1	Bank #2
04:05:00	259				
04:12:30	277				
04:20:00	258				
04:27:30	146				
04:35:00	334				
04:42:30	282				
04:50:00	214				
04:57:30	233				
05:05:00	183				
05:12:30	66				
05:20:00	216				
05:27:30	122				
05:35:00	276				

Note: Blank spaces indicate no measurements.

Table 53. BIOCHEMICAL OXYGEN DEMAND CONCENTRATIONS OF THE STORM RUNOFF OF MAY 13, 1971 - BLOODY RUN CATCHMENT

	Concentrations, mg/l				
Time, hr:min:sec	Outlet	Longview #1	Longview #2	Bank #1	Bank #2
04:05:00	195				
04:12:30	173				
04:20:00	176				
04:27:30	118				
04:35:00	117				
04:42:30	135				
04:50:00	150				
04:57:30	63				
05:05:00	56				
05:12:30	58				
05:20:00	40				
05:27:30	255				
05:35:00	253				
05:42:30	216				
05:50:00	119				

Note: Blank spaces indicate no measurements.

Table 54. RAINFALL DATA OF THE STORM OF AUGUST 25 AND 26, 1971 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
19:00	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:05	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:10	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:15	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:20	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:25	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:30	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:35	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:40	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:45	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
19:50	0.0	0.00	0.0	0.00	1.0	0.04	23.0	0.91	5.5	0.22	12.0	0.47
19:55	0.0	0.00	0.0	0.00	1.3	0.05	23.0	0.91	5.5	0.22	12.1	0.48
20:00	0.0	0.00	0.0	0.00	1.0	0.04	23.0	0.91	5.5	0.22	12.1	0.48
20:05	0.0	0.00	1.5	0.06	1.5	0.06	23.0	0.91	5.6	0.22	12.3	0.48
20:10	0.0	0.00	1.5	0.06	0.0	0.00	23.0	0.91	5.7	0.23	12.3	0.48
20:15	0.0	0.00	3.0	0.12	0.0	0.00	23.0	0.91	6.0	0.24	12.3	0.48
20:20	0.0	0.00	3.0	0.12	0.0	0.00	23.0	0.91	6.2	0.25	12.3	0.48
20:25	0.0	0.00	1.5	0.06	0.0	0.00	23.0	0.91	6.4	0.25	12.3	0.48
20:30	0.0	0.00	3.0	0.12	0.0	0.00	23.0	0.91	6.6	0.26	12.3	0.48
20:35	0.0	0.00	1.5	0.06	0.0	0.00	23.0	0.91	6.7	0.27	12.3	0.48
20:40	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	6.7	0.27	12.3	0.48
20:45	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	6.7	0.27	12.3	0.48
20:50	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	6.7	0.27	12.3	0.48
20:55	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	6.7	0.27	12.3	0.48
21:00	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	6.7	0.27	12.3	0.48
21:05	0.0	0.00	0.0	0.00	3.0	0.12	23.0	0.91	6.7	0.27	12.5	0.49
21:10	0.0	0.00	0.0	0.00	3.0	0.12	23.0	0.91	6.7	0.27	12.8	0.50
21:15	0.0	0.00	0.0	0.00	3.0	0.12	23.0	0.91	6.7	0.27	13.0	0.51
21:20	0.0	0.00	30.5	1.20	0.0	0.00	23.0	0.91	9.3	0.37	13.0	0.51
21:25	0.0	0.00	30.5	1.20	0.0	0.00	23.0	0.91	11.8	0.47	13.0	0.51

Table 54 (Continued). RAINFALL DATA OF THE STORM OF AUGUST 25 AND 26, 1971 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
16:30	0.0	0.00	1.5	0.06	7.6	0.30	10.7	0.42	1.8	0.07	11.9	0.47
16:35	15.2	0.60	1.5	0.06	0.0	0.00	11.9	0.47	1.9	0.08	11.9	0.47
16:40	9.1	0.36	3.0	0.12	0.0	0.00	12.7	0.50	2.2	0.09	11.9	0.47
16:45	6.1	0.24	6.1	0.24	0.0	0.00	13.2	0.52	2.7	0.11	11.9	0.47
16:50	9.1	0.36	12.2	0.48	0.0	0.00	14.0	0.55	3.7	0.15	11.9	0.47
16:55	3.0	0.12	12.2	0.48	0.0	0.00	14.2	0.56	4.7	0.19	11.9	0.47
17:00	0.0	0.00	9.1	0.36	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:05	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:10	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:15	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:20	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:25	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:30	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:35	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:40	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:45	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:50	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
17:55	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
18:00	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
18:05	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
18:10	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
18:15	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
18:20	0.0	0.00	0.0	0.00	0.0	0.00	14.2	0.56	5.5	0.22	11.9	0.47
18:25	12.2	0.48	0.0	0.00	0.0	0.00	15.2	0.60	5.5	0.22	11.9	0.47
18:30	15.2	0.60	0.0	0.00	0.0	0.00	16.5	0.65	5.5	0.22	11.9	0.47
18:35	21.3	0.84	0.0	0.00	0.0	0.00	18.3	0.72	5.5	0.22	11.9	0.47
18:40	23.4	0.92	0.0	0.00	0.0	0.00	20.2	0.80	5.5	0.22	11.9	0.47
18:45	30.5	1.20	0.0	0.00	0.0	0.00	22.8	0.90	5.5	0.22	11.9	0.47
18:50	3.0	0.12	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47
18:55	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	5.5	0.22	11.9	0.47

Table 54 (Continued). RAINFALL DATA OF THE STORM OF AUGUST 25 AND 26, 1971 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
14:00					0.0	0.00					0.0	0.00
14:05			0.0	0.00	30.5	1.20	0.0	0.00	0.0	0.00	2.5	0.10
14:10			1.5	0.06	15.2	0.60	0.0	0.00	0.1	0.01	3.8	0.15
14:15	0.0	0.00	1.5	0.06	9.1	0.36	0.0	0.00	0.3	0.01	4.6	0.18
14:20	18.3	0.72	1.5	0.06	6.1	0.24	1.5	0.06	0.4	0.02	5.1	0.20
14:25	18.3	0.72	1.5	0.06	9.1	0.36	3.0	0.12	0.5	0.02	5.8	0.23
14:30	39.6	1.56	1.5	0.06	6.1	0.24	6.4	0.25	0.6	0.03	6.4	0.25
14:35	52.8	2.04	1.5	0.06	3.0	0.12	10.7	0.42	0.8	0.03	6.6	0.26
14:40	0.0	0.00	1.5	0.06	3.0	0.12	10.7	0.42	0.9	0.04	6.9	0.27
14:45	0.0	0.00	1.5	0.06	0.0	0.00	10.7	0.42	1.0	0.04	6.9	0.27
14:50	0.0	0.00	1.5	0.06	0.0	0.00	10.7	0.42	1.1	0.05	6.9	0.27
14:55	0.0	0.00	1.5	0.06	0.0	0.00	10.7	0.42	1.3	0.05	6.9	0.27
15:00	0.0	0.00	1.8	0.07	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:05	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:10	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:15	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:20	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:25	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:30	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:35	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:40	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:45	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:50	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
15:55	0.0	0.00	0.0	0.00	0.0	0.00	10.7	0.42	1.4	0.06	6.9	0.27
16:00	0.0	0.00	0.0	0.00	1.0	0.04	10.7	0.42	1.4	0.06	6.9	0.27
16:05	0.0	0.00	0.0	0.00	13.7	0.54	10.7	0.42	1.4	0.06	8.1	0.32
16:10	0.0	0.00	0.0	0.00	12.2	0.48	10.7	0.42	1.4	0.06	9.1	0.36
16:15	0.0	0.00	0.0	0.00	9.1	0.36	10.7	0.42	1.4	0.06	9.9	0.39
16:20	0.0	0.00	1.3	0.05	9.1	0.36	10.7	0.42	1.5	0.06	10.6	0.42
16:25	0.0	0.00	1.5	0.06	7.6	0.30	10.7	0.42	1.7	0.07	11.3	0.44

Table 54 (Continued). RAINFALL DATA OF THE STORM OF AUGUST 25 AND 26, 1971 - BLOODY RUN CATCHMENT

Time, hr:min	Rain intensities						Cumulative rain					
	Woodward raingage		Ridge raingage		Laidlaw raingage		Woodward raingage		Ridge raingage		Laidlaw raingage	
	mm/hr	in./hr	mm/hr	in./hr	mm/hr	in./hr	mm	in.	mm	in.	mm	in.
21:30	0.0	0.00	21.3	0.84	0.0	0.00	23.0	0.91	13.6	0.54	13.0	0.51
21:35	0.0	0.00	0.0	0.00	15.2	0.60	23.0	0.91	13.6	0.54	14.3	0.56
21:40	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	13.6	0.54	14.3	0.56
21:45	0.0	0.00	0.0	0.00	0.0	0.00	23.0	0.91	13.6	0.54	14.3	0.56
21:50	0.0	0.00	12.2	0.48	0.0	0.00	23.0	0.91	14.6	0.58	14.3	0.56
21:55	0.0	0.00	12.2	0.48	0.0	0.00	23.0	0.91	15.6	0.62	14.3	0.56
22:00	0.0	0.00	12.2	0.48	0.0	0.00	23.0	0.91	16.6	0.66	14.3	0.56
22:05	3.0	0.12	0.0	0.00	0.0	0.00	23.3	0.92	16.6	0.66	14.3	0.56
22:10	3.0	0.12	0.0	0.00	0.0	0.00	23.6	0.93	16.6	0.66	14.3	0.56
22:15	3.0	0.12	0.0	0.00	0.0	0.00	23.8	0.94	16.6	0.66	14.3	0.56
22:20	0.0	0.00	0.0	0.00	0.0	0.00	23.8	0.94	16.6	0.66	14.3	0.56
22:25	0.0	0.00	0.0	0.00	9.1	0.36	23.8	0.94	16.6	0.66	15.1	0.59
22:30	0.0	0.00	0.0	0.00	6.1	0.24	23.8	0.94	16.6	0.66	15.6	0.61
22:35	0.0	0.00	0.0	0.00	15.2	0.60	23.8	0.94	16.6	0.66	16.8	0.66
22:40	0.0	0.00	4.6	0.18	6.1	0.24	23.8	0.94	17.0	0.67	17.4	0.68
22:45	0.0	0.00	4.6	0.18	0.0	0.00	23.8	0.94	17.4	0.69	17.4	0.68
22:50	0.0	0.00	4.6	0.18	0.0	0.00	23.8	0.94	17.8	0.70	17.4	0.68
22:55	0.0	0.00	3.0	0.12	0.0	0.00	23.8	0.94	18.0	0.71	17.4	0.68
23:00	0.0	0.00	1.5	0.06	0.0	0.00	23.8	0.94	18.2	0.72	17.4	0.68
23:05	0.0	0.00	0.0	0.00	0.0	0.00	23.8	0.94	18.2	0.72	17.4	0.68
23:10	0.0	0.00			0.0	0.00	23.8	0.94			17.4	0.68
23:15	0.0	0.00			0.0	0.00	23.8	0.94			17.4	0.68
23:20	0.0	0.00			0.0	0.00	23.8	0.94			17.4	0.68
23:25	9.1	0.36			0.0	0.00	24.6	0.97			17.4	0.68
23:30	0.0	0.00			0.0	0.00	24.6	0.97			17.4	0.68
23:35	0.0	0.00			0.0	0.00	24.6	0.97			17.4	0.68
23:40	0.0	0.00			0.0	0.00	24.6	0.97			17.4	0.68
23:45	0.0	0.00			0.0	0.00	24.6	0.97			17.4	0.68
23:50	12.2	0.48			0.0	0.00	25.6	1.01			17.4	0.68
23:55	9.1	0.36			0.0	0.00	26.3	1.04			17.4	0.68

Note: Intensities represent averages for time inverval preceding indicated clock time.

Table 55. RUNOFF DATA OF THE STORM OF AUGUST 25 AND 26, 1971 - BLOODY RUN CATCHMENT

Time	Runoff									
	Outlet		Longview #1		Longview #2		Bank #1		Bank #2	
hr:min:sec	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
13:30:00							0.018	0.65		
13:37:30							0.018	0.65		
13:45:00							0.026	0.91		
13:52:30							0.075	2.64		
14:00:00							0.530	18:70		
14:07:30							0.000	0.00		
14:15:00										
14:22:30										
14:30:00	1.356	47.87			0.204	7.21				
14:37:30	21.539	760.66			1.410	49.78			0.000	0.00
14:45:00	18.661	659.02			4.259	150.39			0.086	3.02
14:52:30	13.778	486.56			2.430	85.81			0.150	5.28
15:00:00	8.727	308.20			1.150	40.61			0.255	8.99
15:07:30	7.242	255.74			0.927	32.75			0.000	0.00
15:15:00	6.127	216.39			0.801	28.30				
15:22:30	4.642	163.93			0.612	21.62				
15:30:00	2.135	75.41			0.490	17.29				
15:37:30	0.836	29.51			0.427	15.07				
15:45:00	0.650	22.95			0.382	13.49				
15:52:30	0.557	19.67			0.341	12.05				
16:00:00	0.371	13.11			0.282	9.96				
16:07:30	0.371	13.11			0.278	9.83				
16:15:00	0.371	13.11			0.278	9.83				
16:22:30	0.371	13.11			0.278	9.83				
16:30:00	1.300	45.90			0.278	9.83			0.000	0.00
16:37:30	2.618	92.46			0.601	21.22			0.255	8.99
16:45:00	7.019	247.87			1.261	44.54			0.150	5.28
16:52:30	8.319	293.77			0.872	30.79			0.255	8.99
17:00:00	9.228	325.90			1.261	44.54			0.197	6.96
17:07:30	8.801	310.82			1.154	40.74			0.000	0.00
17:15:00	7.873	278.03			0.909	32.10				
17:22:30	6.610	233.44			0.798	28.17				
17:30:00	4.215	148.85			0.779	27.51	0.018	0.65		
17:37:30	2.655	93.77			0.779	27.51	0.018	0.65		
17:45:00	1.114	39.34			0.742	26.20	0.075	2.64		
17:52:30	0.761	26.88			0.649	22.93	0.120	4.25		
18:00:00	0.483	17.05			0.341	12.05	0.434	15.34		
18:07:30	0.483	17.05			0.371	13.10	0.040	1.40		
18:15:00	0.483	17.05			0.430	15.20	0.024	0.83		
18:22:30	2.302	81.31			0.501	17.69	0.018	0.65		
18:30:00	2.878	101.64			0.594	20.96	0.018	0.65		
18:37:30	16.526	583.61			1.402	49.52				
18:45:00	19.552	690.49			2.460	86.86				
18:52:30	15.449	545.57			4.025	142.14			0.000	0.00
19:00:00	11.234	396.72			1.410	49.78			0.255	8.99
19:07:30	9.693	342.30			1.161	41.00			0.197	6.96
19:15:00	7.242	255.74			0.909	32.10			0.126	4.46
19:22:30	6.555	231.48			0.779	27.51			0.049	1.73
19:30:00	4.884	172.46			0.605	21.35			0.011	0.39
19:37:30	3.547	125.25			0.549	19.39				
19:45:00	1.727	60.98			0.501	17.69				
19:52:30	1.003	35.41			0.445	15.72				
20:00:00	0.743	26.23			0.371	13.10				
20:07:30	0.743	26.23								
20:15:00	0.650	22.95								
20:22:30	0.464	16.39								

Table 55 (Continued). RUNOFF DATA OF THE STORM OF AUGUST 25 AND 26, 1971 - BLOODY RUN CATCHMENT

Time	Runoff									
	Outlet		Longview #1		Longview #2		Bank #1		Bank #2	
hr:min:sec	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs	m^3/sec	cfs
20:30:00	0.520	18.36								
20:37:30	0.650	22.95								
20:45:00	0.650	22.95								
20:52:30	0.557	19.67								
21:00:00	0.427	15.08								
21:07:30	0.390	13.77								
21:15:00	0.390	13.77								
21:22:30	0.390	13.77								
21:30:00	8.412	297.05								
21:37:30	7.892	278.69								
21:45:00	6.610	233.44								
21:52:30	6.610	233.44								
22:00:00	6.945	245.25								
22:07:30	6.945	245.25								
22:15:00	6.146	217.05								
22:22:30	5.069	179.02								
22:30:00	2.859	100.98								
22:37:30	2.859	100.98								
22:45:00	5.849	206.56								
22:52:30	8.727	308.20								
23:00:00	8.022	283.28								
23:07:30	7.000	247.21								
23:15:00	4.698	165.90								
23:22:30	3.547	125.25								
23:30:00	1.523	53.77								
23:37:30	0.891	31.48								
23:45:00	0.650	22.95								
23:52:30	0.557	19.67								
24:00:00	0.464	16.39								
24:07:30	0.409	14.43								
24:15:00	0.371	13.11								
24:22:30	0.371	13.11								
24:30:00	0.371	13.11								
24:37:30	0.371	13.11								
24:45:00	0.371	13.11								

Note: Runoff values were scaled from figures published by the University of Cincinnati (1972). Blank spaces indicate no measurements.

Table 56. SUSPENDED SOLIDS CONCENTRATIONS OF THE STORM RUNOFF OF AUGUST 25, 1971 - BLOODY RUN CATCHMENT

	Concentrations, mg/l				
Time, hr:min:sec	Outlet	Longview #1	Longview #2	Bank #1	Bank #2
14:45:00	276				
15:07:30	321				
15:22:30	208				
15:30:00	170				
16:45:00	125				
17:07:30	140				
17:30:00	106				
17:37:30	97				

Note: Blank spaces indicate no measurements.

Table 57. BIOCHEMICAL OXYGEN DEMAND CONCENTRATIONS OF THE STORM RUNOFF OF AUGUST 25, 1971 - BLOODY RUN CATCHMENT

	Concentrations, mg/l				
Time, hr:min:sec	Outlet	Longview #1	Longview #2	Bank #1	Bank #2
14:30:00	113				
14:37:30	12				
14:45:00	24				
14:52:30	11				
15:00:00	38				
15:07:30	11				
15:15:00	8				
15:22:30	7				
15:30:00	9				
16:30:00	33				
16:37:30	27				
16:45:00	13				
16:52:30	30				
17:00:00	14				
17:07:30	21				
17:15:00	4				
17:22:30	21				
17:30:00	4				
17:37:30	4				
18:22:30	35				
18:30:00	12				
18:37:30	6				
18:45:00	11				
18:52:30	27				

Note: Blank spaces indicate no measurements.

SECTION VII

TEST RESULTS

	Page
Introduction	243
Hypothetical Catchment Data Tests	244
Model Comparison	268
Two-Hour Triangular Rainstorm	269
Hypothetical Pipe Data Tests	272
Model Comparison	296
Two-Hour Triangular Inflow	298
Real Catchment Data Tests	301
Oakdale Avenue Catchment Simulations	301
Bloody Run Catchment Simulations	319

INTRODUCTION

The following seven models were tested by computer runs with the hypothetical catchment and pipe data described in the previous section:

1. Battelle Urban Wastewater Management Model (BNW)
2. Dorsch Consult Hydrograph-Volume Method (DORSCH)
3. Environmental Protection Agency Stormwater Management Model (SWMM)
4. Massachusetts Institute of Technology Urban Watershed Model (MIT)
5. Metropolitan Sanitary District of Greater Chicago Flow Simulation Program (FSP)
6. SOGREAH Looped Sewer Model (SOGREAH)
7. Water Resources Engineers Stormwater Management Model (WRE)

The Hydrocomp Simulation Program was also to be included in the testing, but Hydrocomp International declined to participate.

Four of the models were also tested by computer runs with real catchment data. The BNW, FSP, SWMM, and DORSCH models were tested with data from the Oakdale Avenue catchment in Chicago, Illinois, and the BNW, FSP, and SWMM models were tested with data from the Bloody Run catchment in Cincinnati, Ohio. The proprietary models were not included in this effort due to their developers doubts about the completeness and accuracy of the real catchment data.

The BNW model was run on a DEC PDP-9 computer at Battelle-Northwest in Richland, Washington. The SWMM and FSP were run on a CDC 6400 computer at Battelle-Columbus, Ohio, by remote terminal from Battelle-Northwest. This required the conversion of the original IBM versions of these two programs into CDC versions.

The four proprietary models were run by the respective firms with data supplied by Battelle-Northwest. The DORSCH model was run by Dorsch-Consult on a Univac 1108 computer in Munich, Germany; the MIT model was run by Resource Analysis on an IBM 370 computer in Boston, Massachusetts; the SOGREAH model was run by SOGREAH on an IBM 360 computer in Grenoble, France;

and the WRE model was run by Water Resources Engineers on a Univac 1108 computer in San Francisco, California.

The original intent was to run all models on the same computer to compare running times and costs. This was not possible for the proprietary models or for the BNW model, which is formulated for a small real-time process computer and would have required major reprogramming to adapt it to a larger computer. The test results, however, are still useful for comparing model sensitivity to parameter variations (using hypothetical data) and for comparing model accuracy (using real catchment data). Although running times cannot be compared directly among all the models, orders of magnitude can be indicated and fairly reliable cost comparisons can be made.

HYPOTHETICAL CATCHMENT TESTS

Hypothetical catchment data tests were performed for two different catchment sizes, two catchment slopes, two orientations of each catchment with respect to the direction of surface flow, five ratios of pervious to impervious areas, two different initial catchment moisture conditions, and four diferent rainstorms, representing a total of 320 data combinations. The effects of different catchment surface roughness coefficients and of different time discretizations on computed runoff were not tested.

A time step of 5 minutes was suggested for the catchment runoff simulations and was actually used for the BNW, FSP, SWMM, DORSCH, and WRE models. The SOGREAH model used a time step of 1 minute. The MIT model sets the time step internally as a function of catchment characteristics but independent of rainstorm characteristics. In the tests it ranged from 2 to 16 minutes for the different runs. The time step remains constant, however, for any particular runoff computation.

The results of the catchment runoff simulations for all seven models are summarized in Tables 58 to 64, which present peak runoffs and times for each of the 320 catchment/rainfall combinations. The times of peak runoff are not tabulated for the constant rainstorm since equilibrium runoff conditions were computed for most data combinations.

It was not possible to plot all computed runoff hydrographs, but plots are presented in Figures 31 to 42 for the runoff from the 2-hr triangular storm with initially dry catchment moisture conditions and the direction of flow parallel to the short length of the rectangular catchments. The plots show the effect on catchment runoff of changes in catchment slope and imperviousness.

Table 58a. PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR TWO-HOUR CONSTANT RAINSTORM (cfs)

Slope, percent	Length, ft	Imperviousness, percent	Catchment number	Dry initial condition							Wet initial condition						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	0.23	0.58	0.34	0.2	0.29	0.57	0.23	0.30	0.58	0.34	0.4	0.37	0.57	0.30
		25	2	0.50	0.72	0.34	0.4	0.50	0.71	0.50	0.55	0.72	0.34	0.6	0.56	0.71	0.55
	316	50	3	0.76	0.87	0.34	0.7	0.72	0.85	0.76	0.79	0.87	0.34	0.8	0.76	0.88	0.79
		75	4	0.99	1.01	0.34	0.9	0.92	1.02	0.99	1.00	1.01	0.34	1.0	0.95	1.02	1.00
0.1		100	5	1.15	1.16	0.85	1.2	1.15	1.17	1.16	1.16	1.16	0.85	1.2	1.15	1.17	1.16
		0	6	0.36	0.58	0.17	0.3	0.46	0.57	0.36	0.43	0.58	0.17	0.5	0.52	0.57	0.43
		25	7	0.60	0.72	0.17	0.5	0.63	0.71	0.60	0.65	0.72	0.17	0.7	0.67	0.71	0.65
	158	50	8	0.82	0.87	0.17	0.7	0.80	0.85	0.82	0.84	0.87	0.17	0.8	0.83	0.88	0.84
		75	9	1.00	1.01	0.17	0.9	0.97	1.02	1.01	1.01	1.01	0.17	1.0	0.99	1.02	1.01
		100	10	1.16	1.16	0.84	1.2	1.14	1.17	1.16	1.16	1.16	0.84	1.2	1.14	1.17	1.16
		0	11	0.56	0.58	1.09	0.5	0.57	0.57	0.56	0.57	0.58	1.09	0.6	0.58	0.57	0.58
		25	12	0.72	0.73	1.09	0.7	0.72	0.71	0.72	0.72	0.73	1.09	0.7	0.72	0.71	0.72
	316	50	13	0.87	0.87	1.09	0.8	0.87	0.88	0.87	0.87	0.87	1.09	0.9	0.87	0.88	0.87
		75	14	1.01	1.02	1.09	1.0	1.02	1.02	1.01	1.01	1.02	1.09	1.0	1.02	1.02	1.01
10.0		100	15	1.16	1.16	0.86	1.2	1.16	1.17	1.16	1.16	1.16	0.86	1.2	1.16	1.17	1.16
		0	16	0.58	0.58	1.06	0.6	0.57	0.57	0.58	0.58	0.58	1.06	0.6	0.58	0.57	0.58
		25	17	0.72	0.73	1.06	0.7	0.73	0.71	0.72	0.72	0.73	1.06	0.7	0.73	0.71	0.72
	158	50	18	0.87	0.87	1.06	0.9	0.88	0.88	0.87	0.87	0.87	1.06	0.9	0.88	0.88	0.87
		75	19	1.01	1.02	1.06	1.0	1.03	1.02	1.01	1.01	1.02	1.06	1.0	1.03	1.02	1.01
		100	20	1.16	1.16	0.86	1.2	1.18	1.17	1.16	1.16	1.16	0.86	1.2	1.18	1.17	1.16
		0	21	0.89	5.10	0.52	0.7	0.93	5.05	0.89	1.24	5.79	0.52	2.3	1.02	5.76	1.24
		25	22	3.75	6.63	1.93	3.4	3.52	6.78	3.75	4.08	7.24	1.93	4.6	3.58	7.20	4.08
	1000	50	23	6.55	8.24	4.11	6.1	6.17	8.41	6.54	6.83	8.68	4.11	6.9	6.22	8.65	6.83
		75	24	9.21	9.89	6.27	8.8	8.83	9.99	9.09	9.29	10.13	6.27	9.2	8.86	10.14	9.29
0.1		100	25	10.91	11.58	8.30	11.5	11.49	11.58	10.85	10.91	11.58	8.30	11.5	11.49	11.58	10.92
		0	26	1.61	5.77	1.09	1.4	1.74	5.33	1.61	2.17	5.79	1.09	3.6	2.14	5.76	2.17
		25	27	4.41	7.22	2.47	3.9	4.16	6.96	4.41	4.89	7.24	2.47	5.6	4.46	7.20	4.89
	500	50	28	7.14	8.67	4.62	6.5	6.61	8.51	7.14	7.49	8.68	4.62	7.6	6.81	8.65	7.50
		75	29	9.66	10.13	6.75	9.0	9.05	10.07	9.66	9.82	10.13	6.75	9.6	9.15	10.14	9.83
		100	30	11.48	11.58	8.34	11.6	11.49	11.58	11.47	11.48	11.58	8.34	11.6	11.49	11.58	11.48
		0	31	4.41	5.83	9.69	4.0	5.24	5.72	4.41	4.99	5.83	9.69	5.6	5.66	5.76	4.99
		25	32	6.54	7.28	9.63	5.9	6.78	7.20	6.54	6.88	7.28	9.63	7.1	7.10	7.20	6.88
	1000	50	33	8.46	8.73	9.40	7.8	8.33	8.65	8.46	8.59	8.73	9.40	8.6	8.54	8.65	8.59
		75	34	10.11	10.18	8.98	9.7	9.88	10.10	10.11	10.12	10.18	8.98	10.1	9.99	10.14	10.12
10.0		100	35	11.57	11.62	8.37	11.6	11.46	11.58	11.58	11.57	11.62	8.37	11.6	11.46	11.58	11.58
		0	36	5.34	5.83	10.37	5.1	5.69	5.72	5.34	5.60	5.83	10.37	5.8	5.76	5.76	5.60
		25	37	7.06	7.28	10.10	6.7	7.15	7.20	7.06	7.17	7.28	10.10	7.2	7.18	7.20	7.17
	500	50	38	8.64	8.73	9.61	8.3	8.64	8.65	8.64	8.67	8.73	9.61	8.7	8.68	8.65	8.67
		75	39	10.13	10.18	9.03	10.0	10.13	10.10	10.13	10.13	10.18	9.03	10.1	10.15	10.14	10.13
		100	40	11.57	11.62	8.34	11.6	11.61	11.58	11.58	11.57	11.62	8.34	11.6	11.61	11.58	11.58

Table 58b. PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR TWO-HOUR CONSTANT RAINSTORM (m^3/sec)

Slope percent	Length, m	Imperviousness, percent	Catchment number	Dry initial condition							Wet initial condition						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	0.007	0.016	0.010	0.006	0.008	0.016	0.007	0.008	0.016	0.010	0.011	0.010	0.016	0.008
		25	2	0.014	0.020	0.010	0.011	0.014	0.020	0.014	0.016	0.020	0.010	0.017	0.016	0.020	0.016
	96.4	50	3	0.022	0.025	0.010	0.020	0.020	0.024	0.022	0.022	0.025	0.010	0.023	0.022	0.025	0.022
		75	4	0.028	0.029	0.010	0.025	0.026	0.029	0.028	0.028	0.029	0.010	0.028	0.027	0.029	0.028
0.1		100	5	0.033	0.033	0.024	0.034	0.033	0.033	0.033	0.033	0.033	0.024	0.034	0.033	0.033	0.033
		0	6	0.010	0.016	0.005	0.008	0.013	0.016	0.010	0.012	0.016	0.005	0.014	0.015	0.016	0.012
		25	7	0.017	0.020	0.005	0.014	0.018	0.020	0.017	0.018	0.020	0.005	0.020	0.019	0.020	0.018
	48.2	50	8	0.023	0.025	0.005	0.020	0.023	0.024	0.023	0.024	0.025	0.005	0.023	0.024	0.025	0.024
		75	9	0.028	0.029	0.005	0.025	0.027	0.029	0.029	0.029	0.029	0.005	0.028	0.028	0.019	0.029
		100	10	0.033	0.033	0.024	0.034	0.032	0.033	0.033	0.033	0.033	0.024	0.034	0.032	0.033	0.033
		0	11	0.016	0.016	0.031	0.014	0.016	0.016	0.016	0.016	0.016	0.031	0.017	0.016	0.016	0.016
		25	12	0.020	0.021	0.031	0.020	0.020	0.020	0.020	0.020	0.021	0.031	0.020	0.020	0.020	0.020
	96.4	50	13	0.025	0.025	0.031	0.023	0.025	0.025	0.025	0.025	0.025	0.031	0.025	0.025	0.025	0.025
		75	14	0.029	0.029	0.031	0.028	0.029	0.029	0.029	0.029	0.029	0.031	0.028	0.029	0.029	0.029
10.0		100	15	0.033	0.033	0.024	0.034	0.033	0.033	0.033	0.033	0.033	0.024	0.034	0.033	0.033	0.033
		0	16	0.016	0.016	0.030	0.017	0.016	0.016	0.016	0.016	0.016	0.030	0.017	0.016	0.016	0.016
		25	17	0.020	0.021	0.030	0.020	0.021	0.020	0.020	0.020	0.021	0.030	0.020	0.021	0.020	0.020
	48.2	50	18	0.025	0.025	0.030	0.025	0.025	0.025	0.025	0.025	0.025	0.030	0.025	0.025	0.025	0.025
		75	19	0.029	0.029	0.030	0.028	0.029	0.029	0.029	0.029	0.029	0.030	0.028	0.029	0.029	0.029
		100	20	0.033	0.033	0.024	0.034	0.033	0.033	0.033	0.033	0.033	0.024	0.034	0.033	0.033	0.033
		0	21	0.025	0.144	0.015	0.020	0.026	0.143	0.025	0.035	0.164	0.015	0.065	0.029	0.163	0.035
		25	22	0.106	0.188	0.055	0.096	0.100	0.192	0.106	0.116	0.205	0.055	0.130	0.101	0.204	0.116
	304.8	50	23	0.185	0.233	0.116	0.173	0.175	0.238	0.185	0.193	0.246	0.116	0.195	0.176	0.245	0.193
		75	24	0.261	0.280	0.178	0.249	0.250	0.283	0.257	0.263	0.287	0.178	0.261	0.251	0.287	0.263
0.1		100	25	0.309	0.328	0.235	0.326	0.325	0.328	0.307	0.309	0.328	0.235	0.326	0.325	0.328	0.309
		0	26	0.046	0.163	0.031	0.040	0.049	0.151	0.046	0.061	0.164	0.031	0.102	0.061	0.163	0.061
		25	27	0.125	0.204	0.070	0.110	0.118	0.197	0.125	0.138	0.205	0.070	0.159	0.126	0.204	0.138
	152.4	50	28	0.202	0.246	0.131	0.184	0.187	0.241	0.202	0.212	0.246	0.131	0.215	0.193	0.245	0.212
		75	29	0.274	0.287	0.191	0.255	0.256	0.285	0.274	0.278	0.287	0.191	0.272	0.259	0.287	0.278
		100	30	0.325	0.328	0.236	0.329	0.325	0.328	0.325	0.325	0.328	0.236	0.329	0.325	0.328	0.325
		0	31	0.125	0.165	0.274	0.113	0.148	0.162	0.125	0.141	0.165	0.274	0.159	0.160	0.163	0.141
		25	32	0.185	0.206	0.273	0.167	0.192	0.204	0.185	0.195	0.206	0.273	0.201	0.201	0.204	0.195
	304.8	50	33	0.240	0.247	0.266	0.221	0.236	0.245	0.240	0.243	0.247	0.266	0.244	0.242	0.245	0.243
		75	34	0.286	0.288	0.254	0.275	0.280	0.286	0.286	0.287	0.288	0.254	0.286	0.283	0.287	0.287
10.0		100	35	0.328	0.329	0.237	0.329	0.325	0.328	0.328	0.328	0.329	0.237	0.329	0.325	0.328	0.328
		0	36	0.151	0.165	0.294	0.144	0.161	0.162	0.151	0.159	0.165	0.294	0.164	0.163	0.163	0.159
		25	37	0.200	0.206	0.286	0.190	0.202	0.204	0.200	0.203	0.206	0.286	0.204	0.203	0.204	0.203
	152.4	50	38	0.245	0.247	0.272	0.235	0.245	0.245	0.245	0.246	0.247	0.272	0.246	0.246	0.245	0.246
		75	39	0.287	0.288	0.256	0.283	0.287	0.286	0.287	0.287	0.288	0.256	0.286	0.287	0.287	0.287
		100	40	0.328	0.329	0.236	0.329	0.329	0.328	0.328	0.328	0.329	0.236	0.329	0.329	0.328	0.328

Table 59a. PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR ONE-HOUR TRIANGULAR RAINSTORM (cfs)

Slope, percent	Length, ft	Imperviousness, percent	Catchment number	Dry initial condition							Wet initial condition						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	0.56	2.36	0.39	0.7	0.63	2.47	0.60	0.69	2.87	0.39	1.3	0.74	3.21	0.73
		25	2	1.36	2.58	0.39	1.3	1.25	2.83	1.36	1.48	3.01	0.39	1.7	1.31	3.39	1.48
	316	50	3	2.24	2.87	0.39	2.1	2.14	3.14	2.23	2.35	3.16	0.39	2.4	2.18	3.53	2.35
		75	4	2.94	3.15	0.39	2.9	3.04	3.50	2.92	3.02	3.30	0.39	3.1	3.06	3.67	3.02
0.1		100	5	3.25	3.43	1.44	3.7	3.94	3.85	3.20	3.24	3.45	1.44	3.8	3.94	3.85	3.25
		0	6	0.96	2.97	0.19	1.1	1.21	2.65	0.99	1.15	3.41	0.19	1.8	1.43	3.36	1.19
		25	7	1.66	3.23	0.19	1.6	1.52	2.97	1.66	1.85	3.56	0.19	2.2	1.69	3.50	1.86
	158	50	8	2.58	3.48	0.19	2.3	2.38	3.28	2.59	2.74	3.70	0.19	2.8	2.45	3.67	2.74
		75	9	3.38	3.74	0.19	3.1	3.31	3.64	3.38	3.47	3.85	0.19	3.4	3.34	3.81	3.48
		100	10	3.75	3.99	1.25	4.0	4.24	3.96	3.74	3.75	3.99	1.25	4.0	4.24	3.96	3.76
		0	11	2.33	3.24	3.26	2.2	2.88	3.50	2.34	2.56	3.67	3.26	3.1	3.19	3.96	2.57
		25	12	2.83	3.49	3.26	2.6	3.12	3.74	2.83	3.04	3.81	3.26	3.3	3.41	4.10	3.04
	316	50	13	3.42	3.74	3.26	2.9	3.42	3.99	3.43	3.61	3.96	3.26	3.6	3.66	4.24	3.61
		75	14	4.02	4.00	3.26	3.6	3.81	4.27	4.03	4.06	4.10	3.26	3.9	3.93	4.38	4.07
10.0		100	15	4.26	4.25	1.44	4.2	4.24	4.52	4.26	4.26	4.25	1.44	4.2	4.24	4.52	4.26
		0	16	2.92	3.24	3.20	2.7	3.25	3.50	2.92	3.10	3.67	3.20	3.4	3.63	3.96	3.11
		25	17	3.23	3.49	3.20	2.9	3.43	3.78	3.24	3.44	3.81	3.20	3.6	3.76	4.10	3.45
	158	50	18	3.78	3.74	3.20	3.2	3.68	4.03	3.78	3.86	3.96	3.20	3.8	3.90	4.27	3.86
		75	19	4.14	4.00	3.20	3.8	3.95	4.27	4.14	4.14	4.10	3.20	4.0	4.04	4.41	4.15
		100	20	4.30	4.25	1.45	4.3	4.25	4.56	4.31	4.30	4.25	1.45	4.3	4.25	4.56	4.31
		0	21	2.06	12.08	0.55	2.7	2.30	18.82	2.18	2.59	15.65	0.55	5.3	2.62	26.91	2.72
		25	22	9.94	13.95	2.60	8.7	8.85	22.28	9.88	10.44	16.72	2.60	10.7	9.06	28.46	10.44
	1000	50	23	15.80	15.82	5.74	15.3	15.82	25.89	15.57	16.28	17.79	5.74	16.8	15.95	30.02	16.28
		75	24	19.55	17.69	8.89	22.0	22.80	29.35	19.28	19.95	18.86	8.89	23.0	22.87	31.54	20.00
0.1		100	25	20.68	19.56	12.10	28.6	29.79	32.89	20.65	20.68	19.93	12.10	29.1	29.79	33.06	21.13
		0	26	3.85	18.86	1.18	4.9	3.64	20.94	4.07	4.78	23.86	1.18	9.2	4.57	28.92	5.02
		25	27	12.08	21.40	3.18	11.1	10.61	24.51	12.05	12.94	25.26	3.18	14.5	11.16	30.48	12.94
	500	50	28	19.71	23.98	6.31	18.7	18.88	27.93	19.52	20.51	26.66	6.31	20.9	19.21	32.00	20.51
		75	29	25.74	26.56	9.46	26.6	27.28	31.43	25.46	26.34	28.06	9.46	27.9	27.44	33.69	26.34
		100	30	28.29	29.14	12.64	34.6	35.67	34.96	27.81	28.29	29.46	12.64	34.9	35.67	35.10	28.29
		0	31	13.00	32.43	29.04	14.1	18.01	33.90	13.45	15.30	36.67	29.04	22.6	21.06	38.53	15.65
		25	32	19.39	34.94	26.58	18.2	20.21	36.41	19.39	21.78	38.12	26.58	25.7	22.63	39.98	21.79
	1000	50	33	28.12	37.44	22.42	24.6	25.96	39.13	28.12	29.85	39.57	22.42	29.9	27.41	41.46	29.85
		75	34	35.54	39.95	17.67	32.7	34.08	41.78	35.53	36.59	41.01	17.67	35.5	34.74	42.94	36.59
10.0		100	35	39.36	42.46	12.74	41.5	42.25	44.43	39.19	39.35	42.46	12.74	41.5	42.25	44.43	39.35
		0	36	19.30	32.43	31.00	19.2	24.01	34.33	19.30	21.93	36.67	31.00	28.3	27.43	38.81	21.94
		25	37	24.70	34.94	27.87	22.3	26.28	36.90	24.70	27.23	38.12	27.87	30.6	29.33	40.29	27.23
	500	50	38	31.57	37.44	22.92	27.4	30.42	39.52	31.57	33.58	39.57	22.92	33.8	32.60	41.74	33.59
		75	39	38.77	39.95	17.74	34.5	36.17	42.13	38.77	39.54	41.01	17.74	38.0	37.26	43.23	39.54
		100	40	41.84	42.46	12.64	42.2	42.37	44.60	41.83	41.84	42.46	12.64	42.2	42.37	44.60	41.84

Table 59b. PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR ONE-HOUR TRIANGULAR RAINSTORM (m^3/sec)

Slope percent	Length, m	Imperviousness, percent	Catchment number	Dry initial condition							Wet initial condition						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	0.016	0.067	0.011	0.020	0.018	0.070	0.017	0.020	0.081	0.011	0.037	0.021	0.091	0.021
		25	2	0.039	0.073	0.011	0.037	0.035	0.080	0.039	0.042	0.085	0.011	0.048	0.037	0.096	0.042
	96.4	50	3	0.063	0.081	0.011	0.059	0.061	0.089	0.063	0.067	0.089	0.011	0.068	0.062	0.100	0.670
		75	4	0.083	0.089	0.011	0.082	0.086	0.099	0.083	0.086	0.093	0.011	0.088	0.087	0.104	0.086
0.1		100	5	0.092	0.097	0.041	0.105	0.112	0.109	0.091	0.092	0.098	0.041	0.108	0.112	0.109	0.092
		0	6	0.027	0.084	0.005	0.031	0.034	0.075	0.028	0.033	0.097	0.005	0.051	0.040	0.095	0.034
		25	7	0.047	0.091	0.005	0.045	0.043	0.084	0.047	0.052	0.101	0.005	0.062	0.048	0.099	0.053
	48.2	50	8	0.073	0.099	0.005	0.065	0.067	0.093	0.073	0.078	0.105	0.005	0.079	0.069	0.104	0.078
		75	9	0.096	0.106	0.005	0.088	0.094	0.103	0.096	0.098	0.109	0.005	0.096	0.095	0.108	0.099
		100	10	0.106	0.113	0.035	0.113	0.120	0.112	0.106	0.106	0.113	0.035	0.113	0.120	0.112	0.106
		0	11	0.066	0.092	0.092	0.062	0.082	0.099	0.066	0.072	0.104	0.092	0.088	0.090	0.112	0.073
		25	12	0.080	0.099	0.092	0.074	0.088	0.106	0.080	0.086	0.108	0.092	0.093	0.097	0.116	0.086
	96.4	50	13	0.097	0.106	0.092	0.082	0.097	0.113	0.097	0.102	0.112	0.092	0.102	0.104	0.120	0.102
		75	14	0.114	0.113	0.092	0.102	0.108	0.121	0.114	0.115	0.116	0.092	0.110	0.111	0.124	0.115
10.0		100	15	0.121	0.120	0.041	0.119	0.120	0.128	0.121	0.121	0.120	0.041	0.119	0.120	0.128	0.121
		0	16	0.083	0.092	0.091	0.076	0.092	0.099	0.083	0.088	0.104	0.091	0.096	0.103	0.112	0.088
		25	17	0.091	0.099	0.091	0.082	0.097	0.107	0.092	0.097	0.108	0.091	0.102	0.106	0.116	0.098
	48.2	50	18	0.107	0.106	0.091	0.091	0.104	0.114	0.107	0.109	0.112	0.091	0.108	0.110	0.121	0.109
		75	19	0.117	0.113	0.091	0.108	0.112	0.121	0.117	0.117	0.116	0.091	0.113	0.114	0.125	0.118
		100	20	0.122	0.120	0.041	0.122	0.120	0.129	0.122	0.122	0.120	0.041	0.122	0.120	0.129	0.122
		0	21	0.058	0.342	0.016	0.076	0.065	0.533	0.062	0.073	0.443	0.016	0.150	0.074	0.762	0.077
		25	22	0.282	0.395	0.074	0.246	0.251	0.631	0.280	0.296	0.474	0.074	0.303	0.257	0.806	0.296
	304.8	50	23	0.447	0.448	0.163	0.433	0.448	0.733	0.441	0.461	0.504	0.163	0.476	0.452	0.850	0.461
		75	24	0.554	0.501	0.252	0.623	0.646	0.845	0.546	0.565	0.534	0.252	0.651	0.648	0.893	0.566
0.1		100	25	0.586	0.554	0.343	0.810	0.844	0.931	0.585	0.586	0.564	0.343	0.824	0.844	0.936	0.598
		0	26	0.109	0.534	0.033	0.139	0.103	0.593	0.115	0.135	0.676	0.033	0.261	0.129	0.819	0.142
		25	27	0.342	0.606	0.090	0.314	0.300	0.694	0.341	0.366	0.715	0.090	0.411	0.316	0.863	0.366
	152.4	50	28	0.558	0.679	0.179	0.530	0.535	0.791	0.553	0.581	0.755	0.179	0.592	0.544	0.906	0.581
		75	29	0.729	0.752	0.268	0.753	0.773	0.890	0.721	0.746	0.795	0.268	0.790	0.777	0.954	0.746
		100	30	0.801	0.825	0.358	0.980	1.010	0.990	0.788	0.801	0.834	0.358	0.988	1.010	0.994	0.801
		0	31	0.368	0.918	0.822	0.399	0.510	0.960	0.381	0.433	1.038	0.822	0.640	0.596	1.091	0.44[illegible]
		25	32	0.549	0.990	0.753	0.515	0.572	1.031	0.549	0.617	1.080	0.753	0.728	0.641	1.132	0.617
	304.8	50	33	0.796	1.060	0.635	0.697	0.735	1.108	0.796	0.845	1.121	0.635	0.847	0.776	1.174	0.84[illegible]
		75	34	1.006	1.131	0.500	0.926	0.965	1.183	1.006	1.036	1.161	0.500	1.005	0.984	1.216	1.0[illegible]
10.0		100	35	1.115	1.202	0.361	1.175	1.197	1.258	1.110	1.114	1.202	0.361	1.175	1.197	1.258	1.11[illegible]
		0	36	0.547	0.918	0.878	0.544	0.680	0.972	0.547	0.621	1.038	0.878	0.801	0.777	1.099	0.[illegible]
		25	37	0.700	0.990	0.789	0.632	0.744	1.045	0.700	0.771	1.080	0.789	0.867	0.831	1.141	0.7[illegible]
	152.4	50	38	0.894	1.060	0.649	0.776	1.031	1.119	0.894	0.951	1.121	0.649	0.957	0.923	1.182	0.[illegible]
		75	39	1.098	1.131	0.502	0.977	1.024	1.193	1.098	1.120	1.161	0.502	1.076	1.055	1.224	1.1[illegible]
		100	40	1.185	1.202	0.358	1.195	1.200	1.263	1.185	1.185	1.202	0.358	1.195	1.200	1.263	1.1[illegible]

Table 60a. PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR TWO-HOUR TRIANGULAR RAINSTORM (cfs)

Slope, percent	Length, ft	Imperviousness, percent	Catchment number	Dry initial condition							Wet initial condition						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	0.31	1.25	1.87	0.3	0.46	1.10	0.31	0.40	1.44	1.87	0.7	0.50	1.48	0.40
		25	2	0.69	1.44	1.87	0.6	0.71	1.34	0.70	0.79	1.59	1.87	0.9	0.74	1.63	0.79
	316	50	3	1.18	1.63	1.87	1.1	1.14	1.59	1.18	1.26	1.73	1.87	1.3	1.15	1.77	1.27
		75	4	1.63	1.82	1.87	1.6	1.60	1.84	1.62	1.68	1.87	1.87	1.7	1.61	1.94	1.68
0.1		100	5	1.90	2.02	1.04	2.1	2.08	2.08	1.89	1.90	2.02	1.04	2.1	2.08	2.08	1.90
		0	6	0.50	1.40	1.88	0.5	0.73	1.17	0.50	0.63	1.58	1.88	0.9	0.81	1.52	0.63
		25	7	0.84	1.59	1.88	0.8	0.89	1.38	0.84	0.98	1.72	1.88	1.2	0.96	1.66	0.98
	158	50	8	1.32	1.78	1.88	1.2	1.23	1.62	1.32	1.44	1.87	1.88	1.5	1.27	1.80	1.44
		75	9	1.79	1.97	1.88	1.7	1.69	1.87	1.79	1.85	2.01	1.88	1.8	1.70	1.98	1.85
		100	10	2.07	2.16	1.03	2.1	2.18	2.12	2.07	2.07	2.16	1.03	2.1	2.18	2.12	2.07
		0	11	1.08	1.47	1.94	1.0	1.33	1.52	1.09	1.23	1.64	1.94	1.4	1.44	1.70	1.24
		25	12	1.36	1.66	1.94	1.2	1.52	1.70	1.36	1.50	1.79	1.94	1.6	1.59	1.84	1.50
	316	50	13	1.70	1.85	1.94	1.4	1.71	1.91	1.70	1.79	1.93	1.94	1.8	1.77	1.98	1.79
		75	14	2.02	2.04	1.94	1.8	1.95	2.08	2.02	2.04	2.08	1.94	2.0	1.99	2.15	2.05
10.0		100	15	2.21	2.22	1.05	2.2	2.19	2.30	2.21	2.21	2.22	1.05	2.2	2.21	2.30	2.21
		0	16	1.31	1.47	1.90	1.1	1.49	1.52	1.31	1.43	1.64	1.90	1.5	1.57	1.70	1.43
		25	17	1.56	1.66	1.90	1.3	1.67	1.70	1.56	1.63	1.79	1.90	1.7	1.72	1.84	1.64
	158	50	18	1.83	1.85	1.90	1.6	1.84	1.91	1.83	1.88	1.93	1.90	1.9	1.88	2.01	1.88
		75	19	2.07	2.04	1.90	1.8	2.02	2.08	2.07	2.07	2.08	1.90	2.1	2.05	2.15	2.08
		100	20	2.23	2.22	1.05	2.2	2.22	2.30	2.23	2.23	2.22	1.05	2.2	2.22	2.30	2.23
		0	21	1.18	7.84	0.54	1.3	1.26	8.72	1.18	1.59	9.99	0.54	3.0	1.44	12.96	1.59
		25	22	5.54	9.76	2.16	4.9	5.16	11.27	5.52	5.90	11.41	2.16	6.3	5.27	14.51	5.91
	1000	50	23	9.45	11.70	4.66	9.1	9.40	13.81	9.36	9.80	12.84	4.66	10.0	9.48	16.10	9.80
		75	24	12.48	13.65	7.17	13.2	13.65	16.46	12.32	12.77	14.26	7.17	13.8	13.68	17.66	12.77
0.1		100	25	14.21	15.59	9.65	17.5	17.89	19.14	13.95	14.21	15.69	9.65	17.6	17.89	19.18	14.21
		0	26	2.15	10.97	1.14	2.3	2.82	9.54	2.15	2.84	13.14	1.14	5.0	3.13	13.63	2.84
		25	27	6.30	12.93	2.73	5.7	6.21	12.01	6.30	6.96	14.59	2.73	8.0	6.41	15.22	6.96
	500	50	28	10.99	14.89	5.21	10.3	10.81	14.51	10.95	11.58	16.03	5.21	11.7	10.93	16.78	11.58
		75	29	14.95	16.85	7.70	14.9	15.46	17.13	14.84	15.40	17.48	7.70	15.7	15.51	18.29	15.40
		100	30	17.31	18.93	9.80	19.7	20.15	19.81	17.15	17.31	18.93	9.80	19.8	20.14	19.81	17.32
		0	31	6.59	14.75	17.22	6.3	9.87	14.59	6.59	8.03	16.44	17.22	11.0	10.62	16.70	8.03
		25	32	9.79	16.62	16.07	8.6	11.23	16.60	9.79	11.28	17.88	16.07	13.1	11.90	18.15	11.29
	1000	50	33	14.28	18.49	14.05	12.1	13.76	18.54	14.28	15.53	19.33	14.05	15.7	14.28	19.63	15.54
		75	34	18.66	20.36	11.89	16.7	17.81	20.55	18.66	19.19	20.78	11.89	18.5	18.04	21.08	19.19
10.0		100	35	21.16	22.23	9.81	21.8	22.17	22.57	21.15	21.15	22.23	9.81	21.8	22.17	22.57	21.16
		0	36	9.15	14.75	18.41	8.4	12.45	14.83	9.15	10.73	16.44	18.41	13.2	13.32	16.81	10.73
		25	37	12.09	16.62	16.85	10.4	14.04	16.78	12.09	13.59	17.88	16.85	15.1	14.81	18.26	13.57
	500	50	38	15.99	18.49	14.41	13.1	15.98	18.72	15.99	17.05	19.33	14.41	17.1	16.62	19.74	17.05
		75	39	19.57	20.36	11.97	17.2	18.79	20.70	19.57	20.04	20.78	11.97	19.6	19.17	21.19	20.04
		100	40	21.80	22.23	9.80	22.1	22.16	22.67	21.81	21.81	22.23	9.80	22.1	22.16	22.67	21.81

Table 60b. PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR TWO-HOUR TRIANGULAR RAINSTORM (m^3/sec)

Slope percent	Length, m	Imperviousness, percent	Catchment number	Dry initial condition							Wet initial condition						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	0.009	0.035	0.053	0.008	0.013	0.031	0.009	0.011	0.041	0.053	0.020	0.014	0.042	0.011
		25	2	0.020	0.041	0.053	0.017	0.020	0.038	0.020	0.022	0.045	0.053	0.025	0.021	0.046	0.022
	96.4	50	3	0.033	0.046	0.053	0.031	0.032	0.045	0.033	0.036	0.049	0.053	0.037	0.033	0.050	0.036
		75	4	0.046	0.052	0.053	0.045	0.045	0.052	0.046	0.048	0.053	0.053	0.048	0.046	0.055	0.048
0.1		100	5	0.054	0.057	0.029	0.059	0.059	0.059	0.054	0.054	0.057	0.029	0.059	0.059	0.059	0.054
		0	6	0.014	0.040	0.053	0.014	0.021	0.033	0.014	0.018	0.045	0.053	0.025	0.023	0.043	0.018
		25	7	0.024	0.045	0.053	0.023	0.025	0.039	0.024	0.028	0.049	0.053	0.034	0.027	0.047	0.028
	48.2	50	8	0.037	0.050	0.053	0.034	0.035	0.046	0.037	0.041	0.053	0.053	0.042	0.036	0.051	0.041
		75	9	0.051	0.056	0.053	0.048	0.048	0.053	0.051	0.052	0.057	0.053	0.051	0.048	0.056	0.052
		100	10	0.059	0.061	0.029	0.059	0.062	0.060	0.059	0.059	0.061	0.029	0.059	0.062	0.060	0.059
		0	11	0.031	0.042	0.055	0.028	0.038	0.043	0.031	0.035	0.046	0.055	0.040	0.041	0.048	0.03
		25	12	0.039	0.047	0.055	0.034	0.043	0.048	0.039	0.042	0.051	0.055	0.045	0.045	0.052	0.042
	96.4	50	13	0.048	0.052	0.055	0.040	0.048	0.054	0.048	0.051	0.055	0.055	0.051	0.050	0.056	0.051
		75	14	0.057	0.058	0.055	0.051	0.055	0.059	0.057	0.058	0.059	0.055	0.057	0.056	0.061	0.058
10.0		100	15	0.063	0.063	0.030	0.062	0.062	0.065	0.063	0.063	0.063	0.030	0.062	0.063	0.065	0.063
		0	16	0.037	0.042	0.054	0.031	0.042	0.043	0.037	0.040	0.046	0.054	0.042	0.044	0.048	0.040
		25	17	0.044	0.047	0.054	0.037	0.047	0.048	0.044	0.046	0.051	0.054	0.048	0.049	0.052	0.046
	48.2	50	18	0.052	0.052	0.054	0.045	0.052	0.054	0.052	0.053	0.055	0.054	0.054	0.053	0.057	0.053
		75	19	0.059	0.058	0.054	0.051	0.057	0.059	0.059	0.059	0.059	0.054	0.059	0.058	0.061	0.059
		100	20	0.063	0.063	0.030	0.062	0.063	0.065	0.063	0.063	0.063	0.030	0.062	0.063	0.065	0.063
		0	21	0.033	0.222	0.015	0.037	0.036	0.247	0.033	0.045	0.283	0.015	0.085	0.041	0.367	0.045
		25	22	0.157	0.276	0.061	0.139	0.146	0.319	0.156	0.167	0.323	0.061	0.178	0.149	0.411	0.167
	304.8	50	23	0.268	0.331	0.132	0.258	0.266	0.476	0.265	0.278	0.364	0.132	0.283	0.268	0.456	0.278
		75	24	0.353	0.387	0.203	0.374	0.387	0.466	0.349	0.362	0.404	0.203	0.391	0.387	0.500	0.362
0.1		100	25	0.402	0.442	0.273	0.496	0.507	0.542	0.395	0.402	0.444	0.273	0.498	0.507	0.543	0.402
		0	26	0.061	0.311	0.032	0.065	0.080	0.270	0.061	0.080	0.372	0.032	0.142	0.089	0.386	0.080
		25	27	0.178	0.366	0.077	0.161	0.176	0.340	0.178	0.197	0.413	0.077	0.227	0.182	0.431	0.197
	152.4	50	28	0.311	0.422	0.148	0.292	0.306	0.411	0.310	0.328	0.454	0.148	0.331	0.310	0.475	0.328
		75	29	0.423	0.477	0.218	0.422	0.438	0.485	0.420	0.436	0.495	0.218	0.445	0.439	0.518	0.436
		100	30	0.490	0.536	0.278	0.558	0.571	0.561	0.486	0.490	0.536	0.278	0.561	0.570	0.561	0.491
		0	31	0.187	0.418	0.488	0.178	0.280	0.413	0.187	0.227	0.466	0.488	0.312	0.301	0.473	0.227
		25	32	0.277	0.471	0.455	0.244	0.318	0.470	0.277	0.319	0.506	0.455	0.371	0.337	0.514	0.320
	304.8	50	33	0.404	0.524	0.398	0.343	0.390	0.525	0.404	0.440	0.547	0.398	0.445	0.404	0.556	0.440
		75	34	0.528	0.577	0.337	0.473	0.504	0.582	0.528	0.543	0.588	0.337	0.524	0.511	0.597	0.543
10.0		100	35	0.599	0.630	0.278	0.617	0.628	0.639	0.599	0.599	0.630	0.278	0.617	0.628	0.639	0.599
		0	36	0.259	0.418	0.521	0.238	0.353	0.420	0.259	0.304	0.466	0.521	0.374	0.377	0.476	0.304
		25	37	0.342	0.471	0.477	0.295	0.398	0.475	0.342	0.385	0.506	0.477	0.428	0.419	0.517	0.384
	152.4	50	38	0.453	0.524	0.408	0.371	0.453	0.530	0.453	0.483	0.547	0.408	0.484	0.471	0.559	0.483
		75	39	0.554	0.577	0.339	0.487	0.532	0.586	0.554	0.568	0.588	0.339	0.555	0.543	0.600	0.568
		100	40	0.617	0.630	0.278	0.626	0.628	0.642	0.618	0.618	0.630	0.278	0.626	0.628	0.642	0.618

Table 61a. PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR FOUR-HOUR TRIANGULAR STORM (cfs)

Slope, percent	Length, ft	Imperviousness, percent	Catchment number	Dry initial conditions							Wet initial conditions						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	0.06	0.46	0.32	0.0	0.15	0.25	0.06	0.11	0.50	0.32	0.2	0.15	0.49	0.12
		25	2	0.29	0.62	0.32	0.3	0.33	0.42	0.29	0.34	0.65	0.32	0.4	0.34	0.64	0.35
	316	50	3	0.56	0.77	0.32	0.5	0.58	0.64	0.56	0.61	0.79	0.32	0.6	0.58	7.77	0.61
		75	4	0.82	0.93	0.32	0.8	0.84	0.85	0.82	0.86	0.94	0.32	0.8	0.84	0.95	0.86
0.1		100	5	1.04	1.08	0.73	1.1	1.09	1.10	1.04	1.04	1.08	0.73	1.1	1.09	1.10	1.05
		0	6	0.10	0.50	0.17	0.1	0.25	0.25	0.10	0.19	0.54	0.17	0.3	0.26	0.49	0.19
		25	7	0.31	0.66	0.17	0.3	0.40	0.46	0.31	0.40	0.68	0.17	0.5	0.41	0.64	0.40
	158	50	8	0.58	0.81	0.17	0.6	0.62	0.64	0.58	0.66	0.83	0.17	0.7	0.62	0.81	0.66
		75	9	0.85	0.96	0.17	0.8	0.86	1.10	0.85	0.91	0.97	0.17	0.9	0.87	0.95	0.91
		100	10	1.09	1.11	0.71	1.1	1.11	0.57	1.09	1.09	1.11	0.71	1.1	1.11	1.10	1.09
		0	11	0.26	0.52	1.06	0.2	0.48	0.39	0.26	0.39	0.55	1.06	0.5	0.48	0.57	0.39
		25	12	0.44	0.67	1.06	0.3	0.63	0.57	0.45	0.58	0.70	1.06	0.6	0.63	0.71	0.58
	316	50	13	0.67	0.82	1.06	0.6	0.79	0.71	0.67	0.79	0.84	1.06	0.8	0.79	0.85	0.79
		75	14	0.92	0.98	1.06	0.8	0.96	0.88	0.92	0.98	0.99	1.06	1.0	0.96	0.99	0.98
10.0		100	15	1.13	1.13	0.73	1.1	1.14	1.17	1.14	1.13	1.13	0.73	1.1	1.14	1.17	1.14
		0	16	0.33	0.52	1.04	0.2	0.52	0.39	0.33	0.47	0.55	1.04	0.5	0.52	0.57	0.47
		25	17	0.51	0.67	1.04	0.4	0.67	0.57	0.52	0.64	0.70	1.04	0.7	0.67	0.71	0.64
	158	50	18	0.72	0.82	1.04	0.6	0.83	0.71	0.72	0.82	0.84	1.04	0.8	0.82	0.85	0.82
		75	19	0.95	0.98	1.04	0.9	0.98	0.88	0.95	0.99	0.99	1.04	1.0	0.98	0.99	0.99
		100	20	1.14	1.13	0.73	1.1	1.14	1.17	1.14	1.14	1.13	0.73	1.1	1.14	1.17	1.14
		0	21	0.20	3.20	0.53	0.1	0.47	1.70	0.20	0.44	3.92	0.53	0.8	0.47	4.24	0.44
		25	22	2.72	4.81	1.66	2.5	2.80	3.81	2.72	2.91	5.36	1.66	3.0	2.80	5.83	2.91
	1000	50	23	5.06	6.43	3.46	5.0	5.29	5.86	5.04	5.26	6.81	3.46	5.3	5.29	7.35	5.26
		75	24	7.10	8.06	5.26	7.4	7.77	8.02	7.05	7.28	8.26	5.26	7.6	7.77	8.90	7.28
0.1		100	25	8.77	9.66	6.92	9.9	10.26	10.38	8.68	8.77	9.71	6.92	10.0	10.26	10.38	8.77
		0	26	0.38	4.23	1.10	0.3	0.96	1.94	0.38	0.80	4.72	1.10	1.4	0.97	4.49	0.80
		25	27	2.84	5.76	2.17	2.7	3.09	3.99	2.84	3.20	6.17	2.17	3.5	3.10	6.04	3.20
	500	50	28	5.44	7.33	3.94	5.3	5.62	6.00	5.43	5.77	7.62	3.94	5.8	5.63	7.59	5.77
		75	29	7.85	8.91	5.70	8.0	8.17	8.16	7.83	8.15	9.07	5.70	8.2	8.18	9.08	8.15
		100	30	9.94	10.51	6.95	10.7	10.73	10.56	9.89	9.94	10.51	6.95	10.7	10.73	10.56	9.94
		0	31	1.36	5.20	9.43	0.9	3.33	3.64	1.36	2.42	5.51	9.43	3.4	3.39	5.54	2.42
		25	32	3.38	6.72	9.10	2.8	4.76	5.26	3.38	4.50	6.96	9.10	5.0	4.82	6.99	4.51
	1000	50	33	5.98	8.25	8.44	5.6	6.59	6.89	5.98	6.97	8.40	8.44	7.0	6.64	8.48	6.98
		75	34	8.70	9.77	7.65	8.4	8.87	8.58	8.70	9.34	9.85	7.65	9.1	8.90	9.92	9.34
10.0		100	35	11.08	11.30	6.95	11.2	11.22	11.41	11.09	11.08	11.30	6.95	11.2	11.22	11.41	11.09
		0	36	2.05	5.20	10.11	1.4	4.34	3.74	2.05	3.34	5.51	10.11	4.2	4.38	5.58	3.34
		25	37	3.99	6.72	9.55	3.0	5.77	5.33	3.99	5.30	6.96	9.55	5.8	5.81	7.03	5.30
	500	50	38	6.35	8.25	8.64	5.6	7.44	6.96	6.35	7.56	8.40	8.64	7.5	7.49	8.51	7.56
		75	39	8.99	9.77	7.71	8.5	9.38	8.62	8.99	9.61	9.85	7.71	9.3	9.41	9.96	9.61
		100	40	11.24	11.30	6.95	11.3	11.41	11.41	11.24	11.24	11.30	6.95	11.3	11.41	11.41	11.24

Table 61b. PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR FOUR-HOUR TRIANGULAR STORM (m^3/sec)

Slope percent	Length, m	Imperviousness, percent	Catchment number	Dry initial condition							Wet initial condition						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	0.002	0.013	0.009	0.000	0.004	0.007	0.002	0.003	0.014	0.009	0.006	0.004	0.014	0.003
		25	2	0.008	0.018	0.009	0.008	0.009	0.012	0.008	0.010	0.018	0.009	0.011	0.010	0.018	0.010
	96.4	50	3	0.016	0.022	0.009	0.014	0.016	0.018	0.016	0.017	0.022	0.009	0.017	0.016	0.220	0.017
		75	4	0.023	0.026	0.009	0.023	0.024	0.024	0.023	0.024	0.027	0.009	0.023	0.024	0.027	0.024
0.1		100	5	0.029	0.031	0.021	0.031	0.031	0.031	0.029	0.029	0.031	0.021	0.031	0.031	0.031	0.030
		0	6	0.003	0.014	0.005	0.003	0.007	0.007	0.003	0.005	0.015	0.005	0.008	0.007	0.014	0.005
		25	7	0.009	0.019	0.005	0.008	0.011	0.013	0.009	0.011	0.019	0.005	0.014	0.012	0.018	0.011
	48.2	50	8	0.016	0.023	0.005	0.017	0.018	0.018	0.016	0.019	0.024	0.005	0.020	0.018	0.023	0.019
		75	9	0.024	0.027	0.005	0.023	0.024	0.031	0.024	0.026	0.027	0.005	0.025	0.025	0.027	0.026
		100	10	0.031	0.031	0.020	0.031	0.031	0.016	0.031	0.031	0.031	0.020	0.031	0.031	0.031	0.031
		0	11	0.007	0.015	0.030	0.006	0.014	0.011	0.007	0.011	0.016	0.030	0.014	0.014	0.016	0.011
		25	12	0.012	0.019	0.030	0.008	0.018	0.016	0.013	0.016	0.020	0.030	0.017	0.018	0.020	0.016
	96.4	50	13	0.019	0.023	0.030	0.017	0.022	0.020	0.019	0.022	0.024	0.030	0.023	0.022	0.024	0.022
		75	14	0.026	0.028	0.030	0.023	0.027	0.025	0.026	0.028	0.028	0.030	0.028	0.027	0.028	0.028
10.0		100	15	0.032	0.032	0.021	0.031	0.032	0.033	0.032	0.032	0.032	0.021	0.031	0.032	0.033	0.032
		0	16	0.009	0.015	0.029	0.006	0.015	0.011	0.009	0.013	0.016	0.029	0.014	0.015	0.016	0.013
		25	17	0.014	0.019	0.029	0.011	0.019	0.016	0.015	0.018	0.020	0.029	0.020	0.019	0.020	0.018
	48.2	50	18	0.020	0.023	0.029	0.017	0.024	0.020	0.020	0.023	0.024	0.029	0.023	0.023	0.024	0.023
		75	19	0.027	0.028	0.029	0.025	0.028	0.025	0.027	0.028	0.028	0.029	0.028	0.028	0.028	0.028
		100	20	0.032	0.032	0.021	0.031	0.032	0.033	0.032	0.032	0.032	0.021	0.031	0.032	0.033	0.032
		0	21	0.006	0.091	0.015	0.003	0.013	0.048	0.006	0.012	0.111	0.015	0.023	0.013	0.120	0.012
		25	22	0.077	0.136	0.047	0.071	0.079	0.108	0.077	0.082	0.152	0.047	0.085	0.079	0.165	0.082
	304.8	50	23	0.143	0.182	0.098	0.142	0.150	0.166	0.143	0.149	0.193	0.098	0.150	0.150	0.208	0.149
		75	24	0.201	0.228	0.149	0.210	0.220	0.227	0.200	0.206	0.234	0.149	0.215	0.220	0.252	0.206
0.1		100	25	0.248	0.274	0.196	0.280	0.291	0.294	0.246	0.248	0.275	0.196	0.283	0.291	0.294	0.248
		0	26	0.011	0.120	0.031	0.008	0.027	0.055	0.011	0.023	0.134	0.031	0.040	0.027	0.127	0.023
		25	27	0.080	0.163	0.061	0.076	0.088	0.113	0.080	0.091	0.175	0.061	0.099	0.088	0.171	0.091
	152.4	50	28	0.154	0.208	0.112	0.150	0.159	0.170	0.154	0.163	0.216	0.112	0.164	0.159	0.215	0.163
		75	29	0.222	0.252	0.161	0.227	0.231	0.231	0.222	0.231	0.257	0.161	0.232	0.232	0.257	0.231
		100	30	0.282	0.298	0.197	0.303	0.304	0.299	0.280	0.282	0.298	0.197	0.303	0.304	0.299	0.282
		0	31	0.039	0.147	0.267	0.025	0.094	0.103	0.039	0.069	0.156	0.267	0.096	0.096	0.157	0.069
		25	32	0.096	0.190	0.258	0.079	0.135	0.149	0.096	0.127	0.197	0.258	0.142	0.137	0.198	0.128
	304.8	50	33	0.169	0.234	0.239	0.159	0.187	0.195	0.169	0.197	0.238	0.239	0.198	0.188	0.240	0.198
		75	34	0.246	0.277	0.217	0.238	0.251	0.243	0.246	0.265	0.279	0.217	0.258	0.252	0.281	0.265
10.0		100	35	0.314	0.320	0.197	0.317	0.318	0.323	0.314	0.314	0.320	0.197	0.317	0.318	0.323	0.314
		0	36	0.058	0.147	0.286	0.040	0.123	0.106	0.058	0.095	0.156	0.286	0.117	0.124	0.158	0.095
		25	37	0.113	0.190	0.270	0.085	0.163	0.151	0.113	0.150	0.197	0.270	0.164	0.165	0.199	0.150
	152.4	50	38	0.180	0.234	0.245	0.159	0.211	0.197	0.180	0.214	0.238	0.245	0.212	0.212	0.241	0.214
		75	39	0.255	0.277	0.218	0.241	0.266	0.244	0.255	0.272	0.279	0.218	0.263	0.266	0.282	0.272
		100	40	0.318	0.320	0.197	0.320	0.323	0.323	0.318	0.318	0.320	0.197	0.320	0.323	0.323	0.318

Table 62. TIMES OF PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR ONE-HOUR TRIANGULAR RAINSTORM (minutes)

Slope, percent	Length, ft	Imperviousness, percent	Catchment number	Dry initial condition							Wet initial conditions						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	55	50	55	50	55	42	60	55	45	55	50	55	39	60
		25	2	45	45	55	45	39	41	45	50	45	55	40	39	38	45
	316	50	3	45	45	55	40	36	40	45	50	45	55	40	36	38	45
		75	4	45	45	55	40	36	39	45	45	45	55	40	36	38	45
0.1		100	5	45	45	45	35	36	38	45	45	45	45	40	36	38	45
		0	6	55	40	55	50	52	40	55	55	40	55	45	52	37	55
		25	7	45	40	55	45	40	39	45	50	40	55	40	40	37	45
	158	50	8	45	40	55	40	34	39	45	45	40	55	40	36	37	45
		75	9	45	40	55	35	34	38	45	50	40	55	35	34	37	45
		100	10	45	40	45	35	34	37	45	45	40	45	35	34	37	45
		0	11	50	35	40	40	39	31	50	45	35	40	40	37	31	45
		25	12	45	35	40	40	37	31	45	45	35	40	35	35	31	45
	316	50	13	40	35	40	35	35	31	40	40	35	40	35	35	31	40
		75	14	40	35	40	35	35	31	40	40	35	40	35	35	31	40
10.0		100	15	40	35	45	35	33	31	40	40	35	45	35	33	31	40
		0	16	45	35	40	40	37	31	45	45	35	40	35	35	31	45
		25	17	45	35	40	40	35	31	45	40	35	40	35	35	31	40
	158	50	18	40	35	40	35	35	31	40	40	35	40	35	35	31	40
		75	19	40	35	40	35	33	31	40	40	35	40	35	35	31	40
		100	20	40	35	45	35	33	31	40	40	35	45	35	33	31	40
		0	21	60	75	60	55	59	49	60	60	75	60	55	59	44	60
		25	22	45	75	50	45	48	47	50	50	75	50	45	48	44	50
	1000	50	23	50	75	50	45	44	46	50	50	75	50	45	44	44	50
		75	24	50	75	50	45	44	45	55	50	75	50	45	44	44	55
0.1		100	25	50	75	50	45	44	44	55	50	75	50	45	44	44	55
		0	26	60	60	70	55	55	46	60	60	55	70	55	55	42	60
		25	27	45	55	55	45	41	45	45	45	55	55	45	41	42	45
	500	50	28	45	55	50	40	38	44	45	45	55	50	40	38	42	45
		75	29	50	55	50	40	38	43	50	50	55	50	40	38	42	50
		100	30	50	55	45	40	38	42	50	50	55	45	40	38	42	50
		0	31	55	35	40	50	47	33	55	50	35	40	45	47	32	55
		25	32	45	35	40	45	45	33	45	45	35	40	40	45	32	45
	1000	50	33	45	35	40	40	37	32	45	45	35	40	40	37	32	45
		75	34	45	35	40	35	35	32	45	40	35	40	35	35	32	40
10.0		100	35	40	35	45	35	35	32	40	40	35	45	35	35	32	40
		0	36	50	35	40	45	42	32	50	50	35	40	40	40	32	50
		25	37	45	35	40	40	40	32	45	45	35	40	40	39	32	45
	500	50	38	45	35	40	40	36	32	45	40	35	40	35	36	32	40
		75	39	40	35	45	35	34	32	40	40	35	45	35	34	32	40
		100	40	40	35	45	35	34	31	40	40	35	45	35	34	31	40

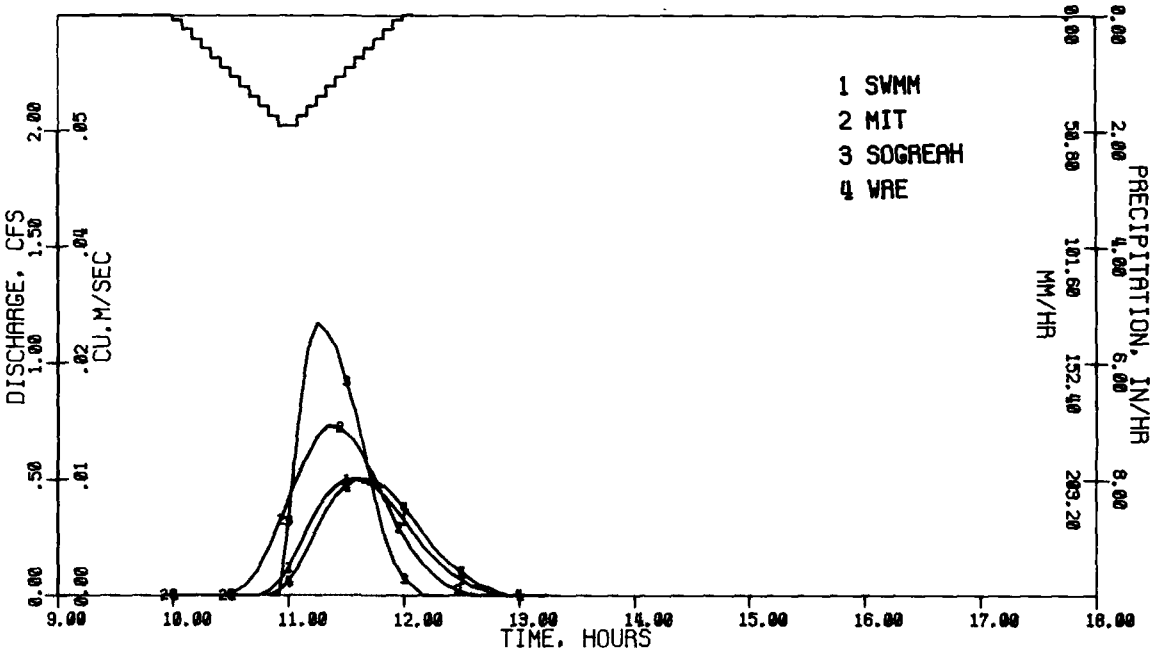
Table 63. TIMES OF PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR TWO-HOUR TRIANGULAR STORM (minutes)

Slope, percent	Length, ft	Imperviousness, percent	Catchment number	Dry initial conditions							Wet initial conditions						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	100	80	70	85	96	79	100	100	75	70	90	96	71	100
		25	2	80	80	70	70	75	77	80	80	75	70	70	75	71	80
	316	50	3	75	80	70	70	63	75	75	75	75	70	70	63	70	80
		75	4	80	80	70	70	63	72	80	80	75	70	70	63	70	80
0.1		100	5	80	75	80	70	61	70	80	80	75	80	70	61	70	80
		0	6	95	70	75	85	89	77	95	95	70	75	75	87	69	95
		25	7	80	70	75	85	79	75	80	80	70	75	75	79	69	80
	158	50	8	75	70	75	70	64	73	75	75	70	75	70	64	69	75
		75	9	75	70	75	70	58	71	75	75	70	75	65	60	68	75
		100	10	75	70	85	65	58	68	75	75	70	85	65	58	68	75
		0	11	85	65	70	80	67	62	85	80	65	70	65	65	61	80
		25	12	30	65	70	75	65	62	80	75	65	70	65	63	61	75
	316	50	13	75	65	70	70	61	62	75	70	65	70	65	61	61	70
		75	14	70	65	70	65	59	62	70	70	65	70	65	59	61	70
10.0		100	15	70	65	80	60	56	61	70	70	65	80	60	56	61	70
		0	16	80	65	70	75	61	62	80	75	65	70	65	59	61	75
		25	17	75	65	70	70	59	62	75	75	65	70	65	59	61	70
	158	50	18	70	65	70	70	59	61	70	70	65	70	65	59	61	70
		75	19	70	65	70	65	59	61	70	70	65	70	65	57	61	70
		100	20	70	65	80	60	57	51	70	70	65	80	60	57	61	70
		0	21	110	115	115	100	100	89	110	110	110	115	95	100	78	110
		25	22	80	110	95	80	76	86	80	80	110	95	85	76	78	80
	1000	50	23	85	110	90	80	76	83	85	85	110	90	80	76	77	85
		75	24	90	110	90	75	76	80	90	90	110	90	75	76	77	90
0.1		100	25	90	110	90	75	76	77	90	90	110	90	75	76	76	90
		0	26	105	90	120	100	99	85	105	105	85	120	95	99	75	105
		25	27	80	90	100	75	71	83	80	80	85	100	80	71	75	80
	500	50	28	80	90	95	75	68	80	80	80	85	95	75	68	74	80
		75	29	80	90	95	70	65	77	80	80	85	95	70	65	74	80
		100	30	85	85	90	70	65	74	85	85	85	90	70	65	74	85
		0	31	95	65	70	90	81	65	95	90	65	70	80	81	63	90
		25	32	85	65	70	85	77	64	85	80	65	70	75	74	63	80
	1000	50	33	75	65	70	70	65	64	75	75	65	70	70	64	63	75
		75	34	75	65	75	65	58	63	75	75	65	75	65	58	63	75
10.0		100	35	75	65	90	65	58	62	75	75	65	90	65	58	62	75
		0	36	90	65	70	85	70	64	90	85	65	70	75	68	62	85
		25	37	80	65	70	80	68	63	80	75	65	70	70	66	62	75
	500	50	38	75	65	75	70	64	63	75	75	65	75	70	64	62	75
		75	39	70	65	80	65	58	62	70	70	65	80	65	58	62	70
		100	40	70	65	90	65	58	62	70	70	65	90	65	58	62	70

Table 64. TIMES OF PEAK RUNOFF FROM HYPOTHETICAL CATCHMENTS FOR FOUR-HOUR TRIANGULAR RAINSTORM (minutes)

Slope, percent	Length, ft	Imperviousness percent	Catchment number	Dry initial conditions							Wet initial conditions						
				SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
		0	1	175	140	200	*	177	161	175	160	135	200	140	177	135	175
		25	2	135	140	200	120	140	155	135	135	135	200	130	140	134	140
	316	50	3	135	140	200	110	131	149	135	140	135	200	120	131	133	135
		75	4	140	140	200	120	128	142	140	140	135	200	115	128	132	140
0.1		100	5	135	135	150	125	128	132	140	135	135	150	125	128	132	140
		0	6	170	130	195	160	166	158	170	170	130	195	140	166	132	165
		25	7	135	130	195	115	148	153	135	140	130	195	140	148	132	140
	158	50	8	135	130	195	125	128	148	135	135	130	195	130	128	131	135
		75	9	135	130	195	115	126	141	140	135	130	195	125	126	130	135
		100	10	135	125	150	120	126	122	135	135	125	150	120	126	130	135
		0	11	160	125	135	155	134	139	160	145	125	135	130	134	122	145
		25	12	145	125	135	110	130	137	150	135	125	135	120	130	122	135
	316	50	13	140	125	135	120	126	137	140	135	125	135	125	126	122	135
		75	14	135	125	135	110	124	137	135	130	125	135	125	124	122	130
10.0		100	15	130	125	150	115	124	122	130	130	125	150	115	124	122	130
		0	16	150	125	135	150	128	138	150	140	125	135	120	128	122	140
		25	17	145	125	135	150	126	137	145	135	125	135	125	126	122	135
	158	50	18	140	125	135	120	124	136	140	130	125	135	120	124	121	130
		75	19	135	125	135	125	124	135	135	130	125	135	120	124	121	130
		100	20	130	125	150	115	124	121	130	130	125	150	115	124	121	130
		0	21	180	175	225	155	178	173	180	180	170	225	155	178	145	180
		25	22	140	175	175	135	141	164	140	145	170	175	145	141	143	140
	1000	50	23	145	175	170	140	141	156	150	150	170	175	140	141	142	150
		75	24	155	170	170	135	141	148	155	155	170	170	135	141	140	155
0.1		100	25	155	175	170	135	141	140	160	155	170	170	140	141	140	155
		0	26	180	155	230	180	181	169	180	180	145	230	160	181	141	175
		25	27	140	150	195	135	141	161	140	140	145	195	145	141	139	140
	500	50	28	140	150	170	130	133	154	140	140	145	170	135	133	138	140
		75	29	145	150	170	135	133	146	145	145	145	170	135	133	137	145
		100	30	145	145	160	135	132	137	145	145	145	160	135	133	137	145
		0	31	175	125	135	165	154	142	175	165	125	135	145	154	124	160
		25	32	155	125	135	125	146	140	155	145	125	135	135	146	123	145
	1000	50	33	140	125	135	125	128	139	140	135	125	135	130	128	123	135
		75	34	135	125	140	125	126	137	135	135	125	140	130	128	123	135
10.0		100	35	135	125	155	125	124	123	135	135	125	155	125	124	123	135
		0	36	165	125	135	165	143	141	165	155	125	135	135	143	123	155
		25	37	155	125	135	150	137	139	155	140	125	135	135	137	123	140
	500	50	38	140	125	135	125	129	138	140	135	125	135	130	129	123	135
		75	39	135	125	145	125	125	136	135	135	125	145	125	125	123	130
		100	40	130	125	160	125	125	123	130	130	125	160	125	125	123	130

* not computed



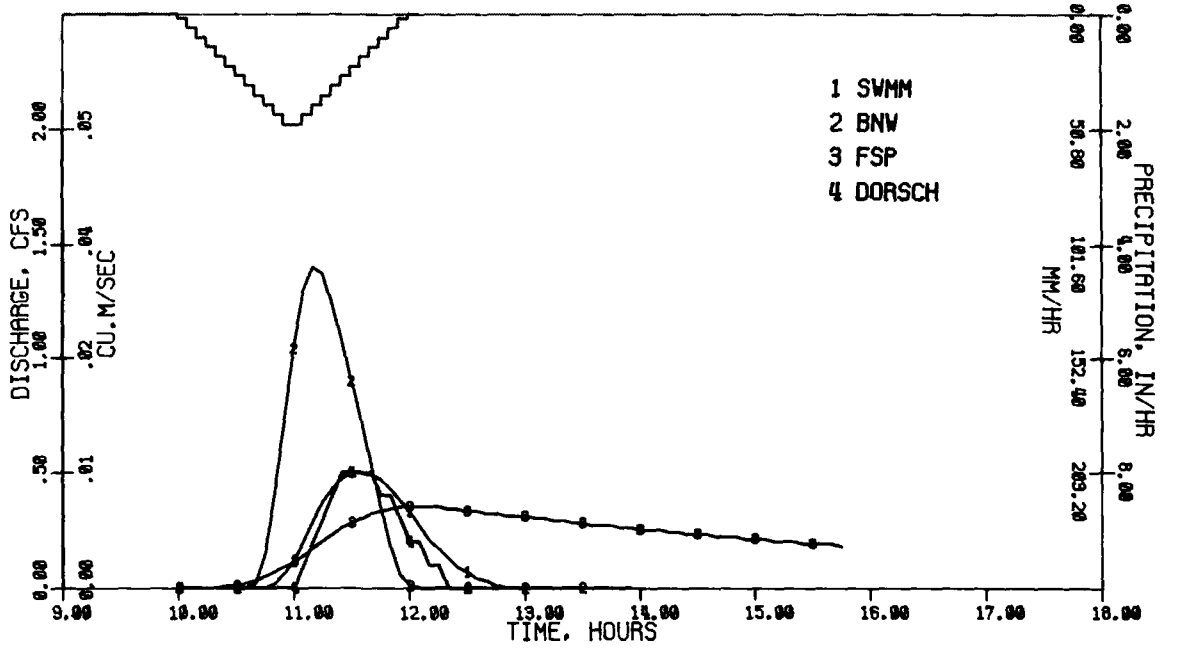
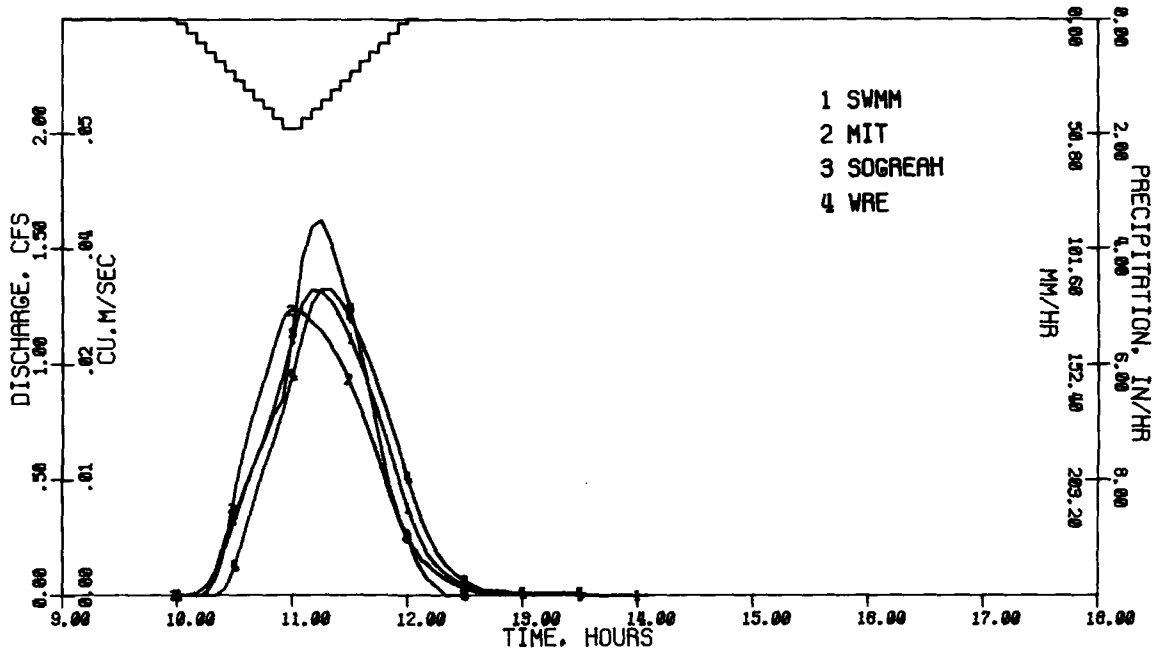


Figure 31. Runoff for Two-Hour Triangular Rainstorm - Small Hypothetical Catchment, 48 m (158 ft) Long, 0.1% Slope, 0% Imperviousness (Catchment #6)



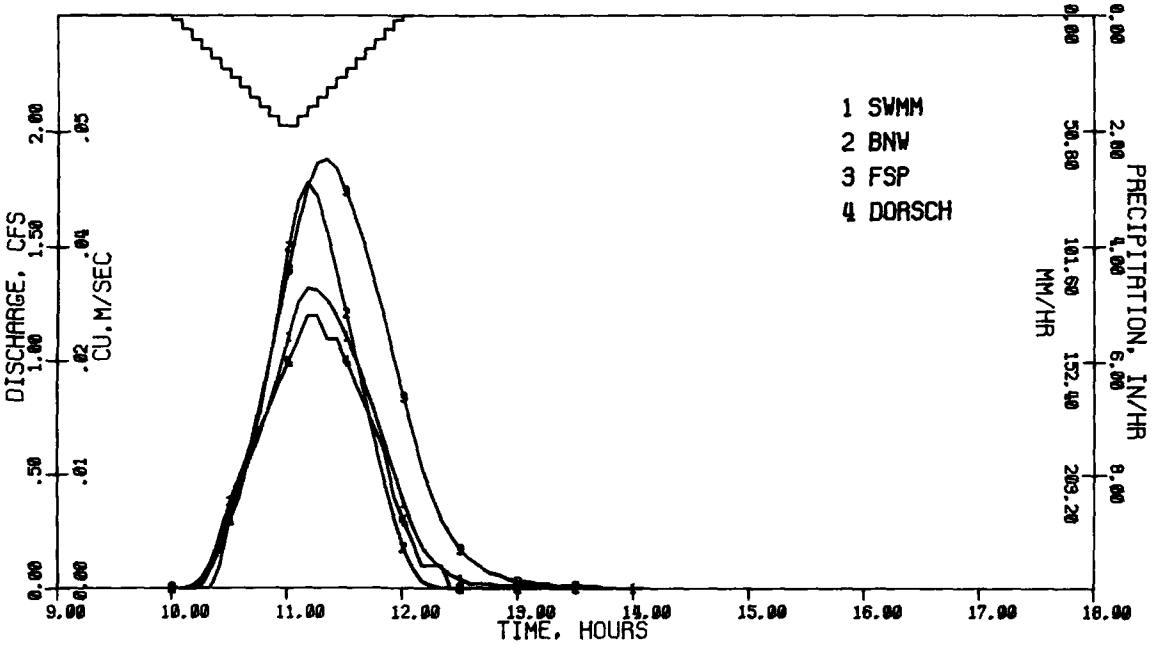
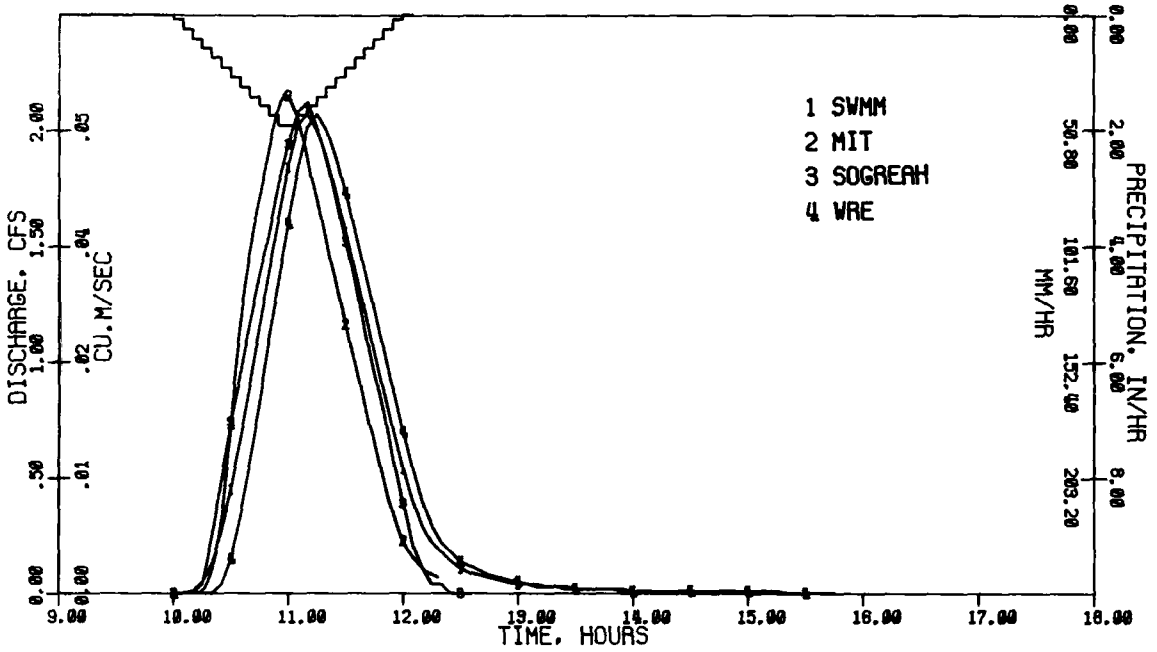


Figure 32. Runoff for Two-Hour Triangular Rainstorm - Small Hypothetical Catchment, 48 m (158 ft) Long, 0.1% Slope, 50% Imperviousness (Catchment #8)



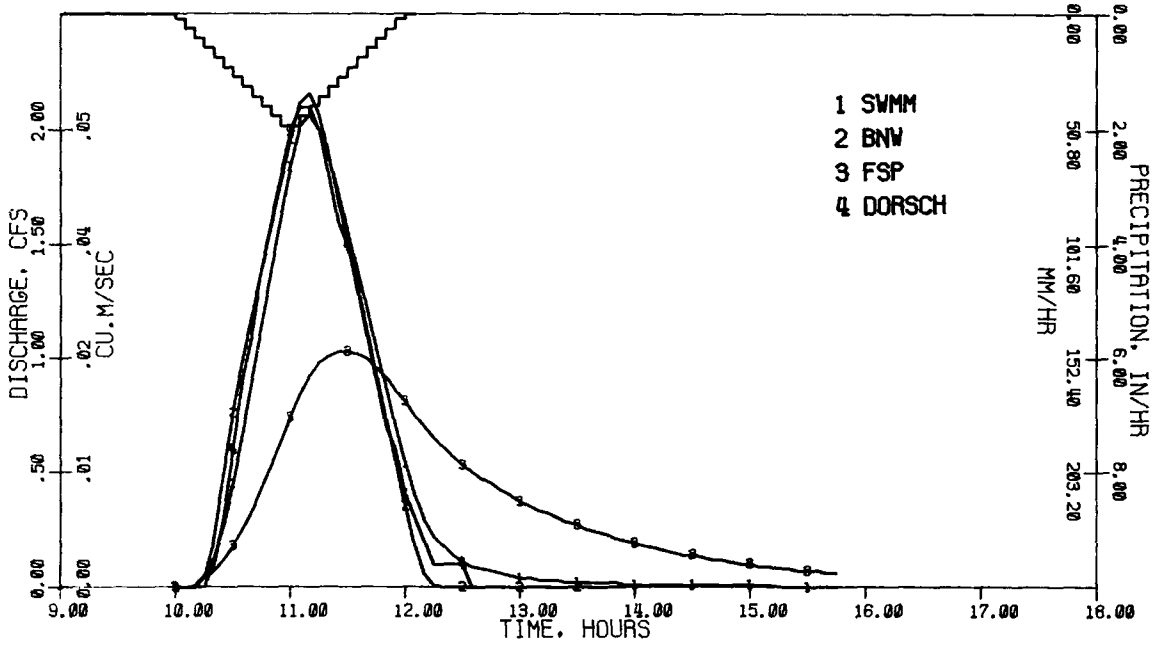
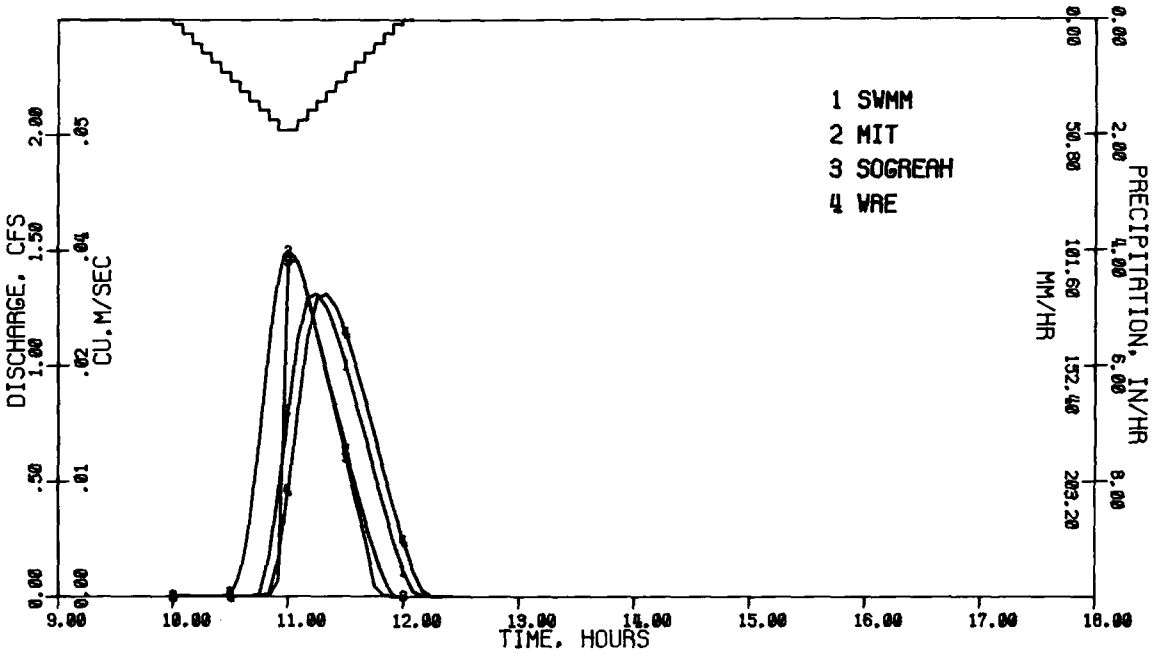


Figure 33. Runoff for Two-Hour Triangular Rainstorm - Small Hypothetical Catchment, 48 m (158 ft) Long, 0.1% Slope, 100% Imperviousness (Catchment #10)



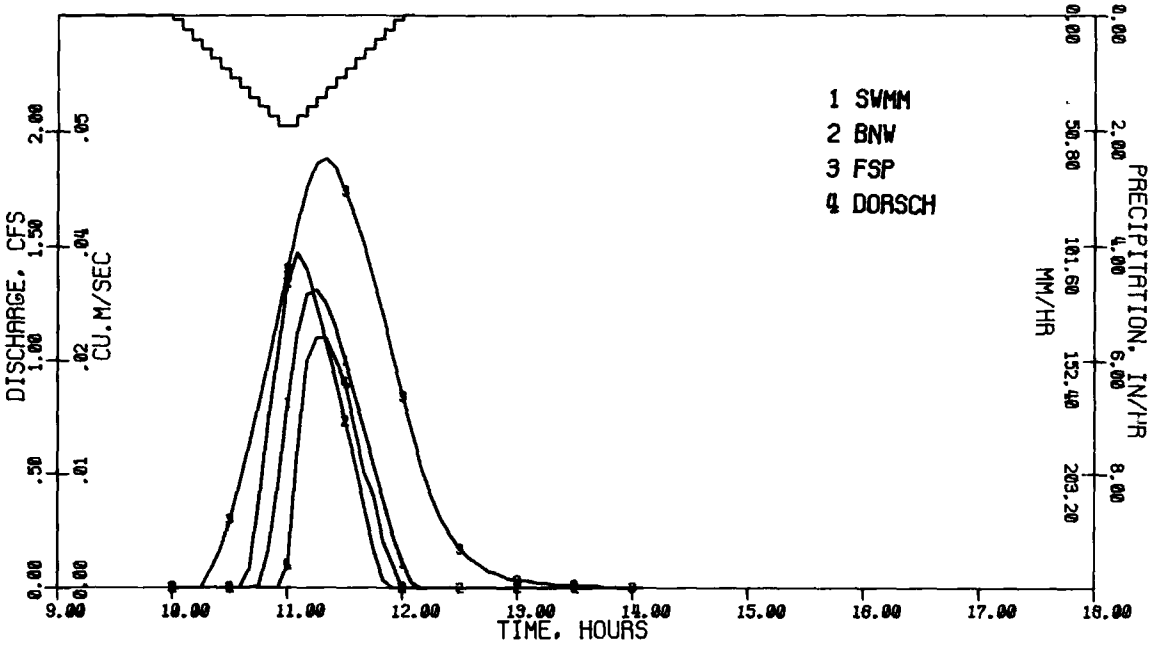
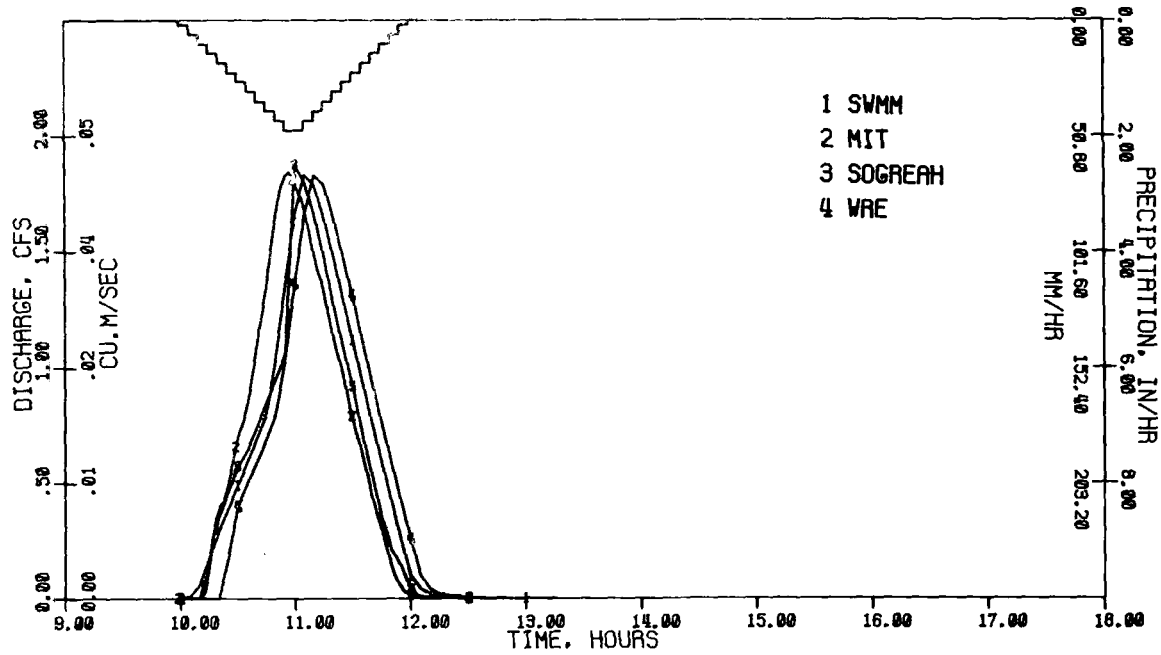


Figure 34. Runoff for Two-Hour Triangular Rainstorm - Small Hypothetical Catchment, 48 m (158 ft) Long, 10% Slope, 0% Imperviousness (Catchment #16)



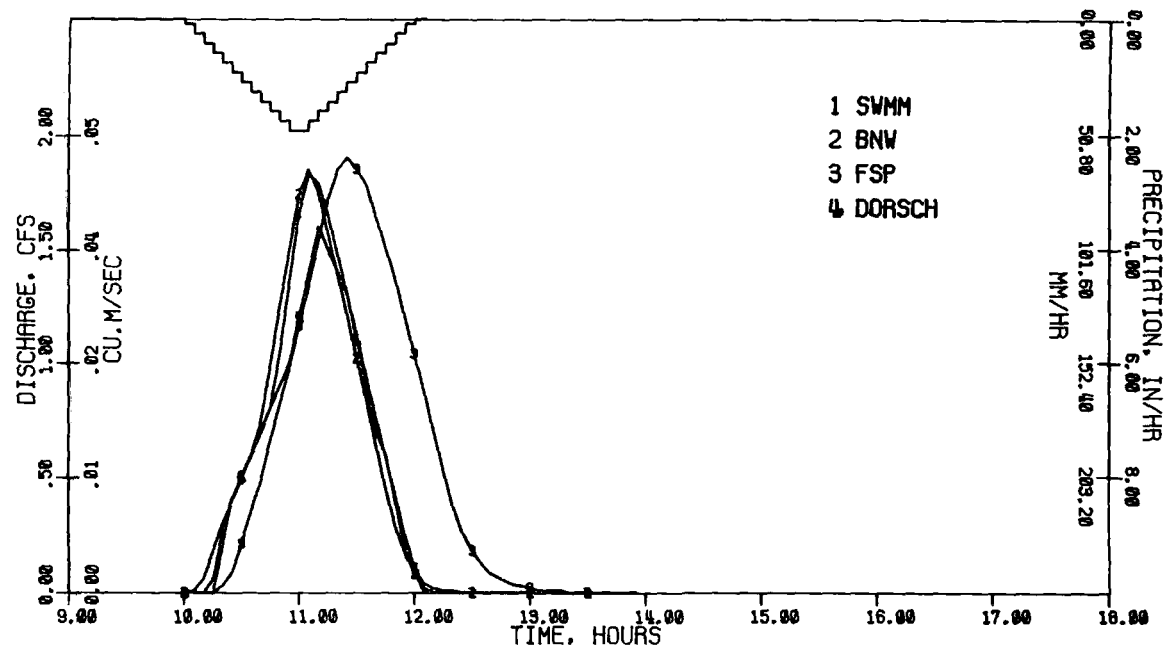
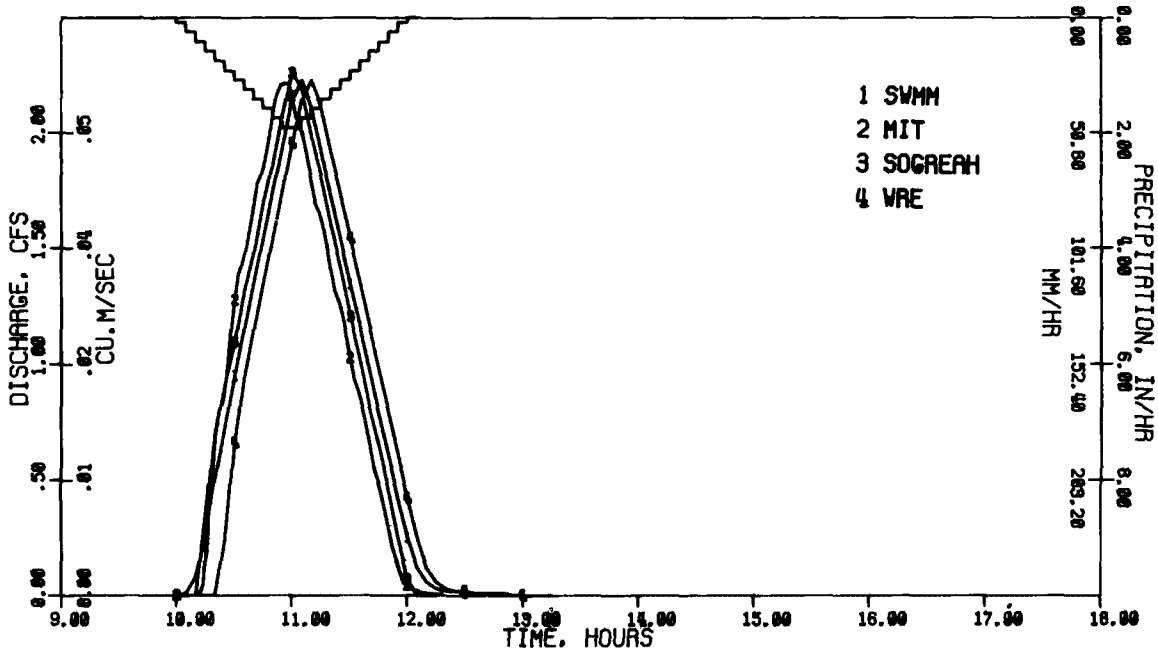


Figure 35. Runoff for Two-Hour Triangular Rainstorm - Small Hypothetical Catchment, 48 m (158 ft) Long, 10% Slope, 50% Imperviousness (Catchment #18)



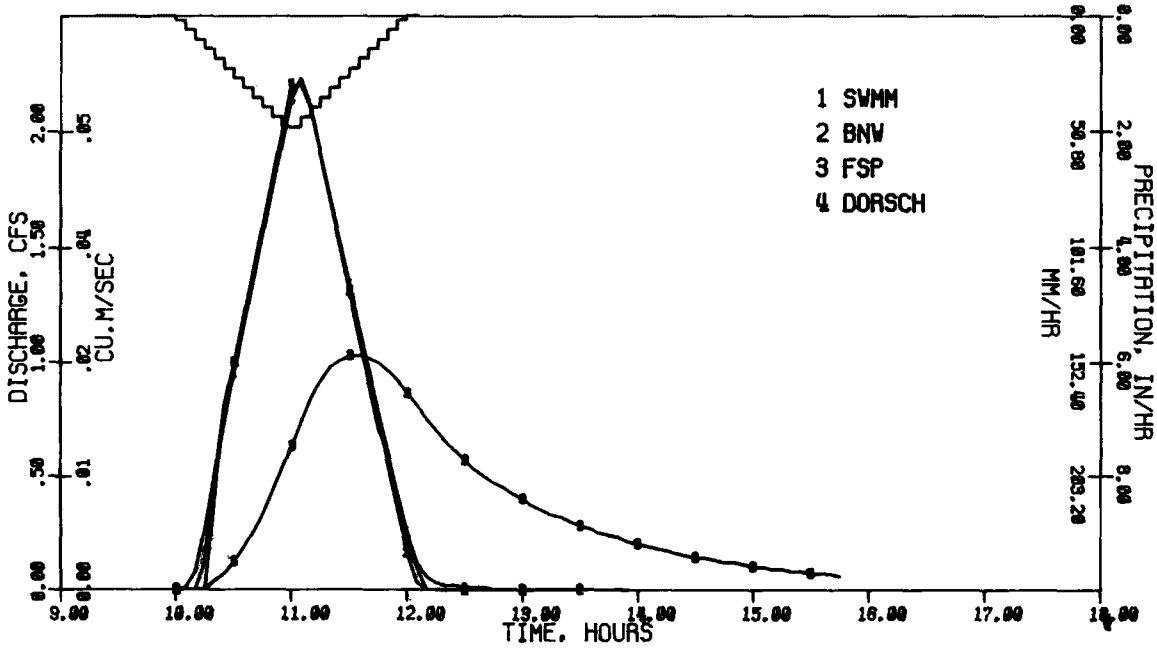
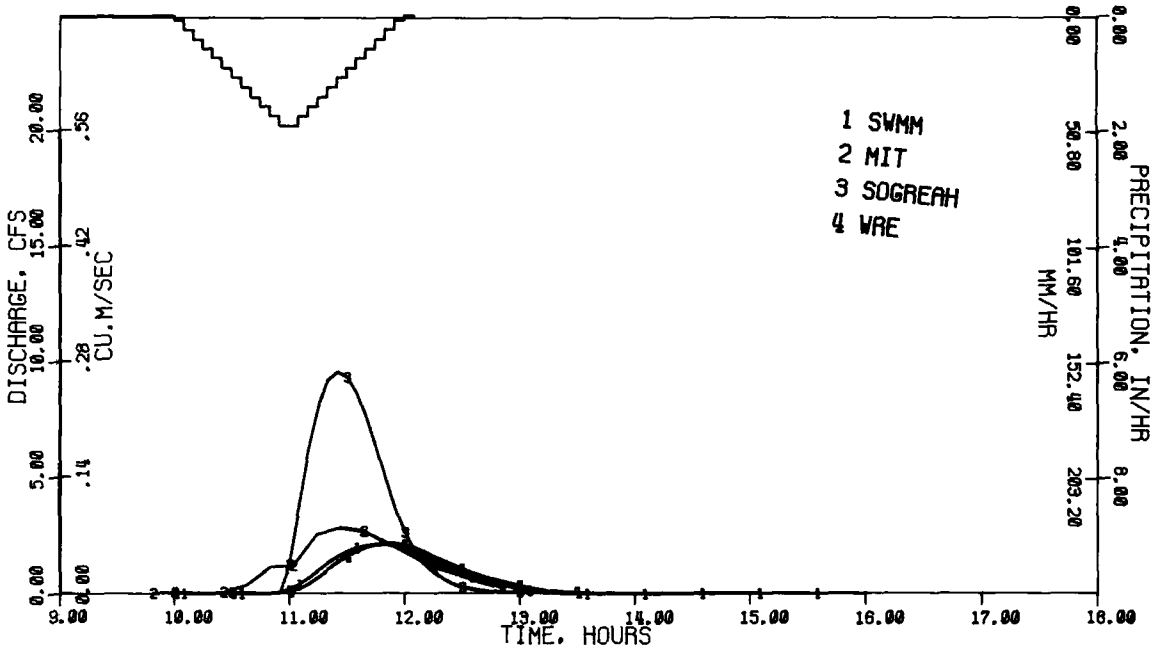


Figure 36. Runoff for Two-Hour Triangular Rainstorm - Small Hypothetical Catchment, 48 m (158 ft) Long, 10% Slope, 100% Imperviousness (Catchment #20)



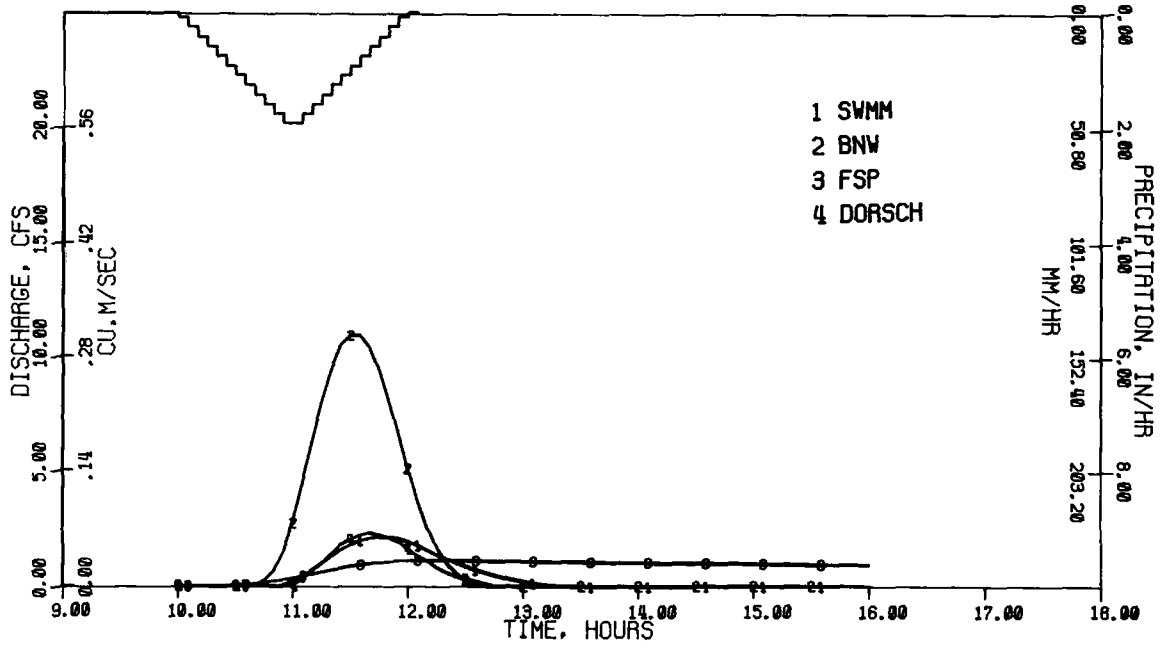
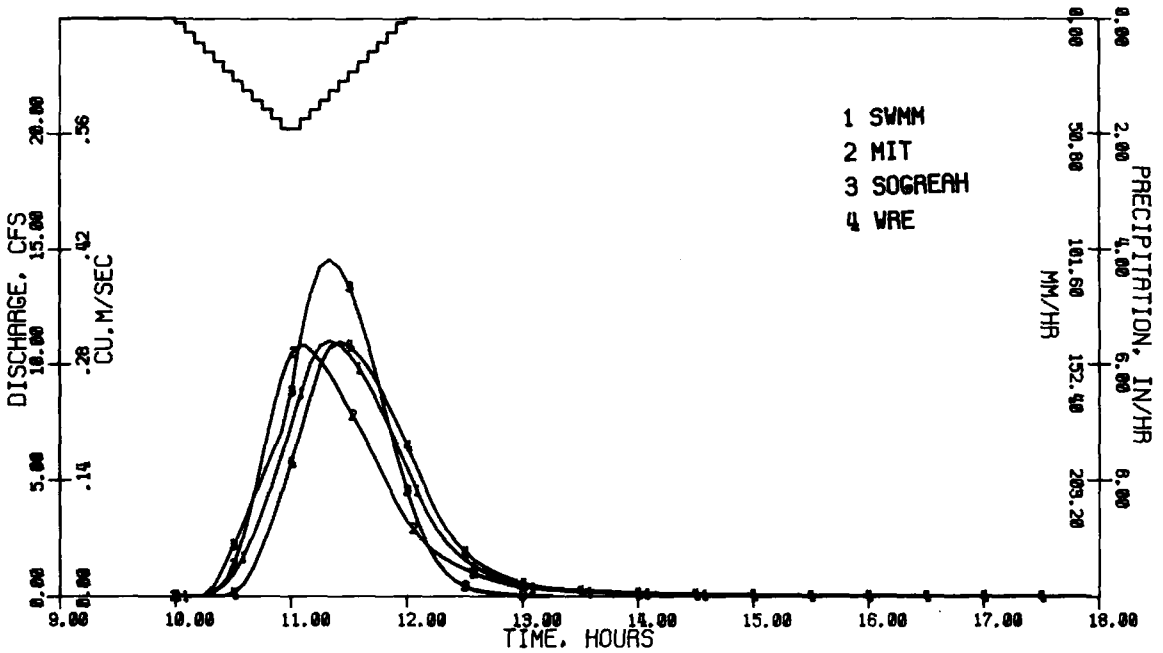


Figure 37. Runoff for Two-Hour Triangular Rainstorm - Large Hypothetical Catchment, 152 m (500 ft) Long, 0.1% Slope, 0% Imperviousness (Catchment #26)



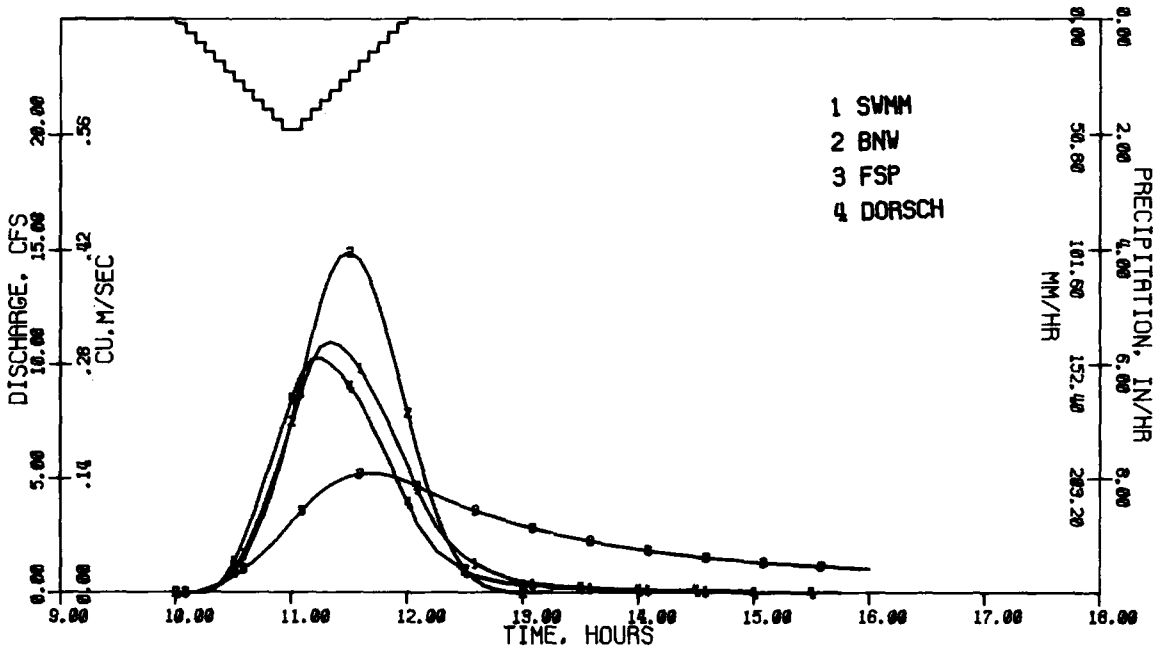
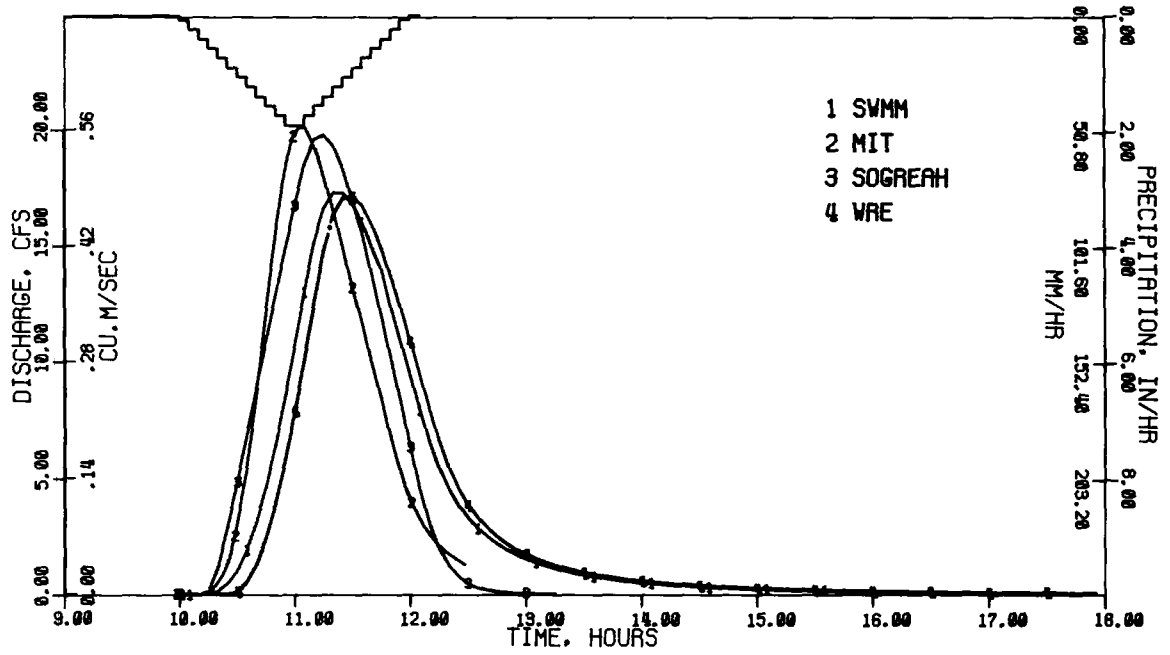


Figure 38. Runoff for Two-Hour Triangular Rainstorm - Large Hypothetical Catchment, 152 m (500 ft) Long, 0.1% Slope, 50% Imperviousness (Catchment #28)



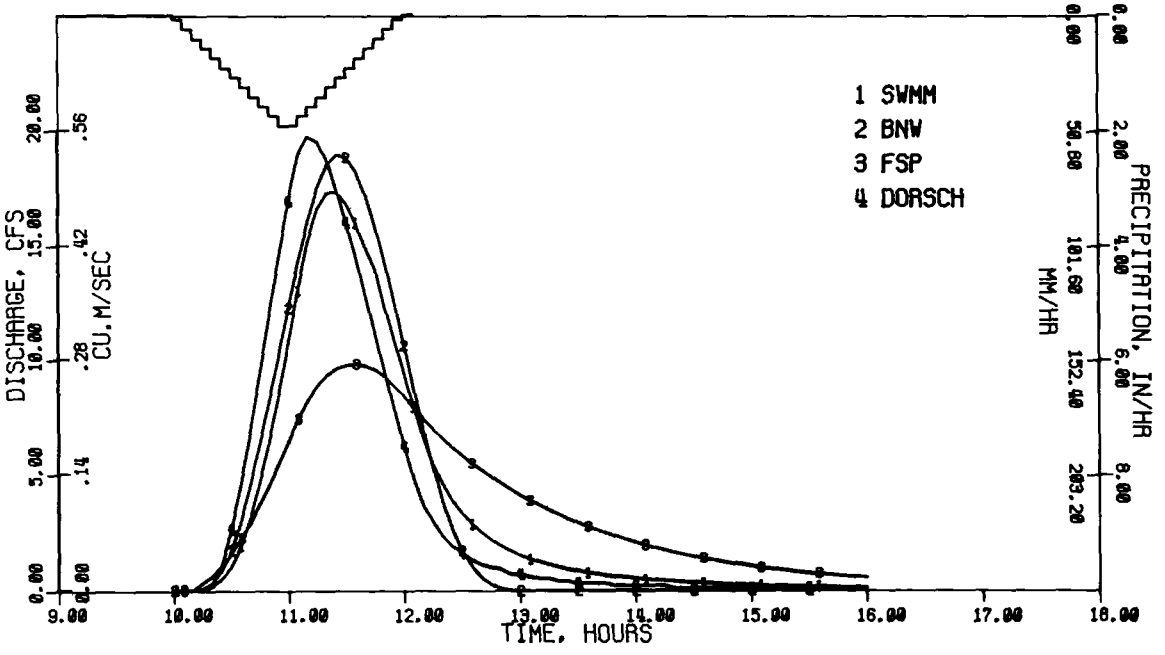
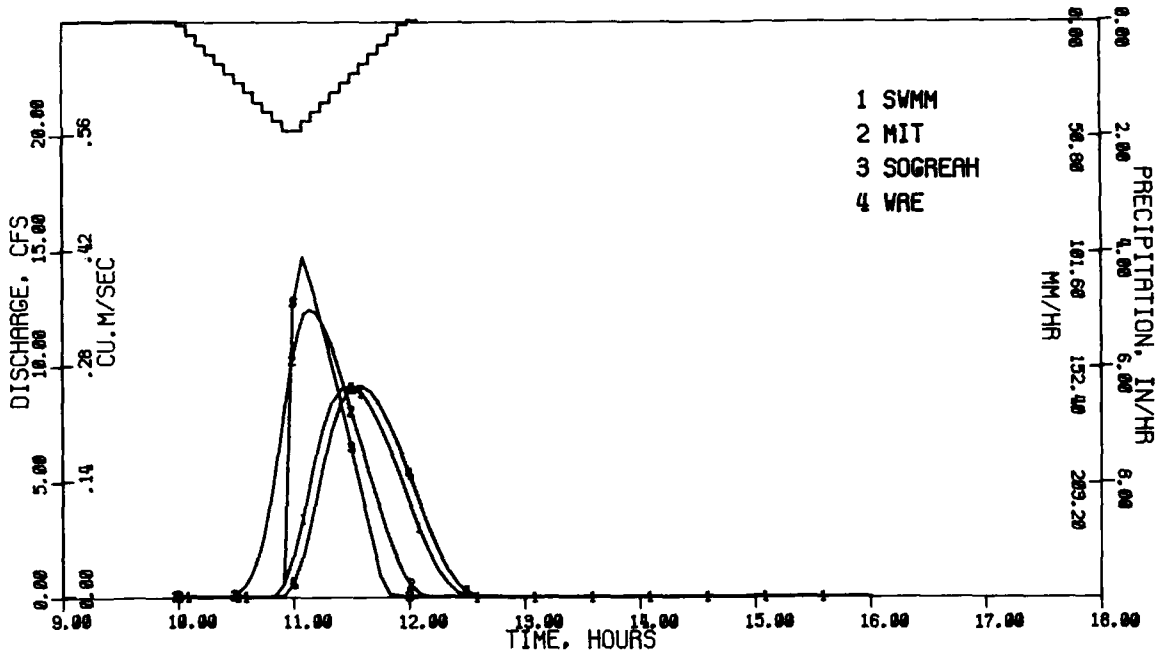


Figure 39. Runoff for Two-Hour Triangular Rainstorm - Large Hypothetical Catchment, 152 m (500 ft) Long, 0.1% Slope, 100% Imperviousness (Catchment #30)



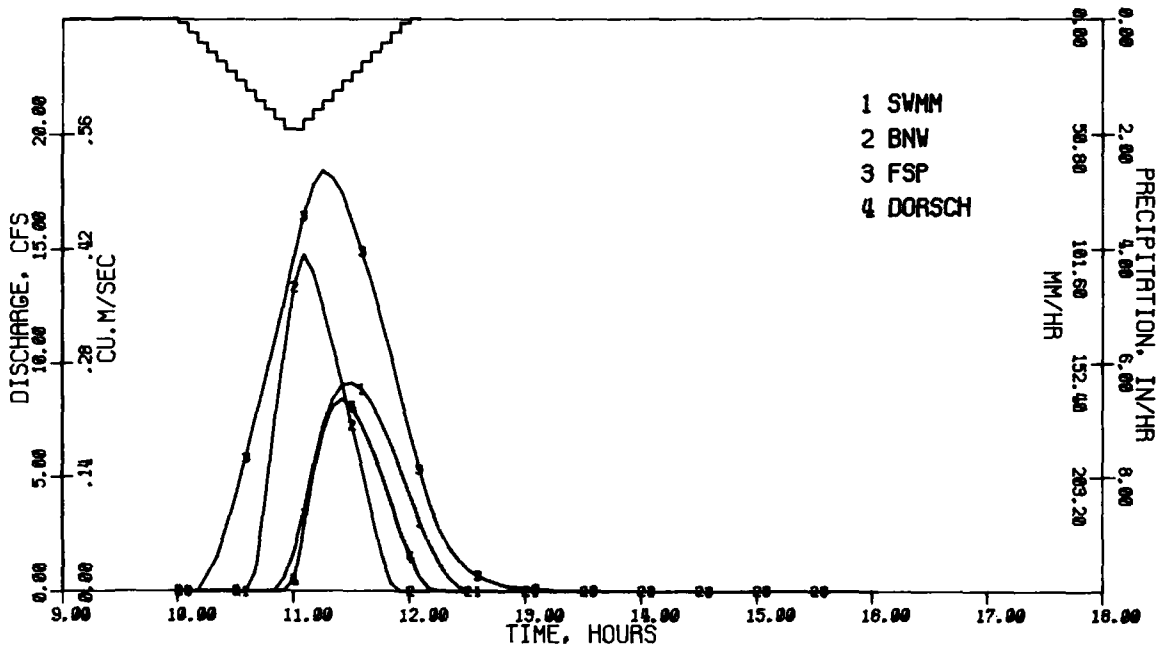
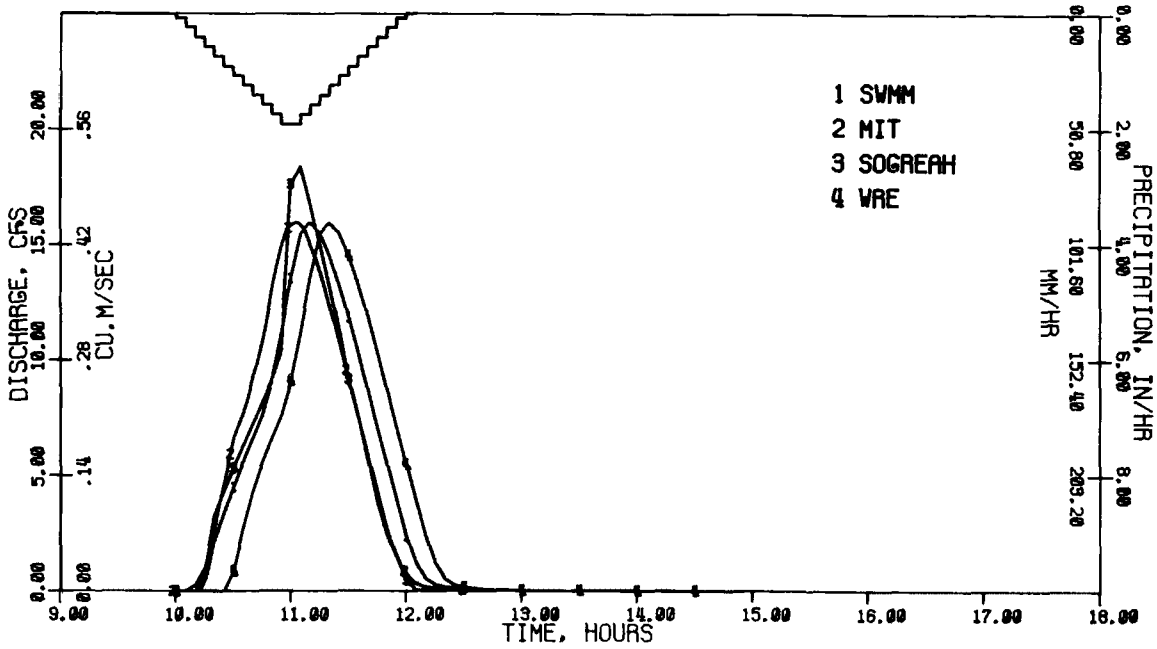


Figure 40. Runoff for Two-Hour Triangular Rainstorm - Large Hypothetical Catchment, 152 m (500 ft) Long, 10% Slope, 0% Imperviousness (Catchment #36)



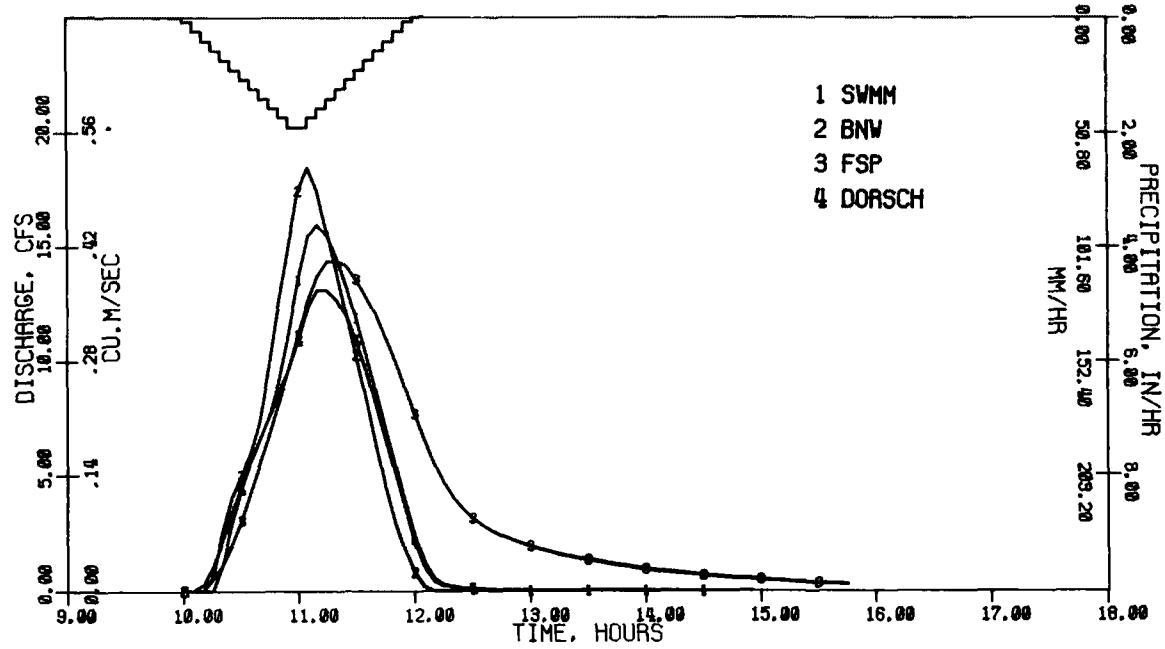
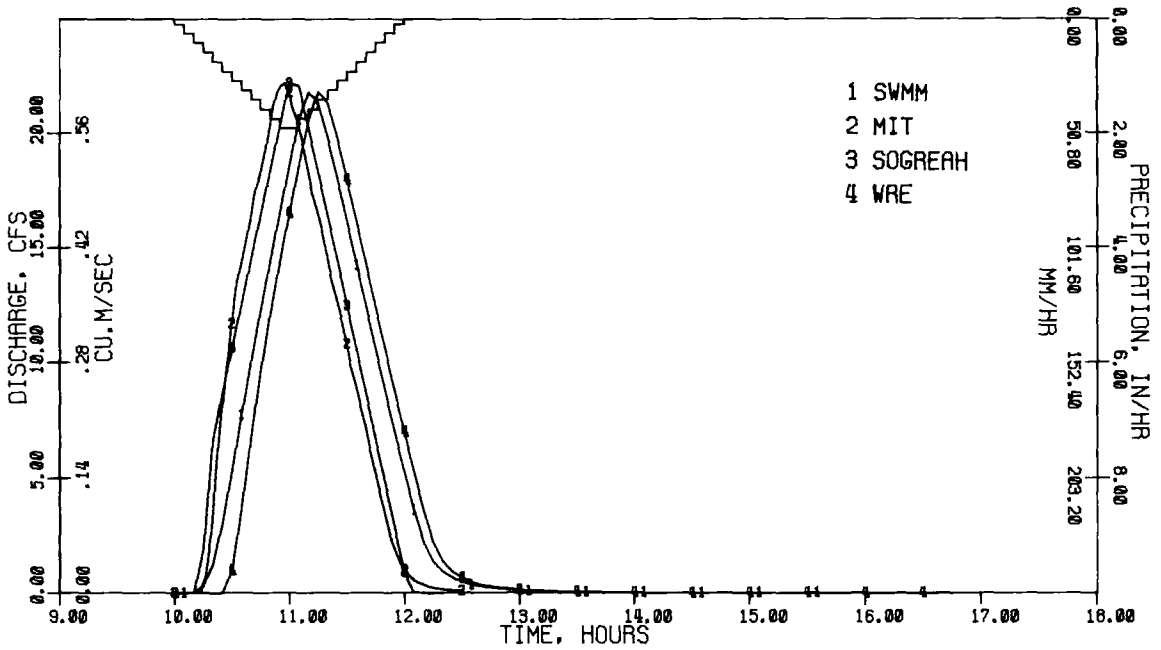


Figure 41. Runoff for Two-Hour Triangular Rainstorm - Large Hypothetical Catchment, 152 m (500 ft) Long, 10% Slope, 50% Imperviousness (Catchment #38)



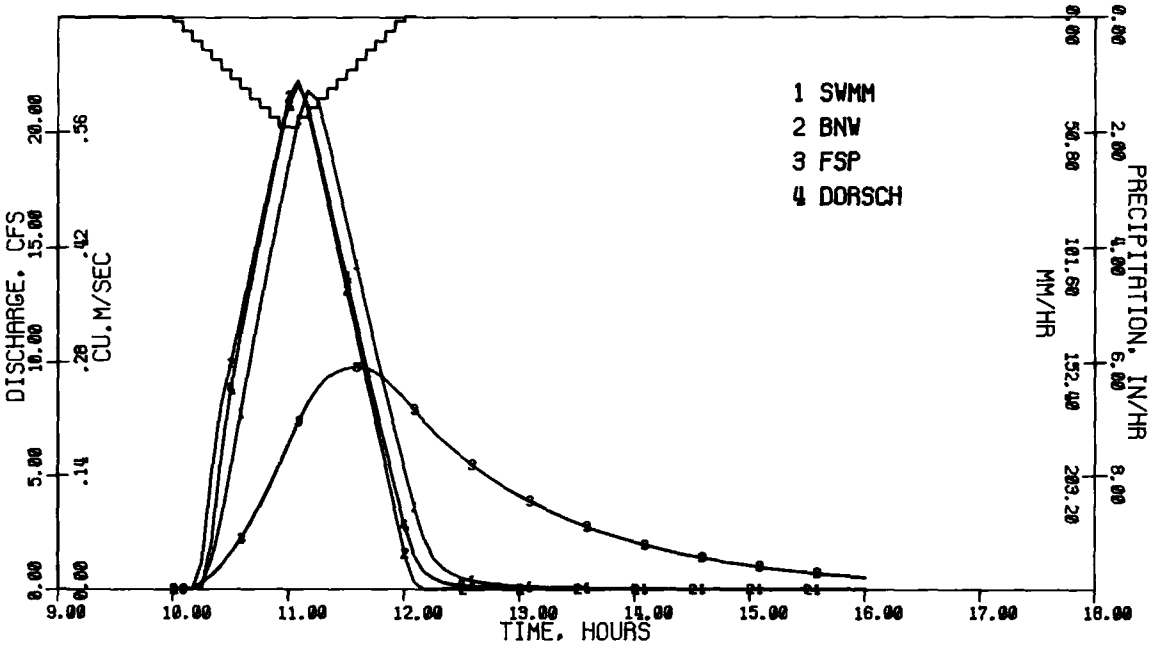


Figure 42. Runoff for Two-Hour Triangular Rainstorm - Large Hypothetical Catchment, 152 m (500 ft) Long, 10% Slope, 100% Imperviousness (Catchment #40)

Model Comparison

Comparable runoff hydrographs are computed by the SWMM, DORSCH, MIT, and WRE models for most catchment data combinations and for many catchment data combinations those computed by the BNW and SOGREAH models are comparable. Peak runoffs computed by the BNW and SOGREAH models are up to several times higher than those by the SWMM, DORSCH, MIT, and WRE models, and the differences between the peak runoff values and times computed by these two groups of models increase with increasing catchment size and decreasing catchment slope, imperviousness and initial moisture condition. The six models compute comparable runoff hydrographs for most fully impervious conditions. The runoff hydrographs computed by the FSP are different from these six models for most catchment data combinations.

The results of the SWMM, DORSCH, MIT, and WRE models are similar since all use the Horton equation for infiltration and the kinematic wave equation for overland flow. Infiltration is computed from overland flow depth and consequently is affected by the runoff delay and storage of the catchment surface. For the large catchment, the DORSCH and MIT models vary slightly from the SWMM and WRE models, particularly for the fully pervious conditions, which may be caused by different solution techniques for the infiltration/overland flow computations and by differences in the retention storage formulations.

The results of the BNW and SOGREAH models are surprisingly similar, although the BNW model uses the Holtan equation for infiltration and the unit hydrograph method for overland flow and the SOGREAH model uses the Horton equation for infiltration and the Muskingum method for overland flow. Both models compute infiltration from rainfall directly rather than the actual depth of water on the catchment surface. This and the different overland flow routing methods may explain the differences in computed runoff hydrographs between the two groups. Although the fully impervious cases do not vary significantly, for some fully pervious data combinations the peak runoffs predicted by the BNW and SOGREAH models are over 5 times greater than those predicted by the SWMM, DORSCH, MIT, and WRE models.

The FSP defines initial moisture conditions internally, so consequently differences between dry and wet initial catchment moisture conditions cannot be considered. Due to program constraints on numerical values of partial results of the catchment runoff computations, consideration of certain combinations of catchment size, shape, slope, and perviousness is limited. Consequently, for the small catchment data

combinations, the percent imperviousness has no influence on the computed peak runoff for the partially to fully pervious catchments, the peak runoff from the short catchment is lower than from the long catchment and the runoff from the steep slope is higher from the pervious than from the impervious areas. For the large catchment data combinations, the peak runoff is lower from the fully impervious area than from the fully pervious area. Only the peak runoff values for the large catchment with the flat slope are similar to the other models; and these values are lower than the other model values. It appears, therefore, that caution should be used in applying the model to small catchments (i.e., with drainage areas of 0.5 ha [1 acre] or less). Criteria could probably be developed for combinations of catchment size, slope, length, and perviousness which would define the applicability of the model.

Examination of the entire computer output indicates that the initial moisture condition has a definite effect on the shape of the computed runoff hydrographs, even for cases in which the peak runoff is not affected. For the dry initial conditions, the peak runoff occurs later and higher rainfall losses during the early part of the rainfall result in lower runoff values until the catchment is saturated and the runoff approaches the values of the wet initial conditions.

Evaluation of model sensitivity to variations in catchment characteristics indicates that catchment slope, shape, and initial moisture conditions have no effect on the computed peak runoff for all fully impervious cases. Initial catchment moisture conditions become more significant as the perviousness and overland flow time increase (i.e., with lower slopes and longer catchments). The catchment length becomes more significant with higher perviousness and lower initial moisture condition. The effect of perviousness is significant for all cases, with differences in peak runoffs between fully pervious and fully impervious catchments being the greatest for the long and flat catchments.

Two-Hour Triangular Rainstorm

The runoff computed by each of the seven tested models is plotted for twelve catchment data combinations in Figures 31 to 42. The models were separated into two groups for plotting purposes to prevent obscuring of model differences by too many overlapping lines. Each plot includes the runoff computed by the SWMM and three other models.

The plotted runoff is for the 2-hr triangular storm which has a peak rainfall intensity of 50.8 mm/hr (2 in./hr). The first six figures show runoff from a 0.465 ha (1.15 acre) rectangular catchment and the second six figures from a 4.65 ha

(11.5 acre) rectangular catchment. The catchment length in the direction of surface flow is 48.2 m (158 ft) for the small and 152 m (500 ft) for the large catchment. The catchment length perpendicular to the direction of surface flow is 96.4 m (316 ft) for the small and 305 m (1000 ft) for the large catchment. A Manning's roughness coefficient of 0.25 is assumed for the pervious area and 0.025 for the impervious area of each catchment. Initially dry catchment moisture conditions are assumed. For pervious areas, the maximum infiltration rate is 50.8 mm/hr (2.0 in./hr), the minimum infiltration rate is 12.7 mm/hr (0.5 in./hr), and the infiltration decay rate is 0.001 sec^{-1} as defined by Horton's infiltration equation. The retention storage capacity for pervious areas is 5.1 mm (0.20 in.) and for impervious areas 1.3 mm (0.05 in.). The plotted examples are for different combinations of catchment slope (0.1 and 10 percent) and impervious areas (0, 50, and 100 percent).

Differences in the mathematical formulations for infiltration produced considerable variation among the models in the computed runoff for the catchments with 0 percent imperviousness (Figures 31, 34, 37, and 40). The differences are particularly great for the 0.1 percent slope (Figures 31 and 37). For both slopes, runoff computed by the BNW and SOGREAH models is considerably higher than for the other models. One reason appears to be that Horton's equation, as used by most models, neglects catchment moisture conditions in computing potential infiltration. When the ground becomes rapidly saturated, Horton's equation overestimates infiltration and underestimates surface runoff. For steeper slopes where runoff occurs more rapidly or for low intensity rain this effect is less pronounced, since less water is available for infiltration.

Although Holtan's equation, as used by the BNW model, accounts for changes in soil moisture, it introduces a different approximation by computing potential infiltration from rainfall, rather than the depth of water on the catchment. It therefore underestimates infiltration for flat slopes where runoff is sufficiently slow, and rainfall occurring during one time step continues to contribute to infiltration during succeeding time steps.

The FSP assumes that a constant fraction of the rain falling on the pervious areas of sewered catchments runs off during the same time step; it neglects catchment slope, shape, and roughness. For the FSP simulations presented here, however, nonsewered catchment runoff was specified, which computes losses from rainfall with an empirical equation accounting for soil moisture changes and evaporation and considers catchment shape and slope. For the flat slope (Figures 31 and 37),

the FSP computes an extremely low and slow response and the runoff hydrograph does not recede during the 6-hour simulation period.

Differences in the computed results are introduced also by the overland flow routing procedures. The BNW model uses a unit hydrograph approach; the SOGREAH model uses the Muskingum method; the SWMM, DORSCH, MIT, and WRE models use kinematic wave formulations; and the FSP uses a linear storage routing which considers catchment shape and slope but neglects surface roughness.

The effect of these differences can be seen by comparing the results of the runoff computations for the 100 percent impervious areas, since for these runs rainfall losses were considered negligible and the computed differences are primarily the result of differences in the overland flow formulations. As Figures 33, 36, 39, and 42 show, the results of all models except the FSP are quite comparable for the steep slope (Figures 33 and 36) and show only small differences for the flat slope (Figures 39 and 42). The FSP assumes that a constant fraction of the overland flow storage on nonsewered areas runs off during each time step. The resulting runoff hydrograph is much flatter and longer than the other model hydrographs. Since the FSP formulation neglects catchment shape, slope, and surface roughness for impervious areas, the computed runoff hydrographs for the flat and steep slopes are the same.

The combined effects of the formulations for infiltration and overland flow is illustrated in Figures 32, 35, 38, and 41 for 50 percent imperviousness. Again, comparable runoff is computed by the BNW and SOGREAH models on the one hand and the SWMM, DORSCH, MIT, and WRE models on the other hand. The differences in computed runoff between the two groups, both in magnitude and timing, are smaller than for the fully pervious case and larger than for the fully impervious case. The peak runoff computed by the FSP is less than half that of the other models for the small flat catchment. The FSP produces more comparable runoff hydrographs for the other cases, but the computed runoff volume is slightly larger, and the computed peaks are slightly higher and later for the small steep and large flat catchment and average for the steep large catchment.

The selected examples accentuate model differences. As can be seen in a later section, many of these differences are damped in actual catchment applications as a result of the smoothing effect of more random rainfall sequences and the routing over many small subcatchments of various sizes, slopes and surface characteristics.

HYPOTHETICAL PIPE DATA TESTS

Hypothetical pipe data tests were performed for two different pipe diameters, three invert slopes, two types of upstream and downstream boundary conditions, four inflow hydrographs, and three inflow quality graphs, representing a total of 48 data combinations. The effect of different pipe roughness coefficients and different time discretizations on computed pipe outflow hydrographs and quality graphs was not tested.

A time step of 5 minutes was suggested for the pipe flow and water quality routing simulations. This time step was actually used by the BNW, FSP, SWMM, and MIT models. The DORSCH model used a time step of 2.5 minutes for the 1-hr triangular inflow and 5-minute time steps for the other three inflows. The SOGREAH model used a time step of 5 minutes for the constant inflow; for the other three inflows, the time steps varied according to changes in flow, with 5-minute steps at the beginning and end of the runoff gradually decreasing to 1 minute near the peaks of the hydrographs.

For these six models the length of time step is selected only on the basis of accuracy, while for the WRE model, it is limited by numerical stability criteria. Consequently, different time steps were used by the WRE model for the different data combinations but the time step was held constant for any particular routing computation. Different time steps were used for flow and water quality routing, which were computed by separate computer runs and have different stability criteria. The time steps for the flow routing ranged from 12 seconds for the large pipe with the steep slope to 60 seconds for the small pipe with the flat slope. The time step for the water quality routing varied from 24 seconds to 120 seconds.

The results of the pipe routing simulations of all seven models are summarized in Tables 65 to 71. These present magnitudes and times of peak outflows, weir overflows and outflow concentrations for each of the 48 pipe/inflow combinations. For cases with upstream storage and downstream diversion structures, the tabulated peak outflow represents the sum of the weir and orifice discharges. The times of the peak outflows and outflow concentrations are not tabulated for the constant continuous inflow and inflow concentrations. Variations from the constant values for these cases are the result of numerical instabilities which are generally oscillatory in nature, and consequently model comparisons are meaningful between the magnitudes, but not the times, of the peaks.

It was not possible to plot all computed outflow hydrographs and quality graphs; however, plots are presented in Figures 43 to 66 for the outflow hydrographs and quality graphs from

Table 65a. PEAK OUTFLOWS FROM HYPOTHETICAL PIPES (cfs)

Slope, percent	Inflow hydrograph number	Storage and diversion	Diameter = 2 ft							Diameter = 12 ft						
			SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
0.05	1	No	5.94	5.93	5.94	5.9	5.94	5.93	5.94	708	705	706	705	717	705	705
		Yes	*	5.93	***	6.0	5.94	5.93	5.93	706	705	***	705	705	705	705
	2	No	3.63	**	**	3.5	3.60	3.88	3.03	502	**	**	570	535	631	570
		Yes	3.63	**	**	3.4	3.44	3.57	2.02	505	**	**	505	443	522	404
	3	No	4.75	5.50	3.47	4.5	4.60	4.77	4.34	595	640	665	614	617	670	638
		Yes	4.65	5.50	***	3.8	4.53	4.56	3.24	593	640	***	572	560	609	533
	4	No	5.27	5.69	4.47	5.4	5.24	5.32	5.12	636	670	683	654	652	685	672
		Yes	5.42	5.71	***	4.0	5.21	5.23	4.44	655	670	***	635	630	659	620
0.5	1	No	20.66	18.77	18.77	18.8	19.51	18.75	18.77	2481	2231	2231	2231	2418	2230	2231
		Yes	*	18.77	***	18.8	18.77	18.75	18.28	2485	2231	***	1879	2231	2230	1088
	2	No	16.86	**	**	15.4	16.88	17.30	16.43	2080	**	**	2042	2088	2146	2141
		Yes	*	**	**	14.7	16.46	16.70	12.30	2076	**	**	1869	1839	1828	1614
	3	No	17.78	17.06	17.57	16.8	17.82	18.01	17.49	2158	2107	2231	2103	2157	2186	2182
		Yes	*	17.06	***	16.2	17.50	17.34	14.80	2153	2107	***	1878	1989	1999	1524
	4	No	18.22	17.92	18.17	17.6	18.30	18.26	18.10	2190	2169	2231	2164	2195	2200	2204
		Yes	*	17.92	***	17.2	18.11	17.83	16.29	2187	2169	***	1881	2094	2226	1606
5.0	1	No	67.12	59.35	59.36	59.4	59.35	59.32	59.35	7411	7054	7056	7054	7399	7053	7024
		Yes	*	50.54	***	59.4	55.24	59.32	48.86	*	3114	***	2866	2888	6049	2845
	2	No	55.71	**	**	53.7	55.73	59.96	56.56	6666	**	**	6683	6660	6902	6594
		Yes	*	**	**	48.5	51.00	52.26	33.24	*	**	**	2863	2785	5646	2822
	3	No	57.43	55.99	59.36	55.9	57.41	58.12	57.14	6861	6860	7056	6868	6861	6978	6656
		Yes	*	50.65	***	49.6	50.52	53.96	39.91	*	3103	***	2867	2786	6036	2836
	4	No	58.26	57.41	59.36	57.8	58.37	58.48	58.70	6946	6946	7056	6958	6958	7000	6987
		Yes	*	50.73	***	49.6	50.33	55.47	45.94	*	3109	***	2861	2778	6234	2844

Notes:
* Value not computed because overflowing storage tank terminates model output.
** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
*** Value not computed because model cannot simulate diversions.

Table 65b. PEAK OUTFLOWS FROM HYPOTHETICAL PIPES (m^3/sec)

Slope, percent	Inflow hydrograph number	Storage and diversion	Diameter = 0.61 m							Diameter = 3.66 m						
			SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
0.05	1	No	0.168	0.168	0.168	0.167	0.168	0.168	0.168	20.1	20.0	20.0	20.0	20.3	20.0	20.0
		Yes	*	0.168	***	0.170	0.168	0.168	0.168	20.0	20.0	***	20.0	20.0	20.0	20.0
	2	No	0.103	**	**	0.099	0.102	0.110	0.086	14.2	**	**	16.1	15.2	17.9	16.1
		Yes	0.103	**	**	0.096	0.097	0.101	0.057	14.3	**	**	14.3	12.5	14.8	11.4
	3	No	0.135	0.156	0.098	0.127	0.130	0.135	0.123	16.9	18.1	18.8	17.4	17.5	19.0	18.1
		Yes	0.132	0.156	***	0.108	0.128	0.129	0.092	16.8	18.1	***	16.2	15.9	17.2	15.1
	4	No	0.149	0.161	0.127	0.153	0.148	0.151	0.145	18.0	19.0	19.3	18.5	18.5	19.4	19.0
		Yes	0.153	0.162	***	0.113	0.148	0.148	0.126	18.5	19.0	***	18.0	17.8	18.7	17.5
0.5	1	No	0.585	0.532	0.532	0.532	0.553	0.531	0.531	70.3	63.2	63.2	63.2	68.5	63.2	63.1
		Yes	*	0.532	***	0.532	0.532	0.531	0.517	70.4	63.2	***	53.2	63.2	63.2	30.8
	2	No	0.477	**	**	0.436	0.478	0.490	0.465	58.9	**	**	57.8	59.1	60.8	60.6
		Yes	*	**	**	0.416	0.466	0.473	0.348	58.8	**	**	52.9	52.1	51.8	45.7
	3	No	0.504	0.483	0.498	0.476	0.505	0.510	0.495	61.1	59.7	63.2	59.6	61.1	61.9	61.8
		Yes	*	0.483	***	0.459	0.496	0.491	0.419	61.0	59.7	***	53.2	56.3	56.6	43.1
	4	No	0.516	0.507	0.515	0.498	0.518	0.517	0.512	62.0	61.4	63.2	61.3	62.2	62.3	62.4
		Yes	*	0.507	***	0.487	0.513	0.505	0.461	61.9	61.4	***	53.3	59.3	63.0	45.4
5.0	1	No	1.901	1.681	1.681	1.682	1.681	1.68	1.680	209.9	199.8	199.8	199.8	209.5	199.7	198.8
		Yes	*	1.431	***	1.682	1.564	1.680	1.383	*	88.2	***	81.2	81.8	171.3	80.5
	2	No	1.578	**	**	1.521	1.578	1.698	1.601	188.8	**	**	189.3	188.6	195.5	186.6
		Yes	*	**	**	1.374	1.444	1.480	0.941	*	**	**	81.1	78.9	159.9	79.9
	3	No	1.626	1.586	1.681	1.583	1.626	1.646	1.617	194.3	194.3	199.8	194.5	194.3	197.6	188.4
		Yes	*	1.434	***	1.405	1.431	1.528	1.129	*	87.9	***	81.2	78.9	170.9	80.3
	4	No	1.650	1.626	1.681	1.637	1.653	1.656	1.661	196.7	196.7	199.8	197.1	197.1	198.2	197.7
		Yes	*	1.437	***	1.405	1.425	1.571	1.300	*	88.0	***	81.0	78.7	176.5	80.5

Notes:
* Value not computed because overflowing storage tank terminates model output.
** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
*** Value not computed because model cannot simulate diversions.

Table 66a. PEAK OVERFLOWS FROM HYPOTHETICAL PIPES (cfs)

Slope, percent	Inflow hydrograph number	Diameter = 2 ft							Diameter = 12 ft						
		SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
0.05	1	*	1.19	***	2.0	1.62	2.65	2.12	3	593	***	319	279	342	336
	2	0.34	**	***	0.1	0.00	0.46	0.00	81	**	***	158	82	192	98
	3	1.00	1.83	***	0.2	0.56	1.17	0.00	124	282	***	210	170	261	197
	4	1.63	1.98	***	0.2	1.07	1.66	1.04	158	306	***	259	223	301	266
0.5	1	*	13.16	***	9.9	11.25	12.36	18.20	30	1686	***	1033	1424	1448	2650
	2	*	**	***	7.8	9.52	10.38	7.21	26	**	***	1026	1130	1166	1136
	3	*	11.63	***	8.6	10.30	10.84	8.17	28	1576	***	1034	1243	1285	1273
	4	*	12.42	***	9.3	10.75	11.19	9.87	29	1631	***	1039	1322	1442	1406
5.0	1	*	42.79	***	31.9	36.96	40.82	47.73	*	2509	***	1595	1917	4076	2686
	2	*	**	***	26.8	35.44	34.82	23.07	*	**	***	1595	1840	3791	2520
	3	*	42.88	***	27.4	35.08	36.02	26.46	*	2491	***	1596	1841	4062	2659
	4	*	42.95	***	27.5	34.94	37.11	34.18	*	2496	***	1596	1835	4118	2697

Notes:
* Value not computed because overflowing storage tank terminates model output.
** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
*** Value not computed because model cannot simulate diversions.

Table 66b. PEAK OVERFLOWS FROM HYPOTHETICAL PIPES
(m^3/sec)

Slope, percent	Inflow hydrograph number	Diameter = 0.61 m							Diameter = 3.66 m						
		SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
0.05	1	*	0.034	***	0.057	0.046	0.075	0.060	0.1	16.8	***	9.0	7.9	9.7	9.5
	2	0.010	**	***	0.003	0.000	0.011	0.000	2.3	**	***	4.5	2.3	5.4	2.8
	3	0.028	0.052	***	0.006	0.016	0.033	0.000	3.5	8.0	***	5.9	4.8	7.4	5.6
	4	0.046	0.056	***	0.006	0.030	0.047	0.029	4.5	8.7	***	7.3	6.3	8.5	7.5
0.5	1	*	0.373	***	0.280	0.319	0.350	0.516	0.8	47.7	***	29.3	40.3	41.0	75.0
	2	*	**	***	0.221	0.270	0.294	0.204	0.7	**	***	29.1	32.0	33.0	32.1
	3	*	0.329	***	0.244	0.292	0.307	0.231	0.8	44.6	***	29.3	35.2	36.4	36.0
	4	*	0.352	***	0.263	0.304	0.317	0.279	0.8	46.2	***	29.4	37.4	40.8	39.8
5.0	1	*	1.212	***	0.903	1.047	1.156	1.351	*	71.1	***	45.2	54.3	115.4	76.0
	2	*	**	***	0.759	1.004	0.986	0.653	*	**	***	45.2	52.1	107.4	71.3
	3	*	1.214	***	0.776	0.993	1.020	0.749	*	70.5	***	45.2	52.1	115.0	75.2
	4	*	1.216	***	0.779	0.990	1.051	0.967	*	70.7	***	45.2	52.0	116.6	76.3

Notes:
* Value not computed because overflowing storage tank terminates model output.
** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
*** Value not computed because model cannot simulate diversions.

Table 67. TIMES OF PEAK OUTFLOWS FROM HYPOTHETICAL PIPES (minutes)

Slope percent	Inflow hydrograph number	Storage and diversion	Diameter = 2 ft							Diameter = 12 ft						
			SWMM	BNW	FSP	DOR	MIT	SOG	WRE	SWMM	BNW	FSP	DOR	MIT	SOG	WRE
0.05	2	No	60	**	**	80	90	78	80	50	**	**	45	50	40	40
		Yes	60	**	***	80	90	84	116	50	**	***	45	55	42	52
	3	No	120	120	190	115	120	105	106	80	80	120	75	80	69	68
		Yes	125	115	***	105	125	109	142	80	80	***	75	85	72	84
	4	No	180	180	250	175	180	165	166	145	140	180	135	135	130	130
		Yes	175	175	***	165	185	170	202	135	140	***	130	145	132	140
0.5	2	No	55	**	**	45	50	47	45	40	**	**	35	35	35	35
		Yes	*	**	***	50	50	49	60	40	**	***	35	40	36	41
	3	No	85	80	125	80	80	77	75	70	70	115	65	65	65	65
		Yes	*	80	***	80	80	82	93	70	70	***	60	70	67	93
	4	No	145	140	125	135	140	138	135	130	125	115	125	125	125	125
		Yes	*	140	***	135	140	141	155	130	130	***	105	130	121	177
5.0	2	No	40	**	**	35	35	36	36	35	**	**	30	30	32	33
		Yes	*	**	***	35	35	38	51	*	**	***	35	40	37	33
	3	No	70	65	115	65	65	66	63	65	60	115	60	60	62	63
		Yes	*	65	***	70	60	70	97	*	65	***	60	25-100	69	63
	4	No	130	125	175	125	125	126	125	125	120	175	120	120	122	123
		Yes	*	135	***	115	110	131	157	*	120	***	110	60-200	118	123

Notes:
- * Value not computed because overflowing storage tank terminates model output.
- ** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
- *** Value not computed because model cannot simulate diversions.
- 24-50 Constant peak outflow for indicated period.

Table 68. EXTREME OUTFLOW CONCENTRATIONS OF SMALL HYPOTHETICAL PIPES (mg/ℓ)

Slope, percent	Inflow hydrograph number	Storage and diversion	Maximum for constant inflow concentration				Maximum for upright triangular inflow concentration				Minimum for inverted triangular inflow concentration			
			SWMM	BNW	SOG	WRE	SWMM	BNW	SOG	WRE	SWMM	BNW	SOG	WRE
0.05	1	No	99	100	100	100	85	91	87	66	15	9	13	34
		Yes	*	100	100	100	*	91	85	66	*	9	15	34
	2	No	100	**	100	100	67	**	53	55	***	**	47	45
		Yes	102	**	100	100	67	**	50	51	***	**	50	49
	3	No	101	100	100	100	77	92	75	71	25	8	25	29
		Yes	104	100	100	100	77	92	73	66	25	8	28	34
	4	No	100	100	100	100	88	96	88	84	14	4	12	16
		Yes	101	100	100	100	88	96	87	78	14	4	13	22
0.5	1	No	105	100	100	100	91	90	90	87	10	10	10	13
		Yes	*	100	100	100	*	90	90	81	*	10	10	19
	2	No	103	**	100	100	86	**	84	83	2	**	16	17
		Yes	*	**	100	100	*	**	82	72	*	**	18	28
	3	No	101	100	100	100	92	90	92	92	9	10	8	9
		Yes	*	100	100	100	*	90	91	78	*	10	9	22
	4	No	101	100	100	100	96	95	97	96	5	5	4	4
		Yes	*	100	100	100	*	95	96	86	*	5	4	14
5.0	1	No	106	100	100	100	96	94	93	96	4	6	7	4
		Yes	*	100	100	100	*	94	92	72	*	6	8	28
	2	No	106	**	100	100	95	**	97	94	7	**	3	7
		Yes	*	**	100	100	*	**	96	70	*	**	4	30
	3	No	107	100	100	100	97	94	99	96	4	6	2	4
		Yes	*	100	100	100	*	94	98	75	*	6	2	25
	4	No	107	100	100	100	99	97	99	98	2	3	1	2
		Yes	*	100	100	100	*	97	99	81	*	3	1	19

Notes:
* Value not computed because overflowing storage tank terminates model output.
** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
*** Value not computed.

Table 69. EXTREME OUTFLOW CONCENTRATIONS OF LARGE HYPOTHETICAL PIPES (mg/ℓ)

Slope, percent	Inflow hydrograph number	Storage and diversion	Maximum for constant inflow concentration				Maximum for upright triangular inflow concentration				Minimum for inverted triangular inflow concentration			
			SWMM	BNW	SOG	WRE	SWMM	BNW	SOG	WRE	SWMM	BNW	SOG	WRE
0.05	1	No	107	100	100	100	96	90	91	86	6	10	9	14
		Yes	110	100	100	100	96	90	90	82	6	10	11	18
	2	No	103	**	100	100	87	**	89	85	14	**	11	17
		Yes	108	**	100	100	87	**	84	70	14	**	16	30
	3	No	101	100	100	100	93	90	94	92	8	10	6	9
		Yes	101	100	100	100	93	90	92	82	8	10	8	18
	4	No	100	100	100	100	96	95	98	96	4	5	2	4
		Yes	100	100	100	100	96	95	96	90	4	5	4	10
0.5	1	No	104	100	100	100	96	94	94	97	4	6	6	3
		Yes	106	100	100	***	96	94	92	***	4	6	8	***
	2	No	104	**	100	100	95	**	98	95	7	**	3	6
		Yes	122	**	100	100	94	**	95	80	7	**	5	20
	3	No	105	100	100	100	97	94	99	97	4	6	1	3
		Yes	122	100	100	100	97	94	98	87	4	6	2	13
	4	No	106	100	100	100	99	97	99	98	2	3	1	1
		Yes	122	100	100	100	99	97	99	92	2	3	1	8
5.0	1	No	103	100	100	100	98	97	97	91	3	3	4	9
		Yes	99	100	100	100	*	97	95	70	*	3	5	30
	2	No	108	**	100	100	97	**	98	97	4	*	2	3
		Yes	*	**	100	100	*	**	98	79	*	*	2	21
	3	No	106	100	100	100	98	97	99	98	3	3	1	2
		Yes	*	100	100	100	*	97	99	87	*	3	1	13
	4	No	106	100	100	100	99	99	99	99	1	2	1	1
		Yes	*	100	100	100	*	99	99	92	*	2	1	8

Notes:
* Value not computed because overflowing storage tank terminates model output.
** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
*** Value not computed.

Table 70. TIMES OF EXTREME OUTFLOW CONCENTRATIONS OF SMALL HYPOTHETICAL PIPES (minutes)

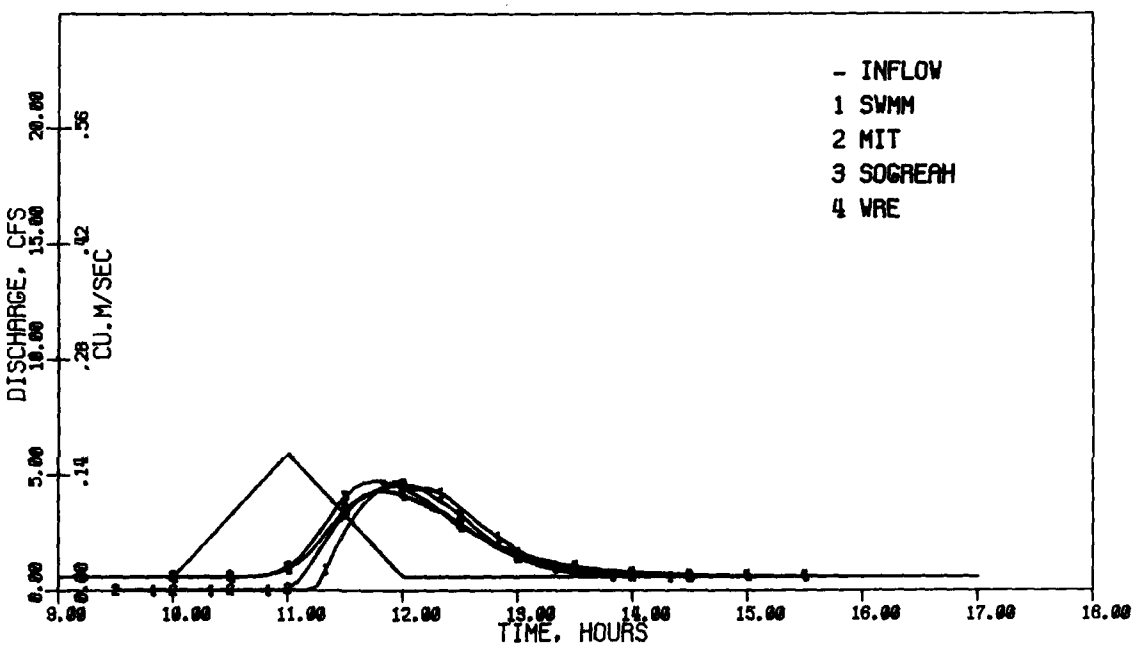
Slope, percent	Inflow hydrograph number	Storage and diversion	For maximum outflow concentration of upright triangular inflow concentration				For minimum outflow concentration of inverted triangular inflow concentration			
			SWMM	BNW	SOG	WRE	SWMM	BNW	SOG	WRE
0.05	2	No	120	**	100	84	***	**	100	84
		Yes	120	**	110	140	***	**	110	140
	3	No	145	120	125	114	135	120	125	114
		Yes	145	120	135	162	135	120	135	162
	4	No	200	180	182	174	195	180	182	174
		Yes	200	180	192	216	195	180	192	216
0.5	2	No	60	**	52	45	60	**	52	45
		Yes	*	**	54	65	*	**	54	65
	3	No	90	80	81	79	40	80	81	79
		Yes	*	80	83	101	*	80	83	101
	4	No	150	140	140	140	150	140	140	140
		Yes	*	140	143	166	*	140	143	166
5.0	2	No	45	**	37	37	45	**	37	37
		Yes	*	**	37	57	*	**	37	57
	3	No	75	65	67	65	75	65	67	65
		Yes	*	65	67	97	*	65	67	97
	4	No	135	125	126	127	135	125	126	127
		Yes	*	125	128	169	*	125	128	169

Notes: * Value not computed because overflowing storage tank terminates model output.
** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
*** Value not computed.

Table 71. TIMES OF EXTREME OUTFLOW CONCENTRATIONS OF LARGE HYPOTHETICAL PIPES (minutes)

Slope, percent	Inflow hydrograph number	Storage and diversion	For maximum outflow concentration of upright triangular inflow concentration				For minimum outflow concentration of inverted triangular inflow concentration			
			SWMM	BNW	SOG	WRE	SWMM	BNW	SOG	WRE
0.05	2	No	55	**	46	48	55	**	46	45
		Yes	55	**	54	66	55	**	54	66
	3	No	85	80	75	76	85	80	75	76
		Yes	85	80	83	96	85	80	83	96
	4	No	145	140	135	136	145	140	135	135
		Yes	145	140	143	155	145	140	143	155
0.5	2	No	45	**	36	36	45	**	36	36
		Yes	45	**	39	47	45	**	39	47
	3	No	75	75	66	66	75	65	66	66
		Yes	75	75	69	83	75	65	69	83
	4	No	135	125	126	125	135	125	126	125
		Yes	135	125	128	149	135	125	128	149
5.0	2	No	40	**	32	33	40	**	32	33
		Yes	*	**	33	42	*	**	33	42
	3	No	70	60	62	63	70	60	62	63
		Yes	*	60	63	75	*	60	63	75
	4	No	130	120	122	121	130	120	122	121
		Yes	*	120	124	137	*	120	124	137

Notes: * Value not computed because overflowing storage tank terminates model output.
** Value not computed because model cannot read and interpolate one-hour triangular inflow hydrograph.
*** Value not computed.



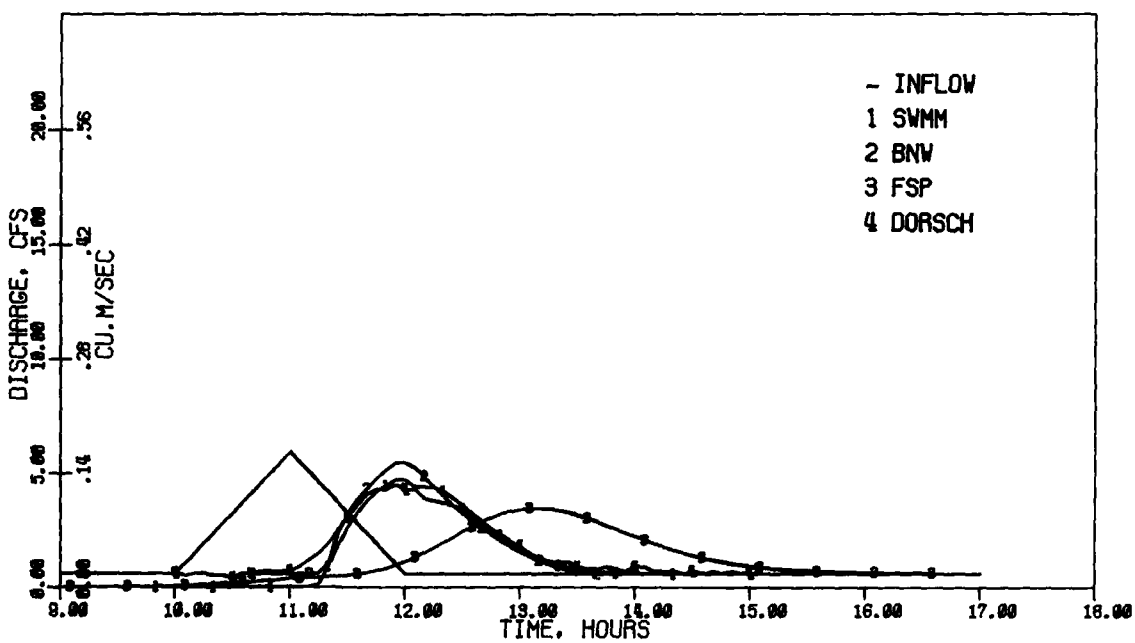
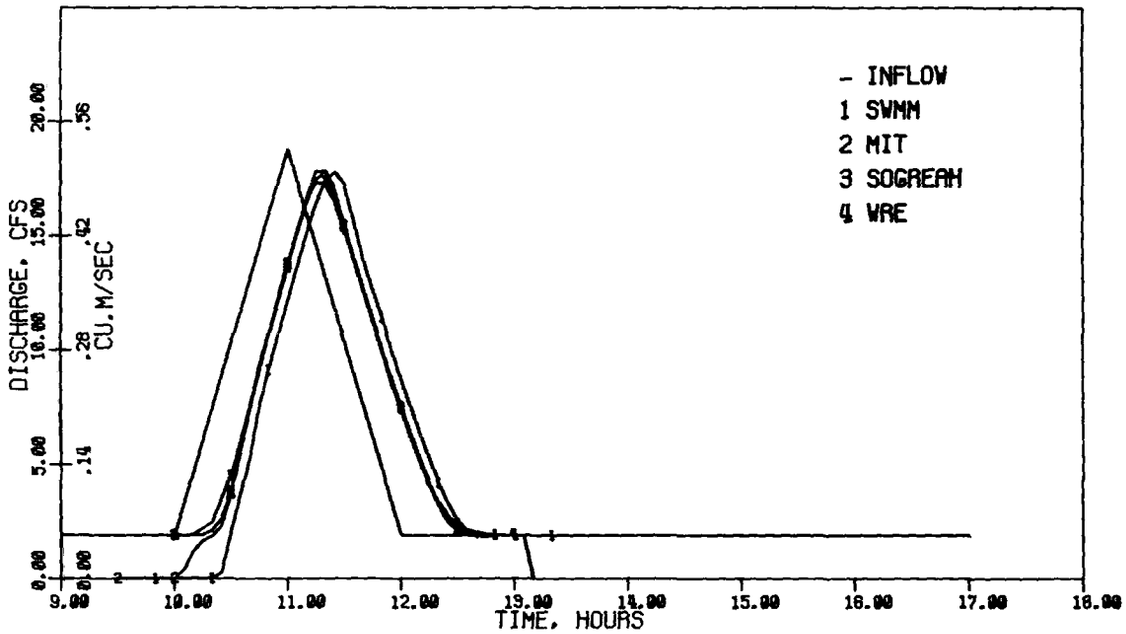


Figure 43. Outflows for Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.05% Slope



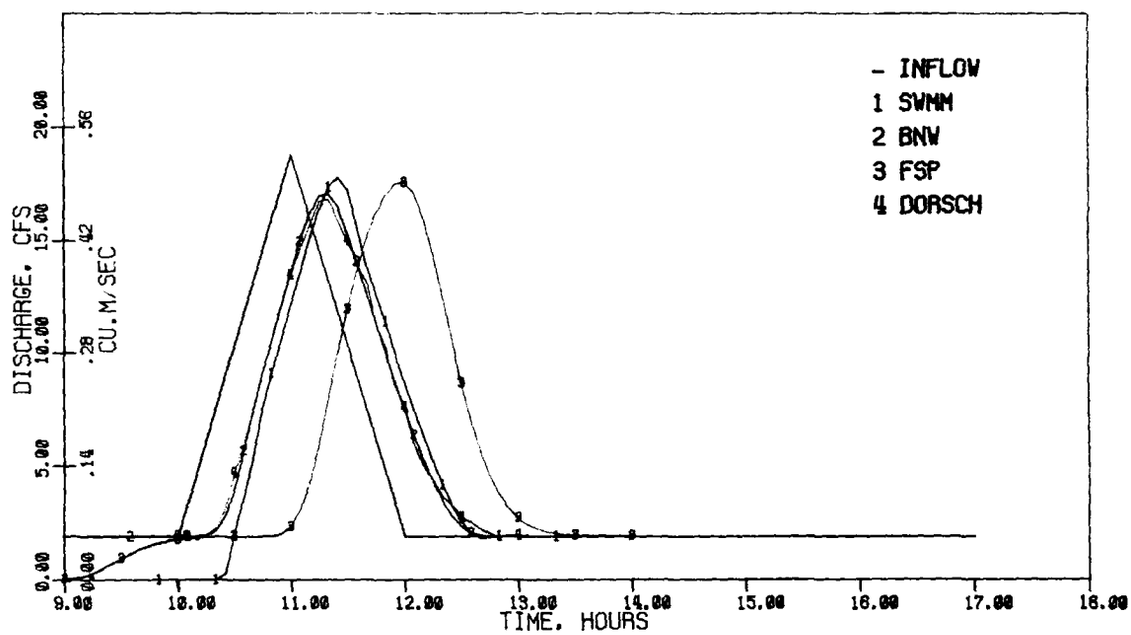
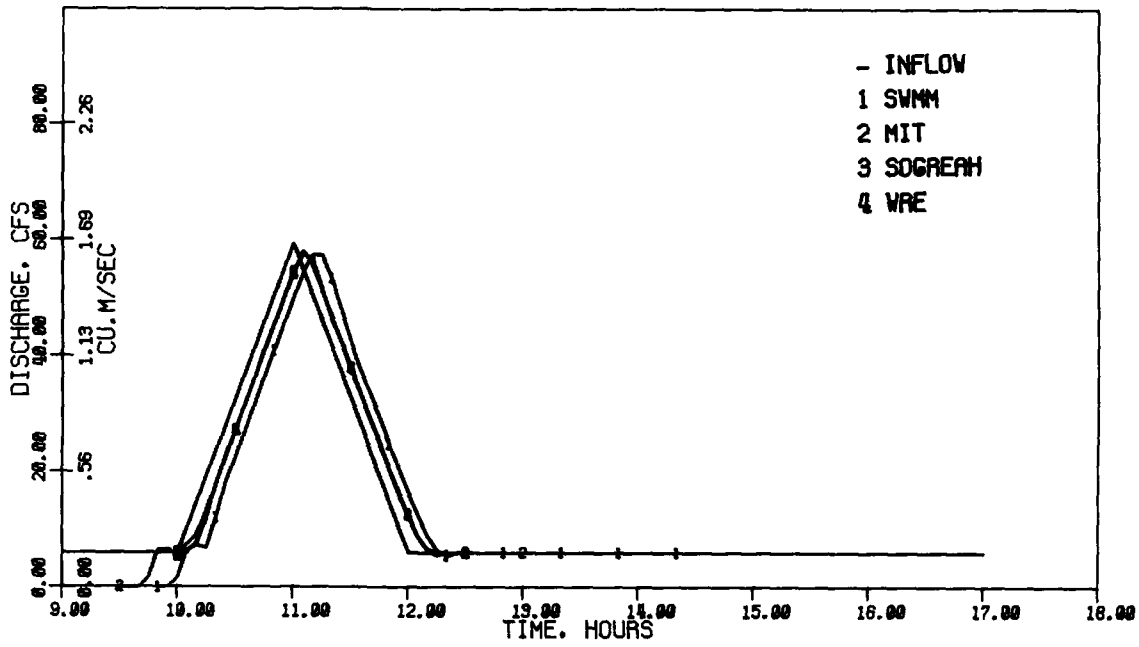


Figure 44. Outflows for Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.5% Slope



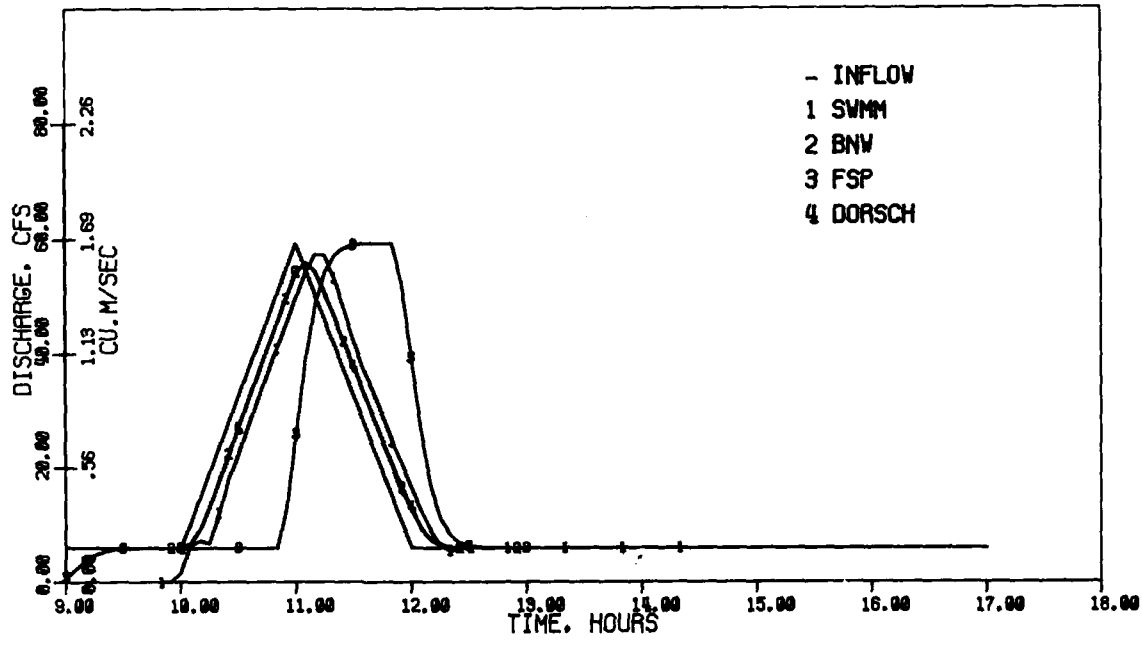
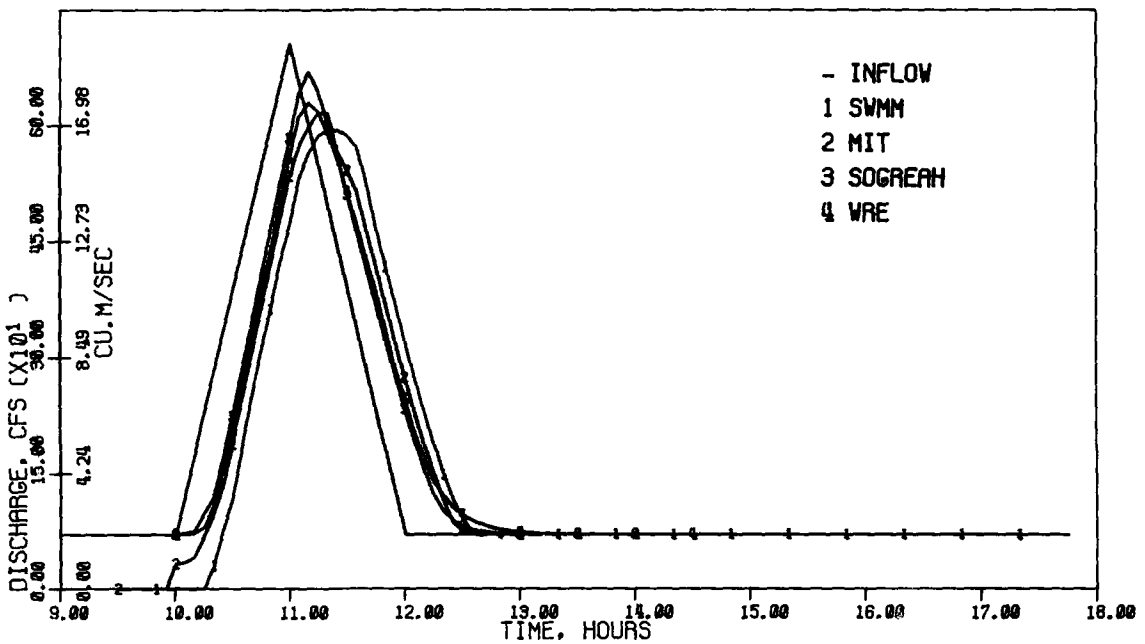


Figure 45. Outflows for Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 5% Slope



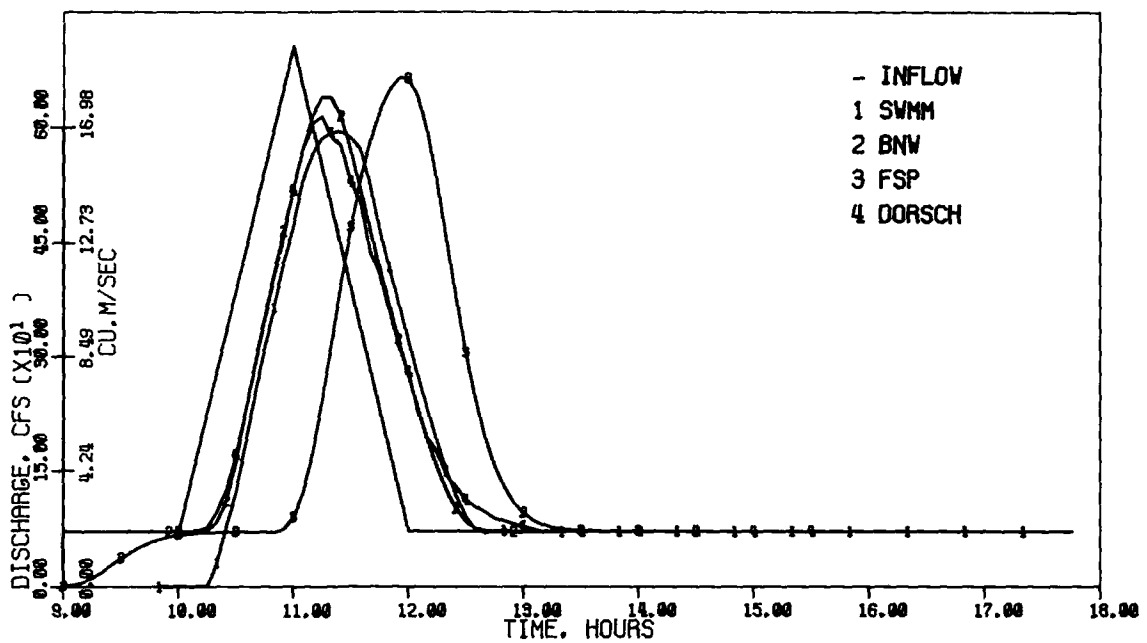
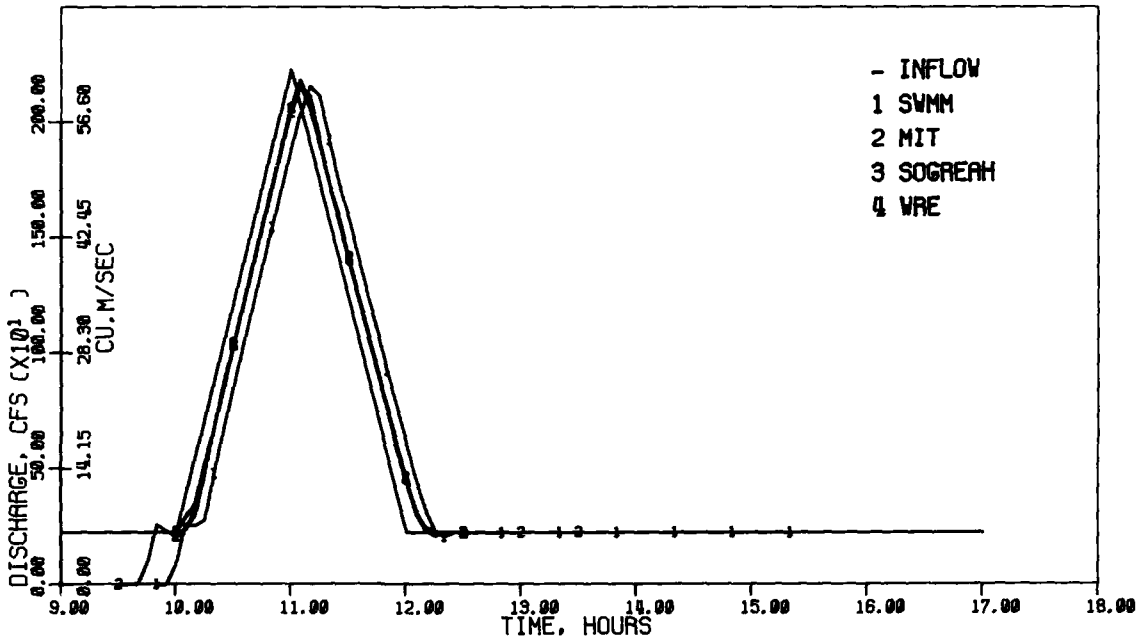


Figure 46. Outflows for Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.05% Slope



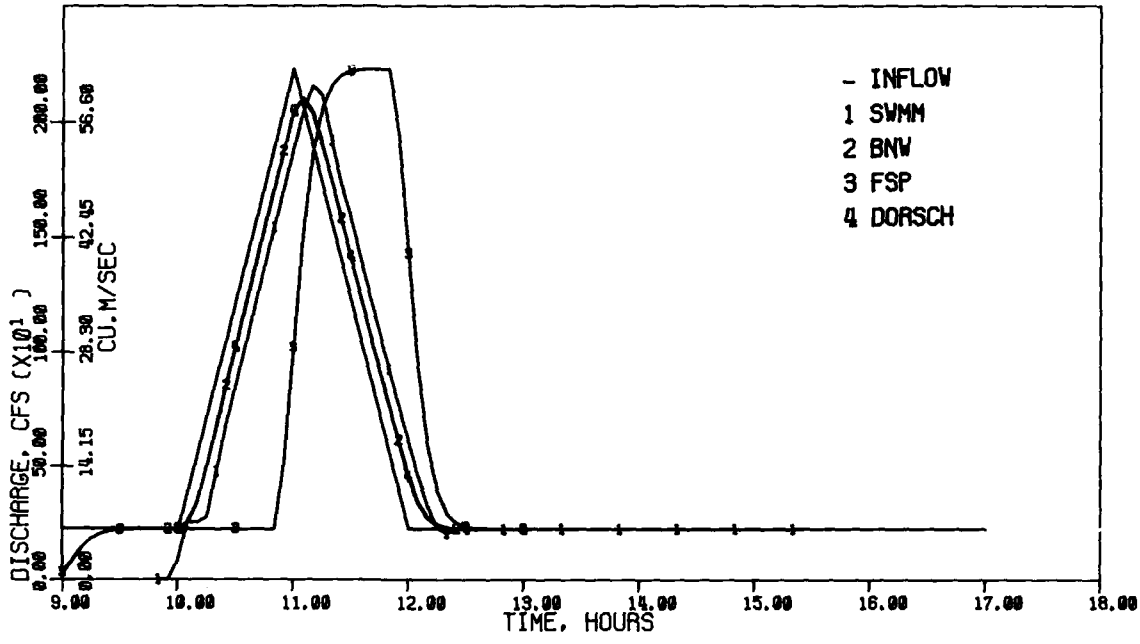
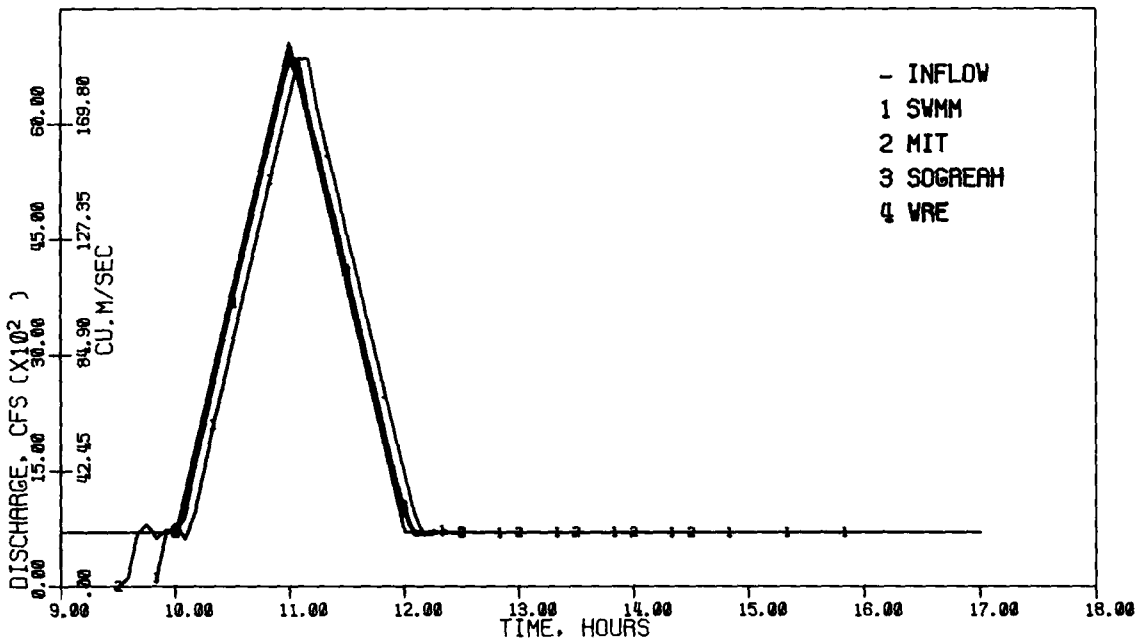


Figure 47. Outflows for Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope



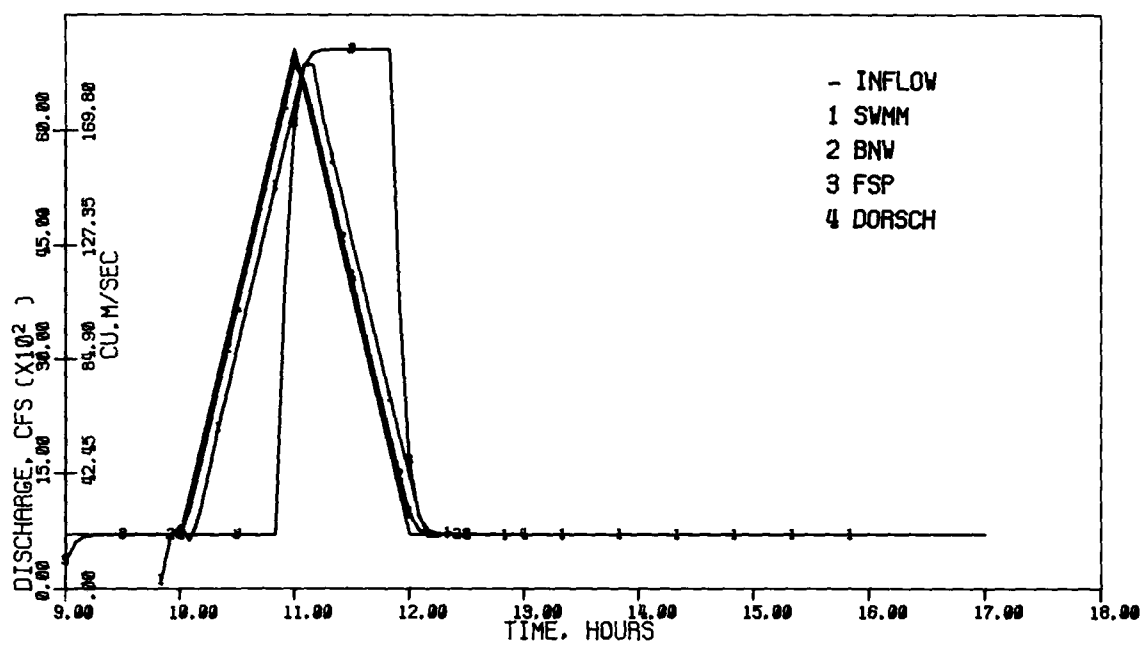


Figure 48. Outflows for Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 5% Slope

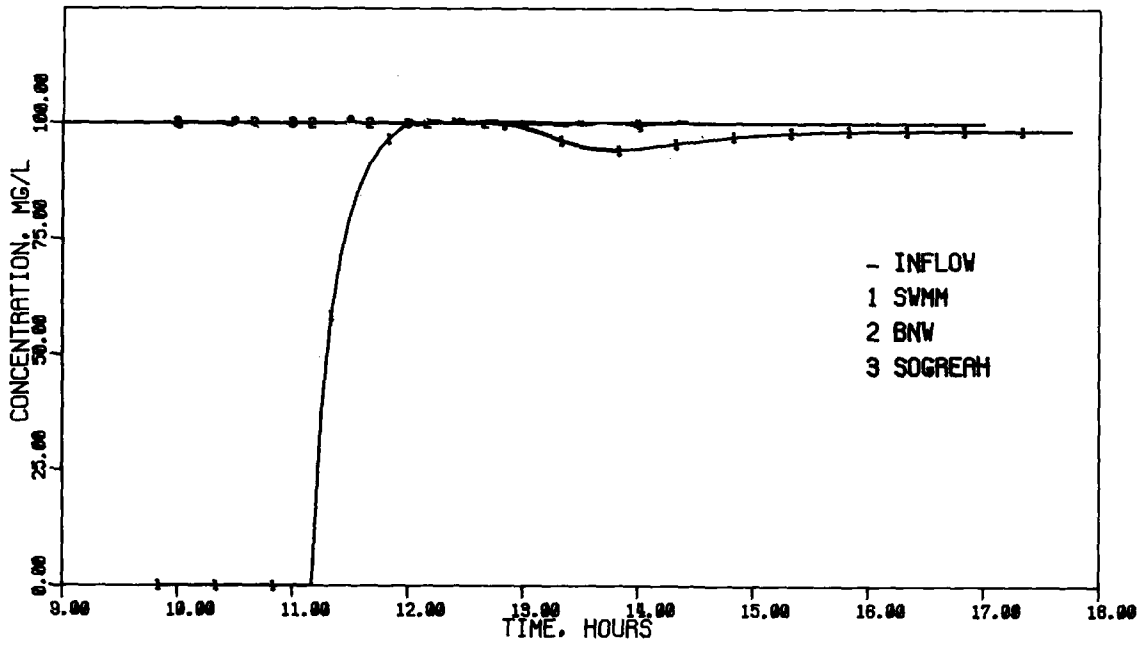


Figure 49. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.05% Slope

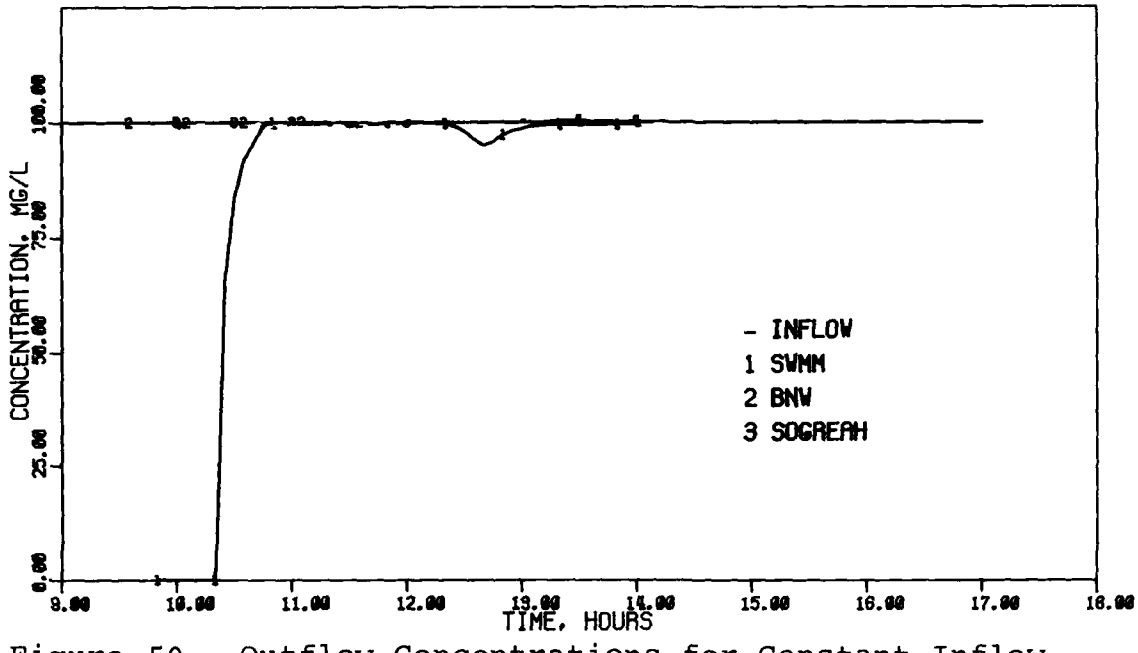


Figure 50. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.5% Slope

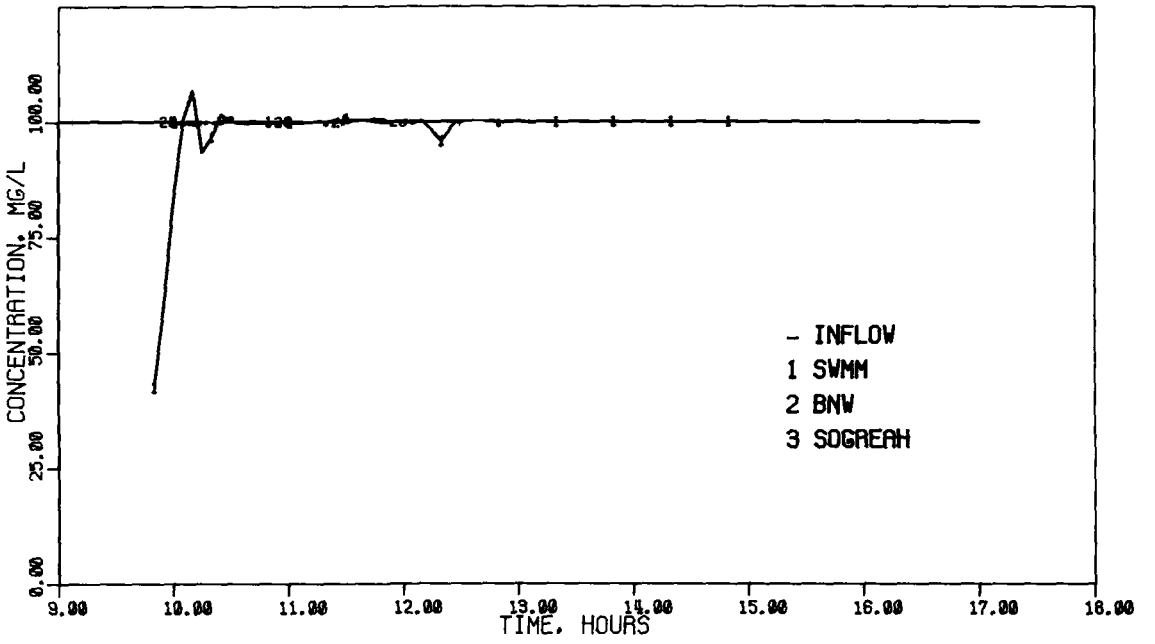


Figure 51. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 5% Slope

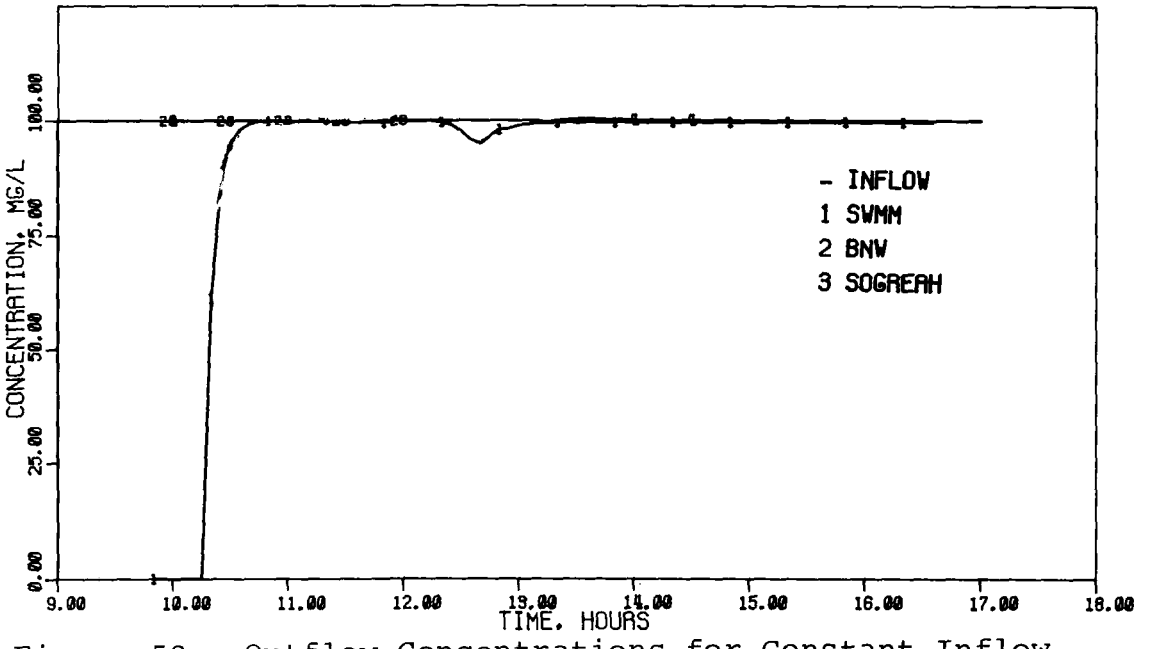


Figure 52. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.05% Slope

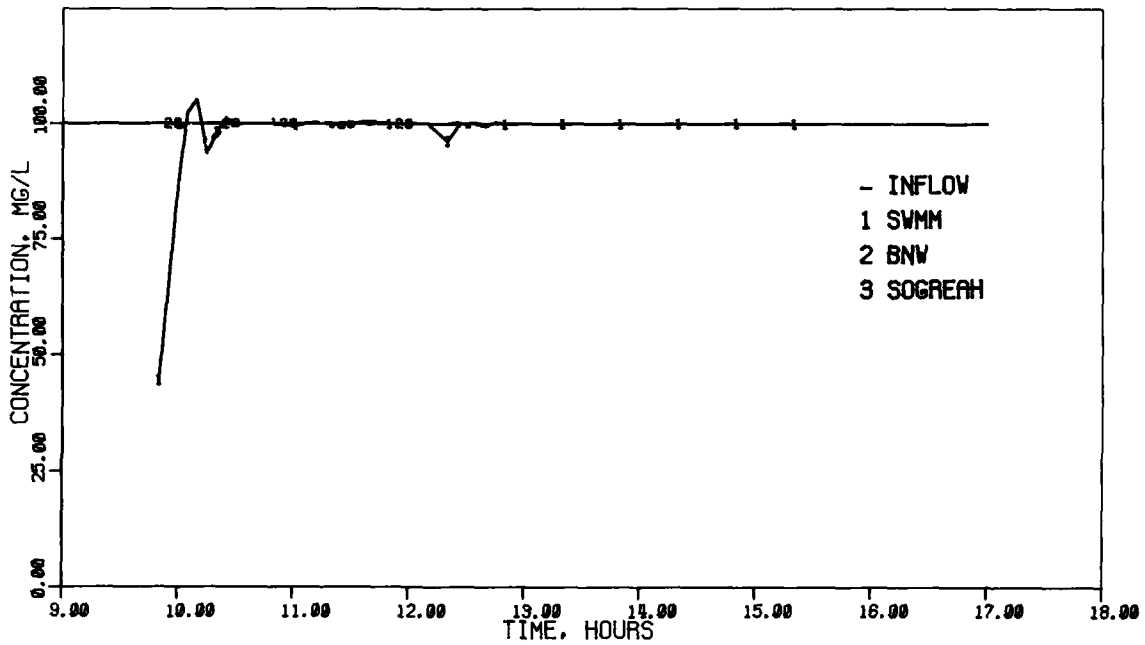


Figure 53. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.5% Slope

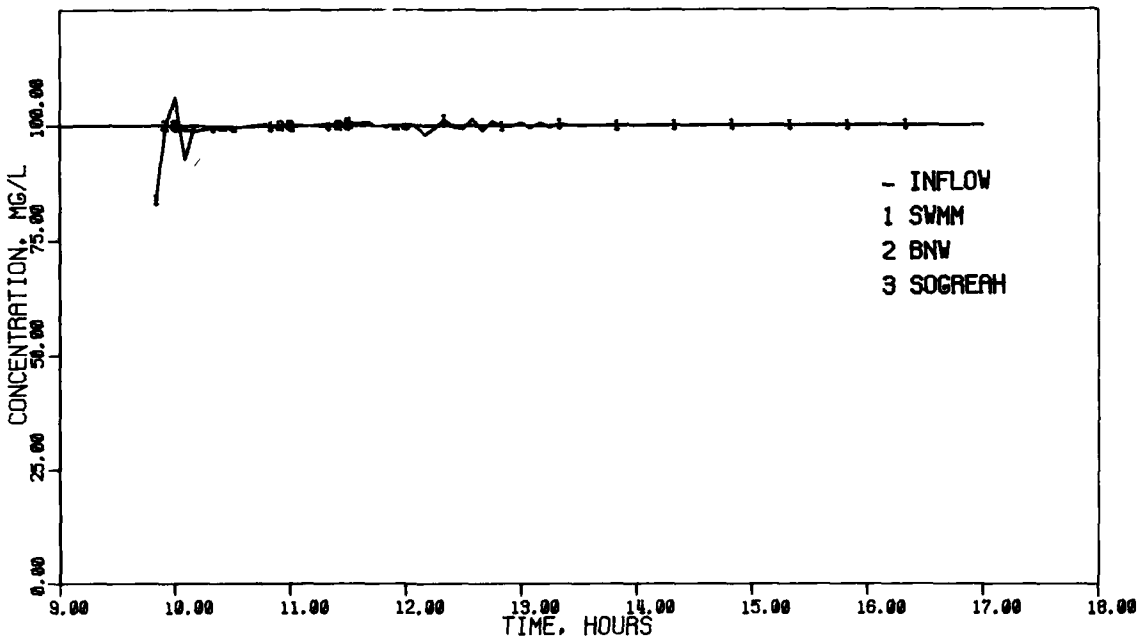


Figure 54. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 5% Slope

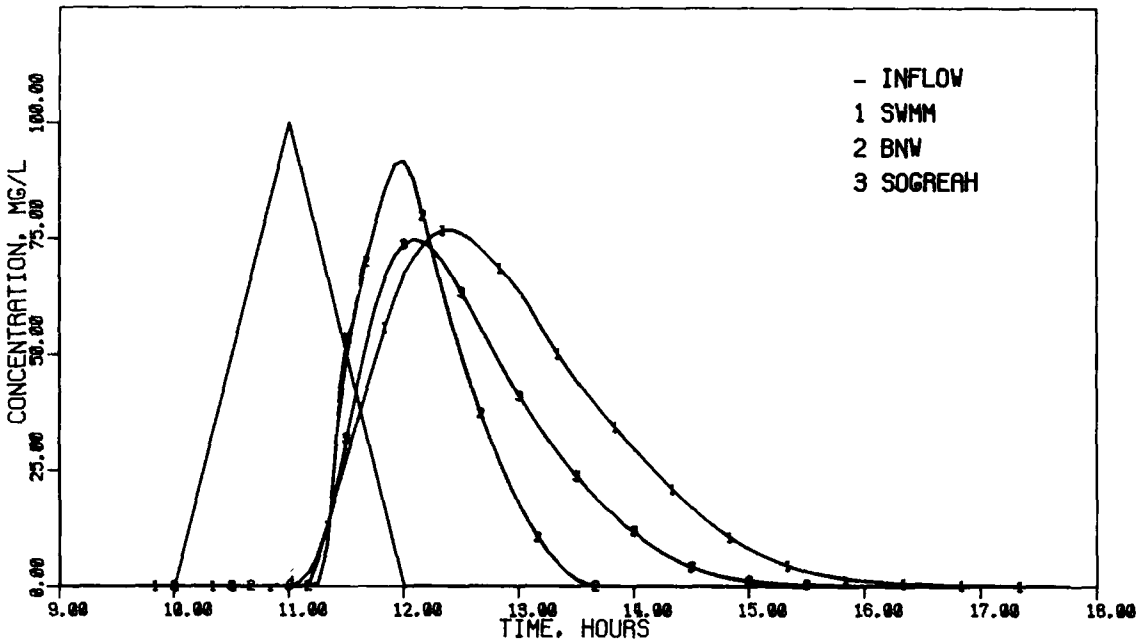


Figure 55. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.05% Slope

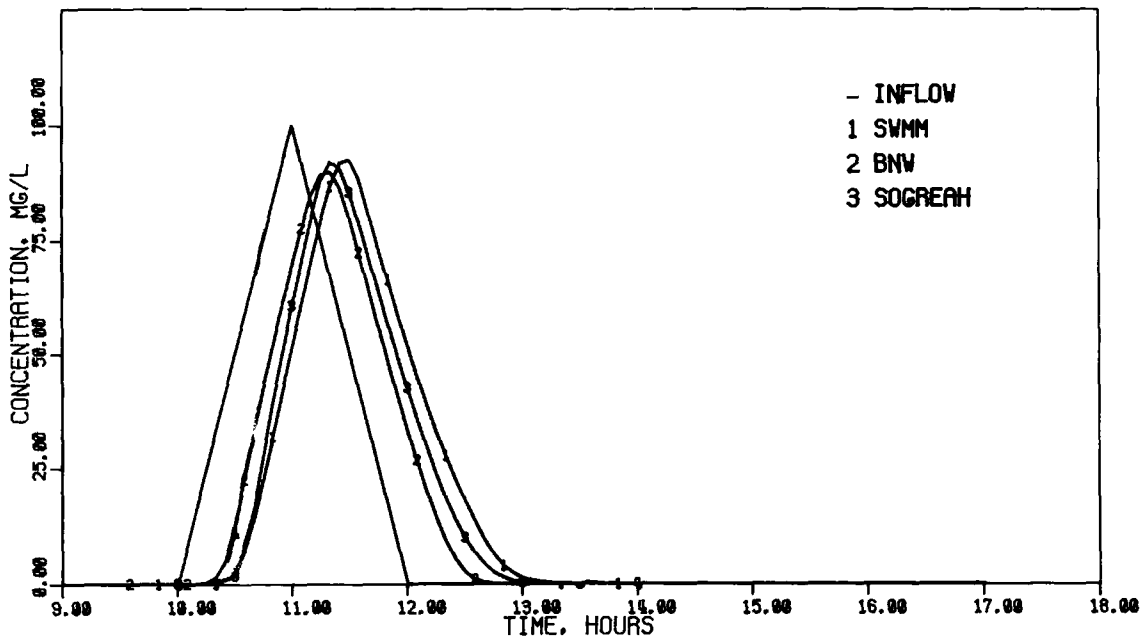


Figure 56. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.5% Slope

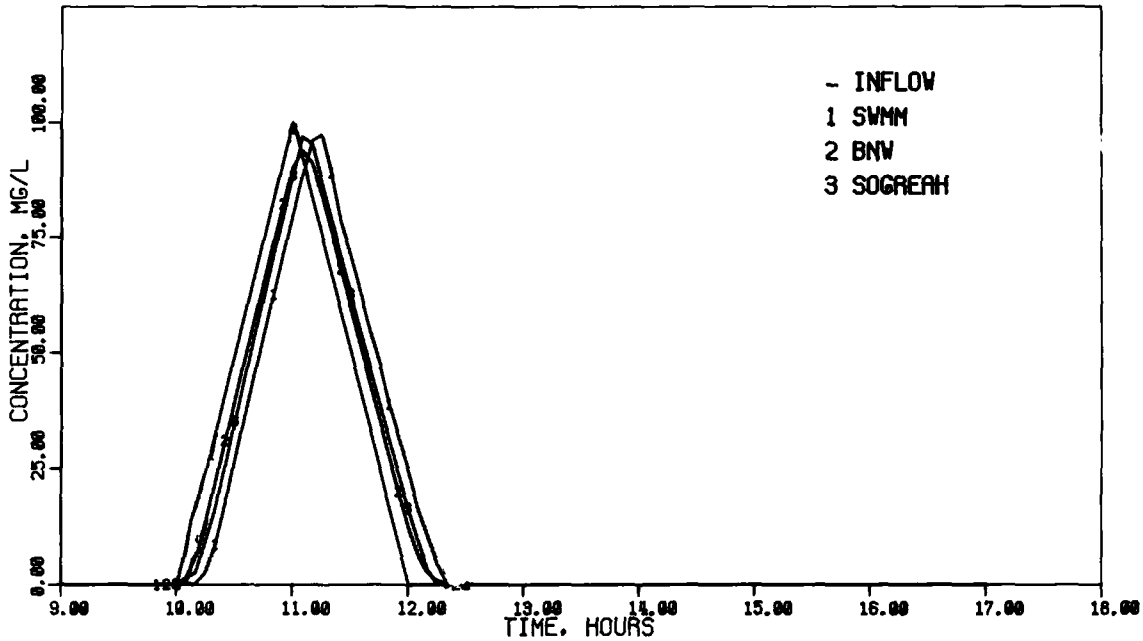


Figure 57. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 5% Slope

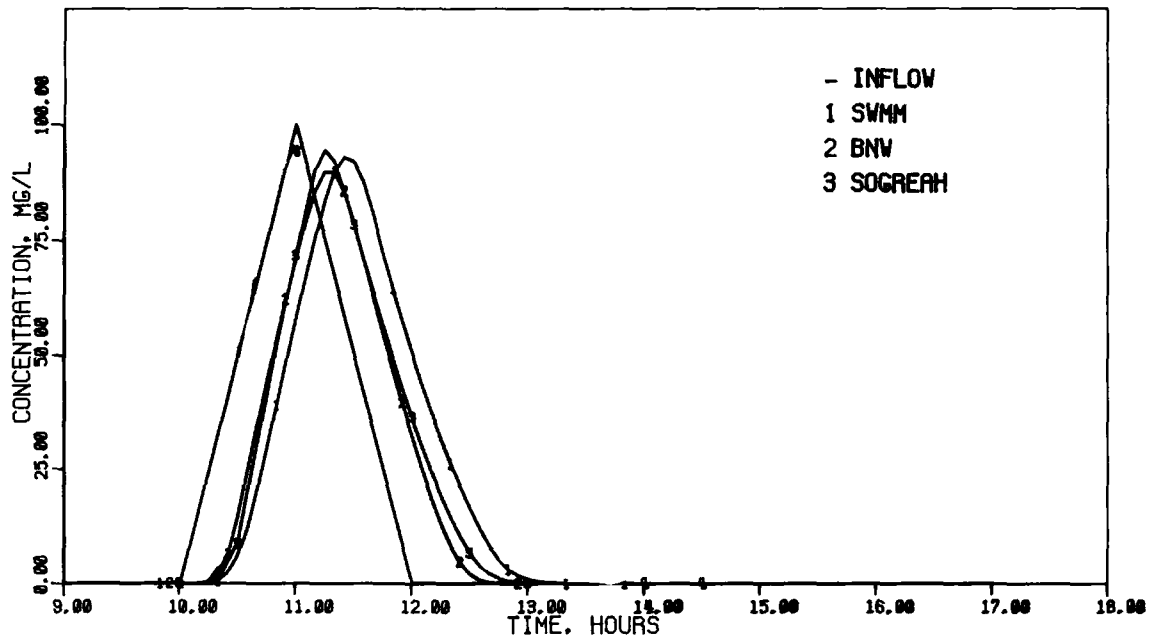


Figure 58. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.05% Slope

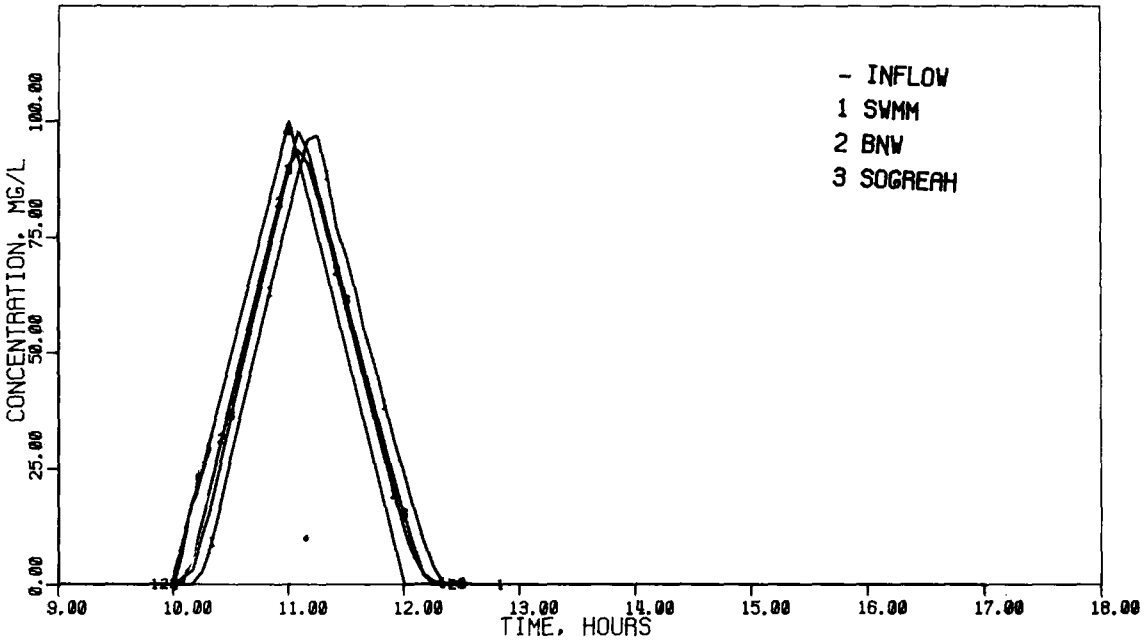


Figure 59. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.5% Slope

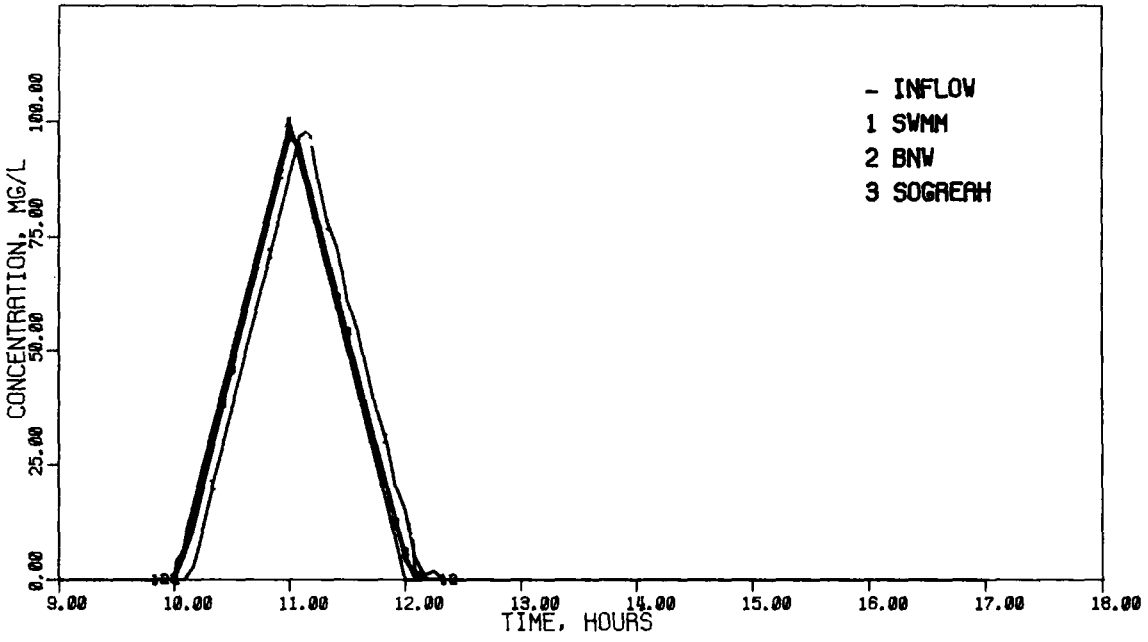


Figure 60. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 5% Slope

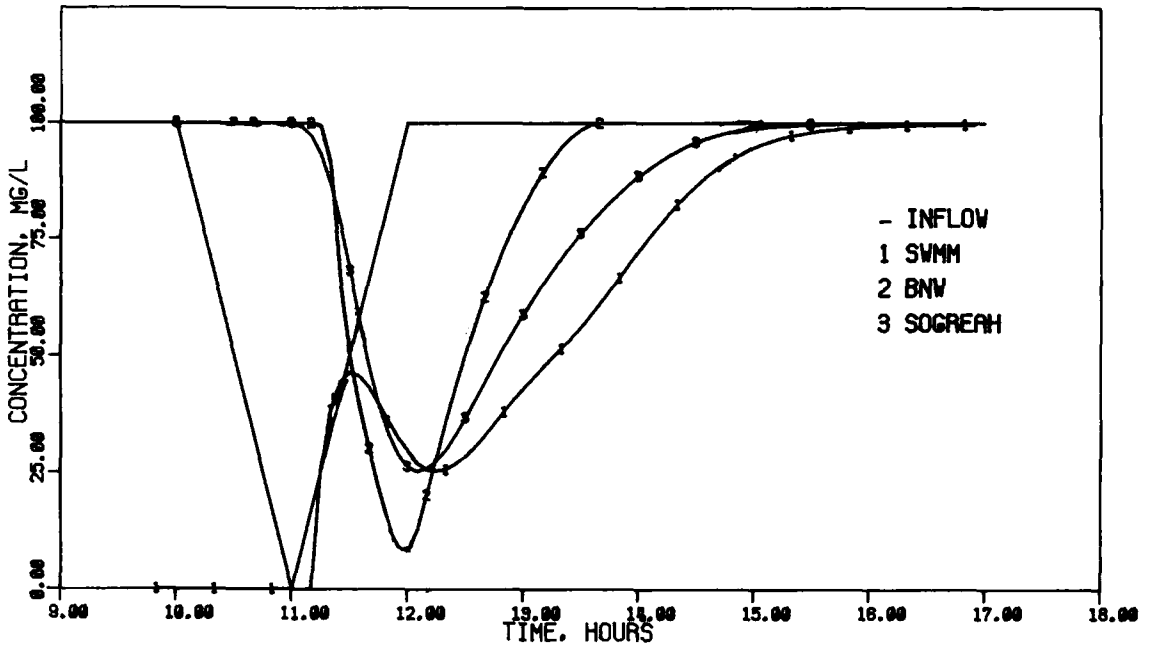


Figure 61. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.05% Slope

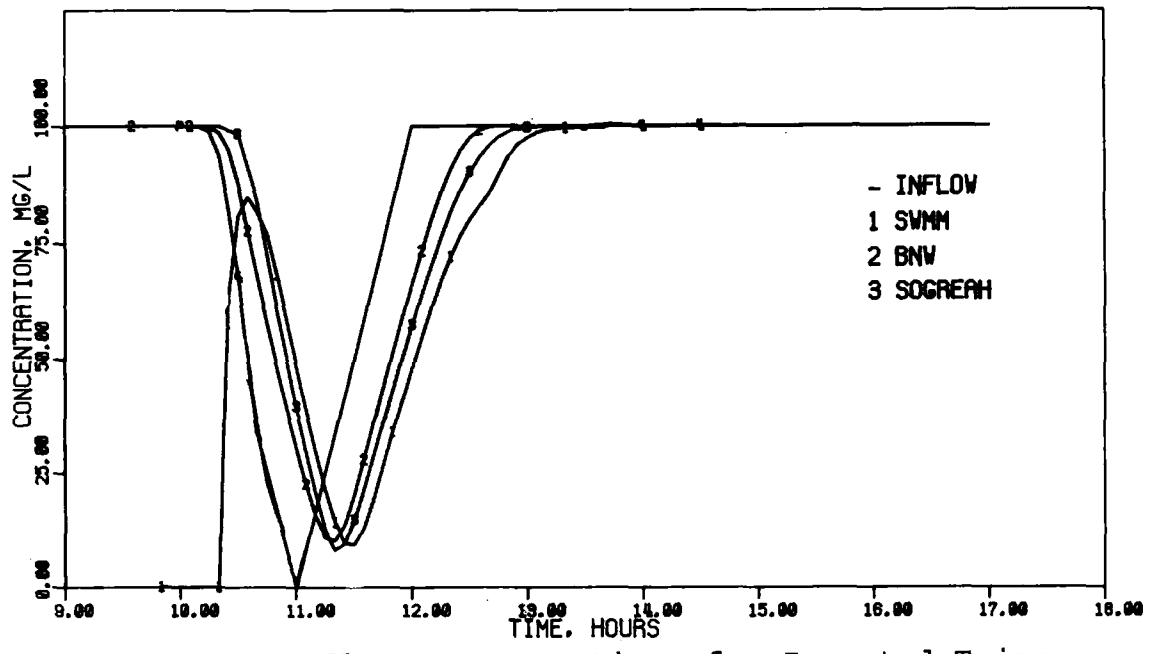


Figure 62. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.5% Slope

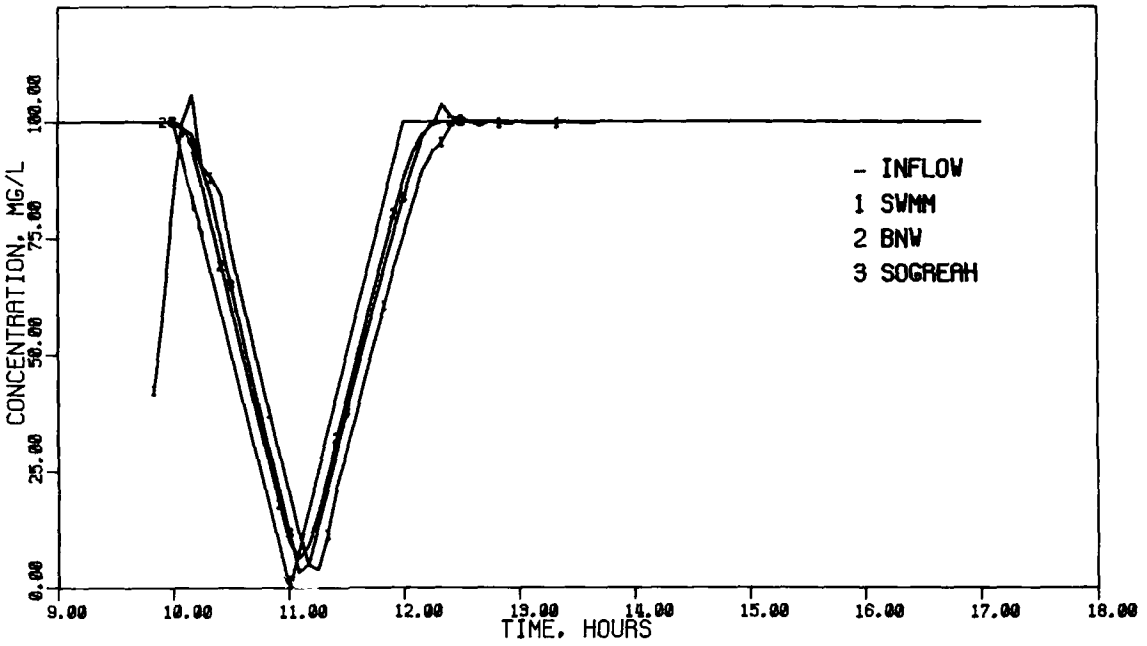


Figure 63. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 5% Slope

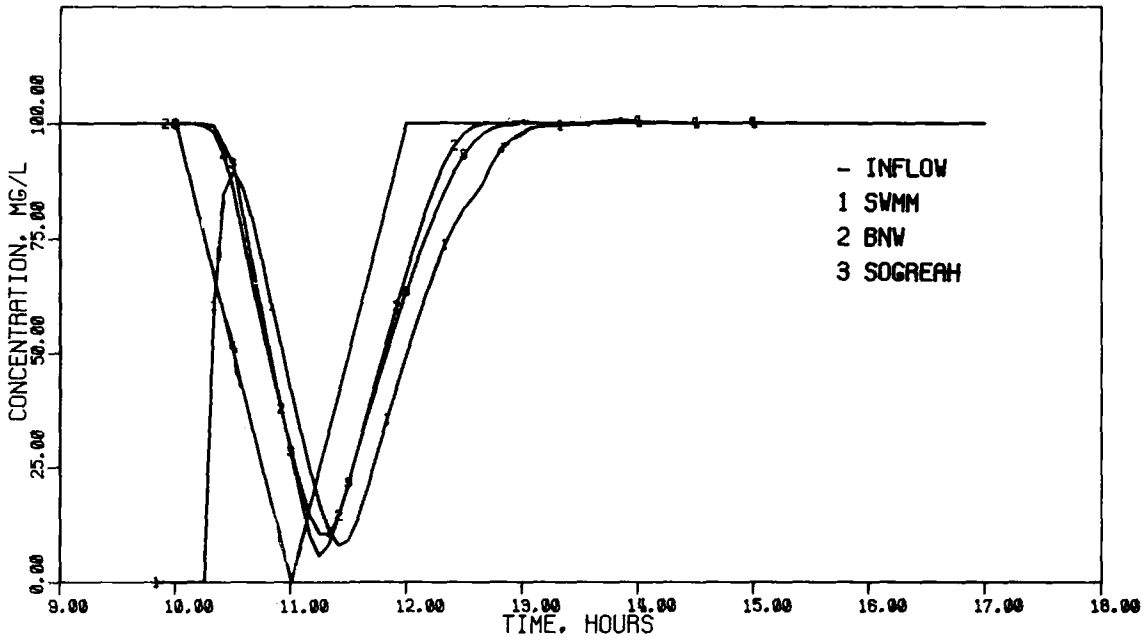


Figure 64. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.05% Slope

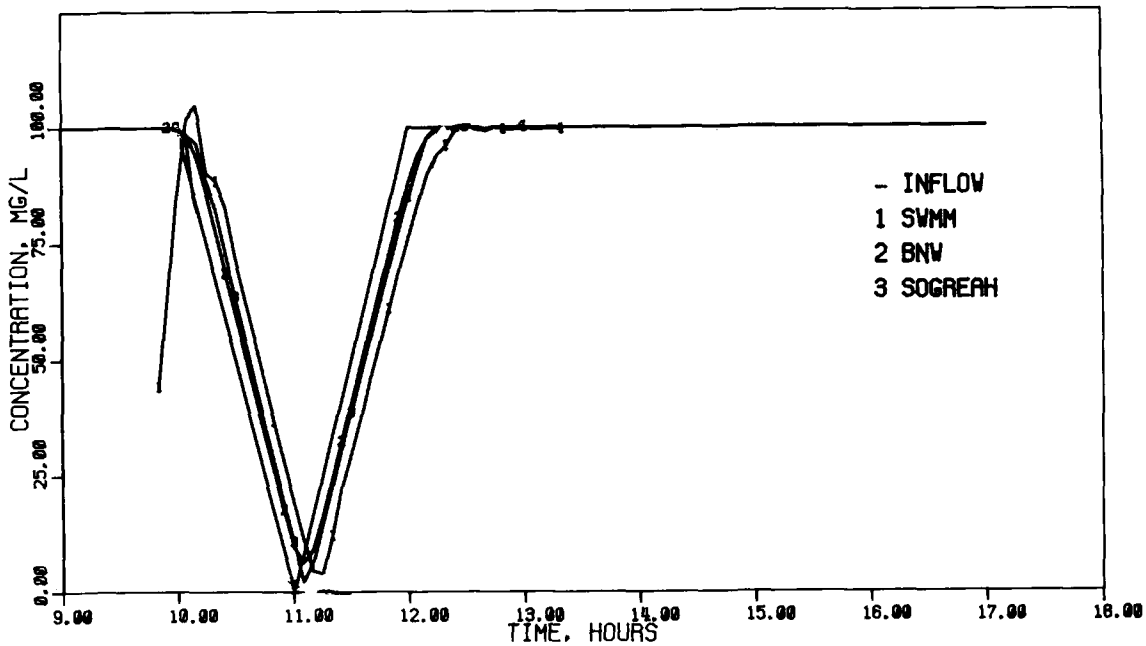


Figure 65. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.5% Slope

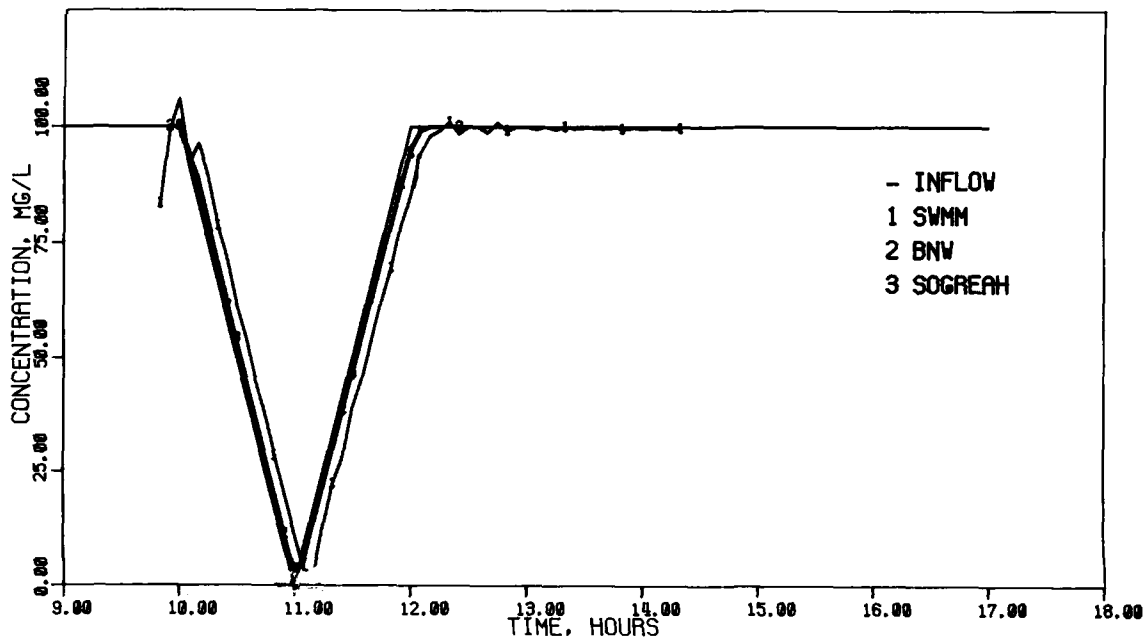


Figure 66. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 5% Slope

the 2-hr triangular inflow and all three inflow qualities for the free inflow and outflow conditions. Outflow hydrographs are plotted for all seven tested models and outflow concentration graphs are plotted for the five models which include water quality simulations (SWMM, BNW, DORSCH, SOGREAH, and WRE). The results of the water quality routing by the DORSCH and WRE models, however, were received too late to be incorporated and evaluated in this section. They are presented in Appendix E as submitted by these two firms.

As indicated in the tables, output is not listed for all data combinations due to various model limitations. The SWMM terminates execution and the routed hydrograph is not printed if the storage tank overflows. This occurs for most inflows into the small pipe and for the steep large pipe for the second boundary condition.

The FSP model does not include diversion formulations and consequently could not simulate the pipes with the upstream storage and downstream diversion structure.

The BNW and FSP models could not simulate the 1-hr triangular inflow data combinations. To read inflow hydrographs for the pipe flow routing directly without computing them from rainfall, the data can be provided only as hourly dry-weather flow and concentration values, which are then interpolated linearly by the models for smaller time steps. Consequently, the 1-hr triangular inflow could not be read since it has a peak at the 1/2 hr. The linear interpolation for time steps of less than 1 hr was added to the FSP by Battelle in order to perform the tests.

Model Comparison

Comparable outflow hydrographs are computed for the free inflow and outflow conditions by all models except the FSP which shows considerable differences from the other models as a result of its linear storage routing scheme (Tables 65 and 67). No significant patterns can be detected either from inspection of the complete outflow hydrographs or from the magnitudes and times of the peak outflows between the SWMM, BNW, DORSCH, MIT, SOGREAH, and WRE models, which would indicate that the models which solve the dynamic wave equations (DORSCH, SOGREAH, and WRE) show similar results while the models which solve the kinematic wave equation (SWMM, BNW, MIT) show different results. It appears that the numerical solution techniques employed by the models introduce enough numerical dispersion to obscure differences in the flow phenomena considered

by the dynamic and kinematic wave equation for the simple flow conditions defined for the first boundary condition. Greater differences between the kinematic and dynamic wave solution could be expected, however, in complex networks with loops and significant backwater and flow reversal.

Although no definite patterns of relative model performance are evident, the differences among model results are generally greatest for the small flat pipe and decrease with increasing pipe diameter and slope. Up to 25 percent differences in the magnitude and 15 minutes in the time of the peak outflow can be observed for the flat pipes, while the corresponding differences are generally less than 15 percent and 5 minutes for the steep pipes.

An exception is the FSP model, which computes much lower and later (over 1 hr) peaks for the small flat pipe and comparable peaks generally more than 45 minutes later for most other pipe data combinations. Only the 4-hr triangular inflows into the medium slope pipes produce comparable peak times.

Considerably greater differences among the models occur for the cases with upstream storage and downstream diversion (Tables 65, 66, and 67). The differences in the total outflow hydrographs among models are comparable to those for the free inflow and outflow cases as long as the pipe does not surcharge and the storage tank does not overflow.

The discharge from a full upstream storage tank into the pipe, assuming free surface flow in the pipe, is 1.484 m^3/sec (52.40 cfs) for the small pipe and 87.24 m^3/sec (3080.5 cfs) for the large pipe if the pipe axis intersects the bottom of the tank. The values are 1.421 m^3/sec (50.17 cfs) and 78.64 m^3/sec (2776.7 cfs), respectively, if the pipe invert intersects the bottom of the tank.

Surcharging occurs when the inflow to the diversion structure exceeds the maximum combined discharge capacity of the orifice and weir under free-surface, near-full pipe flow conditions. This is 0.264 m^3/sec (9.33 cfs) for the small pipe and 23.32 m^3/sec (823.3 cfs) for the large pipe as computed from the standard orifice and weir equations, neglecting the effect of the velocity of approach. This implies that both the small and large pipes are expected to surcharge for the medium and steep slopes, since the peak inflows are several times these values.

Considerable differences exist among the models, however, in the diversion even if the total outflows are similar (Table 66). Since all but the SWMM use similar weir and orifice

equations for the diversion computations, the differences can be explained only by differences in computing the upstream heads for the weir and orifice and interfacing of the routing with the diversion equations. Sufficient information is not available to explain differences in the proprietary models.

The SWMM model terminates execution if it computes an overflowing storage tank. The BNW, DORSCH, MIT, and WRE models do not terminate execution but assume free orifice flow from the upstream tank into the pipe and neglect the effect of the surcharging on the orifice discharge. The SOGREAH model considers the entire system as surcharged and computes much higher inflow from the tank into the pipe and (consequently much higher downstream outflow). This results in peak outflows which vary with the inflow hydrograph for the SOGREAH model, but the same peak outflows regardless of hydrograph shape for the other five models for the same slope and diameter. Differences in the times of the peak discharges between the models are generally less than 15 minutes and do not seem to be affected by surcharging.

Comparable routed water quality concentrations are computed by the BNW and SOGREAH models (Tables 68, 69, 70, and 71). As usual, the differences between model results are greater for small pipes and low slopes. The BNW shows earlier and higher maxima and lower minima than the SWMM and SOGREAH models due to its characteristic solution of the kinematic wave equation for the flow routing and the use of the kinematic wave celerity, rather than flow velocity, for pollutant routing. The differences become smaller as the pipe diameter and slope increase.

Two-Hour Triangular Inflow

The outflow hydrographs computed by each of the seven tested models are plotted for six pipe data combinations in Figures 43 to 48. The models were separated into two groups for plotting the outflow hydrographs to prevent too many overlapping lines from obscuring model differences. Each plot includes the outflow computed by the SWMM and three other models. The corresponding outflow concentrations computed by the SWMM, BNW, and SOGREAH models are plotted in Figures 49 to 66.

The plotted outflow is for the 2-hr triangular inflow which has a peak flow rate of 90 percent of full pipe flow as computed by Manning's equation. A constant flow rate of 10 percent of the peak inflow rate (9 percent of full pipe flow) precedes and follows the constant inflow. Three conservative inflow water quality constituents were specified for each inflow hydrograph: a constant continuous concentration of 100

mg/l, a triangular concentration distribution with 100 mg/l maximum concentration and 0 mg/l before and after the triangular distribution, and inverted triangular concentration distribution with an 0 mg/l minimum concentration and 100 mg/l before and after the triangular distribution.

The plots are for 0.61 m (2 ft) and 3.66 m (12 ft) diameter circular pipes, each 3048 m (10,000 ft) long with a Manning's roughness coefficient of 0.01 and with 0.05, 0.5, and 5 percent slopes. Free upstream inflow and downstream outflow are assumed for the selected examples. The plots consequently show the effects of differences in pipe diameter and slope on the routed hydrographs and water quality graphs.

The SWMM, FSP, and MIT models are not formulated to start the simulation with nonzero initial flow conditions in the pipe, consequently the outflow hydrographs begin with a value of zero flow, rather than the constant 10 percent of the peak flow which was specified for the period preceding the triangular inflow. The simulation was started several time steps prior to the beginning of the triangular hydrographs for all models to compute initial conditions in the pipes for the initially constant inflows.

The overall shape of the computed outflow hydrographs, including the magnitudes and times of the peak outflows, are quite similar for all models except the FSP. The linear storage routing scheme of the FSP produces outflow hydrographs with long time lags between the beginning of the rising portion of the inflow hydrograph and the first outflow. For the small pipe, the outflow hydrograph rises and falls very slowly, resulting in a 25 percent lower and 1 hr later peak than the other models (Figure 43). As the slope increases, the computed outflow hydrographs rise and fall more rapidly. For the medium slope, the shape and peak of the outflow hydrograph are similar to the other models but the peak occurs about 45 minutes later (Figure 44). For the steep slope, a very steeply rising outflow hydrograph reaches a constant outflow equal to the maximum inflow for a short time and then drops very rapidly following cessation of the triangular inflow (Figure 45).

For the large flat pipe, the hydrograph shape is similar to the other models, the peak discharge is approximately the same as for the SOGREAH model, which is about 5 percent to 10 percent higher than the other models, but it occurs about 40 to 50 minutes later than all other models (Figure 46). For the medium and steep slope, a sharply rising and falling outflow hydrograph is computed with a fairly constant outflow equal to the maximum inflow for a short period (Figures 47

and 48). In large steep pipes, the linear storage routing scheme apparently stores the inflow for some time before any outflow occurs and then dumps it at a high rate until the inflow ceases.

The outflow hydrographs of the remaining six models are very similar in shape and in the magnitude and time of the peak discharge. Differences in the magnitude of the peak discharge among these models for the plotted data combinations are less than 20 percent for the flat slope and less than 5 percent for the steep slope. The differences are greatest for the flat slope and decrease as the slope increases. Differences in the times of the peak discharges are less than 15 minutes for the flat slope and less than 5 minutes for the steep slope. This indicates that both the kinematic wave and dynamic wave formulations can be expected to compute similar routed hydrographs for flows ranging from sub- to supercritical conditions, provided backwater effects, surcharging, and flow reversal do not occur or are insignificant.

No consistent pattern in the differences of performance is apparent for the DORSCH, SOGREAH, and WRE models, which use the dynamic wave equations, or for the SWMM, BNW, and MIT models, which use the kinematic wave equation for the conduit flow routing. Numerical dispersion caused by the different numerical solution techniques obscures differences in the basic routing equations for the first boundary condition.

Plots of outflow concentrations are shown only for the SWMM, BNW, and SOGREAH models. Concentration plots for the DORSCH and WRE models were received too late to be incorporated with the other plots. They are shown in Appendix E. The other tested models do not include water quality formulations. All plotted outflow concentrations are the result of routing the specified inflow concentrations with the 2-hr triangular inflow hydrographs.

The BNW and SOGREAH models route the constant inflow concentration without modification for both pipe sizes and all three slopes (Figures 49 to 54). The SWMM assumes a zero initial condition and consequently computes a rising limb which reaches the constant concentration value after the start of the inflow hydrograph. The lag corresponds to the travel time through the pipe. The computed outflow concentration then fluctuates about the constant concentration value until they coincide several hours after the beginning of the triangular inflow.

Similarly shaped outflow concentration graphs are computed by the SWMM, BNW, and SOGREAH model for the triangular inflow concentration for the medium and steep slopes (Figures 55 to

60). The differences increase, however, with decreasing pipe size and slope. The differences are still small for the large flat pipe but become significant for the small flat slope. The BNW model computes an earlier peak with a higher maximum for the upright triangular concentration and a lower minimum for the inverted triangular concentration than do the SWMM and SOGREAH models, since its mass routing uses the kinematic wave celerity rather than the water flow velocity to facilitate interfacing with the real-time control optimization of the model. The results show, however, that the effect of this approximation is insignificant for large pipes, such as trunk and interceptor sewers for which the model was designed.

The SWMM shows the same phenomenon with the routed inverted triangular concentrations as with the routed constant concentration (Figures 61 to 66). The computed outflow concentrations start first with zero rather than 100 mg/l, then rise for some time until they reach a peak and finally follow the shape of the inverted concentration graphs. This implies that for practical applications, the user has to estimate the longest flow travel time from an upstream inflow point to the downstream outflow point and initiate the simulation of a corresponding time period earlier to initialize the flows and concentrations in the system, and not use the values computed for this initial period.

REAL CATCHMENT DATA TESTS

Real catchment data tests were performed with data from the Oakdale catchment in Chicago, Illinois and the Bloody Run catchment in Cincinnati, Ohio. Four runoff events of the Oakdale catchment were simulated. The initial testing raised certain doubts with respect to the completeness and accuracy of the Bloody Run data, and consequently tests were completed for only two of the four storms reported in Section VI.

Oakdale Avenue Catchment Simulations

The Oakdale Avenue catchment is located in an urban area about 6 miles northwest of downtown Chicago, Illinois. It has a drainage area of 5.22 ha (12.9 acres, approximately 2-1/2 blocks long by 1 block wide) and consists entirely of residential lots and adjoining streets.

The DORSCH model simulations were performed by Drosch Consult and the SWMM, FSP, and BNW model simulations by Battelle. The same spatial discretization of catchment and sewer elements was used by all four models. Runoff was routed directly from the catchment into the sewers without routing it separately

through the gutters except the Dorsch model. The same empirical model coefficients were used by all models for phenomena being modeled by the same equations.

The maximum infiltration, the minimum infiltration, and the decay rate of infiltration were assumed to be 63.5 mm/hr (2.50 in./hr), 11.4 mm/hr (0.45 in./hr), and 0.00115 sec^{-1}, respectively. The overall depths of retention storage on the pervious and the impervious areas were assumed to be 5.08 mm (0.20 in.) and 2.03 mm (0.08 in.), respectively. For storms which had to be simulated in two parts, the initial infiltration rate for the second part was set equal to the minimum infiltration rate.

The rainstorms on the following four dates were selected to test the catchment runoff and sewer routing formulations of the four models: May 19, 1959; July 2, 1960; July 26, 1960; and August 2, 1963. These rainstorms represent a typical range of possible combinations of rainfall intensities and durations.

The model results are summarized in Table 72, which lists the magnitudes and times of the peak discharges for each storm.

Figures 67 to 82 show comparisons between the measured runoff and the runoff computed by the BNW, SWMM, FSP, and DORSCH models. Each computed hydrograph was plotted in a separate figure to better show the variation from the measured runoff. Plotting of all computed runoff hydrographs for a given storm in the same figure may provide better comparisons among models, but preliminary plots showed that the lines would be too close together to adequately distinguish among the different model hydrographs. Runoff periods lasting longer than 2 hours and 15 minutes are plotted on more than one figure, with a 15-minute overlap between successive figures.

The measured rainfall and runoff data were reported at 1-minute time intervals. The same interval was chosen for the SWMM and DORSCH model simulations. Due to model limitations, 5-minute time intervals were used for the BNW model and FSP. The BNW model is restricted to a minimum time step of 2 minutes and a maximum of 56 time steps. The 5-minute time interval was selected to reduce the need for dividing a long runoff period into too many segments for the simulation. The FSP requires input data and simulation for a minimum of one day and is restricted to a minimum time step of 5 minutes.

Figures 67 to 82 indicate that the SWMM, BNW, and DORSCH models can be expected to produce satisfactory runoff hydrographs in most cases for small catchments similar to the

Oakdale Avenue catchment. The FSP, however, does not produce satisfactory results, which confirms the model developers' original intent to use the model only for the continuous simulation of runoff from large, mostly unsewered catchments (greater than 1.3 km^2 or 0.5 mi^2) using 1-hr time steps. FSP formulations are not adequate to accurately simulate runoff from small sewered areas requiring short time steps. The model's assumption of immediate runoff from sewered areas produces peaks which are generally too high and occur too early. The assumption that all rain falling on impervious areas and a constant fraction falling on pervious areas becomes runoff generally produces also a higher volume of runoff than measured.

The use of 5-minute time steps for the BNW model results in smoothing of the peaks and valleys which appear in the measured values. Exceptions are the second peaks of the July 2, 1960 (Figure 72) and August 2, 1963 (Figure 80) storms which show computed peaks and runoff volumes higher than the measured values. This appears to be the result of the model's lack of formulations for catchment moisture balance between storms. The model does not compute the recovery of depression storage and infiltration capacity during dry periods, and consequently computes insufficient rainfall losses for the second runoff period of successive storms.

Similar overestimates of the second peaks and runoff volumes are observed for the SWMM for the August 2, 1963 storm (Figure 79) for the same reasons. The second peak and runoff volume of the storm of July 2, 1960 (Figure 71) are underestimated, however. This storm had to be split into two simulation periods since it was too long for a single run. The underestimate occurs although the infiltration value was set to the minimum for the second period.

Dorsch Consult simulated only the major runoff periods of each storm and split longer runoff periods into separate simulation periods. This generally produces satisfactory results, with the exception of overestimating the second peak and runoff volume of the storm of August 2, 1963 (Figure 82). This is probably caused by uncertain initial moisture conditions at the beginning of the second runoff period, since the model does not simulate a catchment moisture balance during dry periods (with the exception of the recovery of depression storage on pervious areas).

The numerical model testing indicates that equally satisfactory results can be obtained for the SWMM, DORSCH, and BNW models for small catchments and sewerage systems where backwater and surcharging conditions are nonexistent or negligible. Difficulties arise with these three models as a result of

Table 72. RUNOFF COMPARISONS FOR OAKDALE STORMS

Date	Peak No.	Peak discharge, cfs					Time of peak discharge, minutes					Peak discharge, m^3/sec				
		Observed	SWMM	BNW	FSP	DOR	Observed	SWMM	BNW	FSP	DOR	Observed	SWMM	BNW	FSP	DOR
5/19/59	1	7.25	7.77	5.90	11.27	8.1	17	17	20	15	15	0.21	0.22	0.17	0.32	0.23
7/02/60	1	4.60	3.89	3.45	7.34	4.3	15	15	15	15	13	0.13	0.11	0.10	0.21	0.12
	2	17.40	15.46	21.74	14.50	17.7	161	165	160	160	167	0.49	0.44	0.62	0.41	0.50
	3	*	11.23	12.47	9.80	**	*	192	185	185	**	*	0.32	0.35	0.28	**
7/26/60	1	2.50	2.01	1.81	6.00	**	12	10	10	5	**	0.07	0.06	0.05	0.17	**
	2	4.30	3.00	2.88	3.87	3.3	212	212	210	210	212	0.12	0.08	0.08	0.11	0.09
	3	2.90	2.44	3.21	3.48	3.2	262	263	265	260	263	0.08	0.07	0.09	0.10	0.09
8/02/63	1	4.85	4.43	3.32	7.31	5.5	18	18	20	15	18	0.14	0.13	0.09	0.21	0.16
	2	5.95	7.18	6.87	8.54	1.0	90	90	90	90	81	0.17	0.20	0.19	0.24	0.03

Notes: * No observation
** Not computed

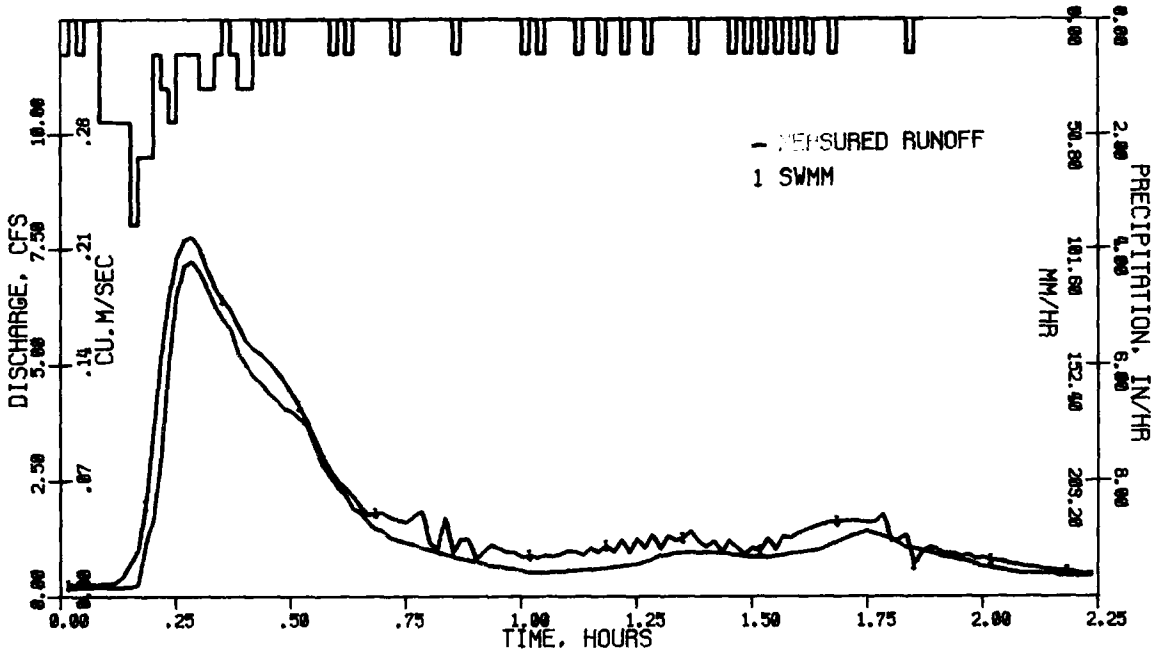


Figure 67. Comparison of Measured and Computed Runoff for the Storm of May 19, 1959 - Oakdale Avenue Catchment - EPA Storm Water Management Model

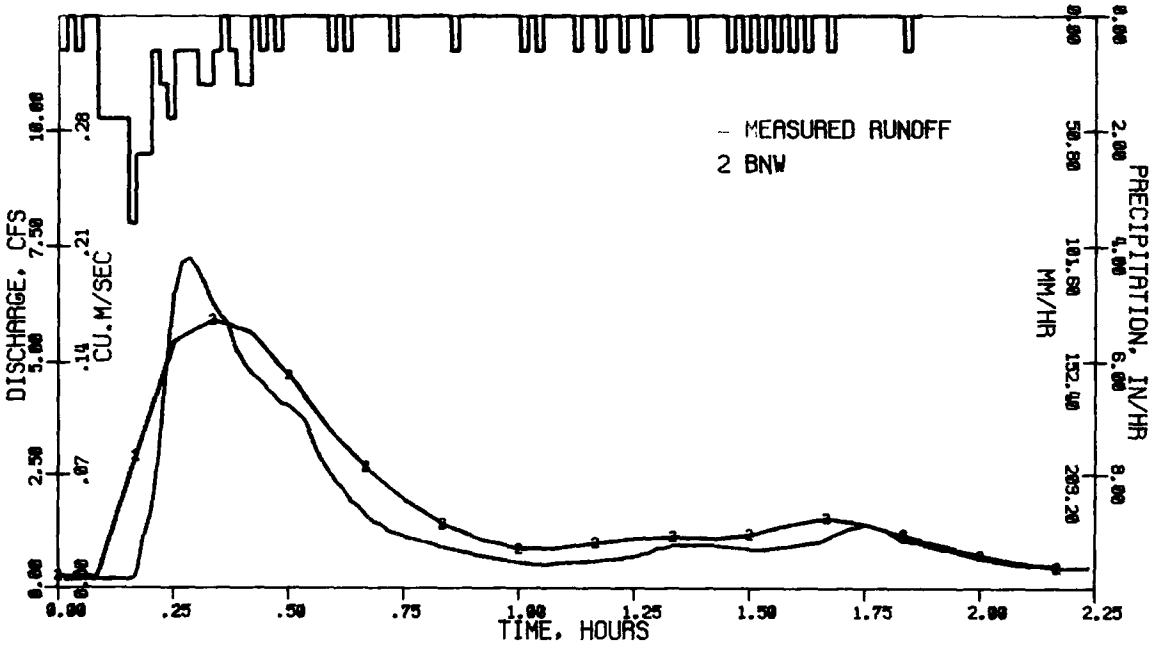


Figure 68. Comparison of Measured and Computed Runoff for the Storm of May 19, 1959 - Oakdale Avenue Catchment - Battelle Urban Wastewater Management Model

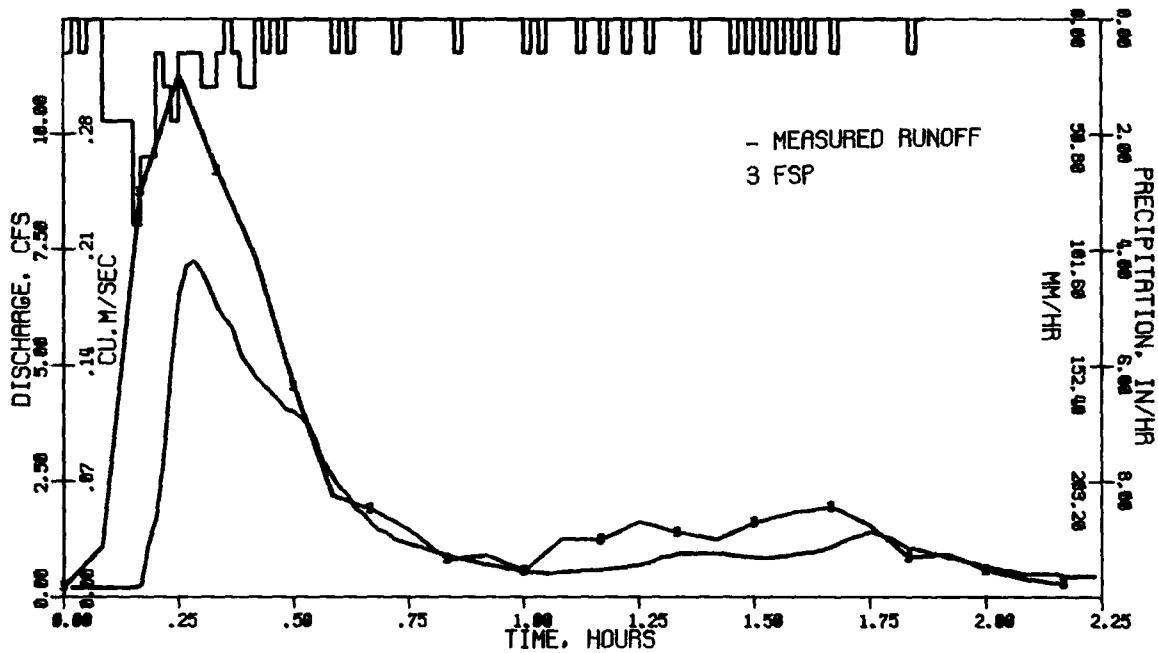


Figure 69. Comparison of Measured and Computed Runoff for the Storm of May 19, 1959 - Oakdale Avenue Catchment - Chicago Flow Simulation Program

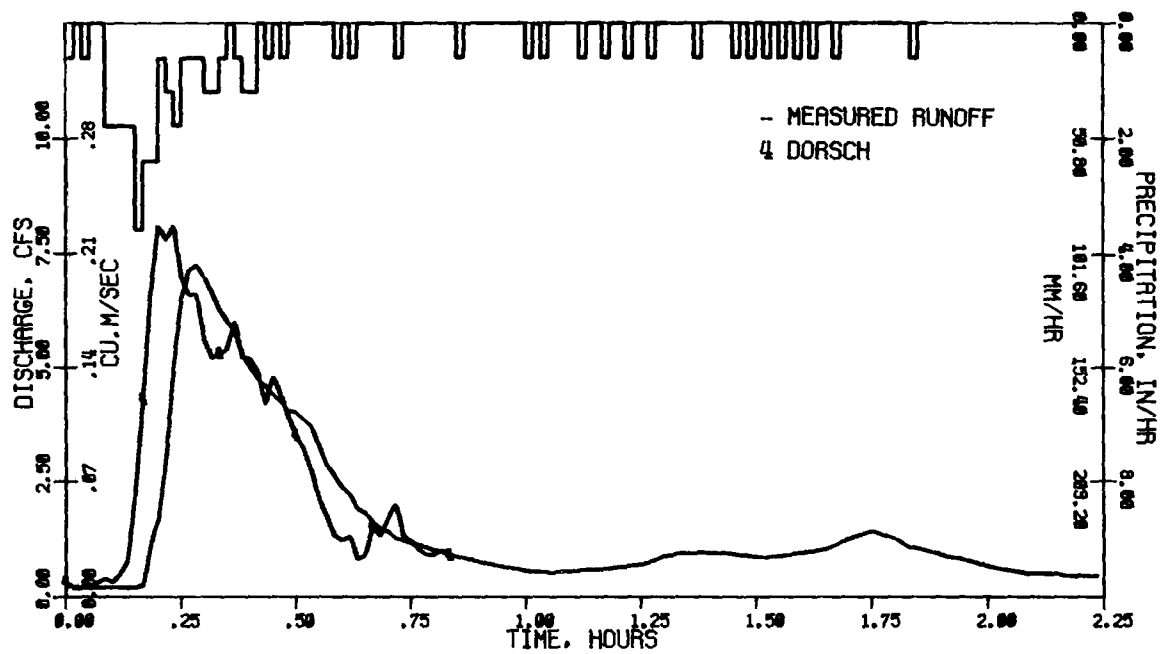


Figure 70. Comparison of Measured and Computed Runoff for the Storm of May 19, 1959 - Oakdale Avenue Catchment - Dorsch Hydrograph Volume Method

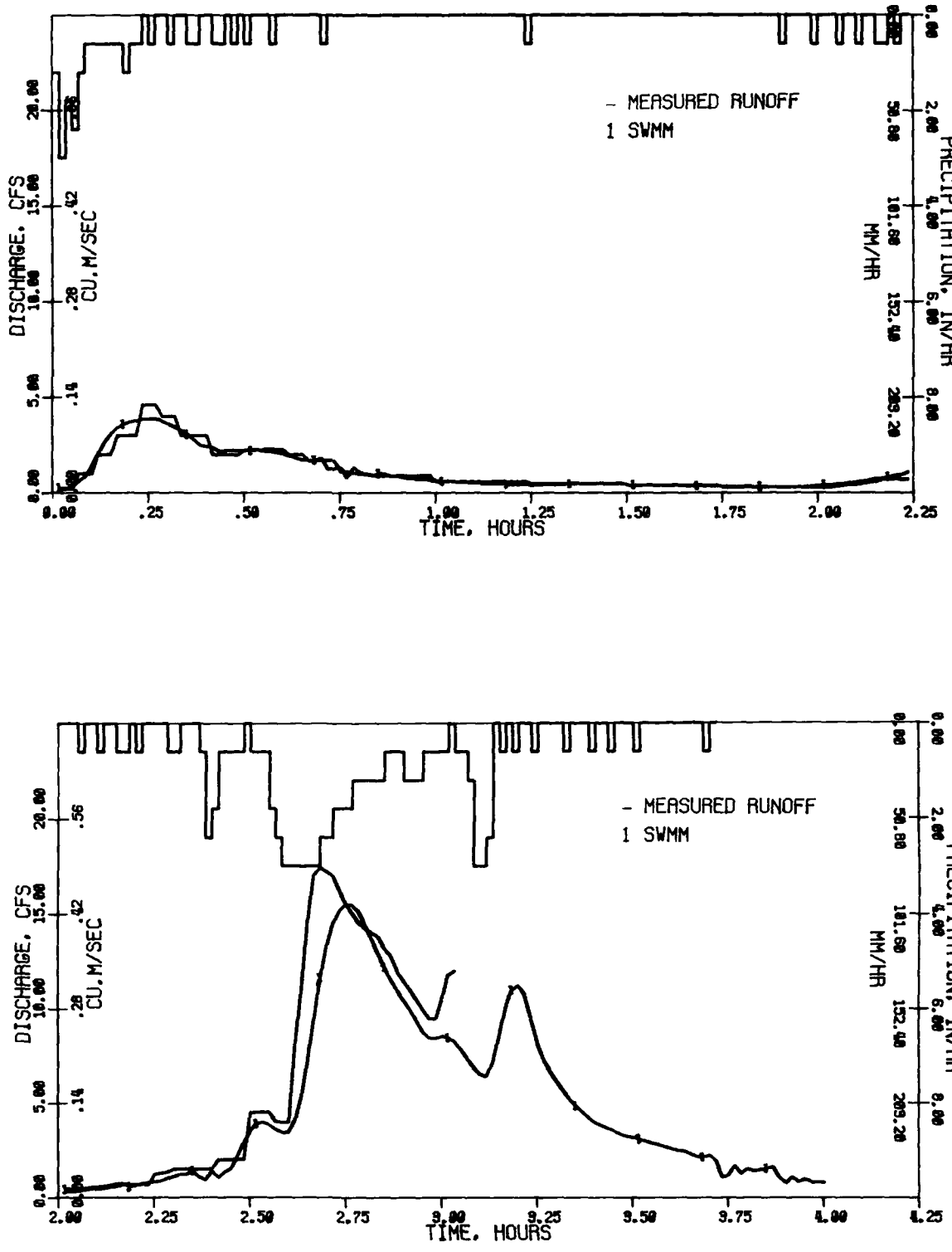
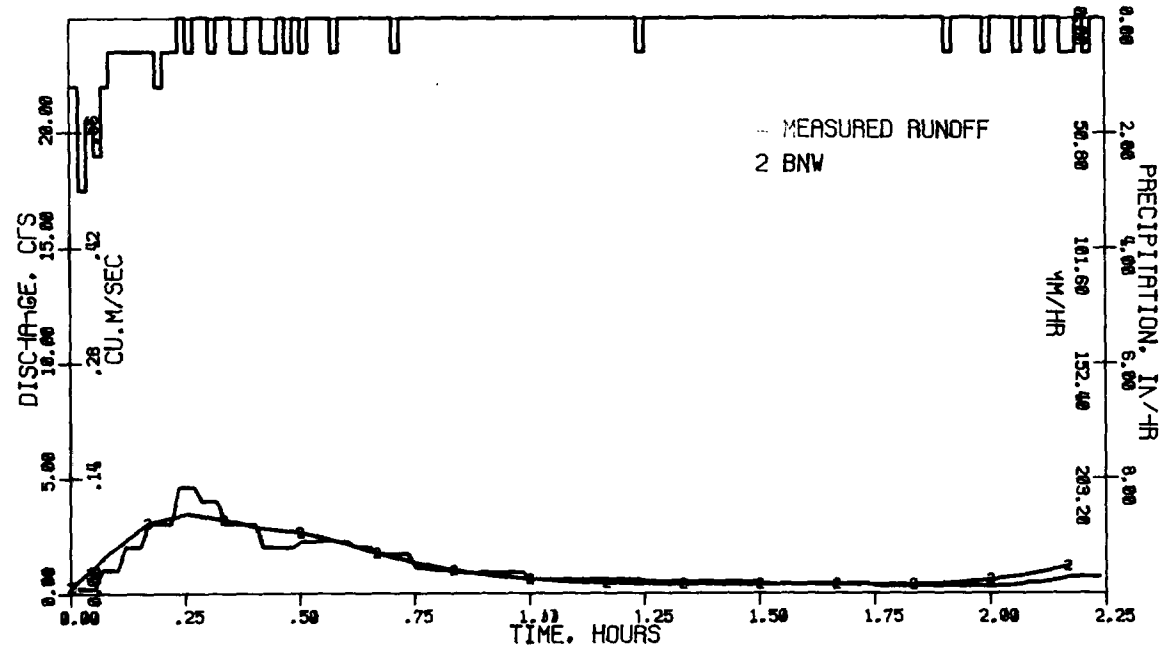


Figure 71. Comparison of Measured and Computed Runoff for the Storm of July 2, 1960 - Oakdale Avenue Catchment - EPA Stormwater Management Model



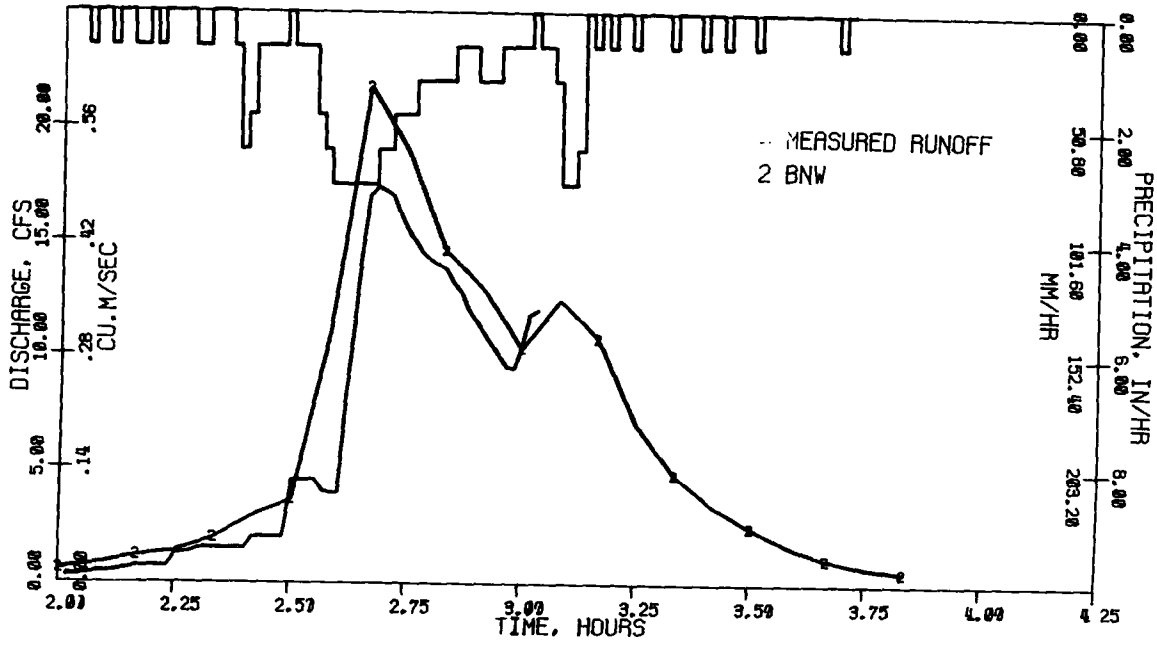
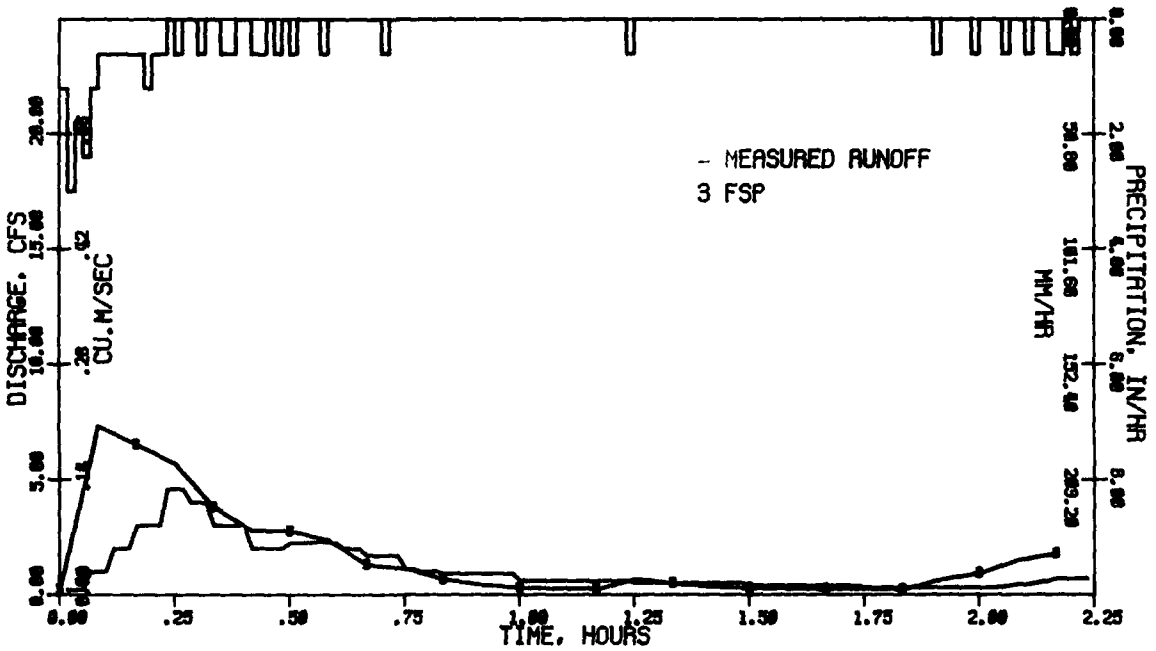


Figure 72. Comparison of Measured and Computed Runoff for the Storm of July 2, 1960 - Oakdale Avenue Catchment - Battelle Urban Wastewater Management Model



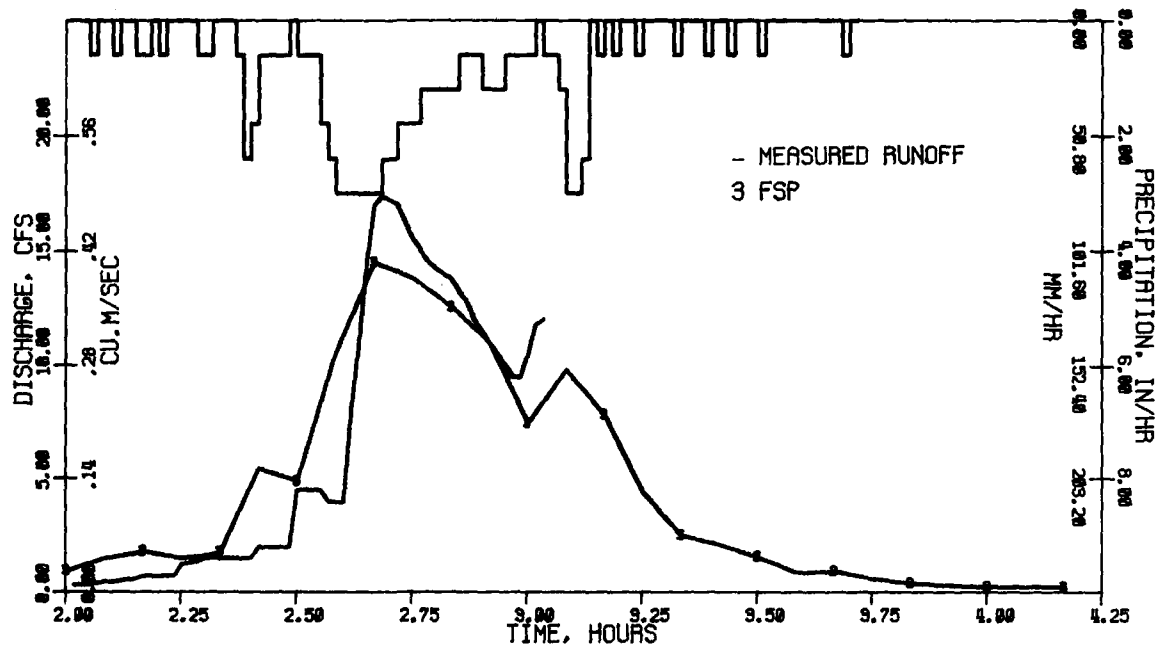
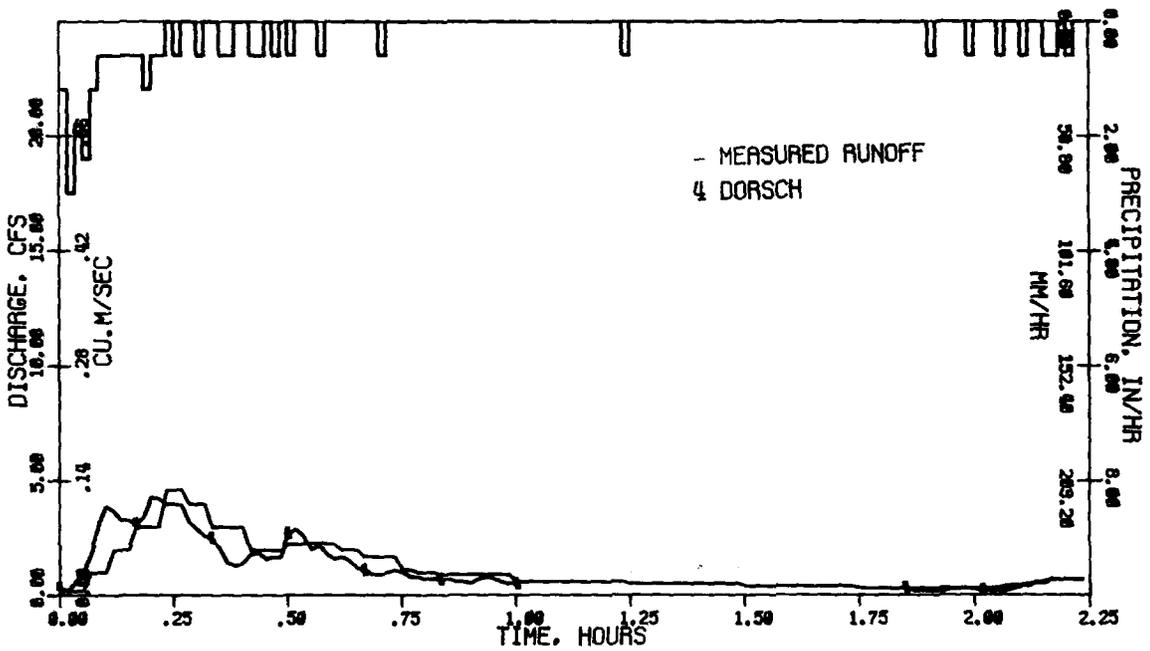


Figure 73. Comparison of Measured and Computed Runoff for the Storm of July 2, 1960 - Oakdale Avenue Catchment - Chicago Flow Simulation Program



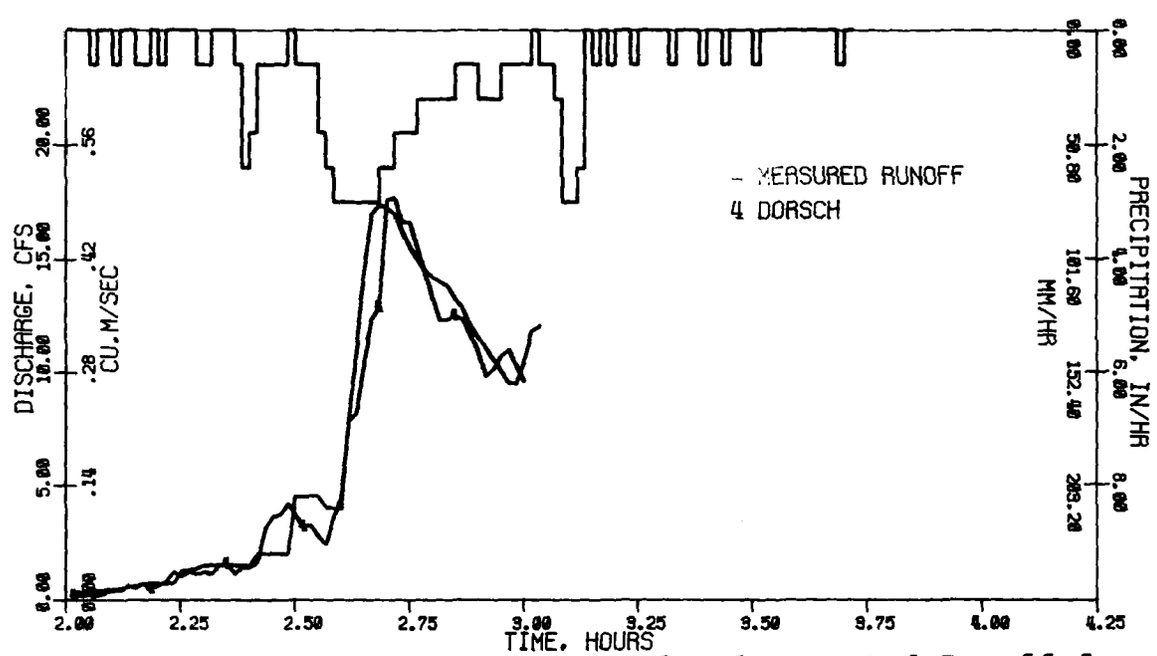
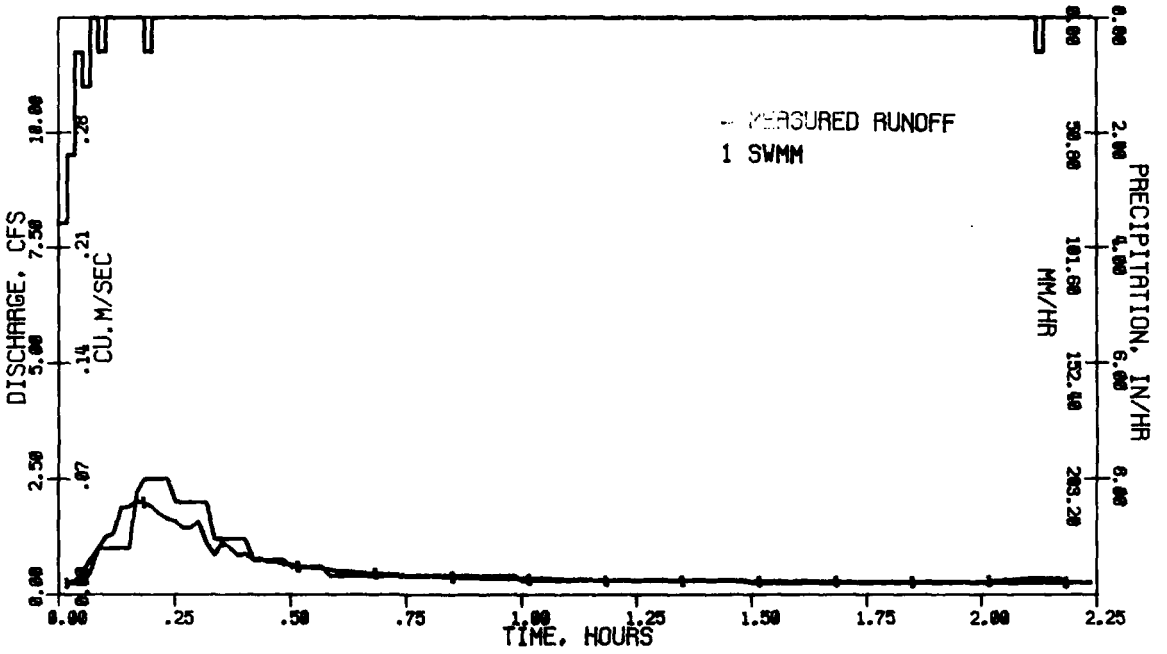


Figure 74. Comparison of Measured and computed Runoff for the Storm of July 2, 1960 - Oakdale Avenue Catchment - Dorsch Hydrograph Method



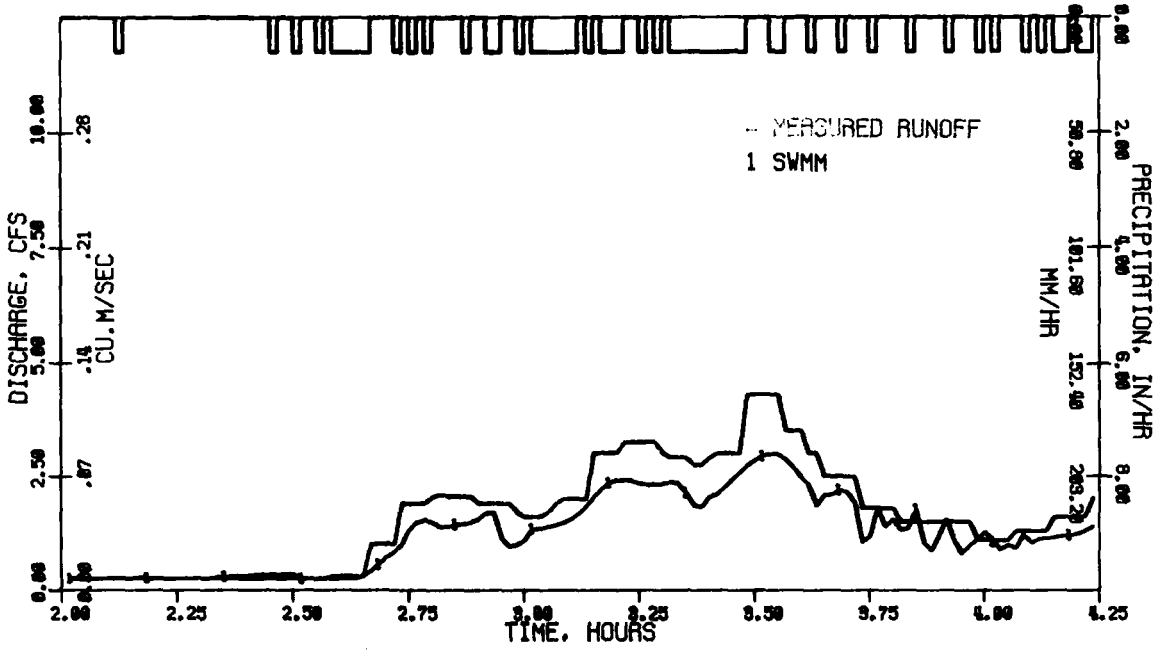


Figure 75a. Comparison of Measured and Computed Runoff for the Storm of July 26, 1960 - Oakdale Avenue Catchment - EPA Stormwater Management Model

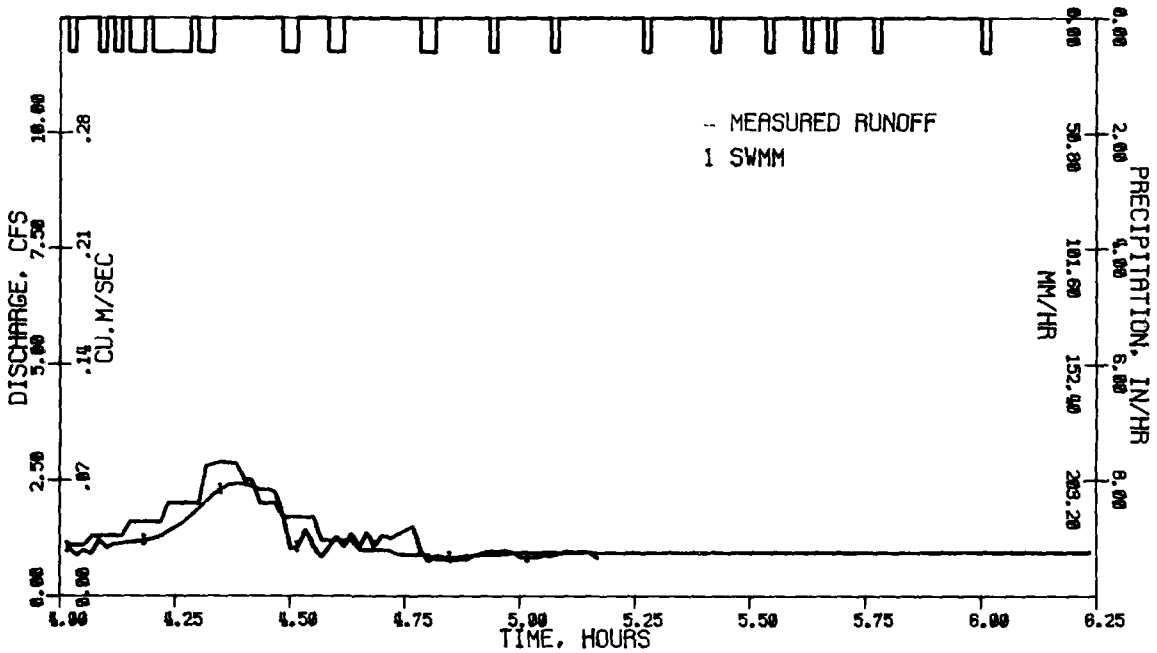


Figure 75b. Comparison of Measured and Computed Runoff for the Storm of July 26, 1960 - Oakdale Avenue Catchment - EPA Stormwater Management Model

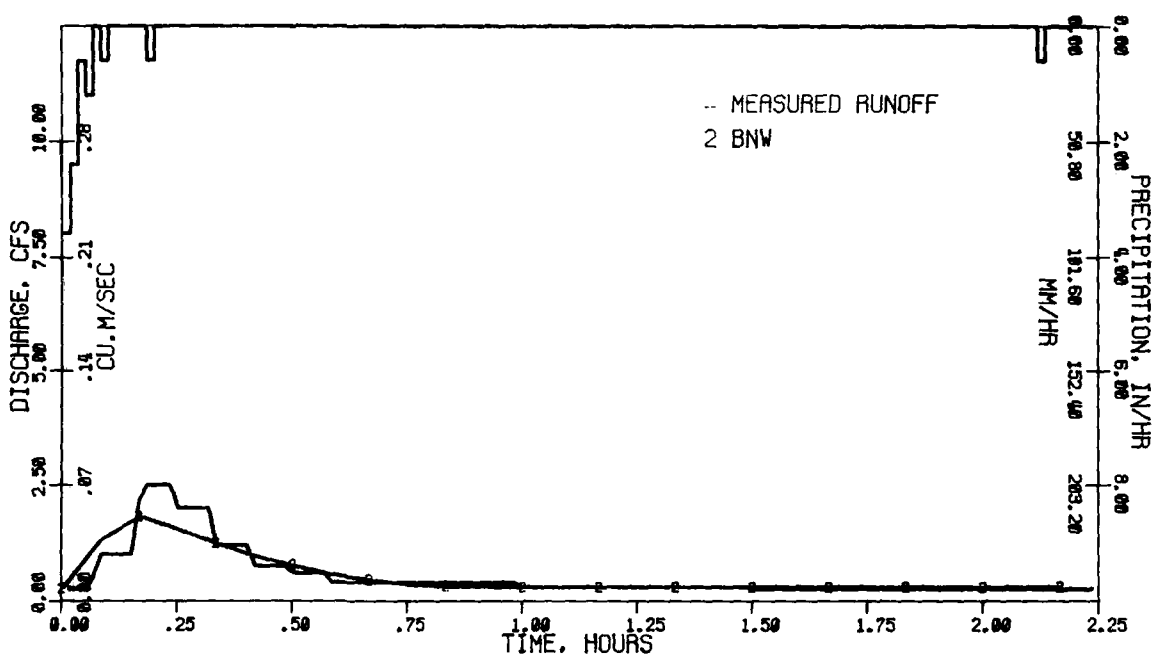
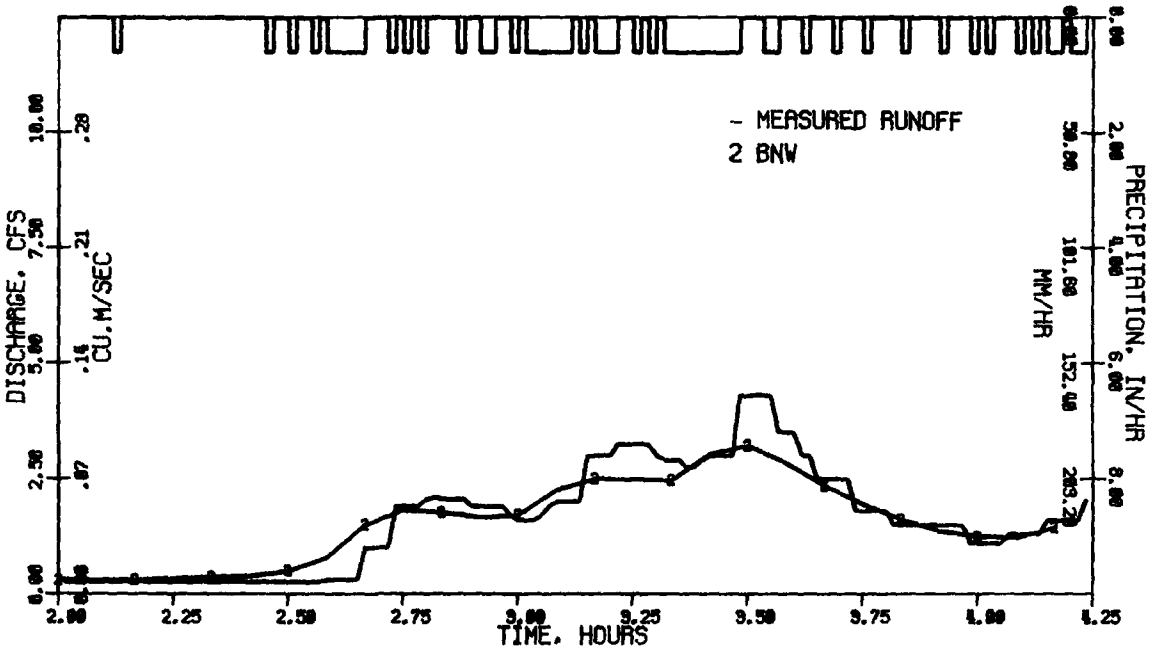


Figure 76a. Comparison of Measured and Computed Runoff for the Storm of July 26, 1960 - Oakdale Avenue Catchment - Battelle Urban Wastewater Management Model



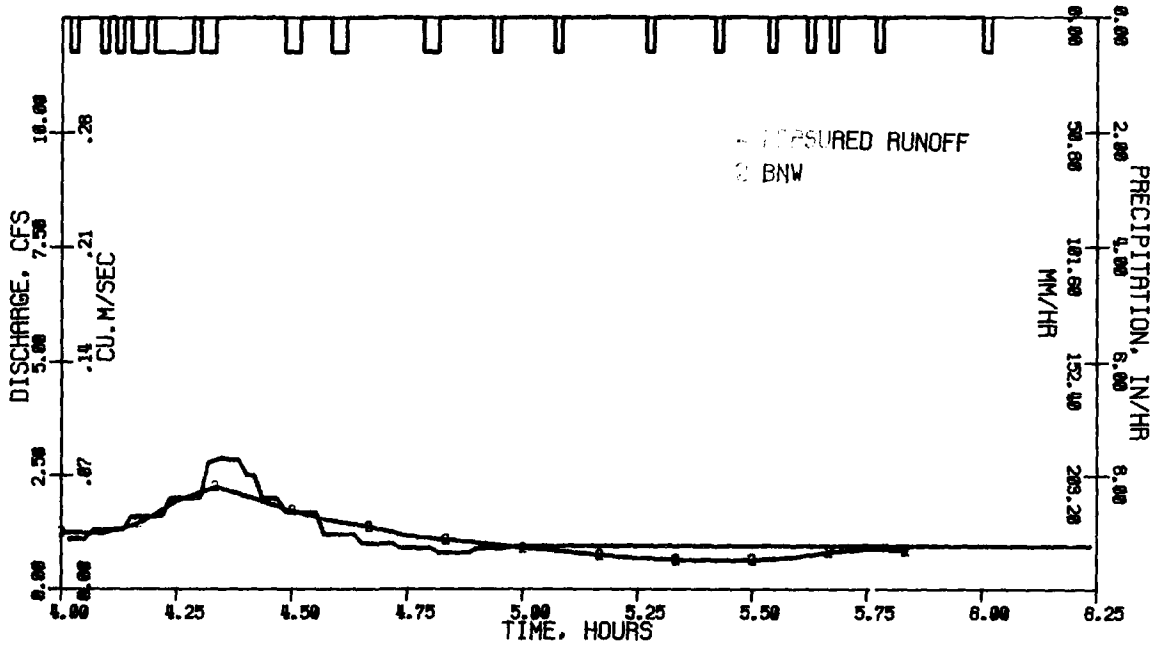
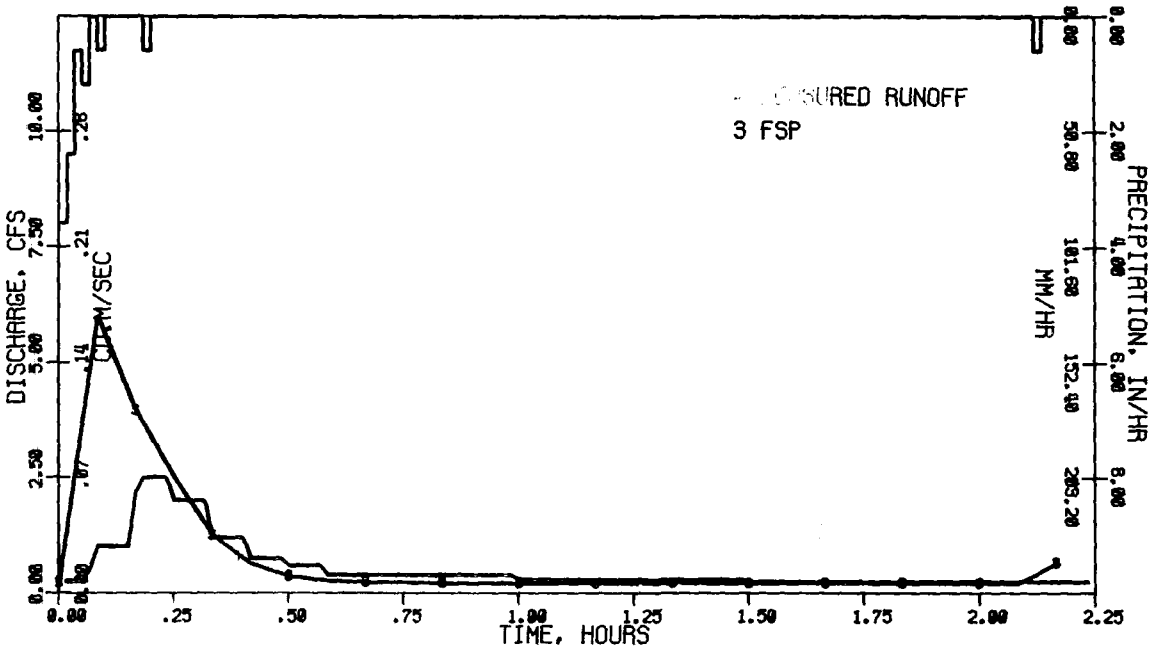


Figure 76b. Comparison of Measured and Computed Runoff for the Storm of July 26, 1960 - Oakdale Avenue Catchment - Battelle Urban Wastewater Management Model



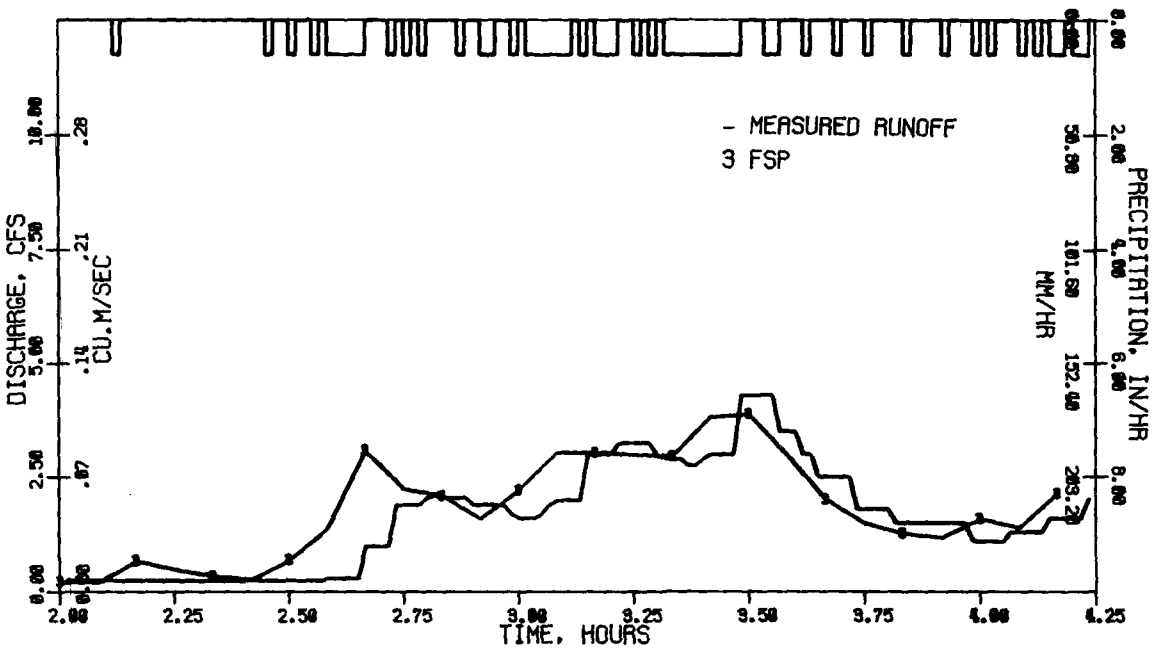


Figure 77a. Comparison of Measured and Computed Runoff for the Storm of July 26, 1960 - Oakdale Avenue Catchment - Chicago Flow Simulation Program

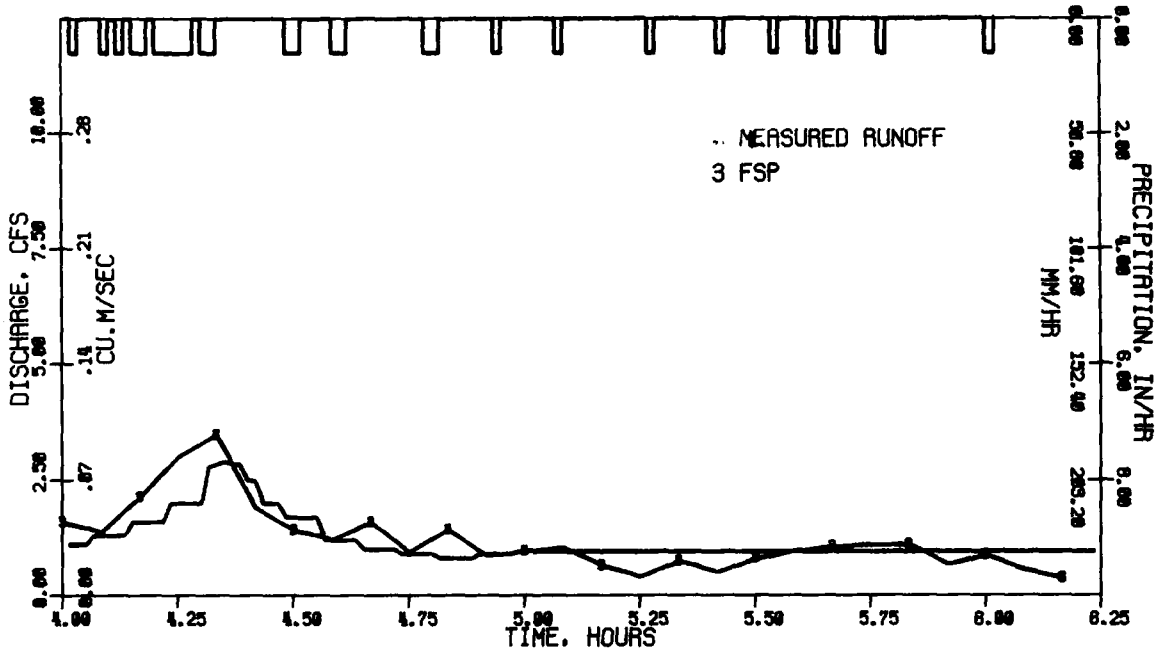


Figure 77b. Comparison of Measured and Computed Runoff for the Storm of July 26, 1960 - Oakdale Avenue Catchment - Chicago Flow Simulation Program

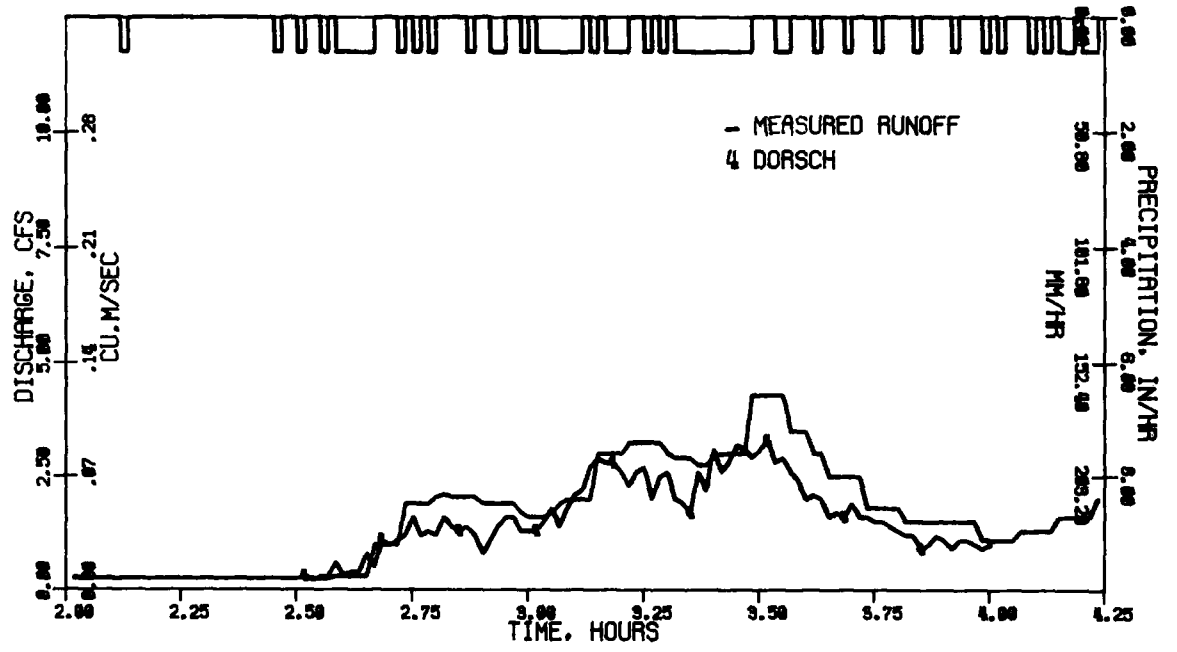


Figure 78a. Comparison of Measured and Computed Runoff for the Storm of July 26, 1960 - Oakdale Avenue Catchment - Dorsch Hydrograph Method

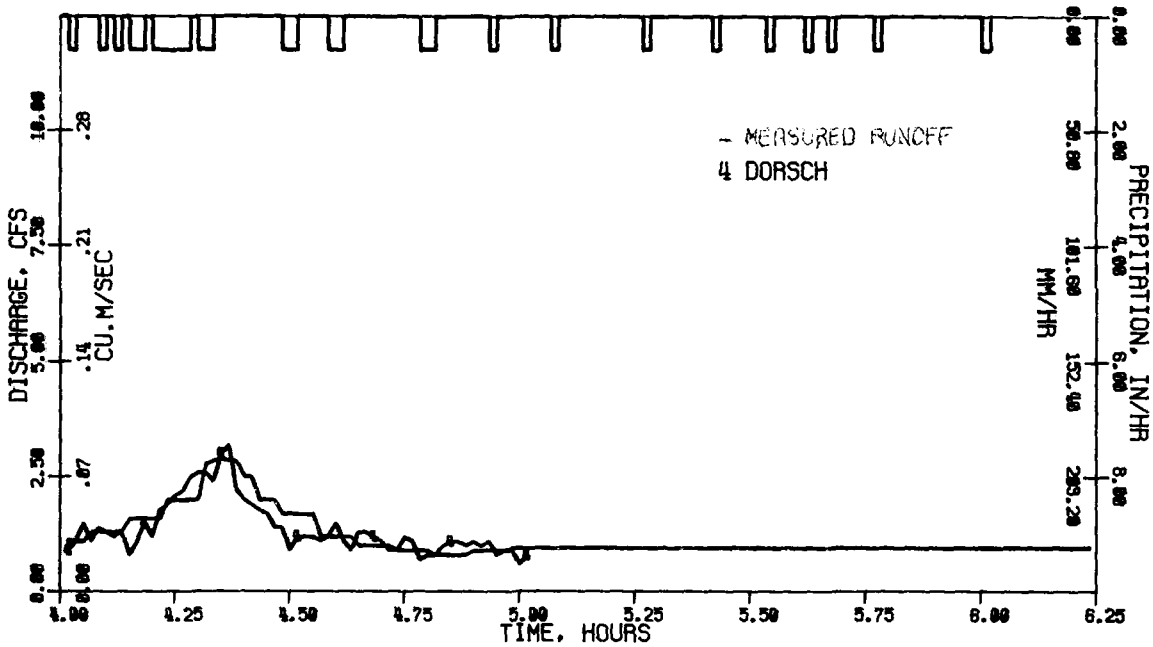


Figure 78b. Comparison of Measured and Computed Runoff for the Storm of July 26, 1960 - Oakdale Avenue Catchment - Dorsch Hydrograph Method

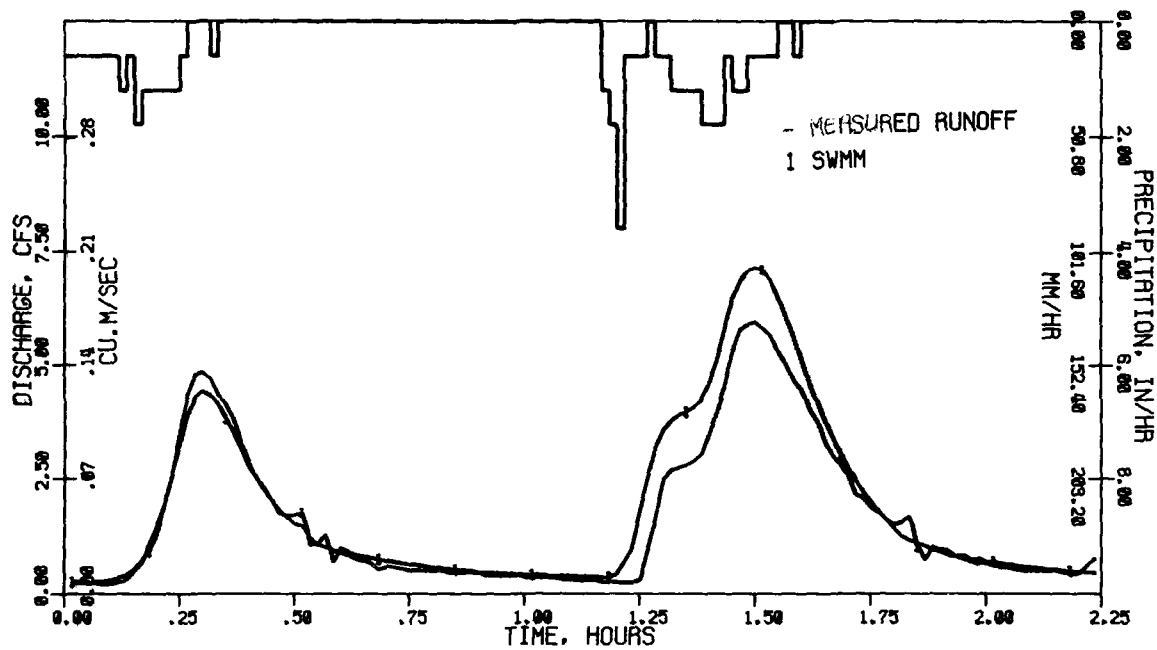


Figure 79. Comparison of Measured and Computed Runoff for the Storm of August 2, 1963 - Oakdale Avenue Catchment - EPA Stormwater Management Model

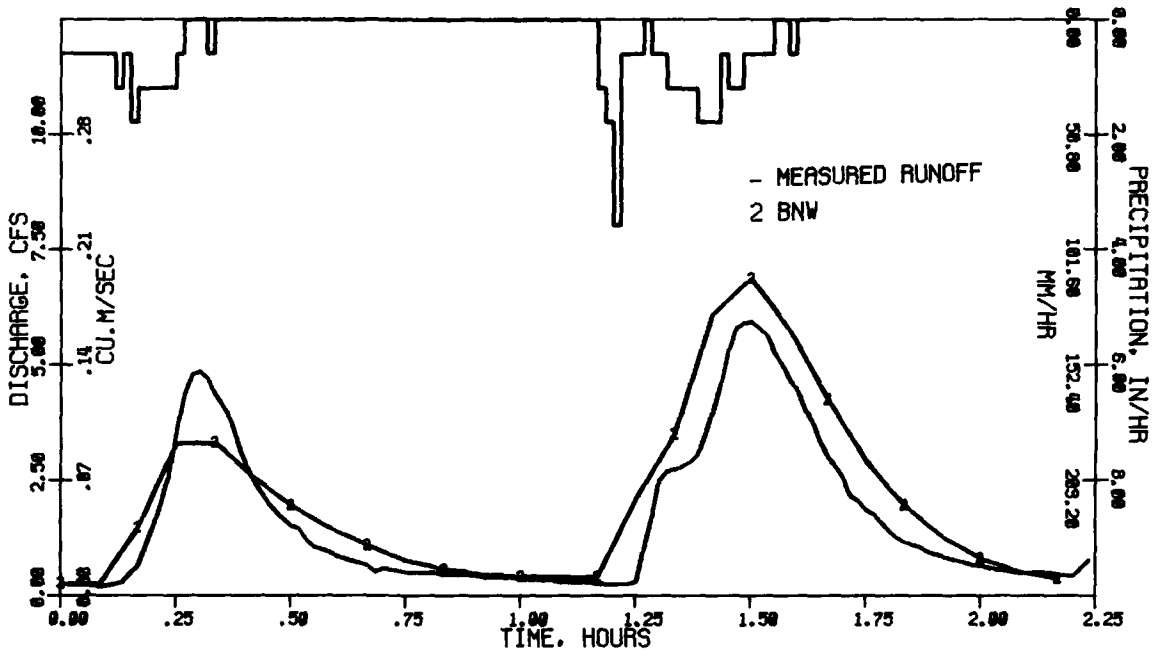


Figure 80. Comparison of Measured and Computed Runoff for the Storm of August 2, 1963 - Oakdale Avenue Catchment - Battelle Urban Wastewater Management Model

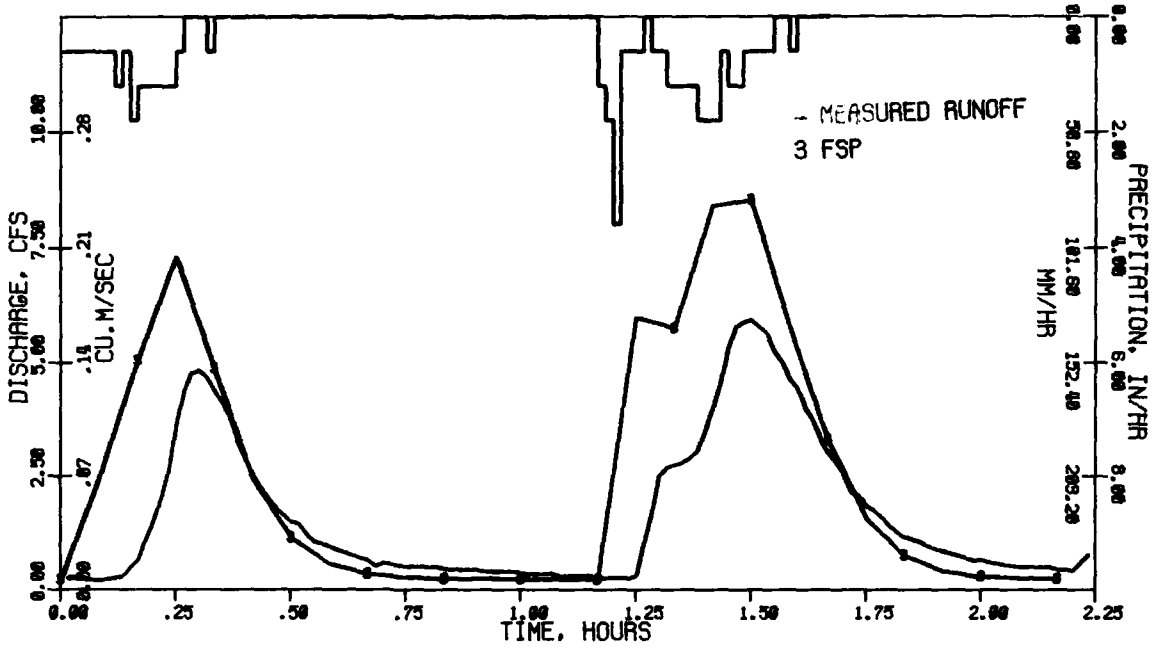


Figure 81. Comparison of Measured and Computed Runoff for the Storm of August 2, 1963 - Oakdale Avenue Catchment - Chicago Flow Simulation Program

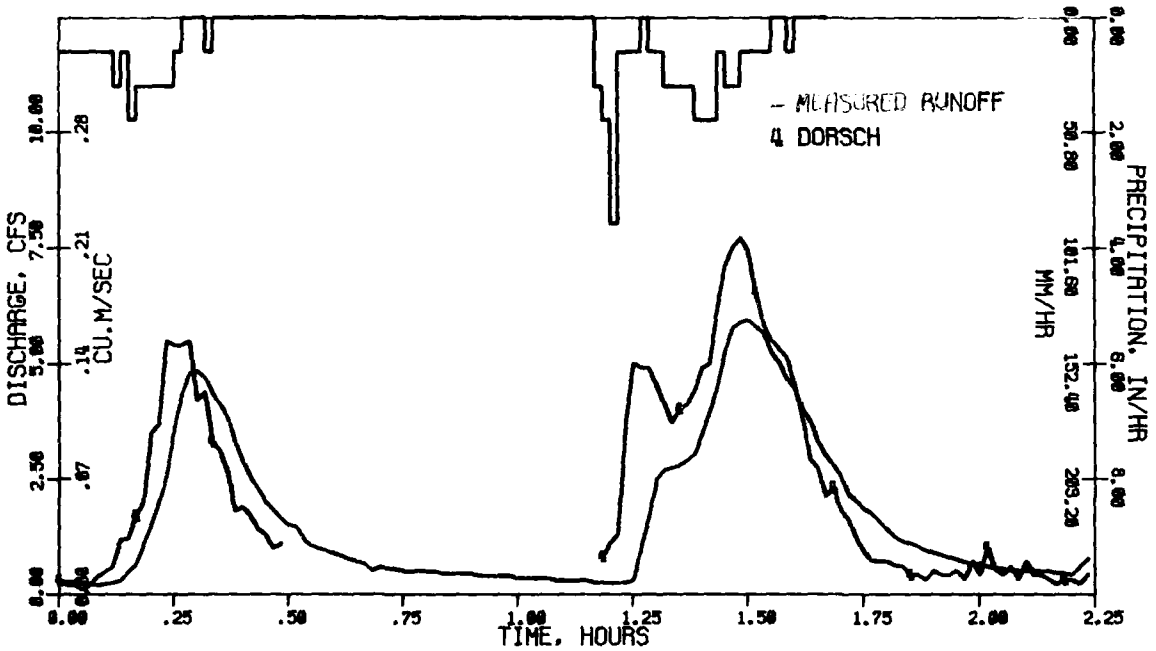


Figure 82. Comparison of Measured and Computed Runoff for the Storm of August 2, 1963 - Oakdale Avenue Catchment - Dorsch Hydrograph Volume Method

approximations in the computation of catchment moisture conditions and uncertainties with respect to initial catchment moisture conditions, particularly for long intermittent storm periods.

Among these four models, the DORSCH model would be most applicable for simulating backwater, downstream flow control, surcharging and pressure flow. The SWMM or BNW model could be the choice if these phenomena were not important. The BNW model has considerable flexibility in that it can model only selected phenomena, if desired, and can suppress flow routing, water quality computations, real-time control optimization, and design optimization as specified by the user. The BNW model, however, does not simulate water quality from land use, unit treatment processes, or receiving water flow and quality, and has not been tested as extensively as the SWMM.

The FSP is suited primarily for the continuous simulation of large nonsewered catchments. Its general applicability may be limited, however, by considerable simplification of the catchment hydrology and channel flow routing and by the need for modifications of model coefficients which are internal to the computer program.

Bloody Run Catchment Simulations

The Bloody Run catchment is located in the northwest section of Cincinnati, Ohio. It has a drainage area of 964 ha (2,380 acres) consisting of rolling terrain with commercial and industrial sections in the valleys and residential districts on the ridges. Approximately 55 percent of the area is residential, 22 percent open land and parks, and the rest commercial and industrial.

Simulations with Bloody Run catchment data were performed by Battelle with the SWMM, FSP, and BNW models. The same spatial discretization of catchment and sewer elements was used by all three models. Runoff was routed directly from the catchment into the sewers rather than separately through the gutters. The same empirical model coefficients were used by all three models for phenomena being modeled by the same equations. Comparable values were selected for the Horton equation (in SWMM) and Holtan equation (BNW model). For the FSP, related values are fixed internally by the program.

For dry initial catchment moisture conditions, the maximum infiltration, the minimum infiltration, and the decay rate of infiltration were assumed to be 63.5 mm/hr (2.50 in./hr), 11.4 mm/hr (0.45 in./hr), and 0.00115 sec^{-1}, respectively. Retention storage depths were assumed to be 4.67 mm (0.184 in.)

for pervious and 1.57 mm (0.062 in.) for impervious areas. For wet initial moisture conditions, a constant infiltration rate of 0.3 mm/hr (0.01 in./hr) was assumed and the available depth of retention storage was set at 0.3 mm (0.01 in.) for both pervious and impervious areas. These values were selected on the basis of previous runs with the SWMM performed by the University of Cincinnati and preliminary runs performed by Battelle.

Default values were used for the water quality model coefficients since the available water quality measurements were inadequate to calibrate the SWMM and BNW models. The FSP does not simulate water quality.

Preliminary runs indicated that the Bloody Run data may not be sufficiently complete and accurate to yield reliable tests of the models. Information on initial moisture conditions, spatial variations in infiltration characteristics in the catchment, and accuracy of the rainfall, runoff, and water quality measurements were unavailable. The sewer discharge was computed from pressure measurements using Manning's equation without establishing stage-discharge relationships or verifying the assumed Manning's roughness coefficient with simultaneous stage and velocity measurements during different flow periods. The four upstream sewer sampling stations were located very short distances upstream of major sewer junctions; the measured stage in one sewer could therefore be affected by backwater caused by the other branch, making the assumption of uniform flow conditions for the discharge computation incorrect for these conditions. Water quality measurements were made only during short periods during the storms and the runoff and water quality were tabulated only for those periods. The values of the remaining portions of the runoff hydrographs had to be obtained from plotted hydrographs, since the original tables were not available.

For these reasons, only the storms of November 9, 1970, and August 25, 1971, were simulated. The SWMM and BNW model simulations were run with both dry and wet initial catchment moisture conditions to demonstrate their effects on computed runoff. The FSP was run only once for each storm, since it sets initial catchment moisture conditions internally. The model results are summarized in Table 73, which lists the magnitudes and times of the peak discharges for each storm at the five Bloody Run gaging stations. The water quality data and simulations were not considered sufficiently reliable to warrant tabular comparisons.

Figures 83 to 111 show comparisons between the measured runoff and the runoff computed by the SWMM, FSP, and BNW models.

Table 73a. RUNOFF COMPARISONS FOR BLOODY RUN STORMS

Date	Monitoring Station	Peak No.	Peak discharge, cfs						Time of peak discharge, minutes					
			Observed	SWMM		BNW		FSP	Observed	SWMM		BNW		FSP
				Dry	Wet	Dry	Wet			Dry	Wet	Dry	Wet	
11/09/70	Outlet	1	120.00	98.06	143.32	84.82	128.63	175.88	67.5	95	90	105	75	70
		2	*	75.11	115.54	64.62	64.62	103.17	*	145	135	170	170	105
	Longview #1	1	9.72	61.97	80.34	31.48	51.45	62.58	60	80	80	95	70	55
		2	*	47.88	71.79	21.96	21.96	48.47	*	125	125	165	165	85
	Longview #2	1	1.02	38.29	55.00	51.19	68.22	86.26	75	75	75	100	65	60
		2	*	23.57	34.74	45.94	45.94	66.32	*	130	130	155	155	85
	Bank #1	1	2.29	22.13	28.36	4.00	6.56	29.92	22.5	70	70	85	65	45
		2	*	15.45	24.33	2.56	2.56	19.23	*	125	125	160	160	85
	Bank #2	1	*	6.90	8.75	11.21	17.67	13.69	*	70	70	85	60	45
		2	*	5.07	7.97	7.35	7.35	7.60	*	125	125	160	160	85
8/25/71	Outlet	1	760.66	522.73	669.93	549.94	1039.29	761.99	68	85	85	80	75	85
		2	325.90	168.20	267.11	228.42	253.70	282.99	210	235	230	220	215	215
		3	690.49	376.96	513.62	611.28	611.28	613.99	315	335	335	330	330	330
		4	297.05	249.90	322.49	266.09	266.09	141.97	600	505	500	495	495	495
	Longview #1	1	*	413.61	519.42	328.58	577.00	569.46	*	75	75	75	75	70
		2	*	102.22	146.38	103.58	125.46	145.57	*	210	210	210	195	205
		3	*	293.79	385.31	386.47	386.47	403.09	*	325	325	325	325	320
		4	*	43.25	69.43	31.70	31.70	45.39	*	500	500	540	540	530
	Longview #2	1	150.39	140.42	174.85	250.77	402.50	556.87	75	75	75	75	70	70
		2	44.54	100.38	133.16	127.67	130.68	170.21	195	225	225	210	210	205
		3	142.14	98.98	133.32	258.35	258.35	380.77	323	325	325	320	320	320
		4	*	220.18	282.94	262.45	262.45	159.18	*	490	490	490	490	490
	Bank #1	1	18.70	169.94	204.14	40.73	80.62	281.42	30	75	75	75	70	70
		2	15.34	35.10	48.52	12.59	14.16	63.07	270	205	205	205	205	195
		3	*	107.32	143.58	53.76	53.76	176.13	*	325	325	320	320	320
		4	*	7.26	10.97	3.47	3.47	20.25	*	540	540	530	530	530
	Bank #2	1	8.99	50.57	59.92	130.13	229.04	117.41	90	70	70	70	70	70
		2	8.99	10.75	16.96	38.24	41.54	31.03	188	210	210	205	205	190
		3	8.99	35.55	46.93	150.45	150.45	70.06	330	320	320	320	320	320
		4	*	2.21	4.59	10.36	10.36	7.67	*	535	535	535	535	530

Note: * No observation

Table 73b. RUNOFF COMPARISONS FOR BLOODY RUN STORMS

Date	Monitoring Station	Peak No.	Peak discharge, m^3/sec Observed	SWMM Dry	SWMM Wet	BNW Dry	BNW Wet	FSP
11/09/70	Outlet	1	3.40	2.78	4.06	2.40	3.64	4.98
		2	*	2.13	3.27	1.83	1.83	2.92
	Longview #1	1	0.28	1.75	2.28	0.89	1.46	1.77
		2	*	1.36	2.03	0.62	0.62	1.37
	Longview #2	1	0.03	1.08	1.56	1.45	1.93	2.44
		2	*	0.67	0.98	1.30	1.30	1.88
	Bank #1	1	0.06	0.63	0.80	0.11	0.19	0.85
		2	*	0.44	0.69	0.07	0.07	0.54
	Bank #2	1	*	0.20	0.25	0.32	0.50	0.39
		2	*	0.14	0.23	0.21	0.21	0.22
8/25/71	Outlet	1	21.54	14.80	18.97	15.57	29.43	21.58
		2	9.23	4.76	7.56	6.47	7.18	8.01
		3	19.55	10.68	14.55	17.31	17.31	17.39
		4	8.41	7.08	9.13	7.54	7.54	4.02
	Longview #1	1	*	11.71	14.71	9.31	16.34	16.13
		2	*	2.89	4.15	2.93	3.55	4.12
		3	*	8.32	10.91	10.94	10.94	11.42
		4	*	1.22	1.97	0.90	0.90	1.29
	Longview #2	1	4.26	3.98	4.95	7.10	11.40	15.77
		2	1.26	2.84	3.77	3.62	3.70	4.82
		3	4.03	2.80	3.78	7.32	7.32	10.78
		4	*	6.24	8.01	7.43	7.43	4.51
	Bank #1	1	0.53	4.81	5.78	1.15	2.28	7.97
		2	0.43	0.99	1.37	0.36	0.40	1.79
		3	*	3.04	4.07	1.52	1.52	4.99
		4	*	0.21	0.31	0.10	0.10	0.57
	Bank #2	1	0.25	1.43	1.70	3.69	6.49	3.33
		2	0.25	0.30	0.48	1.08	1.18	0.88
		3	0.25	1.01	1.33	4.26	4.26	1.98
		4	*	0.06	0.13	0.29	0.29	0.22

Note: * No observation

Measured and computed biochemical oxygen demand and suspended solids concentrations for the SWMM and BNW models are compared in Figures 112 to 122. Plots of computed runoff hydrographs are shown for all five gaging stations, so that all models could be compared regardless of whether measurements existed at each station or not. Plots of the computed water quality are shown for the Outlet station and only those upstream stations which had measured water quality data. This was considered sufficient to indicate the difficulty of simulating wastewater quality with the SWMM and BNW models.

The measured rainfall data were reported at 5-minute intervals and the measured runoff and water quality data at 7.5-minute intervals. Five-minute time steps were selected for all model simulations.

All three models predict considerably higher runoff than measured for the storm of November 9, 1970, at the Bank #2, Longview #1, and Longview #2 stations, and the differences between models are significant (Table 73). For the Outlet station, however, the measured runoff lies between the runoff predicted by the SWMM and the BNW models for the dry and wet initial catchment moisture condition, and the FSP overestimates the measured runoff by a much smaller percentage than for the upstream stations.

The sum of the Longview #1 and Longview #2 runoff should be almost as high as the Outlet runoff. As shown by Table 73 and Figures 91 to 97, the sum of the measured runoff at Longview #1 and #2 for the storm of November 9, 1970, is less than 10 percent of the runoff at the Outlet. It is highly improbable that the missing 90 percent of the runoff entered the sewers between these stations. The runoff predicted by the models, however, appears to be in the approximate ratio of the drainage areas of these stations.

It is interesting to note that no significant difference exists in the times of the peak runoff values computed by the SWMM for dry and wet initial conditions, although the magnitudes of the peaks differ considerably.

The BNW model predicts the first peak from 15 to 25 minutes later than the SWMM for the dry initial condition and from 5 to 10 minutes earlier for the wet initial condition. The BNW model predicts the same times for the second peaks regardless of initial moisture condition, but they are from 25 to 40 minutes later than the second peaks computed by the SWMM. The plots for this storm show also that the BNW model computes saturated catchment moisture conditions after only about 1 hour of storm runoff, while over 6 hours are required by the SWMM.

The FSP model predicts the correct time for the first peak at the Outlet, but the predicted value is considerably higher than the measured value. Both the FSP and BNW models predict a second peak, which is only slightly visible in the SWMM prediction and does not exist in the measured data. This seems to indicate that the catchment did not become saturated and that the second small rainfall peak did not have a large effect on the runoff hydrograph.

Inspection of Figures 95 and 96 indicates that a better fit could probably be obtained between measured and computed runoff by adjusting model coefficients. This would not provide better information with regard to model accuracy, however, considering the uncertainties in the measured data accuracy, particularly at the upstream stations.

Similar problems occur for the storm of August 25, 1971, which consists of several intermittent rainfall bursts and associated short runoff periods. Again, the measured runoff at the Bank #1 and #2 stations is considerably lower than the runoff computed by any model. The SWMM model compares more favorably with the Longview #2 measurements (Figure 107), but then underestimates the runoff at the Outlet (Figure 110) for both dry and wet initial conditions. The BNW model considerably overestimates the runoff at the Longview #2 station (Figure 108) but its values compare more favorably with the measured values at the Outlet station (Figure 111). The FSP model predicts the runoff at the Outlet quite well (Figure 112) although its values are considerably higher than the measured values at Longview #2 (Figure 109). The models agree surprisingly in predicting the times of the four major peaks of this storm (Table 73).

The models cannot predict the measured water quality for the selected Bloody Run storms (Figures 113 to 122). The SWMM and BNW model predictions disagree significantly with each other and with the measured values. The BNW model predictions for both suspended solids and biochemical oxygen demand are considerably higher than the SWMM predictions. The pollutographs are roughly mirror images (upside down) of the runoff hydrographs, although this is more apparent for the BNW model (due to its higher values) than for the SWMM. The measured concentrations, where available, generally lie between the BNW and SWMM predictions, but their duration is too short to draw valid conclusions about model performance.

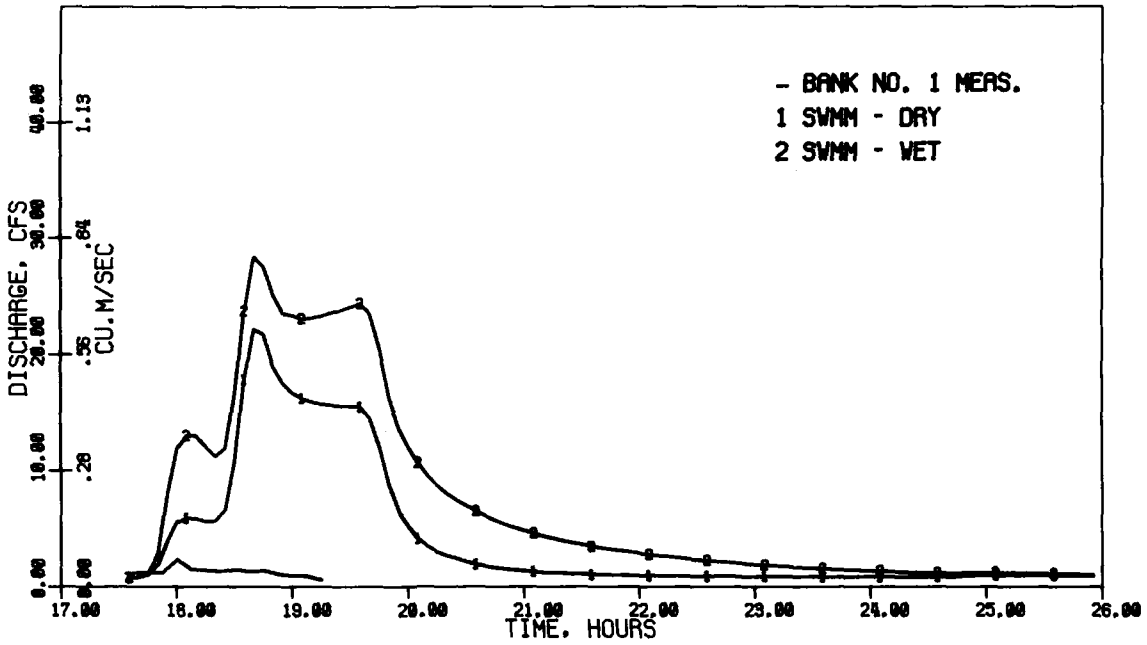


Figure 83. Comparison of Measured and Computed Runoff at the Bank #1 Station for the Storm of November 9, 1970 - Blood Run Catchment - EPA Stormwater Management Model

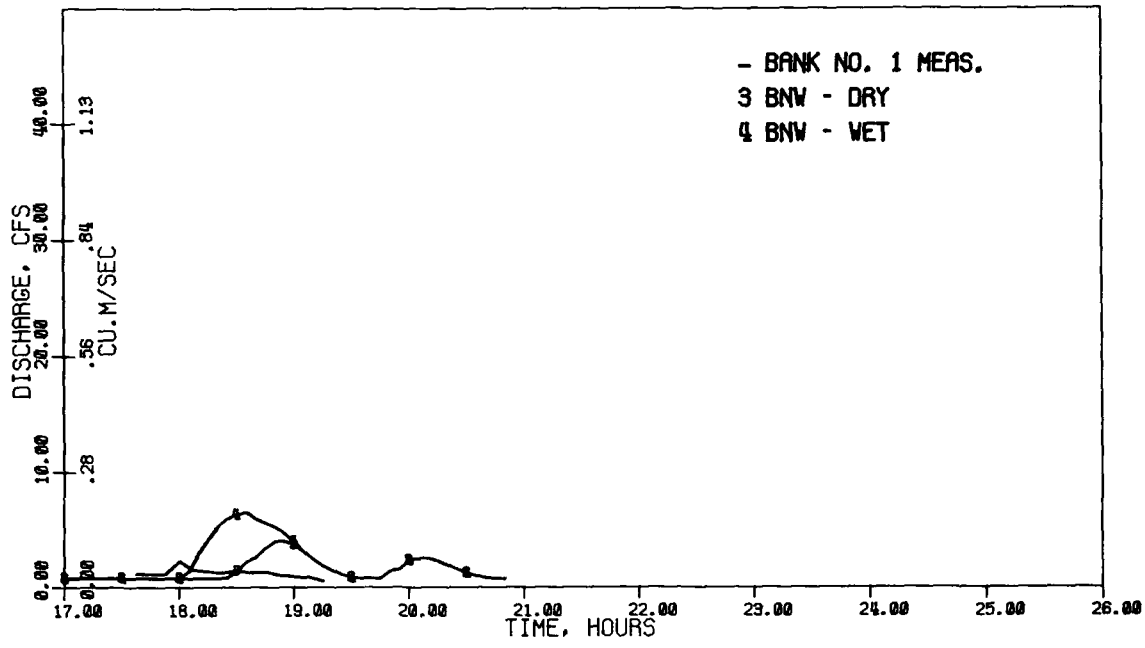


Figure 84. Comparison of Measured and Computed Runoff at the Bank #1 Station for the Storm of November 9, 1970 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

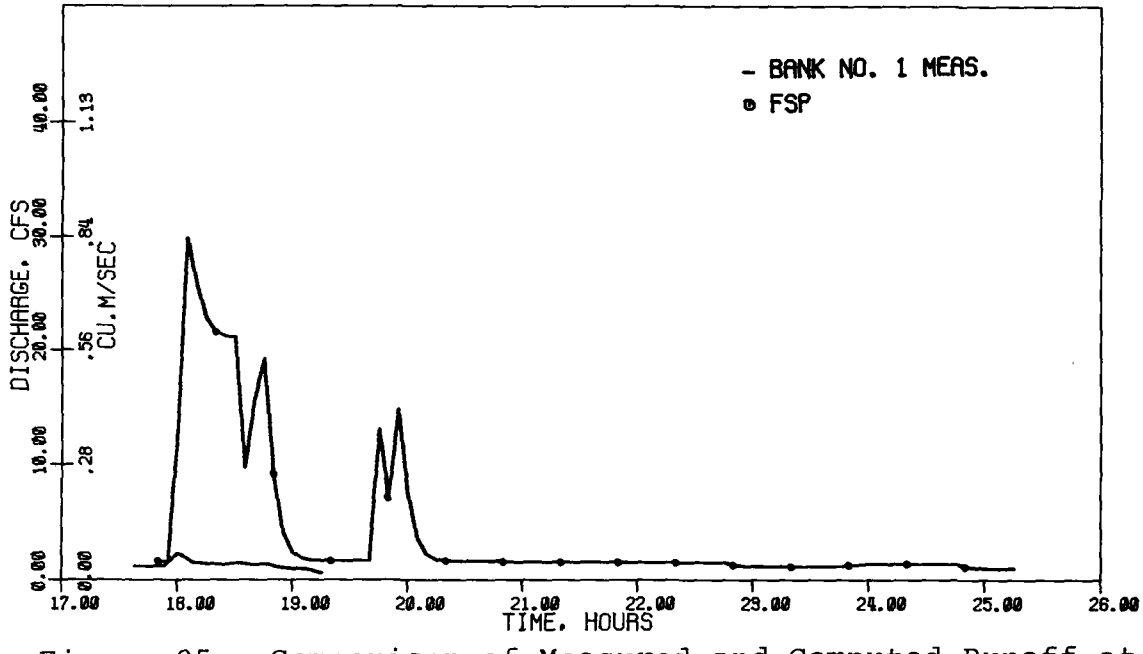


Figure 85. Comparison of Measured and Computed Runoff at the Bank #1 Station for the Storm of November 9, 1970 - Bloody Run Catchment - Chicago Flow Simulation Program

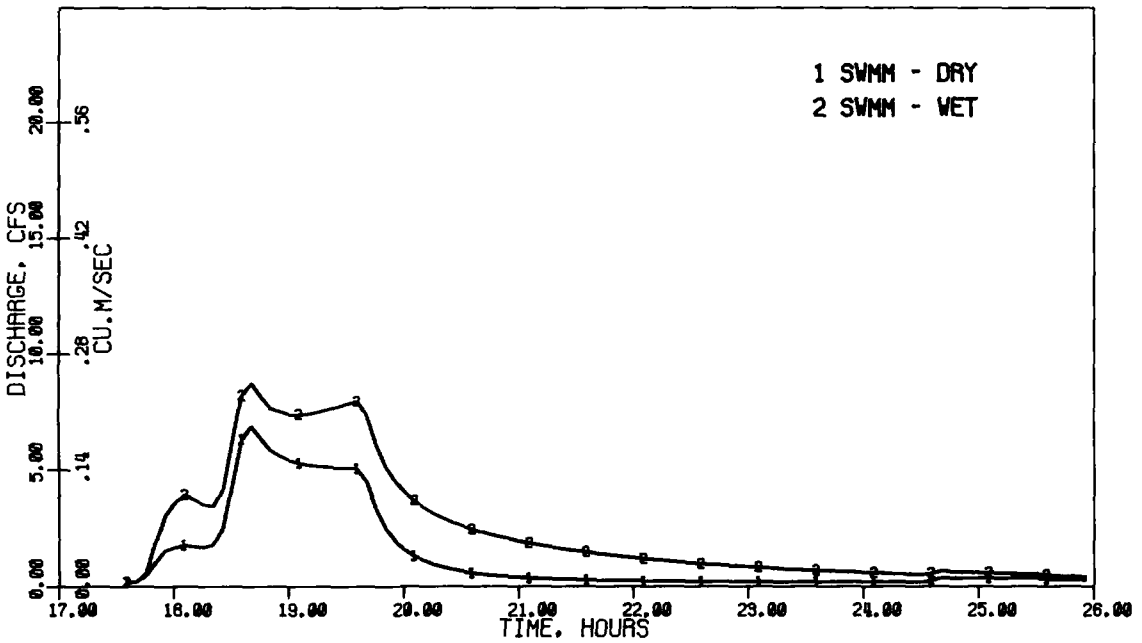


Figure 86. Computed Runoff at the Bank #2 Station for the Storm of November 9, 1970 - Bloody Run Catchment - EPA Stormwater Management Model

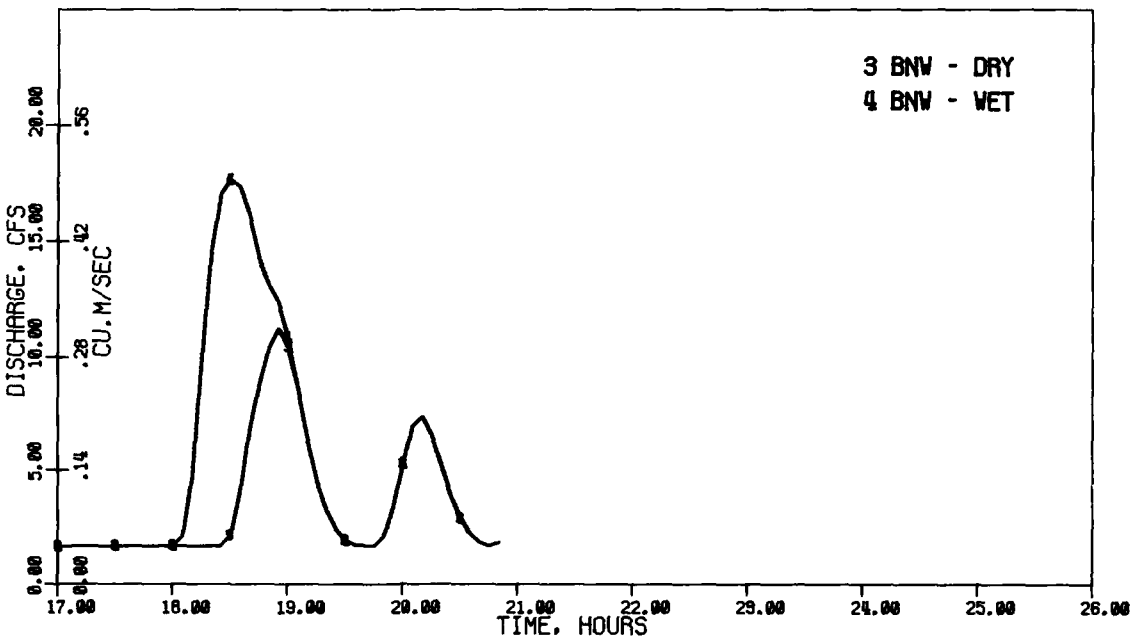


Figure 87. Computed Runoff at the Bank #2 Station for the Storm of November 9, 1970 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

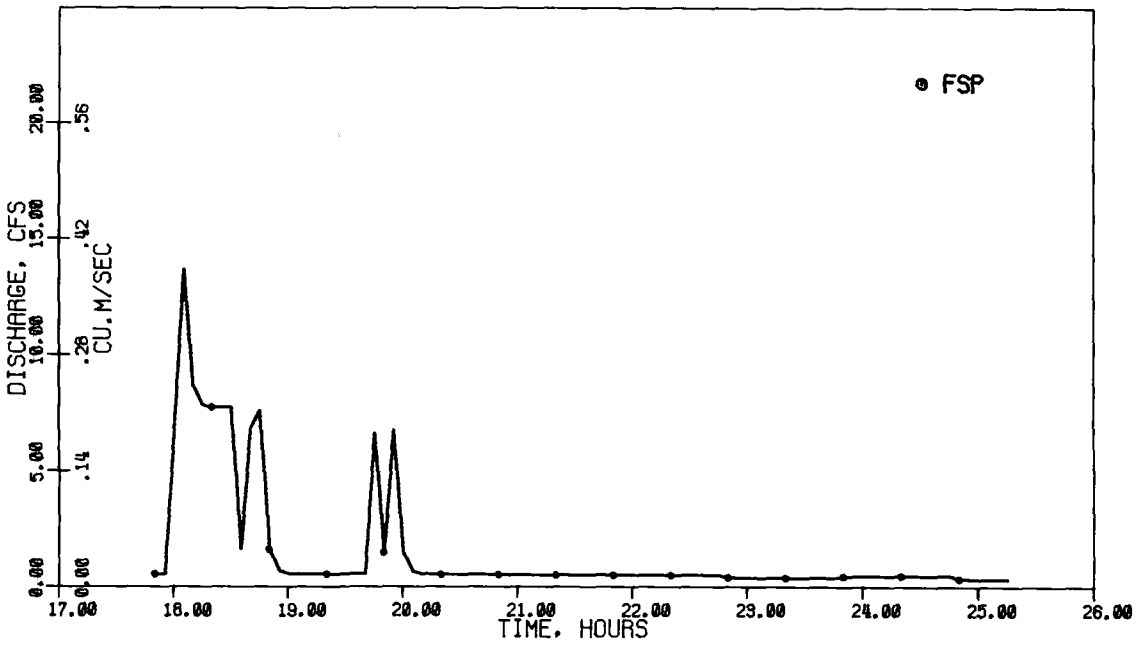


Figure 88. Computed Runoff at the Bank #2 Station for the Storm of November 9, 1970 - Bloody Run Catchment - Chicago Flow Simulation Program

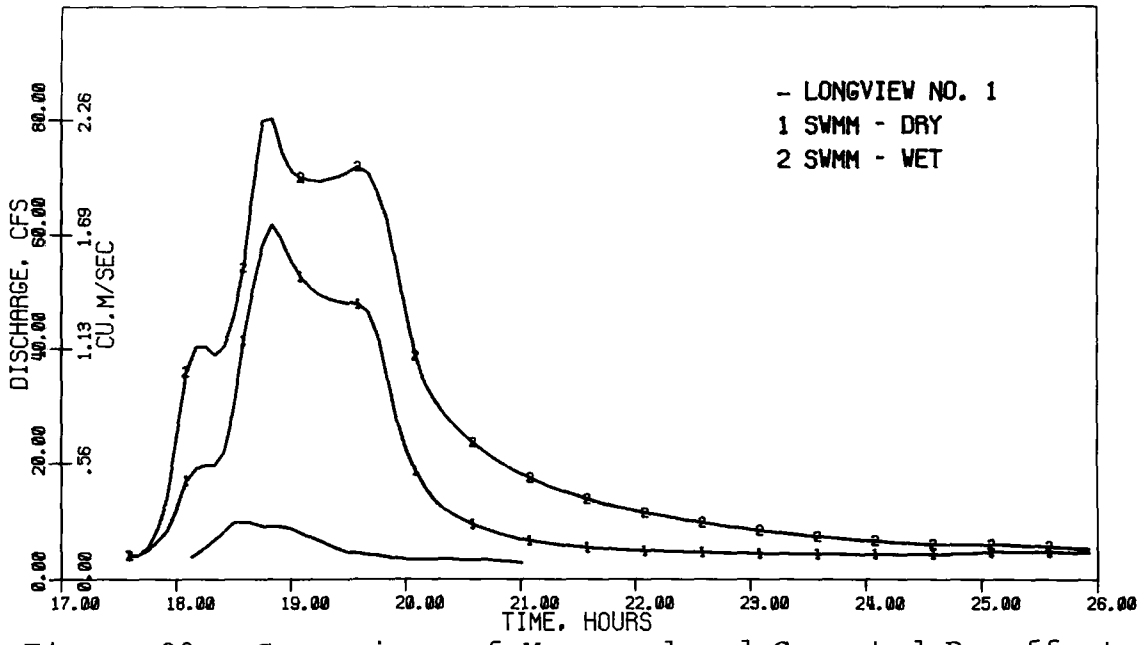


Figure 89. Comparison of Measured and Computed Runoff at the Longview #1 Station for the Storm of November 9, 1970 - Bloody Run Catchment - EPA Stormwater Management Model

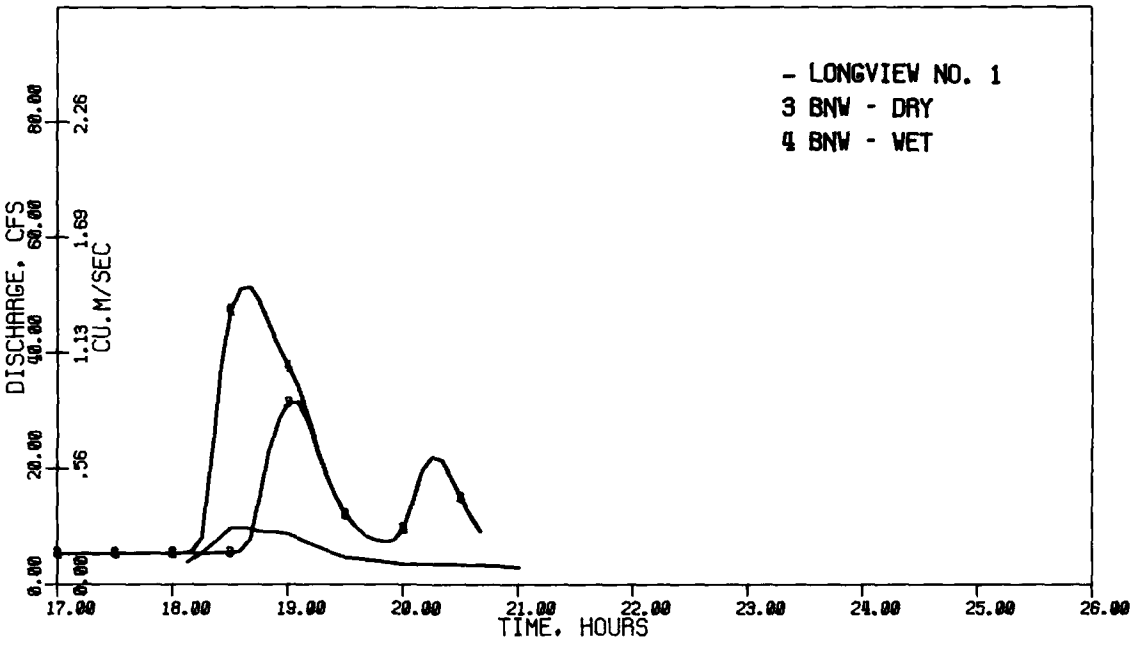


Figure 90. Comparison of Measured and Computed Runoff at the Longview #1 Station for the Storm of November 9, 1970 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

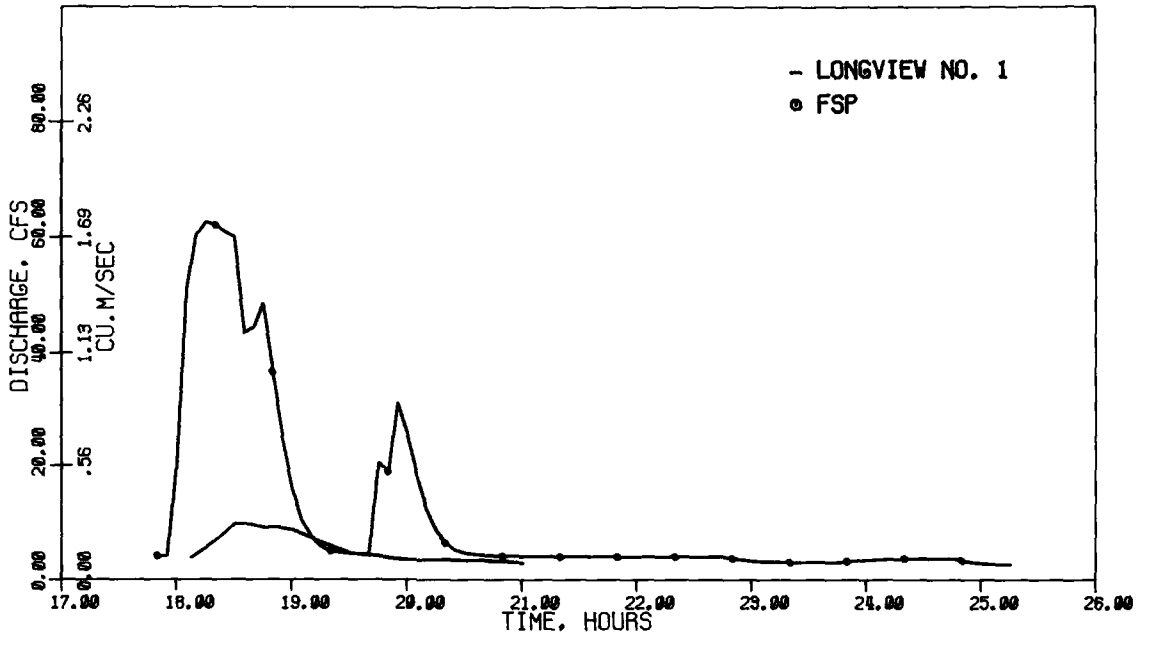


Figure 91. Comparison of Measured and Computed Runoff at the Longview #1 Station for the Storm of November 9, 1970 - Bloody Run Catchment - Chicago Flow Simulation Program

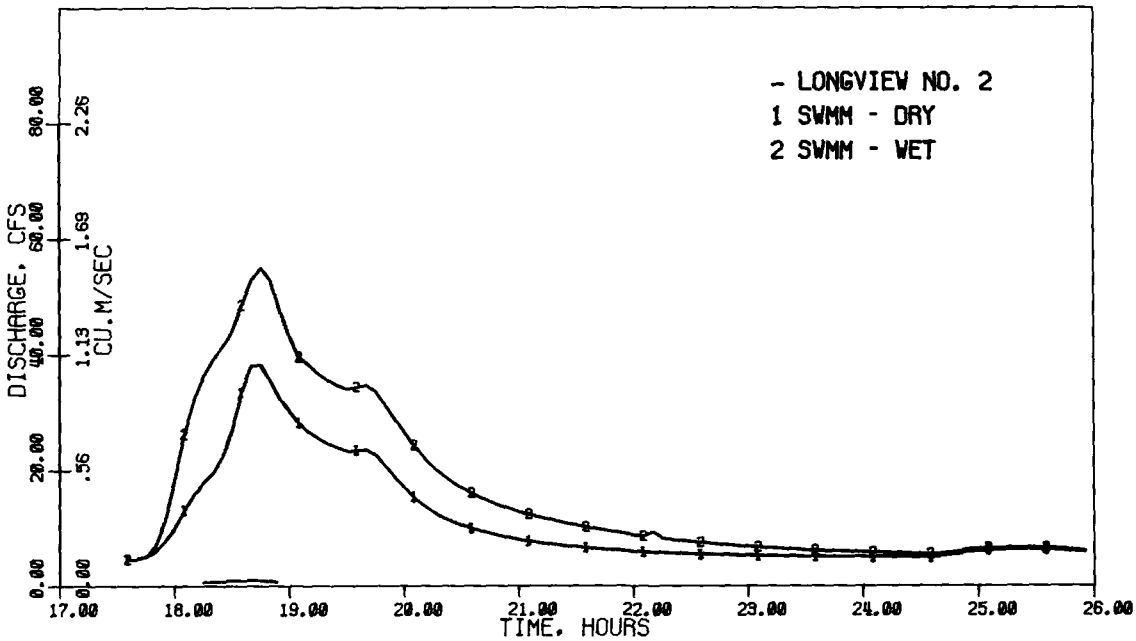


Figure 92. Comparison of Measured and Computed Runoff at the Longview #2 Station for the Storm of November 9, 1970 - Bloody Run Catchment - EPA Stormwater Management Model

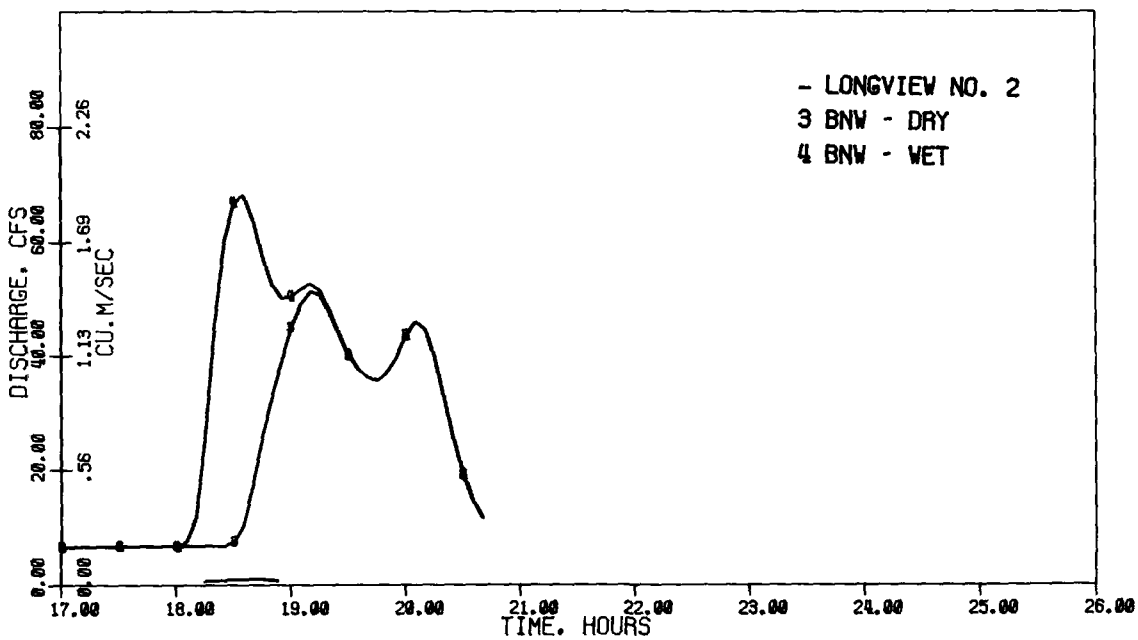


Figure 93. Comparison of Measured and Computed Runoff at the Longview #2 Station for the Storm of November 9, 1970 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

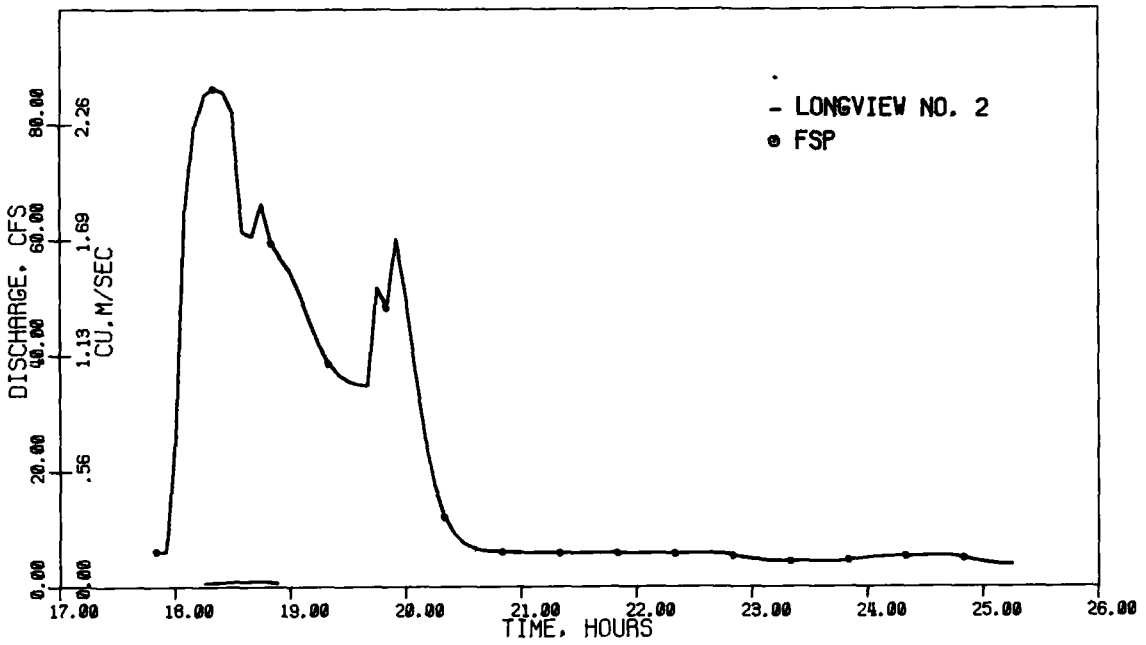


Figure 94. Comparison of Measured and Computed Runoff at the Longview #2 Station for the Storm of November 9, 1970 - Bloody Run Catchment - Chicago Flow Simulation Program

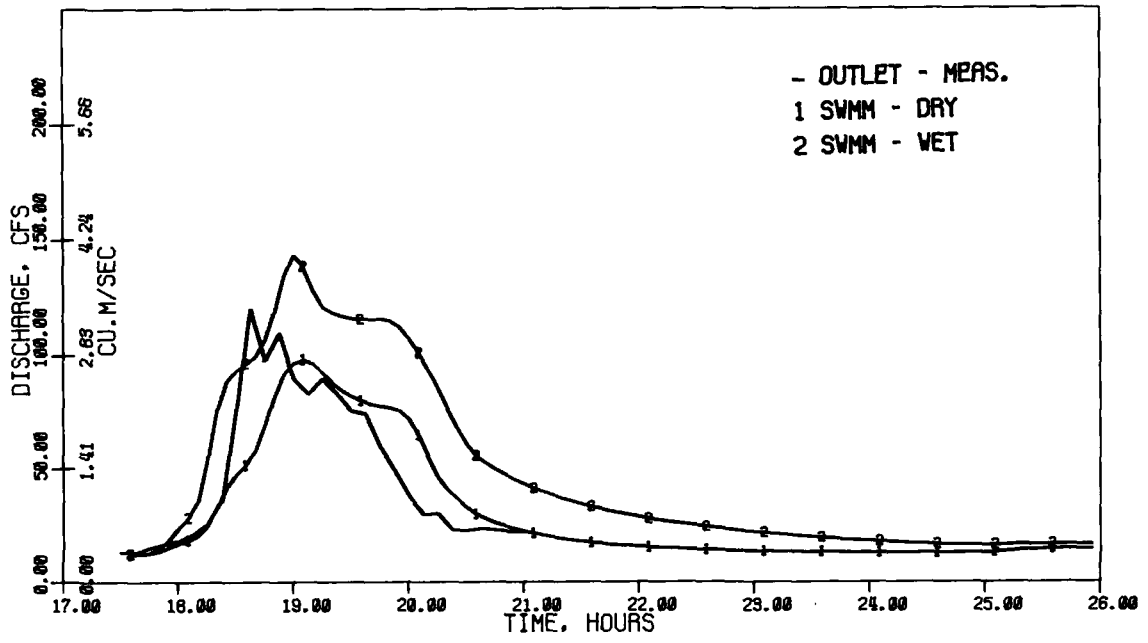


Figure 95. Comparison of Measured and Computed Runoff at the Outlet Station for the Storm of November 9, 1970 - Bloody Run Catchment - EPA Stormwater Management Model

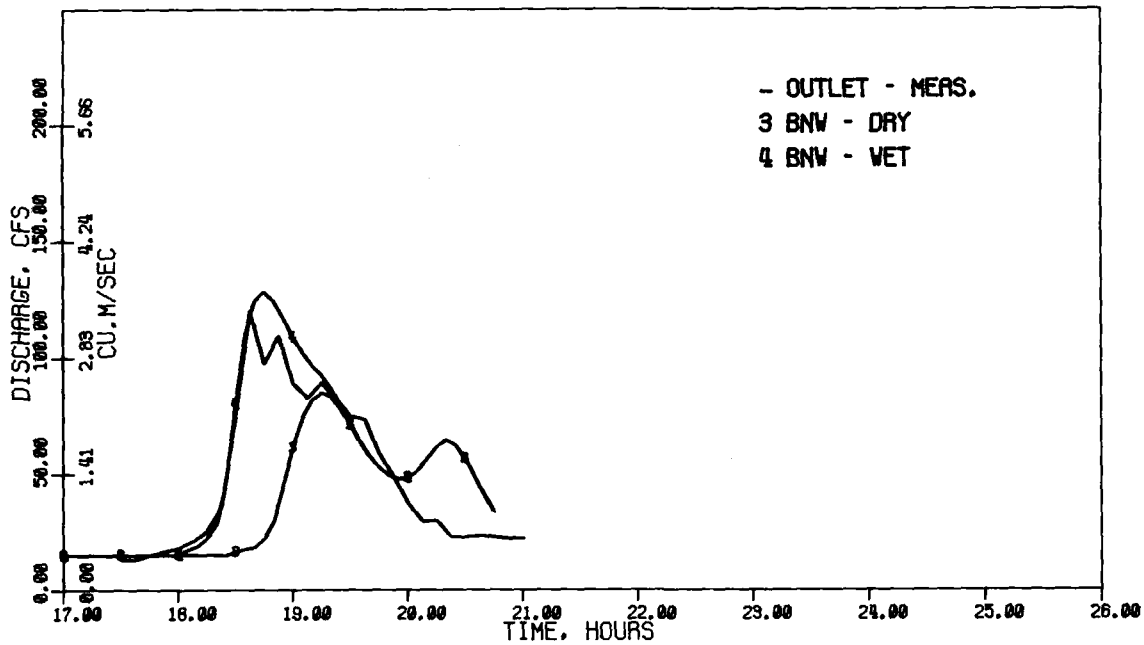


Figure 96. Comparison of Measured and Computed Runoff at the Outlet Station for the Storm of November 9, 1970 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

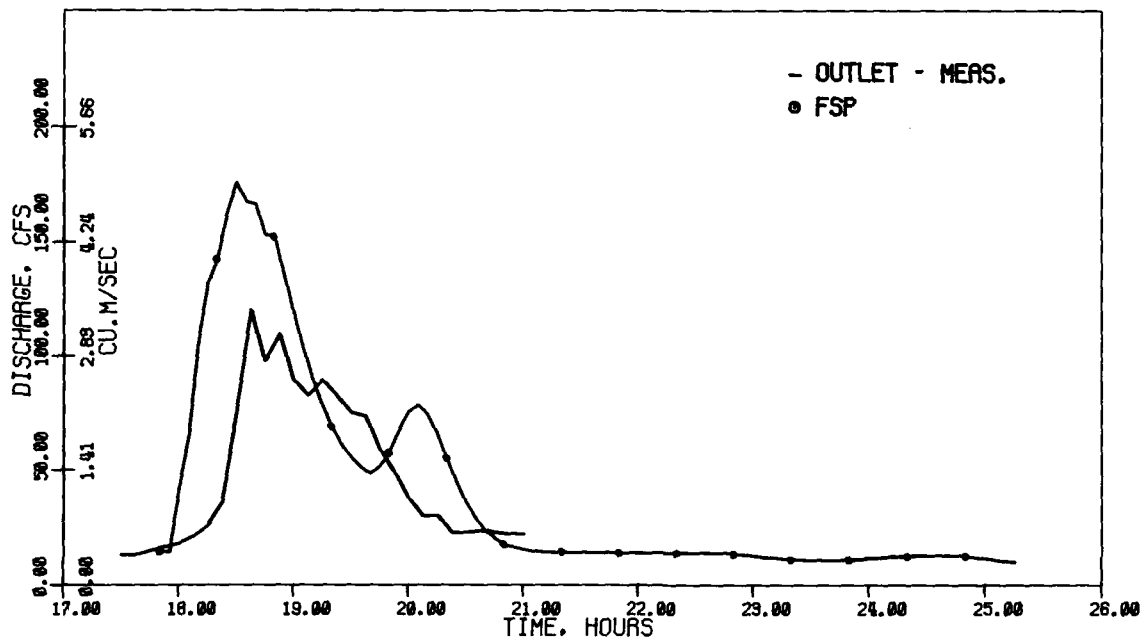


Figure 97. Comparison of Measured and Computed Runoff at the Outlet Station for the Storm of November 9, 1970 - Bloody Run Catchment - Chicago Flow Simulation Program

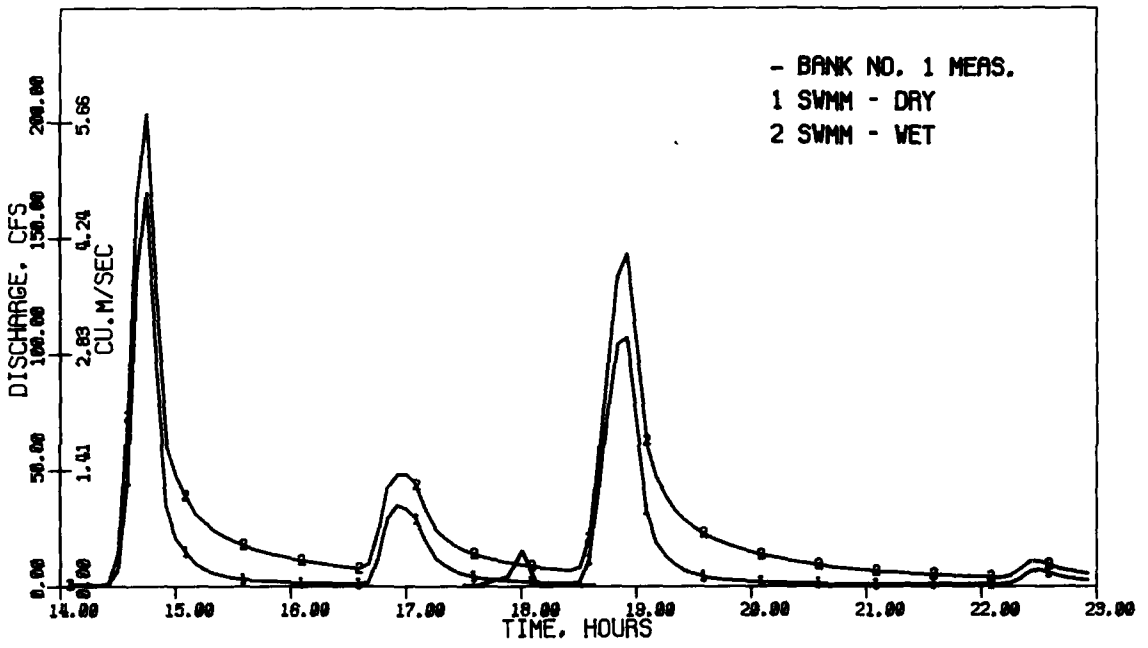


Figure 98. Comparison of Measured and Computed Runoff at the Bank #1 Station for the Storm of August 25, 1971 - Bloody Run Catchment - EPA Stormwater Management Model

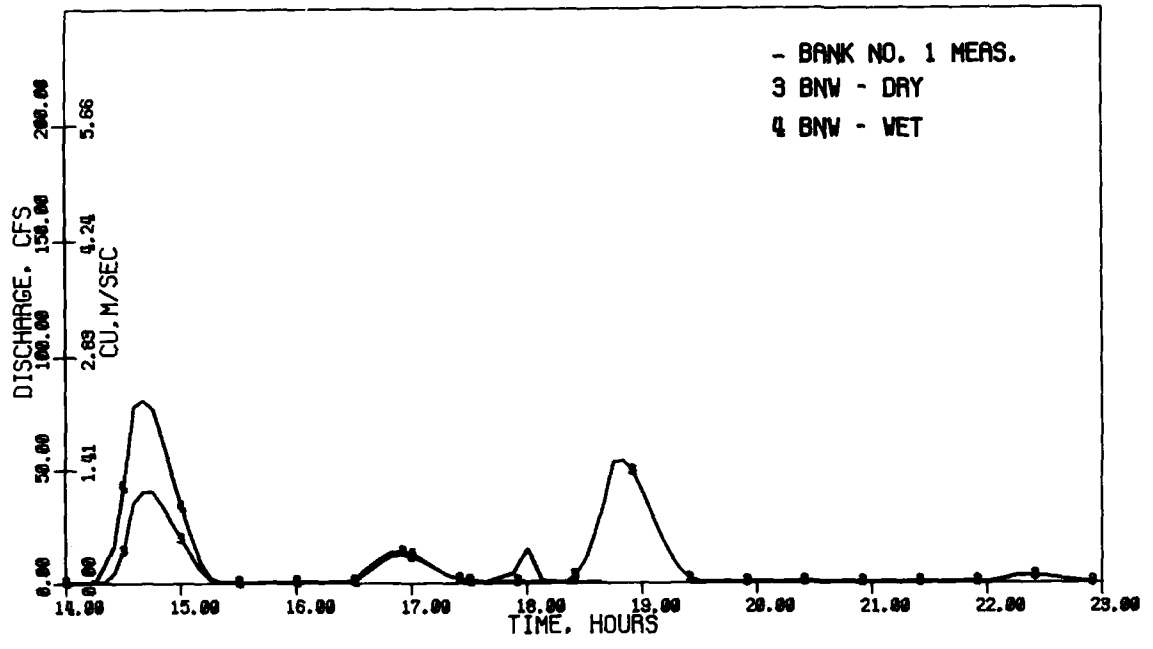


Figure 99. Comparison of Measured and Computed Runoff at the Bank #1 Station for the Storm of August 25, 1971 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

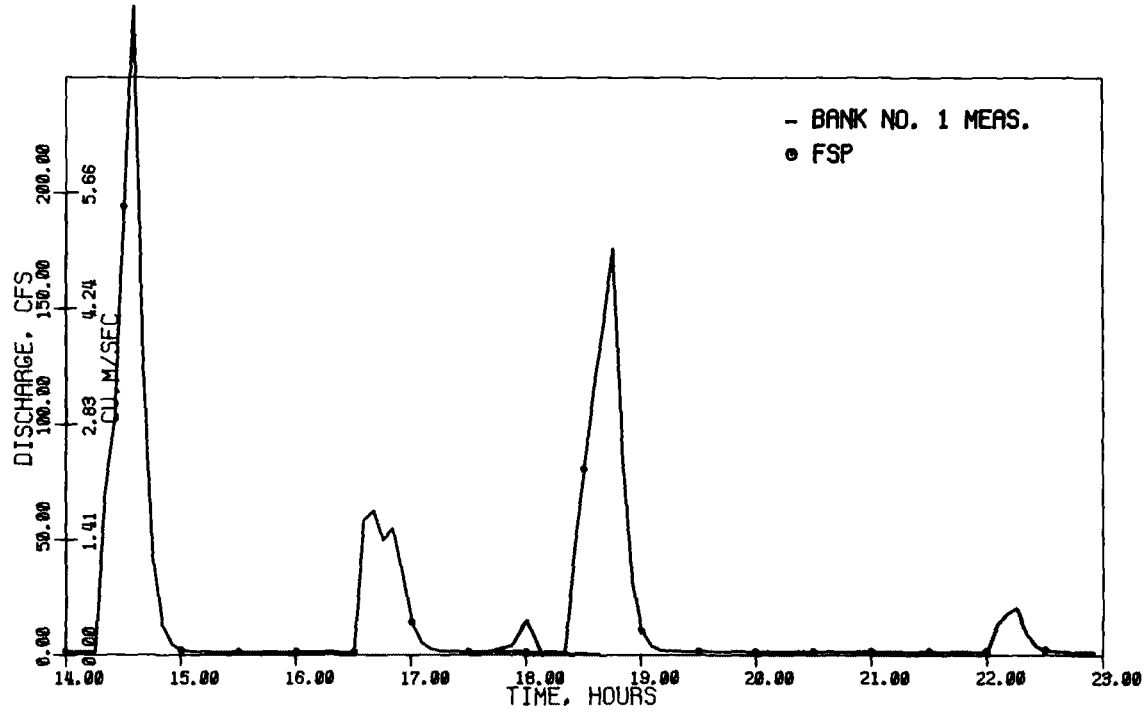


Figure 100. Comparison of Measured and Computed Runoff at the Bank #1 Station for the Storm of August 25, 1971 - Bloody Run Catchment - Chicago Flow Simulation Program

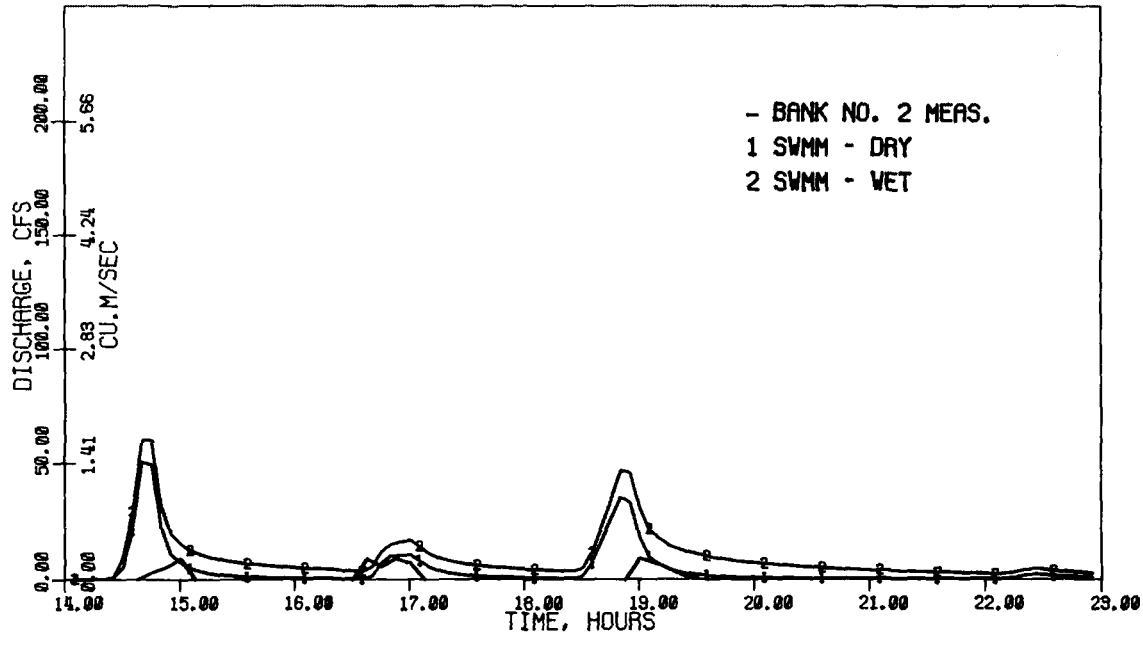


Figure 101. Comparison of Measured and Computed Runoff at the Bank #2 Station for the Storm of August 25, 1971 - Bloody Run Catchment - EPA Stormwater Management Model

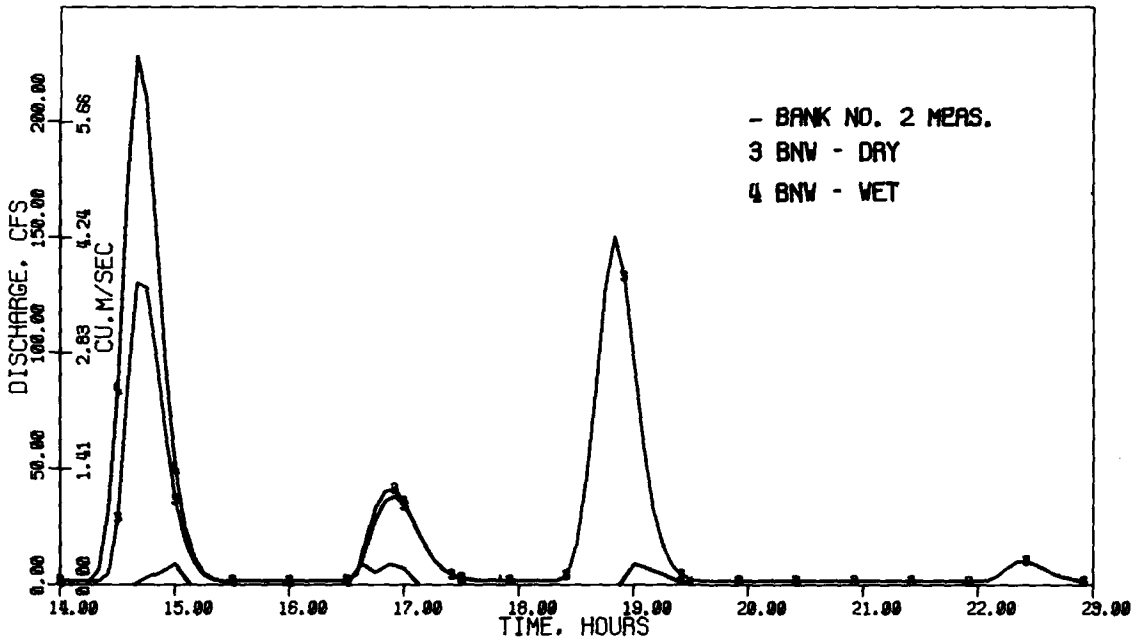


Figure 102. Comparison of Measured and Computed Runoff at the Bank #2 Station for the Storm of August 25, 1971 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

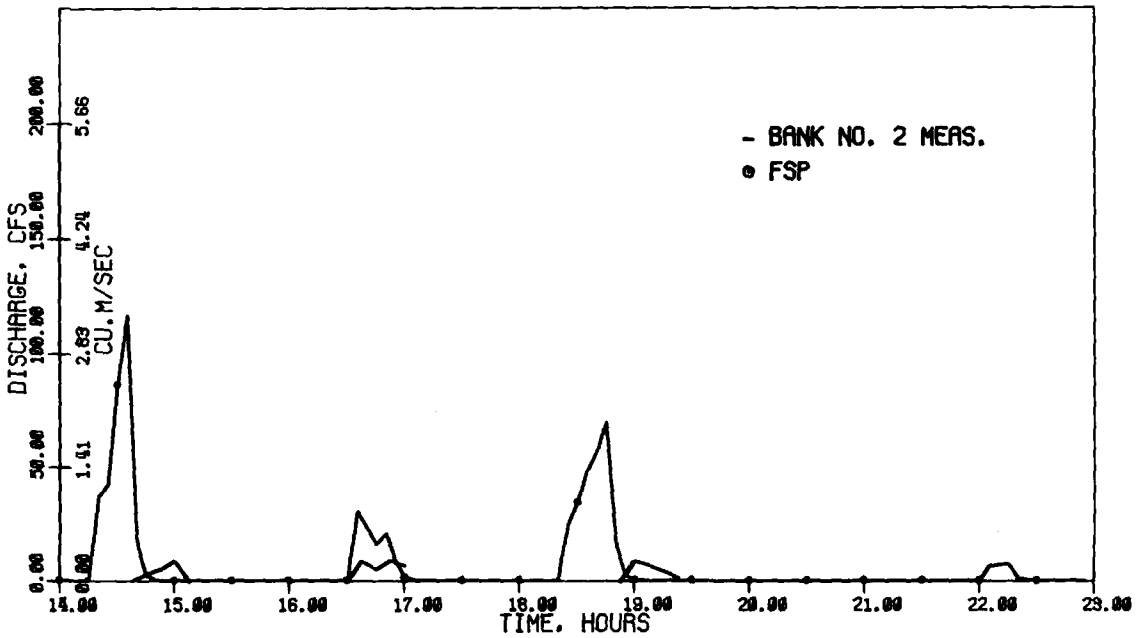


Figure 103. Comparison of Measured and Computed Runoff at the Bank #2 Station for the Storm of August 25, 1971 - Bloody Run Catchment - Chicago Flow Simulation Program

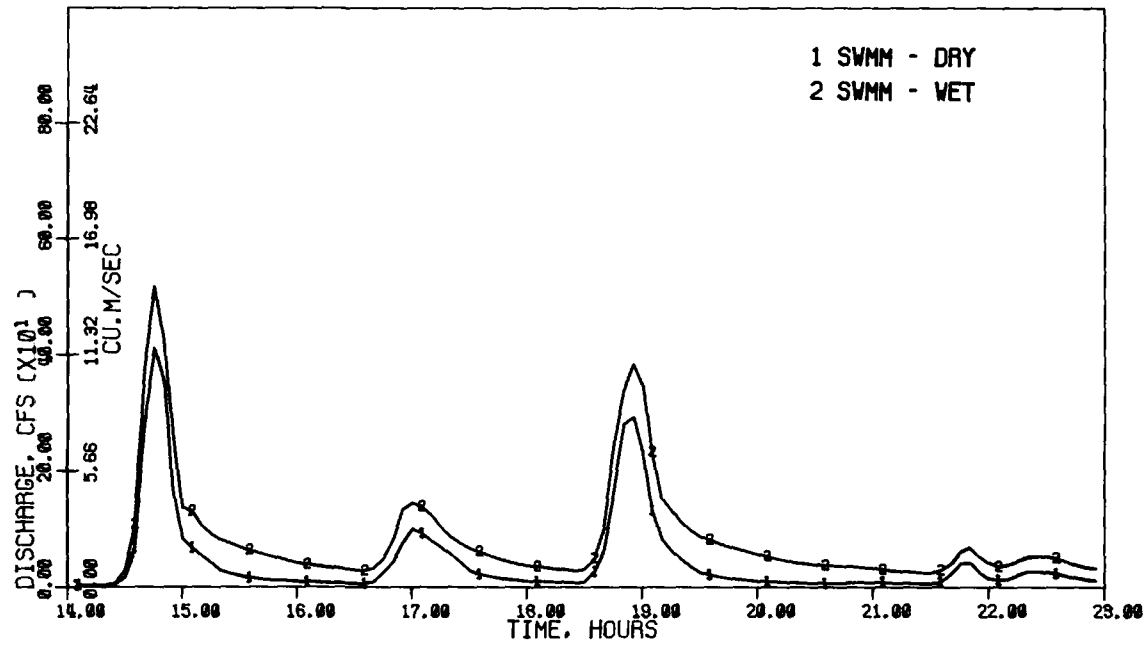


Figure 104. Computed Runoff at the Longview #1 Station for the Storm of August 25, 1971 - Bloody Run Catchment - EPA Stormwater Management Model

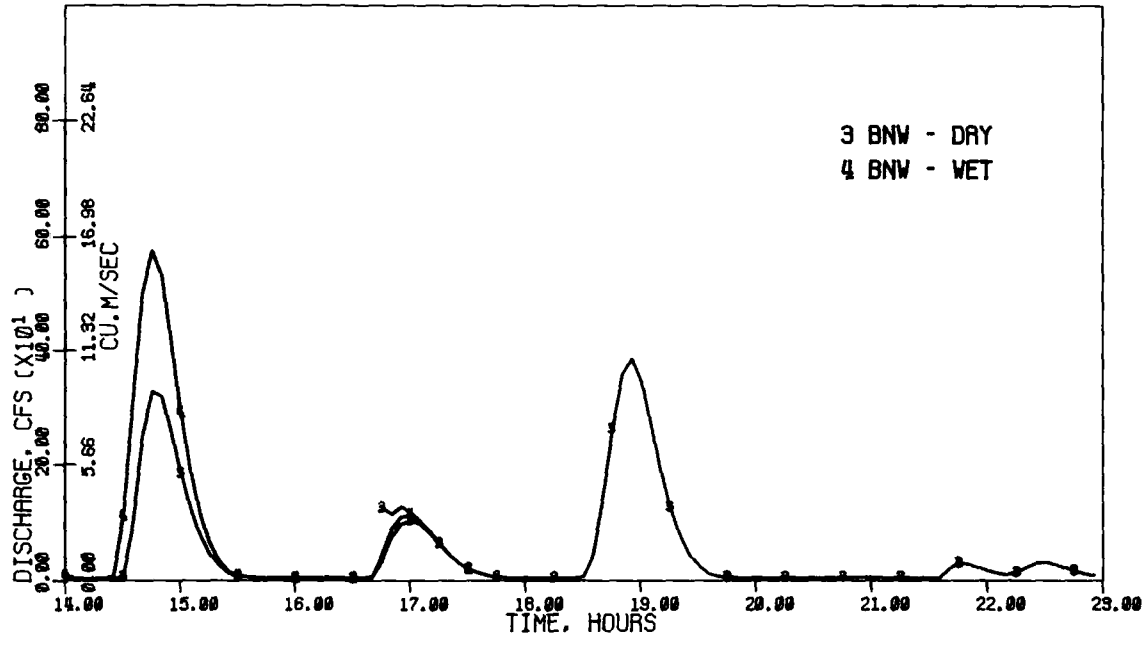


Figure 105. Computed Runoff at the Longview #1 Station for the Storm of August 25, 1971 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

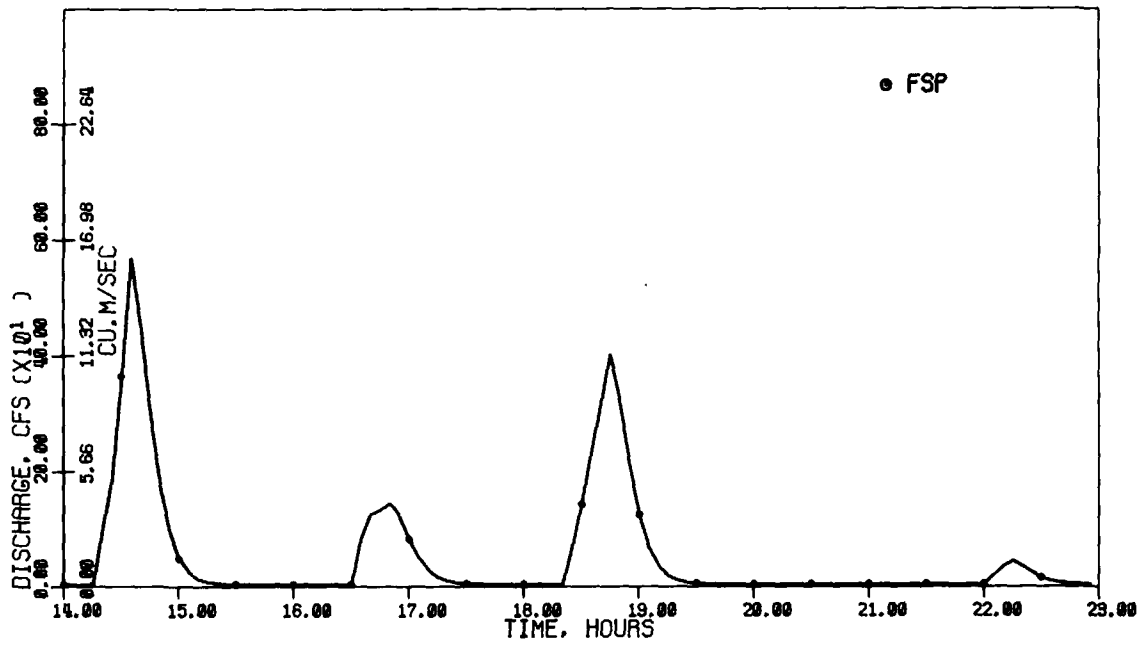


Figure 106. Computed Runoff at the Longview #1 Station for the Storm of August 25, 1971 - Bloody Run Catchment - Chicago Flow Simulation Program

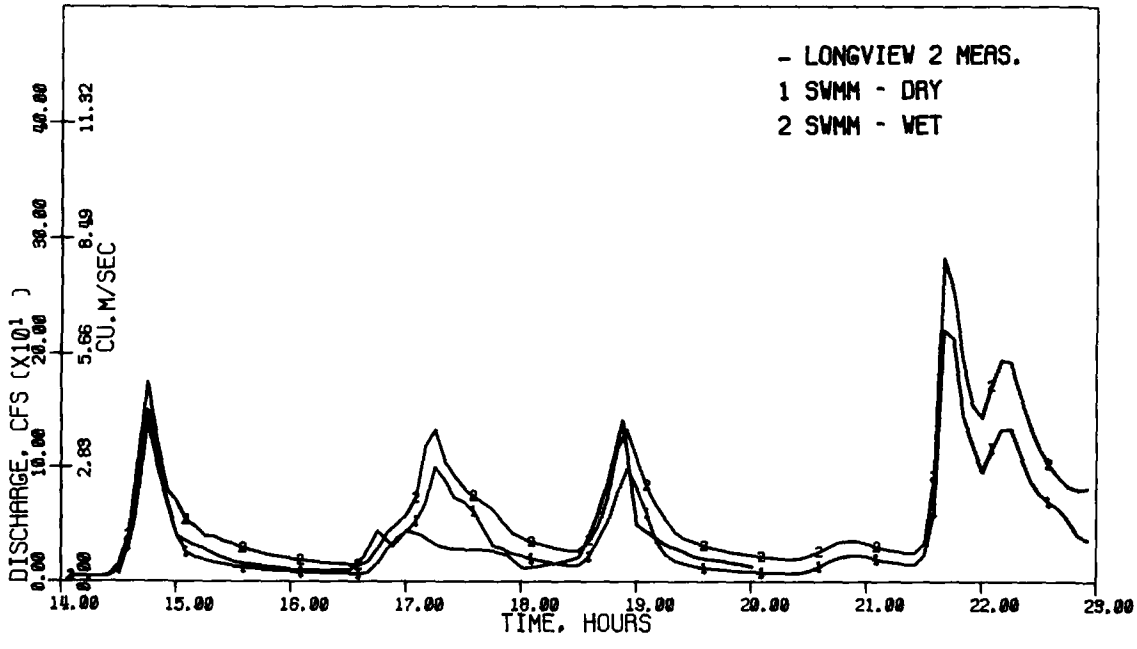


Figure 107. Comparison of Measured and Computed Runoff at the Longview #2 Station for the Storm of August 25, 1971 - Bloody Run Catchment - EPA Stormwater Management Model

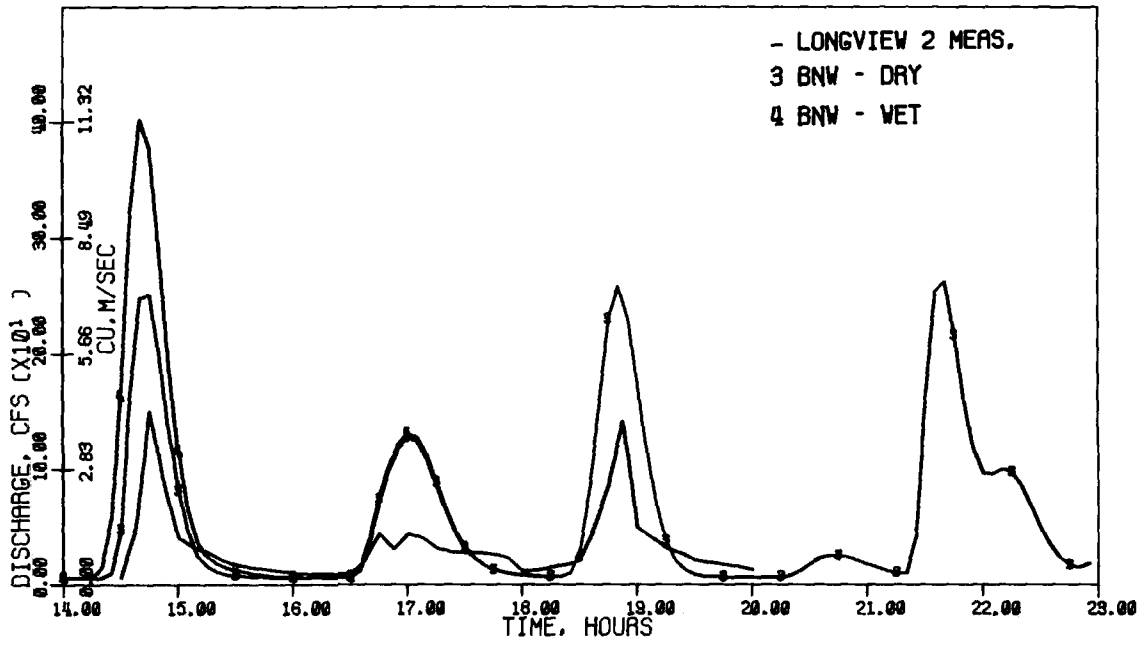


Figure 108. Comparison of Measured and Computed Runoff at the Longview #2 Station for the Storm of August 25, 1971 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

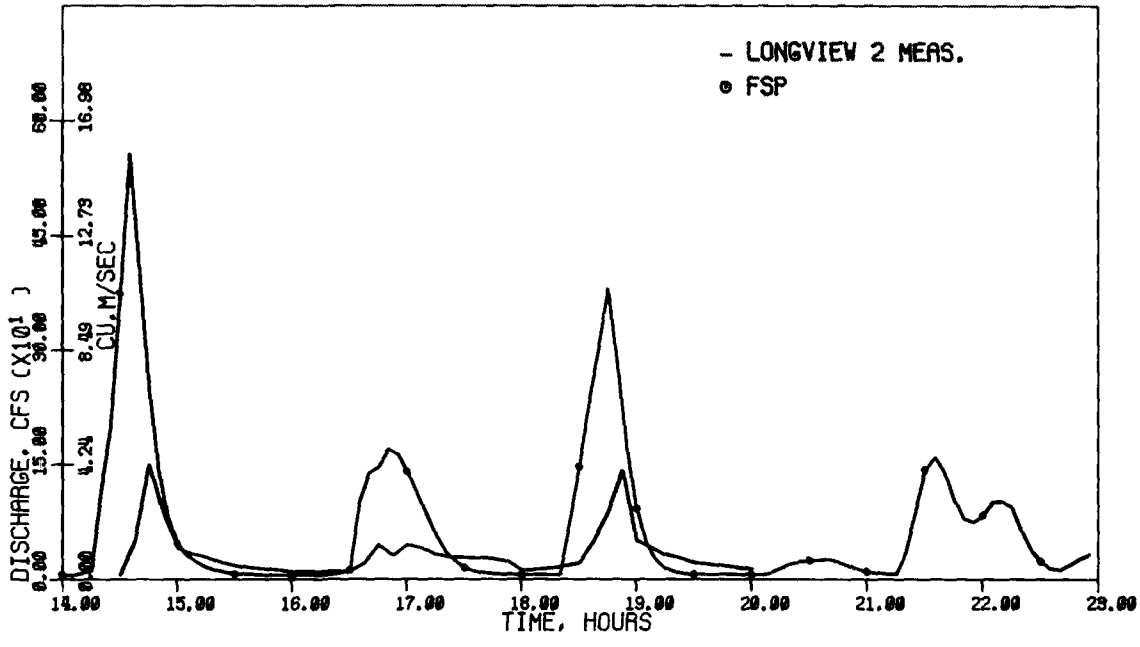


Figure 109. Comparison of Measured and Computed Runoff at the Longview #2 Station for the Storm of August 25, 1971 - Bloody Run Catchment - Chicago Flow Simulation Program

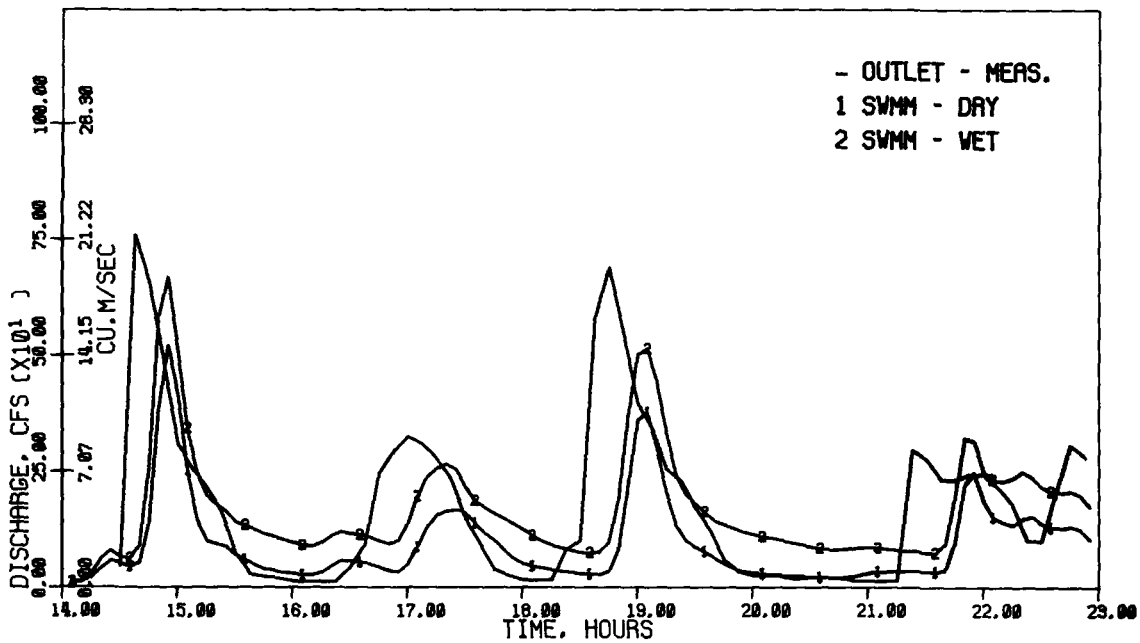


Figure 110. Comparison of Measured and Computed Runoff at the Outlet Station for the Storm of August 25, 1971 - Bloody Run Catchment - EPA Stormwater Management Model

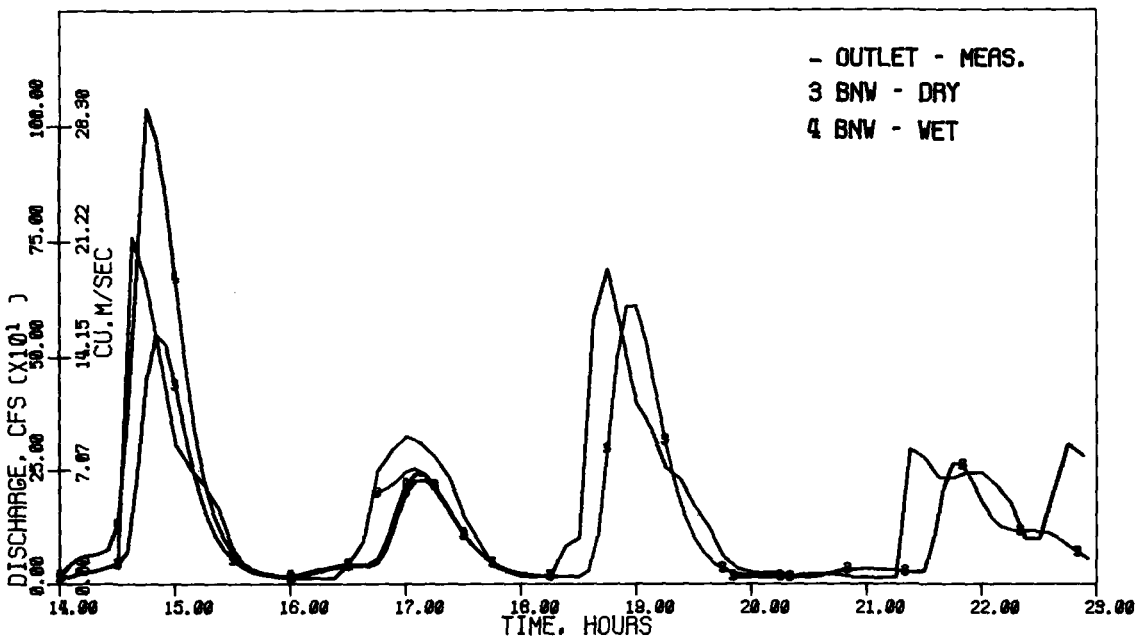


Figure 111. Comparison of Measured and Computed Runoff at the Outlet Station for the Storm of August 25, 1971 - Bloody Run Catchment - Battelle Urban Wastewater Management Model

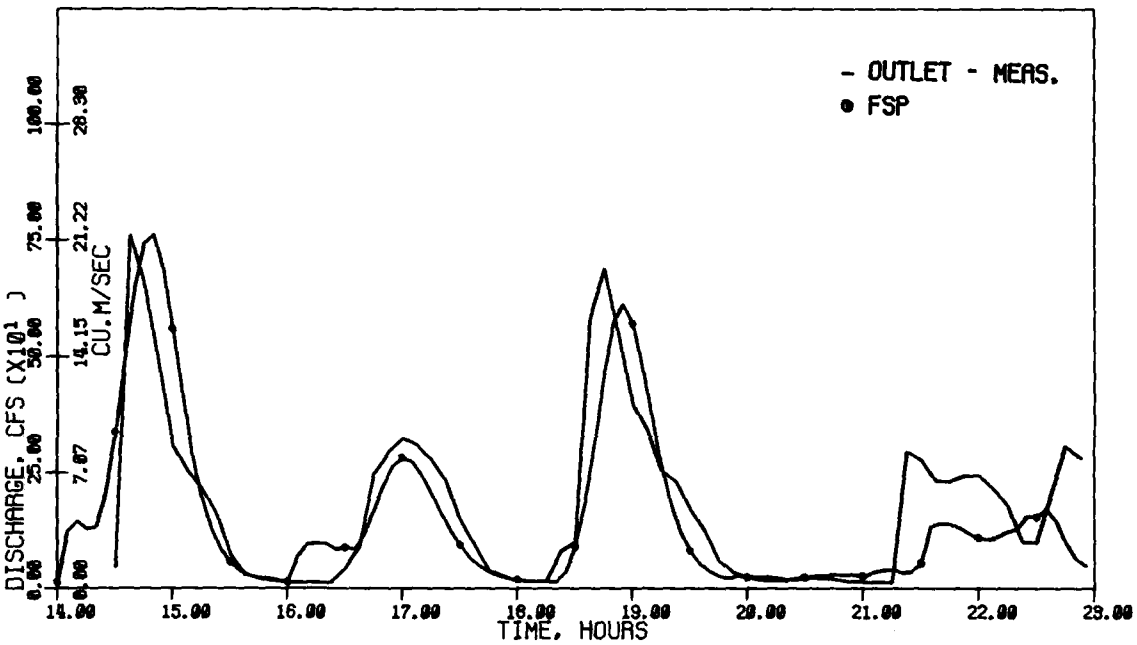


Figure 112. Comparison of Measured and Computed Runoff at the Outlet Station for the Storm of August 25, 1971 - Bloody Run Catchment - Chicago Flow Simulation Program

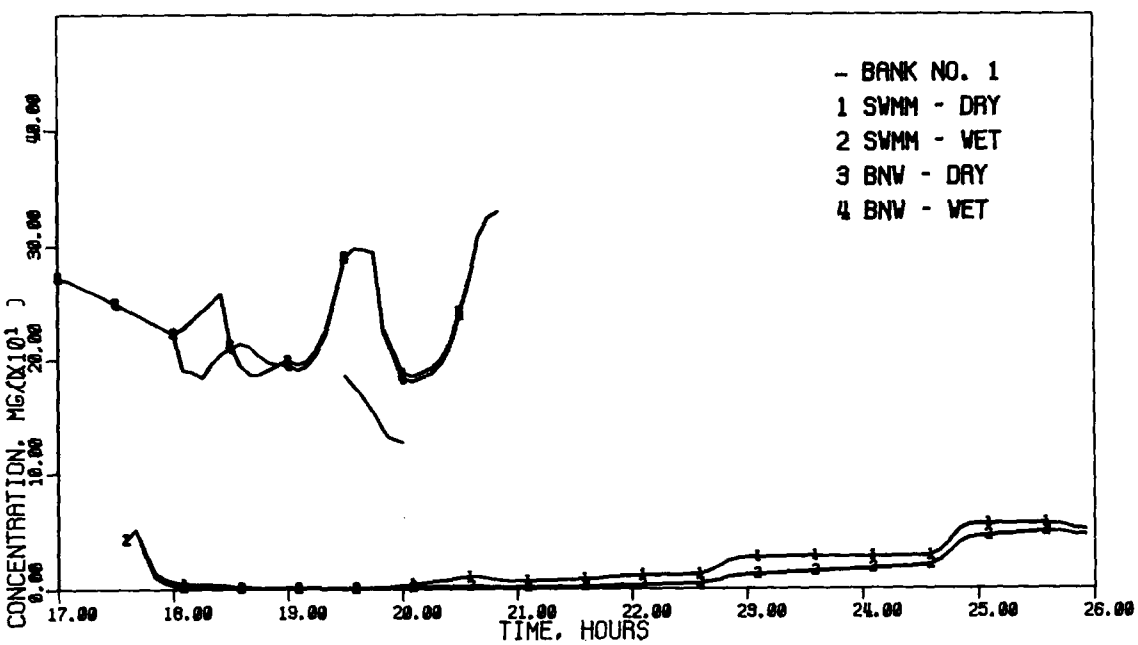


Figure 113. Comparison of Measured and Computed Suspended Solids Concentrations at the Bank #1 Station for the Storm Runoff of November 9, 1970 - Bloody Run Catchment

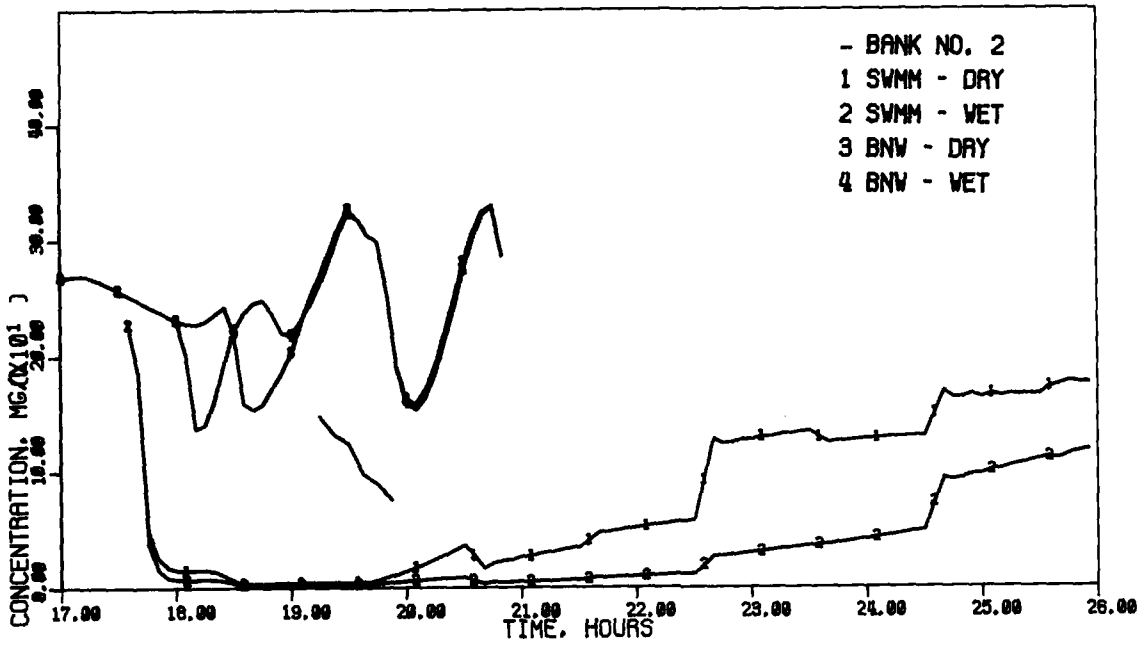


Figure 114. Comparison of Measured and Computed Suspended Solids Concentrations at the Bank #2 Station for the Storm Runoff of November 9, 1970 - Bloody Run Catchment

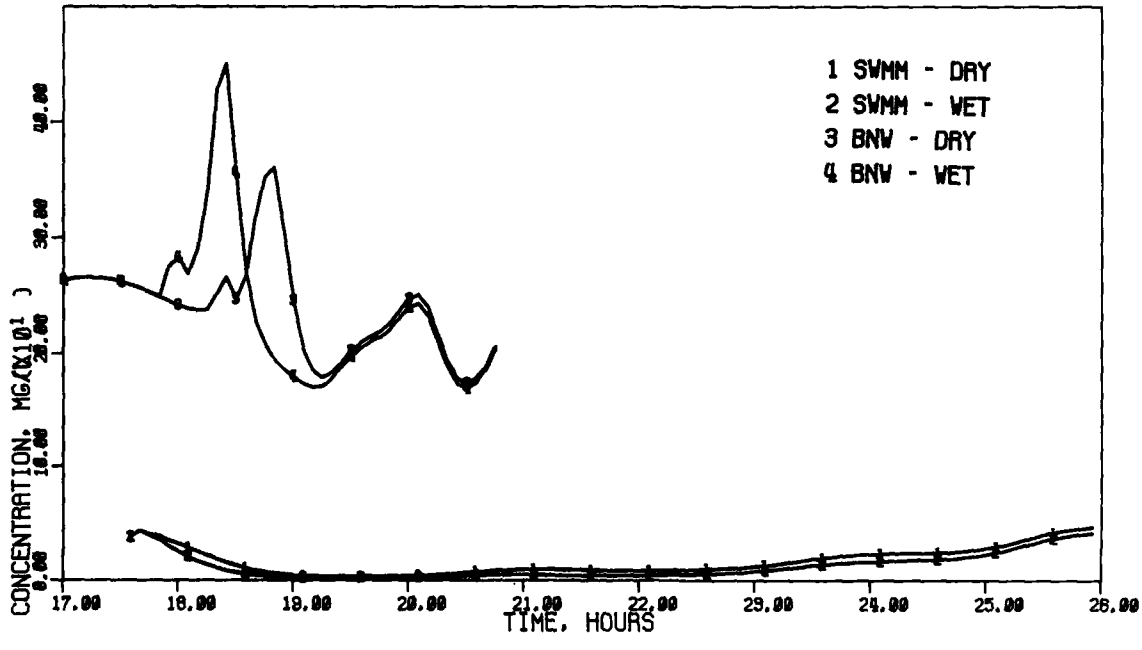


Figure 115. Computed Suspended Solids Concentrations at the Outlet Station for the Storm Runoff of November 9, 1970 - Bloody Run Catchment

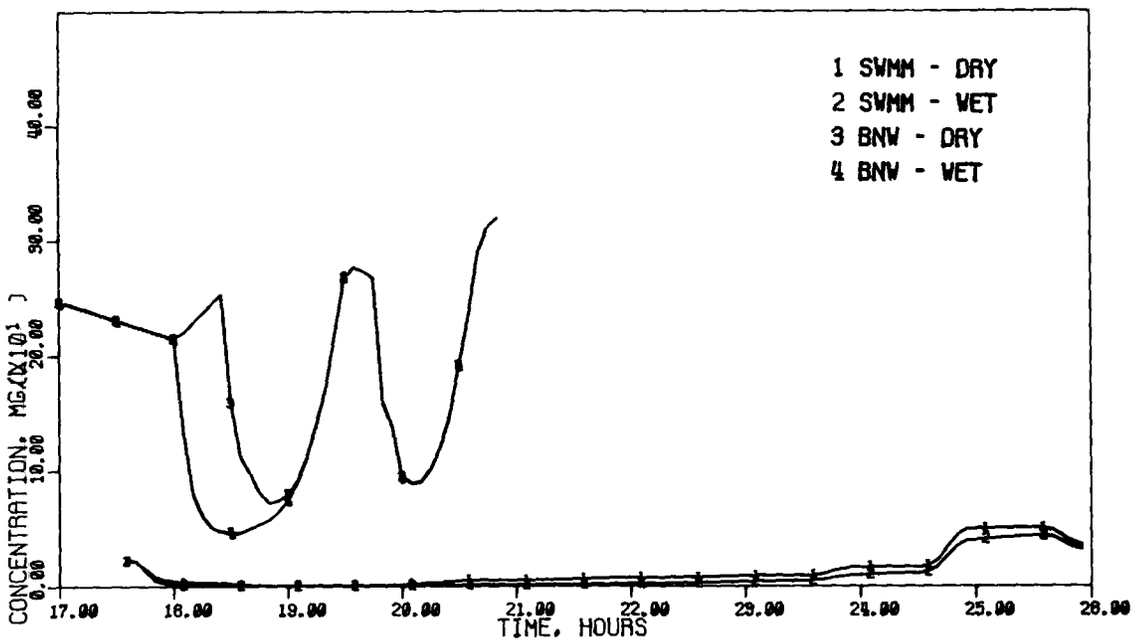


Figure 116. Computed Biochemical Oxygen Demand Concentrations at the Bank #1 Station for the Storm Runoff of November 9, 1970 - Bloody Run Catchment

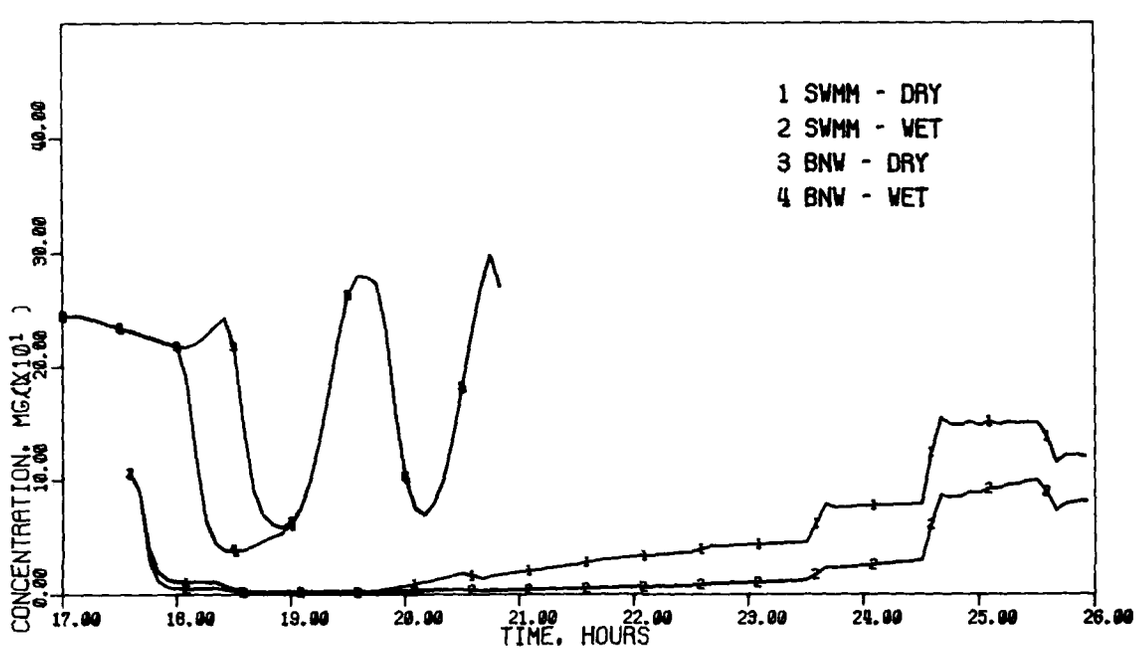


Figure 117. Computed Biochemical Oxygen Demand Concentrations at the Bank #2 Station for the Storm Runoff of November 9, 1970 - Bloody Run Catchment

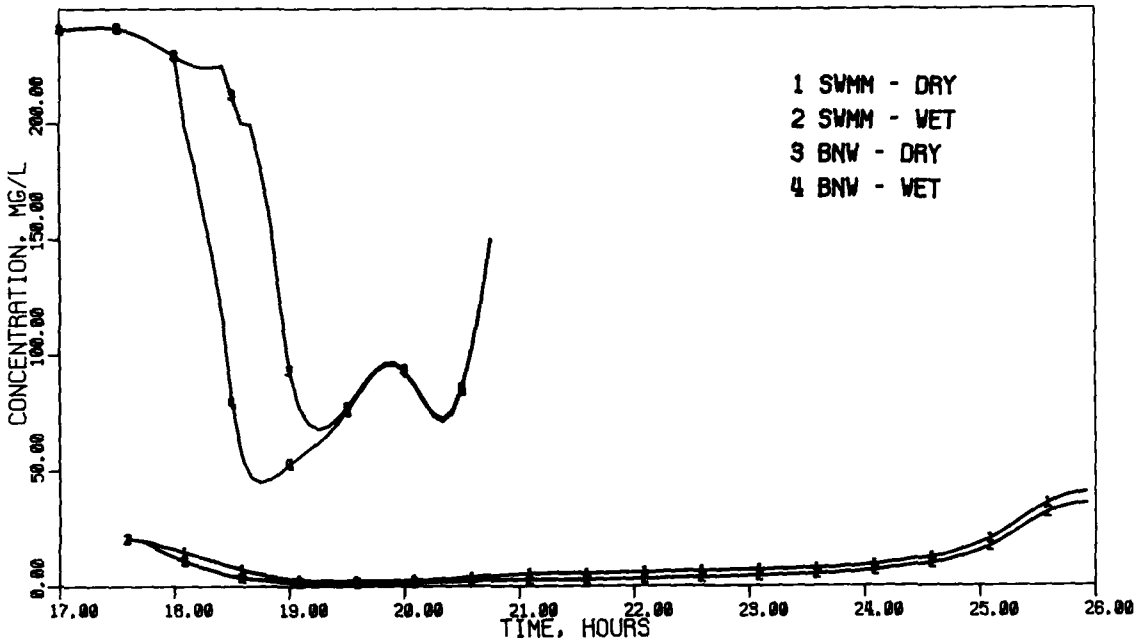


Figure 118. Computed Biochemical Oxygen Demand Concentrations at the Outlet Station for the Storm Runoff of November 9, 1970 - Bloody Run Catchment

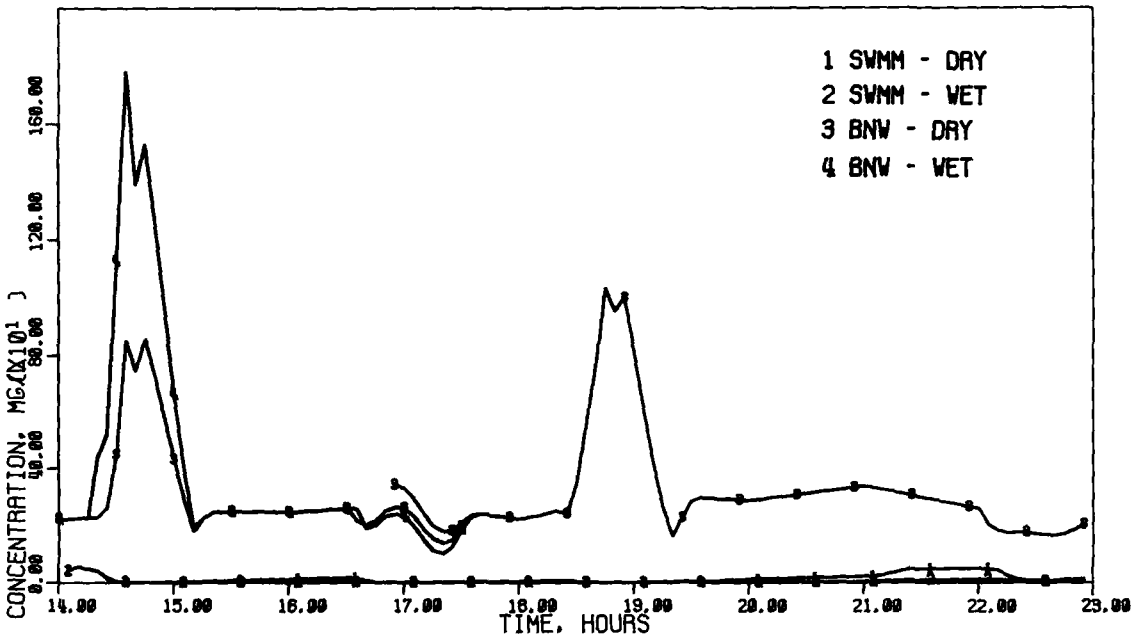


Figure 119. Computed Suspended Solids Concentrations at the Bank #1 Station for the Storm Runoff of August 25, 1971 - Bloody Run Catchment

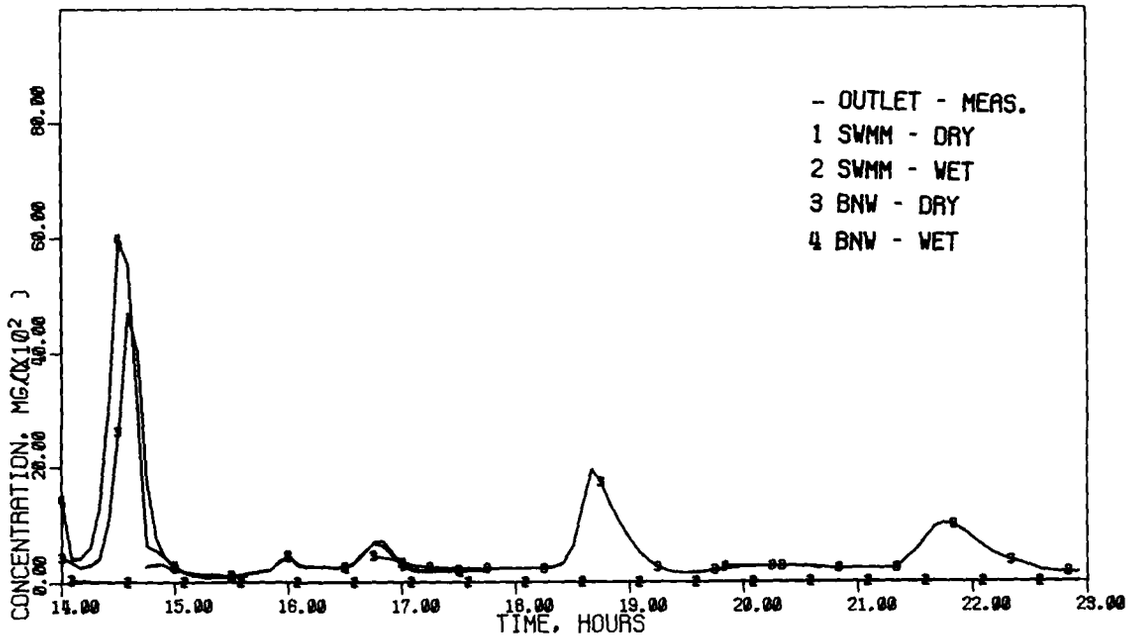


Figure 120. Comparison of Measured and Computed Suspended Solids Concentrations at the Outlet Station for the Storm Runoff of August 25, 1971 - Bloody Run Catchment

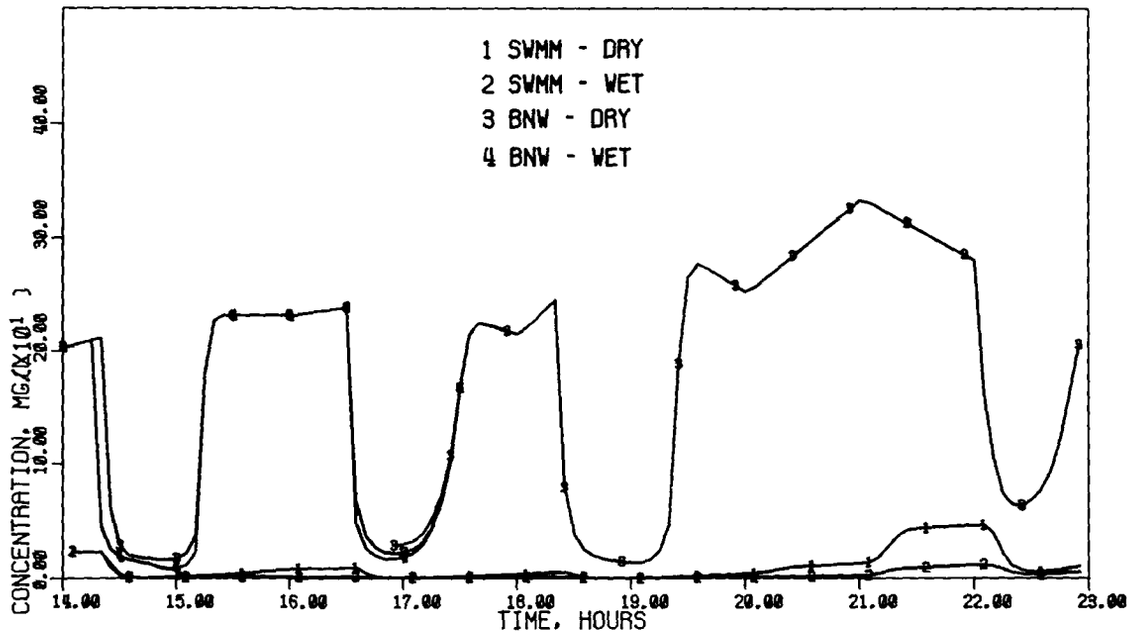


Figure 121. Computed Biochemical Oxygen Demand Concentrations at the Bank #1 Station for the Storm Runoff of August 25, 1971 - Bloody Run Catchment

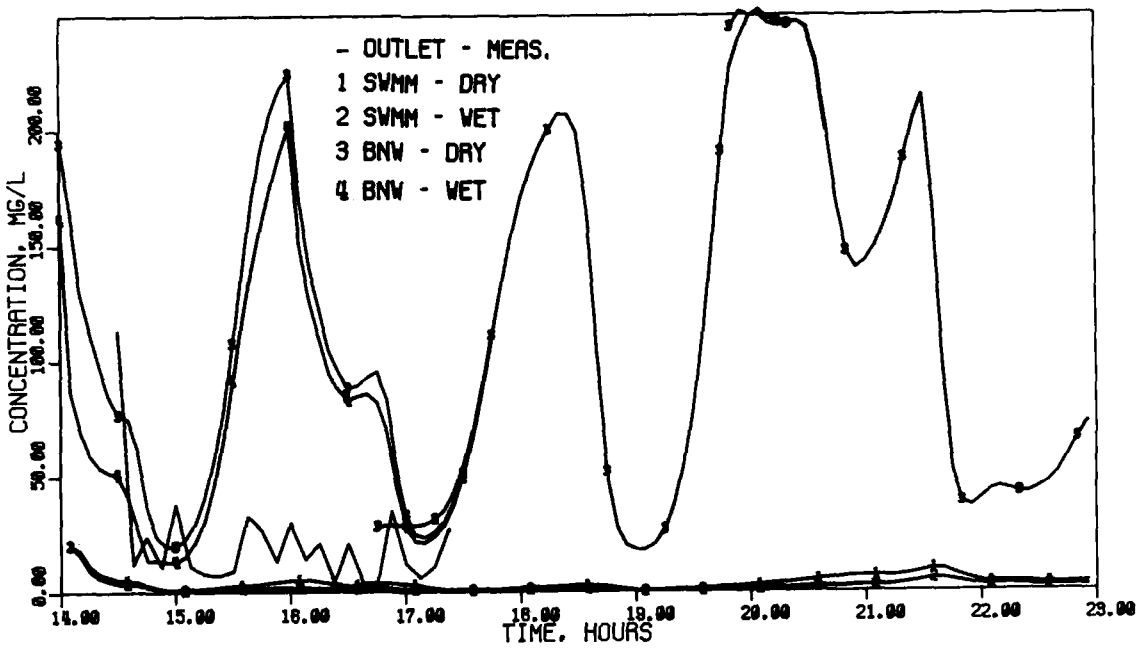


Figure 122. Comparison of Measured and Computed Biochemical Oxygen Demand Concentrations at the Outlet Station for the Storm Runoff of August 25, 1971 - Bloody Run Catchment

SECTION VIII

COSTS OF COMPUTER APPLICATIONS

CONTENTS

Page

Introduction. 347

Input/Output Features of Tested Models. 348

Environmental Protection Agency Stormwater Management Model 348

Battelle Urban Wastewater Management Model 348

Chicago Flow Simulation Program. 349

Dorsch Consult Hydrograph-Volume Method. 350

Massachusetts Institute of Technology Urban Watershed Model. 350

SOGREAH Looped Sewer Model 350

Water Resources Engineer Stormwater Management Model 350

Computer Times and Costs of Test Runs 350

Computer Processing Time 351

Computer Processing Costs. 353

Summary 356

INTRODUCTION

The cost of applying any of the reviewed models depends on the objectives of model use and the specific location of application. Objectives may include preliminary planning of new systems, evaluation of existing system performance, design of new system components, and real-time control of an existing or planned system.

The costs of implementing a model for any of these objectives are highly site specific. They depend upon the size of the system to be modeled, whether small subcatchments or only the major trunk and interceptors have to be simulated, and whether only flow or both flow and quality have to be considered. The costs also depend upon the complexity of the system, its interconnections, special facilities (such as diversions, pumps, siphons, tide gates, storage reservoirs, and treatment plants), the nature of the runoff events, and the importance of specific water quality parameters.

The availability of data and their form will greatly influence costs. Some cities maintain very efficient files of sewerage system plans and specifications and have monitored rainfall, runoff and water quality. In other cases, extensive field surveys and new monitoring programs may have to be implemented to collect sufficient data to characterize the catchments, sewerage system components, and hydrologic and water quality aspects.

The cost of running a program on a computer and performing computer simulations may be small in comparison to the costs of problem definition, data collection, and interpretation of the results. Only for very small catchments and simple problems may computer costs become a major item.

Significant differences can be expected, nevertheless, in the implementation and application of different computer programs, depending on their complexity and the available computer system. In general, these costs will increase with the number of phenomena being modeled, the number of options in a program, the size of the system, and the length of the time period to be simulated. The costs will decrease with the clarity of the input data arrangement, the adequacy of data error diagnostics, the efficiency of the numerical program algorithms, and the flexibility, completeness, and format of the desired computer output.

INPUT/OUTPUT FEATURES OF TESTED MODELS

Environmental Protection Agency Stormwater Management Model

The SWMM, being the most comprehensive of the tested models, may be the most expensive of the generally available models to implement and apply, provided all of its features are needed. Implementation is difficult because discrepancies exist between the model documents and the computer program and the documentation does not clearly separate the model's theoretical basis, testing and input data instructions. The user's manual is difficult to follow and not sufficiently clear to prevent misinterpretations of data definitions. Limits in numerical values are not indicated for variables which cause numerical problems if their value is not within a certain range. The model does not include sufficient error diagnostics.

The printout of hydrographs and pollutographs is limited to 20 sewerage system elements, which may not be adequate for large systems. Repeated simulations of the same system are required if printout is needed for more elements. The tables of hydrographs and pollutographs do not associate clock times with the discharge and pollutant concentrations or mass values. Their arrangement in blocks by system element, rather than columns, makes it difficult to obtain visual comparisons of tabulated values for selected times.

Battelle Urban Wastewater Management Model

The catchment runoff and sewer flow and water quality routing formulations of the BNW model are fairly simple. The complete model, however, is rather complex since it includes mathematical optimization for real-time control and for design. Implementation costs are expected to be high since the model is programmed for a small process computer and would require major reprogramming for applications to other computers. Documentation for the BNW model is only in the draft stage. The user's manual is separate from documentation of model testing and theoretical formulations; some variables are not defined sufficiently, however, to be understood without special instructions. The input data are arranged into logical groups, but the program does not have sufficient data error diagnostics to prevent false starts.

Stage, velocities, and discharge can be printed automatically for all conduit elements but discharge and pollutant concentrations have to be retrieved by light pen action on a cathode ray tube separately for each desired junction or diversion structure, which is time consuming. This is offset, however, by the low cost of the small computer. The model is highly flexible,

however, in that output can be limited to only the elements of interest, but output can also be obtained for all system elements if desired. The stage, velocity, and discharge in sewers are printed in columns as functions of time and easy to read, but they are tabulated at irregular time intervals. Hydrographs and pollutographs are printed at constant time intervals for junctions and diversion structures, with only the beginning time of the hydrograph or pollutograph indicated on the printout. The characteristic solution of the flow routing results for different sewerage system elements, making comparisons somewhat difficult. Plots can be obtained on an electrostatic printer, however, for any hydrograph and pollutograph and several graphs can be overlaid for easy comparison. A plot of the system schematic is a useful feature.

Contracting with Watermation, Inc., may be advantageous since the firm has improved and programmed the BNW model for routine applications using an IBM 360.

Chicago Flow Simulation Program

The FSP does not include water quality simulation and most model coefficients are set internally. It is consequently fairly easy to implement the model and to prepare the input data. The user's instructions are quite clear and the input data are well arranged. Limiting parameter values are not given. For large areas, the specification of the channel network by x-y coordinates has certain advantages over other methods of defining element connectivity.

Depth, flow, and storage are printed for each channel or conduit. The arrangement of the output, however, does not identify the sewer elements or the clock time. The user must therefore refer to the input data, which are not listed by the program automatically, to identify each line of the printout. Each line of computer printout contains the time step number and values of stage, discharge, and storage for only one element and one time step, which produces unnecessarily long computer printouts. The printout is arranged so that the stage, flow and storage can be seen easily at all elements for a given time step, but the complete hydrograph at a single element is not printed and is difficult to extract. Output is limited to 20 channel elements. Part of the printout related to the channel network configuration is not labeled.

The four proprietary models are generally applied by the model developers for interested clients, although they may be released under certain contractual arrangements. Sufficient information was not available in time to evaluate input data arrangements, consequently only output features are summarized.

Dorsch Consult Hydrograph-Volume Method

The DORSCH model printout is very well organized. This is the only model which prints both the depth and discharge separately for each of the three types of catchment surfaces. Clock time, stage, dry-weather and combined flow are printed in clearly-marked tables, facilitating inspection of channel stage and hydrographs. The user must memorize abbreviations used in the summary tables for output interpretation.

Massachusetts Institute of Technology Urban Watershed Model

The MIT model prints clock time, stage, velocities and discharge in columnar form for each sewerage system element, which is easy to read and to interpret. A useful feature is a line printer plot of a labeled schematic of the modeled system. A separate line printer plotting program is used to plot hydrographs.

SOGREAH Looped Sewer Model

The SOGREAH model prints one catchment runoff hydrograph per page, clearly labeling the clock time for all runoff values. Stage, discharge and concentrations at sewerage network elements are printed concurrently for the entire system for one value of time making it easy to obtain an overview for the entire network for a given time but difficult to extract hydrographs for a particular element. A separate plotter program is used for this purpose.

Water Resources Engineer Stormwater Management Model

The WRE model output is very well organized. Separate tables are printed for junctions and conduits. The junction tables list clock time and water surface elevation and depth while the conduit tables list clock time, discharge and velocity. Separate tables list clock time, discharge and concentration for each pollutant at selected junctions. Line printer plots of stage, discharge, and concentrations are also available.

COMPUTER TIMES AND COSTS OF TEST RUNS

Comparisons of computer processing time and costs are provided for the hypothetical data tests performed with the SWMM, BNW, FSP, DORSCH, MIT, SOGREAH and WRE models. These tests are equivalent to the simulation of four runoff events, each of six hours' average duration for a watershed containing 80 subcatchments, 37 kilometers (23 miles) of conduit, 6 diversion structures and 6 storage basins. Three pollutants were routed with each runoff hydrograph for the models simulating water quality (Table 74).

Table 74. SUMMARY OF HYPOTHETICAL DATA TESTS

4	Runoff events
6	Hour average duration
80	Catchments
37	Kilometers of pipe (23 miles)
6	Diversion structures
6	Storage basins
3	Pollutants

Computer Processing Time

The computer processing time required for these tests is listed for the seven models in Table 75. Due to the proprietary nature of four of the models and the special requirements of the BNW model, it was not possible to run all models on the same computer to obtain good running-time comparisons. The table nevertheless presents useful information for comparison of at least those models which were run on similar computers. Since not all models performed all hypothetical data tests, some of the listed running times are estimates based on the assumption that the same number of data combinations were performed by all models.

Computer running times for the pollutant routing are not listed for the FSP and MIT models since they do not include water quality formulations. Computer running times for the pollutant routing by the DORSCH model were not available. For the SOGREAH and WRE models it was possible to list the times required for the flow and pollutant routing separately since these models performed separate computer runs for them. For the SWMM and BNW models, the flow and pollutant routing is performed in the same run; consequently only the total running time for both can be listed.

The computer processing times of the FSP, DORSCH, MIT, SWMM, SOGREAH and WRE models for the catchment runoff computations are quite comparable, although they were performed on different computers. The SWMM version used for these tests performed water quality computations, although they were not needed, and consequently performed more computations than the other models. A new SWMM version allows the suppression of water quality

Table 75. COMPUTER PROCESSING TIME FOR HYPOTHETICAL DATA TESTS (minutes)

Model	Flow only			Flow and quality			
	FSP	DORSCH	MIT	BNW*	SWMM*	SOGREAH	WRE
Computer	CDC 6400	UNIVAC 1108	IBM 370/195	PDP 9	CDC 6400	IBM 360/65	UNIVAC 1108
Runoff	29	15	25	300	38	8	14
Flow routing	18	40	32	-	-	23	26
Pollutant routing	-	-	-	-	-	17	9
Flow and pollutant routing	-	-	-	624	81	40	35
Total	47	55	57	924	119	48	49

Note: *With plotting.

computations, which would result in shorter running time for the hypothetical catchment tests.

The FSP, DORSCH, MIT, WRE and SOGREAH models show comparable running times for the flow routing, and the SWMM, WRE and SOGREAH models are comparable for the combined flow and pollutant routing. The BNW model running times are much longer because of the slower execution speed of the PDP-9.

Computer Processing Costs

Processing cost, a more meaningful comparison than processing time, is listed in Table 76. The FSP and SWMM were run on a CDC 6400 computer at a cost of U.S. $640/hr, the MIT model on an IBM 370/195 computer at a cost of U.S. $860/hr, the WRE model on a UNIVAC 1108 computer at a cost of U.S. $750/hr, and the BNW model on a PDP-9 computer at a cost of U.S. $35/hr. Sufficient information was not available for these models to estimate computer running time and cost as a function of number of sewerage system elements and number of time steps. Test runs with the WRE surface runoff transport model performed by Water Resources Engineers indicated nearly identical costs for the CDC 6600, IBM 360/65 and UNIVAC 1108 computers.

The DORSCH model was run on a UNIVAC 1108 computer at a cost of U.S. $600/hr. Dorsch Consult provided the following information to estimate the cost of applying the DORSCH model:

> Computer running time for flow routing = 0.02 to 0.07 sec x number of sewerage system elements x number of time steps; 0.02 sec for sewerage systems with few diversions, weirs, special structures, and backwater effects; 0.07 sec for sewerage systems with loops and extensive backwater effects throughout the entire network; 0.05 sec for average configurations.
>
> Input data preparation: 0.25 to 0.60 hr per sewerage system element for evaluating existing systems and 0.35 to 0.60 hr per sewerage system element for evaluating system improvements depending on the availability and quality of sewerage system plans and data.

Dorsch Consult estimates that the run time of a CDC 6600 computer would be 55.5 percent of the run time on a UNIVAC 1108.

The SOGREAH model was run on an IBM 360/65 computer at a cost of U.S. $900/hr. SOGREAH provided the following information to estimate the computer cost for applying the SOGREAH model:

Table 76. COMPUTER COSTS FOR HYPOTHETICAL DATA TESTS
(dollars)

	Flow only			Flow and quality			
Model	FSP	DORSCH	MIT	BNW*	SWMM*	SOGREAH	WRE
Computer	CDC 6400	UNIVAC 1108	IBM 370/195	PDP 9	CDC 6400	IBM 360/65	UNIVAC 1108
Cost per hour	640	600	860	35	640	900	750
Runoff	310	150	360	175	405	120	170
Flow routing	190	400	460	-	-	345	325
Pollutant routing	-	-	-	-	-	255	110
Flow and pollutant routing	-	-	-	365	865	600	435
Total	500	550	820	540	1270	720	610

Note: *With plotting.

Computer running time for flow routing = 0.002 sec x number of sewerage system elements x number of time steps + 0.00012 sec x (number of sewerage system elements)^3.

The costs of the catchment runoff computations are comparable for the DORSCH, BNW, SOGREAH and WRE models. The cost of the SWMM catchment simulations could be reduced with the new version by suppressing the catchment water quality simulations. It is not clear why the FSP and MIT model runs of the catchment simulations would cost two to three times more than the other models. The actual cost of the MIT catchment simulations was less, since only fully pervious and fully impervious catchments were simulated and the runoff for the various combinations of pervious and impervious catchments was computed from these in proportion to the percent imperviousness. The listed cost would be representative, however, for running all 320 catchment data combinations.

The FSP shows the lowest cost for flow routing due to the simple linear routing scheme. Comparable costs are shown for the DORSCH, MIT, WRE, and SOGREAH models. The MIT cost is the highest of these four, although it uses the simplest routing scheme; the cost difference, however, may not be significant.

For the combined flow and pollutant routing, the BNW model shows the lowest cost, primarily because it runs on the inexpensive PDP-9 computer. Surprisingly, the SOGREAH and WRE models are less expensive than the SWMM model although they solve the full dynamic wave equations and the SWMM solves the simpler kinematic wave equation.

These comparisons indicate that the tested models feature efficient numerical solution algorithms for the full dynamic wave equation which are competitive with the more approximate kinematic wave equation. The tests also indicate that no significant differences exist in the computer running costs of models which use either an explicit (WRE model) or implicit (DORSCH and SOGREAH models) finite difference solution of the dynamic wave equation. Preliminary runs by the model developers indicate, however, that computer running times several times longer than necessary may result if the time step is chosen shorter than necessary with respect to the time and space variability of the modeled phenomena. Explicit solutions generally require much smaller computational time steps than implicit solutions due to stringent numerical stability conditions, but fewer iterations are required for the solution at any one time step for the explicit than the implicit solutions. For example, to obtain the same accuracy for a particular application, the implicit scheme may be able to use time

steps which are ten times longer than the explicit scheme. On the other hand, the explicit scheme may need only four iterations to converge on a solution at a particular time step, while the implicit scheme may need as many as 30 iterations. Whether one scheme is more efficient than the other depends on the physical characteristics of the channel system and flow phenomena being modeled. No general criteria have been developed.

SUMMARY

It should be pointed out that these computer processing costs can be used only as guidelines. Relative running times and costs for the models may be quite different for real catchments as a result of special features which permit the identification of typical catchment elements and reduce duplication of data input and simulations for similar subcatchments and sewerage system components. The BNW, MIT and WRE models have features of this type.

At the beginning of the model comparison, it was planned to run all models selected for the numerical testing on the same computer to provide a common base for comparing computer processing time and cost. Subsequently, the FSP and SWMM were converted to run on the CDC 6400 computer of Battelle-Columbus, since an IBM computer was not readily available to Battelle-Northwest at that time. Conversion of the BNW model, however, from the small PDP-9 computer to the CDC 6400 was not performed due to expected high cost. In addition, the four proprietary models had to be run on the computers being used by the respective firms since the programs were not released.

The cost comparisons are still realistic, however, since the potential user would run a particular model on the computer for which the model is written unless he is willing to pay for the conversion cost. The costs in Table 76 are consequently valid comparisons of computer costs for running the simple hypothetical data tests. Additional numerical tests simulating large sewerage networks of varying complexity would be required to obtain a more comprehensive comparison of computer costs for running the different models, to evaluate the effect on computer processing costs of model differences in input data arrangement for complex networks, of economies in computing hydrologic phenomena for typical subcatchments only rather than every subcatchment, and of different network flow routing algorithms.

SECTION IX

DISCUSSION

CONTENTS

	Page
Introduction.	358
Data.	358
Models.	359

INTRODUCTION

Several observations can be made as a result of the preceding model reviews and numerical testing. It is hoped that they may provide some guidance with respect to future model development and implementation.

DATA

Although highly sophisticated mathematical models exist for the simulation of water flow and quality in urban drainage systems, their verification and application has been greatly hindered by the lack of reliable and comprehensive data on physical characteristics of urban catchments, rainfall runoff, and water quality.

Past data collection programs have generally been designed to provide answers to specific problems or to develop relationships of limited scope, rather than to provide a general body of information such as is needed to assess the comprehensive models reviewed in this study. Quite often, reliable rainfall and runoff data are collected but data are insufficient or unavailable to describe other important meteorological variables and catchment moisture conditions at the beginning of rainfall and during the rainstorm. Quite commonly, the physical dimensions of the sewerage system have not been described adequately, such as length, areas, slopes, and roughness characteristics of the runoff surfaces; sizes, slopes and materials of conduits and open channels; and configurations and dimensions of special control structures. Expensive programs would be needed to obtain these measurements and in some cases, the watershed characteristics may have changed sufficiently since the rainfall and runoff data were collected to make post-facto measurements meaningless.

This lack of comprehensive data has forced model developers to test portions of their models on data for different watersheds. This requires replacing unavailable data with assumptions, and consequently the tests cannot be considered as conclusive as if all needed model data had been available for the same catchment.

Very rarely is the accuracy of the measured data documented. Data accuracy can be estimated if descriptions of the instrumentation and measurement procedures are available, but this is still less than satisfactory. In one case it was found that portions of the reportedly observed runoff and water quality data were not measured but interpolated from discontinuous measurements. In another case, runoff was computed from water pressure measurements with Manning's equation without attempts to calibrate the equation (Manning's n) with measured depth and velocity measurements. In computing the discharge, uniform flow conditions were assumed where backwater conditions may have existed for certain

flow conditions, particularly where the measuring stations were a short distance upstream of major trunk sewer junctions.

A fair amount of reliable runoff data is now available, but reliable water quality data are hard to find. Particularly lacking is information which would permit definitive conclusions or correlations between catchment land uses and water quality.

MODELS

This review of eighteen sewerage system simulation models and testing of seven of them indicates that it is extremely difficult to select the best model for a particular application. Some models have been designed for specific objectives not considered by others, and consequently they would be preferred if the user has the same objectives in mind. Some proprietary models have features which appear superior to the publicly available models, but a user may prefer to run his own model and consequently has to be satisfied with a model that does not exactly meet his requirements.

Quite often it is found that the model does not perform exactly as claimed by the model developers or that certain features which appeared to be working for the developer are not adequate for the needs of a new user. The user should therefore expect to perform some model modifications and improvements. Most models that are actively used are consequently in a continuous state of development by both the original developers and by various users. As a result, several versions of the same model may exist and the new user has the added problem of making a selection from different versions of the same model.

Because of this continuing development work, model documentation is rarely up-to-date and the user is faced with reconciling differences among the model descriptions, user's manuals, and the actual computer program. Sometimes the model reports describe features which were planned for a model which to the model user's surprise were never implemented in the computer program he receives.

The numerical testing of the models indicates that a direct relationship between model complexity and its cost of implementation and application exists with respect to the number of major phenomena which are modeled. Efficient solution algorithms, however, may reduce this difference significantly. This is true particularly for proprietary models due to their need to stay competitive. A model which simulates many special sewerage system facilities will be more complex in structure and require more data and computer storage than a model that computes only runoff from a single catchment without routing flows or which does only flow routing in a simple converging network without computing runoff from precipitation and land use.

The simulation of water quality adds considerable complexity to a model, even if it routes only conservative substances. The complexity increases substantially if both storm and dry-weather water quality is computed from land use characteristics. Additional complexities are added if wastewater treatment and receiving water flow and quality are being modeled.

To reduce this complexity, it would be desirable to have a series of models which are interfaced by common input and output, that can be run independently provided the required data are available, or that can be run in any desired sequence. This would allow model implementation in stages, reduce implementation cost and computer storage requirements, and facilitate acceptance of the models. A series of models, rather than one large comprehensive model, would reduce the need to compromise in specific submodel formulations, a course which is often required to save computer space.

Such model series already exist, but some streamlining and improvement of components is needed to facilitate their use, either independently or in any sequence. An example is the EPA's Stormwater Management Model, which simulates storm and dry-weather runoff and water quality from catchments, routes them through complex sewerage networks, simulates storage and various treatment processes, and computes the effects of sewerage system effluents on receiving water quality. The model needs several improvements, however, before it becomes sufficiently general to model all important phenomena and to allow arbitrary sequencing of major model components. Needed improvements, for instance, would be the simulation of catchment moisture balance between rainstorms to provide continuous simulation capability, the addition of snowmelt and evapotranspiration, the simulation of backwater, flow reversal and surcharging, a more accurate formulation for storage and flow diversion facilities, the simulation of main treatment plants, and the interfacing with more comprehensive receiving water quality models for river basins, including estuaries and deep lakes and impoundments.

The testing of seven models indicates that approximations in mathematical formulations to simplify their solution do not necessarily reduce computer processing times, as shown by comparisons in the flow routing between models using different numerical schemes for solving the kinematic and dynamic wave equations. Solutions of more complex and comprehensive equations can apparently be more efficient than attempts to approximate comprehensive phenomena with simplified equations. The testing and review of the models indicates also that the flow and water quality routing, although complex for looping and converging and diverging branch systems with special structures, are the best understood phenomena and that the selection of a particular mathematical formulation

and numerical solution technique is governed only by the preference and needs of the model developer and user. Considerable uncertainties exist, however, in the modeling of catchment phenomena, both the flow and water quality of storm and dry-weather runoff. The definition of adequate formulations for soil infiltration, the filling of depression storage, evapotranspiration, groundwater seepage and soil moisture are extremely difficult considering the heterogeneity of catchment land uses, geometry, vegetation, and soils. The adequacy of catchment water quality computations from catchment land use and runoff has not been sufficiently demonstrated. Although various models have shown good agreement between measured and computed catchment runoff water quality, the comparisons have been too limited to assign confidence limits to predictions for catchments without measurements.

Another problem is related to the calibration of the more comprehensive models, that is the determination of the best values of empirical model coefficients. Traditionally, much hard work is required to obtain a best fit between measured and computed runoff (and water quality, if modeled), and generally the model user must rely on his experience and intuition to adjust model coefficients in an optimal manner. Automatic optimization schemes would be needed to obtain optimal values of coefficients and to indicate confidence limits to be expected in model applications for predictions.

In summary, the reviewed models provide very useful tools to the engineer and planner for assessing, designing, planning and controlling storm and combined sewerage systems. It is extremely important, however, that the potential model user study the formulations of the models, their limitations and approximations, if he is to use the models in an appropriate manner. It is hoped that this review will aid him in assessing model capabilities and limitations and thus to select models most suited for his purposes.

SECTION X

REFERENCES

Akan, A. O. Unsteady Gutter Flow Into Grate Inlets. University of Illinois at Urbana-Champaign, Department of Civil Engineering, M.S. Thesis, 1973.

Amorocho, J. The WH-1 Urban Watershed System for Hydrologic Simulation and Sewer Design. Wilsey & Ham, Consulting Engineers, Foster City, California, Unpublished Manuscript, 1972.

Anderson, J. J. Real-time Computer Control of Urban Runoff. Journal of the Hydraulics Division, American Society of Civil Engineers, 96(HY1):153-164, Proc. Paper 7028, January 1970.

Argaman, Y., U. Shamir, and E. Spivak. Design of Optimal Sewerage Systems. Journal of the Environmental Engineering Division, American Society of Civil Engineers, 99(EE5):703-716, Proc. Paper 10050, October 1973.

Blank, D., J. W. Delleur, J. Parker, A. R. Rao, and G. H. Toebes. Fortran Hydro CDC 6500. Purdue University, Lafayette, Indiana, Water Resources and Hydrodynamics Laboratory, Technical Report No. 36, April 1972, Errata August 1973.

Bertle, F. A. Effect of Snow Compaction on Runoff From Rain on Snow. U.S. Bureau of Reclamation, Engineering Monograph No. 35, June 1966.

Brandstetter, A., R. L. Engel, and D. B. Cearlock. A Mathematical Model for Optimum Design and Control of Metropolitan Wastewater Management Systems. Water Resources Bulletin, 9(6):1188-1200, December 1973.

Bugliarello, G., W. D. McNally, J. T. Gormley, and J. T. Onstott. A Pilot Problem-Oriented Computer Language for Hydrology and Hydraulic Engineering ("HYDRO"). Paper Presented at the 47th Annual Meeting, American Geophysical Union, Washington, D. C., April 1966.

Bugliarello, G., W. D. McNally, J. T. Formley, and J. T. Onstott. Design, Philosophy, Specifications and Implications of "HYDRO", A Pilot Computer Language for Hydrology and Hydraulic Engineering. Water Resources Research, 3(2):736-642, 1967.

Bugliarello, G., and J. T. Onstott. A Progress Report on "HYDRO". Proceedings IBM Scientific Computing Symposium on Water and Air Resources Management, Form No. 320-1953, 1968.

Cawood, P. B., R. Thunvik, and L. Y. Nilsson. Hydrological Modeling--An Approach to Digital Simulation. Royal Institute of Technology, Stockholm, School of Surveying, Department of Land Improvement and Drainage, Report 3:4a, 1971.

Chen, C. W. Concepts and Utilities of Ecological Model. Journal of the Sanitary Engineering Division, American Society of Civil Engineers, 96(SA5):1085-1097, Proc. Paper 7602, October 1970.

Chen, C. W., and R. P. Shubinski. Computer Simulation of Urban Storm Water Runoff. Journal of the Hydraulics Division, American Society of Civil Engineers, 97(HY2): 289-301, Proc. Paper 7924, February 1971.

Chen, C. L., and V. T. Chow. Formulation of Mathematical Watershed-Flow Model. Journal of the Engineering Mechanics Division, American Society of Civil Engineers, 97(EM3):809-828, Proc. Paper 8199, June 1971.

Chen, C. W., and G. T. Orlob. Ecologic Simulation for Aquatic Environments. Water Resources Engineers, Inc., Walnut Creek, California, for Office of Water Resources Research, December 1972.

Chow, V. T., and V. C. Kulandaiswamy. General Hydrologic System Model. Journal of the Hydraulics Division, American Society of Civil Engineers, 97(HY6):791-804, Proc. Paper 8176, June 1971.

Chow, V. T., and A. Ben-Zvi. The Illinois Hydrodynamic Watershed Model III (IHW Model III). University of Illinois at Urbana-Champaign, Department of Civil Engineering, Hydraulic Engineering Series No. 26, September 1973.

City of Chicago. Development of a Flood Control and Pollution Control Plan for the Chicagoland Area. Bureau of Engineering, Department of Public Works, City of Chicago, with Metropolitan Sanitary District of Greater Chicago and Illinois Institute for Environmental Quality, Computer Simulation Programs, Technical Report Part 2, December 1972.

Claborn, B. J., and W. Moore. Numerical Simulation in Watershed Hydrology. University of Texas, Austin, Hydraulic Engineering Laboratory, Technical Report HYD 14-7001, 1970.

Crawford, N. H., and R. K. Linsley. Digital Simulation in Hydrology: Stanford Watershed Model IV. Stanford University, Palo Alto, California, Department of Civil Engineering, Technical Report No. 39, July 1966.

Crawford, N. H. Studies in the Application of Digital Simulation to Urban Hydrology. Hydrocomp International, Inc., Palo Alto, California, for Office of Water Resources Research, September 1971.

Crawford, N. H. Computer Simulation for Design Criteria for Urban Flow Storage Systems. Hydrocomp International, Inc., Palo Alto, California, for Office of Water Resources Research, January 1973.

Dajani, J. S., R. S. Gemmell, and E. K. Morlok. Optimum Design of Urban Wastewater Collection Networks. Journal of the Sanitary Engineering Division, American Society of Civil Engineers, 98(SA6):853-867, Proc. Paper 9414, December 1972.

DeCicco, P. R., H. F. Soehngen, and J. Takagi. Use of Computers in Design of Sanitary Sewer Systems. Journal of Water Pollution Control Federation, 40(2):269-284, February 1968.

Delleur, J. W., and G. H. Toebes. FORTRAN-HYDRO. Journal of the Hydraulics Division, American Society of Civil Engineers, 95(HY6):1993-2012, Proc. Paper 6901, November 1969.

Donigian, A., Jr., and W. H. Waggy. Simulation--A Tool for Water Resource Management. Water Resources Bulletin, 10(2):229-244, April 1974.

Dorsch-Consult. Pollution of Receiving Water Bodies from Combined and Storm Sewer Systems. Dorsch-Consult, Ltd., Toronto, Canada, June 1973.

Eagleson, P. S. Unit Hydrograph Characteristics for Sewered Areas. Journal of the Hydraulics Division, American Society of Civil Engineers, 88(HY2):1-25, Proc. Paper 3069, March 1962.

Evelyn, J. P., V. V. D. Narayana, J. P. Riley, and E. K. Israelsen. Hydrograph Synthesis for Watershed Subzones from Measured Urban Parameters. Utah State University, Logan, Utah Water Research Laboratory, August 1970.

Fisher, J. M., G. M. Karadi, and W. W. McVinnie. Design of Sewer Systems. Water Resources Bulletin, 7(2):294-302, April 1971.

Geiger, F. W. Urban Runoff Pollution Derived from Long-Time Simulation. Paper Presented at the National Symposium on Urban Hydrology and Sediment Control, Lexington, Kentucky, July 28-31, 1975.

Gibbs, C. V., S. M. Alexander, and C. P. Leiser. A Computer-Directed System for Regulation of Combined Sewage Overflows. Paper Presented at the National Water Resources Engineering Meeting, American Society of Civil Engineers, Atlanta, Georgia, January 1972a.

Gibbs, C. V., and A. L. Poole. Basin Management Techniques for Sewerage Agencies. Journal of the Sanitary Engineering Division, American Society of Civil Engineers, 98(SA3):491-504, Proc. Paper 8960, June 1972.

Gibbs, C. V., S. M. Alexander, and C. P. Leiser. System for Regulation of Combined Sewage Flows. Journal of the Sanitary Engineering Division, American Society of Civil Engineers, 98(SA6):951-972, Proc. Paper 9402, December 1972b.

Grigg, N. S., J. W. Labadie, G. L. Smith, D. W. Hill and B. W. Bradford. Metropolitan Water Intelligence Systems. Colorado State University, Fort Collins, Department of Civil Engineering, Completion Report - Phase II, for Office of Water Resources Research, June 1973.

Hagarman, J. A., and F. R. S. Dressler. Storm Water Management Model: Dissemination and User Assistance. University City Science Center, Philadelphia, Pennsylvania, to U.S. Environmental Protection Agency, Report EPA-670/2-75-041, May 1975.

Königer, W., G. Vogel, and H. Klym. Anwendung eines komplexen Abflußmodells in der Stadtentwässerung. Paper Presented at the Symposium Elektronische Berechnung von Rohr- und Gerinneströmungen held by the Technische Universität München, Germany, October 1972.

Lager, J. A., R. P. Shubinski, and L. W. Russel. Development of a Simulation Model for Stormwater Management. Journal of the Water Pollution Control Federation, 43(12): 2424-2435, December 1971.

Lanyon, R. F., and J. P. Jackson. Metropolitan Sanitary District of Greater Chicago Flow Simulation System. Paper Presented at the American Society of Civil Engineers Hydraulics Conference, Ithaca, New York, August 1972.

Lanyon, R. F., and J. P. Jackson. A Streamflow Model for Metropolitan Planning and Design. American Society of Civil Engineers, Urban Water Resources Research Program, Technical Memorandum No. 20, January 1974.

Lanyon, R. F., and J. Jackson. Flow Simulation System. Journal of the Hydraulics Division, American Society of Civil Engineers, 100 (HY8):1089-1105, Proc. Paper 10743, August 1974.

Leary, A. G., and P. Zepp. Sewer Design and Cost Estimation by Computer. Public Works, 101(6):90-91, June 1970.

Leclerc, G., and J. C. Schaake, Jr. Methodology for Assessing the Potential Impact of Urban Development on Urban Runoff and the Relative Efficiency of Runoff Control Alternatives. Massachusetts Institute of Technology, Cambridge, Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Report No. 167, March 1973.

Leiser, C. P. Computer Management of a Combined Sewer System. Municipality of Metropolitan Seattle for U.S. Environmental Protection Agency, Report No. EPA-670/2-74-022, July 1974.

Lichty, R. W., D. R. Dawdy, and J. M. Bergman. Rainfall-Runoff Model for Small Basin Flood Hydrograph Simulation. International Association of Scientific Hydrology, Publ. No. 81, The Use of Analog and Digital Computers in Hydrology, pp. 356-367, December 1968.

Liebman, J. C. A Heuristic Aid for the Design of Sewer Networks. Journal of the Sanitary Engineering Division, American Society of Civil Engineers, 93(SA4):81-90, Proc. Paper 5381, August 1967.

Lindholm, O. G. Modeling of Sewerage Systems. Paper Presented at the NATO Advanced Study Institute on Systems Analysis for Environmental Pollution Control, Baiersbronn-Tonbach, Germany, December 9-16, 1972.

Lindholm, O. G. Factors Affecting the Performance of Combined Versus Separate Sewer Systems. Pergamon Press Ltd., Proceedings of 7th International Conference on Water Pollution Research, Paris, France, September 9-13, 1974.

Linsley, R. K. A Critical Review of Currently Available Hydrologic Models for Analysis of Urban Stormwater Runoff. Hydrocomp International, Inc., Palo Alto, California, for Office of Water Resources Research, August 1971.

Liou, E. Y. OPSET: Program for Computerized Selection of Watershed Parameter Values for the Stanford Watershed Model. University of Kentucky, Lexington, Research Report No. 34, 1970.

McDonnell Douglas Automation Co. ICES SEWER, Storm Sanitary Design/Analysis System. McDonnell Douglas Automation Co., St. Louis, Missouri, Report No. MDC N0138-031, undated (1973?).

McLaughlin, R. T. Combining Optimization with Surface-Water Flow. Proceedings of the International Symposium on Mathematical Modeling Techniques in Water Resources Systems, Department of Environment, Ottawa, Canada, May 1972.

Mallory, T. W., and C. P. Leiser. Control of Combined Sewer Overflow Events. Paper Presented at the 1973 International Public Works Congress and Equipment Show, Denver, Colorado, September 1973.

Merritt, L. B. Optimizing the Design of Urban Wastewater Collection Systems. University of Washington, Seattle, Department of Sanitary and Municipal Engineering, Ph.D. Thesis, 1971.

Merritt, L. B., and R. H. Bogan. Computer-based Optimal Design of Sewer Systems. Journal of the Environmental Engineering Division, American Society of Civil Engineers, 99(EE1):35-53, Proc. Paper 9578, February 1973.

Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc. Storm Water Management Model. U.S. Environmental Protection Agency Report 11024 DOC 07/71, 4 Volumes, October 1971.

Mevius, F. Analysis of Urban Sewer Systems by Hydrograph-Volume Method. Paper Presented at the National Conference on Urban Engineering Terrain Problems, Montreal, Canada, May 1973.

Minneapolis-St. Paul Sanitary District. Dispatching System for Control of Combined Sewer Losses. U.S. Environmental Protection Agency Report 11020 FAQ 03/71, March 1971.

Monro, J. C. Direct Search Optimization in Mathematical Modeling and a Watershed Model Application. National Oceanic and Atmospheric Administration, Technical Memorandum NWS HYDRO-12, April 1971.

Municipality of Metropolitan Seattle. Maximizing Storage in Combined Sewer Systems. U.S. Environmental Protection Agency Report 11022 ELK 12/71, December 1971.

Narayana, V. V. D., J. P. Riley, and E. K. Israelsen. Analog Computer Simulation of the Runoff Characteristics of an Urban Watershed. Utah State University, Logan, Utah Water Research Laboratory, January 1969.

Normand, D. Etude Experimentale du Ruissellement Urbain. Societe Hydrotechnique de France, XImes Journees de l'Hydraulique, Paris, France, Question II, Rapport 1, 1970.

Omnidata Services. SEWNET. Omnidata Services, Inc., New York, N. Y., Undated (1973?).

O'Neel, W. G., A. L. Davis and K. W. VanDusen. SAM - Wastewater Collection System Analysis Model - User's Manual. CH2M-Hill, Corvallis, Oregon, to the City of Portland, Oregon, March 1974.

Onstad, C. A., and D. G. Jamieson. Subsurface Flow Regimes of a Hydrologic Watershed Model. Proceedings of 2nd Seepage Symposium, U.S. Department of Agriculture, Report ARS 41-147, p. 46-55, 1968.

Papadakis, C., and H. C. Preul. University of Cincinnati Urban Runoff Model. Journal of the Hydraulics Division, American Society of Civil Engineers, 98(HY10):1789-1804, Proc. Paper 9298, October 1972. Discussion: 99(HY7): 1194-1196, July 1973. Closure: 100(HY4):608-611, April 1974.

Papadakis, C. N., and H. C. Preul. Infiltration and Antecedent Precipitation. Journal of the Hydraulics Division, American Society of Engineers, 99(HY8): 1235-1245, Proc. Paper 9940, August 1973a. Discussion: 100(HY5):707-708, May 1974.

Papadakis, C. N., and H. C. Preul. Testing of Methods for Determination of Urban Runoff. Journal of the Hydraulics Division, American Society of Civil Engineers, 99(HY9):1319-1335, Proc. Paper 9987, September 1973b. Discussion: 100(HY7):1081-1082, July 1974.

Pew, K. A., R. L. Callery, A. Brandstetter, and J. J. Anderson. The Design and Operation of a Real-Time Data Acquisition System and Combined Sewer Controls in the City of Cleveland, Ohio. Paper Presented at the Annual Conference, Water Pollution Control Federation, Atlanta, Georgia, October 1972.

Preul, H. C., and C. N. Papadakis. University of Cincinnati Urban Runoff (UCUR) Model--User's Manual. University of Cincinnati, Ohio, Department of Civil Engineering, October 1973.

Ray, D. L. Simulation of Control Alternatives for Combined Sewer Overflows. University of Massachusetts, Amherst, Department of Civil Engineering, Report No. EVE 33-73-4, April 1973.

Resource Analysis. Analysis of Hypothetical Catchments and Pipes with the M.I.T. Catchment Model. Resource Analysis, Inc., Cambridge, Massachusetts, for Battelle-Pacific Northwest Laboratories, 2 Volumes, October 1974.

Ritter, F. G., and G. Warg. Upgrading City Sewer Installations. Engineering Digest, April 1971.

Rockwood, D. M. Columbia Basin Streamflow Routing by Computer. Journal of the Waterways and Harbors Division, American Society of Civil Engineers, Vol. 84, pp. 1-15, December 1958.

Rockwood, D. M. Application of Streamflow Synthesis and Reservoir Regulation--"SSARR"--Program to the Lower Mekong River. International Association of Scientific Hydrology, Publ. No. 80, The Use of Analog and Digital Computers in Hydrology, pp. 329-344, December 1968.

Roesner, L. A., H. M. Nichandros, R. P. Shubinski, A. D. Feldman, J. W. Abbott, and A. O. Friedland. A Model for Evaluating Runoff-Quality in Metropolitan Master Planning. American Society of Civil Engineers Urban Water Resources Research Program, Technical Memorandum No. 23, April 1974.

Ross, G. A. The Stanford Watershed Model: The Correlation of Parameter Values Selected by a Computerized Procedure with Measurable Physical Characteristics of the Watershed. University of Kentucky, Lexington, Water Resources Institute, Research Report No. 35, for the Office of Water Resources Research, 1970.

Schaake, J. C., Jr., G. LeClerc, and B. M. Harley. Evaluation and Control of Urban Runoff. American Society of Civil Engineers Annual and National Environmental Engineering Meeting, Preprint 2103, New York, N.Y., October/November 1973.

Schermerhorn, V. P., and D. W. Kuehl. Operational Streamflow Forecasting with the SSARR Model. International Association of Scientific Hydrology, Publ. No. 80, The Use of Analog and Digital Computers in Hydrology, pp. 317-328, December 1968.

Sevuk, A. S. Unsteady Flow in Sewer Networks. University of Illinois, Urbana-Champaign, Department of Civil Engineering, Ph.D. Thesis, 1973.

Sevuk, A. S., B. C. Yen, and G. E. Peterson. Illinois Storm Sewer System Simulation Model: User's Manual. University of Illinois, Urbana-Champaign, Water Resources Center, Research Report No. 73, October 1973.

Sevuk, A. S., and B. C. Yen. Comparison of Four Approaches in Routing Flood Wave Through Junction. International Association for Hydraulic Research, Proceedings 15th Congress, Vol. 5, pp. 169-172, 1973a.

Sevuk, A. S., and B. C. Yen. A Comparative Study on Flood Routing Computation. International Association for Hydraulic Research, Proceedings International Symposium on River Mechanics, Bangkok, Thailand, Vol. 3, pp. 275-290, January 1973b.

Sharon, J. D., and J. A. Gutzwiller. Verification and Testing of the EPA Storm Water Management Model. University of Cincinnati, Department of Civil Engineering, M.S. Research Report, 1972.

Shubinski, R. P., J. C. McCarty, and M. R. Lindorf. Computer Simulation of Estuarial Networks. Journal of the Hydraulics Division, American Society of Civil Engineers, 91(HY5):33-49, Proc. Paper 4470, September 1965. Discussion: 92(HY3):96, May 1966. Closure: 93(HY1):68, January 1967.

Shubinski, R. P., and L. A. Roesner. Linked Process Routing Models. Paper Presented at American Geophysical Union Annual Spring Meeting, Washington, D. C., April 1973.

Smith, G. F. Adaptation of the E.P.A. Storm Water Management Model for Use in Preliminary Planning for Control of Urban Storm Runoff. University of Florida, Gainesville, Department of Environmental Engineering Sciences, M.E. Thesis, April 1975.

Smith, G. L., N. S. Grigg, L. S. Tucker, and D. W. Hill. Metropolitan Water Intelligence System. Colorado State University, Fort Collins, Department of Civil Engineering, Completion Report--Phase 1, for Office of Water Resources Research, June 1972.

SOGREAH. Mathematical Model of Flow Simulation in Urban Sewerage Systems, CAREDAS Program. Societe Grenobloise d'Etudes et d'Applications Hydrauliques, Grenoble, France, undated (1973a).

SOGREAH. Modele Mathematique de Simulation des Ecoulements dans les Reseaux d'Assainissement Urbains, Programme Caredas. Societe Grenobloise d'Etudes et d'Applications Hydrauliques, Grenoble, France, Draft Report, April 1973b.

SOGREAH. Modele Mathematique de Simulation des Ecoulements dans les Reseaux d'Assainissement Urbains, Programme Hydrologiques. Societe Grenobloise d'Etudes et d'Applications Hydrauliques, Grenoble, France, October 1973c.

SOGREAH. Mathematical Flow Simulation Model for Urban Sewerage Systems, Caredas Program. Societe Grenobloise d'Etudes et d'Applications Hydrauliques, Grenoble, France, Partial Draft Report, April 1973. Translated from French by David Vetrano, December 1973d.

SOGREAH. Looped Sewer Model. Societe Grenobloise d'Etudes et d'Applications Hydrauliques, Grenoble, France, Unpublished Manuscript, 1973e.

Stall, J. B., and M. L. Terstriep. Storm Sewer Design--An Evaluation of the RRL Method. U.S. Environmental Protection Agency Report EPA-R2-068, October 1972.

Surkan, A. J. HYDRA: Dynamic Model for Urban Hydrologic Systems. University of Nebraska, Lincoln, Department of Computer Science, to U.S. Office of Water Resources Research, July 1973.

Surkan, A. J., and P. Kelton. Binary Tree Model Simulation of the Behavior of Urban Hydrologic Systems. International Journal of Systems Science Preprint, 1974.

Systems Control. Sewerplan. Systems Control, Inc., Palo Alto, California, Urban Systems Application Series No. 2, Promotional Folder, December 1972.

Tang, W. H., L. W. Mays, and B. C. Yen. Optimal Risk-Based Design of Storm Sewer Networks. Journal of the Environmental Engineering Division, American Society of Civil Engineers, 101(EE3):381-398, Proc. Paper 11360, June 1975.

Terstriep, M. L., and J. B. Stall. Urban Runoff by Road Research Laboratory Method. Journal of the Hydraulics Division, American Society of Civil Engineers, 95(HY6): 1809-1834, Proc. Paper 6878, November 1969. Discussions: 96(HY4):1100-1102, April 1970; 96(HY7):1625-1631, July 1970; 96(HY9):1879-1880, September 1970. Closure: 97(HY4):574-579, April 1971.

Terstriep, M. L., and J. B. Stall. The Illinois Urban Drainage Area Simulator. Illinois State Water Survey, Bulletin 58, 1974.

Tholin, A. L., and C. J. Keifer. The Hydrology of Urban Runoff. Journal of the Sanitary Engineering Division, American Society of Civil Engineers, 85(SA2): 47-106, Proc. Paper 1984, March 1959. Discussions: 85(HY8):119, August 1959; 85(SA5):37-51, September 1959. Closure: 86(SA2):112, March 1960.

Tucker, L. S. Oakdale Gaging Installation, Chicago--Instrumentation and Data. American Society of Civil Engineers, Urban Water Resources Research Program, Technical Memorandum No. 2, August 1968.

Tucker, L. S. Control of Combined Sewer Overflows in Minneapolis-St. Paul. Colorado State University, Fort Collins, Department of Civil Engineering, Metropolitan Water Intelligance Systems Project Technical Report No. 3, October 1971.

University of Cincinnati. Urban Runoff Characteristics. U.S. Environmental Protection Agency Report 11024 DQU 10/70, October 1970.

University of Cincinnati. Urban Runoff Characteristics: Volume I-- Analytical Studies, Volume II--Field Data. U.S. Environmental Protection Agency Draft Report 11024 DQU 10/72, October 1972.

U.S. Corps of Engineers. Snow Hydrology, Summary Report of the Snow Investigations. North Pacific Division, Portland, Oregon, June 1956.

U.S. Corps of Engineers. Runoff from Snowmelt. Washington, D. C., Manual EM-1110-2-1406, January 1960.

U.S. Corps of Engineers. HEC-3, Reservoir System Analysis. U.S. Army Engineer District, Sacramento, California, Hydrologic Engineering Center Computer Program 23-X6-L253, December 1968.

U.S. Corps of Engineers. HEC-1, Flood Hydrograph Package. U.S. Army Engineer District, Sacramento, California, Hydrologic Engineering Center Computer Program 23-X6-L270, March 1969.

U.S. Corps of Engineers. Urban Runoff: Storage, Treatment and Overflow Model "STORM". U.S. Army, Davis, California Hydrologic Engineering Center Computer Program 723-S8-L2520, May 1974.

Waddel, W. W., R. G. Baca, C. R. Cole, A. Brandstetter, and K. D. Feigner. A Dynamic Hydraulic and Water Quality Model for River Basins. Paper Presented at National Water Resources Engineering Meeting, American Society of Civil Engineers, Washington, D. C., January 1973.

Walsh, S., and L. C. Brown. Least Cost Method for Sewer Design. Journal of the Environmental Engineering Division, American Society of Civil Engineers, 99(EE3):333-345, Proc. Paper 9796, June 1973.

Watkins, L. H. The Design of Urban Sewer Systems. Department of Scientific and Industrial Research, London, England, Road Research Technical Paper 55, 1962.

Watt, W. E. QUURM - Queen's University Urban Runoff Model. Queen's University at Kingston, Canada, Department of Civil Engineering, Unpublished Manuscript, May 1975.

Williams, J. R., and R. W. Hann. HYMO: Problem-Oriented Computer Language for Hydrologic Modeling. U.S. Agricultural Research Service, Report ARS-S-9, Mary 1973.

Wood, E. F., B. M. Harley, and F. E. Perkins. Operational Characteristics of a Numerical Solution for the Simulation of Open Channel Flow. Massachusetts Institute of Technology, Cambridge, Ralph M. Parsons Laboratory for Water Resources and Hydrodynamics, Report No. 150, June 1972.

Yen, B. C. Methodologies for Flow Prediction in Urban Storm Drainage Systems. University of Illinois, Urbana-Champaign, Water Resources Center, Research Report No. 72, September 1973a.

Yen, B. C. Open-Channel Flow Equations Revisited. Journal of the Engineering Mechanics Division, American Society of Civil Engineers, 99(EM5):979-1009, Proc. Paper 10073, October 1973b.

Yevjevich, V., and A. H. Barnes. Flood Routing through Storm Drains. Colorado State University, Fort Collins, Hydrology Papers 43, 44, 45, and 46, November 1970.

Young, C. P., and J. Prudhoe. The Estimation of Flood Flows From Natural Catchments. Department of the Environment, Transport and Road Research Laboratory, Crowthorne, Berkshire, England, TRRL Report LR 565, 1973.

SECTION XI

GLOSSARY

Advective transport - mass transport resulting from average velocity component of flow only, neglecting turbulent dispersion and molecular diffusion.

Backwater - the increased depth of water upstream from a dam or obstruction in a stream channel due to the existence of such obstruction, and the raising by it of the water level some distance upstream.

Binary - the representation of a numerical quantity by use of the two digits 0 and 1.

Bit - abbreviation of binary digit: smallest possible unit of computer storage.

BOD - biochemical oxygen demand, a standard test used in assessing wastewater strength; the quantity of oxygen used in the biological-chemical oxidation of organic matter in a specified time under standard conditions.

Branch network - network with tree type structure without closed loops.

Byte - a sequence of adjacent binary digits operated as a unit.

Catchment - the area tributary to a lake, stream or drain; also called drainage area, catchment area, watershed.

Closed conduit - any closed artificial or natural duct for conveying liquids or possibly other fluids.

Coliform - bacteria found in intestines of mammals, therefore used as an indication of the pollution level of a sample or body of water.

Combined sewer - a sewer receiving both intercepted surface runoff and municipal sewage.

Conservative pollutant - pollutant whose total mass does not change in the system under consideration.

Core - computer storage area where binary data is represented by the direction of the magnetic field in each unit of an array of tiny donut-shaped rings (generally).

Crown - the ceiling, top, of highest portion of the internal cross section of a closed conduit.

Dispersion - the mixing of a pollutant with a large volume of water in a stream or other body of water.

Diversion structure - a structure that may be closed to divert flow from the channel in which it is located to some other channel; also called regulator.

DO - dissolved oxygen, the amount of gaseous oxygen dissolved into a liquid sample.

Dry-weather flow - the flow of wastewater in a combined sewer during dry weather; such flow consists mainly of wastewater, with no stormwater included.

Dynamic regulator - a regulator which automatically makes adjustments in control settings by responding to water levels in the combined sewer or interceptor.

Dynamic wave - open channel flow defined as a function of water depth, water surface slope, advective acceleration and local acceleration.

Evapotranspiration - water withdrawn from soil by evaporation and plant transpiration; also called consumptive use.

First flush - heavy load of material, previously settled in sewers, which is washed along by the initial flow resulting from a storm.

Fixed regulator - a regulator which has no moving parts.

Froude number - a numerical quantity used as an index to characterize the type of flow in a hydraulic structure that has the force of gravity (as the only force producing motion) acting in conjunction with the resisting force of inertia; it is equal to the square of a characteristic velocity (the mean, surface, or maximum velocity) of the system, divided by the product of a characteristic linear dimension, such as diameter or depth, and the gravity constant or acceleration due to gravity - all expressed in consistent units so that the combinations will be dimensionless; the number is used in open-channel flow studies or in cases in which the free surface plays an essential role in influencing motion.

Hydrograph - a graph showing the discharge of water with respect to time for a given point for open-channel or closed conduit.

Hyetograph - a graph showing average rainfall, rainfall intensities or volume over specified areas with respect to time.

Infiltration - the entrance of water into the soil or other porous material through the interstices or pores of a soil or other porous medium; the water entering a sewer system and service connections from the ground, through such means as, but no limited to, defective pipes, pipe joints, connections, or manhole walls. Infiltration does not include, and is distinguished from, inflow.

Inflow - water discharged into a sewer system and service connections from such sources as roof gutters, cellars, yard and area drains, foundation drains, cooling water discharges, drains from springs and swampy areas, manhole covers, cross-connections from storm sewers and combined sewers, catchbasins, stormwaters, surface waters, street washwaters or drainages.

Inline storage or treatment - storage or treatment of wastewater within the sewer network.

Interceptor sewer - a sewer that generally receives most dry-weather flow and a portion of the combined flow from a number of transverse combined sewers and conveys such waters to a point for treatment.

Interface - a common boundary between parts of a computer system.

Invert - the floor, bottom or lowest portion of the internal cross section of a closed conduit; used particularly with reference to aqueducts, sewers, tunnels, and drains.

Inverted siphon - a pipeline crossing a depression or passing under a structure and having a reversal in grade on a portion of the line, thus creating a V- or U-shaped section of conduit; the line is under positive pressure from inlet to outlet and should not be confused with a siphon; also called depressed sewer.

Kinematic wave - open-channel flow defined as a function of water depth alone, neglecting the effects of the water surface slope, advective and local acceleration on wave celerity.

Leaping weir - a leaping weir is formed by a gap in the invert of a sewer; dry-weather flow falls through the gap to an intercepter, high flow with increased velocity leaps the gap and continues in the sewer, generally to an overflow outlet.

Looped network - channel and conduit network containing closed loops.

NH_4^+, NO_3^- - chemical symbol for ammonia and nitrate in solution.

Non-conservative pollutant - a pollutant whose total mass in the system under consideration changes as a result of decay or chemical-biological reactions.

Overflow storage or treatment - storage or treatment of overflow from high flow periods outside the sewer network.

Overland flow - the flow of water over the ground surface before it becomes channelized.

Perpendicular weir - a regulator whose weir crest is perpendicular to the direction of flow.

Plug flow - the passage of liquid through a chamber such that all increments of liquid move only in the direction of flow and at equal velocity.

PO_4^{--} - chemical symbol for phosphate in solution.

Pressure flow - full pipe flow with energy gradient or hydraulic gradeline above conduit crown.

Rainfall excess - that part of a rain of a given storm which falls at intensities exceeding the infiltration capacity and is thus available for direct runoff.

Rational method - a method of estimating the runoff in a drainage basin at a specific point and time by means of the rational runoff formula; for each drainage area, the rainfall rate under a stated intensity-duration relationship, the fraction that will appear as runoff, and the basin area above the specific point are estimated; their product is the flow; this method is used to estimate storm runoff in urban areas and flood flows in streams.

Real-time control - control of a system by using computers and timing such that the speed of response to the input information is fast enough to effectively influence the performance of that system; in a combined sewerage system, regulation of wastewater flows during rainstorms by changing regulator gate openings and weir heights to minimize overflows.

Regulator - a structure which controls the amount of sewage entering an interceptor by storing in a trunk line or diverting some portion of the flow to an outfall.

Retention - the prevention of runoff from entering the sewer system by storing on a surface area or in a storage basin.

Routing - computing the downstream outflow hydrograph of an open channel from known values of upstream inflow.

Sanitary sewer - a sewer that carries liquid and water-carried wastes from residences, commercial buildings, industrial plants, and institutions, together with relatively low quantities of ground, storm, and surface waters that are not admitted intentionally.

Scour - the action of a flowing liquid as it lifts and carries away the material on the sides or bottom of a waterway, conduit, or pipeline.

Sedimentation - the process of subsidence and deposition of suspended matter carried by water, wastewater, or other liquids, by gravity; it is usually accomplished by reducing the velocity of the liquid below the point at which it can transport the suspended material; also called settling.

Settleable solids - That matter in wastewater which will not stay in suspension during a preselected settling period, such as one hour, but either settles to the bottom or floats to the top; in the Imhoff cone test, the volume of matter that settles to the bottom of the cone in one hour.

Sewer - a pipe or conduit generally closed, but normally not flowing full, for carrying sewage or other waste liquids.

Sewerage - system of piping, with appurtenances, for collecting and conveying wastewaters from source to discharge.

Sidespill weir - a regulator which is essentially a long slot cut into the side of a sewer; normal dry-weather flow continues through the sewer while the increased depth during a storm will allow excessive flows to exit through the slot to some alternate point as an overflow.

Stage - the elevation of a water surface above its minimum or above or below an established low-water plane or datum of reference.

Static regulator - a regulator which is either fixed or can be adjusted only by manual actions of a human operator.

Storm sewer - a sewer that carries intercepted surface runoff, street wash and other surface drainage, but excludes domestic sewage and industrial wastes.

Surface runoff - precipitation that falls onto the surfaces of roofs, streets, ground, etc., and is not absorbed or retained by that surface, thereby collecting and running off.

Surcharge - the flow condition occurring in closed conduits when the hydraulic grade line is above the conduit crown; or the transition from open channel to pressure flow.

Suspended solids - Solids that either float on the surface of, or are in suspension in, water, wastewater, or other liquids, and which are largely removable by laboratory filtering; the quantity of material removed from wastewater in a laboratory test, as prescribed in "Standard Methods for the Examination of Water and Wastewater" and referred to as nonfilterable residue.

Synthetic unit hydrograph - a unit hydrograph developed for an ungaged drainage area, based on known physical characteristics of the basin.

Thiessen polygon - the points of location of raingages on a map are joined by straight lines and their perpendicular bisector drawn; the polygon formed around each raingage station by these perpendiculars is called, after its originator, a Thiessen polygon; the drainage area within a polygon is assumed to receive the same precipitation as the raingage at the center of the polygon.

Trunk sewer - a sewer that receives many tributary branches and serves a large territory.

Unit hydrograph - the hydrograph of storm runoff at a given point on a given stream which will result from an isolated rainfall excess of unit duration occurring over the contributing drainage area and resulting in a unit of runoff; also called unitgraph.

Unit treatment process - component of wastewater treatment facility, e.g., screens, trickling filter, etc.

Wastewater - the spent water of a community, including domestic, industrial and commercial water carrying solids plus storm or other water sources; replacing the term sewage.

Wave - a rise in open-channel flow to a crest in response to runoff generated by precipitation and its subsequent recession after the cessation of precipitation; the crest moves downstream at a velocity (wave celerity) different from the velocity of flow.

Wave celerity - the velocity of propagation of a wave through a fluid medium relative to the undisturbed velocity of the fluid through which the disturbance is propagated.

Word - a set of 16 or more bits stored and transferred as a unit by the computer.

SECTION XII

APPENDICES

		Page
A	List of Sources of Computer Programs	385
B	English to Metric Units Conversion Table	393
C	Mathematical Formulations of Tested Models	394
D	Mathematical Symbols for Tested Models	463
E	Additional Model Test Results.	475
F	Selected Computer Input and Output	509 and Vol. 2

APPENDIX A

LIST OF SOURCES OF COMPUTER PROGRAMS

Battelle Urban Wastewater Management Model

Dr. Albin Brandstetter
Research Associate
Environmental Management Section
Battelle-Northwest
P. O. Box 999
Richland, Washington 99352
Telephone (509) 946-2412

For IBM version:

Mr. David J. Anderson
Vice President
Watermation, Inc.
2304 University Avenue
St. Paul, Minnesota 55114
Telephone (216) 687-0220

Mr. Kenneth A. Pew
Chief, Control Systems Group
Department of Public Utilities
City of Cleveland
3090 Broadway Avenue
Cleveland, Ohio 44115
Telephone (216) 641-6000

British Road Research Laboratory Model

Mr. Harry C. Torno, P. E.
Staff Engineer
Environmental Protection Agency
Office of Research and Development
Municipal Pollution Control Division
Washington, D. C. 20460
Telephone (202) 426-8553

Chicago Flow Simulation Model

Mr. Richard Lanyon, P. E.
Supervising Engineer of Flood Control

Mr. Frank Kudrna
Engineer of Flood Control
Metropolitan Sanitary District of Greater Chicago
100 East Erie Street
Chicago, Illinois 60611
Telephone (312) 751-5600

Chicago Hydrograph Method and Chicago Runoff and Pollution Model

Mr. Louis Koncza
Acting Chief Engineer
City of Chicago Department of Public Works
Bureau of Engineering
320 North Clark Street
Chicago, Illinois 60610
Telephone (312) 744-3544

CH2M-Hill Wastewater Collection System Analysis Model

Dr. Allen L. Davis
CH2M-Hill
1600 S. W. Western Boulevard
P. O. Box 428
Corvallis, Oregon 97330
Telephone (503) 752-4271

Colorado State University Urban Runoff Modeling

Hydrologic and hydraulic modeling:

Dr. V. Yevjevich
Professor
Telephone (303) 491-8651

Dr. A. H. Barnes
Associate Professor
Telephone (303) 491-6767 or (303) 491-8640
Department of Civil Engineering
Colorado State University
Fort Collins, Colorado 80521

Real-time control investigations:

Dr. Neil S. Grigg
Associate Professor
Telephone (303) 491-8649
Department of Civil Engineering
Colorado State University
Fort Collins, Colorado 80521

Corps of Engineers STORM Model

Mr. Jesse Abbott
Research Hydraulic Engineer
The Hydraulic Engineering Center
Sacramento District
Corps of Engineers
609 Second Street
Davis, California 95616
Telephone (916) 449-2329 or (916) 449-3286

Dorsch Consult Hydrograph-Volume Method and Quantity-Quality Simulation Program

Mr. Helmut R. Dorsch
Managing Partner
Dorsch Consult
Ingenieurgesellschaft GmbH
D-8000 Munchen 21
Postfach 210243
Elsenheimerstrasse 63
Germany
Telephone (089) 5797-1

Mr. Fritz Mevius, P. E.
Chief Engineer
Dorsch Consult, Ltd.
Consulting Engineers
45 Richmond St. West
Suite 1203
Toronto 110, Ontario
Canada M5H 1Z2
Telephone (416) 368-7030 or (416) 368-7039

Environmental Protection Agency Stormwater Management Model

Mr. Harry C. Torno, P. E.
Staff Engineer
Environmental Protection Agency
Office of Research and Development
Municipal Pollution Control Division
Washington, D. C. 20460
Telephone (202) 426-8553

Dr. Wayne C. Huber
Associate Professor
Department of Environmental Engineering
University of Florida
Gainesville, Florida
Telephone (904) 392-0846

Hydrocomp Simulation Program

Dr. Norman Crawford
President
Hydrocomp International, Inc.
1502 Page Mill Road
Palo Alto, California 90430
Telephone (415) 493-5522

Illinois State Water Survey Urban Drainage Simulator

Mr. John B. Stall
Head, Hydrology Section
Telephone (217) 333-2210

Mr. Michael L. Terstriep
Associate Engineer
Telephone (217) 333-4959
Illinois State Water Survey
Box 232
Urbana, Illinois 61801

Massachusetts Institute of Technology Urban Watershed Model

Dr. Brendan M. Harley
President
Resource Analysis, Inc.
1050 Massachusetts Avenue
Cambridge, Massachusetts 02138
Telephone (617) 354-1922

Minneapolis-St. Paul Urban Runoff Model

Mr. David J. Anderson
Vice President
Watermation, Inc.
2304 University Avenue
St. Paul, Minnesota 55114
Telephone (216) 687-0220

Norwegian Institute for Water Research Sewerage System Models

Mr. Oddvar G. Lindholm
Norwegian Institute for Water Research
Gaustadalleen 25
P. O. Box 260 Blindern
Oslo 3
Norway
Telephone 23 52 80 or 46 69 60

Queen's University Urban Runoff Model

Dr. W. E. Watt
Professor
Department of Civil Engineering
Ellis Hall
Queen's University
Kingston
Canada K7L 3N6
Telephone (613) 547-6184

Seattle Computer Augmented Treatment and Disposal System

Mr. Curtis P. Leiser
Manager of Computer Services
Municipality of Metropolitan Seattle
410 West Harrison Street
Seattle, Washington 98199
Telephone (206) 284-5100

Mr. Stuart M. Alexander
Associate
R. W. Beck and Associates
200 Tower Building
Seattle, Washington 98102
Telephone (206) 622-5000

SOGREAH Looped Sewer Model

Mr. G. Chevereau or Dr. Jean Cunge
SOGREAH Ingenieurs Conseils
BP 172
Centre De Tri/38042
Grenoble Cedrex
France
Telephone (76) 09.80.22

SOGREAH, Inc.
375 Park Avenue
New York, New York 10022
Telephone (212) 758-8333

Lasalle Hydraulic Laboratory
0250 St. Patrick Street
Lasalle, Province of Quebec
Canada
Telephone (514) 366-2970 or (514) 366-2464

University of Cincinnati Urban Runoff Model

Dr. Constantine N. Papadakis
Hydraulic Engineer
Department of Civil Engineering
University of Michigan
Ann Arbor, Michigan 48104
Telephone (313) 764-4280 or (313) 764-4303

Dr. Herbert C. Preul
Professor
University of Cincinnati
Department of Civil and Environmental Engineering
Cincinnati, Ohio 45221
Telephone (513) 475-2634 or (513) 475-3648

University of Illinois Storm Sewer System Simulation Model

Dr. Ven Te Chow
Professor
Telephone (217) 333-0107

Dr. Ben C. Yen
Associate Professor
Telephone (217) 333-4934
Department of Civil Engineering
University of Illinois
Urbana, Illinois 61801

University of Massachusetts Combined Sewer Control Simulation Model

Dr. Donald D. Adrian
Professor
Department of Civil Engineering
University of Massachusetts
Amherst, Massachusetts 01002
Telephone (413) 545-2508

University of Nebraska Urban Hydrologic Simulator

Dr. Alvin J. Surkan
Department of Computer Science
University of Nebraska
Lincoln, Nebraska 68508

Water Resources Engineers Storm Water Management Model

Dr. Larry Roesner
Principal Engineer
Water Resources Engineers, Inc.
710 South Broadway
Walnut Creek, California 94596
Telephone (415) 933-4500

Mr. Harold Coffee
Civil Engineer
Department of Public Works
City of San Francisco
770 Golden Gate Avenue
San Francisco, California 94102
Telephone (415) 558-2131

Mr. T. Clark Lyons
F. H. Kocks KG
Büro Hamburg
D-2000 Hamburg 76
Pappelallee 28
Germany
Telephone (0411) 20 69 42

Wilsey and Ham Urban Watershed Model

Mr. James H. Iverson
Director of Systems Analysis
Wilsey and Ham
1035 East Hillsdale Boulevard
Foster City, California 94404
Telephone (415) 349-2151

APPENDIX B

ENGLISH TO METRIC UNITS CONVERSION TABLE

English unit	Abbr.	Multiplier	Abbr.	Metric unit
acre	acre	0.4047	ha	hectare
acre-foot	acre-ft	1,233.5	m^3	cubic meter
cubic foot	ft^3	28.32	l	liter
cubic feet per second	cfs	0.02832	m^3/sec	cubic meters per second
cubic inch	in^3	16.39	cm^3	cubic centimeter
cubic yard	yd^3	0.7646	m^3	cubic meter
degree Fahrenheit	°F	(°F-32)/1.8	°C	degree Celsius
feet per minute	fpm	0.005080	m/sec	meters per second
feet per second	fps	0.3048	m/sec	meters per second
foot (feet)	ft	0.3048	m	meter(s)
gallon(s) (U.S. liquid)	gal.	3.785	l	liter(s)
gallons per minute	gpm	0.06309	l/sec	liters per second
inch(es)	in.	2.540	cm	centimeter
inches per hour	in./hr	2.540	cm/hr	centimeters per hour
million gallons per day	mgd	0.04381	m^3/sec	cubic meters per second
mile	mi	1.609	km	kilometer
miles per hour	mph	1.609	km/hr	kilometers per hour
pound(s)	lb	0.4536	kg	kilogram
square foot	ft^2	0.09290	m^2	square meter
square inch	in^2	6.452	cm^2	square centimeter
square mile	mi^2	2.590	km^2	square kilometer
square yard	yd^2	0.8361	m^2	square meter
ton (short)	ton	0.9072	metric ton	metric ton
yard	yd	0.9144	m	meter

CONVERSIONS OF CONCENTRATION x DISCHARGE TO MASS FLOW RATE

Define:

C = pollutant concentration

Q = wastewater discharge

M = pollutant mass flow rate

Then:

M[lb/hr] = 0.224 741 x C[mg/l] x Q[cfs]

M[kg/hr] = 0.101 941 x C[mg/l] x Q[cfs]

M[lb/hr] = 7.936 641 x C[mg/l] x Q[m^3/sec]

M[kg/hr] = 3.600 000 x C[mg/l] x Q[m^3/sec]

APPENDIX C

MATHEMATICAL FORMULATIONS OF TESTED MODELS

CONTENTS

	Page
Introduction	397
Battelle Urban Wastewater Management Model	397
Summary	397
Watershed Representation	398
Losses on Impervious Areas	398
Losses on Pervious Areas	399
Stormwater Runoff	401
Dry-Weather Runoff	402
Combined Catchment Runoff	402
Conduit Flow Routing	404
Diversion Structures	407
Storage Facilities	411
Water Quality	411
Chicago Flow Simulation Program	412
Summary	412
Watershed Representation	412
Snowmelt	413
Runoff from Pervious Areas of Nonsewered Catchments	414
Runoff from Impervious Areas of Nonsewered Catchments	416

CONTENTS (Continued)

Page

Runoff from Pervious and Impervious Areas of Sewered Catchments 417

Channel and Conduit Flow Routing 417

Storage Facilities 419

Dorsch Consult Hydrograph-Volume Method 419

Summary 419

Catchment Surface Runoff 420

Channel and Conduit Flow Routing 421

Environmental Protection Agency Stormwater Management Model 423

Summary 423

Watershed Representation 424

Stormwater Runoff 424

Gutter Flows 426

Dry-Weather Runoff 427

Infiltration into Sewers 428

Channel and Conduit Flow Routing 429

Diversion Structures 433

Pumping Stations 434

Storage Facilities 434

Water Quality 435

Massachusetts Institute of Technology Urban Watershed Model 442

Summary 442

CONTENTS (Continued)

Page

Watershed Representation 442

Stormwater Runoff 442

Conduit and Open Channel Flow Routing 444

SOGREAH Looped Sewer Model 447

Summary 447

Catchment Runoff 447

Conduit and Open Channel Flow Routing 450

Water Resources Engineers Stormwater Management Model 451

Summary 451

Watershed Representation 451

Catchment Runoff 452

Conduit and Open Channel Flow Routing 453

Flow Control Devices 457

Water Quality 459

INTRODUCTION

Brief descriptions and the principal equations are presented for each of the seven models which were tested with hypothetical and real catchment data. The descriptions are intended primarily as a quick reference to the main equations and assumptions of each model, so the reader will not have to wade through a maze of theoretical developments and derivations to gain a basic understanding of the models' capabilities. The original model documentation should be consulted, however, for detailed derivations and justifications of model assumptions.

Equations are given for the catchment and sewerage system flow and water quality computations if available in model documents. The inclusion of equations for other model features, such as receiving water flow and quality simulation, and real-time and design optimization was beyond the scope of this study.

Complete mathematical formulations were not available for some proprietary models. Consequently only the available equations are presented, and the potential user is advised to contact the model developer for additional details.

Consistent nomenclature is used for the equations of all models to facilitate model comparisons. The selected mathematical symbols and their definitions are listed in Appendix D. They are also defined following their first occurrence in each model description.

BATTELLE URBAN WASTEWATER MANAGEMENT MODEL

Summary

The Battelle Urban Wastewater Management Model (Brandstetter et al., 1973) is intended primarily for the simulation of major sewer system components, such as trunk and interceptor sewers, regulators, overflow storage facilities, and treatment plants. It provides a means of evaluating the time-varying performance of a planned or existing sewerage system under a variety of rainfall conditions, considering both the time and spatial variations of rainfall rates without requiring the simulation of every small sewer, manhole, etc., in the system. Stormwater and sanitary runoff and quality of major catchment areas are computed using hydrologic lumping techniques to arrive at hydrographs and quality graphs at major regulators and at junctions of major trunks and interceptors. The runoff is then routed through trunk and interceptor sewers to the treatment plant. The model is limited to the simulation of single runoff events.

The model can simulate up to seven conservative wastewater quality parameters. Quality reactions, with the exception of changes at treatment facilities, are not modeled. The model determines the required diversion at control regulators during real-time rainstorm events in order to minimize wastewater discharges to receiving waters. The model can also be used for design and planning studies. It computes sizes and costs of structural sewer system modifications (such as sewers, storage and treatment facilities) which will result in the least-cost combination of alternatives for improving system performance.

Watershed Representation

Catchments are represented by their areas and the length of the main drainage channel; catchment shape does not enter the computations. Overland flow and flow routing in gutters and minor channels are lumped into a single computation. The model includes provisions to compute runoff from only major catchments and to increase the computed runoff in proportion to drainage area to account for runoff from adjacent small catchments not modeled separately. Cumulative rainfall can be provided at irregular time intervals for several raingages. Weighted rainfall intensities are computed for each subcatchment by the Thiessen polygon method.

Losses on Impervious Areas

Rainfall losses on impervious areas are assumed to depend on the rainfall, the antecedent moisture condition, the accumulated losses, and three coefficients. The accumulated loss represents the amount of moisture retained on the impervious surfaces and intercepted by trees. The first coefficient represents the accumulated loss which must be reached before any runoff will occur. Once the initial loss has been satisfied, the additional loss at each time step is assumed to decrease exponentially. The accumulated loss until the beginning of a time step is given by

$$V_i'' = \begin{cases} V_i \text{ at beginning of rain} \\ V_i' + f_i \text{ thereafter} \end{cases} \tag{1}$$

where

V_i = antecedent moisture condition, in.

f_i = rainfall loss on impervious area during interval Δt, in.

V_i', V_i'' = accumulated loss at beginning and end of time interval Δt, in.

The loss during the time interval Δt is then computed from

$$f_i = \begin{cases} r_t \text{ if } V_i' \leq h_i \\ r_t \; a \exp[-\, b \, V_i'] \text{ if } V_i' > h_i \end{cases} \tag{2}$$

where

h_i = minimum accumulated loss which must be reached before runoff will occur, in.

a,b = impervious loss rate coefficients

r_t = rainfall during time interval Δt, in.

The rainfall excess from the impervious area during the time interval Δt is then given by

$$r_i = r_t - f_i \tag{3}$$

where

r_i = rainfall excess on impervious areas during time interval Δt, in.

Losses on Pervious Areas

Rainfall losses from the pervious areas are assumed to depend on the rainfall, the antecedent moisture condition, the accumulated losses, the infiltration rate under dry conditions, the equilibrium infiltration rate, the total available moisture storage capacity in the soil, and a rate coefficient. As before, an initial loss must be satisfied before any runoff can occur. The accumulated loss until the beginning of a time step is given by

$$V_p'' = \begin{cases} V_p \text{ at beginning of rain} \\ V_p' + f_p \text{ thereafter} \end{cases} \tag{4}$$

where

V_p = antecedent moisture condition, in.

f_p = rainfall loss on pervious area during time interval Δt, in.

V_p', V_p'' = accumulated loss at beginning and end of time interval Δt, in.

The loss is then determined from

$$f_p = \begin{cases} r_t \text{ if } V_i' \leq h_p \text{ or } r_t > f \\ f \text{ if } V_i' > h_p \text{ and } r_t > f \end{cases} \tag{5}$$

where

h_p = minimum accumulated loss which must be satisfied before runoff will occur, in.

f = potential infiltration on pervious area during time interval Δt, in.

The potential loss is approximated by the Holtan infiltration equation as modified by Huggins and Monke:

$$f = \begin{cases} \left[f_e + (f_o - f_e) \left(\frac{s - V_p'}{s} \right)^{\alpha} \right] \frac{\Delta t}{60} \text{ if } V_p' < s \\ f_e \frac{\Delta t}{60} \text{ if } V_p' \geq s \end{cases} \tag{6}$$

where

f_e = equilibrium infiltration rate, in./hr

f_o = infiltration rate under dry conditions, in./hr

α = pervious infiltration rate coefficient

s = total available soil moisture storage capacity, in.

Δt = constant time interval, minutes

The rainfall excess from the pervious area during the time interval Δt is then given by

$$r_p = r_t - f_p \tag{7}$$

where

r_p = rainfall excess on pervious area during time interval Δt, in.

Stormwater Runoff

The combined rainfall excess of the catchment from both pervious and impervious areas during the time interval Δt is given by

$$r_e = p_i \, r_i + (1 - p_i) \, r_p \tag{8}$$

where

r_e = rainfall excess on catchment during time interval Δt, in.

p_i = fraction of impervious area of catchment drainage area, percent/100

Storm runoff is computed for each modeled catchment using the unit-hydrograph method. Triangular unit-hydrographs are derived from catchment characteristics (including catchment length, average catchment slope, and type of soil and soil cover) using the formulas of the U.S. Soil Conservation Service:

$$t_p = \frac{L_c^{0.8} \left[\left(1000/c - 10 \right) + 1 \right]^{0.7}}{31.67 \, S_c^{0.5}} \tag{9}$$

$$t_b = 2.67 \, t_p \tag{10}$$

$$U_p = \frac{2}{t_b} \tag{11}$$

where

c = SCS soil cover complex number

L_c = Length of main runoff channel, ft

S_c = average slope of catchment, percent

t_p = time to peak of unit hydrograph, minutes

t_b = base time of unit hydrograph, minutes

U_p = peak ordinate of unit hydrograph, minutes^{-1}

The rainfall excess is then convoluted with the unit hydrograph to derive the stormwater runoff hydrograph:

$$q_k = \sum_{j=1}^{m+1} U_j \, r_{k-j+1} \, \Delta t \tag{12}$$

where

m = length of memory of catchment ($m\Delta t = t_b$)

q_k = catchment stormwater runoff during k-th time interval, in.

U_j = unit hydrograph ordinate, minutes^{-1}

r_{k-j+1} = rainfall excess during (k-j+1)-th time interval, in.

The total stormwater runoff from a catchment at the end of the k-th time interval is then

$$q_s = 1.008 \, A_c \, q_k \, \frac{60}{\Delta t} = 60.48 \, A_c \, q_k/\Delta t \tag{13}$$

where

A_c = drainage area of catchment, acres

q_s = catchment stormwater runoff, cfs

Dry-Weather Runoff

Dry-weather flow rates are estimated from measurements in the modeled catchments or similar areas. Average dry-weather flows for each catchment are multiplied by hourly, daily, and seasonal factors to obtain hourly dry-weather flow rates for a given day of the week and season. The average flow and adjustment factors are specified by input data for each catchment. The hourly values are then interpolated linearly to obtain dry-weather flow rates at the selected time intervals corresponding to the stormwater runoff hydrographs.

Combined Catchment Runoff

The storm runoff and dry-weather flow hydrographs are then added to obtain the combined catchment runoff hydrograph:

$$q_c = q_d + q_s \tag{14}$$

where

q_d = dry-weather runoff from catchment, cfs

q_c = combined dry-weather and stormwater runoff from catchment, cfs

This combined hydrograph is then reduced by the overflows from small upstream regulators not modeled separately. The mathematical formulation is based on the clipping action of regulators on the peaks of the hydrograph:

$$q_c^* = \begin{cases} q_c \text{ if } q_c \leq \alpha\, \bar{q}_d \\ \alpha\bar{D} + \beta(q_c - \alpha\, \bar{q}_d) \text{ if } q_c > \alpha\, \bar{q}_d \end{cases} \tag{15}$$

where

α = regulator clipping factor

β = overflow reduction factor, ≤ 1

$\bar{q}_d$ = average dry-weather runoff rate, cfs

q_c^* = combined dry-weather and stormwater runoff from catchment, reduced by upstream overflows not modeled separately, cfs

The overflow reduction factor is approximated by

$$\beta = \frac{A_c - \sum_{j=1}^{n} A_j}{A_c} \tag{16}$$

where

n = number of upstream regulators not modeled separately

A_j = combined drainage areas tributary to regulators not modeled separately, acres

A_c = drainage area of modeled catchment, acres

To reduce data requirements and computational effort, only major catchments are modeled in the detail described above. The runoff from smaller catchments is lumped into the runoff from the adjacent major catchments in proportion to their drainage areas:

$$Q_c = k\, q_c^* \tag{17}$$

where

k = catchment lumping factor

Q_c = combined dry-weather and stormwater runoff from catchment, increased by runoff from adjacent catchments not modeled separately, cfs

The lumping factor is approximated by

$$k = \frac{A_c + \sum_{j=1}^{n} A_j}{A_c} \tag{18}$$

where

n = number of adjacent catchments not modeled separately

A_j = drainage areas of adjacent catchments not modeled separately, acres

A_c = drainage area of modeled catchment, acres

Conduit Flow Routing

The Battelle model is formulated for flow routing in circular closed conduits only. The flow routing uses a characteristic solution of the kinematic wave equation, which is defined by the

continuity equation:

$$\frac{\delta Q}{\delta x} + \frac{\delta A}{\delta t} = 0 \tag{19}$$

and Manning's equation:

$$Q = \frac{1.49}{n} A\, R^{2/3}\, S^{1/2} = \frac{1.49}{n} A^{5/3}\, P^{-2/3}\, S^{1/2} \tag{20}$$

where

Q = discharge, cfs

A = cross-sectional area, ft^2

x = distance, ft

t = time, sec

n = Manning's roughness coefficient

P = wetted perimeter, ft

$R = A/P$ = hydraulic radius, ft

S = invert slope, ft/ft

The following geometric relationships for circular pipes are introduced into the kinematic wave equation:

$$A = \frac{D^2}{8}\,\theta\,\left(1 - \frac{\sin\theta}{\theta}\right) \tag{21}$$

$$Y = \frac{D}{2}\left(1 - \cos\frac{\theta}{2}\right) \tag{22}$$

$$\cos\frac{\theta}{2} = 1 - 2Y/D \tag{23}$$

$$\theta = 2\cos^{-1}\left(1 - 2\,\frac{Y}{D}\right) \tag{24}$$

$$P = \frac{D}{2}\,\theta \tag{25}$$

$$R = \frac{A}{P} = \frac{D}{4}\left(1 - \frac{\sin\theta}{\theta}\right) \tag{26}$$

where

Y = depth of water, ft

θ = angle of pipe center with water surface, radians, $0 \leq \theta \leq 2\pi$

d = diameter, ft

This results in the following formulation of the kinematic wave equation for circular pipes:

continuity equation:

$$cF(\theta)\ \frac{\delta\theta}{\delta x} + \frac{\delta\theta}{\delta t} = 0 \tag{27}$$

where

$$c = \frac{0.197}{n}\ S^{1/2}\ D^{2/3} \tag{28}$$

$$F(\theta) = \left(1 - \frac{\sin\theta}{\theta}\right)^{2/3} \left(3 - 5\cos\theta + 2\frac{\sin\theta}{\theta}\right) \Big/ (1 - \cos\theta) \tag{29}$$

and Manning's equation:

$$Q = c'\ \theta\ \left(1 - \frac{\sin\theta}{\theta}\right)^{5/3} \tag{30}$$

where

$$c' = \frac{0.0737}{n}\ S^{1/2}\ D^{8/3} \tag{31}$$

The characteristic form of Equation (27) is

$$\frac{dx}{dt} = c\ F(\theta) \tag{32}$$

This equation is then solved by finite differences in the form

$$\Delta t = \frac{\Delta x}{c\ F(\theta)} \tag{33}$$

where

Δt = travel time of kinematic wave through conduit reach, sec

Δx = length of conduit reach, ft

The flow routing for a pipe reach starts with the upstream inflow hydrograph, specified at constant time intervals. Assuming uniform flow conditions, the angle θ is computed by the method of bisection from Equation 30. Equation 33 then gives the kinematic wave travel time to the downstream end of the pipe reach. Since this results in a hydrograph specified at irregular time intervals, it is interpolated linearly at constant time intervals to obtain the inflow hydrograph for the next downstream pipe reach.

This formulation neglects downstream flow control and cannot compute backwater, flow reversal, and surcharging. Warning statements are printed by the computer program if the computed flow depth exceeds 93 percent of the diameter.

Diversion Structures

The diversion equations are based on standard weir and orifice discharge equations and neglect downstream effects on the regulator discharge.

The discharge over a weir is assumed to follow the relationship

$$Q_w = \begin{cases} 0 \text{ if } Y \leq h_w \\ c_w L_w (Y-h_w)^{\alpha} \text{ if } Y > h_w \end{cases} \tag{34}$$

where

Q_w = weir discharge, cfs

c_w = weir discharge coefficient

Y = depth of flow above sewer invert upstream of weir, ft

h_w = height of weir crest above sewer invert, ft

L_w = length of weir crest, ft

α = weir equation exponent

The discharge through an orifice is assumed to follow the relationship

$$Q_o = \begin{cases} 0 \text{ if } Y \leq h_o \\ c_o A_o \sqrt{2g(Y-h_g/2-h_o)} \text{ if } Y > h_o \end{cases} \tag{35}$$

where

Q_o = orifice discharge, cfs

A_o = area of flow through the orifice, ft^2

c_o = orifice discharge coefficient

g = gravitational acceleration, 32.2 ft/sec^2

Y = depth of flow above sewer invert upstream of orifice, ft

h_o = height of orifice invert above sewer invert, ft

$$h_g = \begin{cases} \text{height of orifice opening if orifice submerged, ft} \\ \text{depth of flow above orifice invert if orifice not submerged, ft} \end{cases}$$

For a diversion structure with an orifice dry-weather outlet immediately upstream of a combined overflow weir, Equations 34 and 35 are solved simultaneously with the equation of continuity:

$$Q = Q_o + Q_w \tag{36}$$

where

Q = inflow rate to the diversion structure, cfs

Equations 34 to 36 have the three unknown variables: Y, Q_o, and Q_w. Elimination of H and Q_w results in the diversion equation

$$Q - Q_o - c_w L_w \left(\frac{Q_w^2}{2g\ c_o^2 A_o^2} + \frac{h_g}{2} + h_o - h_w \right)^{\alpha} = 0 \tag{37}$$

for $Y \geq h_w$.

This equation can be solved for Q_o in terms of the known inflow Q. Since the equation cannot be solved explicitly for Q_o, it is determined in the computer program by the method of bisection.

Three types of orifice shapes are modeled at present:

1. rectangular (a rectangular gate sliding over a rectangular opening),

2. segment of a circle (a rectangular gate sliding over a circular opening), and

3. crescent (a circular gate sliding over a circular opening).

The area of the orifice A_o is determined by the orifice opening if the depth of water is deeper than the top of the orifice, or by the depth of water itself if it is below the top of the orifice.

The area of the rectangular opening is simply

$$A_o = W_o \, h_g \tag{38}$$

where

W_o = width of rectangular orifice, ft

The area of a segment of a circle is

$$A_o = \frac{D_o^2}{8} (\theta - \sin \theta) \tag{39}$$

where

D_o = diameter of circular orifice, ft

θ = angle formed by center of circle and chord formed by top of gate or water surface, radians, $0 \leq \theta \leq 2\pi$

The angle θ is computed from

$$\theta = \begin{cases} 0 \text{ if } h_g \leq 0 \\ 2\pi \text{ if } h_g > D_o \\ 4 \arctan \dfrac{h_g}{D_o - h_g} \text{ otherwise} \end{cases} \tag{40}$$

This relationship is valid for all values of h_g between 0 and D_o (or all angles of θ between 0 and 2π).

The leaping weir consists of an opening in the invert of the combined sewer. Intercepted flow drops through the opening to enter the dry-weather outlet. At high flow rates, a portion of the flow leaps across the opening and continues to the stormwater outlet. At present it is assumed that no

overflow occurs until the flow in the pipe reaches the capacity of the sanitary outlet as given by the orifice equation

$$Q_{min} = c_o A_o \sqrt{2g(Y - h_o - D_o/2)} \tag{41}$$

where

Y = normal depth of flow in combined sewer, ft

h_o = depth of invert of sanitary outlet below invert of combined sewer, ft

Q_{min} = minimum combined flow rate for overflow to occur, cfs

and the remaining variables are defined as before.

The normal depth Y for the combined inflow Q to the leaping weir is computed from Manning's equation as described elsewhere. A_o is taken as the smaller of the areas of the sanitary outlet or combined invert opening.

The intercepted flow is then computed from

$$Q_w = \begin{cases} Q \text{ if } Q \leq Q \text{ min} \\ \\ Q - Q_{min} \text{ if } Q > Q_{min} \end{cases} \tag{42}$$

where

Q_w = intercepted flow through sanitary outlet, cfs

From continuity,

$$Q_o = Q - Q_w \tag{43}$$

where

Q_o = combined overflow, cfs

This is a very crude approximation of the leaping weir flow diversion and probably results in overestimates of the intercepted flows. It neglects the velocity of approach of the combined flow and the geometry of the opening in the combined sewer invert.

Control regulators encompass all types of diversion structures which have controllable gates and/or weirs that can be adjusted continuously or at periodic intervals to divert the incoming combined flow to the diversion structure. The percent diversion, that is the fraction of the inflow that is to be intercepted by the sanitary sewer, is determined by the control optimization. The real-time control optimization determines the amount of flow division at each control regulator for each time interval, so that the total mass of pollutant overflow from the entire sewerage system during a storm is minimized.

Storage Facilities

A storage facility can be specified anywhere on the overflow outlet of a diversion structure. In-line storage facilities are not simulated. Only the inflow and cumulative storage volume are computed. The real-time control optimization determines the optimal allocation of available storage space during the inflow period of a rainstorm but neglects the optimal emptying of the facility during or after the rainstorm.

Water Quality

Up to seven arbitrary conservative water quality constituents can be simulated. The quality of the dry-weather flow is specified by input data defining average concentrations and 24 hourly, seven daily, and up to six seasonal adjustment factors. The average value is multiplied by the hourly, daily, and seasonal adjustment factors to obtain a value for a given hour of a weekday and season, and then interpolated linearly between the values at the beginning and end of the hour to obtain the concentration for a given time step.

The stormwater quality is computed from the storm runoff by the linear regression equation

$$M = \begin{cases} 0 \text{ for } q_s \leq 0 \\ K_1 + K_2 q_s + K_3 V_s \text{ for } q_s > 0 \end{cases} \tag{44}$$

where

K_1, K_2, K_3 = empirical coefficients

q_s = stormwater runoff rate per unit catchment area at time t, cfs/acre

$$V_s = \int^t q_s(t)dt$$

= cumulative volume of storm runoff per unit area from beginning of runoff until time t, ft^3/acre

M = pollutant mass flow rate per unit area at time t, lb/hr/acre

The coefficient values have to be determined for each modeled pollutant by regression between measured storm runoff and quality.

The mass flow rate of each pollutant for the dry-weather and stormwater runoff are then added to obtain the combined inflow at each inlet. Mass flow routing in the sewers is accomplished by simple advection according to the kinematic wave travel time computed by Equation 33. Although this moves the pollutants faster than the actual mass travel time, this assumption was necessary to facilitate the optimization scheme for real-time control of overflow regulators. A pseudo-dispersion is attained by averaging consecutive mass flow rate values at the end of each sewer reach. At regulators, the mass flow rates are divided in proportion to the flow diversion. In storage facilities, the mass of each pollutant is simply accumulated. No other computations, such as chemical reactions or interactions, are performed. Wastewater treatment is computed independently for each pollutant by linear interpolation of tables relating discharge, concentration and percent removal of a pollutant.

CHICAGO FLOW SIMULATION PROGRAM

Summary

The Chicago Flow Simulation Program is intended primarily for the simulation of large catchments consisting of both sewered and nonsewered areas (Lanyon and Jackson, 1972 and 1974). It simulates the time-varying runoff in combined sewerage systems and nonurban drainage basins consisting of several catchments and a converging branch sewer and open channel network. The flow routing formulation for natural channels includes provisions for flow and storage in flood plains. The model can perform continuous simulation using hourly or shorter time steps. Water quality, real-time control, and design features are not included.

Watershed Representation

The watershed being modeled is defined by a system of open channel and closed conduit reaches. It is suggested that the

reach network of natural streams be defined by permanent and intermittent streams shown by blue lines on a USGS 7-1/2 minute quadrangle map. All forks in the channel network are considered ends of reaches and the maximum length of any single reach is restricted to 3 km (2 mi).

The model simulates a maximum of two upstream reaches to be connected directly to one downstream reach. Reaches are defined by the x and y coordinates of the upstream and downstream ends of open channels or closed conduits. The model was designed for catchments larger than 5 km^2 (2 mi^2).

Snowmelt

Snowmelt is computed by a method of the U.S. Bureau of Reclamation from precipitation, daily average air temperature and daily average wind velocity. The snowmelt computations assume a linear relationship between snowpack depth and accumulated water content:

$$s_y = 147.4 - 0.474p_w \tag{45}$$

where

s_y = snowpack depth in percent of initial depth

p_w = accumulated water content in percent of initial water content

It is assumed that drainage from the snow occurs only after the snowpack has reached a specified threshold density. The conditions of the snowpack at the adopted threshold density can be computed by the following three equations derived from Equation 45:

$$p_{wt} = 147.4d_{pt}/(\rho_{so} + 0.474\rho_{pt}) \tag{46}$$

$$s_{yt} = 147.4d_{so}/(\rho_{so} + 0.474\rho_{pt}) \tag{47}$$

$$\rho_{st} = 0.678\ (\rho_{so} + 0.474\rho_{pt}) \tag{48}$$

where

p_{wt} = threshold accumulated water content in percent of initial water content

s_{yt} = threshold depth in percent of initial depth

ρ_{so} = density of initial dry snowpack in percent

ρ_{pt} = threshold density of compacted wet snowpack in percent

ρ_{st} = threshold density of dry snow in compacted wet snowpack in percent

The procedure requires knowledge of the initial snow depth and density. For succeeding time steps, the precipitation in inches of water or inches of snow and its density must be known. The potential snowmelt during each time step is computed by the empirical relationships

$$s_m = (0.029 + 0.0084k_o u + 0.007r_d)(T_a - 32) + 0.09 \quad (49)$$

for open or partly forested areas (mean canopy cover less than 80 percent), and

$$s_m = (0.074 + 0.007r_d)(T_a - 32) + 0.05 \quad (50)$$

for heavily forested areas (mean canopy cover more than 80 percent),

where

s_m = total daily snowmelt, in.

k_o = empirical coefficient reflecting the relative exposure of the basin to wind

u = mean wind velocity at the 50-foot level, mph

r_d = total daily precipitation, in.

T_a = mean daily air temperature of saturated air at the 10-foot level, °F

Runoff from Pervious Areas of Nonsewered Catchments

Infiltration losses on pervious areas of nonsewered catchments are computed at 15-minute intervals by an empirical function of rainfall or snowmelt, catchment shape, and soil moisture:

$$f_j = \frac{K_1 A_p r_t}{4 L_c s} \quad (51)$$

where

f_j = the 15-minute infiltration, in.

r_t = hourly rainfall, in.

A_p = pervious drainage area, acres

K_1 = empirical coefficient

L_c = length of open channel reach of catchment, ft

s = soil moisture, in.

The change in soil moisture due to evapotranspiration and deep percolation is assumed to be proportional to the air temperature and the available soil moisture:

$$s'' = f_j + s' \left[1 + K_2 \left(1 + \frac{T_a - 50}{K_3}\right)\right] \tag{52}$$

where

T_a = average daily air temperature, °F

K_2, K_3 = empirical coefficients

s', s'' = soil moisture at beginning and end of 15-minute time step, in.

The total hourly infiltration is the sum of the four 15-minute infiltrations:

$$f_p = \sum_{j=1}^{4} f_j \tag{53}$$

where

f_p = hourly infiltration, in.

Linear storage routing is used to compute overland flow on the pervious areas of nonsewered catchments. The surface runoff is assumed to be proportional to the overland flow storage, ground slope, and flow distance and independent of the surface characteristics (roughness):

$$q_p = K_4 \, V_p \, S_c \, \frac{L_c}{A_p} \tag{54}$$

where

q_p = hourly surface runoff from pervious area of nonsewered catchment, in.

K_4 = empirical coefficient

V_p = overland flow storage on pervious areas of nonsewered catchment, in.

S_c = ground surface slope of catchment, ft/ft

The overland flow storage at every time step is defined by

$$V''_p = V'_p + r_t - f_p - q_p \tag{55}$$

where

V'_p, V''_p = overland flow storage on pervious areas of nonsewered catchment at beginning and end of hour, in.

Runoff from Impervious Areas of Nonsewered Catchments

No moisture losses are computed for impervious areas of nonsewered catchments. The surface runoff is assumed to be a fraction of the overland flow storage neglecting catchment shape, slope, and roughness:

$$q_i = K_5 \, V_i \tag{56}$$

where

q_i = hourly surface runoff from impervious area of nonsewered catchment, in.

K_5 = empirical coefficient

V_i = overland flow storage on impervious areas of nonsewered catchment, in.

The storage on the impervious surface at every time step is computed from

$$V''_i = V'_i + r_t - q_i \tag{57}$$

where

V_i', V_i'' = overland flow storage on impervious areas of nonsewered catchment at beginning and end of hour, in.

Runoff from Pervious and Impervious Areas of Sewered Catchments

The model assumes that for sewered catchments all of the rainfall or snowmelt on impervious areas and a constant fraction of the rainfall or snowmelt on pervious areas becomes surface runoff during the same hour:

$$q_s = 3630\ r_t(A_i + 0.3A_p) \tag{58}$$

where

q_s = hourly surface runoff rate from impervious and pervious area of sewered catchment, ft^3

A_i = impervious area of catchment, acres

This formulation neglects the effects of catchment shape, slope, and roughness on the surface runoff.

Wastewater inflow to the sewers is computed by

$$q_d = q_j\ P_c \tag{59}$$

where

q_d = hourly dry-weather or sanitary wastewater inflow to sewer reach, ft^3

q_j = wastewater flow during j-th hour of day, $ft^3/hr/capita$

P_c = catchment population

Channel and Conduit Flow Routing

The model assumes a nonsewered catchment drained by an open channel if Manning's roughness coefficient is greater than 0.019 and a sewered catchment drained by a closed conduit if Manning's roughness coefficient is less than 0.019. It consequently does not simulate flows in open channels draining sewered areas or in closed conduits draining nonsewered areas.

The total inflow (from a nonsewered catchment) into the upstream end of an open channel reach is given by

$$I = 3600Q + 3630\ (q_p A_p + q_i A_i) \tag{60}$$

where

I = total hourly inflow into upstream end of open channel reach crossing nonsewered catchment, ft^3

Q = inflow into upstream end of open channel reach from upstream reach, cfs

The total inflow from a sewered catchment into the upstream end of a sewer reach is given by

$$I = 3600Q + q_d + q_s \tag{61}$$

where

I = total hourly inflow into upstream end of sewer reach crossing sewered catchment, ft^3

Q = inflow to upstream end of sewer reach from upstream sewer reach, cfs

The flow routing is formulated for trapezoidal open channels and flood plains of trapezoidal cross section and for circular closed conduits using a storage routing technique. The change in channel or conduit storage is computed from

$$V'' = V' + I - Q \tag{62}$$

where

V', V'' = volume stored in reach at beginning and end of hour, ft^3

I = total hourly inflow into upstream end of reach, ft^3

Q = total hourly outflow from downstream end of reach, ft^3

The channel or conduit discharge is defined by Manning's equation:

$$Q^* = \frac{1.49}{n} \frac{A^{5/3}}{P^{2/3}} S^{1/2} \tag{63}$$

where

Q^* = discharge in channel or conduit, cfs

n = Manning's roughness coefficient

A = cross-sectional area of flow, ft^2

P = wetted perimeter, ft

S = slope of channel or conduit invert, ft/ft

The outflow from the channel or conduit reach is then simply

$$Q = 3600Q^* \tag{64}$$

Appropriate geometric relationships are defined for the cross-sectional area of flow and for the wetted perimeter of trapezoidal channels and flood plains. For circular closed conduits, it is assumed that the discharge varies linearly with the depth of flow. If the depth of flow is computed to be greater than the pipe diameter (i.e., the pipe surcharges) then the model assumes a discharge equal to full pipe flow under free surface flow conditions. Geometric relationships also define the volume of water stored in the reach for both open channels, flood plains and closed conduits.

Storage Facilities

A storage facility can be specified anywhere at the end of a channel or pipe reach. The model formulation assumes that all inflow exceeding a specified maximum outflow value is stored. If water is stored and the inflow falls below the maximum outflow value, the outflow is set equal to the maximum outfall value until the storage volume is reduced to zero. Thereafter the outflow is set equal to the inflow if the inflow becomes less than the maximum specified outflow value.

DORSCH CONSULT HYDROGRAPH-VOLUME METHOD

Summary

The Dorsch Consult Hydrograph-Volume Method simulates the time-varying runoff in combined sewerage systems consisting of several catchments and a converging branch and looping sewer and open channel network (Ritter and Warg, 1971; Koniger and Klym, 1972; Koniger et al., 1972; Klym et al., 1972; Mevius, 1973; Dorsch Consult, 1973; and Klym, 1975). The flow routing is based on a simplified solution of the full dynamic wave equations and simulates backwater, surcharging, and pressure flow, but not flow reversal. The model simulates diversion structures and storage basins. The model is limited to the simulation of single runoff events. Water quality, real-time control, and design features are not included.

Catchment Surface Runoff

Either measured rainfall or design rainfall intensities computed by regression analysis of measured rainfall are provided as input data. The hyetographs may be defined by discrete rainfall intensities at constant or variable intervals or be various depth-duration frequency formulas, e.g., the Eltinge formula.

Storm runoff from urban watersheds is computed separately for pervious and impervious surfaces using the kinematic wave equation:

continuity equation:

$$\frac{\partial y}{\partial t} = r - f - \frac{\partial q_s}{\partial A_c} - e - \frac{\partial h}{\partial t} \tag{64}$$

and Manning's equation:

$$\frac{q_s}{A_c} = \frac{y^{5/3} \; S_c^{1/2}}{n \; L_c} \tag{65}$$

where

y = water depth, m

r = rainfall intensity, m/sec

f = infiltration intensity, m/sec

A_c = surface area of the catchment, m^2

q_s = catchment storm runoff, m^3/sec

n = Manning's roughness coefficient

S_c = catchment slope, m/m

L_c = catchment length, m

e = evapotranspiration, m/sec

h = depression storage, m

Both equations are solved iteratively.

Infiltration and the filling and emptying of surface depressions are simulated separately for roofs, streets and for pervious surfaces subject to both infiltration and depression storages. The infiltration rate, for instance, is only reached when the given depth is supplied by precipitation and/or surface storage. As the surface depression depths differ greatly, runoff begins when the shallowest depressions are filled. After rainfall ceases, the puddles on impervious surfaces are reduced by evaporation and those on pervious surfaces slowly drain through infiltration. The individual tributary areas are characterized by combinations of the above three surface types. The hydrographs of the three surface types are superimposed according to the relative portion of each surface to obtain the tributary area hydrograph.

Channel and Conduit Flow Routing

Flow routing in closed conduits and open channels of various cross sections is accomplished by an implicit finite difference solution of the dynamic wave equations for unsteady gradually varied open channel flow (St. Venant equations):

momentum equation:

$$\frac{\partial Y}{\partial x} + \frac{\partial}{\partial x}\left(\frac{v^2}{2g}\right) + \frac{1}{g}\frac{\partial v}{\partial t} + \frac{Q_c}{gA}(v - v_q \cos\alpha) = S - s_e \qquad (66)$$

continuity equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = Q_c \qquad (67)$$

where

A = cross-sectional area of flow, m^2

g = acceleration due to gravity, m/sec^2

v = average velocity of flow, m/sec

S_e = energy slope, m/m

S = invert slope, m/m

v_q = average velocity of tributary inflow, m/sec

Q_c = tributary inflow at the sewer inlet, m^3/sec

α = angle between the flow through the sewer inlet and the flow in the sewer

Q = flow rate in the sewer, m^3/sec

Y = depth of flow in the sewer, m

These two equations describe the flow process in one network element, such as a sewer section, stormwater overflow structure, or retention basin. The energy slope S_e is calculated using the customary Manning or Prandtl-Colebrook equations. All additional losses, for example through manholes, unclean sewers, bends and curves, etc., are included in the friction factor.

Principles of mass and energy conservation are used to obtain the necessary equations for solving the flow conditions at sewer junctions. The simplest nodal point used in the program is one inflow and one outflow, and the maximum case consists of three inflowing and two outflowing sewers.

The equations for the simplest nodal point (one inflow, one outflow) are as follows:

continuity equation:

$$Q_{i1} = Q_{o2} \tag{68}$$

energy equation:

$$Z_{i1} + Y_{i1} + \frac{v_{i1}^2}{2g} \geq Z_{o2} + Y_{o2} + \frac{v_{o2}^2}{2g} \tag{69}$$

backwater constraint:

$$Z_{i1} + Y_{i1} \geq Z_{o2} + Y_{o2} \tag{70}$$

The equations programmed for the most complex case (three inflows, two outflows) are:

continuity equation:

$$\sum_{j=1}^{j=3} Q_{ij} = \sum_{j=4}^{j=5} Q_{oj} \tag{71}$$

energy equation:

$$\sum_{j=1}^{j=3} \left(Z_{ij} + Y_{ij} + \frac{v_{ij}^2}{2g}\right) Q_{ij} \geq \sum_{j=4}^{j=5} \left(Z_{oj} + Y_{oj} + \frac{v_{oj}^2}{2g}\right) Q_{oj} \qquad (72)$$

backwater constraint:

$$\min\ (Z_{ij} + Y_{ij}) \geq \max\ (Z_{oj} + Y_{oj}) \qquad (73)$$

where

Z = elevation of channel or conduit invert, m

i = subscript designating inflow variable

o = subscript designating outflow variable

1,2,3,4,5 = number of inflow or outflow

The system of equations is solved by an iterative method. It starts with estimated flow depths to calculate unknown discharges in the downstream direction and then the corresponding depths in the upstream direction. The entire procedure is repeated with the help of convergence formulations until the convergence criteria are fulfilled. Normally not more than 30 iterations are necessary to obtain convergence.

ENVIRONMENTAL PROTECTION AGENCY STORMWATER MANAGEMENT MODEL

Summary

The U.S. Environmental Protection Agency's Stormwater Management Model is one of the most comprehensive mathematical models available for the simulation of storm and combined sewerage systems. It computes the combined storm and sanitary runoff from several catchments and routes the flows through a converging branch sewer network (Metcalf & Eddy et al., 1971). Flow diversion structures can be modeled and storage can be simulated for both inline and overflow retention basins. An additional feature is a receiving water model which includes nonsteady formulations of hydrodynamics and mass transport for two-dimensional (vertically mixed) water bodies receiving sewerage system effluents.

Both dry-weather and stormwater quality are computed for suspended and settleable solids, biochemical and chemical oxygen demand, coliform bacteria, phosphorus, nitrogen, and oil and grease for each modeled catchment and routed through the

sewerage system. Mathematical formulations which simulate various combinations of overflow treatment processes for one treatment facility are included to evaluate the effectiveness of overflow treatment. The model does not include real-time control and design features. The model is limited to the simulation of single runoff events and inline treatment cannot be simulated. Cost functions are built into the program to compute the cost of overflow storage and treatment.

Watershed Representation

The watershed being modeled is conceptually represented by a network of hydraulic elements (i.e., catchments, gutters and pipes). A catchment is envisioned as a unit of uniform watershed characteristics, such as surface cover and ground slope. Each catchment is defined by area, width, ground slope, the detention depth requirements, the roughness factor (such as Manning's coefficient), and the coefficients describing the infiltration loss by Horton's equation. Gutters and pipes are characterized by the Manning's coefficient, length, invert slope, and geometric description of the network.

Stormwater Runoff

Each catchment is split into pervious areas and impervious areas with and without surface detention for runoff computations from rainfall. Instant runoff is assumed from impervious areas without detention, no infiltration is computed on impervious areas with detention, and the full computations outlined below are used for pervious areas.

For pervious areas and impervious areas with detention, the depth of overland flow is computed by

$$y'' = y' + r_t \Delta t \tag{74}$$

where

y', y'' = overland flow depth of the catchment at beginning and end of time interval Δt, ft

r_t = rainfall intensity during time interval Δt, ft/sec

Δt = time interval, sec

The potential infiltration rate on pervious areas is computed by Horton's exponential function:

$$f = f_e + (f_o - f_e)\, e^{-kt} \tag{75}$$

where

f = potential infiltration at time t, ft/sec

f_o = inintial infiltration rate, ft/sec

f_e = equilibrium infiltration rate, ft/sec

e = base of natural logarithm

k = constant based on soil and vegetation, sec^{-1}

An intermediate water depth, y_t, after accounting for infiltration, is given by

$$y_t = y' - f\,\Delta t \tag{76}$$

If the value of y_t in the catchment is larger than the available detention depth, overland flow is computed using Manning's equation:

$$v_c = \frac{1.49}{n}\,(y_t - h)^{2/3}\,S_c^{1/2} \tag{77}$$

and

$$q_s = v_c\,W_c(y_t - h) \tag{78}$$

where

v_c = velocity of overland flow, fps

n = Manning's roughness coefficient

S_c = ground slope of catchment, ft/ft

W_c = width of subcatchment, ft

q_s = stormwater runoff from catchment, cfs

h = detention depth, ft

The water depth in the subcatchment resulting from the rainfall, infiltration, and outflow after time interval Δt is:

$$y'' = y' - \left(\frac{q_s}{A_c}\right)\Delta t \tag{79}$$

where

A_c = surface area of the catchment, ft^2

Gutter Flows

The depth of water in a gutter resulting from the inflow and outflow is computed as follows. The rate of inflow into a gutter, q_g, is the summation of the tributary subcatchments' outflow, q_s, and the flow rate of the immediate upstream gutter, i_g, so that

$$i_g = \sum q_s + \sum q_g \tag{80}$$

The water depth at the upstream end of the gutter after time interval Δt is

$$Y''_g = Y'_g + \left(\frac{i_g}{A_s}\right) \Delta t \tag{81}$$

where

Y'_g, Y''_g = water depth in the gutter at beginning and end of time step Δt, ft

A_s = mean water surface area between the depths Y''_g and Y'_g, ft^2

The outflow from the gutter is computed using Manning's equation:

$$V_g = \frac{1.49}{n} R_g^{2/3} S_g^{1/2} \tag{82}$$

and

$$q_g = V_g A_g \tag{83}$$

where

V_g = gutter flow velocity, fps

R_g = hydraulic radius, ft

S_g = gutter invert slope, ft/ft

A_g = cross-sectional area of gutter at Y_g, ft^2

q_g = outflow from gutter, cfs

The water depth at the downstream end of the gutter resulting from the inflow and outflow after time interval Δt is

$$Y''_g = Y'_g + (i_g - q_g) \frac{\Delta t}{A_s} \tag{84}$$

Flows that reach the point of concern are added to produce a hydrograph ordinate for each succeeding time period until the complete hydrograph is computed.

Dry-Weather Runoff

The quantity and quality of dry-weather flow (sanitary runoff) is estimated from three data categories: drainage basin data, subarea data, and decision and adjustment parameters. The dry-weather flow from commercial and industrial areas is estimated from water use values. The dry-weather flow from residential areas is computed from regression equations.

For metered dwelling units with public sewer:

$$\bar{q}_d = 206 + 3.47\ V_m - 1.30\ P_w \tag{85}$$

where

$\bar{q}_d$ = average annual dry-weather runoff, gpd/dwelling unit

V_m = market value of the dwelling unit, thousands of dollars

P_w = sum of water and sewerage charges

For flat rate dwelling units and apartments with public sewers:

$$\bar{q}_d = 28.9 + 4.39\ V_m + 33.6\ P_u \tag{86}$$

where

P_u = number of persons per dwelling unit

For metered dwelling units with septic tanks:

$$q_d = 30.2 + 39.5\ P_u \tag{87}$$

Infiltration into Sewers

Infiltration into sewers is assumed to be a part of the dry-weather flow but it is computed separately. The effect of infiltration upon quality is assumed negligible, unless the water is known to be contaminated by passage through soils with significant soluble impurities (of the type being modeled). Quantitatively, infiltration is assumed to include moisture from miscellaneous sources causing a base dry-weather inflow, frozen residual moisture, antecedent precipitation, and high groundwater. If the groundwater table is below the sewer invert, infiltration is defined by

$$q_f = f_d + f_r + f_s \tag{88}$$

where

q_f = total infiltration, cfs

f_d = dry-weather infiltration, cfs

f_r = wet-weather infiltration - f_d, cfs

f_s = melting residual ice and snow infiltration, cfs

f_g = groundwater infiltration, cfs

If the groundwater table rises above the sewer invert, the groundwater infiltration predominates and is determined by the regression equation

$$f_g = \sum_{j=1}^{n} \alpha_j \, Y_{gw}^{\beta_j} \tag{89}$$

where

G_e = groundwater elevation above sewer invert, ft

α_j = coefficient for j-th term in equation

β_j = power to which Y_{gw} is raised in j-th term (e.g., 0,1,2,1/2 for j = 0,1,2, and 3, respectively)

The wet-weather infiltration is estimated from antecedent precipitation using the following linear relationship

$$f_r = a + a_o \, r_o + a_1 \, r_1 + \ldots a_9 \, r_9 \tag{90}$$

where

$f_r = \overline{Q}_s - f_d - Q_d$

a_n = coefficient to rainfall for n days prior to estimate, n = 0,1,....9

r_n = precipitation on n days prior to estimate, in.

$\overline{Q}_d$ = daily average sewer flow excluding surface runoff, gpm

Q_d = sewage flow, gpm

Residual melting ice and frost infiltration is determined by

$$f_s = \begin{cases} s_m \left\{ \sin \left[180 \left(\frac{N_t - N_b}{N_e - N_b} \right) \right] \right\} \\ 0.0 \text{ if the day under study is not in the melting period; that is, the number of degree days does not exceed 750} \end{cases} \tag{91}$$

where

s_m = residual moisture peak contribution, cfs

N_t = day on which infiltration estimate is desired

N_b = day on which melting period begins; that is, number of degree days exceeding 750

N_e = day on which melting period ends

After the total infiltration, q_f, has been computed, this flow must be apportioned throughout the designated study area. The opportunity factor, F, which represents the relative number and length of openings susceptible to infiltration, determines the criterion for apportionment:

$$F = \sum_{\text{conduits}} \begin{pmatrix} \text{conduit} \\ \text{perimeter} \end{pmatrix} \begin{pmatrix} \text{number of} \\ \text{joints in} \\ \text{each conduit} \end{pmatrix} \tag{92}$$

Channel and Conduit Flow Routing

Flows are routed in open channels and closed conduits by the kinematic wave equation:

continuity equation:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \tag{93}$$

and Manning's equation:

$$Q = \frac{1.49}{n} A R^{2/3} S^{1/2} \tag{94}$$

where

Y = depth, ft

x = longitudinal distance, ft

t = time, sec

g = gravitational acceleration, ft/sec^2

S = invert slope, ft/ft

Q = flow rate, cfs

A = flow area, ft^2

R = hydraulic radius, ft

n = Manning's roughness coefficient

Normalizing the above equation by the full-flow values of Q, A, and R gives

$$\frac{Q}{Q_f} = \frac{A R^{2/3}}{A_f R_f^{2/3}} \quad . \tag{95}$$

This formulation neglects backwater effects, which may be appreciable for low slopes. To improve the predictions but retain the kinematic wave formulation's simplicity, the terms of the dynamic wave equation are used in defining the friction slope for the full-flow computation:

$$Q_f = \frac{1.49}{n} A_f R_f^{2/3} \left(S - \frac{\partial Y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x}\right)^{1/2} \tag{96}$$

where

v = flow velocity, fps

This system of equations is solved by an iterative finite difference scheme.

The cross-sectional areas of various conduit shapes are defined by the following equations

circular:

$$A = \pi D^2/4 \tag{97}$$

rectangular:

$$A = H\ W \tag{98}$$

egg-shaped:

$$A = 0.5105\ H^2 \tag{99}$$

horseshoe:

$$A = 0.829\ H^2 \tag{100}$$

gothic:

$$A = 0.655\ H^2 \tag{101}$$

catenary:

$$A = 0.703\ H^2 \tag{102}$$

semi-elliptic:

$$A = 0.785\ H^2 \tag{103}$$

basket-handle:

$$A = 0.786\ H^2 \tag{104}$$

semi-circular:

$$A = 1.27\ H^2 \tag{105}$$

modified basket-handle:

$$A = W\ (H_s + \pi W/8) \tag{106}$$

rectangular with triangular bottom:

$$A = W\ (H - H_t/2) \tag{107}$$

rectangular with round bottom:

$$A = H_s W + D_i^2 \ (\theta - \sin\theta)/8 \tag{108}$$

$$\theta = 2 \sin^{-1}(W/D_i) \tag{109}$$

where

A = cross-sectional area, ft^2

D = diameter, ft

H = height, ft

W = width, ft

H_s = side height, ft

H_t = height of triangle, ft

D_i = invert diameter, ft

θ = angle formed by center of circle with chord intersecting circle, radians

Surcharging is approximated by storing water exceeding the full pipe flow capacity in the next upstream manhole. No further upstream propagation of the surcharging is computed. The program execution terminates if the storage capacity of the manhole is exceeded.

Backwater is approximated by routing a portion of the flow normally through the conduit and allocating the rest to storage by specifying a storage element. The flow division is computed by

$$Q_1 = Q \sqrt{p} \tag{110}$$

and

$$Q_2 = Q - Q_1 \tag{111}$$

where

Q_1 = flow directly into storage, cfs

Q_2 = flow routed through conduit, cfs

Q = inflow to backwater element, cfs

p = ratio of current to maximum storage volume in the downstream storage element

Diversion Structures

Three types of flow diversion structures are modeled. The formulations neglect downstream effects on the flow division.

The flow diversion by the first type specifies that a diversion occurs only if a given inflow is exceeded. At that point, the nondiverted flow remains constant and the surplus is diverted:

$$\left.\begin{aligned} Q_o &= Q \\ Q_w &= 0.0 \end{aligned}\right\} \quad \text{if } Q \leq Q_{max} \tag{112}$$

$$\left.\begin{aligned} Q_o &= Q_{max} \\ Q_w &= Q - Q_{max} \end{aligned}\right\} \quad \text{if } Q > Q_{max}$$

where

Q = inflow to diversion structure, cfs

Q_o = undiverted flow, cfs

Q_w = diverted flow, cfs

Q_{max} = maximum undiverted flow, cfs (provided as input data)

The same relationship is assumed for a cunnette type flow divider, except that the value of Q_{max} is computed internally as a function of the sewer size, assuming that the full flow capacity of the divider pipe is one-half of the main sewer.

Equation 112 is used to define the continuity relationships between inflow and outflow of weir diversion structures, but the diverted flow is computed from the weir formula and a linear flow-depth relationship:

$$Q_w = c_w L_w H_w^{1.5} \tag{113}$$

$$Y_w = (Q - Q_{max})(Y_{max} - Y_w)/(I_{max} - Q_{max}) \quad (114)$$

where

c_w = weir discharge coefficient

L_w = weir length, ft

Y_w = depth of flow above weir crest, ft

Y_{max} = water depth at maximum flow, ft

Q_{max} = maximum undiverted flow, cfs

I_{max} = maximum inflow to structure, cfs

Values for variables in the above list have to be provided as input data.

Pumping Stations

Lift stations are assumed to operate when wet well volume reaches a designated value which actuates a single, constant-discharge pump. Pumping continues until the wet well is emptied. Flooding caused when wet well capacity is exceeded is not considered.

Storage Facilities

Flows are routed through storage facilities by solving the continuity equation in conjunction with a weir or orifice equation for the outlet of the storage facility:

continuity equation:

$$0.5\ (I' + I'')\ \Delta t = 0.5\ (Q' + Q'')\ \Delta t + (V'' - V') \quad (115)$$

where

I', I'' = inflow at beginning and end of time interval Δt, cfs

Q', Q'' = outflow at beginning and end of time interval Δt, cfs

V', V'' = stored volume at beginning and end of time interval Δt, ft^3

Δt = time interval, sec

orifice equation:

$$Q_o = c_o A_o \sqrt{64.4 Y_o} \tag{116}$$

where

Y_o = water depth above orifice centerline, ft

A_o = orifice area, ft^2

c_o = coefficient of discharge

weir equation:

$$Q_w = 3.33 L_w Y_w^{1.5} \tag{117}$$

where

L_w = length of weir, ft

Y_w = depth of flow above weir crest, ft

Water Quality

The modeled water quality constituents are BOD, COD, suspended solids, settleable solids, coliforms, phosphorus, nitrogen, and oil and grease. Average daily values and the quality of dry-weather flow are either computed from measured values or from average per capita values. Hourly variations in the dry-weather flow quality are computed by adjusting the daily average values using hourly variation factors.

The stormwater quality computations consider the quantity of pollutants on each subcatchment before the storm event and the rate of their removal during the storm. The pounds of pollutant washed off in any time interval, Δt, are assumed to be proportional to the pounds remaining on the ground. It is further assumed that a uniform rate of 1/2 in./hr would wash away 90 percent of the pollutant in 1 hr. This leads to the equation

$$M_o - M = M_o(1 - e^{-4.46 q_s t}) \tag{118}$$

where

M_o = pollutants on surfaces which produce runoff in a subcatchment at beginning of runoff, lb

M = pollutants on surfaces at time t, lb

t = time, hr

q_s = runoff rate, in./hr

Concentrations of pollutants washed away from the subcatchment are expressed as

$$C = \frac{268}{\Delta t} \cdot \frac{M_o - M}{q_s A_c} \tag{119}$$

C = pollutant concentration in mg/l

A_c = catchment drainage area, acres

Δt = time interval, minutes

How much dirt and dust accumulates on the ground before the start of a storm depends upon the sweeping efficiency and the cleaning frequency. The equivalent number of days of accumulation, N_e, is computed by

$$N_e = \frac{N_d}{N_c} \tag{120}$$

where

N_d = number of dry days before storm event

N_c = number of days between cleaning frequency

The pounds of dust and dirt on the ground before the start of the storm, M_d, are given by

$$M_d = N_c \, m_d \, [1 + (1-E_c) + \ldots + (1-E_c)^{N_e}] \tag{121}$$

where

m_d = dust and dirt accumulation rate/subarea

E_c = efficiency of street cleanings

Pollutant concentrations in the sewer are initialized by the quality of the wastewater inflows diluted by infiltration, which is assumed to contain no pollutants. These pollutants are then routed through the sewer system, with accounting for decay during transit. The model is based on the assumptions that mixing within each sewer element in the system is instantaneous and complete and the non-conservative pollutants decay according to a first-order reaction.

The general equation of the model representing continuity of mass is as follows:

$$\begin{matrix}\text{pounds in} \\ \text{element at} \\ \text{new time-step}\end{matrix} = \begin{matrix}\text{pounds in} \\ \text{element at} \\ \text{old time-step}\end{matrix} + \begin{matrix}\text{pounds} \\ \text{entering}\end{matrix} - \begin{matrix}\text{pounds} \\ \text{leaving}\end{matrix} \pm \begin{matrix}\text{pounds} \\ \text{decayed or} \\ \text{generated}\end{matrix} \pm \begin{matrix}\text{pounds entering} \\ \text{or leaving from} \\ \text{source or sink}\end{matrix} \qquad (122)$$

Using the finite difference scheme, the final computer model for routing pollutants is in the form

$$C_{n+1} = \left\{ C_n \left[V_n \frac{2}{\Delta t} - (K_1+K_2) - (Q_{out})_n \right] + (C_{in}\, Q_{in}) + (C_{in}\, Q_{in})_{n+1} + K_2\, C_m\, (V_n+V_{n+1}) \right\} \Big/ \left\{ V_{n+1} \left[\frac{2}{\Delta t} + (K_1+K_2) \right] + (Q_{out})_{n+1} \right\} \qquad (123)$$

where

C = concentration of pollutant in element, lb/ft^3

V = volume in element, ft^3

n = time-step number

Q = flow rate, cfs

K_1 = decay rate or oxygen utilization rate, sec^{-1}

K_2 = growth rate or reaeration rate, sec^{-1}

C_m = maximum growth or oxygen saturation value, mg/l

Δt = time increment, sec

Sediment deposition and scum in the sewer considers the particle diameter in suspension corresponding to the velocity in the sewer conduit:

$$d = \frac{RS}{k_s \ (g_s - 1)} \tag{124}$$

where

d = particle diameter, ft

R = hydraulic radius, ft

S = conduit invert slope, ft/ft

k_s = Shield's k as criterion for deposition and resuspension

g_s = specific gravity of the sediment

The amount of settled sediment to the beginning of the time step is

$$M_s' = M_s' + F_s\ (T_1 + T_2 + T_3)\ \Delta t \tag{125}$$

The amount of suspended sediment transported during the time step is

$$C_s = \frac{[(1-F_s)(T_1+T_2+T_3)]}{Q_{in}} + \frac{(1-F_s)\ M_s'}{V_{n+1}} \tag{126}$$

The amount of settled sediment at the end of the time step is

$$M_s'' = F_s\ M_s' \tag{127}$$

where

C_s = concentration of suspended solids, lb/ft^3

M_s'', M_s' = amount of settled sediment, at start and end of time step Δt, lb

F_s = fraction of sediment in suspension with diameter greater than or equal to the critical particle diameter

Sedimentation, scour and water quality reactions are not simulated in storage facilities. The pollutant loads are traced and redistributed using complete mixing or plug flow options.

Nine unit treatment processes at or near a sewer outfall can be modeled in any combination. Storage, either alone or combined with sedimentation, may be used to reduce the hydraulic capacity of the treatment units and is included in the model. The removal efficiency of each unit treatment process is defined by internal empirical equations for each pollutant. Only a few physical dimensions of the treatment facilities have to be provided as input data.

For dissolved air flotation the average removal of suspended solids is given by

$$\Delta C_s = 0.656 + \frac{0.06\ C_s}{190} - 0.40 \left\{ \frac{Q - 300}{2000} \right\} \tag{128}$$

where

Q = overflow rate, $gpm/ft^2/day$

The above equation is applicable if the overflow rate is at least 300 gpm/ft^2/day and the removal efficiency is not less than 0.30 nor more than 0.76. If chemicals are added, the removal of suspended solids will be given by

$$\Delta C_s = 0.656 + \frac{0.06\ C_s}{190} - 0.40 \left(\frac{Q - 1000}{7000} \right) + k_1 \left(\frac{20,000 - Q}{100,000} \right) \quad (129)$$

where

ΔC_s = suspended solids removed, ppm

C_s = suspended solids in inflow, ppm

k_1 = 1 if chemicals are added or 0 if not added

Q = overflow rate, gpm/ft^2/day

The amount of chemicals added is defined as 12 mg/l.

The rate of BOD removal is computed by

$$\Delta C_b = 0.59 + \frac{0.05\ C_b}{100} - 0.36 \left(\frac{Q - 1000}{7000} \right) + 0.02\ k_1 + 0.15\ k_2 \quad (130)$$

where

ΔC_b = BOD removal, ppm

C_b = BOD concentration, ppm

k_1 = 1 if chemicals are added or 0 if not added

k_2 = 1 if chlorine is added or 0 if not added

The model restricts the BOD removal to not less than 0.18 and not more than 0.60. The amount of chlorine is defined as 10 mg/l if the BOD of the influent is 130 or less and as 15 mg/l if it is more.

If the dissolved air flotation process is preceded by fine screens, a different removal efficiency is applied to the fine screen effluent. It is assumed that fine screens remove 27 percent of the suspended solids and 22 percent of the BOD. The equations used in case of fine screen effluent are

$$\Delta C_s = 0.528 + \frac{0.06\ C_s}{190} - 0.486 \left(\frac{Q - 1000}{7000}\right) \tag{131}$$

$$+ 1.37\ k_1 \left(\frac{20,000 - Q}{100,000}\right)$$

$$\Delta C_b = 0.475 + \frac{0.05\ C_b}{100} - 0.405 \left(\frac{Q - 1000}{7000}\right) \tag{132}$$

$$+ 0.026\ k_1 + 0.195\ k_2$$

The capacity of a microstrainer is given by

$$C_m = \frac{169}{F_i + 1.8} \tag{133}$$

where

C_m = capacity, gpm/ft^2 submerged area

F_i = filterability index

The removal of suspended solids is given by

$$\Delta C_s = \frac{10,000\ F_c}{C_m} \tag{134}$$

where

$$F_c = \left(\frac{400}{A_c}\right)^{0.5}$$

A_c = catchment drainage area, acres

The factor F_c allows for the effect of comminution and disintegration of the solids during flow in the sewers. It has a maximum value of 1.0. The model restricts the capacity to 40 gpm/ft^2 of submerged area regardless of the solids concentration.

The following equations are used for estimating the removal of BOD by a microstrainer

$$\Delta C_b = C_b - 10.0 \text{ if } C_b \geq 27 \text{ ppm} \tag{135}$$

$$\Delta C_b = 17.0\ C_b/27.0 \text{ if } C_b < 27 \text{ ppm} \tag{136}$$

MASSACHUSETTS INSTITUTE OF TECHNOLOGY URBAN WATERSHED MODEL

Summary

The Massachusetts Institute of Technology (MIT) Urban Watershed Model (MITCAT) simulates the time-varying runoff of several catchments and a converging sewer and open channel network, including loops and converging and diverging branches (Harley et al., 1970). The model can simulate only single runoff events. Water quality and real-time control features are not included.

A separate model includes design features to compute the sizes and costs of sewers, storage and treatment facilities which will result in the least-cost combination of alternatives for the elimination of untreated overflows and the reduction of flooding and surcharging (Kirshen et al., 1972).

The original model was developed at MIT for the U.S. Office of Water Resources Research but the model has been modified by Resource Analysis, Inc., for its routine application (Schaake et al., 1973).

Watershed Representation

The model replaces the natural complexities with a number of simple elements such as overland flow planes, stream segments, pipe lengths, etc. A suitable combination of these simple elements is assumed sufficient to model the behavior of an entire catchment. Two basic runoff elements were chosen. Flow distribution over the surface of the catchment is modeled as planes of overland flow. The inputs to an overland flow plane can be spatially uniform lateral inflow from rainfall, lateral outflow to infiltration, and upstream inflow from adjacent overland flow segments. Flow from the overland flow planes is collected by streamflow segments as lateral inflow and then passed downstream to other stream segments. The term stream is used in a generic sense and may include any open or closed form of conveyance.

Stormwater Runoff

Several alternative representations of infiltration are available in the MIT catchment model. These are

1. Horton's equation
2. Holtan's method
3. U.S. Soil Conservation Service method
4. Runoff coefficient method

The runoff coefficient method assumes that the potential infiltration f(t) at time t is a portion of the rainfall rate r(t) at that time:

$$f(t) = \beta(t)\ r(t) \tag{137}$$

where $\beta(t)$ depends on the antecedent precipitation volume V_r calculated over the past N days and the rainfall depth during the storm up to time t:

$$\beta(t) = 1 - \exp\left\{\alpha\ V_r + \int_o^t r(t)\ dt\right\} \tag{138}$$

where α is determined from observed volumes of rainfall and runoff or from past experience in the region.

Each of the methods permits initial infiltration rates to depend on some measure of antecedent soil moisture and to vary with time during the storm.

These methods are applied differently to the input rainfall. For methods 3 and 4 the infiltration is subtracted from the rainfall to obtain an effective rainfall which is then input to the catchment element. For methods 1 and 2 the original rainfall is applied directly to the catchment and then the infiltrated portion is subtracted within the routing through the element.

The kinematic wave equation is used for overland flow routing:

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = (r - f)/43200 \tag{139}$$

$$q = \alpha_c\ y^m\ c \tag{140}$$

where

y = depth of overland flow, ft

q = rate of flow per catchment element width, cfs/ft

t = time, sec

x = distance in direction of flow, ft

r = rainfall intensity, in./hr

f = infiltration rate, in./ hr

α_c, m_c = empirical coefficients dependent on surface geometry and roughness

Runoff occurs only if the rainfall intensity exceeds the potential infiltration rate.

The coefficients α_c and m_c for overland flow are defined by Manning's equation:

$$q = \frac{1.49}{n_c} y_c^{5/3} S_c^{1/2} \tag{141}$$

as

$$\alpha_c = \frac{1.49}{n_c} S_c^{1/2} \tag{142}$$

and

$$m_c = 5/3 \tag{143}$$

where

n_c = Manning's roughness coefficient for catchment element

S_c = overland surface slope of catchment element, ft/ft

Conduit and Open Channel Flow Routing

The kinematic wave equation is also used for flow routing in conduits and open channels. The kinematic wave equation for open channel segments is

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{144}$$

$$Q = \alpha_s A^{m_s} \tag{145}$$

where

A = cross-sectional area of flow, ft^2

Q = discharge rate, cfs

q = lateral inflow rate of overland flow, cfs/ft

α_s, m_s = empirical coefficients

The coefficients α_s and m_s for a triangular open channel are defined by Manning's equation:

$$Q = \frac{1.182}{n_s} \left[\frac{\sqrt{z}}{1+\sqrt{1+z^2}} \right]^{2/3} A^{4/3} S^{1/2} \tag{146}$$

as

$$\alpha_s = \frac{1.182}{n_s} \left[\frac{\sqrt{z}}{1+\sqrt{1+z^2}} \right]^{2/3} S^{1/2} \tag{147}$$

$$m_s = 4/3$$

where

n_s = Manning's roughness coefficient of stream segment

S = invert slope of stream segment, ft/ft

$z = W/(2Y)$

W = width of water surface, ft

Y = water depth, ft

The routing procedure used to represent pipe flow is an adaptation of the flow routing methods developed for EPA'S Stormwater Management Model. The model has the capability of routing through all common sewer conduit shapes. The solution procedure follows the basic kinematic wave approach used in other routing elements. The problem of flow routing is basically one of determining the downstream conditions (flow and area) in a conduit, given the upstream conditions and the conditions at the previous time step:

continuity equation

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{149}$$

and Mannings equation:

$$Q = \frac{1.49}{n} AR^{2/3} S_e^{1/2} \tag{150}$$

where

n = Manning's roughness coefficient

R = hydraulic radius, ft

S_e = energy slope, ft/ft

In order to improve the accuracy of results at low slopes, additional dynamic terms for the water surface and velocity head slopes are included in the slope term of Manning's equation. The energy slope is computed as follows:

$$S_e = S - \frac{\partial Y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} \tag{151}$$

where

S = invert slope, ft/ft

Y = depth of flow, ft

v = flow velocity, fps

g = gravitational constant, ft/sec^2

A normalized continuity equation in finite difference form using the dimensionless area α and the dimensionless flow rate Ψ is

$$\Psi + F_1\alpha + F_2 = 0 \tag{152}$$

where F_1 and F_2 are variables that may be computed at any time step from the known upstream conditions and the conditions in the previous time step. Equations 150 and 152 may then be solved for α and Ψ by determining the intersection of the straight line $-F_1\alpha-F_2$ with the normalized flow area relationship developed from Manning's equation. Since α represents the ratio of the cross-sectional area A of flow to the full-flow area and Ψ represents the ratio of the flow Q to the full flow value, the current time step values of A and Q are directly determined from α and Ψ.

Thus, the kinematic wave approach allows simple finite difference solutions of the unsteady flow equations governing typical sewer flow. In this form all disturbances propagate only in the downstream direction. As a result, precise simulation of transient backwater conditions is not possible. Backwater effects are included for flow routing in a given conduit element by means of an iterative flow averaging

scheme. However, since upstream conduits are routed independently of those downstream a continuous backwater profile is not simulated. Backwater effects due to ponding as a result of a flow control structure can be simulated more directly by a combination of conduit routing and in-system storage.

Pressure-flow conditions are not modeled in this flow routing procedure. Surcharging is represented to an extent, however, by storing flows in excess of the full flow capacity at an upstream manhole (an assumed upstream manhole is associated with every pipe element) until sewer capacity exists to accept the stored volume.

SOGREAH LOOPED SEWER MODEL

Summary

The Looped Sewer Model of the French consulting firm Societe Grenobloise d'Etudes et d'Applications Hydrauliques (SOGREAH) simulates the time-varying runoff of combined sewerage systems consisting of several catchments and a sewer and open channel network including loops and converging and diverging branches (SOGREAH, 1973 - 5 reports). The model includes formulations for most hydraulic phenomena encountered in closed conduit and open channel networks. The flow routing solves the dynamic wave equations coupled with equations for special sewer system facilities, such as diversion structures, pumping stations, inverted siphons, and retention basins. The solution considers both upstream and downstream boundary conditions, backwater, surcharging, pressure flow, and flow reversal.

The model appears to be limited to the simulation of single runoff events. Water quality, real-time control and design features are not included. This is a proprietary model and not all details of the model formulations are available. The firm has North American branch offices in New York City and Lasalle, Quebec, Canada.

Catchment Runoff

Dry-weather flow data for each inlet are input in the form of hydrograph values at constant time intervals. Storm runoff is computed from hyetographs provided as input or from design storms on the basis of frequency of occurrence and catchment characteristics. Options exist to use two different methods to compute design storms.

Using the first method, the maximum rainfall is computed by the formula:

$$r_{max} = \bar{r}\ A_c \tag{153}$$

where

r_{max} = maximum rainfall of frequency F, m^3/sec

$\bar{r}$ = average rainfall intensity of selected duration and frequency, m/sec

A_c = catchment drainage area, ha

Using the second method, the maximum catchment rainfall is computed by a formula suggested by Caquot:

$$r_{max} = S_c^a\ p_i^b\ A_c^c \tag{154}$$

where

S_c = slope of main drainage channel, percent

p_i = fraction of total drainage area which is impervious

a,b,c = universal empirical coefficients independent of location

Using either method, the maximum rainfall is transformed into a design rainfall hyetograph by

$$r_t = \alpha\ t^{\beta} \tag{155}$$

where

r_t = rainfall intensity at time t, m^3/sec

α, β = empirical coefficients

The empirical coefficients in this equation are determined by regression analysis of measured rainfall and runoff data.

Either measured or design hyetographs are then transformed into rainfall excess hyetographs by the rational formula or Horton's equation. The runoff coefficient of the rational formula can be computed with the equation

$$c' = 0.14 + 0.65\ p_i + 0.05\ S_c \tag{156}$$

Overland flow routing is accomplished by the Muskingum method:

$$Q'' = c_1 I' + c_2 I'' + c_3 Q' \tag{157}$$

where

Q', Q'' = catchment runoff at the beginning and end of a time step, respectively, m^3/sec

I', I'' = rainfall excess or upstream inflow at beginning and end of time step, m^3/sec

c_1, c_2, c_3 = Muskingum routing coefficients

The Muskingum routing coefficients are defined by

$$c_1 = \frac{2\ KX + \Delta t}{2K\ (1 - X) + \Delta t} \tag{158}$$

$$c_2 = \frac{\Delta t - 2\ KX}{2K\ (1 - X) + \Delta t} \tag{159}$$

$$c_3 = \frac{2K\ (1 - X) - \Delta t}{2K\ (1 - X) + \Delta t} \tag{160}$$

where

Δt = time step, sec

X = 0.2 (empirical coefficient)

K = lag time, sec

A first approximation of the lag time is computed with an equation suggested by Schaake and Geyer which considers physical catchment characteristics:

$$K = \frac{1.40\ L_c^{0.24}}{S_c^{0.16}\ p_i^{0.26}} \tag{161}$$

where

L_c = length of main drainage channel, m

This value is adjusted if rainfall and runoff measurements are available. If measurements are not available, the Caquot adaptation of the rational formula is used:

$$K = d\ S_c^e\ A_c^f\ r_{max}^g \tag{162}$$

where

d,e,f,g = empirical coefficients and r_{max} is defined by Equation 154.

Conduit and Open Channel Flow Routing

Flow routing in conduits and open channels is accomplished by an implicit finite difference solution of the full dynamic wave equations for nonsteady gradually varied open channel flow (St. Venant equations):

continuity equation:

$$\frac{dA}{dt} + \frac{dQ}{dx} = 0 \tag{163}$$

momentum equation:

$$\frac{1}{g}\left(\frac{dv}{dt} + v\frac{dv}{dx}\right) + \frac{dY}{dx} + \frac{Q|Q|}{K^2} = 0 \tag{164}$$

equation at the network nodes:

$$\Sigma Q = 0 \tag{165}$$

where

v = mean velocity in a section, m/sec

Y = water stage, m

A = cross-sectional area, m^2

Q = vA = discharge, m^3/sec

K = conveyance factor

The length of the computational time steps can be varied throughout the simulated period to save computation time. Longer time steps are used during periods of small changes in flow and shorter time steps for fast rising or falling hydrographs.

The program can simulate unsteady flow in any looped and branching network of sewers. The only limits are imposed by the computer equipment. Each sewer is divided into a certain

number of computational points where discharges (and/or velocities) and water stages are computed. Any common structure can be simulated, including weirs, lateral weirs, gates (automatic and time dependent), storage basins linked with a sewer by gates or weirs, siphons, pumping stations (pumping water from one sewer to another), and local head losses. As the program is written, nearly any other structure (such as an inflatable weir) can be introduced with a minimum loss of time. The following sewer cross sections are provided for: circular, oval, trapezoidal, and arbitrary (defined as a function of depth). The formulation computes backwater and surcharging due to any structure or to any cross-sectional variation.

WATER RESOURCES ENGINEERS STORMWATER MANAGEMENT MODEL

Summary

The Stormwater Management Model of Water Resources Engineers, Inc. (WRE) is a modified version of the Stormwater Management Model of the U.S. Environmental Protection Agency. It simulates the time-varying combined storm and sanitary runoff and wastewater quality of several catchments and a sewer and open channel network including loops and converging and diverging branches (Shubinski and Roesner, 1973).

The flow routing procedure is based on the dynamic wave equations which consider backwater, flow reversal and both upstream and downstream flow control. A special formulation is built in for the solution of surcharging and pressure flow. The solution is coupled with the hydraulic equations for flow control and diversion structures considering both upstream and downstream flow conditions. The model simulates overflow storage basins but not in-line storage facilities.

Both dry-weather and stormwater quality are computed for six constituents: suspended solids, settleable solids, biochemical oxygen demand, nitrogen, phosphorus, and oil and grease. The pollutants are routed through the sewer system but treatment processes are not modeled. The model does not include real-time control and design features and is limited to the simulation of single runoff events. A separate model for simulating both flow and quality in receiving waters is available (Chen and Orlob, 1972).

Watershed Representation

The model takes into account typical characteristics of urban drainage systems and permits modeling of large areas without a lot of detailed, geometric data. The collection system is

divided into blocks; each block in a catchment is considered to have similar average characteristics and to be an average distance from the low point. This simulates the hydrograph for both rising and falling limbs more accurately than does lumping all blocks into one large catchment. This technique also permits a reasonably large area to be represented as only two elements: a block-sized overland flow element, and an average length collection element. The overland flow portion is further subdivided into pervious and impervious parts.

The user is required to specify only a single element with basic geometric information. The computer will break this down into pervious, impervious and collection elements and provide the required linkages. For example, dry-weather flows are assigned to the collection element, while infiltration is applied mainly to the pervious area.

Catchment Runoff

Dry-weather flow is input as average values which are then adjusted internally for fluctuations during the day and week. Storm runoff is computed from rainfall data.

Infiltration is computed by the Horton formula:

$$f = f_e + (f_o - f_e)e^{-kt} \tag{166}$$

where

f = potential infiltration at time t

f_e, f_o = minimum and maximum infiltration rates

k = exponential rate of loss in infiltration capacity

If the computed potential infiltration rate exceeds the rainfall rate, then the infiltration rate is set equal to the rainfall rate.

Overland flow routing is accomplished with the kinematic wave equation:

continuity equation:

$$\frac{y''-y'}{\Delta t} = r - f + \frac{i_c + q_s + q_d}{A_c} \tag{167}$$

Manning's equation:

$$q_c = \frac{1.49}{n_c} S_c^{1/2} W_c \left(\frac{y' + y''}{2} - h_p\right)^{5/3} \tag{168}$$

where

Δt = time step

y'', y' = overland flow depth at beginning and end of time step

h_p = maximum depth of detention storage

r = rainfall during Δt

f = infiltration to groundwater during Δt

i_c = inflow from upstream catchment

q_c = outflow from catchment

q_d = other flow sources (e.g., dry-weather flow)

A_c = surface area of the catchment element

n_c = Manning's roughness coefficient

W_c = width of the catchment element

The continuity and Manning's equations are solved simultaneously by a Newton-Raphson method which handles the nonlinearity involved.

Conduit and Open Channel Flow Routing

The flow routing requires geometric definition of the network and operating rules for pumps and flow diverters. Such items as the flow across weirs or through orifices, the flow split in looping systems, surcharge conditions, and backwater are handled automatically by the model and need not be prespecified.

The conduit system is represented by a series of links (pipes) which are joined at nodes. Any shape cross section can be incorporated into the computer program; at present it simulates

rectangular, circular, horseshoe, basket-handle or eggshape pipes and trapezoidal open channels.

Conduit and open channel flow routing is accomplished by an explicit finite difference solution of the dynamic wave equations for nonsteady gradually varied open channel flow (St. Venant equations). The momentum equation is

$$\frac{\partial Q}{\partial t} = gAS_e + 2v\frac{\partial A}{\partial t} + v^2\frac{\partial A}{\partial x} - gA\frac{\partial Y}{\partial x} \tag{169}$$

where

Q = discharge

v = velocity

A = cross-sectional area of the flow

Y = hydraulic head

S_e = friction slope

x = distance along conduit

t = time

The friction slope is expressed as

$$S_e = \frac{k}{gAR^{4/3}} Q|v| \tag{170}$$

where

$$k = g\left(\frac{n}{1.49}\right)^2 \tag{171}$$

R = hydraulic radius

g = gravitational acceleration

n = Manning's roughness coefficient

The following finite difference form of the momentum equation is used:

$$\left(\frac{\partial Q}{\partial t}\right)_t = \frac{1}{1 + \frac{k}{\bar{R}^{4/3}|\bar{v}_t|}\,\Delta t} \left\{ - \frac{k}{\bar{R}^{4/3}} |\bar{v}_t|\, Q_t + 2\bar{v}_t \frac{\bar{A}_t - \bar{A}_{t-\Delta t}}{\Delta t} \right.$$

$$\left. + \bar{v}_t^{\,2} \frac{A_j - A_i}{L} - g\bar{A}_t \left[\frac{\left(Y_j - c_1 \frac{v_j^2}{2g}\right) - \left(H_i + c_2 \frac{v_i^2}{2g}\right)}{L} \right] \right\} \quad (172)$$

The term $c\,\frac{v^2}{2g}$ represents the entrance and exit losses in the conduit. The coefficients c_1 and c_2 are input values, while subscripts i and j refer to the nodal values at the ends of the link. The barred symbols for R, v, and A indicate a weighted average along the link, e.g.,

$$\bar{R}_t = 1/4\ (R_i + 2R_m + R_j)_t \quad (173)$$

where the subscript m refers to the midpoint of the link.

The continuity equation for a node is written directly in finite difference form:

$$\left(\frac{\partial Y}{\partial t}\right)_t = \frac{\Sigma Q_t}{A_{st}} \quad (174)$$

where

ΣQ_t = algebraic sum of all conduit flows, inflows and outflows at the node at time t

A_{st} = water surface area associated with the node at time t

When all of the pipes entering a node are at maximum flow, the node is said to be surcharged. The usual form of the continuity equation is not strictly applicable in this case, because no additional storage is available at the node. Continuity then becomes

$$\Sigma Q_t = 0 \quad (175)$$

and the earlier form is indeterminate. The exact change in hydraulic head cannot be determined directly; the situation is now a dynamic version of the traditional pipe network problem. However, an approximate, first-order correction

based on the Hardy Cross method can be computed. The Hardy Cross head correction for a node is

$$\Delta Y = \frac{-n\Sigma Q}{\Sigma\left(\frac{Q}{Y}\right)} \tag{176}$$

where

ΔY = change in nodal head for one iteration

Q = flow in a line entering the node

Y = pressure drop along that line

Σ = summation over all lines entering node

Successive application of Equation 176 to the nodes of the pipe network leads to the satisfaction of the continuity Equation 175. A derivation similar to that used to obtain Equation 176, but using the dynamic equation of motion, yields

$$\left(\frac{dY}{dt}\right)_t = K \frac{\Sigma Qt}{g(\Delta t)^2 \Sigma\frac{A}{L}} \tag{177}$$

where K = constant, usually taken as 0.25 to introduce some underrelaxation in the system. Note that A is the cross-sectional area of the links; the term

$$\frac{K}{g(\Delta t)^2 \Sigma\frac{A}{L}}$$

is a measure of the capacity of the system to absorb excess flow. It is a kind of equivalent surface area. For a surcharged node, the calculations are carried out using the above equation, which is identical in form to Equation 174.

Equations 172 and 174 are numerically integrated by a modified Euler method which yields a completely explicit approach. The momentum equation is applied to each link and the continuity equation to each node, entirely without implicit coupling.

Explicit methods involve fairly simple arithmetic and require little storage space compared to implicit methods, but they are generally less stable and often require very short time steps. Experience showed that a severe stability relationship is required for the solution technique of this model:

$$\Delta t \leq \frac{c\ A_s\ Y_{max}}{\Sigma Q} \tag{178}$$

where c = constant determined by experience, usually about 0.10. When c = 0.10 in the above equation, the rise in water surface during Δt should not exceed 0.10 of the depth available in the pipe. This criterion is approximate and must be approached with caution.

The solution of the dynamic wave equation considers special conditions which may exist all or part of the time in a sewer. First, the invert elevations of pipes which join at a node may be different; sewers are frequently built with invert discontinuities. Second, critical depth may occur in the conduit. Third, normal depth may control. Finally, the pipe may be dry.

Flow Control Devices

Weir diversions provide relief to the sanitary system during periods of storm runoff. Flow over a weir is computed by the equation

$$Q_w = c_w L_w [(Y_w + \frac{v^2}{2g})^a - (\frac{v^2}{2g})^a]$$

where

c_w = weir discharge coefficient

L_w = weir length

Y_w = driving head on the weir

v = approach velocity

a = weir exponent; 3/2 for transverse weirs, 5/3 for sideflow weirs

Normally, the driving head on the weir is given by

$$Y_w = Y_1 - h_w \tag{180}$$

where

Y_1 = water depth upstream of weir

h_w = height of weir crest above invert

However, several other existing conditions can modify the situation. If the downstream water depth also exceeds the weir crest height, the weir is submerged, and the flow computed by the above weir formula is corrected by applying the following equation:

$$Q_w^* = Q_w \left[1 - \left(\frac{Y_2 - h_w}{Y_1 - Y_2} \right)^a \right]^{0.385} \tag{181}$$

where

Q_w^* = submerged weir discharge

Y_2 = water depth downstream of weir

If the upstream pipe is surcharged, the weir will behave like an orifice and the flow is computed by the equation

$$Q_w = c_w^* L_w \left(Y_{max} - Y_c \right) \sqrt{2gY_w + v^2} \tag{182}$$

where

c_w^* = discharge coefficient for submerged case

Y_{max} = maximum flow depth

For all of the above cases, the direction of flow may be reversed under certain conditions. This does not affect the equations, except that Y_1 and Y_2 are switched.

An orifice is usually installed as a dropout to divert sanitary sewage out of the storm system to an interceptor and to limit the flow of stormwater into the sanitary system. Flow through an orifice is computed by

$$Q_o = c_o A_o \sqrt{gY_o} \tag{183}$$

where

c_o = discharge coefficient

A_o = cross-sectional area of orifice

Y_o = driving head on the orifice

The driving head is usually the depth of flow in the upstream pipe, Y_1. For a few installations, when the orifice is side flowing instead of a dropout, the head difference $Y_1 - Y_2$ may control. The variables Y are defined identically for orifices and for weirs, and backflow can occur.

A pump is represented conceptually as an off-line storage node (the wet well) from which the contents are pumped to another node according to a programmed rule curve. One, two, or three stage pumping is permitted, and the turning on and off of the pumps is computed automatically.

The program simulates weir outfalls and free outfalls with or without tide gates. The characteristics of the weir outfall were described above. For truly free outfall, the water surface at the free outfall is taken as critical or normal depth, whichever is less. If backwater exists, the receiving water elevation is taken as the water surface value at the outfall.

Water Quality

The following six water quality constituents are modeled: suspended solids, settleable solids, biochemical oxygen demand (BOD), total nitrogen, phosphorus, and oil and grease. Although this list could be modified within the form of the existing model, the present arrangement is ordered and linked to some extent. For example, BOD is produced both in dissolved form and as a percentage of the weights of suspended and settleable solids. Hence, the calculations in the program depend to a small degree on the order of the constituents.

The six water quality constituents are computed for three sources within the watershed: dry-weather flow, catchbasin washout and surface runoff.

Both flow and mass emission rate of the sanitary sewage are input as average values. Built-in program functions vary the average value to account for time and land use differences. The following land uses are presently included in the program: single-family residential, multiple-family residential, commercial, industrial, and open (or park). If the system consists of storm sewers only, all dry-weather flow contributions are omitted.

Catchbasins collect and store pollutants during dry weather, flushing rather rapidly in the early hours of a storm. The washout of material from the catchbasins is computed by

$$\Delta M_c = \frac{M_c}{\Delta t}\left(1 - e^{-\frac{Q_c\ t}{1.6\ V_c}}\right) \tag{184}$$

where

ΔM_c = mass emission rate from catchbasins, mass/unit time

M_c = total mass in catchbasins at beginning of time interval

Q_c = flow through catchbasins (runoff from pervious and impervious areas)

V_c = total catchbasin volume

Surface runoff of pollutants is treated much the same as catchbasin washout:

$$\Delta M = \frac{M}{\Delta t}\left(1 - e^{-4.6\ q\ \Delta t}\right) \tag{185}$$

where

ΔM = mass emission rate from the watershed

q = outflow per unit area

M = mass available at time t

Auxiliary computations account for the efficiency of the street cleaning process and other minor variations.

Most water quality constituents move in the sewer system by advective transport. The differential equation which describes this process is usually in one of two forms. In terms of mass rate of change

$$\frac{\partial M}{\partial t} = QC \tag{186}$$

where

M = mass

C = concentration

In terms of concentration rate of change

$$\frac{\partial C}{\partial t} = - \frac{C}{V} \frac{\partial v}{\partial t} + v \frac{\partial C}{\partial X} \tag{187}$$

where

v = flow velocity

V = volume

These equations are completely interchangeable; some aspects of both are used in the numerical solution.

The finite difference form for the concentration change in a conduit is

$$\frac{\Delta C_j}{\Delta t} = v \frac{\Delta C_n}{L} \tag{188}$$

where

L = length of conduit

ΔC_n = change in concentration along the conduit

ΔC_j = change in nodal concentration, due only to advection in a single conduit

This equation is applied to each conduit entering a node, provided the flow in that conduit is into the node. The concentration at the node does not take into account events which occur downstream.

Combining the conduit computations at a node, and adding the effects of volume change, source-sink contributions, pumps, weirs, and orifices produce the total change in concentration at the node:

$$\frac{\Delta C_j}{\Delta t} = - \frac{C_j}{V_j} \frac{\Delta V_j}{\Delta t} + \frac{1}{\alpha_n} \sum_u Q_n \frac{\Delta C_j}{\Delta t} + \sum_s \frac{\Delta C_j}{\Delta} \tag{189}$$

where

V_j = nodal volume

$\sum_u$ = summation over all upstream conduits

$\alpha_n = \sum_u Q$

Q_n = flow in conduit

$\sum_s$ = summation of all source-sinks, pumps, etc.

The latter term in the above equation is computed by a mass balance technique.

The time history of concentration at all nodes can be obtained by sucessfully integrating Equation 189 through time in a single step for each time step:

$$\Delta C_j(t+\Delta t) = C_j(t) + \left(\frac{\Delta C_j}{\Delta t}\right)_t \Delta t \tag{190}$$

The integration step, Δt, must be restricted so that

$$\Delta t \leq \left|\frac{L}{v}\right| \tag{191}$$

for each conduit, or an unstable solution may occur. In practice, this restraint can be violated occasionally without causing the solution to become unstable, although some error will be introduced. The model contains a trap to guard against this occurrence.

Scour, deposition, and decay are not considered during routing.

APPENDIX D

MATHEMATICAL SYMBOLS FOR TESTED MODELS

VARIABLES AND COEFFICIENTS

Symbol	Definition
α	angle between sewer flow and inflow or coefficient
β	coefficient
δ	partial differential
Δ	difference
θ	angle between center of circle and chord intersecting circle
π	ratio of circumference to diameter of circle
ρ_{so}	density of initial dry snowpack in percent of water density
ρ_{pt}	threshold density of compacted wet snowpack in percent of water density
ρ_{st}	threshold density of dry snow in compacted wet snowpack in percent of water density
ψ	coefficient
a	coefficient
A	cross-sectional area of channel flow
A^*	normalized area
A_c	catchment drainage area
A_f	cross-sectional area of conduit flowing full
A_g	cross-sectional area of gutter flow

VARIABLES AND COEFFICIENTS (Continued)

Symbol	Definition
A_i	impervious catchment drainage area
A_p	pervious catchment drainage area
A_s	water surface area
b	coefficient
c	coefficient
c'	coefficient
c_o	orifice discharge coefficient
c_w	weir discharge coefficient
C	water quality concentration
C_b	biochemical oxygen demand
C_m	capacity of micro-strainer
C_s	suspended solids concentration
d	particle diameter
D	channel diameter
D_i	invert diameter of rectangular conduit with round bottom
D_o	orifice diameter
e	coefficient, base of natural logarithm, evapotranspiration
E_c	efficiency of street cleaning
f	coefficient, infiltration
f_c	saturated infiltration capacity of soil

VARIABLES AND COEFFICIENTS (Continued)

Symbol	Definition
f_d	dry-weather infiltration into sewers
f_e	equilibrium infiltration rate into soil under saturated conditions
f_g	groundwater infiltration into sewers
f_i	rainfall loss on impervious areas
f_j	rainfall loss or infiltration into soil during j-th time interval
f_o	initial potential infiltration into soil
f_p	rainfall loss on pervious areas or infiltration into soil
f_r	storm water infiltration into sewers
f_s	ice and snowmelt infiltration into sewers
f_t	rainfall loss or infiltration into soil during time interval Δt
F	factor, function, or ratio
g	coefficient, gravitational acceleration
g_s	specific gravity of sediments
h	height or depth
h_g	height of diversion gate or orifice opening
h_i	catchment detention depth on impervious areas
h_o	height of orifice invert above or depth of orifice invert below channel invert
h_p	catchment detention depth on pervious areas
h_w	weir height

VARIABLES AND COEFFICIENTS (Continued)

Symbol	Definition
H	height of noncircular conduit
H_s	side height of modified basket handle or rectangular conduit with round bottom
H_t	height of triangle of rectangular conduit with triangular bottom
i_c	combined inflow to catchment
i_g	inflow to gutter
I	inflow to channel or storage
k	coefficient
$k_o, k_1, ..$	coefficients
K	channel conveyance coefficient or Muskingum routing coefficient
$K_o, K_1, ..$	coefficients
L	channel length
L_c	catchment length
L_w	length of weir crest
m	coefficient
m_c	overland flow coefficient
m_d	dust and dirt accumulation rate
m_s	channel flow coefficient
M	mass
M_c	pollutant mass in catch basin
M_d	mass of dirt and dust

VARIABLES AND COEFFICIENTS (Continued)

Symbol	Definition
M_s	mass of settled sediments
M_v	market value
n	number of items
n	channel roughness coefficient, English units
N	number of day or days
N_b	day at beginning of time period
N_c	number of days between street cleaning
N_d	number of dry days before storm event
N_e	day at end of time period, equivalent number of days of catchment pollutant accumulation
N_t	specific day
p	percent or fraction
p_i	percent or fraction of catchment surface which is impervious
p_p	percent or fraction of catchment surface which is pervious
p_w	accumulated water content of snow in percent of initial water content
p_{wt}	threshold accumulated water content of snow in percent of initial water content
P	hydraulic radius
P_c	catchment population
P_u	number of persons per dwelling unit
P_w	sewer and water charge

VARIABLES AND COEFFICIENTS (Continued)

Symbol	Definition
q	discharge per unit width or unit area
q_c	combined runoff from catchment
q_d	dry-weather or sanitary runoff from catchment
q_f	infiltration into sewers
q_i	runoff from impervious area of catchment
q_j	runoff during j-th time step
q_p	runoff from pervious area of catchment
q_s	storm water runoff from catchment
Q	channel flow or outflow from downstream end of channel or from storage
Q_c	combined catchment runoff entering sewer inlets, catchbasin throughflow
Q_f	full conduit flow
Q_g	gutter flow
Q_o	orifice discharge
Q_w	weir discharge
r	instantaneous rainfall intensity
r_d	daily precipitation
r_e	rainfall excess
r_i	rainfall excess on impervious area
r_p	rainfall excess on pervious area
r_t	rainfall during time interval Δt

VARIABLES AND COEFFICIENTS (Continued)

Symbol	Definition
R	hydraulic radius
R_f	hydraulic radius of conduit flowing full
R_g	hydraulic radius of gutter
s	soil moisture content or capacity
s_m	snowmelt
s_y	snowpack depth in percent of initial depth
s_{yt}	threshold depth of snowpack in percent of initial depth
S	channel invert slope
S_c	ground surface slope of catchment
S_e	energy slope
S_g	gutter invert slope
t	time
t_b	base time or duration of hyetograph or hydrograph
t_c	time of concentration of overland flow
t_e	end of time period
t_n	n-th time interval or time value
t_o	beginning of time period
t_p	time from beginning to peak of hyetograph or hydrograph
T	temperature
T_a	air temperature

VARIABLES AND COEFFICIENTS (Continued)

Symbol	Definition
u	wind velocity
U	unit hydrograph ordinate
U_p	peak ordinate of unit hydrograph
v	channel flow velocity
v_c	overland flow velocity
v_g	gutter flow velocity
V	volume of water stored in channel or storage
V_c	catchbasin volume
V_i	overland flow storage or cumulative rainfall loss on impervious area
V_m	market value
V_p	overland flow storage or cumulative rainfall loss on pervious area
V_r	antecedent precipitation volume
V_s	cumulative storm water runoff from catchment
W	channel width
W_c	catchment width
W_o	orifice width
W_y	width of water surface at depth y
x	distance
X	Muskingum routing coefficient

VARIABLES AND COEFFICIENTS (Continued)

Symbol	Definition
y	overland flow depth
y_i	overland flow depth on impervious area
y_p	overland flow depth on pervious
Y	depth of water in channel or storage
Y_g	depth of water in gutter
Y_{gw}	depth of groundwater above sewer invert
Y_o	depth of water above orifice center line
Y_w	depth of water above weir crest
z	geometric channel ratio
Z	elevation above reference plane

SUBSCRIPTS

Symbol	Definition
b	base, biochemical oxygen demand, beginning of time period
c	capacity, catchment, cleaning, hydrograph concentration time
d	decay, dirt and dust, dry-weather
e	end of time period, equilibrium, excess
f	full
F	return frequency
g	gate, gutter
gw	groundwater
i	i-th item, impervious, inflow, invert
j	j-th item
k	k-th item
m	snowmelt
max	maximum
min	minimum
n	n-th item
o	initial, orifice, outflow
p	peak, pervious, snowpack
r	rain
s	sediment, side, snow, suspended solids
t	threshold, triangle, average during time interval Δt

SUBSCRIPTS (Continued)

Symbol	Definition
u	per unit
w	water, weir
y	depth

SUPERSCRIPTS

Symbol	Definition
-	average
*	normalized, adjusted
'	at beginning of time interval
"	at end of time interval

APPENDIX E

ADDITIONAL MODEL TEST RESULTS

Two requests were mailed to model developers to comment on the draft report:

1. Request to review model comparison Tables 1 to 5, dated June 17, 1974.

2. Request to review sections of rough draft of final report, dated May 9, 1975.

The final report includes revisions based on the responses of the model developers. Following are new model test results which were received too late to be incorporated in the main body of the report. They were provided by Dorsch Consult and Water Resources Engineers and are included without editing by Battelle-Northwest or the Environmental Protection Agency.

DORSCH CONSULT

INGENIEURGESELLSCHAFT MBH

DORSCH CONSULT · Ingenieurges. mbH · 8 München 21 · Postfach 210243

Mr. Albin Brandstetter
Research Associate,
Environmental Management Section
Battelle Pacific Northwest Laboratories

Richland, Washington 99352
U S A

Ihre Nachricht vom	Ihr Zeichen	Unsere Nachricht vom	Unser Zeichen	München	28.5.1975
			GEG/ck		

Dear Sir,

In reply to your telex please find enclosed a brief description of the Dorsch Consult "QQS-MODEL" and of its capabilities. Test runs according to your requirements are under way and will be mailed within one week.

Please note that the Dorsch Consult "QQS-MODEL" is a model on its own not connected with the "Hydrograph - Volume - Method" of Dorsch Consult.

Sincerely yours

W.F. Geiger, P. Eng.

DORSCH CONSULT Ingenieurgesellschaft mbH · 8 München 21 · Postfach 210243
Elsenheimerstraße 63 · Telefon 57 97 - 1
Durchwahl: 267
Telex: dors 05-212 862 · Telegramme: dorschconsult München
Dresdner Bank München, Promenadeplatz, Konto Nr. 3 086 070, BLZ 700 800 00
Postscheckkonto München 1169 19-809

Sitz der Gesellschaft: München · HRB 42 898
Geschäftsführende Gesellschafter:
Regierungsbaumeister Xaver Dorsch, Ministerialdirektor a. D.
Dipl.-Ing. Dieter Dorsch · Dipl.-Ing. Helmut Dorsch
Geschäftsführer: Dipl.-Ing. Hermann Krebs
Dipl.-Ing. Günter Riedel · Dipl.-Kfm. Werner Schleber

DORSCH CONSULT QUANTITY - QUALITY - SIMULATION (QQS)

Summary

The Dorsch Consult QQS-MODEL (Geiger, 1975) allows for continuous simulation of four conservative water quality parameters. Runoff and its pollution from catchment areas, characterized as residential, commercial, industrial or mixed areas, are calculated by an unit hydrograph method, modified for the calculation of water quality. Flow routing through the network is based on the dynamic wave equations. Statistical analysis of the results provides annual and monthly frequency and duration curves for flows and pollutants at any node of the network. Still, single event simulation is possible providing pollutographs and hydrographs at any node of the network.

The model is generally applicable to any urban drainage basin and to any pollutant. The input has been established for the pollution indicating parameters BOD and settleable solids. Presently efforts are being made to complete the input sets for COD, suspended solids, bacteria coli and if possible for nutrients and chloride.

The Dorsch Consult QQS-MODEL includes the following features:

1. continueous simulation or single event simulation;
2. dry-weather flow and quality of several catchments;
3. three rain records;
4. stormwater runoff and quality from pervious and impervious areas of several catchments;
5. routing of combined wastewater flow and quality in a network of loops and converging and diverging branches;
6. various closed conduit and open channel cross-sections, but linearized partial filling curves;
7. backwater, upstream and downstream flow and quality control, surcharging and pressure flow;

8. weirs and diversion structures considering both upstream and downstream flow conditions;
9. pumping stations;
10. retention storage basins;
11. wastewater quality improvement at treatment plants and overflow treatment facilities; and
12. statistical analysis of results.

The model does not include the following features:

1. snow accumulation and melt;
2. flow reversal;
3. detailed flow and quality routing in gutters;
4. realtime control;
5. sedimentation and scour in channels;
6. wastewater quality decay, reactions and interactions in sewers and receiving waters;
7. design; and
8. costs.

Methods

The surface runoff from small catchment areas including the flow process through underlying small sewers, such as street sewers and laterals is obtained by means of the unit hydrograph method which is modified, however, for the qualitative part. The combined consideration of surface runoff and flow in small sewers and the use of a systemized network, each segment of which can be composed by several sewer elements totaling a length of up to 500 m, result in a tremendous reduction of network nodes. The assumption of linearity of the runoff process, being sufficiently valid for small catchment areas, could not be employed for the flow routing through the system of trunk and interceptor sewers, the flow behaviour of which may be formed by backwater effects and interaction of branching points, retention facilities and overflow structures. Thus, flow routing is based on the dynamic wave equations. This combination provides the necessary accuracy and still allows for continuous long-time simulation. The hydrographs and their flow velocities obtained by the hydraulic calculations form the basis for the pollutant transport within the network.

In case of outfalls the hydrographs and pollutographs resulting from the network calculations form the loading of the receiving waters. These loadings are applied at the nodes of a receiving water system. They are superimposed and routed through this system according to the same principles applied in the sewer network. Quality reactions and interactions in receiving waters, due to their complexity, have to be treated in a separate model.

The results are the continuous time-dependent pollutant loadings of receiving waters. Various properties are subjected to linear and logarithmic regression analysis. Two-dimensional frequency distributions are formed for the three pairs of variables showing the best correlations and for any other combination of variables given by input. All results are printed in block form together with their relevant statistics as well as in computer generated graphs.

In principle at each node of the sewer and receiving water network a statistical analysis of hydrographs and pollutographs can be produced. Usually only a selection of the possibilities given below are desired, mainly for outfalls, retention facilities and treatment plants. Annual and monthly frequency and durations curves of

- by-passed flows
- intercepted flows
- receiving water flows
- used storage capacities
- effluent pollutant loads or concentrations
- intercepted pollutant loads or concentrations
- receiving water pollutant loads or concentrations
- retained pollutant loads

can be evaluated.

All parts of the model have been verified checking quality against measurements taken in Augsburg, Munich and Stuttgart, Germany, and quantity against the Dorsch Consult HVM-Method and measurements, (Geiger, 1975). The model presently is applied to Augsburg, Germany (360,000 inhabitants). Completion of this project is scheduled for Sept. 1975.

Computer Program

The computer program consists of three main programs which are run sequentially. Model documentation does not describe details of the computer program. This is a proprietary model developed by the engineering consulting firm Dorsch Consult Ingenieurgesellschaft mbH of Munich, Germany. The model may be released under certain use and distribution restrictions. A user's manual is available upon request.

The complete program package is written in Standard-Fortran IV and consists of nearly 30,000 statements. Usable core storage of 400 K-bytes and a configuration with fast external mass storage are required. The package is presently installed in a Univac 1108 computer. The program, however, can be used on all BATCH processing systems with Fortran IV - compilers.

For single event simulation program output includes both tables and plots of discharges and water quality (mass rates) for any nodal point of the sewerage system and receiving water network. Flow velocities are not printed. For continuous simulation principal output is tables of statistical analysis and graphs of frequency and durations curves.

Hypothetical Pipe Tests

Comparison runs were performed for the hypothetical pipes described in Section VI.

Results of the flow and quality routing for the Two-Hour Triangular Inflow using the Dorsch Consult QQS-MODEL are presented in Figures 1 to 24.

Simulations used five minutes time steps. Fluctuations are apparent at the beginning and at the end of the triangular quality graphs. Later improvements in the flow routing portion of the model eliminated these fluctuations. In general the results are very similar to the SWMM-Model results for conditions without backwater. Slightly lower minimum concentration and later arrival time are computed for the 0.05% slope.

Reference

Geiger, F. W. "Urban Runoff Pollution Derived from Long-Time Simulation." Presented at the National Symposium on Urban Hydrology and Sediment Control, Lexington, Kentucky, July 28-31, 1975.

Table 1. SUPPLEMENT TO COMPARISON OF MODEL FEATURES
(Tables 2 to 5 in main text)

COMPARISON OF HYDROLOGIC FEATURES OF MODELS (Table 2 in main text)

	MULTIPLE CATCHMENT INFLOWS	DRY-WEATHER FLOW	SUBCATCHMENT PRECIPITATION	EVAPORATION	SNOW ACCUMUL. AND MELT	GROUNDWATER SIMULATION	GUTTER FLOW
DORSCH CONSULT QQS - MODEL	YES	DIURNAL PATTERN COMPUTED FROM LAND USE	ONE HYETOGRAPH* PER SUBC., MAX. THREE PREC. REC. PER CALC. AREA	NONLINEAR FUNCTION OF AIR TEMPERATURE	IN PREPARATION	NO	NO

	INFILTRATION ON IMPERVIOUS AREAS	OTHER RAINFALL LOSSES ON IMPERVIOUS AREAS	STORM RUNOFF FROM IMPERVIOUS AREAS	INFILTRATION ON PERVIOUS AREAS	OTHER RAINFALL LOSSES ON PERVIOUS AREAS	STORM RUNOFF FROM PERVIOUS AREAS	WATER BALANCE BETWEEN STORMS
DORSCH CONSULT QQS - MODEL	NO	INITIAL LOSS* BEFORE RUNOFF BEGINS	UNIT HYDRO-GRAPH	CONSTANT LOSS RATE*	INITIAL LOSS* BEFORE RUNOFF BEGINS	UNIT HYDRO-GRAPH	RECOVERY OF DEPRESSION STORAGE DUE TO EVAPORATION

COMPARISON OF HYDRAULIC FEATURES OF MODELS (Table 3 in main text)

	OPEN CHANNEL NETWORK	FREE SURFACE FLOW	BACKWATER EFFECTS	FLOW REVERSAL	SURCHARGING AND PRESSURE FLOW	DIFFERENT PIPE CROSS-SECTIONS	INFILTRATION INTO SEWERS OR OPEN CHANNELS
DORSCH CONSULT QQS - MODEL	CONV. AND DIV. BRANCH NETWORK	DYNAMIC WAVE EQU. IMPLICIT FINITE DIFF. SOL., MANNING OR PRANDTL EQU.	YES	NO	YES	VAR. STAND. PLUS ARBITR. SHAPES, BUT LINEARIZED PARTIAL FILLING CURVES	NO

	DIFFERENT DIVERSION STRUCTURES	DIVERSION COMPUTATION	PUMPING STATIONS	STORAGE FACILITIES	STORAGE COMPUTATION	COMPUTES AND PRINTS STAGE HYDROGRAPHS	COMPUTES AND PRINTS FLOW VELOCITIES
DORSCH CONSULT QQS - MODEL	WEIR, DIVERGING PIPE BRANCHES	WEIR EQU. CONSIDERING UPSTREAM AND DOWNSTREAM CONDITIONS	PUMPING CAPACITY CURVE*	YES	FUNCTION OF SHAPE, INFLOW AND OUTFLOW, CONTINUITY EQU.	FOR EACH SEWER SYSTEM ELEMENT PRINT OPTIONAL	COMPUTED FOR EACH SEWER SYSTEM ELEMENT, BUT NO PRINTS

* PROVIDED AS INPUT DATA

Table 1. SUPPLEMENT TO COMPARISON OF MODEL FEATURES (CONTD.)
(Tables 2 to 5 in main text)

COMPARISON OF WATER QUALITY FEATURES OF MODELS (Table 4 in main text)

	QUALITY CONSTITUENTS	DRY-WEATHER QUALITY	STORM RUNOFF QUALITY	QUALITY INTERACTIONS ON CATCHMENTS	QUALITY ROUTING IN CHANNELS	SEDIMENTATION AND SCOUR IN CHANNELS	QUALITY REACTIONS IN CHANNELS
DORSCH CONSULT QQS - MODEL	4 ARBITRARY CONSERVATIVE CONSTITUENTS	DIURNAL PATTERNS CALCULATED FROM LAND USE	NONLINEAR FUNCT. OF CATCHM. CHAR., POLL. ACCUMUL. AND RUNOFF	NO	ADVECTION WITH PARTIAL MIXING BETWEEN SUCCESSIVE TIME STEPS	NO	NO

	QUALITY ROUTING THROUGH STORAGE	QUALITY REACTIONS IN STORAGE	TREATMENT FACILITIES	QUALITY INTERACTIONS DURING TREATMENT	QUALITY BALANCE BETWEEN STORMS	RECEIVING WATER FLOW SIMULATION	RECEIVING WATER QUALITY SIMULATION
DORSCH CONSULT QQS - MODEL	PLUG FLOW	NO	TABULAR FUNCTION OF FLOW AND CONCENTRATION	NO	FUNCTION OF POLLUTANT ACCUMULATION AND STREET SWEEPING	YES	YES, BUT NO QUALITY REACTIONS AND INTERACTIONS

COMPARISON OF MISCELLANEOUS FEATURES OF MODELS (Table 5 in main text)

	CONTINUOUS SIMULATION	TIME INTERVAL	MIN. AND MAX. TIME PERIOD FOR SIMULATION	ALLOWS INPUT OF INITIAL CONDITIONS	DESIGN COMPUTATIONS	REAL-TIME CONTROL	TESTED ON URBAN DATA AND APPLIED TO REAL PROBLEMS
DORSCH CONSULT QQS - MODEL	YES	CONSTANT INTERVALS IN MINUTES	NO MIN., MAX. 2×10^6 TIME STEPS	YES	NO	NO	LIMITED TESTING AND APPLICATIONS

	ERROR MESSAGES	PRINCIPAL PRINTED OUTPUT	PRINCIPAL GRAPHIC OUTPUT	UNITS OF MEASUREMENT	COMPUTER VERSIONS AND CORE STORAGE REQUIREMENT	COMPUTER LANGUAGE	COMPUTER PROGRAM AVAILABLE
DORSCH CONSULT QQS - MODEL	EXTENSIVE, PLUS SEPARATE DATA CHECKING PROGRAM	TABLES OF STATISTICAL ANALYSIS	FREQUENCY AND DURATION CURVES	METRIC	UNIVAC 1108, 400 K BYTES	FORTRAN IV, 18 000 STATEMENTS	UNDER USE AGREEMENT

* PROVIDED AS INPUT DATA

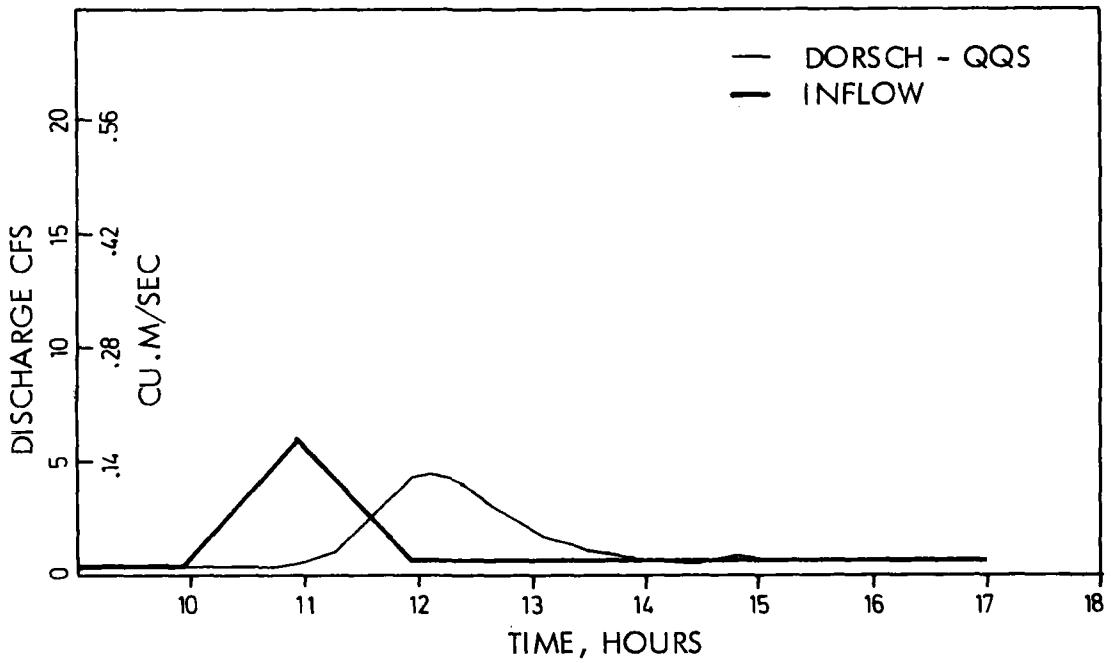


Figure 1: Outflow for Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.05 % Slope

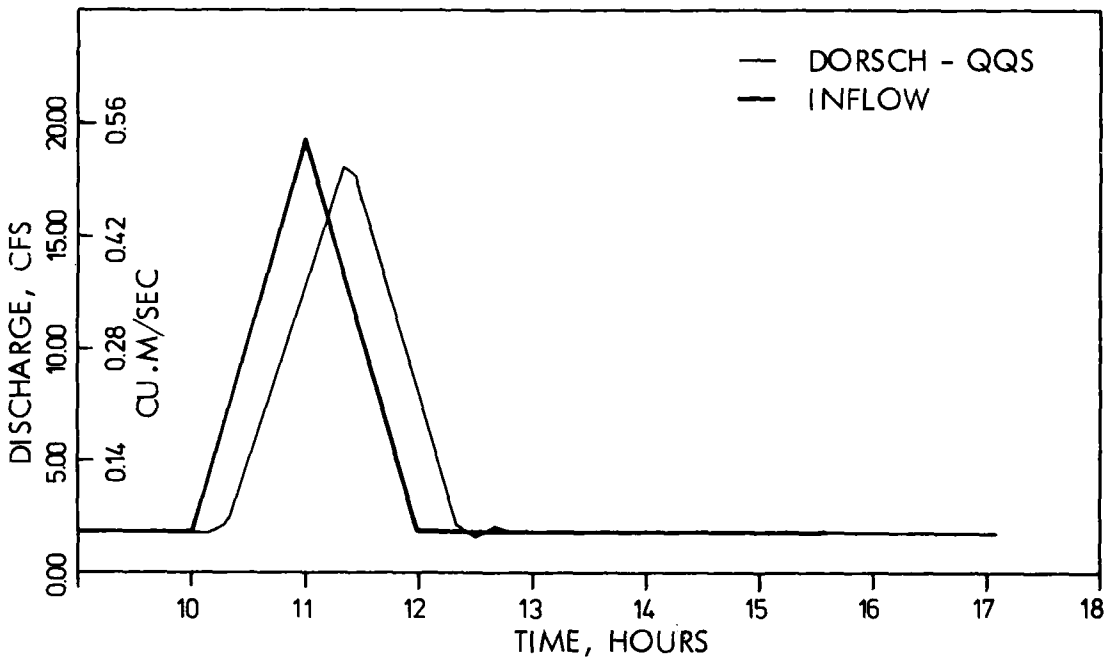


Figure 2: Outflow for Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope

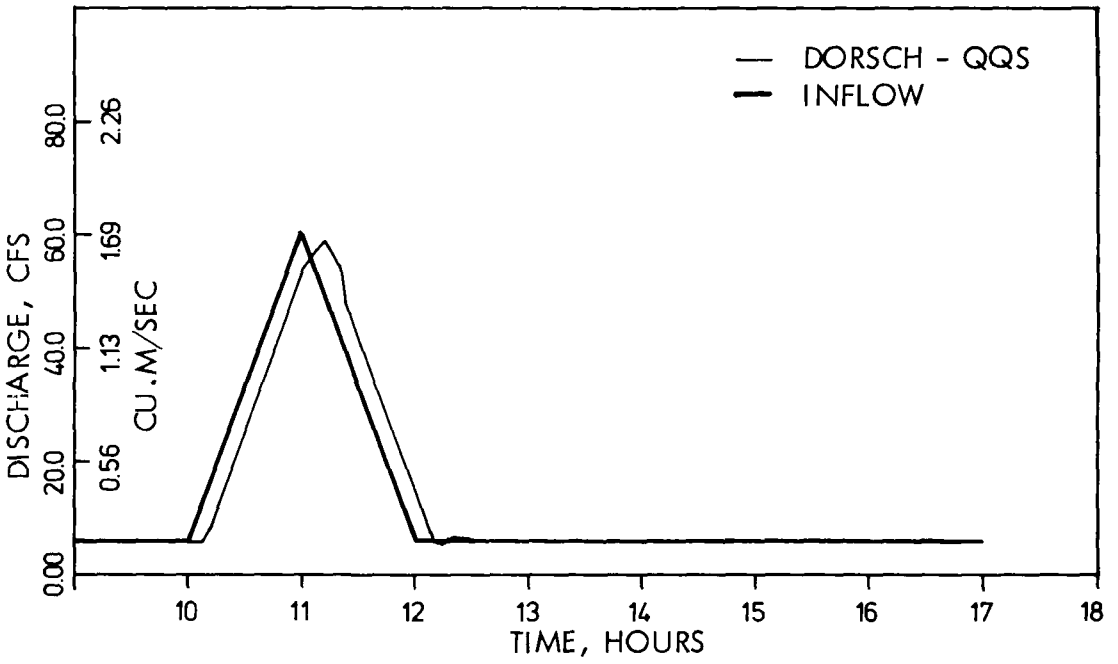


Figure 3: Outflow for Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 5 % Slope

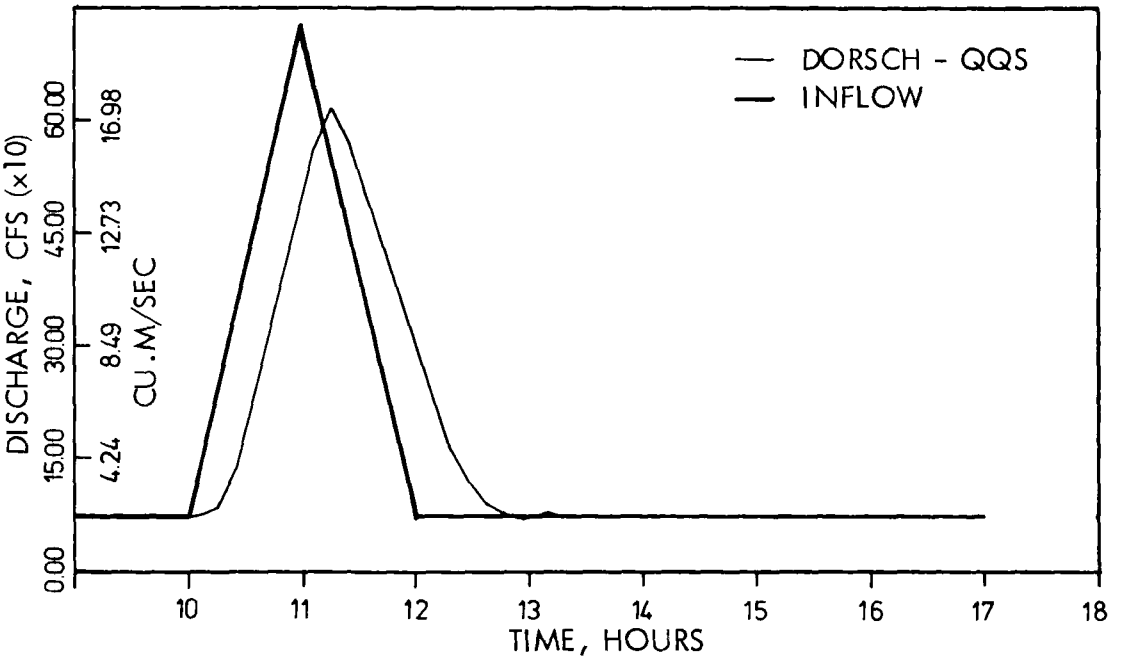


Figure 4: Outflow for Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.05 % Slope

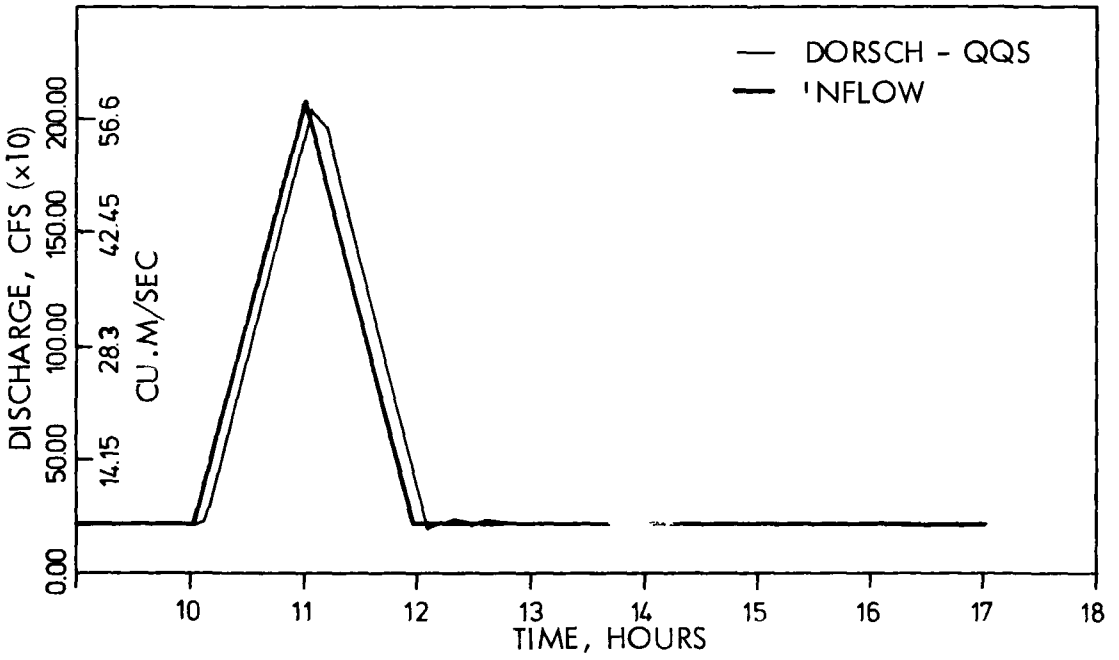


Figure 5: Outflow for Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope

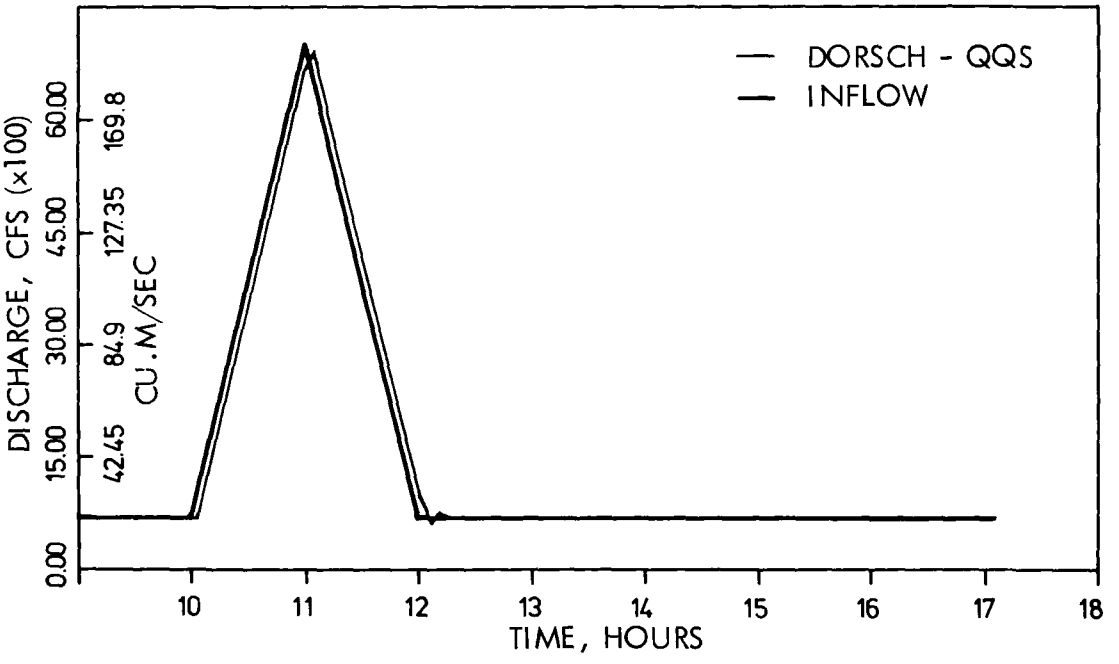


Figure 6: Outflow for Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 5 % Slope

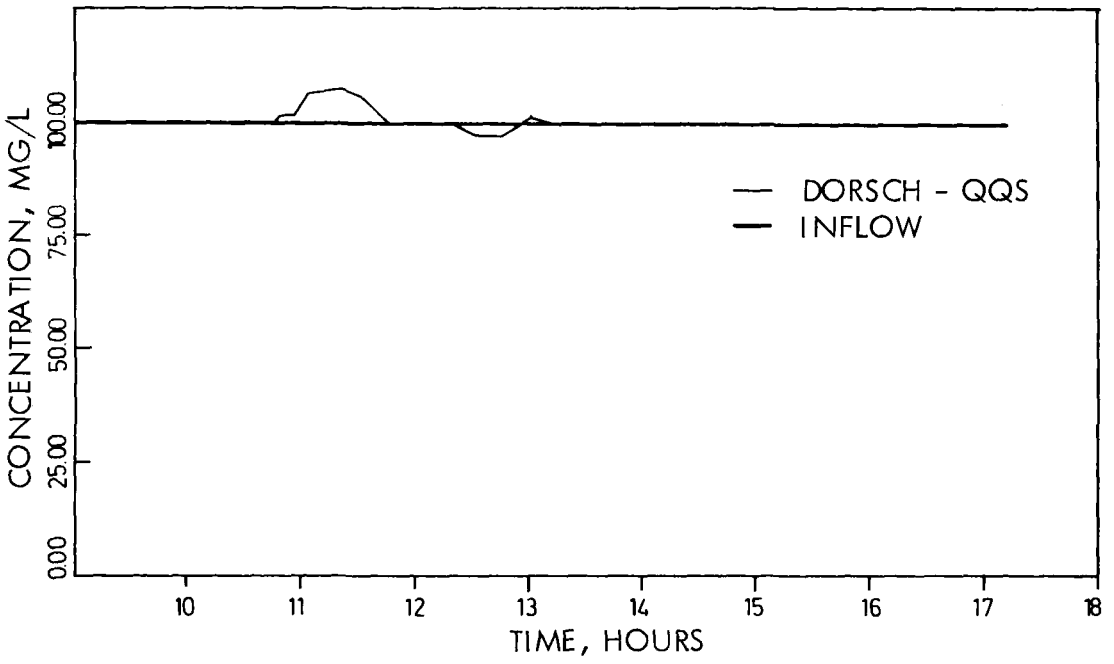


Figure 7: Outflow Concentration for Constant Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.05 % Slope

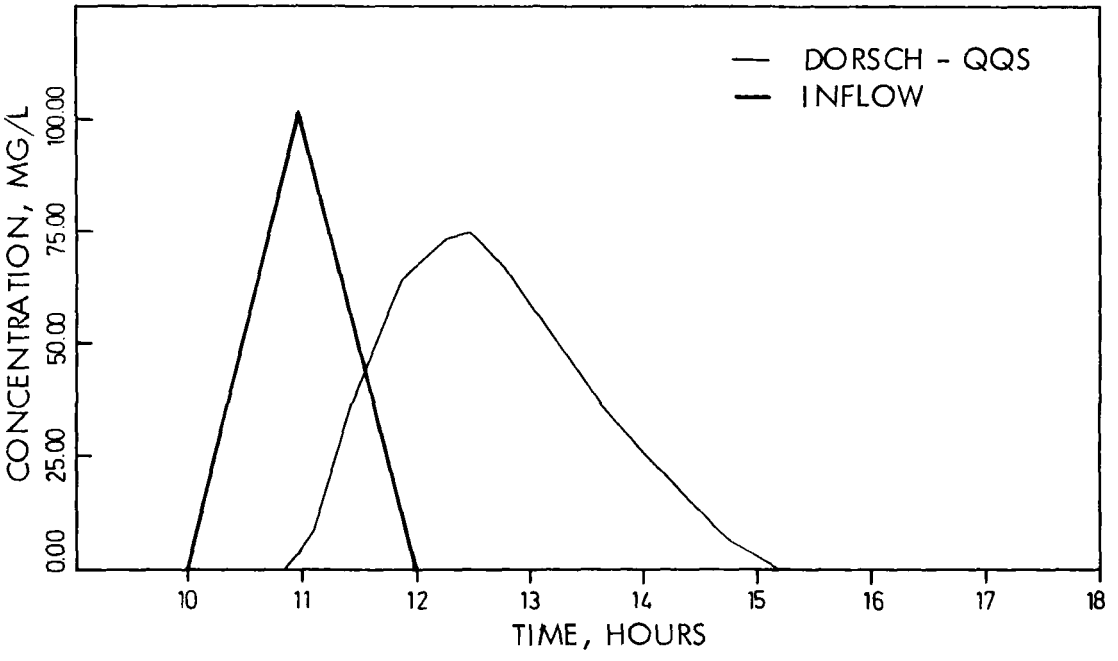


Figure 8: Outflow Concentration for Triangular Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.05 % Slope

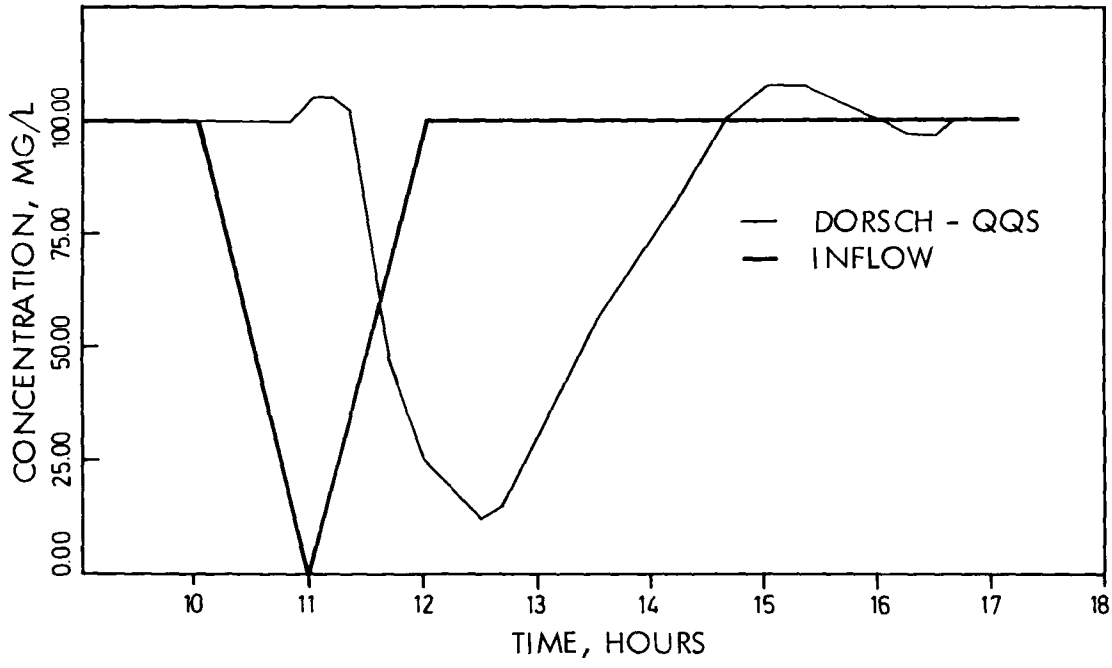


Figure 9: Outflow Concentration for Inverted Triangular Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.05 % Slope

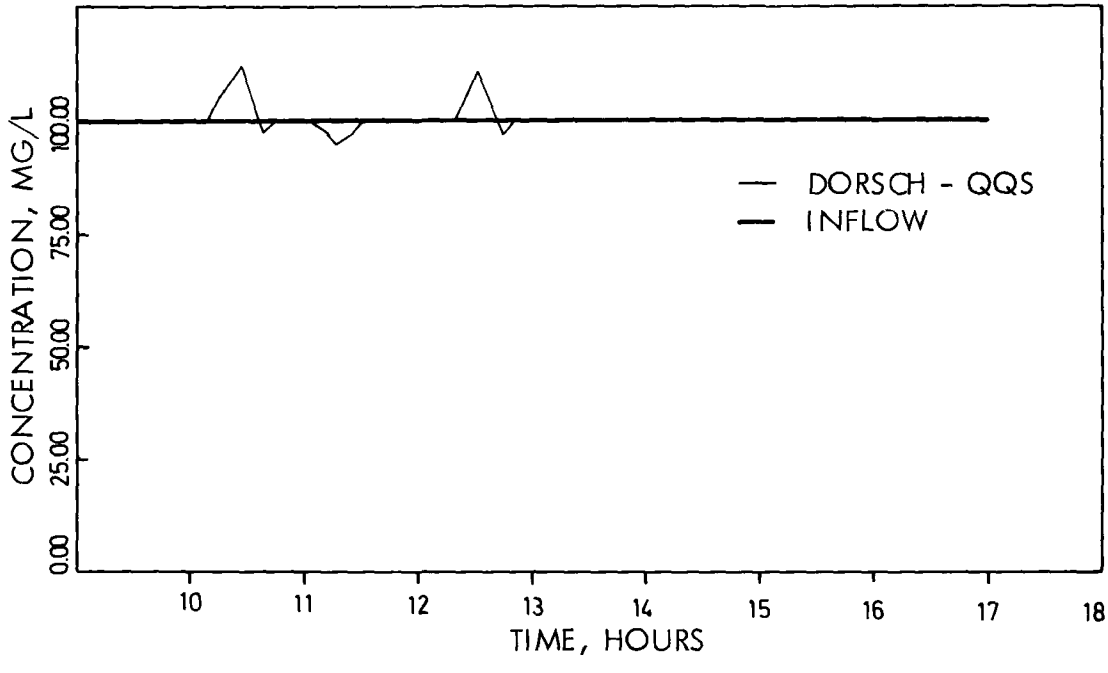


Figure 10: Outflow Concentration for Constant Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope

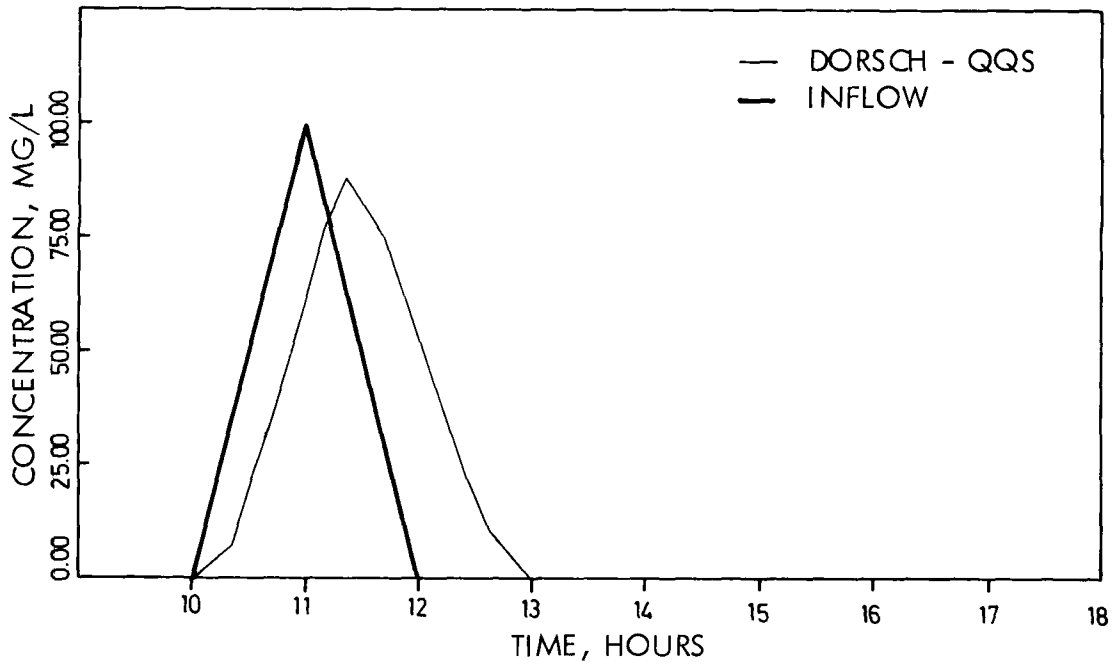


Figure 11: Outflow Concentration for Triangular Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope

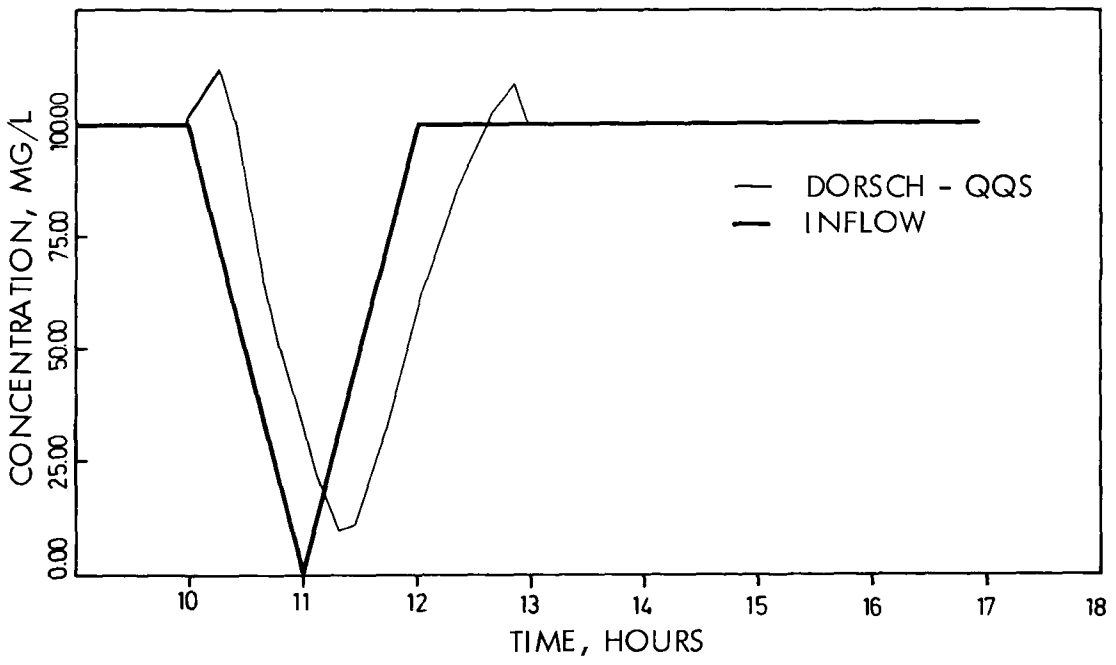


Figure 12: Outflow Concentration for Inverted Triangular Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope

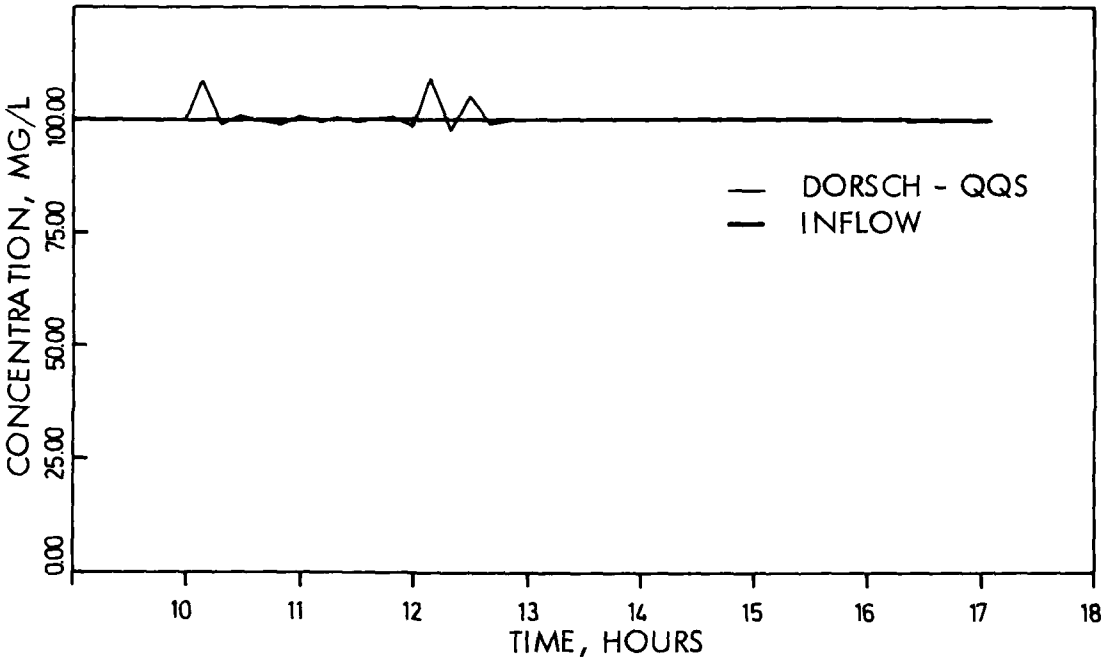


Figure 13: Outflow Concentration for Constant Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 5 % Slope

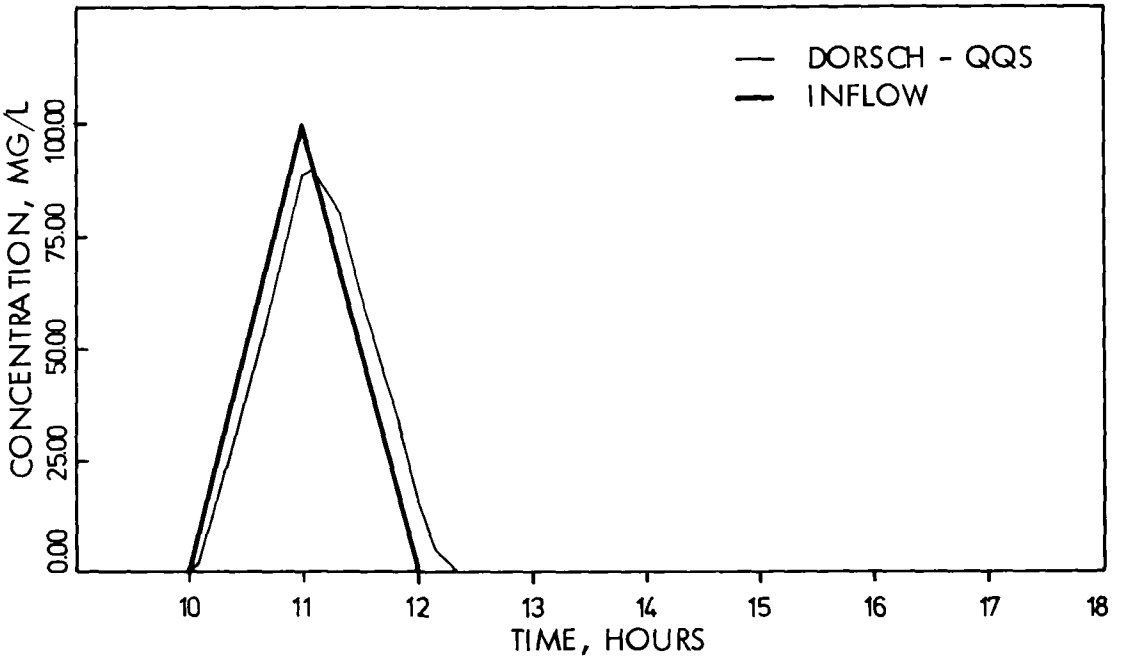


Figure 14: Outflow Concentration for Triangular Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 5 % Slope

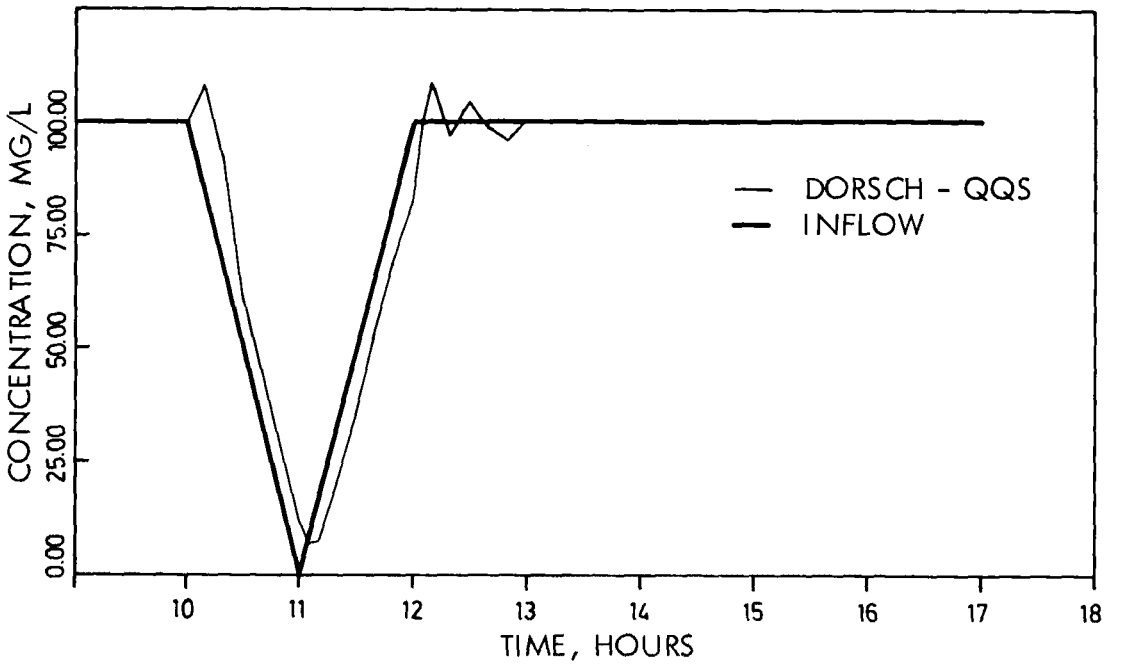


Figure 15: Outflow Concentration for Inverted Triangular Inflow Concentration with Two-Hour Triangular Inflow - Small Hypothetical Pipe, Free Inflow and Outflow, 5 % Slope

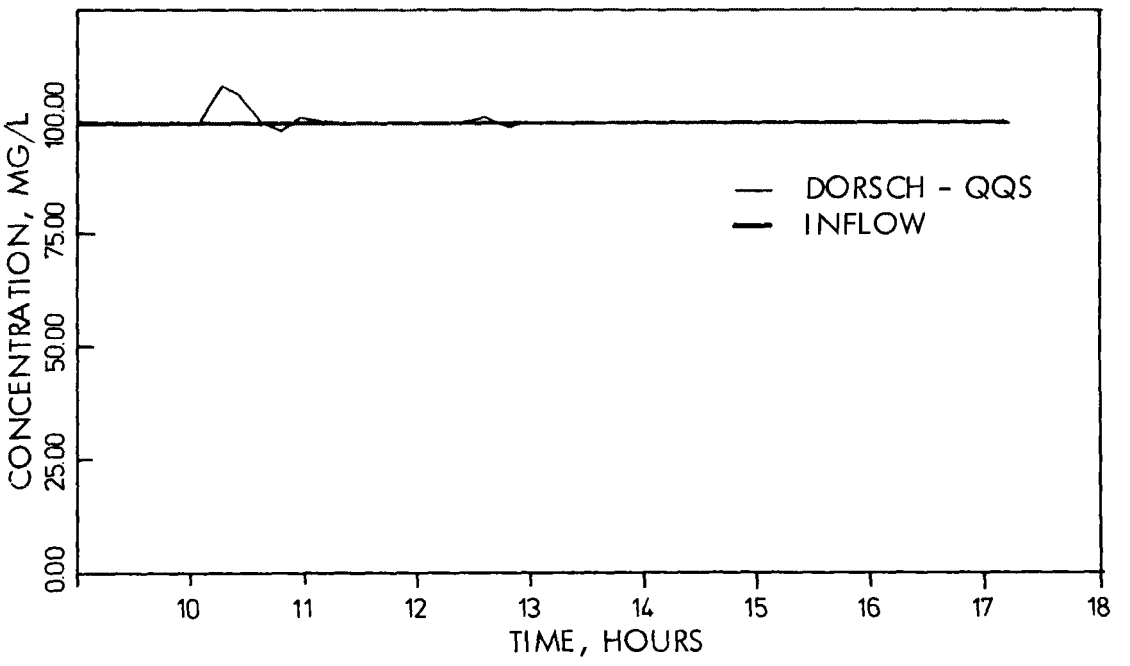


Figure 16: Outflow Concentration for Constant Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.05 % Slope

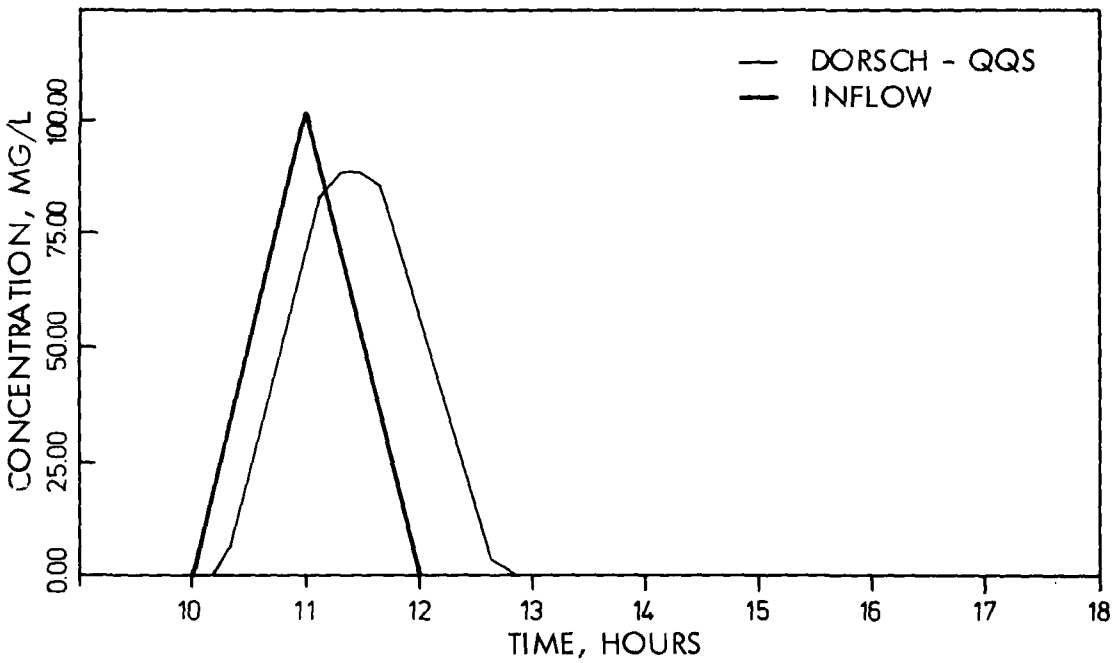


Figure 17: Outflow Concentration for Triangular Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.05 % Slope

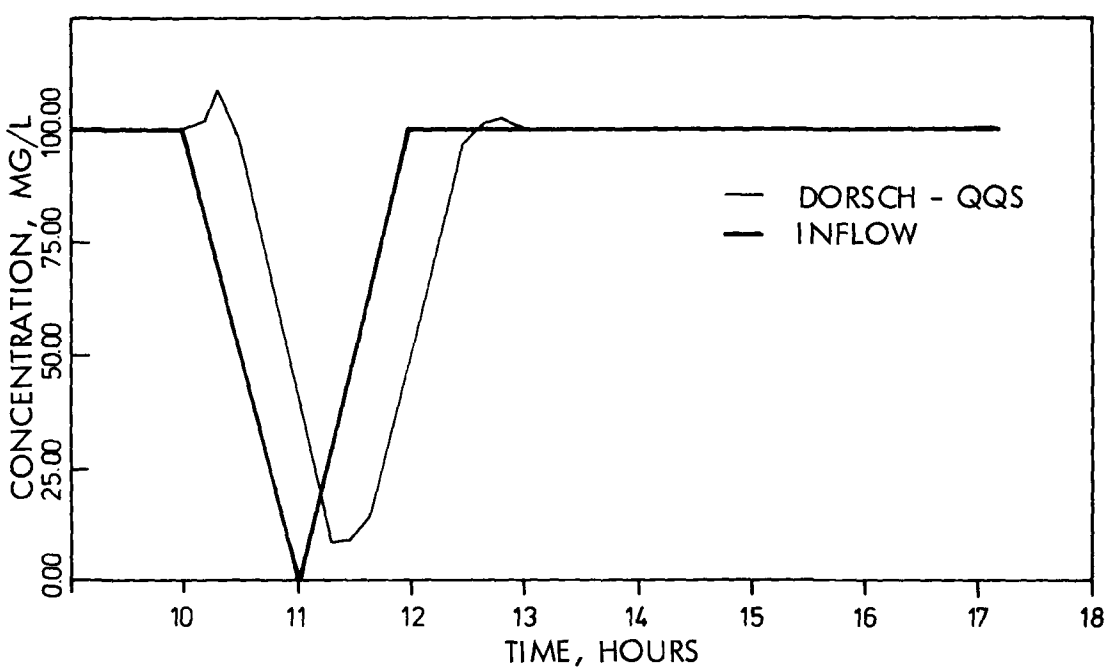


Figure 18: Outflow Concentration for Inverted Triangular Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.05 % Slope

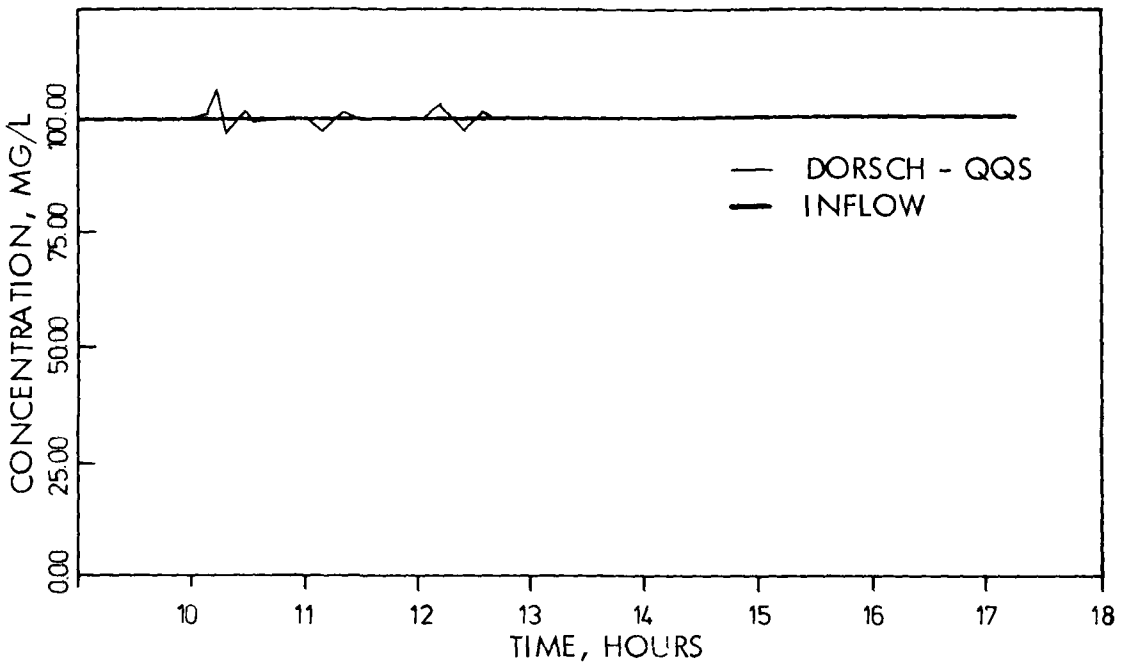


Figure 19: Outflow Concentration for Constant Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope

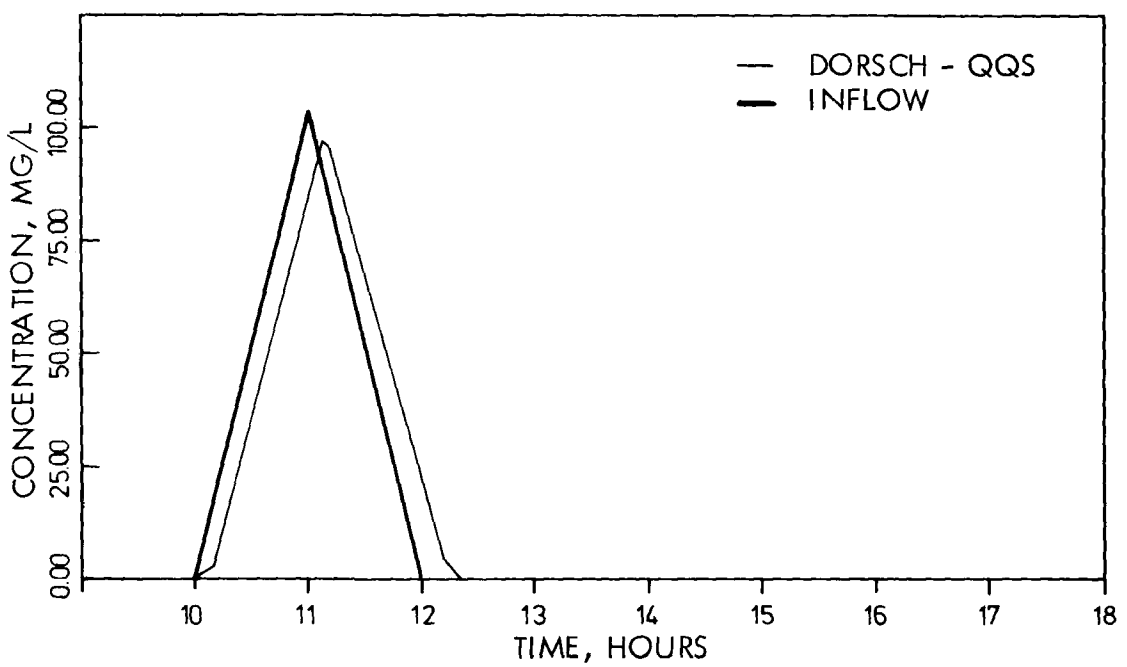


Figure 20: Outflow Concentration for Triangular Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope

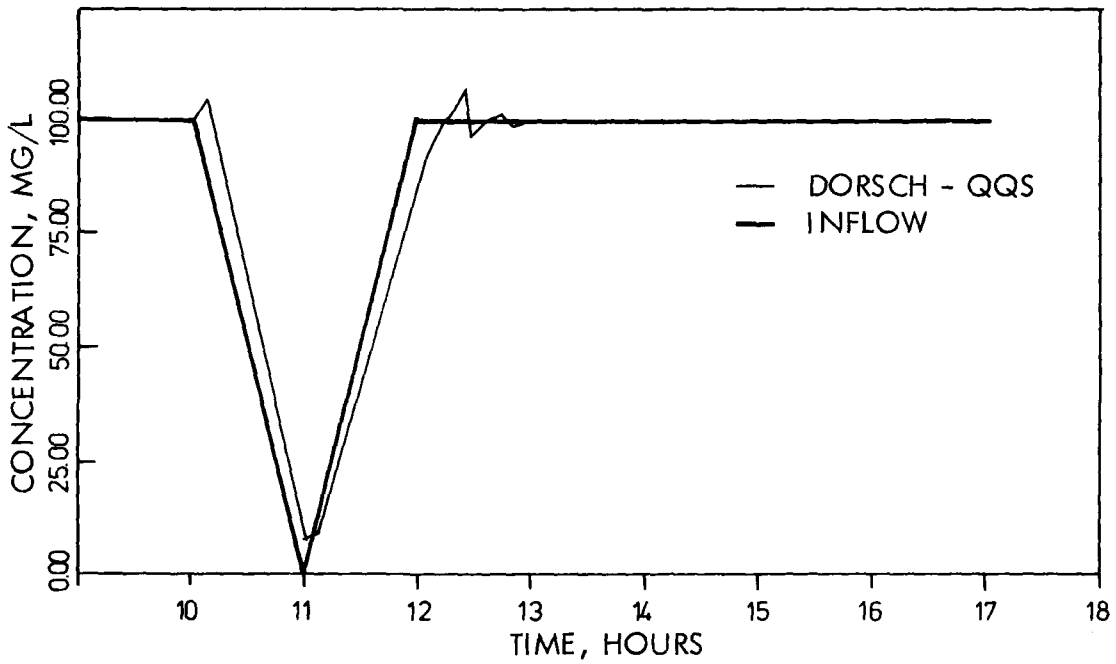


Figure 21: Outflow Concentration for Inverted Triangular Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 0.5 % Slope

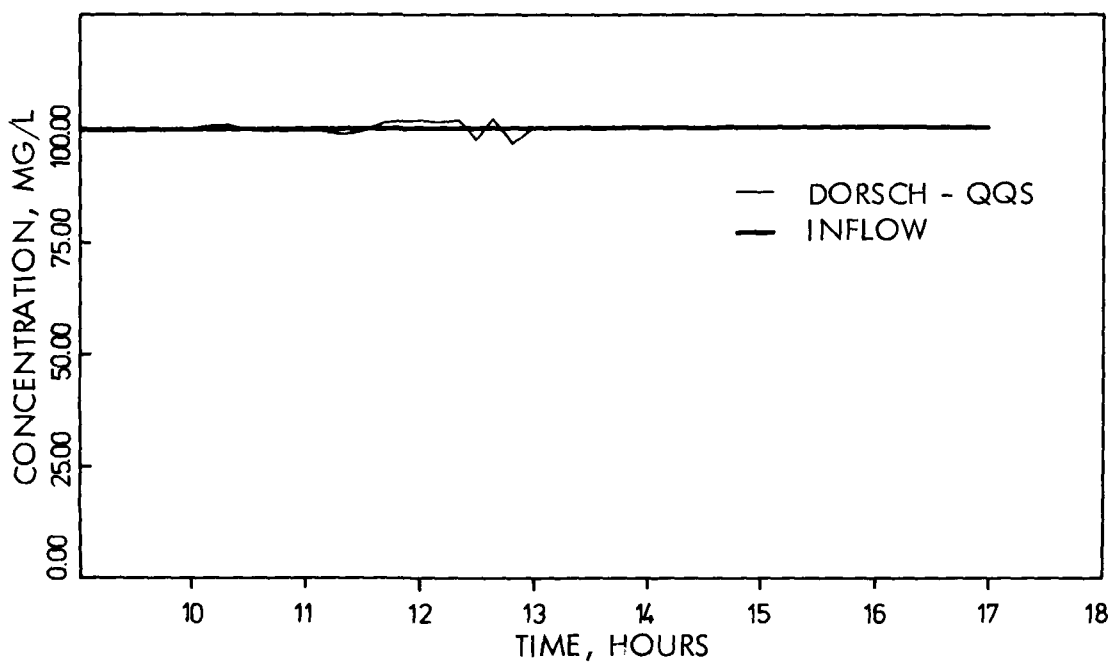


Figure 22: Outflow Concentration for Constant Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 5 % Slope

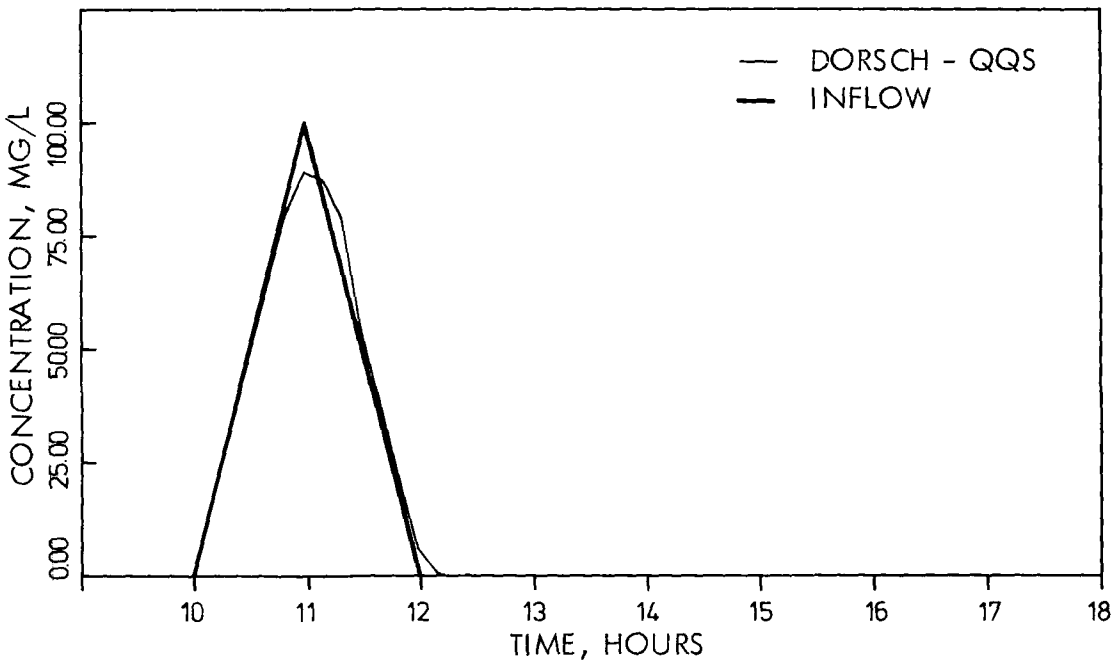


Figure 23: Outflow Concentration for Triangular Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 5 % Slope

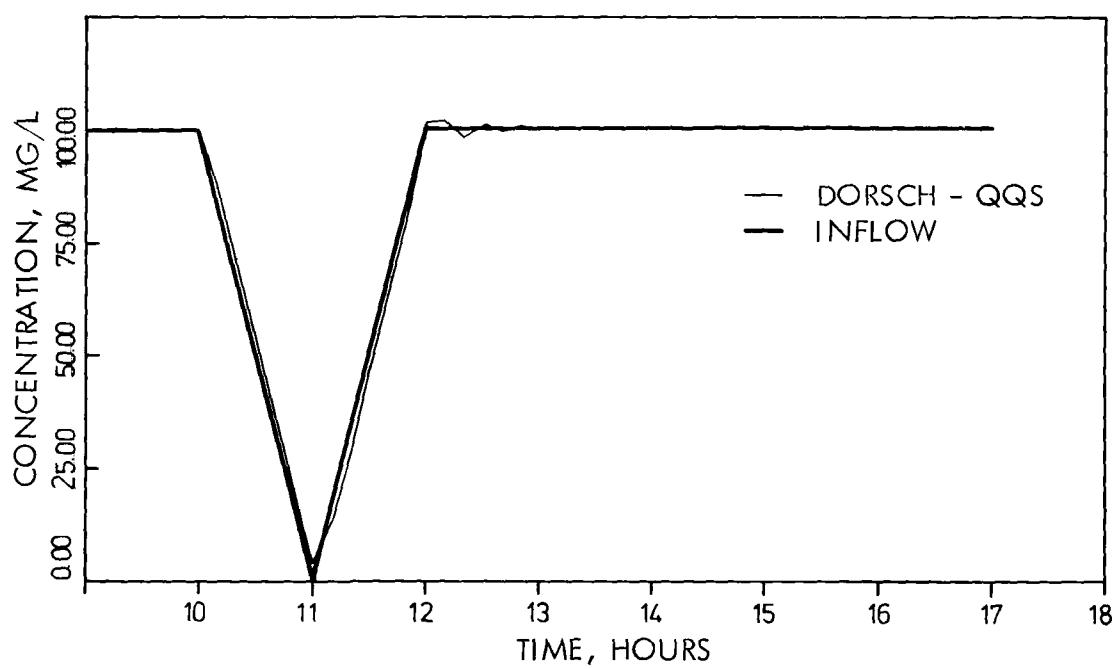


Figure 24: Outflow Concentration for Inverted Triangular Inflow Concentration with Two-Hour Triangular Inflow - Large Hypothetical Pipe, Free Inflow and Outflow, 5 % Slope

WRE WATER RESOURCES ENGINEERS
a Systems Associates Company

August 14, 1975

Dr. Albin Brandstetter
Research Associate
Water and Land Resources Department
Pacific Northwest Laboratories
Battelle Memorial Institute
P. O. Box 999
Richland, Washington 99352

Dear Dr. Brandstetter:

Subsequent to our submittal of the hypothetical pipe simulations, we discovered the following problems in the water quality simulation model:

1. The time step used in these simulations was too long (the criteria is $\Delta t < L/2$, where L is the length of pipe, or distance between adjacent nodes). When this situation occurs, the mass balance is no longer maintained.

2. We discovered a programming error in the computation of the volume under the drawdown profile of the water surface at free outfalls. Thus computed qualities at these points were in error.

3. We found another programming error in the computation of volumes at storage nodes when the storage node is full at the start of the simulation. (This situation occurs for the case of the constant inflow hydrograph.)

Fortunately (or perhaps unfortunately, since we would have discovered these problems earlier), previously reported comparisons of computed results to measured results on prototype systems involved systems or points of measurement which were not affected by the two errors we discovered, and the time step used in these comparisons was proper.

With your kind permission, we have rerun all simulations, correcting the errors discovered in the initial runs. Tables 68 through 71 in the main text, which summarize the results of the water quality simulations, have been revised from the values reported in the draft report to show the correct simulation results. Outflow concentration plots shown in Figures 49 through 66 of the main text are shown for the WRE model on the following pages. SWMM results for these cases are shown also for comparison. Note that we have shown the concentration curves for the case of storage and diversion also in these figures.

710 SOUTH BROADWAY / WALNUT CREEK, CALIFORNIA 94596 / TEL (415) 933-4500
Walnut Creek, California / Springfield, Virginia / Austin, Texas

With respect to the hydraulic model, some significant improvements have been made during its use and application since the original simulations were performed. One of the most significant changes has been a change in the stability criteria which allows longer time steps to be used. Since the hydraulic simulations had to be rerun to generate input data for the quality model, the improved hydraulic model was used. Tables 75 and 76 in the main text have been revised to reflect the run times with the refined model.

In Hamburg, West Germany, the WRE model has been used by KOCKS, Engineers, Germany, and WRE to develop SESIM (Stadt-Entwässerungs-Simulationsmodell). The most important changes are as follows:

SURFACE RUNOFF MODEL

- Contribution of impervious areas to runoff:
  An inputted portion of the impervious areas (e.g. 30 percent) is allowed to create surface runoff immediately after rain start. This portion is increased exponentially until the surface retention is satisfied.

- Capacity of program:
  The basic capacity has been increased to 450 catchment areas. An additional program can mix up to 3,150 runoff hydrographs to create a unique input file to the sewer transport model.

SEWER TRANSPORT MODEL

- Computer time:
  The necessary computer time has been greatly reduced by:

  (1) Converting very short pipes to orifices and thus avoiding offenses against the Courant-criterium. Thus, larger time steps may be used.

  (2) Optional skipping of entry and exit loss computations if they are included in the roughness coefficients. This by itself saves 39 percent of the Sewer Transport central processing time.

  (3) Optional dampening of large oscillations supports convergence of weir flow, thus enabling larger time steps.

- Energy losses:
  Friction losses may be computed optionally with Manning's or Prandtl-Colebrook's (rough, turbulent) formula.

- Real-time control:
  To test the influence of real-time control the sewer transport model was given the capability of simulating local control of weirs.

- Capacity of program:
  The sewer transport model has been changed to handle up to 3,000 conduits and nodes by using background storage.

I wish to personally thank you for including this appendix. Your project has been one of the best I have seen from the standpoint of objectivity in reporting comparisons of models.

Sincerely yours,

WATER RESOURCES ENGINEERS, INC.

Larry A. Roesner

Larry A. Roesner, Ph.D.
Principal Engineer

LAR/djs

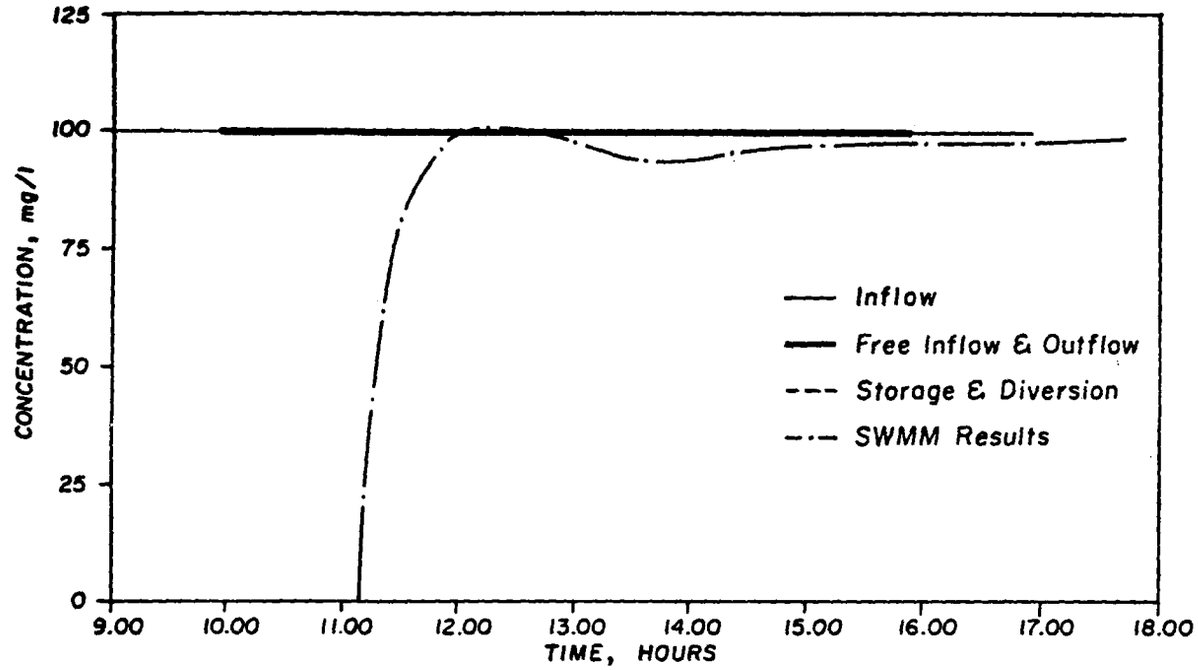


Figure 49. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, 0.05% Slope

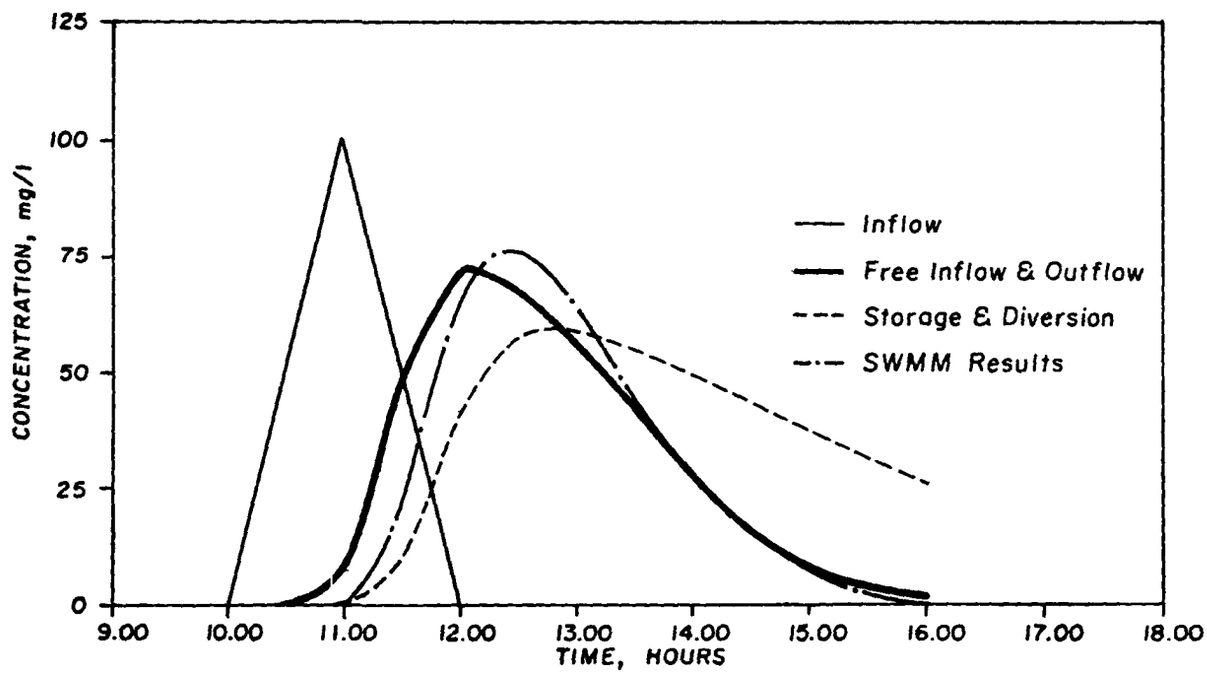


Figure 50. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, 0.05% Slope

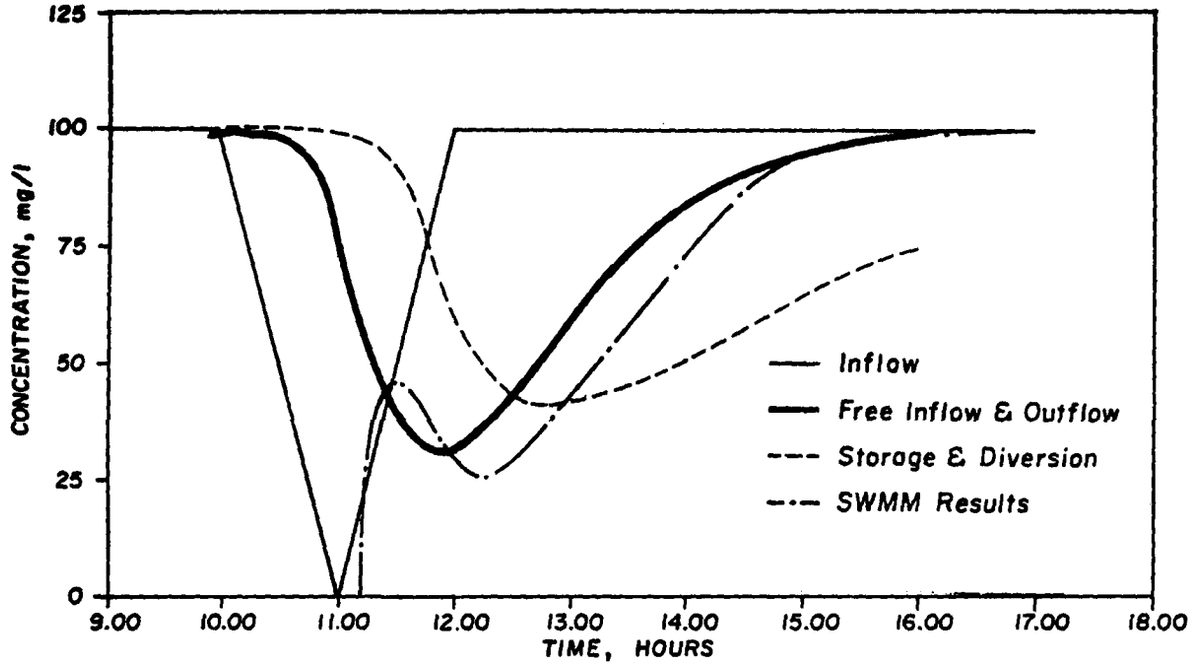


Figure 51. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, 0.05% Slope

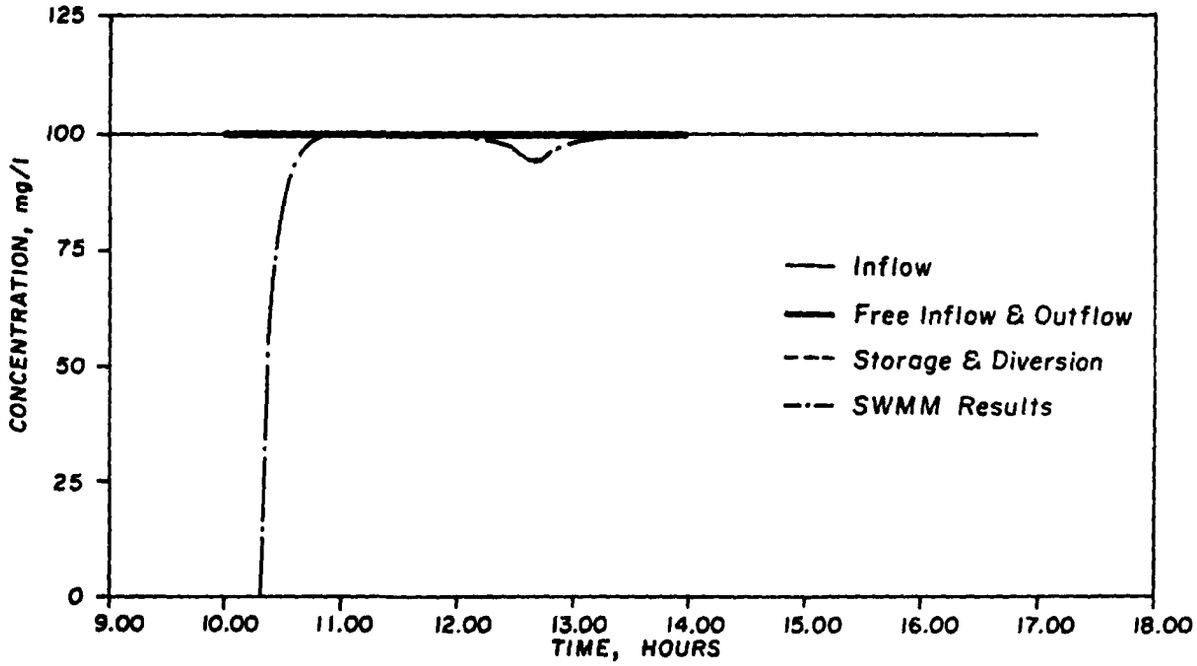


Figure 52. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, 0.5% Slope

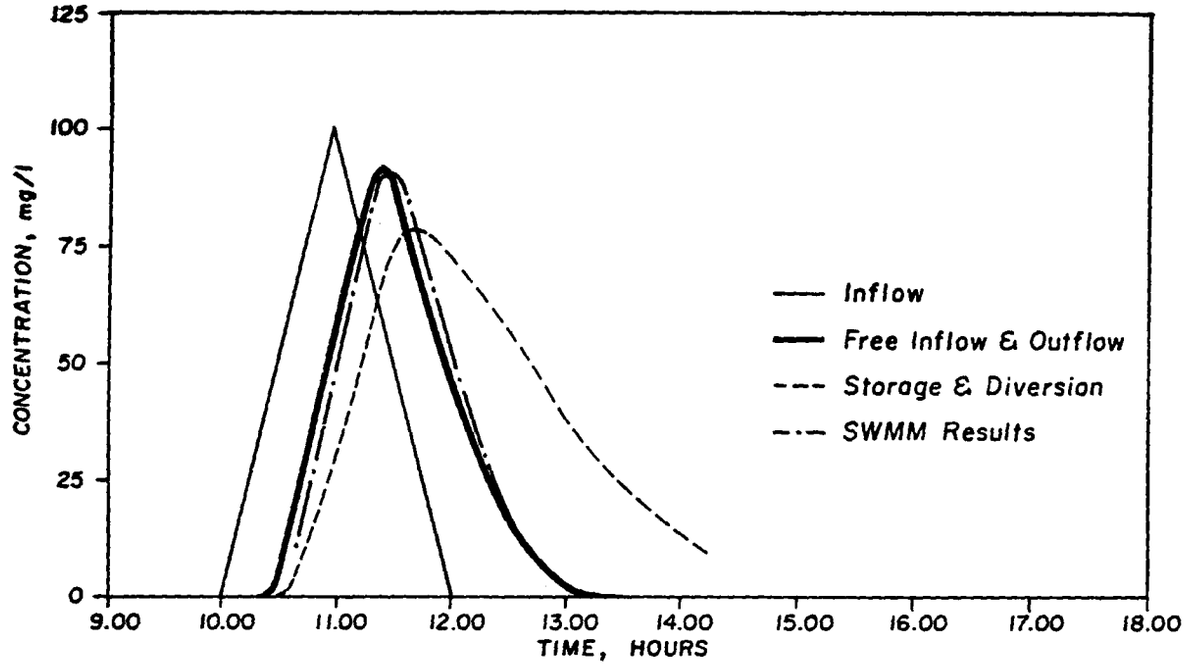


Figure 53. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, 0.5% Slope

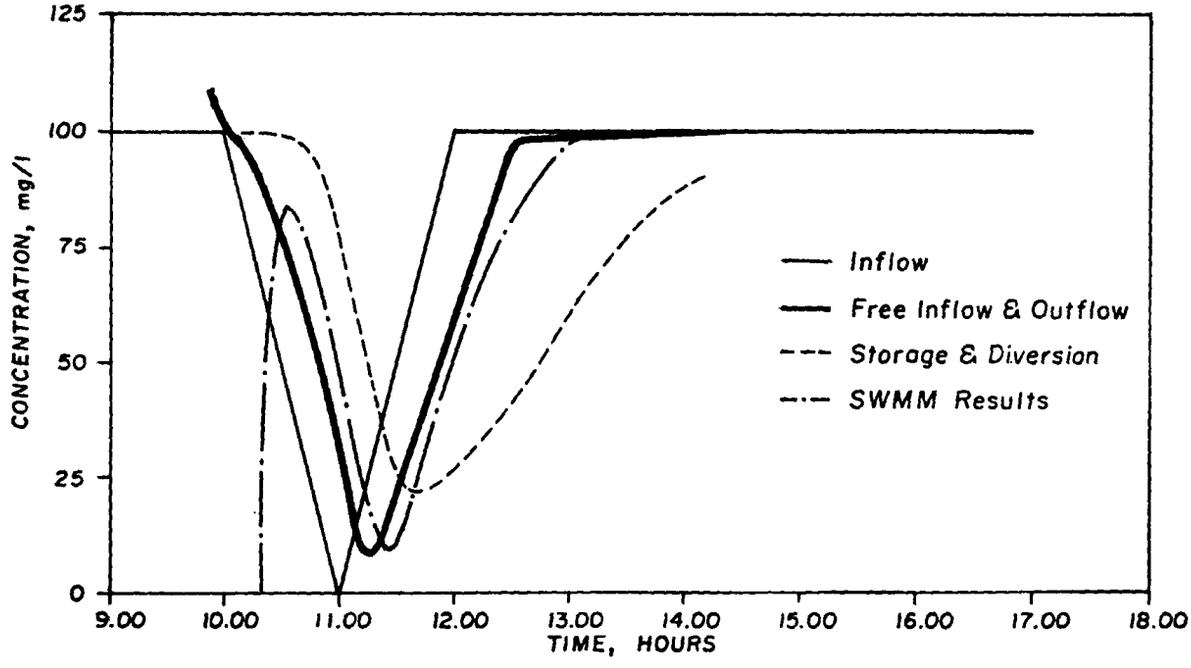


Figure 54. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, 0.5% Slope

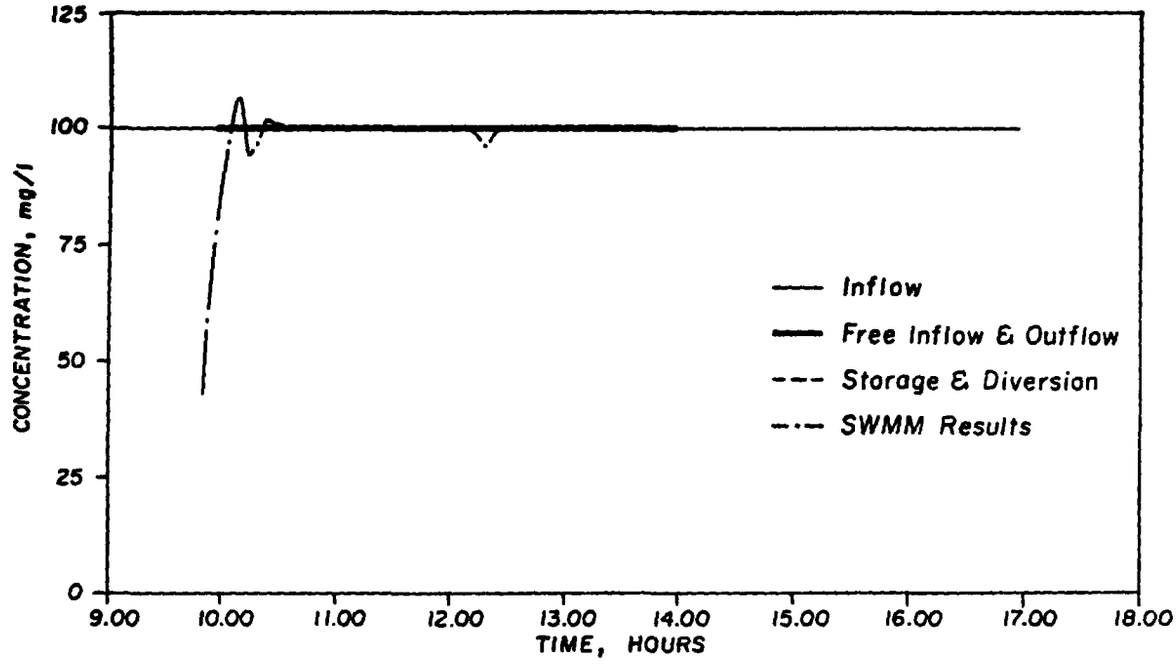


Figure 55. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, 5% Slope

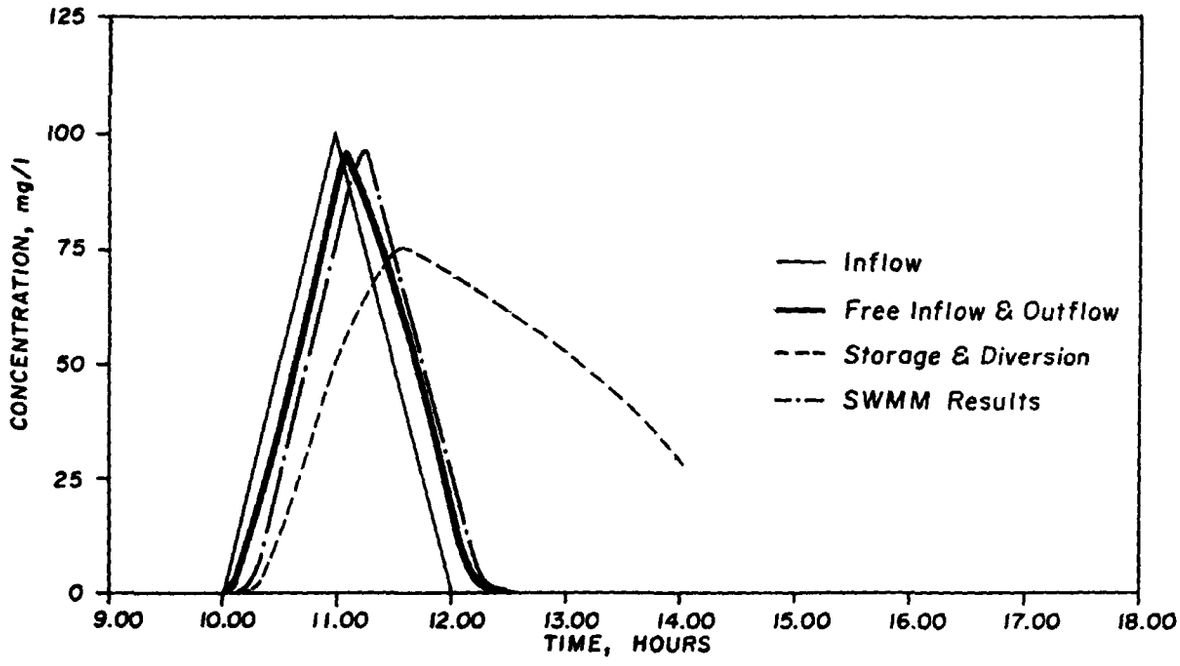


Figure 56. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow Small Hypothetical Pipe, 5% Slope

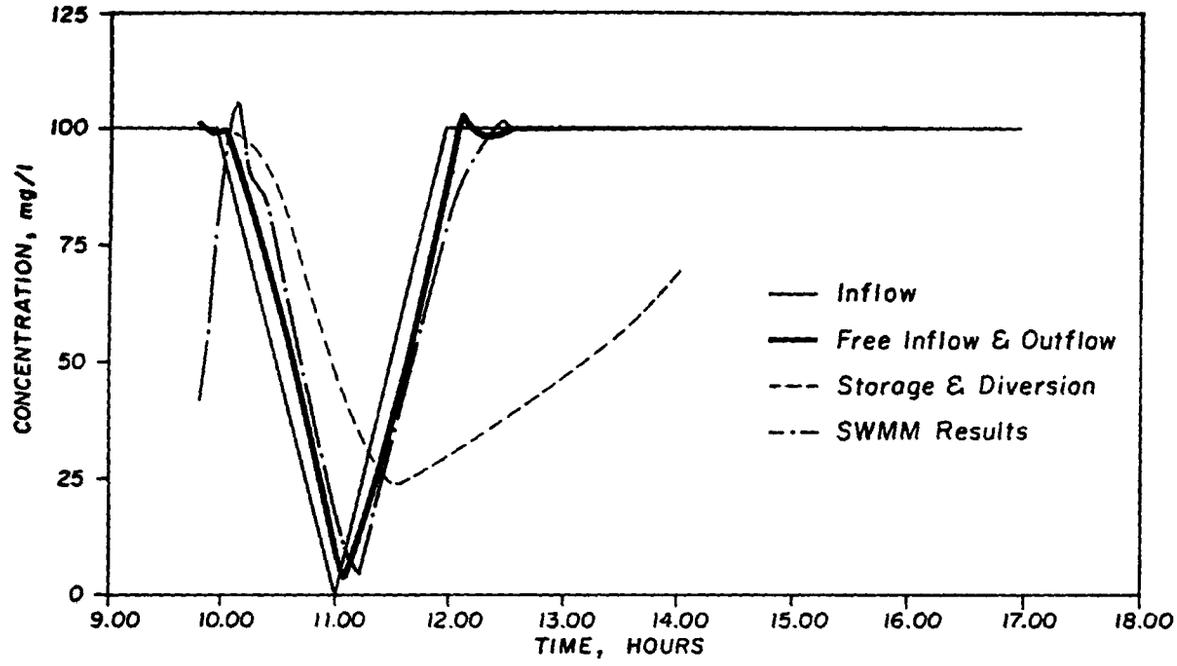


Figure 57. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Small Hypothetical Pipe, 5% Slope

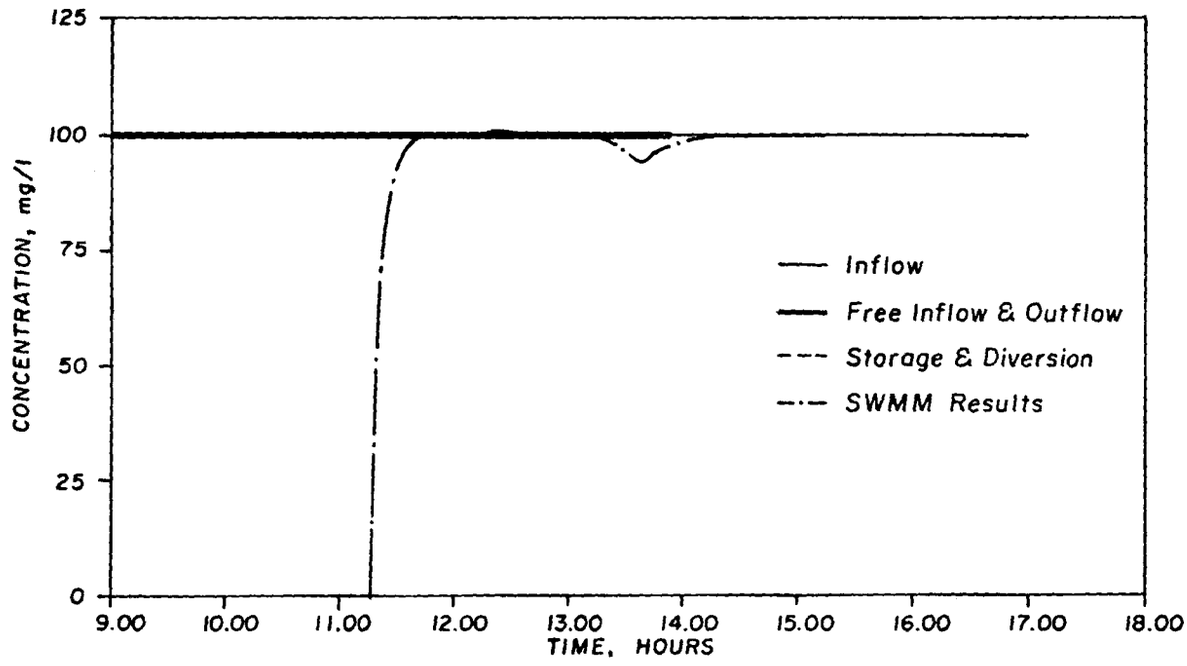


Figure 58. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 0.05% Slope

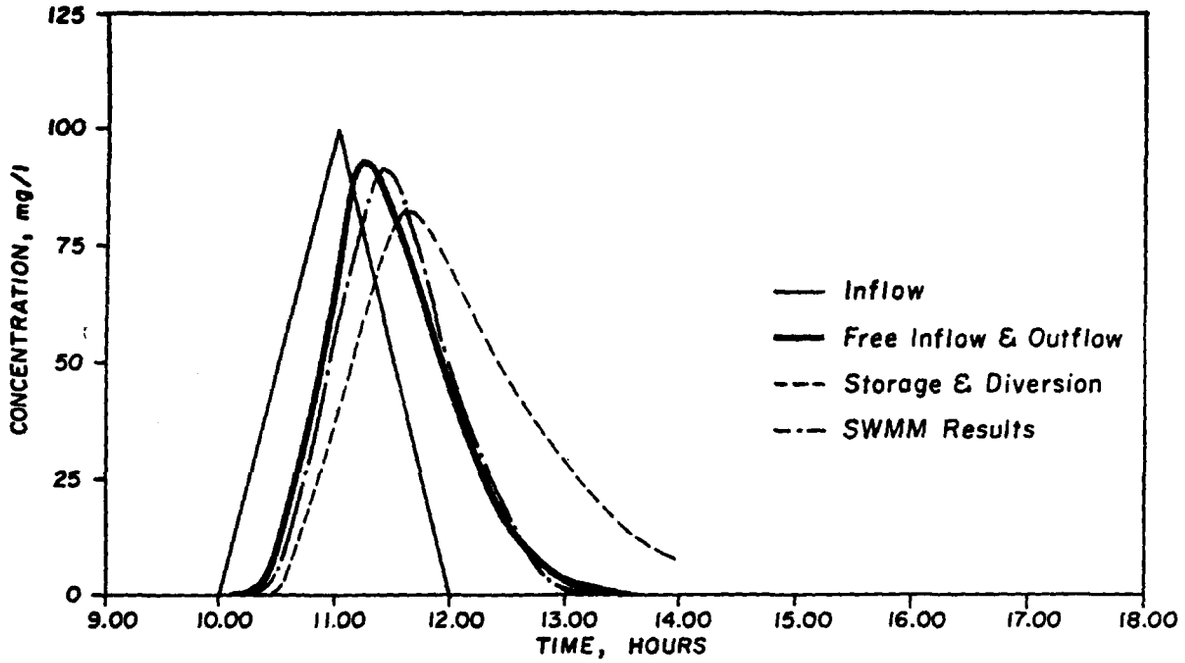


Figure 59. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 0.05% Slope

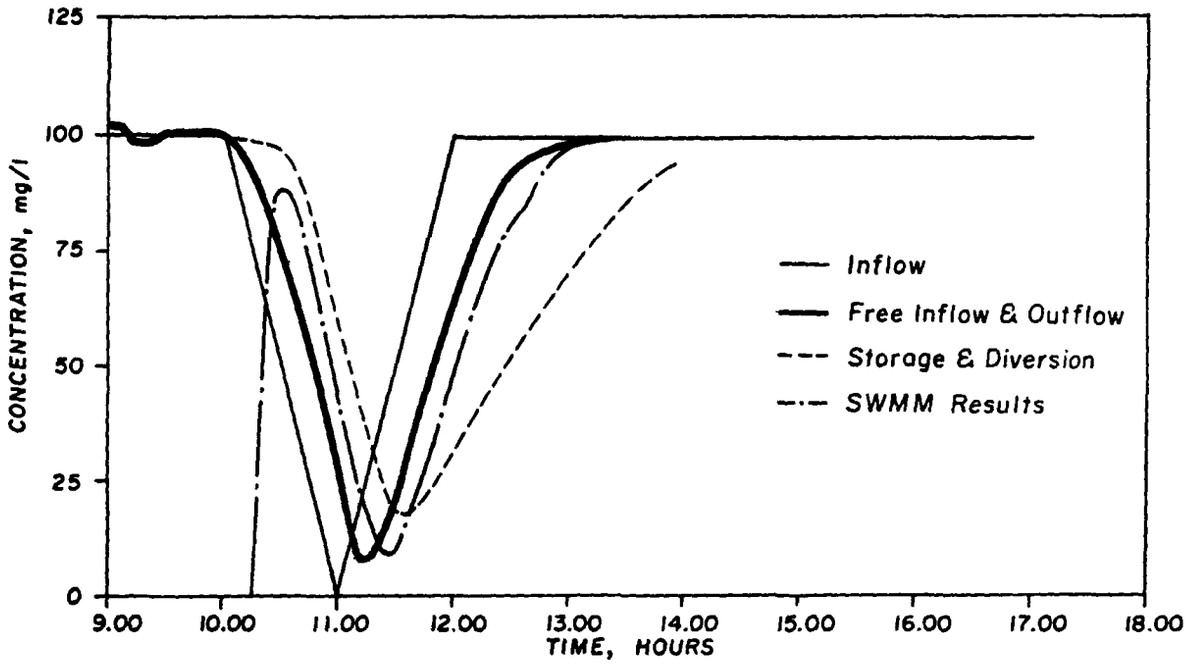


Figure 60. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 0.05% Slope

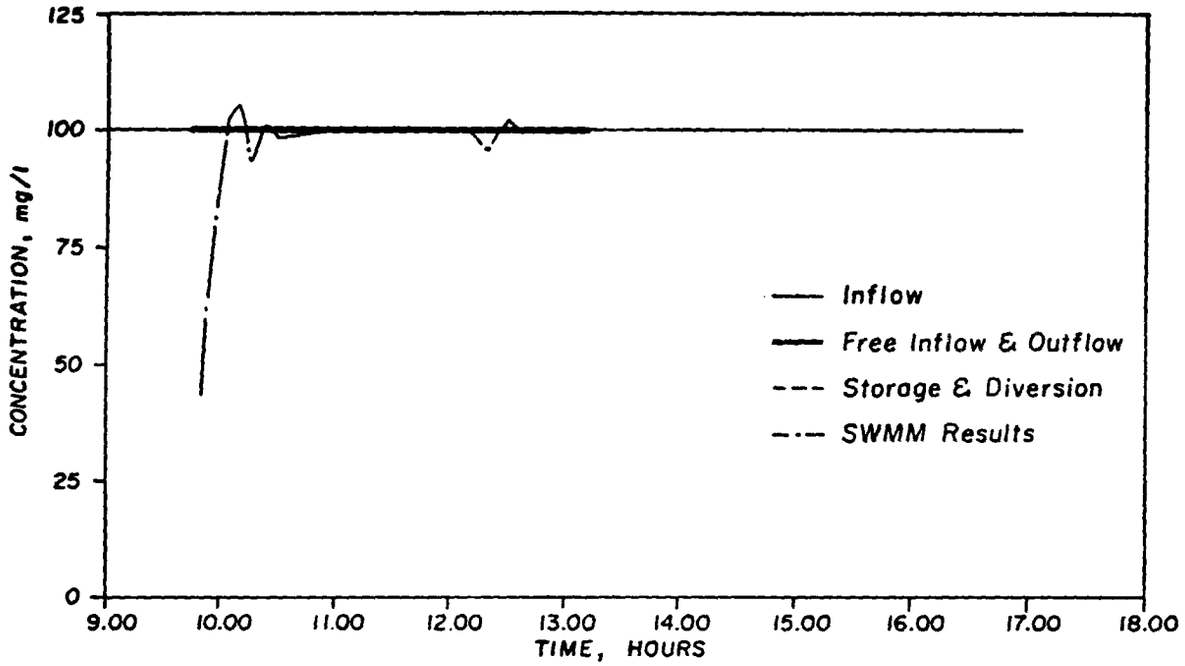


Figure 61. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 0.5% Slope

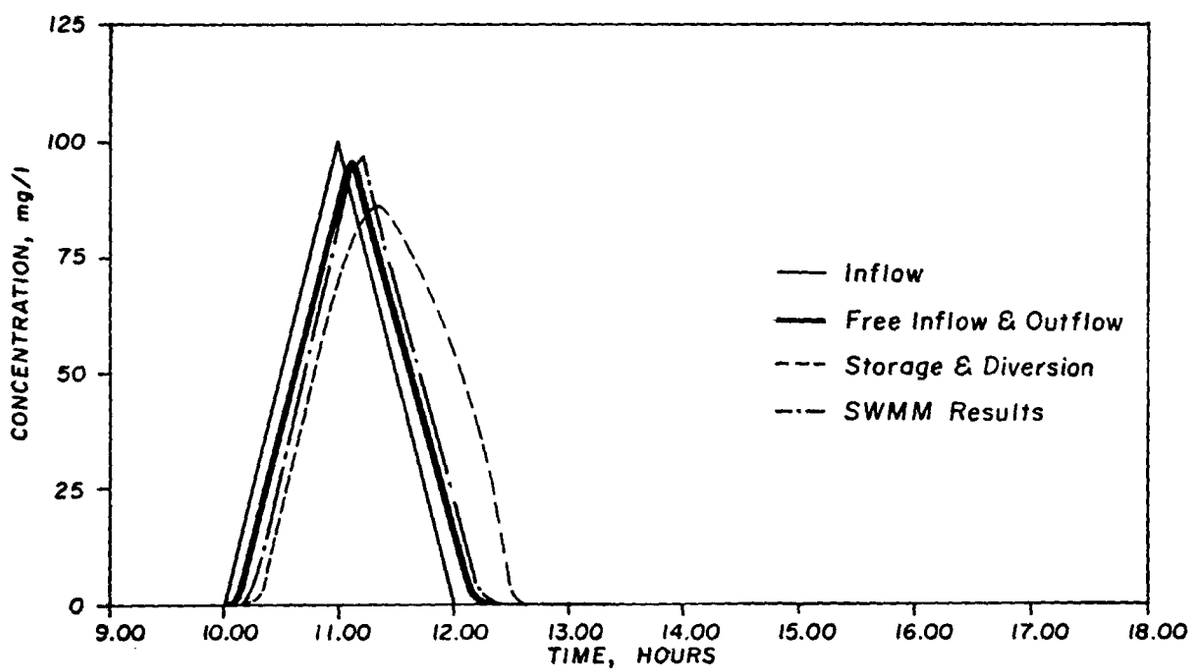


Figure 62. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 0.5% Slope

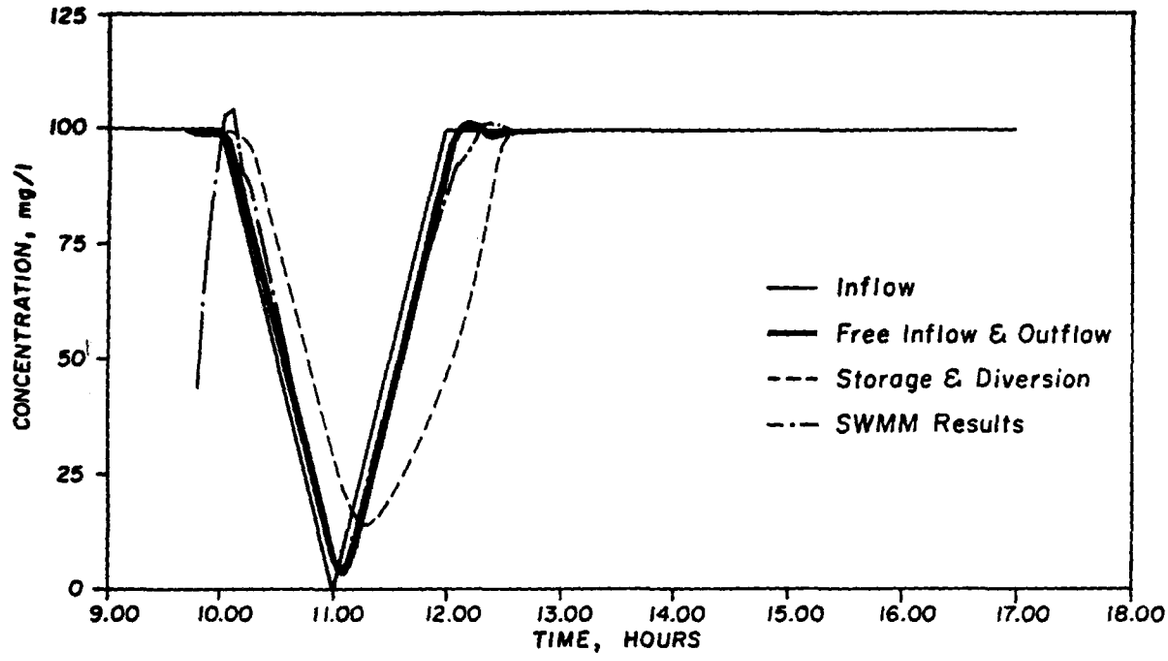


Figure 63. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 0.5% Slope

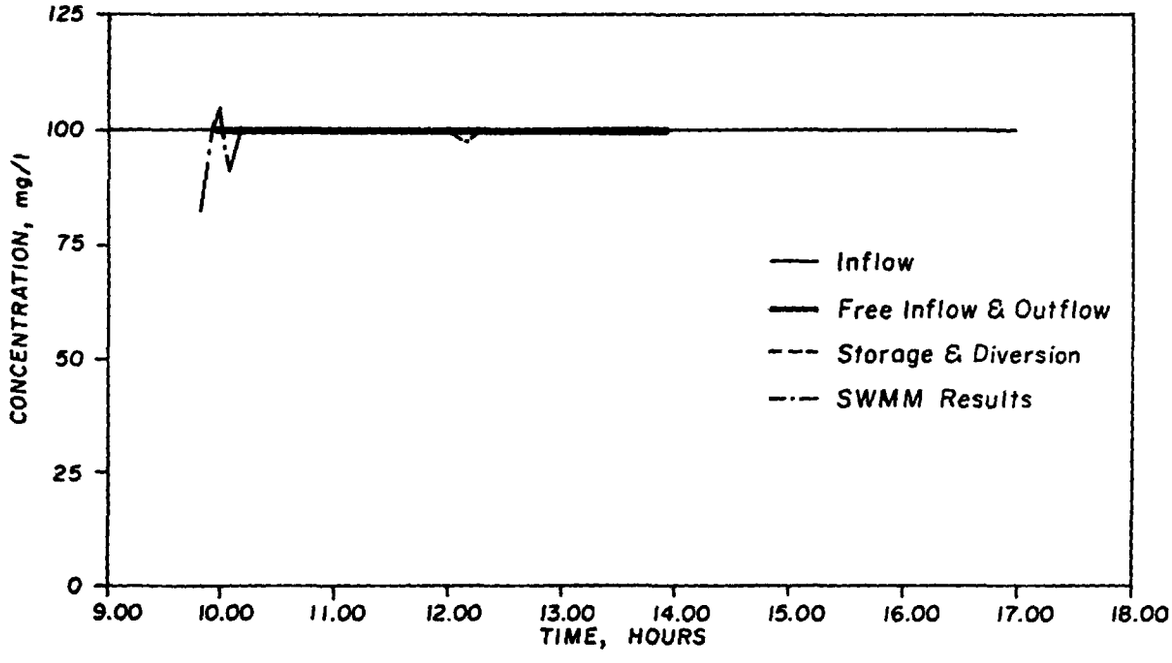


Figure 64. Outflow Concentrations for Constant Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 5% Slope

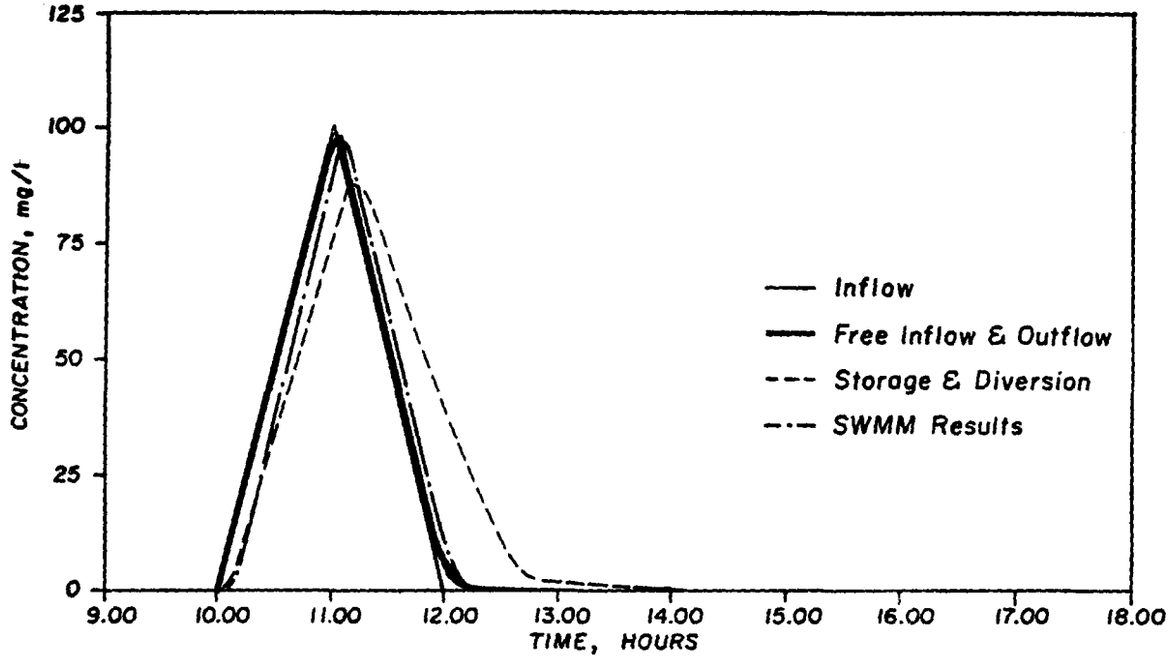


Figure 65. Outflow Concentrations for Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 5% Slope

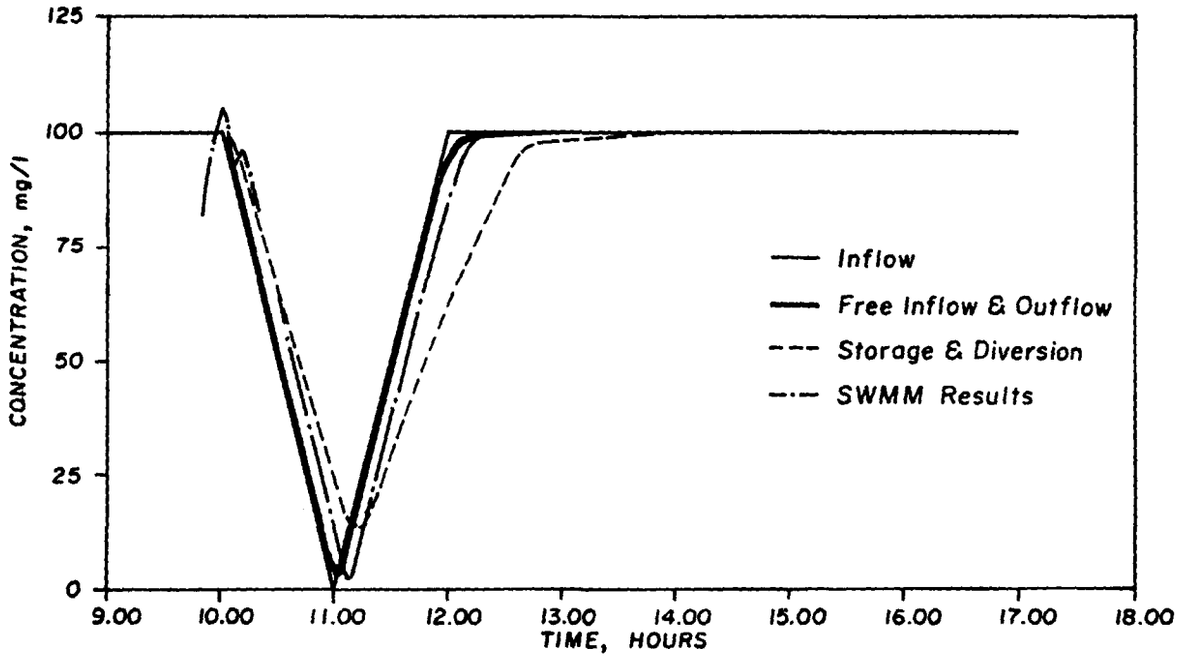


Figure 66. Outflow Concentrations for Inverted Triangular Inflow Concentrations with Two-Hour Triangular Inflow - Large Hypothetical Pipe, 5% Slope

APPENDIX F

SELECTED COMPUTER INPUT AND OUTPUT

Examples of computer input and output for each of the seven tested models are assembled in a separate volume.* The output, in most cases, is self-explanatory. Reference to the respective user's manuals is needed in some instances where output data categories are identified by abbreviations and to identify input data categories. It was not within the scope of this study to duplicate the user's manuals to provide all details required for input and output interpretation.

The input and output examples are selected from computer runs which include the following simulations:

1. large hypothetical catchment, 0.1% slope, dry condition, 2-hour triangular rainstorm;

2. small hypothetical pipe with upstream storage and downstream diversion, 0.5% slope, 2-hour triangular inflow, all three inflow concentrations (if modeled).

*The title of this separate volume, an extension of this work is "Assessment of Mathematical Models for Storm and Combined Sewer Management, Appendix F: Selected Computer Input and Output" (EPA-600/2-76-175a) and can be obtained at the National Technical Information Service, Springfield, Virginia 22151.

TECHNICAL REPORT DATA
(Please read Instructions on the reverse before completing)

1. REPORT NO. EPA-600/2-76-175a	2.	3. RECIPIENT'S ACCESSION NO.
4. TITLE AND SUBTITLE Assessment of Mathematical Models for Storm and Combined Sewer Management		5. REPORT DATE August 1976 (Issuing Date)
		6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Albin Brandstetter		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS Battelle Pacific Northwest Laboratories, Water and Land Resources Department, Battelle Blvd., Richland, Washington 99352		10. PROGRAM ELEMENT NO. 1BC611
		11. CONTRACT/~~GRANT NO.~~ 68-03-0251
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory, Office of Research and Development, U. S. Environmental Protection Agency, Cincinnati, Ohio 45268		13. TYPE OF REPORT AND PERIOD COVERED Final
		14. SPONSORING AGENCY CODE EPA-ORD

15. SUPPLEMENTARY NOTES

See also Appendix F, EPA-600/2-76-175b

16. ABSTRACT

Mathematical models for the nonsteady simulation of urban runoff were evaluated to determine their suitability for the engineering assessment, planning, design and control of storm and combined sewerage systems. The models were evaluated on the basis of information published by the model builders and model users. Seven models were also tested by computer runs using both hypothetical and real catchment data. Most of the models evaluated include the nonsteady simulation of the rainfall-runoff process and flow routing in sewers; a few include also the simulation of wastewater quality, options for dimensioning sewerage system components, and features for real-time control of overflows during rainstorms.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Combined sewers, Hydraulics, Hydrology, *Mathematical models, Open Channel flow, Overflows, Runoff, Sanitary sewers, Sewage, *Sewers, Storm sewers, Unsteady flow, *Urban planning, Wastewater, *Water pollution, Water quality.	Combined sewer overflow, Optimal control, Optimal design, Urban hydrology, *Urban hydrologic models, *Water quality control.	13B

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS (This Report) UNCLASSIFIED	21. NO. OF PAGES 530
	20. SECURITY CLASS (This page) UNCLASSIFIED	22. PRICE

EPA Form 2220-1 (9-73)