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Soil Conservation Service

HYDROLOGIC MODELING FOR FLOOD HAZARDS

Aug. 1985

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HYDROLOGIC MODELING FOR FLOOD HAZARDS

PROGRAM

Wednesday, August 7

8:00 a.m. Registration Check-in, Executive Tower Inn, Beethoven Rm
8:30 a.m. Welcome and Overview
Oren Strom, Assistant Dean,
CU-Denver School of Engineering
"Methods and Requirements for FEMA Flood Studies"
Dr. John Liou, FEMA Region VIII
"Guidelines for Computer-Based Hydrologic Design Studies"
Dr. Lynn Johnson, CU-Denver, Civil Engineering
9:30 a.m. Hydrologic Model - TR-20
Wendell Styner, SCS West National Technical Center
11:30 a.m. Hydrologic Model - TR-55
Wendell Styner, SCS West National Technical Center
12:30 p.m. Lunch and Speaker
"Rainfall-Runoff Modeling from Consultant's Point-of-View"
Dr. Anand Prakash, Chief Hydrologist, Dames and Moore
1:30 p.m. Hydrologic Model - HEC-1
John Peters, Hydrologic Engineering Center,
Corps of Engineers
3:30 p.m. TR-20/HEC-1 Combination
Claron Koontz, Kansas City District,
Corps of Engineers
4:30 p.m. Microcomputer Demonstrations

Thursday, August 8

8:30 a.m. Hydrologic Model - ILLUDAS
Doug Noel, Illinois State Water Survey
11:00 a.m. Hydrologic Model - Revised SWMM
Jody Farhat, Omaha District,
Corps of Engineers
12:30 p.m. Lunch (on your own)
1:30 p.m. Hydrologic Model - MITCAT
William Taggart, McLaughlin Water Engineers, Inc.
Roy Evans, CAmp, Dresser, & McKee
4:30 p.m. Microcomputer Demonstrations

Friday, August 9
8:00 a.m. Hydrologic Model - Sacramento Watershed Model
12:00 a.m. Lunch (on your own)
1:00 p.m. Hydrologic Model - Colorado Urban Hydrograph Procedure
3:00 p.m. Panel Discussion and Wrap-Up

HYDROLOGIC MODELING

FOR

FLOOD HAZARDS PLANNING AND MANAGEMENT

Lecturers

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HYDROLOGIC MODELING

FOR

FLOOD HAZARDS PLANNING AND MANAGEMENT

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8/7

FEMA/DR. John Liou Region VIII

Flood Hazard mitigation

(1)

non-structural

- Floodplain management
- Flood warning
- Evacuation plans
- ~~NFIF~~ NFIP (Flood Ins.)

Structural

- channelization / levees
- Dam / Resivoir
- Others.

(2) Basic Engineering data - Hydrology / Hydraulics

- Peak Flow & Volume
- Time to peak
- Duration

(3) Classification System

Statistical

- Frequency
  - Plotting positions
  - Frequency Fuctn
- Regional Analysis
  - Direct estimate (USGS)
  - Estimate of moment
  - Index

Non - Statistical

- Empirical (Rational)
- Simulation
  - rainfall-runoff maleling

Liou

II

Selection criteria of method

- Purpose of analysis - Planning vs design
- Time Constraints
- cost limits
- man power limits
- Data availability.

(4) Streams

- gauged (10 + years)
  - Gage site
  - ungauged site

Streams

- non gauged
  - Unregulated
  - Regulated

Frequency

- Gage Site — Log Pierson Type III or other if data file
- ungaged — Transfer techniques

Let document name

ungauged streams

unregulated vs regulated - specify method
various methods

For proper regulation - must master plan basins

FEMA studys

1. Gauged sites
2. Regional method
3. last choice rainfall / runoff modeling ; only with FEMA Approval.

Lynn Johnson

1. models must be credible / replicable.

→ validate model → does → 1" rainfall no storage yield
1" runoff
→ Does complete storage yield no runoff etc.

Sensitivity → hold variables constant & vary → see what parameter is most sensitive to peak Q.

Wendell Stiger
Author TR-20

~~[illegible]~~ → developed for
TR-20 - Small ungaged watershed (400 mi² or less)
little Data possibly - soils -
minimum Data - Soil series (loam etc)
Precipitation - NWS TP-40
(western) or NOAA Atlas 2

(1) models a single storm is not a continuous day to day model.
(2) subsurface runoff is not considered, direct runoff only.
(3) Routes through a resivoir or through natural reaches.
Documented in NEH-4
1964 TR-20 published 1982 - Revised
latest Draft at N.T.I.S.

Basic operations

Standard ~~Runoff~~ control
(1) must develop hydrograph.
(2) Route hydrograph thru resivoir must supply data on stage/stor
(3) Route through natural reach.
(4) Add hyd → Add all hydrographs at confluence.
(5) Diversion → split hydrograph / add or remove.

Executive control (environmental)
(1) Basflo - what is existing stage (constant cfs or hydrograph)
(2) Increm (what time increment)
(3) comput → where to stop/start, depth, antecedent moi

Storm II

New changes

Old - convex routing

- new - Atkin Routing & Attenuation / Kinematic routing.
- runoff routines ↑ Valley routing

- Reccomended New data requirements - include cross section / ~~stat~~ rating curve.

TR-20 Data sheet allows one through to check all data,

Future

① complete revised version.

② Hydrology / Hydraulics

③ SAWCAM - Soil + water model encompasses TR-20, WSP-2, Dams-2 +

TR-55

A Type II storm with shorter time increments may make up the difference for

Sheet Flow < 300 feet - No channel

$$T_t = \frac{.007\,(nL)^{.8}}{P_2^{.5}\,S^{.4}}$$

L = ft

P_2 = 24 24 hour inches

S = ft/ft

8/7
HEC 1 John Peters

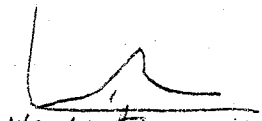
HEC 1 First out 1968
last major revision 1981

Flood Hydrograph Package

~~Sample~~

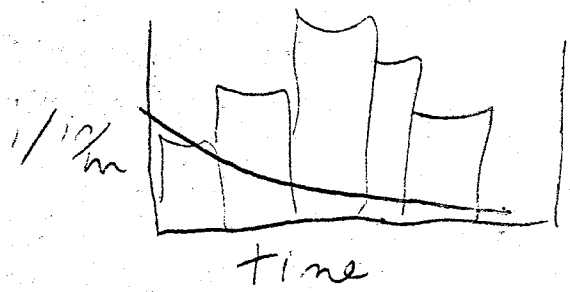
- Precip
  1. - Precip for each subbasin entered
  2. Enter data for gauges / ~~proc~~ ~~utilize~~ input weight + gauges for each basin.
  3. Input hypothetical depth.

- Input depth/duration data.
  - program will create Temporal variation

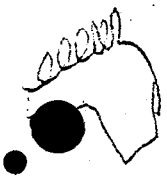


  - can overide and input own temporal Storm

- Rainfall excess
  4 methods allowed



- hydrographs
  - unit - 4 options (Nakrown, Clark, Snyder, SCS Dimen)
  - Kinematic wave hydrograph



Hec 1
pg II

- ~~Rout~~
- Routing

Muskingum - Travel time.

Modified Puls - storage inflow - outflow.

Kinematic Wave

Typically used after a Hec II run, Hec II gives storage discharge, plug into Modified Puls.

options

① parameter calibration - input observed rainfall/response on a watershed, program determines parameter for the single basin.

- look at parameters carefully, See if optimization is reasonable.

"Comparison of Rainfall-runoff modeling on small upland Catchments"

by

Keith M. Loague & R. Allen Freeze

Water Resources Research

Vol 21 No. 2 Feb 1985

TR20/HEC1 combined

Claron Koontz

Doug Noel

Illinois 8/8 500 - 1000 acres maximum

Non structural - type routing - routes a peak through.

- Design tool mainly -
  - you pick discharges/size through pipes will route
  - You pick discharge, will provide basin size + volume, can provide rating curve to specify storage volume outfall - p.c. model.

If backwater is anticipated utilize a backwater model.

State of Illinois is no longer funding the model

Has not been developed, but are trying to develop initial runoff prior to initial abstraction.

Top down model - infinite complexity.

Revised Swmm
Jody Farhat

- originally EPA
- Revised in 73 Florida and modified by HEC & Missouri corp. 74

Many versions out, may not include all modification

Swmm - urban + Rural - largest 1000 mi², limit of 700 basins at one time, carryover of inflow hydrographs for large basins.

looks at idealized sub basins - rectangular sections, and ~~ph~~ uniform physical parameters.

input rainfall intensity curve, overland sheet flow, routed in roads etc by Manning.
subbasins at int homogeneous 1/2 mi² basin - Subbasins are approximated by a rectangle using a mean width approach.

- Does not work well on very flat or steep basins.

- Presently Swmm is linked with other models to do hydraulic routing.

→ overflow section - helps prevents surcharge + ~~storage~~ ~~under~~ overfilling channel → swmm will not flag you to utilize this.

problems
→ overflow → unless overflow is larger than pipe, will not fill pipe.
→ ~~outlet~~

you can not directly divert to storage

MitCAT

Taggart & Evans

82 Urban storm Drainage Management -
Schaffer, Wright, Taggart, Wright
Dekker Publ

→ Routes by fluid mechanics - user inputs physical surface that water flows.

$$\underbrace{\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + \underbrace{g\frac{\partial h}{\partial x} + \underbrace{g(S_o - S_f)}_{\text{kinematic}}}_{\text{Diffussion}}}_{\text{Dynamic}} = 0$$

(centroid)

Kinematic applies $K = \frac{S_o L}{h_o F^2} > 10$

F = Froude
h_o = centroid of depth

Diffusion only if flat or hysteris (tide)

~~Am~~ model Also routes flow into pipe up to low flow capacity, holds H_2O back at inlet till it is routed through

(1) Do not use where Incrusted Predominates

(2) Not good for small planning - Subdivision

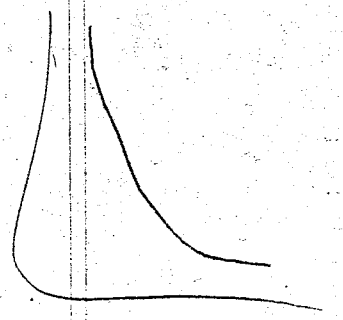
(3) Good 300 + acres → 30-40 mi².

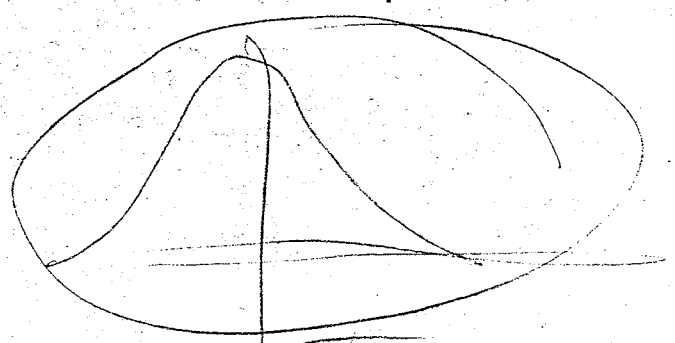
(4) Extremely user friendly - Free Format, NO real for order.

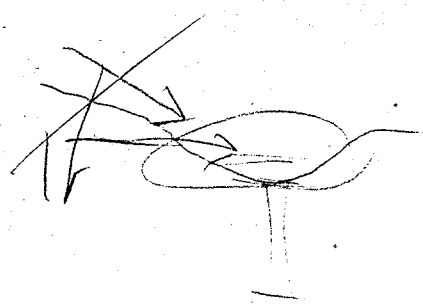
2.0
1.30
7.8
8.0
3.3 all
Burnoch

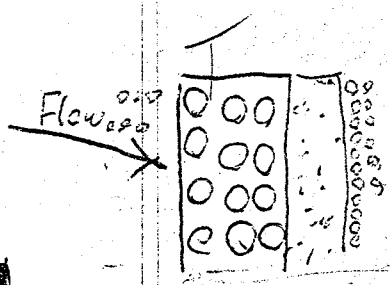
windspeeds up to 20 mph ↓
up to 60% of total
catch measured.

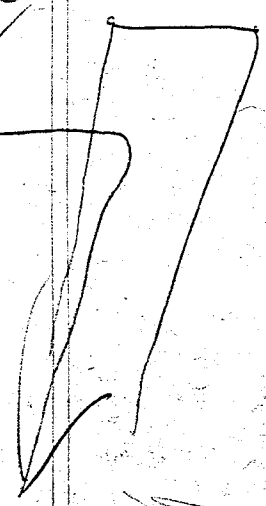
→ Find out if our rainfall data was adjusted by windspeeds. ←

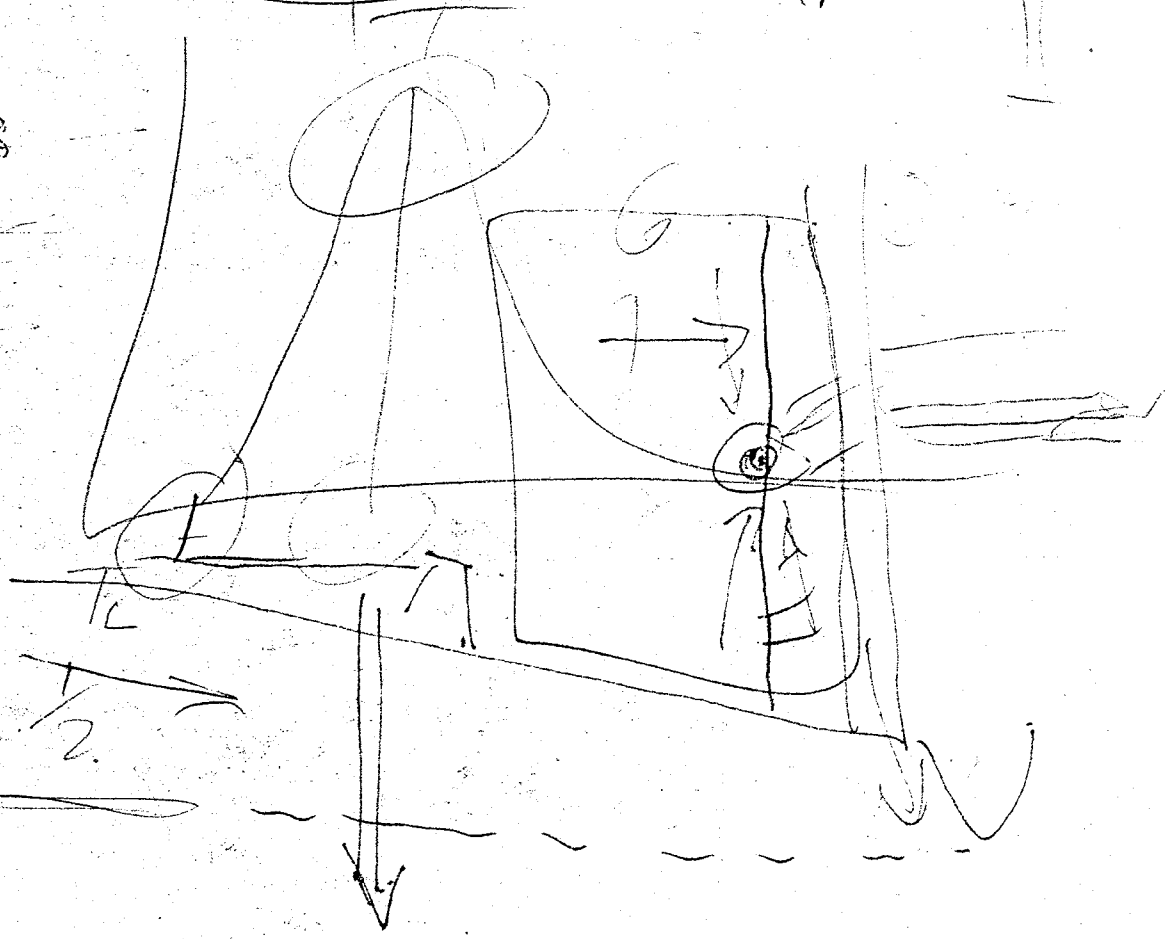












hydrophobic
soils

CUHP Ben Urbonas + Young Yoon

Development

69 ~~Development~~ Data collection began →

77 Began Analysis 12-13 years data

82 Completed Analysis.

Original CUHP → 1969 Base

Limited data base

$$t_p = C_t (L L_{ca})^{.3}$$

$$q_p = \frac{640 C_p}{t_p}$$

Utilized CIA = Q , Literature C

(Utilized 1.25x larger runoff) x Factors for larger storm.

Latitude for interp.

Original Rainfall in 1969

- local data
- Short records w/ Long record
- Showed large variation in Depth in District

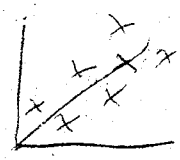
Shortly after – Utilized NOAA.

in 1977 : Reviewed Data

Threw out defective data,

Screening.

① Plot obs. rainfall vs obs Runoff.



60
.04
24
240 minutes

Reduced 30 basins to 11 basins - because no one checked data to see how it was, if problems are found fix it or move equipment.

unscreened data should not be used for calibration.

unit hydrographs were verified on Hec 1.

Choose U.H based on

$$t_p = f(L, L_{ca}, S)$$ <also Imperviousness>

$$q_p = f(t_p, A)$$

$t_p \rightarrow$ utilizes a multiple regression fit.

$$C_{t_p} = f(i_u)$$

$$C_{q_p} = \rightarrow$$

$$C_p = P\, C_t A^{.15}$$

① How does this methodology compare to existing ADOT methods without routing. Are they adequate?

② Could ADOT methods be computerized with modifications?

③ Could ADOT be given uniform criteria.

→ 150 acre basin how much data needed, time to input, run and develop answer
- how fast is the review process.
- is model applicable or economically practicle for this use.

question + Answer

→

mitcat
swmm
Sac
Illudas
Fcs
CHUP

HYDROLOGIC MODELING FOR FLOOD HAZARDS

GUIDELINES FOR COMPUTER-BASED DESIGN STUDIES

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GUIDELINES FOR COMPUTER-BASED DESIGN STUDIES

INTRODUCTION

- o Need for Guidelines/Standards
  - Enhance model credibility and confidence.
  - Aid understanding of model results.
  - Assist in procurement of modeling services.

- o Problems of Model Choice
  - Models vary in spatial and temporal detail, as well as the processes represented.
  - Models are operable on differing computing systems (e.g. desktop micro to mainframe).
  - Person who developed model is not the one applying it.
  - Programs get modified without followup documentation.
  - Client may be sceptical of model results.

- o Guidelines are standard precautions or specifications
  - Minimum number of standard tests for:
    - + model verification,
    - + sensitivity analysis,
    - + output interpretation.
  - To avoid using:
    - + wrong model,
    - + wrong data,
    - + wrong interpretation of results,
    - + misunderstanding of model and its relationship to design objectives.

MORE COMPLEX COMPUTER-BASED DESIGN ENVIRONMENT

- o Factors Causing Complexity
  - More programs available,
  - More processes modeled,
  - Increased variety of computer hardware,
  - Decreased cost of computing,
  - More sophisticated micro-computer software,
  - Easier communication with remote mainframes,
  - Desire for hands-on use of models.

- o Computer Models
  - Several hundred currently available.
  - Main frame code being ported to micros.
  - Preference for use of a model used before.
  - Most models being enhanced each year.
  - General-purpose software (e.g. spreadsheets) make computing easier.

o Hardware
- Rapid evolution of microcomputers.
- Decreasing costs of computing.
- Distributed computing linking micros with mainframes.

o Systems Ware
- High resolution graphics processors for micros.
- Enhanced communications speeds (e.g. 1200 baud)
- National and international networks to access large computers.

COMPUTER MODELING ASPECTS OF STUDY REVIEW

o Problem Review
- At outset of study, determine drainage system characteristics so that computer model can represent those characteristics.
- Examples:
  + Diversions
  + Pipe surcharging
  + Interior design points

o Study Objectives and Criteria Functions
- ID questions to be answered and how computer numbers will relate to answers.
- Determine performance criteria:
  + How computer numbers are to be compared between several runs.
  + Examples: peak flow, time to peak, volume.

o Accuracy of Model Should Match Data
- Field measurements and calibrations will establish accuracy of model.
- Examine sources of error:
  + input data
  + model concepts
  + solution methods
  + computational accuracy
  + field equipment and measurements

o Model Selection Criteria
- Use the simplest model which answers the questions.
- Study resources.
- Availability of the model.
- Availability of supporting advice.
- Experience with the model.
- Availability of data.
- Requirements of regional authorities.
- Special costs of proprietary programs.

GUIDELINES FOR COMPUTER-BASED DESIGN STUDIES

STEPS TO ESTABLISH MODEL CREDIBILITY

- o Validation Tests
  - Compare model responses with theoretically anticipated results computed by hand.
  - Examples:
    1) Zero rain ----> no runoff.
    2) Steep catchments ----> hydrograph shaped like hyetograph.
    3) 100 % impervious ----> runoff volume = rain volume.
    4) Light rain and high infiltration ----> no runoff hydrograph.
    5) Flat pipe gradients ----> surcharge check.
    6) Storage routing check.
    7) Addition of two hydrographs at junctions.

- o Calibration and Verification
  - Calibration implies comparison of simulation results with:
    1) field measurements,
    2) another model known to be correct, or
    3) some other criteria.
  - Calibration involves progressive adjustment of model algorithms or model input data until satisfactory confidence level is achieved.
  - Verification implies testing the system model with an independent data set not used in the calibration process.

- o Level of Discretization
  - Need to represent interior design points.
  - Time step selected to accurately represent unit hydrograph.
  - Trade-offs between study costs and number of subspaces.

- o Sensitivity Analysis
  - Sensitivity analysis conducted by holding all parameters but one constant.
  - Vary the single parameter over expected range.
  - If model criteria function is sensitive to variation then data collection/analysis effort should be conducted with care.
  - If not sensitive, then accurate measurement not necessary.
  - Peak flow: slopes (pipe & land), Manning's "n", subbasin shape.
  - Volume: % impervious, infiltration, depression storage.

- o Control of Errors
  - Validation tests prove no serious coding errors.
  - Calibration and verification tests for simple systems show model is reasonable.
  - Verification tests for full system help to determine data input errors.
  - Sensitivity tests indicate appropriate level of effort of estimating parameters.
  - Graphics displays aid in data review (input & output).

- o Documentation
  - Identify model version and reference documentation.
  - List input and output data files.
  - Backup data on computer storage media.

GUIDELINES FOR COMPUTER-BASED DESIGN STUDIES

TOPICS FOR COMPUTER-BASED DESIGN STUDIES

- o PROBLEM REVIEW
- o STUDY OBJECTIVES
- o PERFORMANCE CRITERIA
- o RREQUISITE ACCURACY
- o REVIEW AVAILABLE PROGRAMS
- o STUDY RESOURCES
- o MODEL SELECTION CRITERIA
- o MODEL VALIDATION
- o MODEL CALIBRATION AND VERIFICATION
- o LEVEL OF DISCRETIZATION
- o SENSITIVITY ANALYSIS
- o DATA PREPARATION
- o OUTPUT INTERPRETATION
- o DOCUMENTATION

REFERENCES

o James, W. and M. Robinson, 1981, "Standards for Computer-Based Design Studies", ASCE, HY7, July.

HYDROLOGIC MODELING FOR FLOOD HAZARDS

METHODS AND REQUIREMENTS FOR FEMA FLOOD STUDIES

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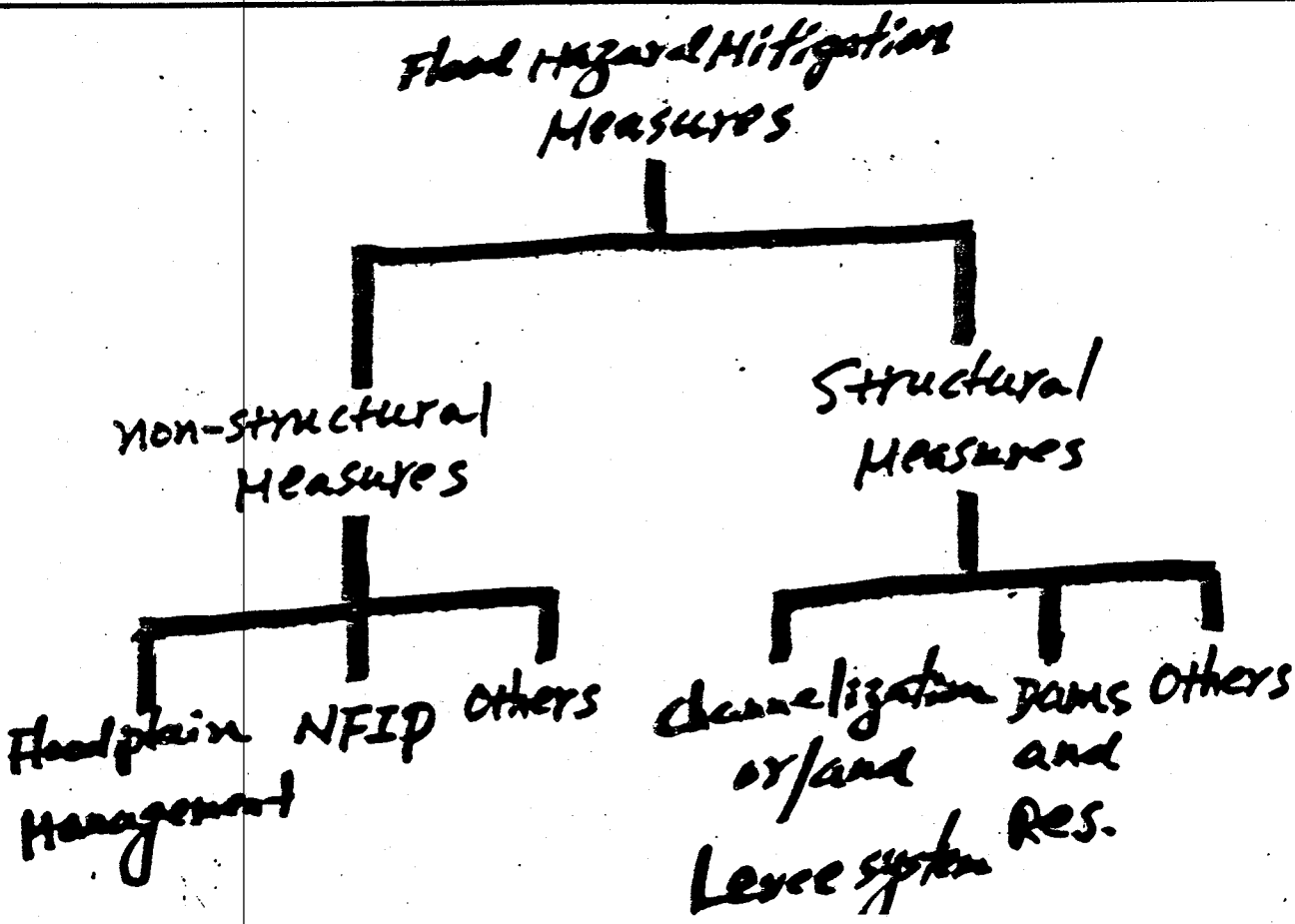
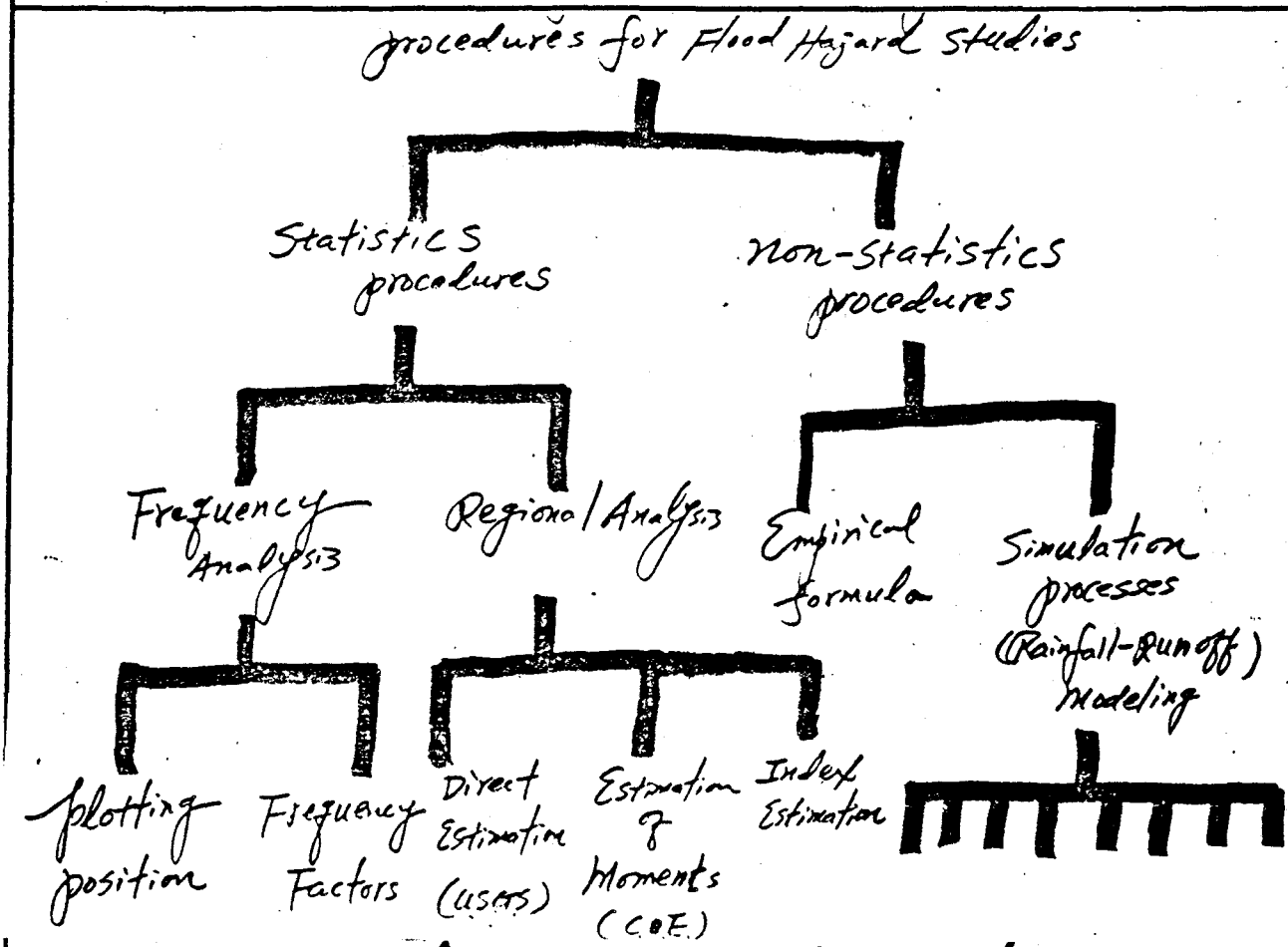
Building 710
Denver, Colorado 80225



Basic Engineering Data
For
Flood Hazard Mitigation projects

- Mappings
- Hydrology
- Hydraulics
- Others



Hydrology Information Required for Flood Hazard Mitigation projects

- peak flow
- Volume of Runoff
- Time to peak flow
- Flood Duration
- Hydrograph

Selection Criteria of Hydrological procedures for Conducting Flood Hazard Studies

- purpose
- Time limitation
- Cost limitation
- man-power limitation
- Data availability

- Streams
  - Gage Streams (10 Years or more Records)
    - Gage site
      - Regulated Streams
      - Unregulated Streams
    - Ungage site
  - Ungage streams (no records or less than 10 years Records)
    - Regulated Stream
    - Unregulated Stream

HYDROLOGIC MODELING FOR FLOOD HAZARDS

Guidelines For Determining

Flood Flow Frequency

Bulletin #17A
of the
Hydrology Committee

Revised June 1977

United States Water Resources Council
2120 L Street, N.W.
Washington, D.C. 20037

Gage Site { Regulated / unregulated
- Frequency distributions, LP III. (GI, LN, 3LN or GMZ for Comparison)
- Weiball plotting position

Gage streams
(House Document 465)
89th Congress,
2nd session, Aug. 1966
Bulletin 17 Series
Statistical procedures"

Ungage Site — Transfer techniques
(Area ratio, Regional Analysis, and Modeling)

Estimating Peak Flow Frequencies for Natural Ungaged Watersheds

A Proposed Nationwide Test

Hydrology Committee—1981

U.S. Water Resources Council

Ungage streams
1981 Hydrology
Committee, U.S. water
Resources Council

- regulated site
- unregulated site
  - U.S.G.S. Regional Analysis
  - COE Regional Analysis
  - Index method
  - Empirical Equat
  - Rainfall-Runoff Modeling

- Rainfall-Runoff Modeling

Application of Rainfall-Runoff Modeling

- Urban Area
- Hydrograph needed
- ungage site on gage streams
- ungage streams
- Flood control Measures Design
- River flood forecasting
- River basin Master Planning

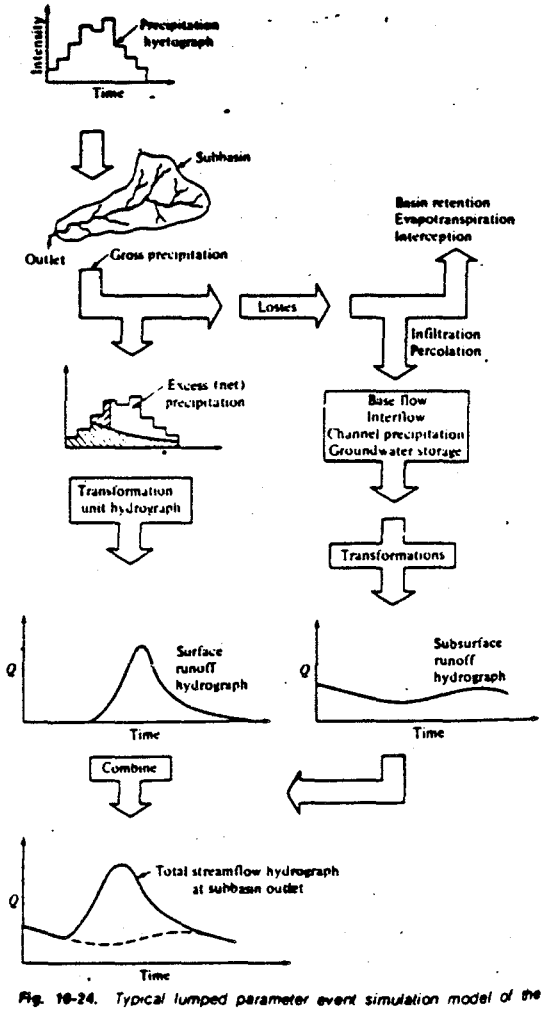


Fig. 10-24. Typical lumped parameter event simulation model of the rainfall-runoff process.

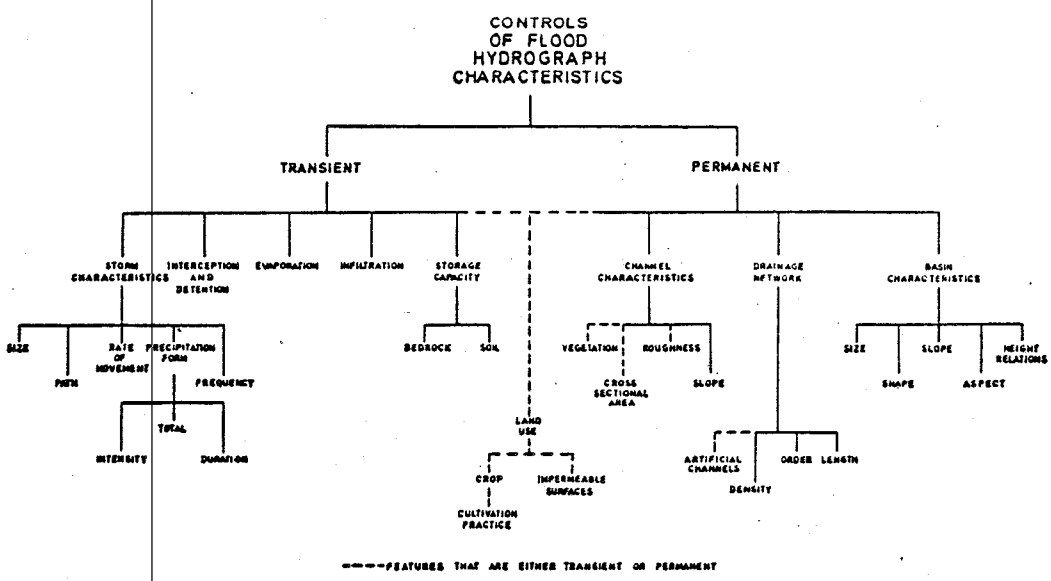


Fig. 9.1.2 Controls of flood hydrograph characteristics.

Nationwide Test

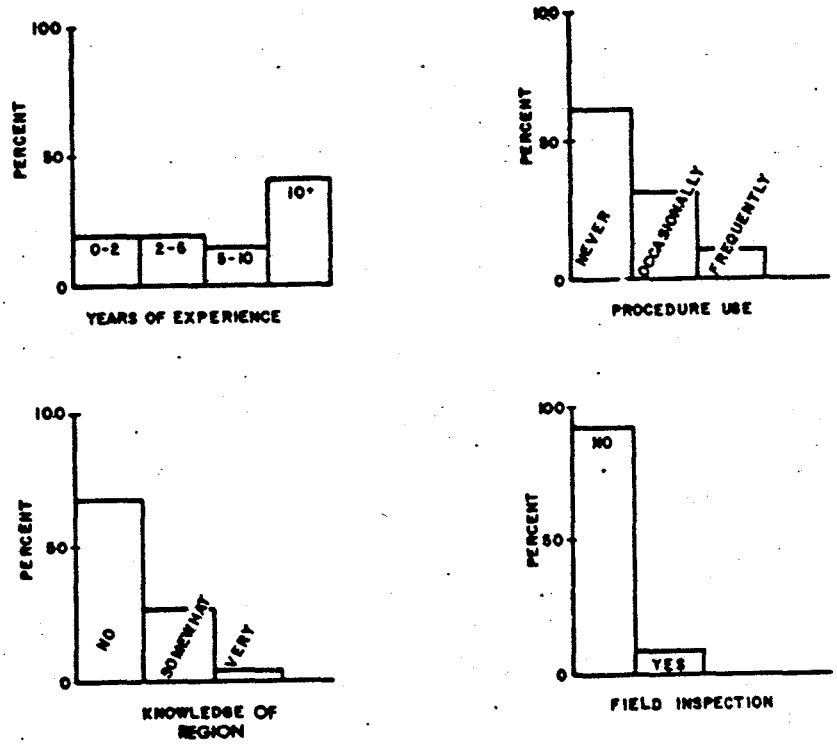
200 Individuals participated in the test

63% Federal Governments

15% State and Local Governments

20% Consultants

2% University personnel



SUMMARY OF TESTER INFORMATION

CATEGORIES OF PROCEDURES

Category	Products: Primary[1]	Products: Other[2]
1. Statistical estimation of Q_p Q_p = f (basin characteristics B, channel geometry G, climatic characteristic C)	Q_p	
2. Statistical estimation of moments Q_p = f (moments) moments = f(B,G,C,)	Frequency curve	
3. Index flood estimation Q_p = f (ratio Q_i) Q_i = f(B,G,C)	Q_p	
4. Estimation by transfer of Q_p	Q_p	
5. Empirical equations	Q_p	
6. Single storm event: rainfall frequency α runoff frequency	Q_p	Hydrograph possible
7. Multiple discrete events	Frequency curve	Flood hydrograph
8. Continuous record	Frequency curve	Continuous hydrograph

1. The major output from the procedure for meeting the objectives of the Work Group.

2. Other products useful in certain applications such as reservoir design.

Note: Q_p = peak discharge for a given exceedance probability.

Q_i = peak discharge from an index flood relationship.

TESTER BACKGROUND INFORMATION

Summary of Tester's Hydrologic Experience (Percent)

		State Eq.	Flet-cher	Index Flood	Reich	Rational Formula	TR-55 Charts	TR-55 Graph	TR-20	HEC-1	Average
Testers Hydrologic Experience:	0-2 Years	22	24	28	17	17	18	20	21	12	22
	2-5 Years	22	23	24	17	20	25	19	27	26	22
	5-10 Years	17	14	14	19	17	12	16	20	23	16
	10 + Years	39	39	34	47	46	45	45	32	39	40
Summary of Testers' Hydrologic Knowledge of the Region (Percent)											
Knowledge of Region:	No	68	70	71	63	65	69	63	68	68	68
	Somewhat	27	27	24	32	32	29	33	26	27	28
	Very	5	3	5	5	3	2	4	6	5	4
Summary of Testers' Frequency of Procedure Use (Percent)											
Frequency of Use:	Never	40	80	57	92	55	58	59	68	60	62
	Occasionally	38	19	39	8	39	37	29	23	34	30
	Frequently	22	1	4	0	6	5	12	9	6	8
Summary of Field Inspections (Percent)											
Made Field Inspection:	No	94	94	93	93	94	97	91	82	84	92
	Yes	6	6	7	7	6	3	9	18	16	8

FREQUENCY OF USE OF PROCEDURE CATEGORIES
(in percent)

Procedure Categories	Federal* Agencies	State Highway	Private Sector
1. Statistical Estimation of Q_p	48	38	34
2. Statistical Estimation by Moments	1	0	4
3. Index Flood Method	1	4	3
4. Transfer Method	1	19	7
5. Empirical Equations	24	38	17
6. Single Storm	24	1	34
7. Multiple Discrete Events	1	0	0
8. Continuous Record	0	0	1

*Based on small samples, modest to important projects

FREQUENCY OF USE OF PROCEDURES BY THE PRIVATE SECTOR
(in percent)

	Procedure	Category		Frequency of Use
1.	USGS State Equations	Statistical Estimation of Q_p	(1)	27.3
2.	Rational Formula	Empirical Equations	(5)	16.1
3.	SCS NEH-4	Single Storm Event	(6)	7.0
4.	Transfer of Q_p	Transfer of Q_p	(4)	6.9
5.	SCS TR-55	Single Storm Event	(6)	6.6
6.	SCS TR-20	Single Storm Event	(6)	5.9
7.	COE HEC-1	Single Storm Event	(6)	5.3
8.	COE Log Pearson III Basin Studies	Statistical Estimation of Moments	(2)	3.3
9.	BPR Potter	Statistical Estimation of Q_p	(1)	2.9
10.	Snyder Unit Hydrograph	Single Storm Event	(6)	2.3
11.	USGS Water Supply Papers	Index Flood	(3)	1.5
*12.	Other Unit Hydrograph	Single Storm Event	(6)	7.3
*13.	Other Regression	Statistical Estimation	(1&2)	4.8
*14.	Other Empirical	Empirical Equations	(5)	1.0
*15.	Other Index Flood	Index Flood	(3)	1.0
*16.	Continuous Simulation	Continuous Simulation	(8)	0.8
	Total			100.0

*Cumulative percentages for all other procedures in the category. Individual procedures accounting for less than 1.0 percent of total applications were not itemized.

TR-20 & TR-55

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HYDROLOGIC MODELING FOR FLOOD HAZARD PLANNING AND MANAGEMENT

TR-20

Wendell A. Styner, P.E., \1

OVERVIEW:

The SCS computer program for Project Formulation--Hydrology, more popularly known as the SCS TR-20 program, assists the engineer in hydrologic evaluation of flood events for use in analysis of water resource projects by analyzing more alternatives. TR-20, one of a series of SCS Technical Releases, is the users manual describing the input and output of the program (1). The program is a single event model which computes direct runoff resulting from any synthetic or natural rainstorm. It develops flood hydrographs from runoff and routes the flow through stream channels and reservoirs. It combines the routed hydrograph with those from tributaries and computes the peak discharges at points of interest in the watershed.

The model uses standard SCS hydrologic techniques and procedures as described in the SCS National Engineering Handbook, Section 4 (NEH4)-Hydrology (2).

Background:

1. A "Computer Program for Project Formulation, Hydrology" was prepared for SCS by C-E-I-R, Inc., January 1964.
2. TR-20 User's Manual.
   a. SCS developed a user's manual, May 1965, "Computer Program for Project Formulation, Hydrology."
   b. Supplement to user's manual, March 1969.
   c. Completely rewritten as Draft User's Manual, May 1982.
   d. Pen and ink changes - 2nd Edition (May 1983).
   e. Latest Draft User's Manual available through National Technical Information Service (NTIS) contains pen and ink changes. The NTIS Accession No. is PB83-223768 with a price of $25.00.
   f. Errata sheets were distributed March/April 1984.
   g. Final User's Manual - incorporated comments waiting for completion of programming.
3. TR-20 Computer Program.
   a. First available May 1965.
   b. Revisions made periodically until last version 1974.
   c. Latest program revision date is 9/1/83.
   d. Available at the USDA's computers at the Washington Computer Center (WCC) and the Fort Collins Computer Center (FCCC).

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e. Available through NTIS Accession No. PB83-223107 with a price of $595.00.
f. Program written in Fortran IV for IBM series of mainframe computer, i.e. S360 or S370, with a Fortran IV G1 compiler and 400K bytes of main storage. The program has also been implemented on other mainframe computers as the UNIVAC 1100 series.
g. In 1976 the SCS began to examine advanced technology in hydrology and hydraulics (H&H). In 1980, the SCS initiated an H&H study that should lead to the development of an unsteady-flow field model that will combine the discharge and stage computation functions of TR-20 and the SCS water surface profile program, WSP2. This study indicated that the coefficient valley routing procedure in TR-20 could be improved using the Att-Kin procedure developed for a simplified dam-breach routing procedure (3). With need to update the TR-20 user's manual, other improvements are also being made. A new version of TR-20 user's manual and program will be released soon and given an appropriate version date following final testing.

Organization of Draft TR-20 User's Manual (4).

1. Chapter 1 describes TR-20 in terms of purpose, computer requirements, and availability of program.
2. Chapter 2 describes the program organization, and capabilities and limitations.
3. Chapter 3 describes input data preparation.
4. Chapter 4 describes output presentation and description.
5. References, Glossary and Conversions.
6. Appendices A-E contain 5 sample jobs showing different levels of complexity.
7. Appendix F contains blank input forms.
8. Appendices G and H have a description of the Modified Att-Kin (MAK) reach routing procedure and guidelines for rating curve coefficients and reach lengths.
9. Appendix I describes the data check computer Program (SCSCHK20).

MODEL THEORY AND STRUCTURE:

The TR20 computer model uses standard SCS hydrologic techniques and procedures described in SCS NEH4. The basic concepts of the model have been essentially unchanged since it was developed in 1964.

The SCS Runoff Curve Number (CN) system is used to convert total storm rainfall to total storm runoff. The model develops subwatershed flood hydrographs from the runoff and routes the

flow through stream channels and reservoirs. It combines routed hydrographs with those from lateral inflow and tributaries and includes procedures for adding baseflow. Hydrographs can be separated by branching or diversion of flow. The model computes peak discharges, their times of occurence, and associated water-surface elevations for any given cross section or reservoir. It can also generate the complete discharge hydrographs with associated durations and elevations.

The program was developed using the philosophy of being as flexible as possible in the use of input data and ability to model actual watersheds. The user can direct the model to generate hydrographs and route them through stream reaches and/or reservoirs, then combine and/or divide the hydrographs in virtually any combination necessary to model and watershed or river basin. Fig 1 illustrates a simple watershed with two sub-watershed areas, a reservoir and 2 stream reaches.

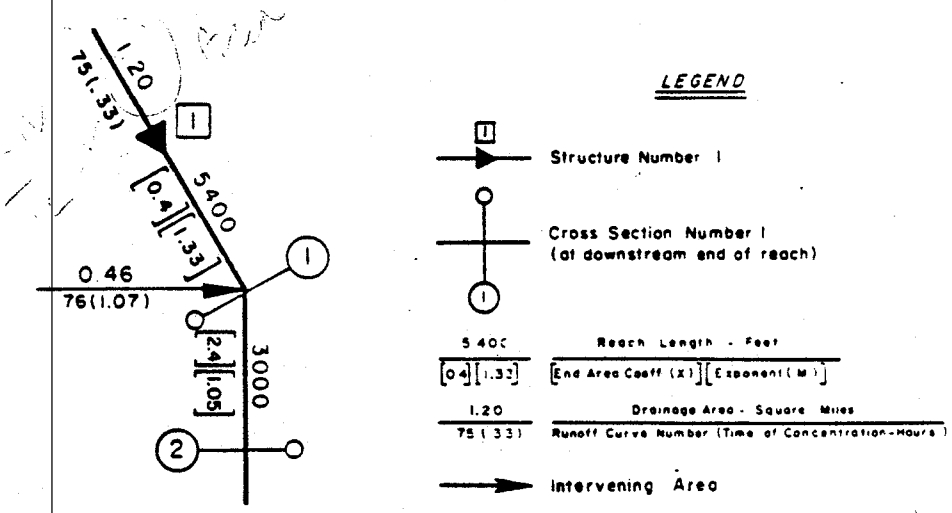

Fig. 1

The model can analyze in a continuous run up to nine different rainfall distributions of a watershed under existing conditions and various combinations of structural and nonstructural measures. Such and analysis can include as many as 200 cross sections (valley reaches) and 99 structures (reservoirs) within up to 600 separate standard control operations. Ninety-nine alternatives with 10 storms each are possible, but only the first 24,000 peaks from these combinations may be saved for the summary table of peaks (Summary Table 3).

Input data required by the program consists of watershed physical parameters, rainfall data and additional information to define the order of processing by the model. Basic input data includes:

a. Drainage areas, (sq.mi.) 20-50 mi² upper limit
b. Runoff Curve Number
c. Time of concentration, hrs.
d. Stream reach lengths, ft.
e. Rainfall data:
   1. duration, hrs.
   2. amount, inches.
   3. temporal distribution, a mass curve of depth vs. time.

4 distributions. 6 & 24 hr values → Based on Ratio 6hr/24hr
Type I - late peak
Type II - early peak
Type IA - long even Duration
Type III - Gulf/East intense early.
PNW less

can be left blank for TR-20 strct.

f. Structure data:
   1. stage or elev, ft.
   2. ~~surface area, acres~~
   3. storage, acre-feet
   4. discharge, cfs.
g. Channel cross section data
   1. stage or elevation, ft.
   2. end area, sq. ft.
   3. discharge, cfs.
   4. acres flooded (optional)

Output information provided by the model which can be selected by the user to be printed out include:

a. Hydrograph peak
   1. Discharge
   2. Time of occurrence
   3. Elevation (if applicable)
b. Hydrograph Ordinates
   1. Discharge
   2. Elevation
c. Runoff volumes
   1. Acre-ft
   2. Watershed-inches
   3. cfs-hrs
d. Additional data
   1. List of input data
   2. Basic data useful for determining validity of results.

modify raintable for the various duration, i.e. model defaults to 24hr?

A brief resume of the computational portions of the model follows:

Initial abstraction .2S is weak point definite possible local variation.

RUNOFF

The first step in the RUNOFF process to generate a hydrograph is to estimate the direct runoff from the rainfall. The SCS procedure use the relationship:

$$Q=((P-0.2*S)^2)/(P+0.8*S) \qquad \text{Eq. 1}$$

where Q = storm runoff
P = storm rainfall
S = maximum water retention factor for the soil and land use.

The value S, is determined from the relationship:

$$S=(1000/CN)-10 \qquad \text{Eq. 2}$$

where CN = the soil-cover runoff curve number.

A description of the derivation of Eq. 1 & 2 is given in Chapter 10 NEH4. The model generates a runoff mass curve from the rainfall mass curve.

A unit hydrograph is computed from a dimensionless unit hydrograph and the watershed time parameter, Tc. The unit hydrograph is then convoluted with the runoff mass curve to form the composite runoff hydrograph.

REACH

The REACH process routes the hydrograph through a stream reach using the modified Att-Kin procedure. The procedure, described in Appendix G, draft TR-20 user's manual, first determines the attenuation of the hydrograph due to storage within the reach, then computes the time lag due to kinematic routing. Storage in the reach is related to outflow from the reach by a single valued relationship:

$$\text{Log } Q = \text{Log } k + m * \text{Log } S \qquad \text{Eq. 3}$$

K - Slope of Storage outflow curve at peak.

or

$$Q = k * S^m \qquad \text{Eq. 4}$$

m -

Steady flow rating curve (Q vs. A) at a cross-section relects storage in reach at any particular time.

$$S = L * A \qquad \text{Eq. 5}$$

Substitute into Eq. 4:

$$Q = k * (L*A)^m \qquad \text{Eq. 6}$$

or $Q = k * L^m * A^m$ Eq. 7

let $x = k * L^m$

$Q = x * A^m$ Eq. 8

This relationship is seen in a log-log plot of Q vs A from cross section rating curves.

The routing equation is derived from the equation of conservation of mass:

$I - O = dS / dt$ Eq. 9

assume I1 is the average inflow over dt
and ((O2 +O1)/2) is the average outflow over dt

then $I1 - ((O2+O1)/2) = dS/dt$ Eq.10

Let S = KO: where K approximates the slope of the storage-outflow curve at peak outflow discharge.

$I1 - ((O2+O1)/2) = K * ((O2-O1)/dt)$ Eq.11

Rearraigning:

$O2 = (2dt/(2K+dt))*I + (1-(2dt/2K+dt))*O1$ Eq.12

Substituting the routing coefficient: $Cr = 2dt/(2K+dt)$

then $O2 = Cr*I1 +(1-Cr)*O1$ Eq.13

RESVOR

The RESVOR routine routes the hydrograph through a storage structure or reservoir using the Storage-Indication method. The basic equation for routing is derived from Eq. 1, i.e.:

$I - O = dS / dt$

Let $I = (I1 + I2)/2$, $O = (O1 + O2)/2$ and $dS = S2 - S1$
Then
$(I1 + I2) / 2) * dt - ((O1 + O2) / 2) * dt = S2 - S1$ Eq.14

After rearranging, this becomes:

$(I1 + I2 - O1) * dt + 2S1 = O2 * dt + 2S2$ Eq.15

The terms on the left hand side of Eq.15 are known and O2 and S2 solved iteratively using the storage-discharge rating table for the structure read from the Tabular Data file.

Program Structure and Implementation

The TR20 program consists of a mainline program with 5 processing subroutines and 13 housekeeping subroutines. The processing subroutines include:

a. RUNOFF-generates a runoff hydrograph
b. RESVOR-storage routes a hydrograph through a storage structure
c. REACH-routes a hydrograph through a stream reach using a modified AH-Kin routing procedure
d. ADDHYD-Adds two hydrographs together
e. DIVERT-Divides a hydrograph into two hydrographs using:
   1. a fixed discharge division
   2. a variable discharge division

Input data is classified into three types:

a. Tabular Data
b. Standard Control
c. Executive Control

The user builds a model of the watershed using appropriate Tabular data and Standard Control instructions. The Executive Control establishes the conditions to be used during the computer run.

The user first enters the tabular data consisting of:

1. Rainfall temporal distribution (up to 9 tables permitted). Entries are in the form of increments of a mass curve in inches of rainfall or runoff at a selected time interval.
2. Cross section data, (up to 199 tables). Data consists of rating curves of Elevation vs Discharge and End Area.
3. Structure data (up to 99 tables). Data consists of rating curves of Elevation vs Discharge and Storage.

The Standard Control provides the sequential operations instruction for the model network. The five Standard Control operations are:

1. RUNOFF-A subarea runoff hydrograph is generated based on the Drainage Area (DA), Time of Concentration (Tc),SCS Runoff Curve Number (RCN) for a rainfall depth and rainfall temporal distribution.
2. RESVOR-Routes a hydrograph through a reservoir using given elevation-discharge and storage data at the site.
3. REACH-Routes a hydrograph through a stream reach using elevation-discharge and end area data at a cross section representative of the reach, or, given routing coefficients.
4. ADDHYD-Adds two selected hydrographs together. For

example, the two hydrographs may be at the junction of a tributary stream, or adding local inflow to a stream reach.

5. DIVERT-Divides a hydrograph into two seperate hydrographs by diversion of fixed discharge or by the branching between two cross-sections.

The Executive Control permits the user to select the beginning and ending standard control instruction, starting time of the rainfall, depth of rainfall, antecedent moisture condition, which rain table to use and storm and alternate label to identify the output in the summary tables. Additional Executive Control instructions define the time increment to be used in the computations, defines BASFLO or the pre-storm flow conditions.

Fig 2 is a listing of the TABULAR DATA, STANDARD CONTROLS, and EXECUTIVE CONTROLS for the watershed of Fig. 1.

Output from the program can vary from a complete printout of every hydrograph computed, routed, added or divided for every pass through the watershed to an abreviated listing of peak flows and times for selected cross sections or structures. A complete output file should be used at least once for the user to check for accuracy. Succeeding runs may then use less output to reduce the volume of printed material to review.

Summary

The TR20 computer program provides the engineer with an easy to use method of modeling the hydrologic response from rural and/or urban watersheds. The model is especially useful in watersheds where there is a paucity of data such as infiltration parameters and where comparative results of land use and/or structural measures are needed

APPENDIX I - REFERENCES

1. Soil Conservation Service, "Computer Program for Project Formulation Hydrology," Soil Conservation Service Technical Release 20 (TR20) Supplement No. 1, Mar., 1969.
2. Soil Conservation Service, "National Engineering Handbook, Section 4, Hydrology, 1972.
3. Comer, G.H., Theurer, F.D., and Richardson, H.H., "the Modified Attenuation-Kinematic (Att-Kin) Routing Model," RAINFALL-RUNOFF RELATIONSHIP, Proceedings of the International Symposium on Rainfall-Runoff Modeling, 1981, pp. 553-564.
4. Soil Conservation Service, "Technical Release 20. Computer Program for Project Formulation - Hydrology," Draft 1982, Errata 1984.

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR20                          FULLPRINT                                 00000010
TITLE 001 SAMPLE JOB 1  USING 24HR TYPE II STORM FROM STR 1 TO XSECT 1      00000020
TITLE     INCLUDES GIVEN REACH COEFF'S, FULLPRINT/ALL OUTPUT OPTIONS ON     00000030
 3 STRUCT       01                                                          00000040
 8                        521.5        0.0         17.0                     00000050
 8                        521.6        3.0         18.0                     00000060
 8                        521.8        15.         20.2                     00000070
 8                        522.0        33.         22.5                     00000080
 8                        522.2        54.         25.0                     00000090
 8                        522.4        79.         28.0                     00000100
 8                        526.2        948.        70.                      00000110
 8                        526.4        1009.       75.                      00000120
 8                        526.6        1071.       80.                      00000130
 8                        527.2        1265.       95.                      00000140
 9 ENDTBL                                                                   00000150
 6 RUNOFF 1     01      6 1.20         75.         0.33        1 1   1   1 00000160
 6 RESVOR 2     01 6    7 521.5                                1 1 1 1   1 00000170
 6 REACH  3 001    7    5 5400.        0.4         1.33        1 1   1   1 00000180
   ENDATA                                                                   00000190
 7 INCREM 6             0.1                                                 00000200
 7 COMPUT 7     01 001                 5.2         1.0         2 2  01  01 00000210
   ENDCMP 1                                                                 00000220
   ENDJOB 2                                                                 00000230

*********************************END OF 80-80 LIST*********************************
```

Fig 2

```
******************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY******************

JOB TR20                          FULLPRINT                                    00000010
TITLE 001 SAMPLE JOB 1  USING 24HR TYPE II STORM FROM STR 1 TO XSECT 1         00000020
TITLE     INCLUDES GIVEN REACH COEFF'S, FULLPRINT/ALL OUTPUT OPTIONS ON        00000030
 3 STRUCT       01        Stage      Discharge      Storage ac/ft               00000040
 8                        521.5       0.0           17.0                        00000050
 8                        521.6       3.0           18.0                        00000060
 8                        521.8       15.           20.2                        00000070
 8                        522.0       33.           22.5                        00000080
 8                        522.2       54.           25.0                        00000090
 8                        522.4       79.           28.0                        00000100
 8                        526.2       948.          70.                         00000110
 8                        526.4       1009.         75.                         00000120
 8                        526.6       1071.         80.                         00000130
 8                        527.2       1265.         95.                         00000140
 9 ENDTBL                                                                       00000150
 6 RUNOFF 1     01      6 1.20        75.           0.33        1 1   1   1    00000160
 6 RESVOR 2     01 6    7 521.5                                 1 1 1 1   1    00000170
 6 REACH  3 001    7    5 5400.       0.4           1.33        1 1   1   1    00000180
   ENDATA                                                                       00000190
 7 INCREM 6              0.1                                                    00000200
 7 COMPUT 7     01 001                5.2           1.0         2 2  01  01      00000210
   ENDCMP 1                                                                     00000220
   ENDJOB 2                                                                     00000230

*******************************END OF 80-80 LIST*******************************
```

FILE NO. 1

A-8

COMPUTER PROGRAM FOR PROJECT FORMULATION - HYDROLOGY USER NOTES

THE USERS MANUAL FOR THIS PROGRAM IS THE MAY 1982 DRAFT OF TR-20. CHANGES FROM THE 2/14/74 VERSION INCLUDE:

REACH ROUTING - THE MODIFIED ATT-KIN ROUTING PROCEDURE REPLACES THE CONVEX METHOD. INPUT DATA PREPARED FOR PREVIOUS PROGRAM VERSIONS USING CONVEX ROUTING COEFFICIENTS WILL NOT RUN ON THIS VERSION.

THE PREFERRED TYPE OF DATA ENTRY IS CROSS SECTION DATA REPRESENTATIVE OF A REACH. IT IS RECOMMENDED THAT THE OPTIONAL CROSS SECTION DISCHARGE-AREA PLOTS BE OBTAINED WHENEVER NEW CROSS SECTION DATA IS ENTERED. THE PLOTS SHOULD BE CHECKED FOR REASONABLENESS AND ADEQUACY OF INPUT DATA FOR THE COMPUTATION OF "M" VALUES USED IN THE ROUTING PROCEDURE.

GUIDELINES FOR DETERMINING OR ANALYZING REACH LENGTHS AND COEFFICIENTS (X,M) ARE AVAILABLE IN THE USERS MANUAL. SUMMARY TABLE 2 DISPLAYS REACH ROUTING RESULTS AND ROUTING PARAMETERS FOR COMPARISON AND CHECKING.

HYDROGRAPH GENERATION - THE PROCEDURE TO CALCULATE THE INTERNAL TIME INCREMENT AND PEAK TIME OF THE UNIT HYDROGRAPH HAVE BEEN IMPROVED. PEAK DISCHARGES AND TIMES MAY DIFFER FROM THE PREVIOUS VERSION. OUTPUT HYDROGRAPHS ARE STILL INTERPOLATED, PRINTED, AND ROUTED AT THE USER SELECTED MAIN TIME INCREMENT.

INTERMEDIATE PEAKS - METHOD ADDED TO PROVIDE DISCHARGES AT INTERMEDIATE POINTS WITHIN REACHES WITHOUT ROUTING.

OTHER - THIS VERSION CONTAINS SOME ADDITIONS TO THE INPUT AND NUMEROUS MODIFICATIONS TO THE OUTPUT. USER OPTIONS HAVE BEEN MODIFIED AND AUGMENTED ON THE JOB RECORD, RAINTABLES ADDED, ERROR AND WARNING MESSAGES EXPANDED, AND THE SUMMARY TABLES COMPLETELY REVISED. THE HOLDOUT OPTION IS NOT OPERATIONAL AT THIS TIME.

PROGRAM QUESTIONS OR PROBLEMS SHOULD BE DIRECTED TO HYDRAULIC ENGINEERS AT THE SCS NATIONAL TECHNICAL CENTERS:

BROOMALL, PA (NORTHEAST) -- 489-3229 (FTS), FORT WORTH, TX (SOUTH) -- 344-5242 (FTS)
LINCOLN, NB (MIDWEST) -- 541-5318 (FTS), PORTLAND, OR (WEST) -- 423-2357 (FTS)
OR HYDROLOGY UNIT, NATIONAL ENGINEERING STAFF, LANHAM, MD -- 436-7383 (FTS).

PROGRAM CHANGES SINCE MAY 1982:

NONE

EXECUTIVE CONTROL OPERATION INCREM MAIN TIME INCREMENT = 0.10 HOURS RECORD ID 00000200

EXECUTIVE CONTROL OPERATION COMPUT FROM STRUCTURE 1 TO XSECTION 1 RECORD ID 00000210
STARTING TIME = 0.0 RAIN DEPTH = 5.20 RAIN DURATION= 1.00 RAIN TABLE NO.= 2 ANT. MOIST. COND= 2
ALTERNATE NO.= 1 STORM NO.= 1 MAIN TIME INCREMENT = 0.10 HOURS

OPERATION RUNOFF STRUCTURE 1
OUTPUT HYDROGRAPH= 6
AREA= 1.20 SQ MI INPUT RUNOFF CURVE= 75. TIME OF CONCENTRATION= 0.33 HOURS
INTERNAL HYDROGRAPH TIME INCREMENT= 0.0440 HOURS

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.10	2097.04	(RUNOFF)
16.46	76.40	(RUNOFF)
17.66	64.19	(RUNOFF)
19.66	51.94	(RUNOFF)
23.66	39.57	(RUNOFF)

TIME(HRS) FIRST HYDROGRAPH POINT = 0.0 HOURS TIME INCREMENT = 0.10 HOURS DRAINAGE AREA = 1.20 SQ.MI.

8.00	DISCHG	0.0	0.0	0.0	0.0	0.01	0.16	0.60	1.30	2.13	3.02
9.00	DISCHG	3.93	4.94	6.17	7.49	8.74	9.93	11.24	12.90	14.64	16.22
10.00	DISCHG	17.67	19.27	21.34	23.62	26.66	30.43	34.64	40.34	46.47	53.42
11.00	DISCHG	61.32	69.36	78.90	88.90	101.41	118.21	175.19	333.20	556.83	1013.89
12.00	DISCHG	1699.99	2096.98	1719.05	1144.40	768.99	553.56	432.04	359.65	314.18	280.14
13.00	DISCHG	250.11	228.85	210.77	196.10	183.98	172.76	163.48	153.15	144.06	137.02
14.00	DISCHG	130.97	126.07	120.78	116.04	111.31	106.20	101.68	96.55	92.15	89.77
15.00	DISCHG	88.69	88.25	88.09	87.74	85.38	81.35	78.42	77.04	76.42	76.17
16.00	DISCHG	76.08	76.08	76.11	76.16	76.23	76.29	75.23	71.56	67.76	65.67
17.00	DISCHG	64.73	64.31	64.12	64.06	64.06	64.08	64.12	64.17	63.86	61.43
18.00	DISCHG	57.31	54.29	52.84	52.15	51.85	51.72	51.67	51.66	51.67	51.70
19.00	DISCHG	51.73	51.75	51.78	51.81	51.84	51.86	51.89	51.92	51.67	49.16
20.00	DISCHG	44.95	41.83	40.36	39.68	39.35	39.21	39.15	39.13	39.12	39.14
21.00	DISCHG	39.15	39.17	39.18	39.20	39.21	39.23	39.24	39.25	39.27	39.28
22.00	DISCHG	39.30	39.31	39.33	39.34	39.36	39.37	39.39	39.40	39.41	39.43
23.00	DISCHG	39.44	39.46	39.47	39.48	39.50	39.51	39.53	39.54	39.26	36.73
24.00	DISCHG	32.35	26.82	17.75	9.14	4.30	2.02	0.93	0.42	0.18	0.07
25.00	DISCHG	0.01									

RUNOFF VOLUME ABOVE BASEFLOW = 2.61 WATERSHED INCHES, 2025.05 CFS-HRS, 167.35 ACRE-FEET; BASEFLOW = 0.0 CFS

OPERATION RESVOR STRUCTURE 1
INPUT HYDROGRAPH= 6 OUTPUT HYDROGRAPH= 7
SURFACE ELEVATION= 521.50

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.36	926.87	526.11

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A-10

TIME(HRS) FIRST HYDROGRAPH POINT = 0.0 HOURS TIME INCREMENT = 0.10 HOURS DRAINAGE AREA = 1.20 SQ.MI.

8.00	DISCHG	0.0	0.0	0.0	0.0	0.0	0.0	0.01	0.03	0.08	0.14
8.00	ELEV	521.50	521.50	521.50	521.50	521.50	521.50	521.50	521.50	521.50	521.50
9.00	DISCHG	0.22	0.32	0.45	0.61	0.79	1.00	1.23	1.50	1.80	2.13
9.00	ELEV	521.51	521.51	521.51	521.52	521.53	521.53	521.54	521.55	521.56	521.57
10.00	DISCHG	2.50	2.89	3.56	4.40	5.31	6.34	7.49	8.81	10.34	12.08
10.00	ELEV	521.58	521.60	521.61	521.62	521.64	521.66	521.67	521.70	521.72	521.75
11.00	DISCHG	14.08	16.90	20.49	24.46	28.89	34.03	41.59	55.84	85.50	195.75
11.00	ELEV	521.78	521.82	521.86	521.91	521.95	522.01	522.08	522.21	522.43	522.91
12.00	DISCHG	378.67	618.08	821.28	917.44	923.62	882.30	820.94	753.98	688.28	626.66
12.00	ELEV	523.71	524.76	525.65	526.07	526.09	525.91	525.64	525.35	525.06	524.79
13.00	DISCHG	569.71	517.69	470.77	428.65	391.07	357.56	327.72	301.03	277.02	255.52
13.00	ELEV	524.55	524.32	524.11	523.93	523.76	523.62	523.49	523.37	523.27	523.17
14.00	DISCHG	236.38	219.39	204.27	190.74	178.60	167.60	157.57	148.36	139.85	132.15
14.00	ELEV	523.09	523.01	522.95	522.89	522.84	522.79	522.74	522.70	522.67	522.63
15.00	DISCHG	125.39	119.57	114.63	110.42	106.66	102.99	99.35	95.94	92.92	90.30
15.00	ELEV	522.60	522.58	522.56	522.54	522.52	522.50	522.49	522.47	522.46	522.45
16.00	DISCHG	88.06	86.18	84.59	83.26	82.14	81.21	80.35	79.26	78.47	77.69
16.00	ELEV	522.44	522.43	522.42	522.42	522.41	522.41	522.41	522.40	522.40	522.39
17.00	DISCHG	76.86	76.03	75.25	74.50	73.81	73.16	72.56	72.00	71.46	70.88
17.00	ELEV	522.38	522.38	522.37	522.36	522.36	522.35	522.35	522.34	522.34	522.33
18.00	DISCHG	70.11	69.16	68.12	67.08	66.07	65.12	64.23	63.39	62.61	61.88
18.00	ELEV	522.33	522.32	522.31	522.30	522.30	522.29	522.28	522.27	522.27	522.26
19.00	DISCHG	61.21	60.58	59.99	59.44	58.94	58.46	58.03	57.62	57.23	56.78
19.00	ELEV	522.26	522.25	522.25	522.24	522.24	522.24	522.23	522.23	522.23	522.22
20.00	DISCHG	56.13	55.28	54.34	53.38	52.45	51.56	50.73	49.95	49.23	48.55
20.00	ELEV	522.22	522.21	522.20	522.19	522.19	522.18	522.17	522.16	522.15	522.15
21.00	DISCHG	47.92	47.33	46.78	46.27	45.80	45.36	44.95	44.56	44.21	43.88
21.00	ELEV	522.14	522.14	522.13	522.13	522.12	522.12	522.11	522.11	522.11	522.10
22.00	DISCHG	43.57	43.28	43.02	42.77	42.54	42.33	42.13	41.95	41.77	41.62
22.00	ELEV	522.10	522.10	522.10	522.09	522.09	522.09	522.09	522.08	522.08	522.08
23.00	DISCHG	41.47	41.33	41.21	41.09	40.98	40.88	40.79	40.71	40.62	40.44
23.00	ELEV	522.08	522.08	522.08	522.08	522.08	522.07	522.07	522.07	522.07	522.07
24.00	DISCHG	40.05	39.35	38.20	36.54	34.54	32.47	30.53	28.66	26.88	25.20
24.00	ELEV	522.07	522.06	522.05	522.03	522.01	521.99	521.97	521.95	521.93	521.91
25.00	DISCHG	23.63	22.15	20.76	19.46	18.24	17.10	16.03	15.02	14.35	13.72
25.00	ELEV	521.90	521.88	521.86	521.85	521.84	521.82	521.81	521.80	521.79	521.78
26.00	DISCHG	13.12	12.54	11.98	11.46	10.95	10.47	10.01	9.57	9.14	8.74
26.00	ELEV	521.77	521.76	521.75	521.74	521.73	521.72	521.72	521.71	521.70	521.70
27.00	DISCHG	8.36	7.99	7.63	7.30	6.98	6.67	6.37	6.09	5.83	5.57
27.00	ELEV	521.69	521.68	521.68	521.67	521.67	521.66	521.66	521.65	521.65	521.64
28.00	DISCHG	5.32	5.09	4.86	4.65	4.44	4.25	4.06	3.88	3.71	3.55
28.00	ELEV	521.64	521.63	521.63	521.63	521.62	521.62	521.62	521.61	521.61	521.61
29.00	DISCHG	3.39	3.24	3.10	2.98	2.91	2.83	2.77	2.70	2.63	2.57
29.00	ELEV	521.61	521.60	521.60	521.60	521.60	521.59	521.59	521.59	521.59	521.59

RUNOFF VOLUME ABOVE BASEFLOW = 2.60 WATERSHED INCHES, 2013.37 CFS-HRS, 166.39 ACRE-FEET; BASEFLOW = 0.0 CFS

OPERATION REACH CROSS SECTION 1
INPUT HYDROGRAPH= 7 OUTPUT HYDROGRAPH= 5
LENGTH = 5400.00 FEET INPUT = COEFFICIENTS RELATED TO CROSS SECTIONAL AREA, X= 0.40, M= 1.33

MODIFIED ATT-KIN ROUTING COEFFICIENT = 0.22 PEAK TRAVEL TIME = 0.43

PEAK TIME(HRS)	PEAK DISCHARGE(CFS)	PEAK ELEVATION(FEET)
12.84	700.68	(NULL)

FIRST HYDROGRAPH POINT = 0.0 HOURS TIME INCREMENT = 0.10 HOURS DRAINAGE AREA = 1.20 SQ.MI.

TIME(HRS)											
8.00	DISCHG	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.01	0.02
9.00	DISCHG	0.04	0.08	0.13	0.20	0.29	0.40	0.53	0.68	0.85	1.06
10.00	DISCHG	1.29	1.55	1.84	2.21	2.68	3.25	3.92	4.69	5.58	6.61
11.00	DISCHG	7.79	9.15	10.82	12.91	15.41	18.32	21.71	26.01	32.45	43.92
12.00	DISCHG	76.73	141.98	244.88	369.45	487.88	582.05	646.94	684.54	699.55	697.11
13.00	DISCHG	681.88	657.64	627.39	593.54	557.91	521.85	486.34	452.06	419.42	388.64
14.00	DISCHG	359.87	333.18	308.59	286.04	265.44	246.67	229.58	214.02	199.83	186.87
15.00	DISCHG	175.04	164.31	154.64	145.99	138.30	131.46	125.31	119.70	114.57	109.89
16.00	DISCHG	105.65	101.85	98.46	95.46	92.82	90.52	88.50	86.74	85.13	83.69
17.00	DISCHG	82.39	81.19	80.08	79.03	78.05	77.14	76.28	75.47	74.72	74.02
18.00	DISCHG	73.34	72.64	71.89	71.07	70.21	69.31	68.41	67.50	66.62	65.75
19.00	DISCHG	64.91	64.11	63.35	62.62	61.93	61.29	60.68	60.10	59.56	59.06
20.00	DISCHG	58.57	58.04	57.44	56.77	56.04	55.26	54.46	53.65	52.85	52.07
21.00	DISCHG	51.31	50.58	49.88	49.21	48.57	47.97	47.41	46.87	46.37	45.90
22.00	DISCHG	45.47	45.05	44.67	44.31	43.98	43.67	43.38	43.11	42.85	42.62
23.00	DISCHG	42.40	42.20	42.01	41.84	41.68	41.53	41.39	41.26	41.14	41.03
24.00	DISCHG	40.90	40.71	40.42	39.94	39.20	38.19	36.96	35.57	34.07	32.52
25.00	DISCHG	30.94	29.36	27.80	26.28	24.80	23.38	22.02	20.73	19.49	18.38
26.00	DISCHG	17.37	16.45	15.61	14.82	14.09	13.41	12.78	12.18	11.61	11.08
27.00	DISCHG	10.57	10.09	9.64	9.20	8.79	8.40	8.02	7.67	7.33	7.00
28.00	DISCHG	6.69	6.40	6.11	5.84	5.58	5.34	5.10	4.88	4.66	4.45
29.00	DISCHG	4.26	4.07	3.89	3.72	3.56	3.42	3.29	3.18	3.07	2.98

RUNOFF VOLUME ABOVE BASEFLOW = 2.60 WATERSHED INCHES, 2011.60 CFS-HRS, 166.24 ACRE-FEET; BASEFLOW = 0.0 CFS

EXECUTIVE CONTROL OPERATION ENDCMP COMPUTATIONS COMPLETED FOR PASS 1 RECORD ID 00000220

EXECUTIVE CONTROL OPERATION ENDJOB RECORD ID 00000230

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SUMMARY TABLE 1 - SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL INSTRUCTIONS IN THE ORDER PERFORMED
(A STAR(*) AFTER THE PEAK DISCHARGE TIME AND RATE (CFS) VALUES INDICATES A FLAT TOP HYDROGRAPH.)

SECTION/ STRUCTURE ID	STANDARD CONTROL OPERATION	DRAINAGE AREA (SQ MI)	RAIN TABLE #	ANTEC MOIST COND	MAIN TIME INCREM (HR)	PRECIPITATION BEGIN (HR)	PRECIPITATION AMOUNT (IN)	PRECIPITATION DURATION (HR)	RUNOFF AMOUNT (IN)	PEAK DISCHARGE ELEVATION (FT)	PEAK DISCHARGE TIME (HR)	PEAK DISCHARGE RATE (CFS)	PEAK DISCHARGE RATE (CSM)
ALTERNATE 1 STORM 1													
STRUCTURE 1	RUNOFF	1.20	2	2	0.10	0.0	5.20	24.00	2.61	---	12.10	2097.04	1747.5
STRUCTURE 1	RESVOR	1.20	2	2	0.10	0.0	5.20	24.00	2.60	526.11	12.36	926.87	772.4
XSECTION 1	REACH	1.20	2	2	0.10	0.0	5.20	24.00	2.60	---	12.84	700.68	583.9

SUMMARY TABLE 2 - SELECTED MODIFIED ATT-KIN REACH ROUTINGS IN ORDER OF STANDARD EXECUTIVE CONTROL INSTRUCTIONS
(A STAR(*) AFTER VOLUME ABOVE BASE(IN) INDICATES A HYDROGRAPH TRUNCATED AT A VALUE EXCEEDING BASE + 10% OF PEAK)

XSEC ID	REACH LENGTH (FT)	HYDROGRAPH INFORMATION INFLOW PEAK (CFS)	INFLOW TIME (HR)	OUTFLOW PEAK (CFS)	OUTFLOW TIME (HR)	OUTFLOW+ INTERV. AREA PEAK (CFS)	OUTFLOW+ INTERV. AREA TIME (HR)	BASE-FLOW (CFS)	ROUTING PARAMETERS VOLUME ABOVE BASE (IN)	MAIN TIME INCR (HR)	ITER-ATION #	Q AND A EQUATION COEFF (X)	Q AND A EQUATION POWER (M)	LENGTH FACTOR (K*)	PEAK RATIO O/I (Q*)	S/Q @PEAK (K) (SEC)	ATT-KIN COEFF (C)	PEAK TRAVEL TIME STOR-AGE (HR)	PEAK TRAVEL TIME KINE-MATIC (HR)
ALTERNATE 1 STORM 1																			
1	5400	924	12.4	700	12.8	---	---	0	2.60	0.10	1	.400	1.33	0.160	0.757	1486	0.22	0.40	0.43

SUMMARY TABLE 3 - DISCHARGE (CFS) AT XSECTIONS AND STRUCTURES FOR ALL STORMS AND ALTERNATES

XSECTION/ STRUCTURE ID	DRAINAGE AREA (SQ MI)	STORM NUMBERS..... 1
STRUCTURE 1	1.20	
ALTERNATE 1		926.87
XSECTION 1	1.20	
ALTERNATE 1		700.68

END OF 1 JOBS IN THIS RUN

Note: Each Summary Table would n ally start on a separate sheet.

HYDROLOGIC MODELING FOR FLOOD HAZARD PLANNING AND MANAGEMENT

TR-55 Urban Hydrology for Small watersheds

Wendell A. Styner, P.E. \1

OVERVIEW:

The Soil Conservation Service (SCS) Technical Release 55, "URBAN HYDROLOGY FOR SMALL WATERSHEDS" (TR-55) (1) presents simplified procedures for estimating hydraulic and hydrologic parameters used in the development of runoff volumes and peak rates of discharge in urban and urbanizing areas. It was originally issued in January 1975 for use by SCS field personnel in meeting the ever increasing requests for service in the urban areas. "SCS technicians and engineers were being inundated with requests from soil and water conservation districts, cities, counties, and others to verify and approve erosion and storm water management plans prepared by developers and their consultants. The SCS staff needed a direct approach with a simple set of procedures that gives reproducible results acceptable to both developers and regulators"(2). The TR uses procedures that follow standard SCS methodology as much as possible, with modification where needed to fit the urban problems.

Background: New issue this FALL → may be Jan 1, 86

TR-55 was first issued in January 1975. In the intervening 10 years considerable research has been done in urban hydrology. The SCS is revising TR-55 to incorporate some of the results of this research (3) as well as changes gained from experience in using the original version. The revised publication is intended to enable the user to systematically examine and use the various methods presented. The draft of the revised version has been reviewed by the SCS staffs in selected states, the National Technical Centers (NTC) and the National Office, Washington, D.C. Printing and distribution will be in the fall of 1985.

The major revisions and additions include:

1. A systematic method for selection of the appropriate procedures .
2. Expansion of the chapter on Runoff Curve Numbers (RCN). - Include Arid low procedure
3. Addition of three rainfall temporal distributions. I, II, III & IA
4. A procedure for calculating overland flow travel time in headwater watersheds. - Improved overland flow calcs.
5. Modification of the Graphical (Tc) Peak Discharge Method and the Tabular Method. - micro package in Basic will be released with

\1 Hydraulic Engineer, Engineering Staff, West National Technical Center, Soil Conservation Service, 511 NW Broadway, Portland, Or. 97209

6. Worksheets for each procedure.
7. A computer program.
8. A new storage routing procedure.

OUTLINE OF THE MODEL:

An edited version of Chapter 1 is included here which briefly describes the parameters that are used to develop a runoff hydrograph or peak flow. A flow chart (Fig.1) is included that leads the user through application of TR-55 procedures.

Effects of Urban Development:
The conversion of rural land to urban land often increases the volume of storm runoff, runoff rates and soil erosion as well as other problems that affect soil and water resources. An urban or urbanizing watershed is an area in which all or part of the watershed is or will be covered by impervious surfaces, such as roads, sidewalks, parking lots and houses. Stream channels may be supplemented by some form of artificial drainage system, such as paved gutters and storm sewers.

Urbanization of a watershed changes its response to precipitation. The most common effects are reduced infiltration and decreased travel time, which result in higher peak rates of runoff. The volume of runoff is determined primarily by the amount of rainfall and by infiltration characteristics related to soil type, antecedent rainfall, type of vegetal cover, impervious surfaces, and surface retention. Travel time is determined primarily by slope, flow length, depth of flow, and roughness of flow surfaces. Peak rates of discharge are based on the relationship of the above parameters as well as the total drainage area of the watershed, the location of the development in relation to the total drainage area and the effect of any flood control works or other man-made storage. Peak rates of discharge are also influenced by the distribution of rainfall within a given storm event. The SCS has developed and uses four standard 24-hr rainfall distributions--Types I, IA, II, and III.

Application of the basic model begins with a rainfall amount that is uniformly imposed on the watershed with a prescribed time distribution. The mass rainfall volume is converted to mass runoff volume using a retention parameter called Runoff Curve Number (RCN) or (CN) which is based on soils, vegetative cover, amount of impervious areas, interception and surface retention. The runoff volume is then transformed into a discharge hydrograph using unit hydrograph theory and routing procedures that are dependent upon the time it takes for runoff to travel through various segments of the watershed. The modeling process, therefore, resolves into two areas -- runoff volume and time.

Rainfall:
The 1975 version of TR55 included only one synthetic 24-hour rainfall distribution, the type II rainfall distribution. A type II storm is representative of the dominant thunderstorm intensities that cover most of the country. Three other synthetic 24-hour rainfall distributions were added to the revision of TR-55. These distributions are for storm types I, IA, and III. Type I and IA storms are representative of the Pacific marine climate with wet winters and dry summers. Type III represents the Gulf and Atlantic Coastal areas where tropical storms bring large 24-hour rainfall amounts. The four regional rainfall time distributions are for a 24-hour time period. The 24-hour duration is used because of the general availablity of daily rainfall data which can be used to estimate 24-hour rainfall amounts and will span most of the applications covered by TR-55. Because duration of rainfall is usually selected to be equal to the time of concentration, the rainfall distribution was designed to contain the intensity of any duration from 1 to 24 hours for the frequency of the event chosen.

Runoff:
Chapter 2 describes how to estimate the parameters that are necessary for the determination of runoff volumes. The method used is the SCS Runoff Curve Number (CN) method (4). Parameters used to estimate CN are soil, cover and initial losses. A discussion follows of how urban development affects each.

Soils: No urban area is completely covered by impervious surfaces; therefore, the soil remains an important factor in runoff estimates. Infiltration rates of soils vary widely and are affected by subsurface as well as surface conditions. The urbanization process has greater effect in watersheds with soils having high infiltration rates (sands and gravels) because the of the large change caused by the impervious cover. The silts and clays have lower infiltration rates and thus are impacted less. Urbanization can also modify soils by mixing or removing native soils or introducing new soils with fill material. Any altering of the soil profile can change infiltration characteristics.

Cover: The cover parameter is influenced by the vegetation and the surface cover of the soil. The urban effects of streets, parking lots, roofs and driveways is to reduce surface storage and infiltration. The type and condition of the vegetation in urban areas is also important because even in intensively developed subdivisions much of the area is pervious. Consideration needs to be given to areas in transition where pervious areas have no cover.

Initial Losses: In the SCS CN model, losses that occur before runoff begins are termed "initial abstraction." These losses include water retained in minor depressions, intercepted by vegetation, evaporation, and infiltration prior to runoff. This is a highly variable condition but generally correlates with the soil and cover parameters. This regression was applied to many small watersheds and it was found that the initial abstraction was approximately twenty percent of the maximum potential retention. Using this approximation removes the initial losses as an independent parameter and allows a given CN and rainfall combination to produce a unique runoff amount. This approximation can be especially important in an urban application because the combination of impervious areas with pervious areas can imply a significant initial loss that may not take place. The user should understand the initial abstraction assumption and be satisfied that the assumption applies to the situation.

Time Parameters:
Chapter 3 describes a method for estimating the parameters used to distribute the runoff into hydrographs. The method presented is empirically derived from actual runoff hydrographs and watershed characteristics and is based on the velocities that water travels through the various segments of the delivery system. Since the method for estimation of Time of Concentration (Tc) and Travel Time (Tt) is based on flow velocities, the parameters involved are the same as those contained in accepted open channel hydraulic analysis. These are surface roughness, channel shape and slope.

In the upstream area of small watersheds, the overland flow can be considered as sheet flow over plane surfaces. With sheet flow, the friction factor is an effective roughness coefficient that includes the effect of raindrop impact; drag over the plane surface; obstacles such a litter and crop ridges; erosion; and transport of sediment. For flow lengths of less than 300 feet, a simplified form of the Manning-Kinematic wave equation (Eq.1) was added to the revision to compute travel time of overland flow.

$$Tt=(0.007*(nL)^{0.8})/(((P2)^{0.5})*S^{0.4}) \qquad \text{Eq.1}$$

where:
- Tt = travel time (hours)
- n = Manning's "n" value
- L = flow length (feet)
- P2 = 2-year 24-hour rainfall depth (inches)
- S = slope (feet/foot)

A table listing Mannings "n" values for various surfaces has been added to the revised technical release.

Surface Roughness: One of the most significant effects of urban development on flow velocity is the reduction of the retardance of flow. Areas that naturally have very slow overland flow through vegetation at shallow depths are modified to deliver the flow to streets, gutters and storm sewers, which transport runoff downstream as rapidly as possible; thus, the travel time through the watershed is decreased.

Channel Shape and Flow Patterns: The channel shape and flow patterns are modified in urban development much the same as surface roughness. In small natural watersheds a significant portion of the travel time results from overland flow in upstream areas. Urbanization tends to reduce overland flow lengths by conveying storm runoff into a channel as soon as possible. Channel designs have efficient hydraulic characteristics to increase runoff flow velocity and decrease the travel time.

Slope: Slopes can be increased or decreased by urbanization depending on how significantly storm sewers and street ditches are used in the design of the water management system, or the extent of site grading. Since it is unlikely that maximum and minimum elevations will be changed, the slope will be determined by the flow length. Slopes will tend to increase when channels are straightened and tend to decrease when natural overland flows are diverted through storm sewers, street gutters and diversions.

PEAK DISCHARGE AND HYDROGRAPHS

Chapter 4 contains an approximate method to make estimates of peak rates of discharge and Chapter 5 has an approximate method for obtaining and/or routing hydrographs. Both of these methods were derived from hydrographs prepared by procedures outlined in Chapter 16 of the National Engineering Handbook, Section 4, Hydrology (NEH-4) and Technical Release 20.

The two methods described in Chapters 4 and 5 are intended to be used in specific ways with specific basic data. If improper basic data or adjustments are used they will exceed those expected. The flow chart in Figure 1 outlines a systematic approach for selection of the appropriate methods.

The first decision is to determine if data are available to estimate the Tc. If the data are not available TR-55 should not be used. If the data are available then the next decision is to determine if a discharge hydrograph is needed. A hydrograph will be needed if a routing (other than an approximate method) will be performed or if subwatersheds are going to be combined to account for dissimilar hydrologic or hydraulic conditions. The Tabular method is the only procedure in TR-55 that yields a hydrograph.

After deciding to use either the Tabular or Graphical methods the user estimates the runoff volume from Chapter 2. The time of concentration (Tc) is then estimated and, if the Tabular method is used, the travel time (Tt) is also estimated from Chapter 3. Then the peak discharge using the Graphical method of Chapter 4 or the runoff hydrograph using the Tabular Hydrograph method is computed.

The flow chart converges for the user to determine if the effect of a storage basin at the outlet is to be analyzed.

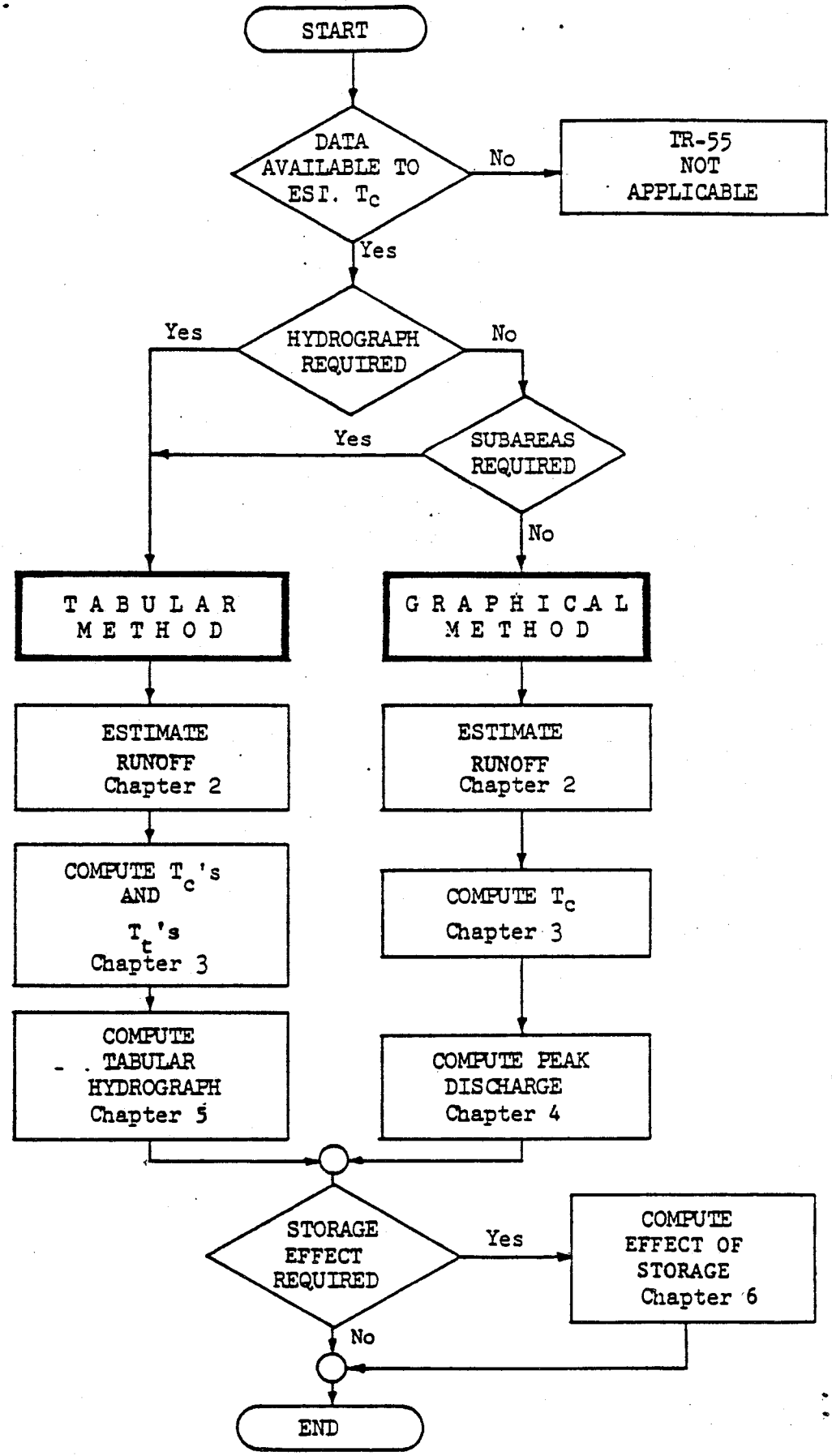

Fig. 1 -- Selecting the Appropriate Procedure in SCS TR-55.

The revision of TR-55 introduces a new parameter, Ia/P, to the methods for estimating peak discharge to discriminate among similar shaped hydrographs and their resulting peak discharges. Ia is initial abstraction and P is 24-hour rainfall. SCS procedures for hydrograph generation are based on unit hydrograph theory, therefore, classifying hydrographs by unit runoff response is desirable. The Ia/P value is a dimensionless ratio that permits grouping of peak flow response from watersheds under a wide range of rainfall amounts.

For a given 24-hour rainfall distribution, Ia/p represents the fraction of rainfall that must occur before runoff begins. Vary the Ia/P and the runoff response time changes. Hold Ia/p constant and the unit runoff response is identical no matter what combination of CN and P is used to form the Ia/P ratio.

STORAGE EFFECTS

Chapter 6 outlines procedures to account for the effect of storage and provides a shortcut method to estimate temporary flood storage based on hydrologic data developed from the Graphical or Tabular methods. A new procedure for storage routing through single-stage structures and a technique for routing through a two-stage structure is described.

Since urbanization can be expected to increase runoff volume and peak discharge, one of the primary uses of TR-55 is to find ways to offset the efect. Chapter 6 also discusses how floodwater detention can be used to modify hydrographs with storage to reduce the downstream impact of urbanization. The most credible course of action is to use storage to reduce the peak discharge to approximate the predevelopment condition. The shortcuts in Chapter 6 can be helpful in sizing detention basins even if the final design requires more detailed analysis.

LIMITATIONS

Decisions on which procedure to use will depend primarily on the type of output desired and the basic input data available. All of the procedures described in TR-55 are simplified procedures with parameter assumptions that lead to reduced time of application. These simplifications also introduce limitations and potentially decrease the accuracy when compared with more detailed methods.

The user should be aware of the inherent simplification in the procedures and should not ignore the effects of the resulting error. It is desirable to examine the sensitivity to a variation of the peak disharge or hydrograph on the analysis being conducted to ensure that the simplifications will not lead to intolerable results. It is expected that the variations due to the parameter assumptions will not significantly affect most designs or conclusions. If the evaluation of the methods indicated that the errors are unacceptable, the user should either allow for the error or choose a more detailed procedure.

The methods described in TR-55 are designed to be used where flow is overland and in channels. Where flow is put into underground conduits, the hydrograph relationships determined by Tc are no longer valid. TR-55 should, therefore, be used only to compute discharge at inlets to closed systems.

The Graphical method and the Tabular method are derived from TR-20 output and differ only in the product. Their accuracy is, therefore, comparable. The use of the Tc permits these two methods to be used for any size watershed within the scope of the curves or tables. The Graphical method is used only for hydrologically homogeneous watersheds because the procedure is limited to a single watershed subarea.

The Tabular method can be used for a non-homogeneous watershed divided into any number of homogeneous subwatersheds because hydrographs can be routed and added. This flexibility greatly increases the potential applications.

The approximate storage-routing curves of Chapter 6 should not be used if the adjustment of ponding is used. These storage-routing curves, like the peak discharge and hydrograph procedures, are approximations derived from TR-20 data and should be used with a complete understanding of the expected error.

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3. McCuen, Richard H., Rawls, Walter J., Wong, Stanley L., "SCS Urban Peak Flow Methods," Journal of the Hydraulic Engineering Division, ASCE, Vol. 110, No. 3, March 1984, pp. 290-299.
4. Soil Conservation Service, National Engineering Handbook, Section 4, Hydrology, 1972.

HEC-1

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HEC-1 -- FLOOD HYDROGRAPH PACKAGE

o OVERVIEW

Publications

HEC-1, Flood Hydrograph Package, Users Manual, Sept. 1981, 192 pp.	$12.00
HEC-1, Flood Hydrograph Package, Programmers Manual, Oct. 1981, 136 pp.	$6.50
Flood Control System Component Optimization - HEC-1 Capability, Training Document 9, Sept. 1977, 208 pp.	$8.50
Introduction and Application of Kinematic Wave Routing Techniques Using HEC-1, Training Document 10, May 1979, 100 pp.	$5.00
Hydrologic Analysis of Ungaged Watersheds with HEC-1, Training Document 15, Apr. 1981, 172 pp.	$8.00
Application of Spatial Data Management Techniques to HEC-1 Rainfall-Runoff Studies, Training Document 19, Oct. 1983, 72 pp.	$4.00
Determining Peak-Discharge Frequencies in an Urbanizing Watershed: A Case Study, Technical Paper 64, July 1979, 15 pp.	$2.00
Corps of Engineers Experience with Automatic Calibration of a Precipitation-Runoff Model, Technical Paper 70, May 1980, 12 pp.	$2.00
Flood-Runoff Forecasting with HEC1F, Technical Paper 106, Feb. 1985, 7 pp.	$2.00

All of the above publications are available from the Hydrologic Engineering Center. To order, send your request, together with payment made payable to "FAO-USAED, SACRAMENTO" to:

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A Publications Catalog, Computer Program Catalog and Video Tape Loan Library Catalog are available, free of charge. The latter lists video tapes made during lectures presented in training courses (including courses on HEC-1) given at the Center. The tapes are in a 3/4 in. cassette format and may be borrowed free of charge. The Video Tape Loan Library Catalog contains an order form.

Program Acquisition

HEC-1, and other computer programs, may be obtained from HEC at the address given above. The handling fee is $200. for the first program of an order, plus $100. for each additional program of a multiprogram order. The handling fee includes a magnetic tape containing the program(s) and test data, and one copy each of a users manual and a programmers manual (if available) for each program ordered.

A microcomputer version of HEC-1 is available. It requires a 10 megabyte hard disc, at least 512 kilobytes of random access memory (RAM). and the MS-DOS 2.0 or newer operating system. The microcomputer must also have at least one disc drive for 5 1/4 inch double density discs. Another accessory which is helpful, but not necessary, is the 8087 Math Processor Chip, which will reduce the run time significantly. Also, to be able to print HEC-1 output, a printer capable of printing 132 columns is necessary. The handling fee for obtaining microcomputer versions of programs is $200. for the first program and $100. for each additional program on the same order.

Training

Several universities periodically offer extension courses in hydrologic engineering which focus on the use of HEC computer programs, including HEC-1. These courses are generally comparable to courses presented by the HEC and, in many instances, utilize publications and training materials developed by the HEC. Information concerning scheduling and fees may be obtained from the following:

Mr. James Lapsley
University Extension
University of California, Davis
Davis, CA 95616
phone: (916) 752-0880

Mr. W, E. Kinnebrew
Continuing Engineering Education
1700 Asp Avenue
University of Oklahoma
Norman, Oklahoma 73019

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212 Sachett Building
University Park, PA 16802
phone: (814) 865-9173

HEC MODELS FOR WATER RESOURCES SYSTEM SIMULATION: THEORY AND EXPERIENCE

ARLEN D. FELDMAN

The Hydrologic Engineering Center
Davis, California

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ADVANCES IN HYDROSCIENCE, VOL. 12

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ISBN 0-12-021812-7

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- MODEL THEORY AND STRUCTURE
- PROGRAM STRUCTURE AND IMPLEMENTATION

The following pages are excerpted from Advances in Hydroscience, Volume 12, edited by Ven Te Chow, Academic Press, 1981.

III. The HEC-1 Flood Hydrograph Package

A. Purpose

The main purpose of the "HEC-1 Flood Hydrograph Package" (HEC, 1980a) is to simulate the hydrologic processes during flood events. The precipitation (rainfall, snowfall/melt) to runoff process can be simulated for large complex watersheds such as depicted in Fig. 4. Flood event analysis was the motivation for developing the HEC-1 program. The Corps of Engineers uses this model as a basic tool for determining runoff from various historical and synthetic (or design) storms in planning flood control measures. The HEC-1 has several major capabilities which are used in the development of a watershed simulation model and the analysis of flood control measures. Those capabilities are the following:

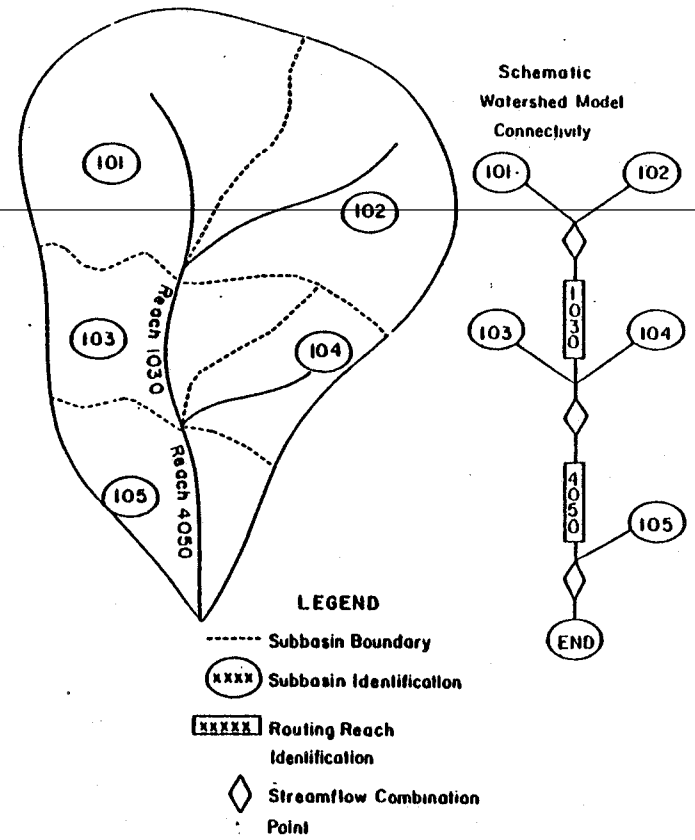

FIG. 4. Typical HEC-1 watershed model components.

Automatic estimation of unit graph, interception/infiltration and streamflow routing parameters.

Simulation of complex river basin runoff and streamflow.

River basin simulation using a precipitation depth versus area function.

Computation of modified frequency curves and expected annual damages for any location in the stream system and automatically for several flood control plans throughout the watershed.

Simulation of flow through a reservoir and spillway for dam safety analysis.

Simulation of dam breach hydrographs.
Optimization of flood control system components.

The automatic parameter estimation capability relates to the determination of subbasin runoff parameters by a univariate search procedure. The unit hydrograph, precipitation interception/infiltration rates (hereafter referred to as precipitation loss rates), and streamflow routing parameters may be optimized for individual storm events based on observed precipitation and streamflow data for a single subbasin. Streamflow routing parameters may also be optimized from known inflow and outflow in a river reach.

Watershed precipitation–runoff simulation is the main function of the program and the basis for the other capabilities. The watershed model as referred to in this discussion includes all aspects of the precipitation and runoff computations necessary to simulate streamflow in a complex river basin. The model may be used to simulate runoff in a simple, single-basin watershed or in highly complex basins with a virtually unlimited number of subbasins and routing reaches in which interconnections may exist. Watershed runoff simulation is the basic mechanism upon which a number of specialized capabilities are based.

The "HEC-1 Flood Hydrograph Package" program is so labeled because the original version of the program was developed by linking together several smaller, special-purpose watershed simulation programs. The watershed runoff component programs were developed by Leo R. Beard, who was director of the HEC at that time. Section VI briefly describes these component programs.

B. Description of the Physical System

The HEC-1 watershed model uses spatially and temporally lumped (or averaged) parameters to simulate the precipitation and runoff process. The time and/or space discretization may be changed by modifying the size of subbasins, routing reaches, and/or the computation interval. There are virtually no limitations on the sizes of the components or the computation interval. The user selects the sizes of these variables that are consistent with the degree of detail desired in the computational results, the allowable modeling efforts, project budget, and the available data.

The HEC-1 is a "single event" model intended to simulate discrete storm events. This is in contrast to a "continuous" watershed model which simulates multiple years of hydrologic record, e.g., the Stanford Watershed Model (Crawford and Linsley, 1966).

A typical watershed with subbasins and routing reach components is shown in Fig. 4. The HEC-1 simulates the precipitation, precipitation loss, and runoff processes in each subbasin, routes streamflow downstream, and combines tributary inflows.

1. *Theoretical Bases*

a. Precipitation. Precipitation is computed (or directly input) separately for each subbasin shown in Fig. 4. There are three ways in which historical storm data may be input:

(i) Historical subbasin average cumulative or incremental precipitation.
(ii) Historical subbasin average storm total precipitation together with an incremental or cumulative time pattern for distribution.
(iii) Historical gage data (recording and nonrecording) together with Thiessen (Chow, 1964) or other weightings with which to compute subbasin average precipitation.

And there are three types of standardized synthetic storms that may be used:

(i) Synthetic storm depth versus duration data.
(ii) Synthetic Probable Maximum Precipitation (National Weather Service, 1956).
(iii) Synthetic Standard Project Precipitation (Corps of Engineers, 1952).

All time series precipitation data are input as end-of-interval values. The date, time, and interval length of the input precipitation series must be specified, as must the desired beginning of the simulation and the simulation time interval. If the desired computation time interval is different from the input data time interval, the program will interpolate (using a three-point spline function) the input precipitation series in order to determine the precipitation for the required computation intervals. The simulation is performed at a constant time interval but the data may be at any time interval.

b. Snowfall and Snowmelt. The foregoing discussion of precipitation applies to both snowfall and rainfall simulation. For snow simulation, each subbasin is discretized into elevation zones. The same amount of precipitation is assumed to fall in each elevation zone, as depicted in Fig. 5. An average air temperature time series is input for the subbasin outlet and a lapse rate is used to compute the average temperature of each elevation zone. If the average elevation zone temperature is less than or equal to a specified snowfall temperature threshold, the precipitation is treated as snow and is accumulated in that zone's snowpack. The water content of the initial snowpack (if any) is specified. All snowpack computations are in terms of the water equivalent (mm or inches) of the snow.

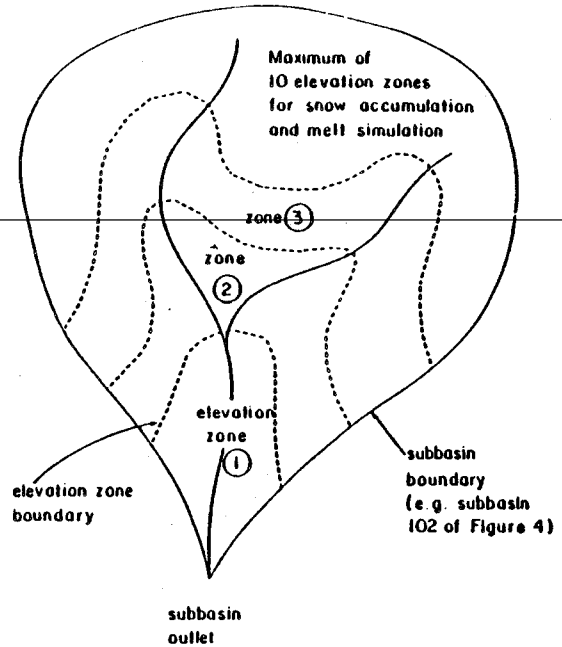

FIG. 5. The HEC-1 subbasin elevation zones for snow simulation.

Snowmelt may be computed by either the simple degree-day (Chow, 1964) or the complex energy budget (Corps of Engineers, 1960b) method. With either method, the snowmelt and pack water balance is computed separately for each elevation zone. A subbasin may be void of snow or partially or totally covered with snow.

c. INTERCEPTION/INFILTRATION (LOSS RATE ANALYSIS). The interception/infiltration part of the rainfall/snowmelt runoff process may be simulated by four different methods using HEC-1, as shown in Table II. These methods simulate the interception of precipitation by vegetation, depressions, etc. and the accumulation of moisture in the soil.

The initial and constant loss rate function (Linsley *et al.*, 1975), is the simplest form of all loss rate functions. The loss L, in millimeters (inches), for a time interval t, in hours, is

$$L = \begin{cases} P & \text{if } \sum L \le I, \\ Ct & \text{if } \sum L > I, \end{cases} \tag{1}$$

TABLE II

HEC-1 INTERCEPTION/INFILTRATION METHODS

Method	Parameters	Description
Initial and Constant	Initial volume loss and a constant infiltration rate	Initial loss volume is satisfied, then constant loss rate begins.
HEC Exponential	Infiltration rate, antecedent moisture condition, rate of change of infiltration with wetness	Initial infiltration rate adjusted for antecedent conditions and continuous function of soil wetness.
SCS Curve Number	Curve number	Initial interception loss satisfied before computing cumulative runoff as a function of cumulative rainfall.
Holtan	Infiltration rate capacity, available soil moisture storage	Infiltration rate computed as exponential function of available soil moisture storage and is limited by ultimate infiltration rate for saturated soil.

where I is an initial loss, in millimeters (inches), representing antecedent soil moisture conditions and interception losses; C is a constant loss rate, in millimeters per hour (inches per hour), which is representative of soil moisture infiltration; and P is the rainfall/snowmelt in millimeters (inches). If I is satisfied during a time interval, C applies only to the remainder of that time interval after I is satisfied. The C is also referred to as the ϕ index ($I = 0$) and represents the average infiltration rate, throughout the entire storm event, which produces the observed precipitation excess for that storm. Precipitation excess is that part of the precipitation which results in runoff during that period and is not lost to interception/infiltration. The initial loss and constant loss rate are often used in synthetic (design) storm runoff simulation and where inadequate data are available to justify use of the more complex methods.

The HEC exponential loss rate function simulates the interception/infiltration process as a function of accumulated soil moisture (losses not available for runoff) as shown in Fig. 6. The equation for computation of loss rate L is

$$L = AP^E \tag{2}$$

where A represents the combined effect of interception and infiltration rates as a function of accumulated losses (soil moisture), P is the precipitation intensity, and E is an exponent regulating the effect of the precipitation intensity term. The precipitation intensity factor is used to represent nonlinearity in the loss rate process for heterogeneous watersheds.

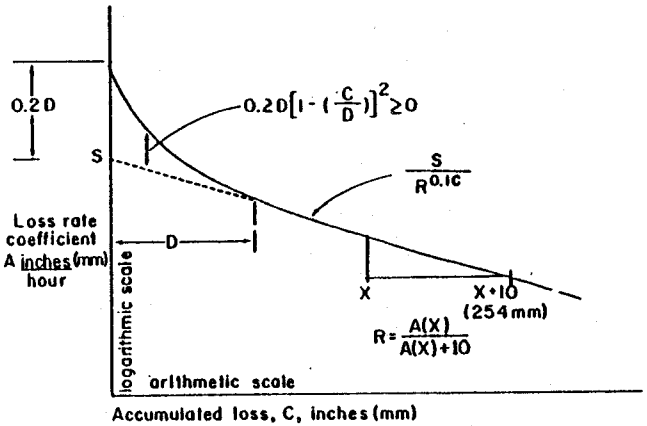

FIG. 6. The HEC exponential loss rate function. S is the loss rate for average soil moisture conditions; D, initial amount of loss for which the loss rate coefficient is increased to represent antecedent soil moisture conditions; R, rate of change of loss rate coefficient as soil moisture increases.

The HEC exponential loss rate equation is a logical function of the soil moisture accumulation; however, it is an analytical function whose parameters are not readily determined from measurable watershed characteristics. Thus, the function is difficult to apply in ungaged areas where the loss rate parameters must be related to the variable soil type and land cover geographic characteristics in a watershed. The parameters are generally obtained using the automated parameter estimation capability of HEC-1. A regional relationship may be developed between the derived parameters and watershed characteristics. Derivation of the loss rate and other watershed parameters is discussed on page 332.

The two other loss rate options in HEC-1 were developed by the U.S. Department of Agriculture's Soil Conservation Service (SCS) and the Science and Education Service (SES). One option employs the SCS "Curve Number" (SCS, 1965) and an additional parameter, a wetness multiplier, which is used to modify the given curve number for antecedent moisture conditions. The most attractive feature of the SCS curve number technique is its direct relation to the land use and soil properties of a watershed. That feature makes it easily applicable to ungaged areas. The curve number technique has been applied in urban areas (SCS, 1975).

H. Holtan of the SES (formerly the Agricultural Research Service) developed a loss rate function (Holtan *et al.*, 1975) which is related to watershed characteristics and also a more sophisticated function of accumulated soil moisture. The Holtan loss rate function has the same general form as the HEC exponential loss rate function but does not consider precipitation intensity; however, the Holtan parameters are derived directly from the soil water infiltration characteristics of the watershed. Although the Holtan loss rate parameters are closely related to watershed interception and infiltration characteristics, the parameters must still be calibrated because of the nonhomogeneity of the soils, vegetation, etc. throughout a subbasin.

The Holtan infiltration function as implemented in HEC-1 is given by the equation

$$L = aS^e + f_c \tag{3}$$

where L is the loss rate in inches per hour; a, is the infiltration capacity in inches per hour per (inch)^e of available storage; S is the available storage in inches water equivalent; e, is the exponent of the storage S; and f_c is the constant rate of infiltration after prolonged wetting in inches per hour.

An impervious area parameter may be used with any of the loss rate functions. Imperviousness is specified as a percent of the subbasin area. The amount of loss (millimeters or inches) computed in any computation time interval is reduced by the impervious area factor. Thus, 100% runoff occurs from that portion of the subbasin that is impervious.

The portion of the rainfall/snowmelt not lost to soil moisture etc. is referred to as precipitation excess. The next step in the HEC-1 simulation is to convert a hyetograph of rainfall/snowmelt excess into a runoff hydrograph from the subbasin.

d. SURFACE RUNOFF. The HEC-1 program simulates direct surface runoff by convoluting the rainfall/snowmelt excesses through a unit hydrograph function or by the kinematic wave transformation. Rainfall/snowmelt excesses are computed for each time interval by subtracting the interception/infiltration loss from the incoming rainfall and/or snowmelt. Several alternative approaches for specifying unit hydrographs may be used in the watershed simulation. If unit graph ordinates are known, they may be input directly. The three unit graph functions in the program are based on methods developed by Clark (1945), Snyder (1938), and SCS (1965). A time-area curve representing the shape of the basin and related travel times to the basin outlet may be used with the Clark or Snyder methods.

Because the Snyder coefficients used in HEC-1 define the coordinates of only the peak of the unit hydrograph, the Clark method is used as a curve-fitting mechanism to produce a complete unit hydrograph that has peak coordinates that are consistent with the user-specified Snyder coefficients. The Clark method is used to compute a unit graph and then the corresponding Snyder parameters T_p (time to peak) and C_p (peaking coefficient) are

determined. The derived T_p and C_p are then compared to the input Snyder data and adjustments are made to the Clark parameters until the desired Snyder coefficients are obtained. After the unit graph ordinates are determined, direct runoff at the basin outlet is calculated.

The kinematic wave (Woolhiser, 1975) runoff transformation in HEC-1 has been especially developed for simulation of runoff in urban areas. The kinematic wave method produces a nonlinear runoff response to rainfall excesses as compared to the linear response of the unit graph. The HEC-1 version of the kinematic wave process was adapted from the MIT Catchment Model (Harley, 1975), and is described in detail in Training Document No. 10, HEC (1979g). The following is a brief summary of that methodology.

The objective of HEC's utilization of the kinematic wave technique was to associate parameters as closely as possible with measurable watershed characteristics such as slopes, land use, and storm drains. Such an approach is desired to reduce ambiguities in the calibration process and to enable parameter estimation for future land use conditions.

Figure 7 shows the three basic components that may be used in application of the kinematic wave transformation. First, a typical drainage area is identified within the HEC-1 subbasin. The subbasin size may be increased or decreased as necessary to include areas of common drainage characteristics. The typical drainage area is one which has similar lot sizes, overland flow lengths, and street and gutter connectivity throughout the area. The overland flow kinematic wave computations are made for the typical drainage area on a unit width basis according to

$$Q = ay^m \tag{4}$$

where Q is the discharge; a and m are kinematic wave routing parameters; and y is the depth of flow.

After determining the overland flow from the typical drainage area, the overland flow may be routed through one or two levels of collector channels before entering the main channel. The collector channels may be of virtually any standard shape found in urban areas: circular, trapezoidal, triangular, etc. The inflow to the collector channels is laterally distributed, both for the land surface runoff and from one collector to the next, as shown in Fig. 8.

The typical drainage area may be any size with respect to the HEC-1 subbasin. The subbasin's total contribution to the main channel is determined by multiplying the runoff from the typical drainage system by the number of typical drainage systems (including fractional parts) in the HEC-1 subbasin area. The following equation describes this computation.

$$QSB = QDA \times \frac{\text{(subbasin area)}}{\text{(typical drainage system area)}} \tag{5}$$

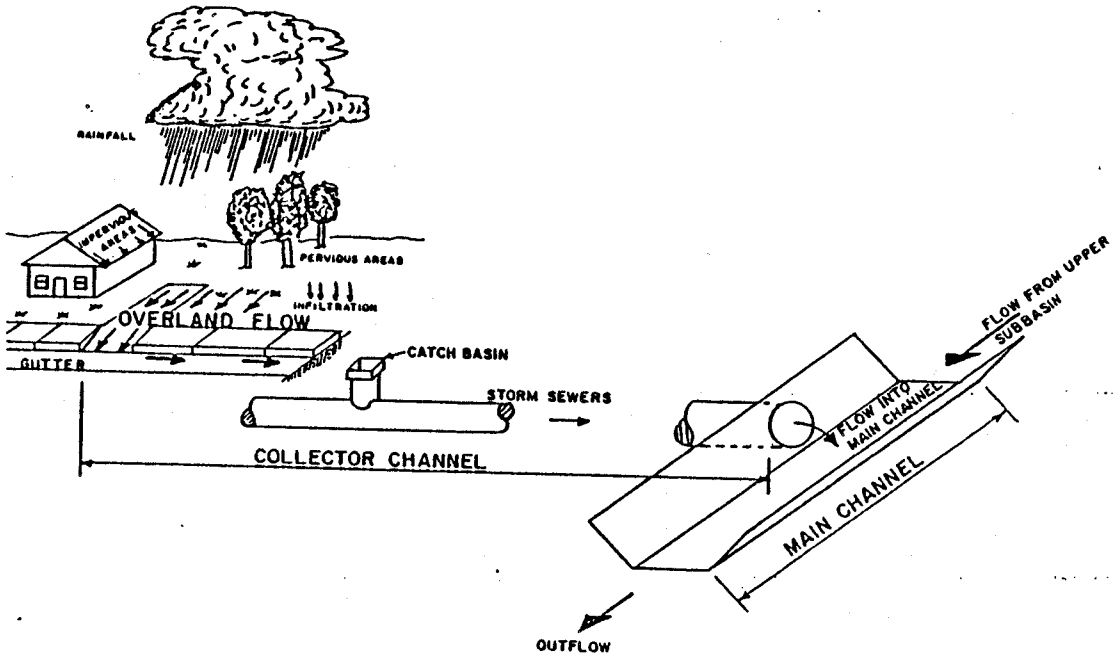

FIG. 7 The HEC-1 kinematic wave simulation of an urban area.

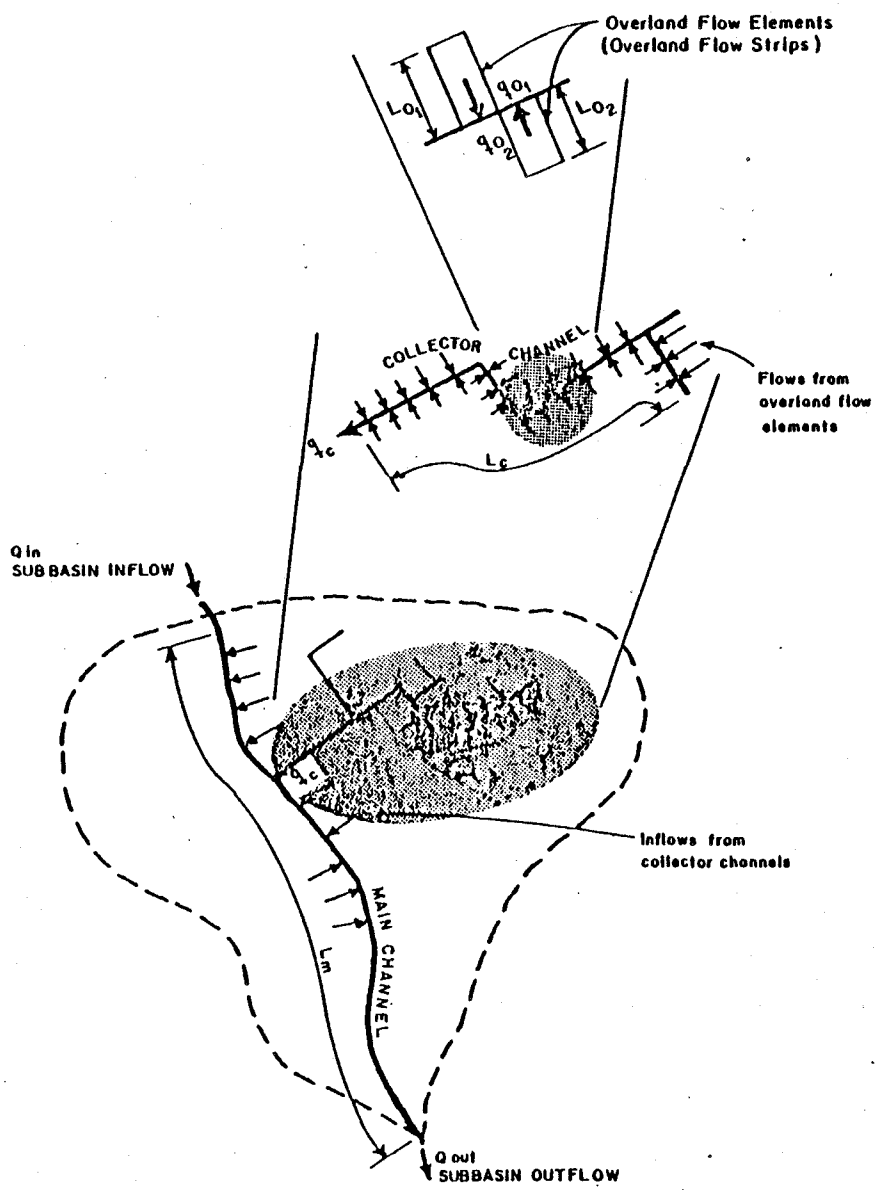

FIG. 8. The HEC-1 kinematic wave subbasin runoff and collector channels.

where QSB is the HEC-1 subbasin outflow during period i, and QDA is the typical drainage system outflow during period i. This proportioning for the lateral distribution is performed for each level of the collector system. The total subbasin outflow QSB may be lumped at either end of the main channel reach or laterally distributed.

c. BASEFLOW AND TOTAL SUBBASIN RUNOFF. The baseflow simulation in HEC-1 employs a logarithmic decay function. There are two baseflow components, each of which use the same decay constant. First, the initial flow (baseflow from some previous storm) is recessed at the decay rate as shown in Fig. 9.

The surface runoff, computed from the precipitation excess and unit graph/kinematic wave, in each time period, is added to the recessed starting flow. This process is continued until a recession flow threshold (Q_R in Fig. 9) is reached. At that time, the flow is under baseflow recession control and the logarithmic decay constant is used to compute subsequent flows. Subsequent rainfall excesses do not influence the computed hydrograph until they are large enough to cause the hydrograph to rise above the recession-controlled discharges. Threshold Q_R may be zero, in which case the only baseflow is that obtained by recessing the initial flow. Any multipeaked, complex hydrograph may be simulated in this manner. It should be emphasized that HEC-1 was developed for use in flood hydrograph studies in which baseflow is relatively unimportant.

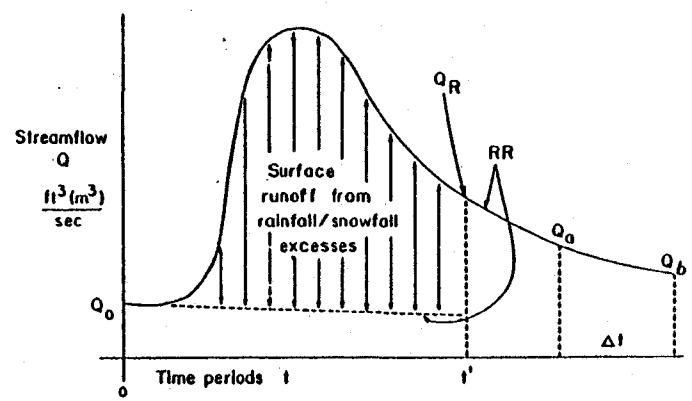

FIG. 9. The HEC-1 baseflow and total hydrograph simulation. Q_R is the recession threshold; RR, recession rate. $RR = (Q_a/Q_b)^{1/\Delta t}$. $Q_t = Q_0/RR^t$ + surface runoff if $t < t'$; $Q_t = Q_R \cdot RR^{t-t'}$ if $Q < Q_R$ on falling limb.

TABLE III

HEC-1 ROUTING METHODS

Method	Parameters	Description
Muskingum	Wedge storage coefficient and travel time through reach	Outflow from reach is computed as a function of several previous inflows to reach (linear).
Modified Puls	Table of storage versus outflow for each river reach	Outflow from river reach computed as function of inflow plus outflow continuity and current storage (nonlinear).
Kinematic Wave	Channel shape, length, slope, and roughness	Outflow from river reach computed as function of depth of flow (nonlinear).

f. STREAMFLOW ROUTING. There are several types of flood routing functions which may be used in the HEC-1 watershed simulation model. Table III summarizes these methods.

The Muskingum method, also described in the Corps of Engineers' EM 1110-2-1408 (1960a) and the work of Chow (1964), describes outflow from a reach in one time interval as a linear function of weighted discharges of the previous time periods. The coefficients are determined from prism and wedge storage of the valley of the routing reach. The Muskingum parameters (a time constant K, a wedge size x, and number of subreaches) may be derived directly from historical streamflows. Good results have been obtained with this method for streams with small or no floodplains (valley storage) where there is a linear relationship between storage and weighted discharge. Typical applications use the Muskingum method to route flows in upstream areas where floodplains are less developed, and a storage routing method (such as modified Puls) in the downstream, larger-floodplain reaches.

The modified Puls technique (Chow, 1964) employs a storage-discharge relationship as shown in the following equation of continuity:

$$\frac{S_2}{\Delta t} + \frac{O_2}{2} = \frac{S_1}{\Delta t} + \frac{O_1}{2} + \frac{(I_1 + I_2)}{2} - O_1 \qquad (6)$$

where O is the outflow from the reach, I is the inflow to the reach, and S is the storage in the reach. The subscripts 1 and 2 refer to the beginning and end of period values. Using this relationship, the average inflow, and the storage discharge relationship computed from steady-flow water surface profiles, the outflow from a reach may be computed directly. The storage discharge curve may be computed with a water surface profile program, such as HEC-2, which computes a continuous water surface profile throughout the river network, or by a simplified independent reach method. The independent reach method utilizes normal depth water surface profile computations for each routing reach independently, i.e., there is no continuity between the water surface elevations of different reaches for the same discharge. Using this method in HEC-1 requires the input of coordinates for a typical cross section, a slope and length for the main channel, and roughnesses for the channel and overbanks. The program computes a series of normal depths for the reach and determines the storage associated with each discharge. The resulting storage–discharge relationship is used in the modified Puls routing.

The kinematic wave (Woolhiser, 1975) channel routing method may also be used, but on an independent reach basis. This method does not allow flood peak attenuation due to overbank storage as the modified Puls method does, but it does allow for laterally distributed local inflow if desired. A detailed description of the kinematic wave routing method is found in HEC's Training Document No. 10, 1979.

g. RIVER NETWORK. The watershed simulation in HEC-1 is accomplished in a converging tree network as shown in Fig. 4. The simulation must begin at the uppermost subbasin of a stream branch. *The connectivity of the watershed subbasins and routing reaches is implied by the order of these elements in the data deck.* For example, in the watershed shown in Fig. 4, the outflows from subbasins 101 and 102 must be combined before being routed through the river reach 1030. That routing must also be accomplished before computing the runoff from subbasin 103. When a routing operation is specified, the program automatically routes the last hydrograph computed (which also remains in the computer storage). After the routing is performed, the local runoff from subbasin 103 is computed and the routed and local flows are combined.

Streamflow diversions may be performed at any point in the stream network. The diverted flows may be treated as a normal hydrograph and routed and combined through a new branch of the stream network. The diverted flows may be returned to virtually any point of the channel network. Figure 10 exemplifies a river network schematic with diversions and return flows. The diversion and return flow capability allows simulation of extremely complex stream networks. The diversion is accomplished by inputting a total channel flow versus diverted flow relationship. The algorithm does not allow diversion to be computed as a function of water surface elevation at this time. One

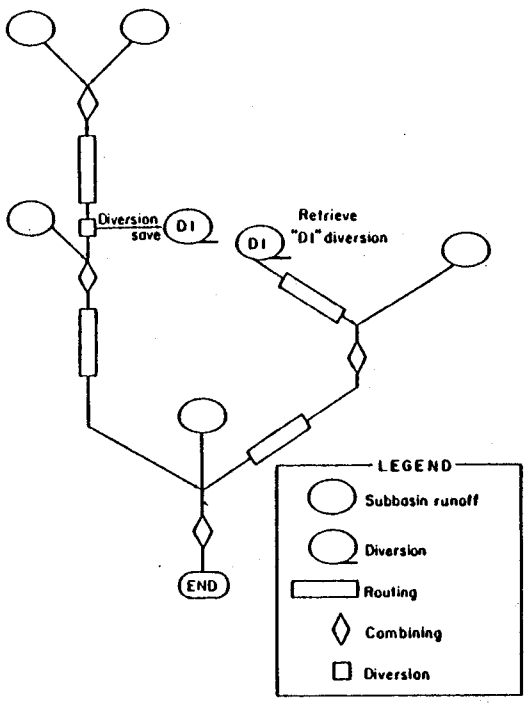

FIG. 10. Schematic illustration of streamflow diversions and return flow.

must first analyze the hydraulics of the diversion location and precompute the main channel versus diverted flow table for use by HEC-1.

A pumping simulation algorithm is also incorporated in HEC-1. Its usage has been primarily for interior drainage (removing water from behind levees etc.) simulation but it can also be used to simulate a diversion which is a function of storage or elevation in the routing reach. A pumping rate versus water surface elevation at the site is input to HEC-1. The water removed by pumping can also be handled as a new hydrograph, which can be routed through a new runoff network in the same manner as the diversion hydrograph.

2. *Assumptions and Limitations*

a. TIME INTERVAL AND SIZE OF WATERSHED COMPONENTS. As discussed at the beginning of the previous section, the HEC-1 model assumes spatially and temporally averaged inputs and parameters (precipitation, infiltration, etc.) within each subunit (subbasin or routing reach) of the watershed. These assumptions limit the size of these subunits. The hydrologist must decide what size of each component is sufficient to model the watershed with the degree of definition required in the study.

b. SUBBASIN RUNOFF. The runoff from a subbasin (from rainfall and/or snowmelt) is assumed to be primarily the result of direct surface runoff; i.e., the infiltration losses cannot be directly included in the runoff hydrograph. The infiltrated water is not used in any further computations. A logarithmic decay assumption is made about the shape of the falling limb of the hydrograph which usually represents the delayed contributions of soil moisture/groundwater contributions to the hydrograph.

Another limitation of the infiltration simulation is that there is no provision for drying out of the soils (recovery of the infiltration rate) between storms; i.e., the loss rates remain constant or monotonically decrease. The Holtan loss rate function includes a percolation out of the bottom of the limiting soil layer but HEC-1 does not include the evapotranspiration term.

c. CHANNEL ROUTING. The channel routing is accomplished on a reach-to-reach basis without considering a direct continuous water surface profile. The storage routing methods, such as modified Puls, may utilize storage versus outflow relationships developed from continuous water surface profiles for steady flow. Such profiles do not consider the dynamic wave travel through the reach and may incorrectly estimate the storage in the reach by using a single-valued function in lieu of the actual looped rating curve. Such steady-state analysis will tend to underestimate the storage used on the rising limb of the flood hydrograph and overestimate the actual storage used on the falling limb of the hydrograph. These limitations can be partially overcome by using shorter routing reaches and more detailed calibration. Only the kinematic wave method uses water surface elevation (depth) directly in the routing computation. The other computations are carried out with discharges and storages. If a water surface elevation is desired at a particular river location, a rating curve must be input for that location and the program will look up the elevations from the provided rating curve. Likewise, a stage hydrograph may be input to the program but a rating curve must also be supplied for conversion to flows before the simulation can proceed. The routing should be performed by an unsteady-flow simulation if flood wave dynamics are important, such as in routing large, dynamic dam-breach flood waves.

Time-dependent tributary backwater effects cannot be accounted for in HEC-1. The generalized backwater effects of a channel network can be

accounted for in the steady-state HEC-2 water surface profiles but not on a dynamic basis at river confluences. A large mainstream or tributary flow may impede the flow in the other channel; this effect cannot be simulated in HEC-1. Again, an unsteady flow technique would be required or possibly a three-point rating curve to relate the flow in one channel to that in another channel.

d. RESERVOIR ROUTING. The routing through reservoir outlets and spillway for dam safety studies assumes a horizontal water surface throughout the reservoir pool. The horizontal water surface is also assumed during a dam breach simulation. These limitations can only be overcome by using a dynamic routing model such as the "Gradually Varied Unsteady Flow Profiles" program described in Section VI. Also the reservoir outflow cannot be a function of downstream flow conditions. Reservoirs are uncontrolled and simply follow a storage (elevation) versus outflow relationship. The HEC-5 model should be used to operate reservoirs for downstream flow conditions.

e. RIVER NETWORK STRUCTURE. Stream systems are treated as converging tree networks. No explicit declaration of the subbasin and routing reach connectivity is made. This requirement may limit somewhat the flexibility in building the data model. Its advantage is in reducing the need to store all hydrographs in the computer. Only the hydrographs needed for subsequent computations are stored.

f. OPTIMIZATION ALGORITHM. The optimization algorithm is a univariate gradient search (Beard, 1966) procedure which does not guarantee arrival at a global optimum; the procedure may stop at a local optimum. The user must start the optimization process with reasonable first approximations of the best values in order to insure that the procedure does not wander off into a local optimum.

3. *Data Requirements*

Precipitation-runoff simulation with HEC-1 requires data describing precipitation, interception/infiltration, surface runoff, baseflow, and channel routing. The specific data requirements are described in detail in the users manual. The general structure of the input data is a series of input blocks which proceed from upstream to downstream. The input blocks specify parameters that control subbasin runoff, routing, combining, diversions, etc. as shown in Fig. 10. Metric or English units may be used.

The input data may be provided in a free or fixed format. In the free format, the input is separated by a comma or blank. The fixed format is HEC's standardized ten fields of eight columns each. The first field is composed of a card/record identifier in columns 1 and 2 and the remaining six columns are available for data. The record identifier is a two-character code uniquely identifying the data on that record. To build the data model, the user simply inputs the card for the desired function. For example, all loss rate function cards begin with an "L" in column 1 and a second character in column 2 which is unique to a particular loss rate. The loss rate codes are as follows: LU, initial and uniform; LE, HEC exponential; LS, SCS curve number; and LH, the Holtan function.

4. *Application Procedure*

The development of an HEC-1 watershed model begins with subdividing the watershed into interconnected subbasin and routing reaches as previously shown in Fig. 4. There are two phases in this process: one for calibration of the model's parameters and the other for the project study. For the calibration process, the stream gage locations are taken to be the outlets of subbasins and beginning and ending of routing reaches. Using the gage data for individual elements, the parameters describing the runoff or routing process in that element may be identified for individual storm events. Several (5–10) individual storm events are usually desirable. The actual number of events which may be used will vary considerably. In ungaged areas, there are no recorded events. In other areas, long-record gages may be available so that there are many potentially good events. In those areas where long-record gages exist, one must be careful to select calibration periods in which the watershed response mechanisms are constant. Urbanization is usually a major cause of change of the watershed runoff response. An urbanizing watershed may require calibration at several stages of urbanization in order to determine a predictive relationship to explain the change in the model's parameters with urbanization.

a. NEED FOR A CALIBRATION PROCESS. A parameter optimization algorithm is available in the HEC-1 program for use in determining estimates of watershed runoff and routing parameters. The following discussion is summarized from an HEC technical paper (Ford *et al.*, 1980). That paper also described the use of optimization for streamflow forecasting and compares HEC's unit gradient method with other search methods.

If HEC-1 were a perfect model of watershed hydrology, and if total precipitation and total direct runoff could be measured accurately, the parameters for a storm event could be determined directly by the inverse solution of the transformation equations. However, these conditions are not satisfied in

reality, and inverse solution of the equations is difficult, so the parameters are found instead by selection of those values that yield the "best" reproduction of a measured runoff event with the available measured precipitation data and the available model. This parameter selection has often been accomplished by a systematic trial-and-error procedure: parameter values are selected, the model is executed with these values, and the resulting runoff hydrograph is compared with the observed hydrograph. If the "fit" is less than satisfactory, different parameter values are adjusted and the entire process is repeated.

An alternative to the trial-and-error approach to parameter selection is an automatic calibration approach in which the necessary tasks for calibration are automated. Automatic calibration requires selection of an explicit index of the acceptability of alternative parameter estimates, definition of the range of feasible values of the parameters, and development of some technique for correction of the parameter estimates until the "best" estimates are determined. Thus, the parameter estimation problem can be classified as an optimization problem: there is an objective function for which an optimal value is sought, subject to certain constraints on the decision variables (the parameters). The HEC-1 program includes the capability to solve this optimization problem, thereby automatically determining optimal estimates of the parameters.

The objective function of the parameter estimation optimization problem must define the difference between the computed runoff hydrograph (with any parameter estimates) and the recorded runoff hydrograph. Presumably, this difference will be at a minimum for the optimal parameter estimates. In HEC-1, the following objective function is employed as an index of the errors:

$$\mathrm{STDER} = \sum_{i=1}^{N} [(\mathrm{QOBS}_i - \mathrm{QCOMP}_i)^2 \times \mathrm{WT}]/N \qquad (7)$$

where STDER is the error index; QOBS_i is the observed runoff hydrograph ordinate for time period i; QCOMP_i is the runoff hydrograph ordinate for time period i computed by HEC-1 with the current parameter estimates; N is total number of hydrograph ordinates; and WT is a weight for the hydrograph ordinate. The weight WT is defined as follows:

$$\mathrm{WT} = (\mathrm{QOBS} + \mathrm{QAVE})/(2 \times \mathrm{QAVE}) \qquad (8)$$

where QAVE is the average computed discharge. This weighting function emphasizes accurate reproduction of peak flows rather than low flows by biasing the objective function. Any errors for discharge ordinates that exceed the average discharge will be weighted more heavily, and hence the optimization scheme should focus on the reduction of these errors.

The range of feasible values of the parameters is bounded because of physical limitations on the values that the various unit hydrograph, loss rate, and snowmelt parameters may have, and also because of numerical limitations imposed by the mathematical functions employed to model watershed behavior. In addition to bounds on the maximum and minimum values of certain parameters, the interaction of some parameters is also restricted because of physical or numerical limitations.

b. HEC OPTIMIZATION TECHNIQUE. The constrained optimization scheme employed in HEC-1 is a univariate search technique that uses Newton's method. Application of such a technique permits use of the simulation capabilities of HEC-1 in a traditional manner and does not require development of analytical derivatives. Steps in application of this technique, as implemented in HEC-1, are as follows:

(1) Initial values are assigned for all parameters. These values may be assigned by the program user, or program-assigned default values may be used.

(2) The response of the watershed is simulated with the initial parameter estimates, and the value of the objective function is computed by comparison of the ordinates of the computed and observed runoff hydrographs using Eq. (7).

(3) The first parameter to be estimated is decreased by 1% and then by 2%, the system response is evaluated, and the objective function calculated for each change. This gives three separate system evaluations at equally spaced values of the parameter with all other parameters held constant. The "best" value of the parameter is then estimated using Newton's method.

(4) Step 3 is repeated for each parameter, using the "best" estimate of the previous parameters as fixed values.

(5) All parameters are cycled through four times.

(6) Step 3 is repeated for the parameter that most improved the value of the objective function in its last change. This process is repeated until no single change in any parameter yields a reduction of the objective function of more than 1%.

The scheme employed for estimating the "best" value of each parameter in Step 3 is based on the concept that the optimum of the objective function occurs at the value of the parameter which caused the second derivative to go to zero. These derivatives cannot easily be evaluated analytically because the objective function indirectly includes all the functions and equations

contained in the HEC-1 watershed response. Therefore, numerical approximations of the derivatives are used.

c. APPLICATION OF THE CALIBRATION TECHNIQUE. Due to the varying quantity and form of data available for precipitation-runoff analysis, the exact sequence of steps in application of the automatic calibration capability of HEC-1 varies from study to study. An often-used strategy employs the following steps when using the Clark unit graph and the HEC exponential loss rate equation:

(1) For each storm selected for use in calibration determine the baseflow and recession parameters that are event dependent and are not included in the set of parameters that can be estimated automatically. These parameters are the recession flow for antecedent runoff (Q_0), the discharge at which recession flow begins (Q_k), and the recession coefficient (RR), as were illustrated in Fig. 9. The HEC-1 users manual suggests techniques for estimating these parameters.

(2) For each storm on each gaged subbasin, determine the optimal estimates of all unknown unit hydrograph and loss rate parameters using the automatic calibration feature of HEC-1.

(3) If loss rate precipitation rate exponent E [Eq. (2)] is to be estimated, select a regional value based on analysis of the results of Step 2 for all storms and subbasins.

(4) Using the optimization scheme, estimate the unknown parameters with E now fixed at the selected value. Select an appropriate regional value for the slope of the loss rate function R (Fig. 6). If the temporal and spatial distribution of precipitation is not well defined, an initial loss followed by a uniform loss rate may be appropriate. In this case, $E = 0$ and $R = 1$. If these values are used, as they often are in studies accomplished at HEC, Steps 2, 3, and 4 are omitted.

(5) With E and R fixed, estimate the remaining unknown parameters using the optimization scheme. Select a value of S for each storm being used for calibration. If parameter values for adjacent basins have been determined, check the selected value for regional consistency.

(6) With E, R, and S fixed, use the parameter estimation algorithm to compute all remaining unknown parameters. The D can be generalized and fixed if desired at this point, although this parameter is considered to be relatively event dependent.

(7) Using the calibration capability of HEC-1, determine values of the Clark unit graph parameters. Select appropriate values for each gaged subbasin.

(8) Once all parameters have been selected, the values should be verified by simulating the response of the gaged basins to other events for which precipitation and runoff records are available.

The HEC-1 optimization scheme has been used by the HEC in numerous studies for nearly 10 years. The applications have focused on developing frequency curves for ungaged locations and on modeling the impact of basin modifications, of channel improvements, or of additional control measures at selected locations.

Many of the recent applications of HEC-1 accomplished at the Hydrologic Engineering Center have employed the automatic calibration scheme in the development of data for ungaged areas. Typically, in these studies data are available from stream and precipitation measurement stations in the proximity of a location for which detailed stage and discharge data are unavailable but are desired. The automatic calibration technique is used to estimate unit hydrograph, loss rate, and routing parameters for the gaged locations, and this data is "transferred" to the ungaged locations using regression techniques. The particular strategy for estimating parameters and the methods for transferring the parameters to ungaged locations is a function of the basin characteristics, the available data, the parameters that are found via calibration, and the time and money available for the study.

While the sequence of steps for estimation of all parameters of this rainfall-runoff model has been employed in at least one major study, the flexibility gained by use of the four parameters in the exponential loss rate equation is not always necessary. Often E is set equal to zero, R is set equal to one, and calibration proceeds with Step 5 of this sequence. This approach has been employed in several studies, e.g., the Lehigh River (HEC, 1978d).

The optimization methodology may be used for the SCS curve number and lag or Holtan's infiltration function parameters. Likewise the methodology may be used for the routing function parameters. The same philosophy is followed for any function with two or more parameters: select the best value for the least variable parameters and optimize the remaining parameters. This process is carried out until only one parameter remains and its best estimate is obtained. That last parameter may be highly storm dependent as in the case of the loss rate function, where it represents antecedent moisture conditions.

d. TECHNIQUES FOR UNGAGED AREAS. The foregoing calibration process is used to determine watershed model parameters at gage locations. Rarely will gages be located at all points in a watershed where hydrologic information is required. In order to simulate hydrographs with the desired definition and at proposed project locations, the watershed must be subdivided into

additional subbasins and routing reaches. The hydrologic engineer is then faced with the problem of determining parameters for those ungaged areas. "Hydrologic Analysis of Ungaged Watersheds with HEC-1" (HEC, 1980b) describes the ungaged area modeling process. As stated in the preface of that report: "General approaches to frequency analyses for ungaged watersheds, effects of extent of data availability on choice of approach, and regionalization of hydrologic parameters are discussed is this report." Hypothetical rainfall data are often the only rainfall data available for a study, and therefore, methods for use of this type of data are presented. The use of HEC-1 for watershed modeling is described and techniques for the estimation and calibration of HEC-1 model parameters are given in detail.

The method used for hydrologic analysis of an ungaged area depends upon many factors including the following: purpose of the study and the type of information required, data available, watershed characteristics, experience of the hydrologist, and available time and personnel resources. The types of information which may be required are peak flow and volume frequency and duration relationships, and/or a long-term runoff hydrograph. The following data might be available: precipitation, temperature and other meteorologic measurements, discharge hydrographs, high-water marks, and basin characteristics. Some of the following methods may be used: regional frequency analysis of observed streamflows, watershed modeling of individual storm events, continuous watershed simulation for the period of record of observed precipitation and streamflow data, and watershed modeling of design storms having various frequencies. The selection of the method of analysis is dependent primarily upon the experience of the analyst and secondarily upon the data available. The experience of the analyst is extremely important in the application of a model in an ungaged area. The analyst must be able to relate his experience with the model's parameters in other areas to the basin characteristics and subjective assessments of the ungaged watershed and its expected runoff response.

The HEC ungaged area modeling report (1980b) discusses several modeling approaches and the impact of data availability on each approach. Procedures for calibrating model parameters and the regionalization of those parameters are treated in detail. Unit hydrograph parameters are typically related to basin characteristics such as drainage area, slope, and percent imperviousness. Thomas and Benson (1970) describe many watershed characteristics and their relation to runoff. The HEC's report describes three studies which have varying amounts of data and use different methods of analysis. The first case addresses the problem when some discharge frequency data are available. The second case has streamflow and precipitation data but records are insufficient to estimate frequencies with much confidence. The third case represents an approach when no data are available for the watershed in question. Because of the importance of synthetic storm analysis in ungaged watershed analysis, one chapter and an appendix are devoted to that subject and their use in the hydrologic analysis.

e. WATERSHED MODEL CALIBRATION. Having determined the parameters of the subbasins and routing reaches through optimization and ungaged area analyses, the entire river basin model may now be formulated. After developing the watershed data model, further calibration should be undertaken with the same historical storms used in the individual component calibration. The historical storms runoff is simulated throughout the watershed and compared with gaged flows. Further adjustment to subbasin and routing reach parameters are made as necessary to best reproduce observed flows. These adjustments cannot be made, however, without maintaining a relative consistency between parameters in similar geographic areas. Storm events which are critical to the water control projects under study should be emphasized in this process.

C. Use of HEC-1 in the Evaluation of Flood Control Measures

1. *Simulation of a Depth–Area–Duration Relationship for a River System*

Flood control studies require the determination of discharge frequency curves at several points of interest in a river network. If a watershed model and the design storm approach are used to estimate that frequency relationship, then it is necessary to make several runoff simulations for a storm of a given frequency at different locations in the basin. This is because a storm which produces the $X\%$ flood at one location in a river basin will not usually produce an $X\%$ flood at other locations in the river basin.

Design storms are computed on the basis of precipitation observed over a given area. For a given frequency and duration (e.g., 10%, 1 day), a precipitation depth–area relationship may be computed from observed precipitation in a region. Such relationships are computed in the U.S.A. by the National Weather Service (1961 and 1977). Simulation of the runoff from such a storm requires a separate application for each area. Thus, many executions of the model may be required in order to compute the flood of a specified frequency and duration throughout a large river basin.

The HEC-1 has a depth–area precipitation–runoff simulation mode in which the depth–area relationship is preserved throughout a river basin in a single simulation. This capability eliminates the need to make a separate simulation for each flood frequency computation point. The HEC-1 accomplishes this by simultaneously simulating the runoff from several storms con-

sistent with the given depth–area relationship. The computed hydrographs, termed "index" hydrographs, are routed and combined throughout the river network. This procedure is described in detail in the HEC-1 users manual (1980a) and summarized here.

Figure 11 shows a schematic representation of the HEC-1 depth–area storm runoff simulation methodology. A separate storm is simulated for each point on the given depth-area relationship as if that precipitation had occurred over the subbasin in question. The resulting runoff hydrograph for a subbasin is computed by interpolation using a linear discharge, log drainage

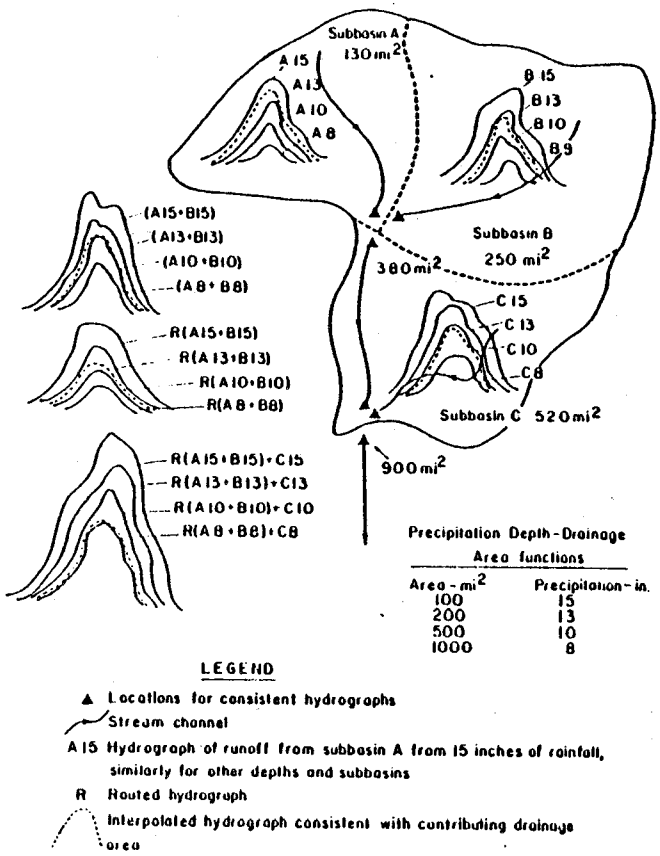

Fig. 11 Consistent storm runoff simulation per precipitation depth area relationship.

area function. At a confluence (e.g., sum of subbasins A and B in Fig. 11), the index hydrographs are added together and the hydrograph is interpolated using the sum of the drainage areas of A and B. The index hydrographs are routed to the next confluence downstream, where a similar combining of index hydrographs takes place.

A potential problem with this analysis is that the watershed model may produce a smaller volume of runoff for the large-area storm than it does for the small-area storm as the contributing area increases. This may occur through the action of nonlinear loss-rates on the lower-intensity large-area storm. Because a monotonically increasing volume of runoff is required as the contributing area increases, HEC-1 has included a method with which to preserve the increase in precipitation excess for each successive larger-area storm. That methodology, described in the HEC-1 users manual (1980a), guarantees that the runoff volume increases with increased index area.

2. *Simulation of Multiple Storms or Floods for a River Network*

Flood control analysis usually requires the investigation of several storms and/or floods of varying magnitude. The objective of such an analysis is to evaluate the impact of runoff for different-sized storms on proposed flood control projects. Any model can accomplish this by simply changing the precipitation input data and rerunning the job. Such a procedure can require substantial data manipulation and computer execution time.

The HEC-1 model can automatically and simultaneously simulate the runoff from several ratios of the same storm. The user simply inputs the base storm and the ratios desired for investigation. Every subbasin runoff calculation is made for each ratio and each resultant flood runoff is routed through the river basin network. The complete hydrograph for any ratio is available at any location in the basin. An output summary shows the peak discharge for each hydrograph computation point for each ratio.

If the flood investigation does not involve a change in watershed rainfall runoff response characteristics, then it is expedient to take ratios of runoff rather than precipitation. This would be the case for a reservoir or channel modification flood control project. In the ratio-of-runoff mode of operation, HEC-1 first computes the runoff from precipitation on a subbasin and then computes the required ratios. The subbasin runoff may also be directly input if it has been precomputed or is from some gaged event.

3. *Simulation of Alternative Flood Control Plans*

A flood control investigation must analyze several different projects to determine the best control strategy. That analysis can involve both different

projects and different-sized projects at the same location. There will also be projects at different locations, such as an upstream reservoir and/or a downstream levee. As in the case of the multiple ratios, multiple project analysis could be accomplished by several executions of the data deck with and without the projects. Such a method is time consuming, requires additional computer time, and does not facilitate comparative analysis of the projects.

The HEC-1 multiple plan mode of operation allows the analyst to compute the hydrologic impact of several flood control projects in a single computer run. In this usage a "plan" refers to a single flood control scheme which may involve several projects. The first plan in such an analysis is always the existing conditions. Each successive flood control plan may then be compared with each other and with existing conditions. The simplest case would be a single project such as a reservoir. "Plan 1" would be the runoff simulation for the existing river network, and "Plan 2" would be the same simulation except that the reservoir would be included. All flows downstream of the reservoir would be affected by the reservoir in "Plan 2."

The multiple-ratio option is usually used in conjunction with the multiple-plan option. Thus, the impact of different flood control projects on different-sized flood events can be investigated simultaneously. Such a combination of ratios and plans is especially convenient in computing expected annual flood damages as discussed in the next section. Figure 12 depicts the multiple-ratio plan capability for a simple system. It is important to emphasize again the computational efficiency of this mode of analysis and its utility in the investigation of alternatives.

4. *HEC-1 Computation of Expected Annual Flood Damage*

Economic efficiency is one of the main measures of the value of a flood control project. The benefits of the project are measured in terms of a reduction in flood damages. In order to evaluate flood damages over the life of a project, the concept of expected annual flood damage is used. The expected annual damage measure results from integrating the probability distribution of future streamflow (frequency curve) knowing the damage to occur at each magnitude of streamflow (or water surface elevation). The HEC-1 model has the capability to compute expected annual flood damage for any point of interest in a river basin. This capability is valuable during project formulation for comparing alternative flood control plans. The process includes simulation of basin hydrology, input of the flow–frequency relationship for existing conditions, frequency curve adjustment to post project conditions, input of flow/stage–damage relationships for existing and postproject conditions, and integration of the resulting damage–frequency relationships for existing and postproject conditions.

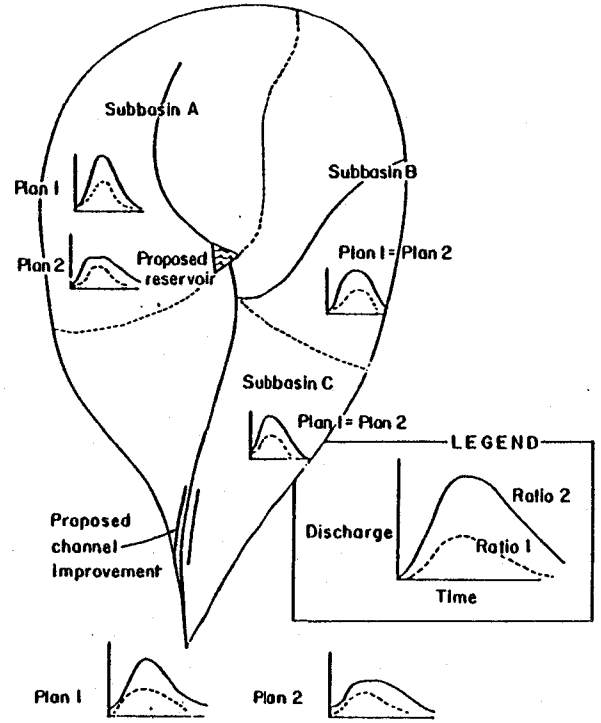

FIG. 12. Multiple floods and flood control projects analysis.

a. SIMULATION OF POSTPROJECT FLOW FREQUENCY CURVE. The computation of flow frequencies makes use of the "multiple-ratio" and "multiple-plan" features of HEC-1 discussed in the previous two sections. The HEC-1 must be provided with a flow– or stage–frequency relationship for existing conditions at all locations of interest. The analyst selects a major flood event and several ratios of which to span the range of the flood frequency curve. Additional criteria for selection of the ratios will be given in the next section.

The existing (Plan 1) river basin is simulated and the frequency of the peak discharge/stage for each ratio is computed from the given flow–frequency curve as shown in Fig. 13. *The method assumes the frequency of the storm/flood ratio will remain the same under postproject conditions* and only the flow and possibly the stage–damage relationship will change. Thus, the peak flow under postproject conditions for a ratio will have the same

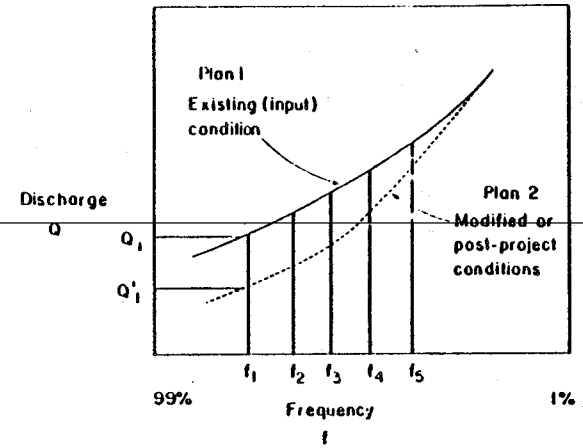

FIG. 13. Computation of modified flood frequency curve. Q_i is the existing condition computed peak flow for RATIO $i = 1, n$; f_i, frequency of Q_i from input flow frequency curve; Q'_i, computed postproject (Plan 2) peak flow for RATIO $i = 1, n$.

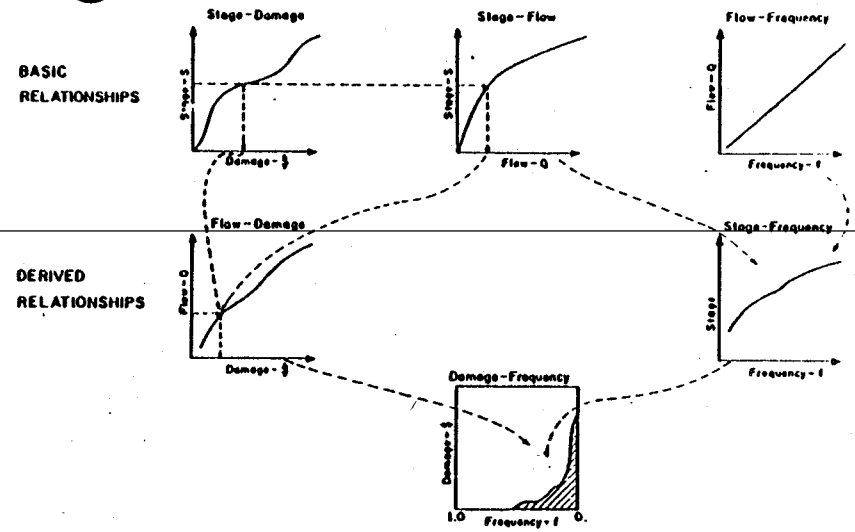

FIG. 14. Expected annual flood damage computation.

frequency as the preproject peak flow for the same ratio. By using several ratios as shown in Fig. 13, the postproject modified frequency curve is computed.

b. COMPUTATION OF EXPECTED ANNUAL FLOOD DAMAGE. Expected annual flood damage is computed for each "plan" and benefits are determined as the reduction in flood damage from pre- to postproject conditions. Damage reaches are established for all potential flood damage locations in the river basin. A damage reach is defined as a length of river channel which is under essentially the same hydraulic control for the range of the frequency curve, at least that range of flows which contributes the most to expected annual damages; i.e., the damage reach is the longest length of river which can adequately be represented by discharge and stage at a representative "index" location on the river. Several damage reaches may be required to calculate damages in a large metropolitan area.

A flow- or stage-frequency and a flow- or stage-damage function is required for each damage reach. A damage-frequency relationship is then derived combining these two functions using the peak flow for each flood ratio as shown in Fig. 14. Then the damage-frequency function is integrated using a Gaussian quadrature procedure. The magnitude of the "ratio" peak flows is very important in defining these functions. Thus, another criteria for selection of the flood ratios is that the expected annual damage integrated between ratios be about equal; i.e., it is not desirable to have a majority of the expected annual damage contributed by one or two ratios.

The economic impact of a flood control project is presented through the modified flow- or stage-frequency function or in the flow- or stage-damage function. Reservoirs and channel improvement projects will alter the flow-frequency functions whereas a levee may only affect the stage-damage function. A discussion of the relative impacts of flood control and flood damage reduction projects on these various functions is given in Exhibit 2 of the "Expected Annual Flood Damage Computation" (EAD) users manual (HEC, 1979b). The HEC-1 flood damage analysis does not take into account the time value of money. The HEC-1 results would typically be input to the EAD program, which computes the present value of future costs and benefits of flood control projects.

There are some limitations to this method of computing expected annual flood damage, but they have generally been far outweighed by the computational efficiency and utility of the method. Also, considering the accuracy of the basic data (flood frequencies and damage over a river reach) the method has been proven very acceptable. A truer picture of the hydrologic impact of proposed flood control projects on the frequency curve could probably be obtained using a continuous watershed simulation model. That

model could evaluate the impact on every significant flood rather than a few selected "ratios." The use of continuous watershed simulation models to determine a flow-frequency relationship is not without its own drawbacks. The primary drawback is that of not being able to determine the spatial and temporal distribution of precipitation over long periods of time. As one goes back in time, the number of gages becomes smaller and they have less consistent operation. Thus, a major effort is required to fill in (simulate) missing data.

The HEC-1 "multiple-ratio" analysis assumes the same time pattern of precipitation is applicable to all ratios, only the magnitude of the storm changes. The depth-duration relationship for the base case (from which ratios are taken) must be representative of storms at several magnitudes. The relative areal distribution of the storm is also considered to be the same for each "ratio." The runoff from each part of the basin is assumed to be representative of that portion of the basin. The same is true of the loss rate distribution over the basin. Detailed output information concerning the hydrologic and economic computations and summary results is printed by HEC-1.

5. *Sizing Flood Control System Components*

The HEC-1 has a very powerful feature to automatically determine the "best" size of each of several components (levee, reservoir, pump, diversion, etc.) of flood control systems (Davis, 1974). This feature makes use of the expected annual damage computation algorithm just discussed together with the univariate gradient optimization capability used for the watershed model parameter estimation. That same optimization algorithm is used to search for the "best" combination of sizes of flood control components. The flood control component size optimization process is described in HEC Training Document No. 9 (1978a).

Cost versus size data are required for each component to be included in the optimization. The objective function to be optimized (minimized) is the sum of the expected annual flood damages and the project cost. Costs include both the initial capital investment and the annual operation and maintenance costs. The objective functions may also be weighted by how well desired discharge targets are met at various stream stations. The univariate gradient search procedure determines the best size of one component while holding all others at a constant size. The method iterates through all components several times, searching for the best combination of sizes. The sizes of the projects may be constrained to specific ranges.

In the simple system shown in Fig. 15, the optimization process would automatically investigate the many trade-offs between a larger reservoir and

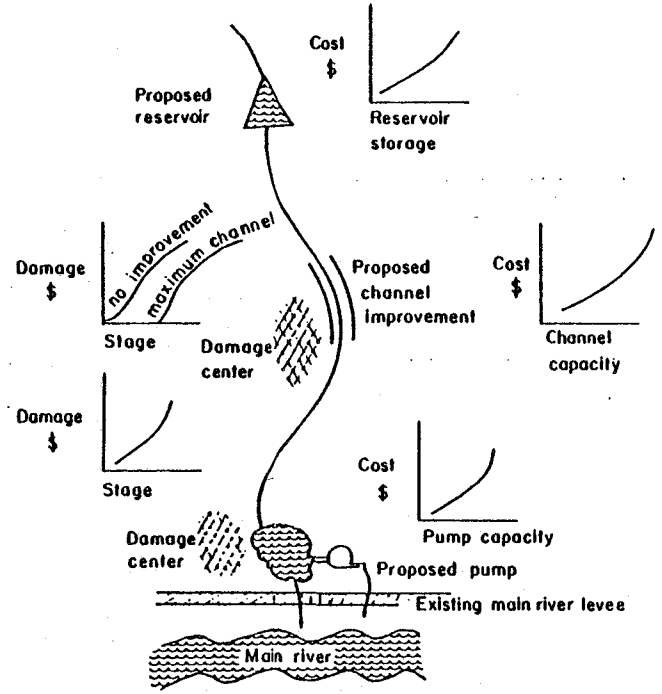

FIG. 15. Optimization of flood control project sizes.

smaller downstream projects, and vice versa. Both of the two damage centers can be helped by the reservoir project. The upstream damage center can also be helped by the levee project. The levee project is represented by both a cost function and a modification in the stage-damage function for that reach. HEC-1 uses upper and lower stage-damage functions to represent the range of possible economic impacts by a channel improvement. An interpolation scheme is used to determine damages for intermediate channel sizes. The downstream damage center can be protected by a pump system which evacuates the water ponding behind the existing main river levee.

Output information is provided during each step of the optimization process. This information is especially important because the method may stop at a local optimum. The results should be checked by starting the optimization process at different initial component sizes.

D. Computer Requirements

HEC-1 is written in FORTRAN IV and requires 115,000 words of core storage on the CDC 7600 computer. Input may be in free or fixed format. The program code is machine independent except in limited areas where special instructions are provided for implementation on some computers.

IV. The HEC-2 Water Surface Profile Program

A. Purpose

The original version of the present HEC-2 program was developed by Bill S. Eichert while employed in the Tulsa, Oklahoma District Office of the U.S. Army Corps of Engineers. Eichert is presently the director of HEC, where he and others continue to improve and expand the capabilities of HEC-2. HEC-2 (HEC, 1979d) is now considered to be the most widely used program in the world for computing water surface profiles.

The HEC-2 was developed to determine water surface elevations for specified discharges in natural channels to aid the Corps floodplain management program. The program computes water surface profiles for given discharges in river channels of any cross section for subcritical or supercritical steady-state flow. The effects of natural obstructions to flow, floodplain encroachment, and hydraulic structures may be simulated by the program. Bridges are given special consideration for their impact on the flow hydraulics. Culverts, weirs, channel improvements, embankments, and levees may also be considered in the flow/profile computation.

The principal use of the HEC-2 program has been in determining inundated areas associated with various flood discharges. The simulated area and depth information is used by the Corps to evaluate flood damages and by federal and local agencies to aid in the management of the nation's floodplains. Although the HEC-2 analysis does not consider the dynamics of the flood hydrograph propagation, the methodology is well founded in hydraulic theory for steady flow conditions, efficient to use, and commensurate with the technical reliability of the other aspects of the studies such as flow-frequency and stage-damage relationships. In cases where floodplain geometry, hydraulic structures, or flood wave shape produce significant rates of change in the water surface, a dynamic simulation should be made using an unsteady-flow model.

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o EXAMPLE OF MODEL APPLICATION

The following pages are excerpted from Hydrologic Analysis of Ungaged Watersheds with HEC-1, Training Document 15, Hydrologic Engineering Center, 1981.

CHAPTER 9

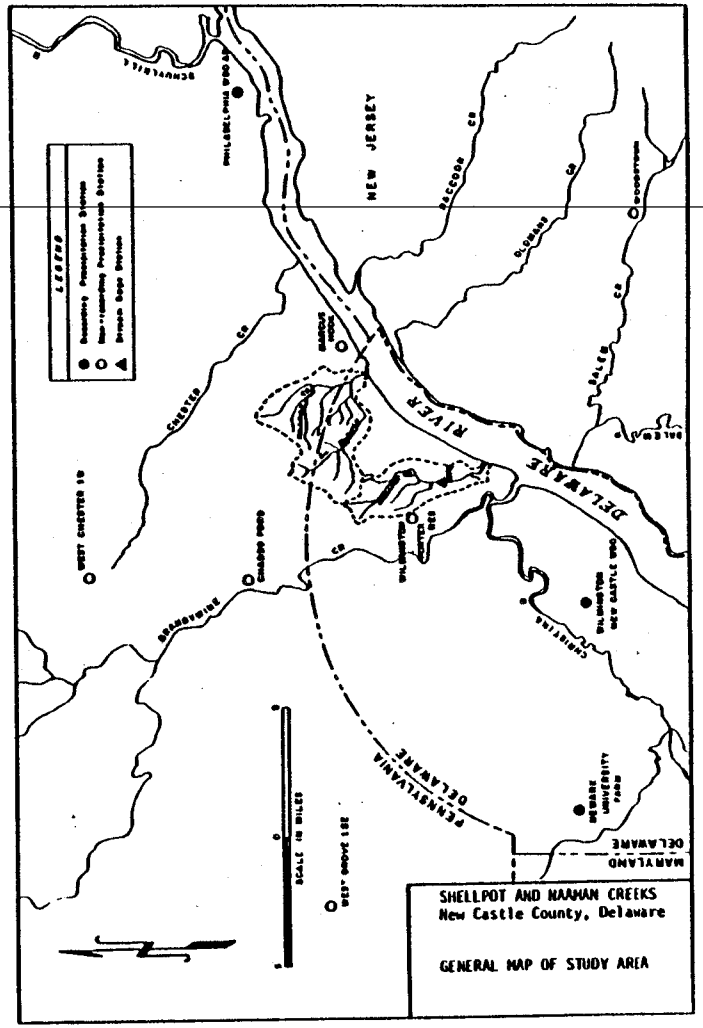
STUDY II - ANALYSIS WITH SOME STREAMFLOW DATA AVAILABLE

Some data (such as flow data from a single stream gage) are available for the watershed in many cases. Usually, however, the gage is not at the location at which the discharge-frequency relationship is needed. An example of such a situation is Shellpot Creek, a 9.43-sq mi basin near Wilmington, Delaware. The study on which this example is based is described in a Special Projects Memorandum by the Hydrologic Engineering Center (1976b). The hydrological data were developed for use in Special Flood Hazard Information reports. Basic rainfall and runoff data were analyzed, and the HEC-1 model was calibrated on several observed events and applied to specific hypothetical events. The following paragraphs describe the procedures and assumptions which were used to develop recommended discharges at selected exceedance frequencies for this basin.

9.1 Basin Description

Shellpot Creek is located in the northeastern corner of the state of Delaware. Figure 9.1 is a general map of the study area. Shellpot Creek enters the Delaware River at Wilmington, Delaware. It drains an area of 9.43 square miles, all within New Castle County, Delaware. The basin is nearly completely urbanized, and the predominant type of land use is single-family residential, with industrial and commercial development concentrated primarily in the lower portions of the basin near the Delaware River.

Channel slopes vary from less than 25 ft per mi in the lower reaches of the stream to over 200 ft per mi in the steeper slopes near the watershed boundary. The main channel slope of Shellpot Creek (between points 10 and 85 percent of the distance along the longest watercourse) is 40 ft per mi. Land surface slopes average about 3 to 5 percent, and soils are predominantly of the moderate-infiltration soils group. Mean annual precipitation is about 43 in, and average annual runoff for Shellpot Creek during the 29-year period 1946-74 was 17.5 in. Streamflow data are available at one location on Shellpot Creek (at Wilmington, Delaware). The drainage area at this point is 7.46 sq mi. Continuous records are available from December 1945 to the present.



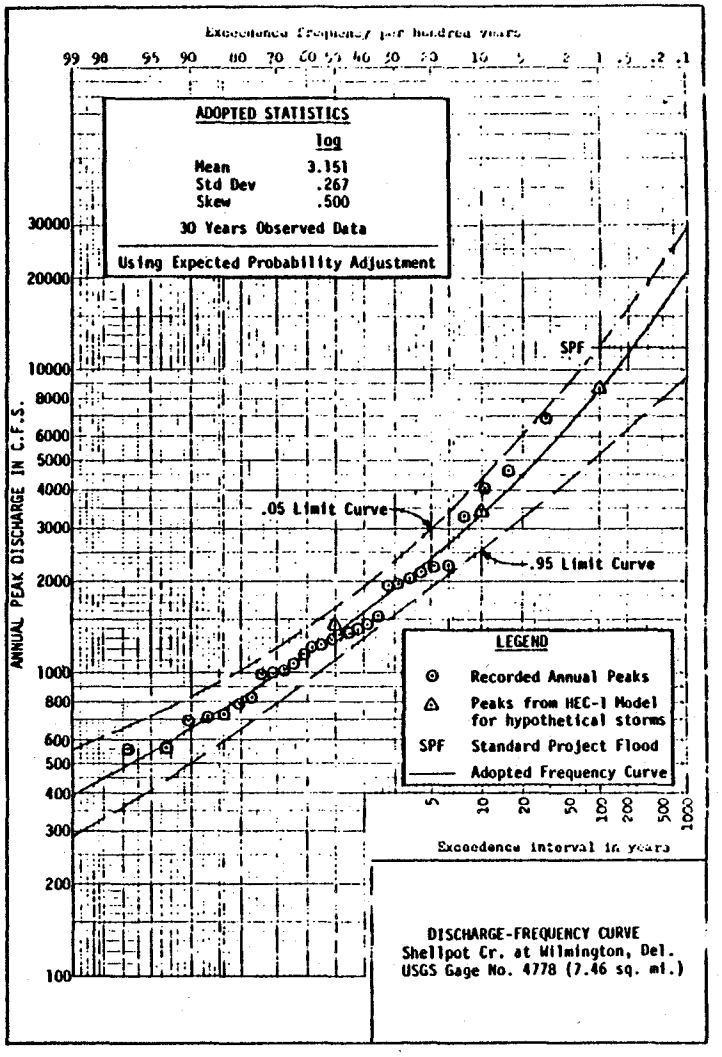
9.1. Shellpot Creek Study Area

9.2 Observed Frequency (Observed Data)

A discharge-frequency analysis was made on the 30-year period 1946-75 based on annual maxima. Water Resources Council Guidelines procedures (WRC, 1976) were applied using a weighted skew value of 0.5. Figure 9.2 shows a plot of these data based on the Weibull plotting position equation. The computed mean, based on the log transformation of the data, is 3.151 (or 1,400 cfs), and the standard deviation is 0.267. Nearby basins having streamflow records with longer periods of record are almost nonexistent. Chester Creek, about 10 miles northeast, has continuous records from 1932, but has a much larger drainage area (61.1 sq mi) and very little urbanization. Correlation of annual peak discharges for Shellpot and Chester Creeks resulted in a correlation coefficient of 0.428. If the Water Resources Council Guidelines are followed, the correlation coefficient would need to be equal to or greater than 0.93 in order to justify adjustment of the gaged annual-series discharge statistics of Shellpot Creek.

The only other nearby station with a longer term of record is Brandywine Creek at Chadds Ford, Pennsylvania, with records from 1911 to 1953 and from 1962 to the present. The area associated with this station is 287 square miles, and there is essentially no urbanization. Because of the great difference in basin size, no correlation studies were made. The curve in Figure 9.2 is considered to be the best estimate of the discharge-frequency characteristics for Shellpot Creek and the best guide for developing reasonable estimates at other locations within the basin.

The six highest recorded events at the Shellpot Creek gage have peak discharges that are all greater than the 5-year event (20 percent chance of annual exceedance). Data for these events are given in Table 9.1. Because of the small size and nearly complete urbanization of this basin, it is reasonable to expect less variability in initial rainfall loss by infiltration at various times of the year; therefore, no additional search of rainfall records was made to find storms that were more severe than those resulting in these peak discharges.



9.2. Discharge-Frequency Curve for Shellpot Creek

TABLE 9.1

DATA FOR HISTORICAL EVENTS

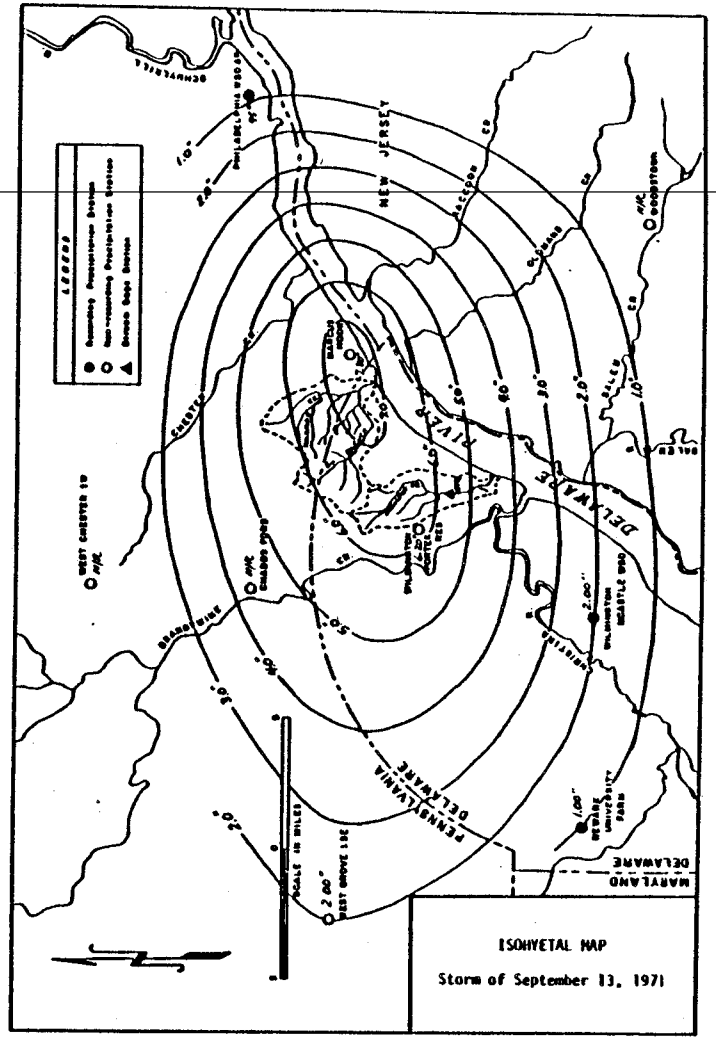
Date of Peak	Peak (cfs)	AVERAGE FLOW 1 Day (cfs)	3 Day (cfs)	TP[1/] (hrs)	RR[2/] (ft/hr)
13 Sep 1971	6850	1300	536	2	4.5
27 Aug 1967	4650	577	263	2	5.4
9 Jul 1952	4080	643	225	3.5	2.5
23 Aug 1974	3300	386	136	4.5	2.0
18 Aug 1955	2280	279	110	2	3.7
22 Jun 1972	2240	501	183	3	2.8

1/ Time from start of rise to peak discharge
2/ Maximum rate of rise

9.3 Unit Hydrograph Studies

Five of the six events listed above were analyzed in detail with HEC-1. Isohyetal maps were prepared for each storm analyzed. Recording and nonrecording gage locations are shown on a typical isohyetal map in Figure 9.3. A great deal of judgment was required in developing isohyetal patterns for each storm because there is a substantial variation in the total storm rainfall amounts observed at the various gages surrounding the basin. The distribution of storm rainfall in time is subject to considerable error when based on stations several basin widths away. Total basin-average precipitation was estimated for each storm event. The HEC-1 unit hydrograph and loss-rate optimization option was used to determine 15-minute unit hydrograph and loss rate parameters based on observed discharge hydrographs. Rainfall and runoff data for the five storms are presented in Table 9.2.

Unit hydrograph data and basin characteristics are presented in Table 9.3. For the full basin at the gage the average Snyder t_p is 2.1 hours, and C_p is 0.77. Figure 9.4 represents a typical reconstituted



9.3. Typical Isohyetal Map

TABLE 9.2

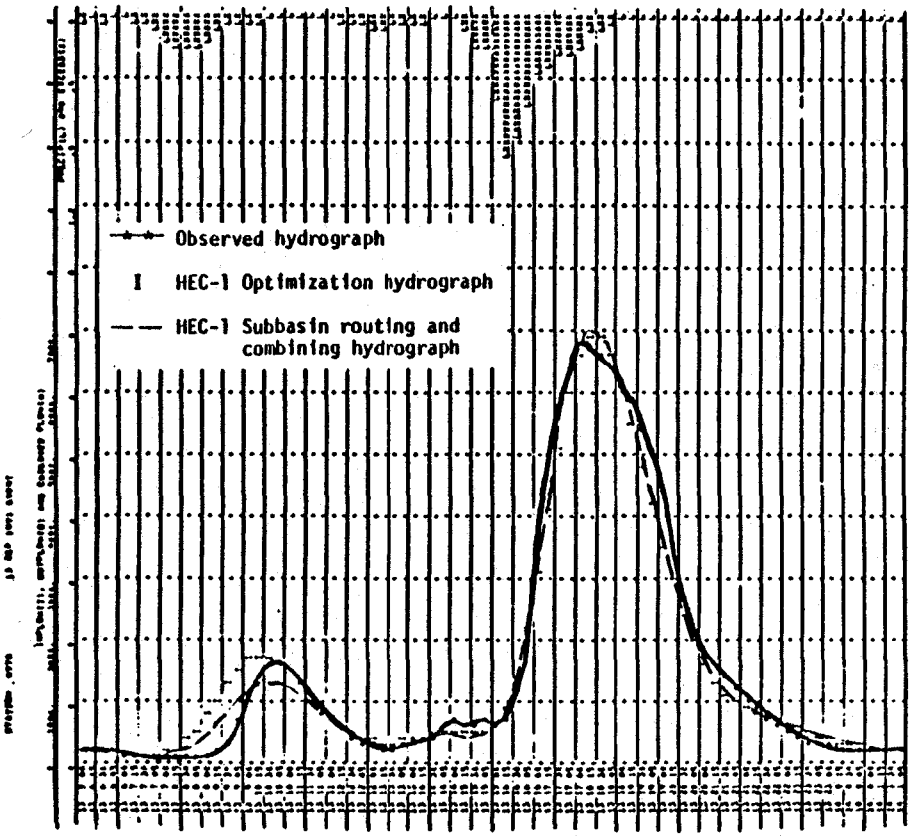
RAINFALL AND RUNOFF FOR STORM EVENTS

Event	Basin Average Rainfall and Runoff Shellpot Cr. at Wilmington, Del. (7.46 sq mi) Rainfall (inches)	Runoff (inches)
18-19 August 1955	2.44	1.52
27 August 1967	2.24	2.07
13 September 1971	6.36	6.24
22 June 1972	3.74	2.43
22-23 August 1974	3.85	1.87

TABLE 9.3

SHELLPOT CREEK BASIN
PHYSICAL CHARACTERISTICS AND
UNIT HYDROGRAPH DATA FOR SUBBASINS

Subbasin Number	DA (sq mi)	L (miles)	L_{ca} (miles)	Slope 10-85 (ft/mi)	Percent Impervious I	t_p (hours)
1	0.64	1.00	0.40	73	23	0.66
2	0.71	1.70	0.80	48	30	0.96
3	0.66	1.80	1.00	61	42	1.04
4	0.94	2.20	1.10	50	35	1.13
5	0.75	1.70	1.00	86	30	1.02
6	1.11	2.10	1.10	97	20	1.12
7	0.93	2.35	1.30	119	25	1.22
8	1.34	3.25	1.60	111	30	1.43
9	0.38	1.05	0.52	240	30	0.73
Gage	7.46	6.10	3.10	40	30	2.10
10	0.41	1.25	0.50	186	30	0.75
11	1.56	2.40	1.40	25	35	1.26



13 SEPTEMBER 1971
SHELLPOT CR. AT WILMINGTON, DEL.

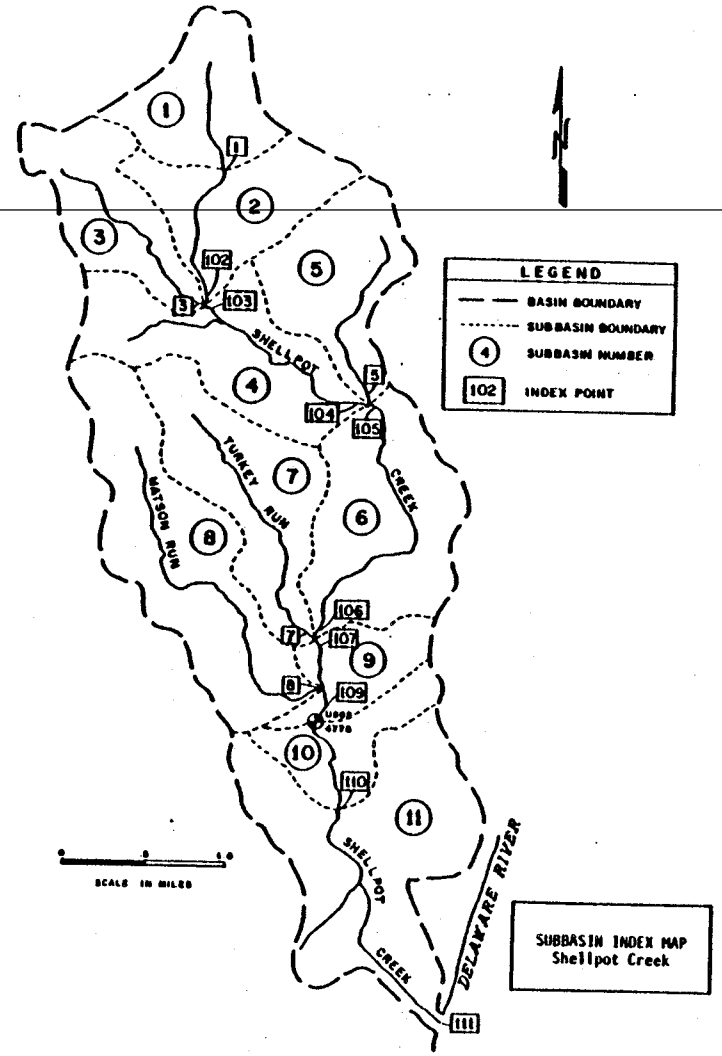
9.4. Simulated and Observed Hydrographs

hydrograph along with the observed hydrograph. Rainfall distribution was based on mass curve plots of available recorder data and an adjusted average curve. Fifteen-minute values were then interpolated from the curves. The rainfall exponent parameter ERAIN was set to zero and the loss rate parameter RTIOL was set equal to 1.0. This forced the program to optimize losses using essentially an initial loss plus a constant loss. The values of t_p and C_p were reasonably stable for the five events studied, but the loss parameters displayed more variability due to an inability to match antecedent soil moisture conditions and the sensitivity of the parameters to errors in estimating total rainfall and the rainfall distribution. The overall hydrograph reconstitutions are acceptable, and differences between observed and reconstituted hydrographs can be explained as random errors, resulting primarily from non-uniformity of rainfall distribution.

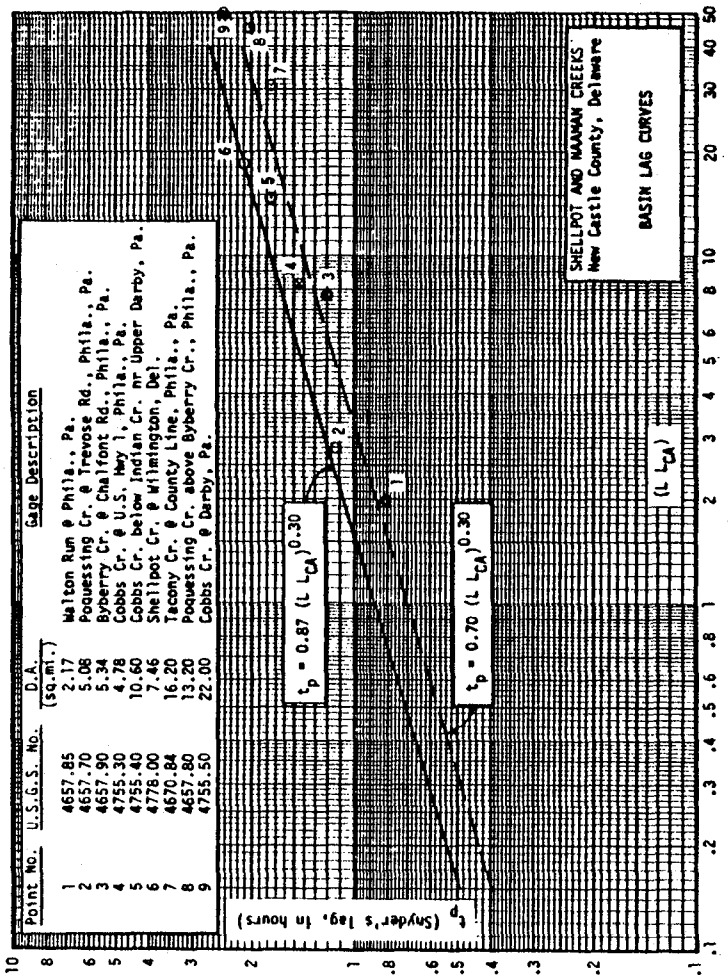
9.4 Basin Subdivision and Model Calibration

In order to determine probable flood discharges at other locations within Shellpot Creek, the basin was subdivided as shown in Figure 9.5. Index points were selected to coincide with major tributaries and spaced along the main watercourse in such a way as to result in only minor changes in discharges between mainstem index points. The percent imperviousness was estimated from areal photographs and published information.

To develop appropriate unit hydrographs for each sub-basin, a relationship between the basin physical characteristics of stream length (L) and length to center of basin (L_{ca}) versus unit hydrograph lag (t_p) was developed from a comparison of Shellpot Creek parameters with those for gaged urban basins in Philadelphia, Pennsylvania. Also given consideration was the t_p versus $L\,L_{ca}$ relation adopted in the Red Lion Creek Study described in Chapter 10. Initial estimates for t_p were based on a line drawn through the Shellpot Creek five-storm average value which was drawn parallel to lines describing relations adopted in other Delaware and southeastern Pennsylvania stream studies. This line (Fig. 9.6) has the equation $t_p = 0.87\,(L\,L_{ca})^{0.30}$. A Snyder peaking parameter C_p of 0.77 was adopted for all subbasins. It is the average value determined in the optimization studies discussed above. Starting discharge (STRTQ) varied from 0.13 to 30 cfs per sq mi for the various events and was based on gage data. For the final loss-rate



9.5. Basin Subdivision



9.6. Regional Parameters

parameters, the initial-loss volume ranged from 0.4 to 2.0 in. and constant loss-rates from 0.06 to 0.65 in. per hr for the five storm events. A constant ratio of the peak (18 percent of the peak discharge) was selected for the start of recession discharge (QRCSN), and a rate-of-recession parameter (RTIOR) equal to 3.0 was used for each subbasin based on a 15-minute time interval.

9.5 Channel Routing Criteria

Channel routing is required in the model to provide the correct translation of the flood hydrograph along the stream from index point to index point. Routing provides the timing and attenuation which reflect the storage characteristics of the channel and overbank sections of the stream reach. Since detailed stream cross-section and bridge data were unavailable, approximate routing methods were used. Channel reach lengths and slopes were estimated from USGS 1:24,000 scale maps that had a contour interval of 10 ft. A value of average velocity was then estimated for each channel reach based on Manning's equation and assuming steady flow with an average slope equal to the channel slope. The Muskingum routing method was used with a discharge weighting coefficient of X = 0.3. A reach travel time K in hours was estimated from the mean velocity. Adjustments were made to K as necessary to make the routed and combined hydrographs for the nine contributing subbasins agree with the observed runoff events at the stream gage.

9.6 Hypothetical Storm Runoff Estimates

Hypothetical storm data were developed by the procedures described in Chapter 4 and the Appendix of this report. Twenty-four-hour storms were developed for return periods of 2-years, 10-years, 100-years, and for the Standard Project Storm (SPS). The computed 24-hour point rainfall values were assumed to be applicable to areas of up to 1 sq mi. Adjustments were made to all durations (15-minute to 24-hour) for areas of 5, 10, and 15 sq mi, and the storms were distributed into successive 15-minute periods. These adjusted and distributed data were used in the stream-system option of the HEC-1 computer program to generate consistent hydrographs for each subbasin and combining point.

The model results at the gage were compared with the adopted discharge-frequency curve shown in Figure 9.2, and adjustments were made to loss rate parameters until reasonable agreement was obtained. However, even with zero loss, the 100-year and Standard Project Storms could not be made to give peak discharges as high as those given by the curve. It is generally accepted that unit hydrographs have a shorter t_p when intense storms occur over basins having confined flood plains. Therefore, it was rationalized that values of t_p for the 100-year and the Standard Project Flood (SPF) could reasonably be adjusted by a factor of 0.8. An adjustment factor of 0.8 was also applied to the channel-routing time-of-travel K values. This allowed reasonable loss-rate parameters to be adopted and resulted in values consistent with the statistical analysis of observed annual maximum discharges and SPF shown in Figure 9.2. Final loss parameters (STRTL and CNSTL) used for all subbasins were:

	2-Yr	10-Yr	100-Yr	SPF
Initial Loss (STRTL) in in.	2.0	1.5	0.5	0.5
Constant Rate (CNSTL) in in./hr	1.0	0.8	0.1	0.05

9.7 Regional Discharge-Frequency Comparisons

Regional discharge-frequency characteristics were determined using data from studies of other basins in the area (Hydrologic Engineering Center, 1974). The following correlation relationship for these regional data was developed:

$$\log Q_m = C_m + 0.87 \log (DA) \quad \ldots\ldots\ldots\ldots\ldots\ldots (9.1)$$

where

Q_m = geometric mean of the annual flood peaks in cfs,
C_m = "mapping coefficient" for a particular basin in the region, and
DA = drainage area in sq mi

A regression equation which contained the main channel slope in addition to DA produced a slightly better correlation (the regression coefficient R^2 was 0.897 when the slope was included, versus 0.871 for the relationship

expressed by equation 9.1) The equation with the slope was not used, however, because it was felt that the great deal of effort needed to determine S for all the ungaged streams was not justified for such a small increase in R^2.

The mapping coefficient C_m is the difference between the observed and computed mean logarithm of the flow. For the Shellpot Creek data the coefficient was relatively high (C_m = 2.392). This was attributed to the high percentage of urbanized area within the basin in comparison to the other basins in the region.

The equation developed in the regional studies for estimating S the standard deviation of the logarithms of annual maximum discharges is:

$$S = C_s - 0.05 \log (DA) \quad \ldots\ldots\ldots\ldots\ldots\ldots (9.2)$$

where C_s is a mapping coefficient for the standard deviation. The C_s value was determined to be 0.311. This value is consistent with data from adjacent basins.

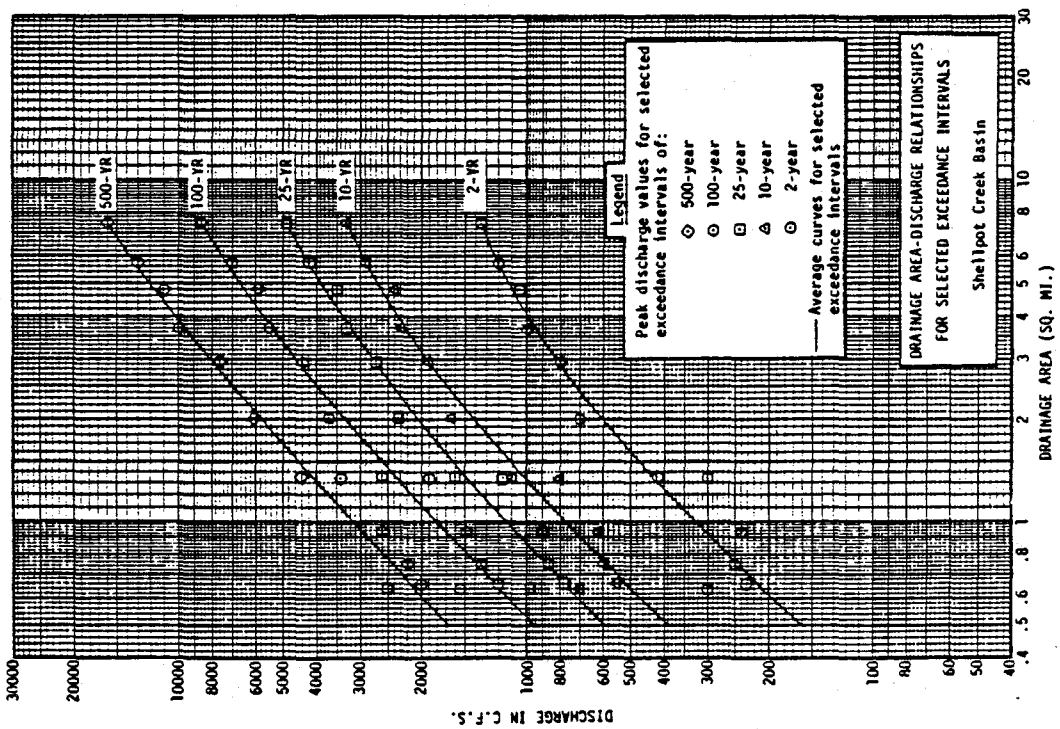
Computed values of log Q_m and S for each index point were used as input to a regional discharge-frequency program, using an assumed equivalent length of record of 20 years and a regional skew of 0.5. Results were plotted and used to evaluate results of the hypothetical-storm approach discussed above. Since the regression equation does not account for differences in land use, channel slopes, or basin shape, it was felt that the answers determined from the HEC-1 model were better. Consequently, the flows computed from the HEC-1 model runs were given more weight in the preparation of recommended curves. Discharges based on the recommended curves are given in Table 9.4 for all index points. The relatively wide scatter displayed on plots of peak discharges versus drainage area in Figure 9.7 are attributable to the sensitivity of peak discharges to differences in land use, basin shape, and channel slopes.

TABLE 9.4

RECOMMENDED PEAK DISCHARGES AT SELECTED INDEX POINTS
(SHELLPOT CREEK BASIN)

Index Point	Location Description	D.A. (sq mi)	Peak Discharges (in C.F.S.) for Selected Exceedance Intervals 2-Year	10-Year	25-Year	100-Year	SPF	500-Year
1	Subbasin No. 1	0.64	300	700	980	1530	1700	2500
102	Subbasins 1 and 2	1.35	420	1100	1600	2600	3000	4400
3	Subbasin No. 3	0.66	230	550	780	1200	1400	2000
103	Subbasins 1 thru 3	2.01	700	1650	2350	3700	4500	6100
104	Subbasins 1 thru 4	2.95	800	1900	2700	4400	5500	7600
5	Subbasin No. 5	0.75	250	600	860	1350	1600	2200
105	Subbasins 1 thru 5	3.70	1000	2300	3300	5500	6900	10000
106	Subbasins 1 thru 6	4.81	1050	2400	3500	5900	7900	11000
7	Subbasin No. 7 (Turkey Run)	0.93	240	620	900	1500	1800	2600
107	Subbasins 1 thru 7	5.74	1200	2900	4200	7000	9500	13000
8	Subbasin No. 8 (Matson Run)	1.34	300	800	1170	1900	2500	3400
109	Subbasins 1 thru 9 (USGS Gage) (Shellpot Cr. at Wilmington, Del.)	7.46	1350	3300	4900	8600	12000	16000

1/Refer to Figure 9.5 for subbasin and index point locations.



9.7. Drainage Area - Discharge Relationships

9.8 Conclusions

Discharges for primary index points on Shellpot Creek are summarized in Table 9.4 for selected exceedance intervals. Discharges at other exceedance intervals can be estimated from plots of the data. If discharge estimates are needed for additional index points along the stream, discharges can be determined by interpolation based on respective drainage areas and the nearest upstream and downstream index point. The 100-year peak discharge estimate for this basin appears high (approximately 1,200 cfs per sq mi) when compared with other streams in Delaware and southeastern Pennsylvania, but with 30 years of observed data on Shellpot Creek it is reasonable to use the discharge-frequency curve from these data as the best indication of what could occur on the urbanized subbasins.

o ASSESSMENT

Pitfalls in Application

The most common pitfall in application of HEC-1 is to apply the program as though it were a black box without understanding the theoretical bases and limitations of the program. Effective use of HEC-1 requires a clear understanding of the problem to be solved, and recognition of the extent and reliability of available data.

Possible Improvements

HEC-1, like any other well-used program, is undergoing continuous development. At the present time, work is underway to improve the program's capability to model base flow. Consideration is being given to provide capability to use a variable time step size, such that the program itself would determine the appropriate time step for each element of the computations, and would interpolate as necessary at the interface between elements. This capability would be especially valuable in situations where runoff from both small and large subbasins is being determined as part of the same analysis.

"FER" COMPUTER PROGRAM
FLOOD ESTIMATES AND ROUTINGS

US ARMY CORPS OF ENGINEERS
KANSAS CITY DISTRICT
CLARON L. KOONTZ
Hydrology and Hydraulic

Prepared for presentation at

WORKSHOP
ON
HYDROLOGIC MODELING
FOR
FLOOD HAZARDS PLANNING & MANAGEMENT
AUGUST 7-9, 1985

Sponsored by
DEPARTMENT OF CIVIL ENGINEERING
UNIVERSITY OF COLORADO
in cooperation with
FEDERAL EMERGENCY MANAGEMENT AGENCY
REGION VIII

Denver, Colorado

OUTLINE

FER Computer Program

A. Program development initiated by State of Nebraska's Natural Resources Commission, Lincoln, Nebraska (NNRC).

1. Needed a program to combine both HEC and TR20.

2. Desired to simplify data input into program.

3. Restructure data input and use more alphabetic control rather than numeric number.

B. Adopted HEC-1 program (Version DD) as base program.

1. Traced program logic and removed some routines.

a. Removed all hydrograph optimization.

b. Removed balanced hydrograph procedure.

c. Removed snow melt computation. (does not work in state of Nebraska.)

d. Removed storm system computations.

e. All other routines were preserved.

2. Restructured and segmented program into modular subroutines.

   a. Removed and change program control.

   b. Program easier to modify and insert new procedures.

C. Restructured input data.

   1. Reads in sequential modules of data.

   2. Uses control words to control execution.

   3. Allows more comments to be placed in data.

   4. Major control card words (two words 1st 8 digits of the first card (line) of each data module).

   5. Multiplan/multistorm procedures retained & simplified.

      a. "STM MULT" control card

      b. "RTE MULT" control card

D. Routines and Procedures inserted into the FER program.

   1. User can change computation time period during execution.

a. Can either increase or decrease time period.

b. Most subroutines automatically consider changes of computation time.

2. Printed output control.

a. Can be selective.

b. Can turn on or off at any time during computation.

c. Major control cards echoed in printout.

d. Prints out only necessary ordinates, truncates trailing zeros.

e. Can control number of ordinates to be used (max = 300).

f. Inserted Creager's formula for maximum peaks (can be used to check for possible errors or unrealistic computation).

g. Can turn on or off hydrograph on-line plotting at any time during execution.

h. Can turn on or off trace routines.

(1) Trace helps locate where program has stopped during an execution due to a program or data problem.

(2) All trace output has the word "TRACE" at the beginning of each trace printed line.

3. New variable and more versatile read-in of storm runoff hydrographs.

   a. Can use old data coded in time periods not equal to current run computation time period.

   b. Do not have to read in a specified number of ordinates.

   c. Can read in ordinate of time vs. discharge to define hydrograph. Program interpolates ordinates in the runs computation time period.

   d. First ordinate read in need not be the same time as the 1st time period of the run computation.

4. Reading in mass curves or Precipitation data.

   a. Can read in precipitation data with different time periods than the run computation time period.

   b. Can read in precipitation data accumulated as mass rainfall amounts (or percent of total rainfall).

   c. Can use SCS rainfall distribution curve or read in any other desired distribution.

d. Recoding of old data to change time periods is not necessary. Program recognized change of time periods that may or may not be different than the run computation time period.

e. Job input precipitation was made variable mass curve type.

5. SCS TR20 computation.

a. Whenever possible FER program uses exact coding of TR20 programming.

b. "FER" programs computer SCS unit hydrographs using the SCS dimensionless hydrograph.

c. Application of rainfall excess to unit hydrograph same as SCS routines.

d. SCS TR20 convex method of routing subroutine is a direct copy of the SCS program (slight modification of inputing hydrograph was required).

e. Initial SCS loss (Ia) was made a variable.

6. Combining & storage of hydrographs.

a. User control storage, retrieval, adding (or subtraction) of hydrograph storage areas.

b. Possible 1 to 8 areas can be used depending upon number of multiplan/storms.

c. Greater user control over storage areas.

7. Dynamic reach routing of hydrographs.

a. Uses cross section data to determine reach hydraulic elements.

b. Same section data can be used to compute a trial initial steady-state condition by using KCD Backwater program.

c. Solution use full dynamic equation solved by implicit method.

d. Routine is not critical of reach lengths or small delta time increments.

e. Small amount of time required for execution.

8. PMP Storm using HMR-51 rainfall (new method).

a. Computed rainfall precipitation amounts saved from HEC Computer program.

b. FER program attaches rainfall data file and searches data for comparable area labels then reads in rainfall amounts.

F. Future Needs and Developments.

1. Continue to locate and remove any unused coding that may or may not remain.

2. Continue changing program logic to operate with alphabetic control words (some numeric control remains).

3. Adopt and place more emphasis on new variable routines of hydrograph and precipitation data input and then remove fixed methods.

4. Complete and insert a new "S-curve" method (now being developed) to allow unit hydrographs to be read in and converted to a unit hydrograph equal to the execution consultation time period.

5. Develop a loose leaf type users handbook to explain in detail methods and procedures used in the FER program (this is being requested by the state of Nebraska).

G. Discussion (if any) (IF TIME)

FLOOD ESTIMATES AND ROUTINGS

Brief Description of "FER"
Computer Program
Major Control Cards
and their use

US Army Corps of Engineers
Kansas City District

FLOOD ESTIMATES AND ROUTING (FER)

SUMMARY OF MAJOR CONTROL CARDS

The FER computer program sequentially reads data modules of one or more lines (cards) of data to perform a specific procedure, computation or function. Computations are made in sequential order as encountered in the input data file. Each data module must have at least one or more lines (or cards) of data. The number of lines (or cards) is dependent upon the operation or computations desired. The first card of each data module is labeled a major control card. Its purpose is to direct the program operation and computations. If only one card is needed to perform the desired operation, then the data module would consist of only one line of data. If the procedure requires additional data to perform the desired computations, then the supporting data must be stacked directly after the first card of the data module. The FER program executes the computations or procedure requested by the data module then returns to read a new module of data. The procedure is reiterated until instructed to terminate execution.

The first card of each data module directs the program to perform one of the following computations and/or procedures.

(1) Run control of program input/output, etc.

(2) Storm runoff hydrographs (input or computations).

(3) Routing of hydrograph thru lakes, valleys, etc.

(4) Storing, adding and subtraction of computed hydrographs.

The first three columns of the major control card in each data module directs the program to perform one of the above four procedures. The program will

accept only one of the following four sets of 3 characters.

(1) "RUN", (2) "STM", (3) "RTE" and (4) "STO"

The fourth field of the major control card must always be blank. If the first three characters of the major control card is not verified to be "RUN", "STM", "RTE" or "STO" then execution of data modules will be terminated.

After the program has verified one of the procedures above, then the second control word in columns 4 thru 8 of the major control word are used to determine the type of computation desired. As an example, if the first 8 columns of the major control card were "RTE TATM", then the program would do a reach routing of a storm run off hydrograph by using Tatum's method of successive averages. The remaining data fields of the major control cards may or may not be used to perform computations.

Following is a listing of control words used on the major control cards and a brief description of the computation methods, procedure or usage of the control cards. The listing shows the first 8 columns of each major control card.

TABLE I. Major control cards for controlling program input, output, etc.

Columns 1-8	Remarks and Usage
"RUN BEG"	Begins execution of new job after a "RUN END" card has been read.
"RUN END"	Ends execution and begins new job. Next card should be a "RUN BEG" card.
	"RUN BEG" and "RUN END" cards are used for controlling job stack.
"RUN TRON"	Turns on program trace routines.
"RUN TROF"	Turns off program trace routines.
"RUN PLON"	Turns on Hydrograph on-line plottings.
"RUN PLOF"	Turns off Hydrograph on-line plottings.
"RUN PRNT"	Controls level of printed output.
"RUN TERM"	Terminates execution of computations. Also "RUN FIN" and "RUN STOP".

TABLE I. con't

"RUN TIME"	Allows changing computation time increment at any time during execution.
"RUN ECON"	Runs economic analysis of Hydrograph('s).
"RUN CMPR"	Compares computed Hydrograph with a hydrograph read into program.
"RUN NQ"	Changes the number of computation ordinates (Max 300).
"RUN TAPE"	Stores computed Hydrographs or other data on a file that can be read in during another execution.

TABLE II. Major Control Cards for storm hydrograph input and computations.

Columns 1-8	"Remarks & Usage"
"STM MULT"	Initiates reading in multi "STM" control cards and data when doing a multiple storm computation. When used program looks for a storm computation "STM" for each storm in a multistorm run.
"STM CMPR"	Compares computed storm with an input hydrograph. Same as "RUN CMPR" control card.
"STM TAPE"	Reads in data from a file that was saved from a previous execution. Reads data saved from a previous execution that has been stored by the major control card "RUN TAPE".
"STM READ"	(HEC) Read in Hydrograph. Hydrograph ordinates must be coded in the same time period as the computation time period and must read in the numbers of ordinates set by "NQ".
"STM HYD"	(KCD) Variable input routine of reading in hydrographs. Allows several ways of reading in Hydrograph data.
"STM SPF" *	Computation of Standard project flood rainfall (older method that was programmed in the HEC-1DD Computer Model).
"STM PMP" *	Computes probable maximum precipitation computed by older version of HEC-1DD Computer Program.

TABLE II. con't

"STM HM52"	*	Reads in 72 hour rainfalls saved from computation made using the HEC computer program HMR52.
"STM PRCP"	*	HEC reading in of precipitation data. Requires "HQ" ordinates to be read in.
"STM MASS"	*	KCD Reading in of precipitation data. Input type, periods, volume, etc. are variable and wide variety of input data can be read. TR20 rainfall distribution Tables I & II are internally programmed for direct application.
"STM PRWT"	*	HEC - Storm precipitation weighting routines using job precipitation data. Input of the job precipitation data can be in any time period because the program uses mass curve routines to determine delta rainfall amounts. This modification allows the method to be used with the FER program "RUN TIME" control card.
"STM RATO"		Multiplies ordinates of the computed hydrograph('s) by a constant.

* Whenever precipitation amounts are read in, computed, or used then the program requires loss control cards and unit hydrograph control cards to follow. These control cards, "LOS" and "UHG" cards, are secondary control cards that are required to compute a runoff hydrograph.

TABLE IIa. Secondary control cards for loss computations.

Columns 1-8	Remarks and Usage
"LOS HEC"	HEC General loss rate function for snow free ground.
"LOS HOLT"	Loss computation by "HOLT" function.
"LOS CNST"	Initial loss and constant infiltration rate.
"LOS SCS"	Uses SCS curve number and equation to compute rainfall losses.
"LOS SAME"	Uses last loss routine that was read in and used for previous computation.

NOTE: Loss control cards are used only when precipitation is used to compute rainfall excess. When an "STM" Major Control Card in used that requires rainfall amounts to be input or computed then the "LOS" control card is required to follow.

"LOS NONE"	No rainfall losses.

TABLE IIb. Secondary control cards to input or compute unit hydrographs.

Columns 1-8	Remarks and Usage
"UHG READ"	HEC unit hydrograph read in routine. (Input ordinates must be in same time period of computation time period.)
"UHG CLRK"	Compute unit hydrograph by Clark's Method with a unit duration equal to the computation time period.
"UHG SNYD"	HEC method uses Snyder coefficients to compute peak Q and time of peak then uses the Clark's method to approximate the Snyders parameters.
"UHG SCS"	Computes an SCS unit hydrograph using the programmed dimensionless unit graph (or read in) converts rainfall excess to a mass curve then computes delta rainfall excess amounts equal to unit hydrograph duration, then computes a storm runoff hydrograph. Ordinates equal to the FER computation time period are interpolated from the computed SCS runoff hydrograph. Computation, equations and procedure are coded directly from SCS TR20 program.

TABLE IIb. con't

"UHG SCRV" <u>Not available at this time.</u> Programming routine to read in a unit hydrograph for any duration and convert the unit duration by S-curve methods to obtain a unit hydrograph with a duration equal to the "FER" computation time period.

NOTE: Similiar as with the loss control card "LOS", a "UHG" control card must be read in whenever precipitation is used to compute a storm runoff hydrograph. The program will require a "UHG" control card after a "LOS" control card has been read in.

TABLE III. Major control card for routing of input or computed runoff hydrographs.

Columns 1-8	Use and Description
"RTE MULT"	Used in multiplan runs. Control card will initiate reading of routing control cards and data for each plan. Routing control cards and data must follow for each plan.
"RTE PULS"	Routes hydrographs thru lakes or valleys using Pul's method of storage discharge.
"RTE MPUL"	Same as "RTE PULS" except program reads in a cross section and develops a reach storage discharge relationship of the reach.
"RTE TATM"	Reach routing of hydrographs using Tatum's method of successive averages.
"RTE MUSK"	Routes hydrographs by Muskinghum's method using coefficients K and X.
"RTE MSKM"	Multiple layer discharge as Muskinghum routing coefficients. Muskinghum coefficients used depend on the discharge ordinates of the hydrographs.

TABLE III. con't

"RTE STST"	KCD method of Straddle-Stagger is used to reach route hydrographs. A specified number of inflows are averaged and logged a specified number of time periods.
"RTE RAD"	Uses the method of working R and D is used to reach route hydrographs. Uses storage versus discharge relationship of the valley reach.
"RTE MRAD"	Same as "RTE RAD" except a valley cross section, valley slope, and reach length are used to compute the reach storage versus discharge relationship.
"RTE DYAM"	Reach routing of hydrographs using full dynamic routing (including the acceleration term of the dynamic equation). Procedures used provide fast execution of hydrograph routing even with very small delta time periods. Solutions are not sensitive to reach lengths between cross section data.
"RTE TMST"	Multiple storage routing using time-of-storage coefficients. Method was developed and tested by HEC for use in Columbia River in Washington and Oregon. The HEC program fixed the power coefficient to -.2 but has been made a variable in FER.

TABLE III. con't

"RTE SCS"	The Soils Conservation Service (SCS) program uses the Convex Method of routing and the same TR20 program routing subroutine was inserted into the FER Computer program. Only slight modification was made to the subroutine to allow the FER inflow hydrograph variable names be used instead of the TR-20 variable names. The FER program allows SCS reach routing data to be read in directly or a fixed coefficient can be used to reach route the hydrographs. The FER program also can read in a valley cross section, reach length and slope to develop the storage discharge relationship needed to perform the SCS convex method of routing.
"RTE LAG"	Reach route hydrograph by averaging successive ordinates of the hydrograph a given number of times (Tatum's method) then log the hydrograph a given number of ordinates. Same procedures as used by Tatum's Method "RTE TATM".

TABLE III. con't

"RTE NULL"	Control card for null/or no routing/hydrograph. Used with multi plans and "RTE MULT" control card. For example, assume that two plans are being considered and the hydrograph for plan 1 was with a reservoir, and plan 2 was without a reservoir. Then, the "RTE MULT" card would be required to inform the program that more than one routing would be required. Following the "RTE MULT" card would be a "RTE PULS" control card and supporting data to reservoir route the hydrograph through the lake for plan 1. Then read in a "RTE NULL" card for the second plan to instruct the program to not route the hydrograph for plan 2.
"RTE NONE"	Same control card as "RTE NULL".
"RTE LOS"	HEC method of channel loss. Method subtracts a constant loss from each ordinate and then multiplies each ordinate of the remaining hydrograph ordinates by a constant value.
"RTE SAME"	Route hydrograph using lasts input routing procedure and data.

TABLE IV. Major control cards for storing, adding and subtraction of hydrographs.

Columns 1-8	Remarks
"STO HYD"	Store computed hydrograph('s) in a designated storage area. Number of storage areas are dependent on number of multi-storm/multi-plans used. Program determines available number of storage areas for single-storm/single plan computations maximum number of 8 storage areas is allowed.
"STO ADD"	Qs = Qw where; Qs represents ordinates in storage area and Qw = ordinates in computation working areas. Add computed hydrograph('s) to hydrograph in a designated storage area. The Qs = Qs + Qw.
"STO SUB"	Subtract computed hydrograph('s) from ordinates stored in a designated storage area. Qs = Qs - Qw
"STO GET"	Copy to the computation working area a hydrograph stored in a designated storage area Qw = Qs.
"STO TOT"	Sum one or more hydrographs stored in storage areas with a hydrograph in the computation working area. Zero out storage areas after computations are completed. Example let: Q1 = ordinates stored in Area 1, Q3 = ordinates stored in area 3, Q5 = ordinates stored in area 5, and Qw = the ordinates in the computation working area, then:

TABLE IV. con't

	Qw = Qw + Q1 + Q3 + Q5 after summation Q1, Q3 and Q5 storage areas will be zeroed out. The computation working area will be the summation of all four areas.
"STO ADDR"	Sum a selected storage area with the hydrograph in the working area then zero out the storage area. Qw = Qs + Qw then Qs = 0.0.
"STO SUBR"	Same as "STO ADDR" except subtract working area from storage area. Qw = Qs - Qw then Qs = 0.0
"STO ZAP"	Set all hydrograph storage array to zero.
"STO ZERO"	Selective setting one or more storage area to zero.

TR-20/HEC-1 COMBINATION

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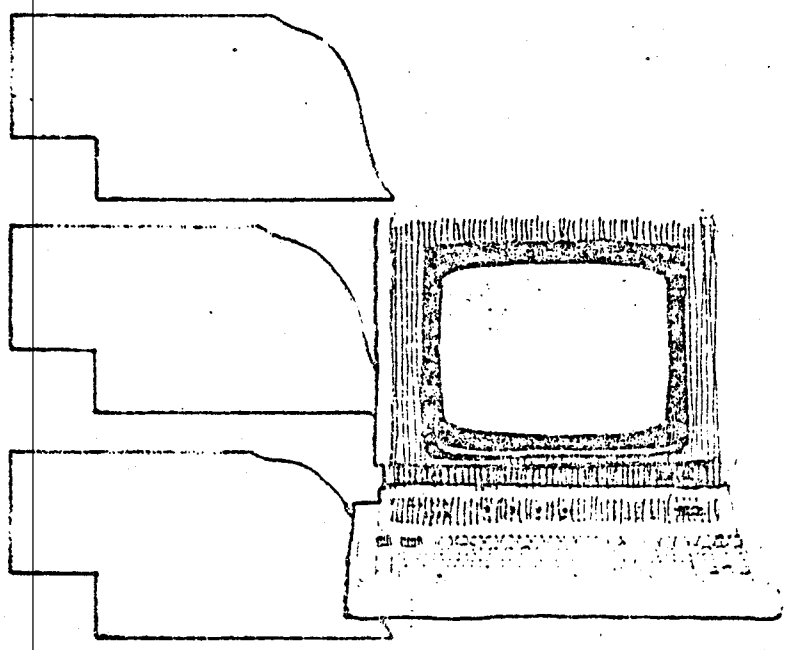
HYDROLOGIC MODELING FOR FLOOD HAZARDS • TR-20/HEC-1

GENERALIZED COMPUTER PROGRAM

FLOOD ESTIMATES AND ROUTING



USERS MANUAL

NEBRASKA

SEPTEMBER 1982

United States Army
Corps of Engineers
Serving the Army
Serving the Nation

Kansas City District

FLOOD ESTIMATES AND ROUTING
USERS MANUAL
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OVERVIEW OF FER PROGRAM

A. Original Source, HEC-1DD Version

B. IBM/CDC FORTRAN Language

C. HEC Routines Removed

(1) Optimization of unit hydrograph and rainfall loss coefficients

(2) Optimization of flood routing coefficients

(3) Snow melt computation

(4) Balanced hydrograph computation

(5) Storm system analysis

D. Program Restructured

(1) Module structure

(2) Easy to insert, change or delete subroutines

E. New or Modified Subroutines

(1) Use of channel cross-sections as input data

(2) Full dynamic routing

(3) Reservoir storage tables expanded

(4) Computation of storage-discharge relationship from cross-section data (MPUL and MRAD)

(5) "SCS" convex routing - with or without cross-section data

(6) "SCS" TR-20 rainfall-runoff relationships

(7) Use of mass curves for rainfall distribution

(8) Input of hydrographs, change from fixed to variable time intervals

(a) Variable intervals

(b) Fixed time intervals but different beginning time intervals

(9) Change of computation time interval during execution of data

(a) Program handles variable precipitation intervals

(b) Interpolates ordinates to new computation time interval

(c) Adding and subtracting of hydrograph is checked

(10) Simplification, consolidation and modification of required input data

(a) Input data easier to interpret

(b) Use of header cards expanded

(c) Control cards with supporting data (modular data)

(d) Control words

(11) Improved multi-plan and multi-storm computations

(a) Multi-storms - single plan

(b) Single storm - multi plans

(c) Multi-storms - multi plans

(12) User's control of adding, subtraction and storing computed hydrographs

(a) One to eight possible storage segments

(b) Time increment safety factor

F. Users Manual - Introduction

OUTLINE

FER Computer Program

A. Program development initiated by State of Nebraska's Natural Resources Commission, Lincoln, Nebraska (NNRC).

1. Needed a program to combine both HEC and TR20.

2. Desired to simplify data input into program.

3. Restructure data input and use more alphabetic control rather than numeric number.

B. Adopted HEC-1 program (Version DD) as base program.

1. Traced program logic and removed some routines.

a. Removed all hydrograph optimization.

b. Removed balanced hydrograph procedure.

c. Removed snow melt computation. (does not work in state of Nebraska.)

d. Removed storm system computations.

e. All other routines were preserved.

2. Restructured and segmented program into modular subroutines.

a. Removed and change program control.

b. Program easier to modify and insert new procedures.

C. Restructured input data.

1. Reads in sequential modules of data.

2. Uses control words to control execution.

3. Allows more comments to be placed in data.

4. Major control card words (two words 1st 8 digits of the first card (line) of each data module).

5. Multiplan/multistorm procedures retained & simplified.

a. "STM_MULT" control card

b. "RTE_MULT" control card

D. Routines and Procedures inserted into the FER program.

1. User can change computation time period during execution.

a. Can either increase or decrease time period.

b. Most subroutines automatically consider changes of computation time.

2. Printed output control.

a. Can be selective.

b. Can turn on or off at any time during computation.

c. Major control cards echoed in printout.

d. Prints out only necessary ordinates, truncates trailing zeros.

e. Can control number of ordinates to be used (max = 300).

f. Inserted Creager's formula for maximum peaks (can be used to check for possible errors or unrealistic computation).

g. Can turn on or off hydrograph on-line plotting at any time during execution.

h. Can turn on or off trace routines.

(1) Trace helps locate where program has stopped during an execution due to a program or data problem.

(2) All trace output has the word "TRACE" at the beginning of each trace printed line.

3. New variable and more versatile read-in of storm runoff hydrographs.

a. Can use old data coded in time periods not equal to current run computation time period.

b. Do not have to read in a specified number of ordinates.

c. Can read in ordinate of time vs. discharge to define hydrograph. Program interp~~rotes~~olates ordinates in the runs computation time period.

d. First ordinate read in need not be the same time as the 1st time period of the run computation.

4. Reading in mass curves or precipitation data.

a. Can read in precipitation data with different time periods than the run computation time period.

b. Can read in precipitation data accumulated as % mass rainfall amounts (or percent of total rainfall).

c. Can use SCS rainfall distribution curve or read in any other desired distribution.

d. Recoding of old data to change time periods is not necessary. Program recognized change of time periods that may or may not be different than the run computation time period.

e. Job input precipitation was made variable mass curve type.

5. SCS TR20 computation.

a. Whenever possible FER program uses exact coding of TR20 programming.

b. "FER" programs computer SCS unit hydrographs using the SCS dimensionless hydrograph.

c. Application of rainfall excess to unit hydrograph same as SCS routines.

d. SCS TR20 convex method of routing subroutine is a direct copy of the SCS program (slight modification of inputing hydrograph was required).

e. Initial SCS loss (Ia) was made a variable.

6. Combining & storage of hydrographs.

a. User control storage, retrieval, adding (or subtraction) of hydrograph storage areas.

b. Possible 1 to 8 areas can be used depending upon number of multiplan/storms.

c. Greater user control over storage areas.

7. Dynamic reach routing of hydrographs.

a. Uses cross section data to determine reach hydraulic elements.

b. Same section data can be used to compute a trial initial steady-state condition by using KCD Backwater program.

c. Solution use full dynamic equation solved by implicit method.

d. Routine is not critical of reach lengths or small delta time increments.

e. Small amount of time required for execution.

8. PMP Storm using HMR-51 rainfall (new method).

a. Computed rainfall precipitation amounts saved from HEC Computer program.

b. FER program attaches rainfall data file and searches data for comparable area labels then reads in rainfall amounts.

F. Future Needs and Developments.

1. Continue to locate and remove any unused coding that may or may not remain.

2. Continue changing program logic to operate with alphabetic control words (some numeric control remains).

3. Adopt and place more emphasis on new variable routines of hydrograph and precipitation data input and then remove fixed methods.

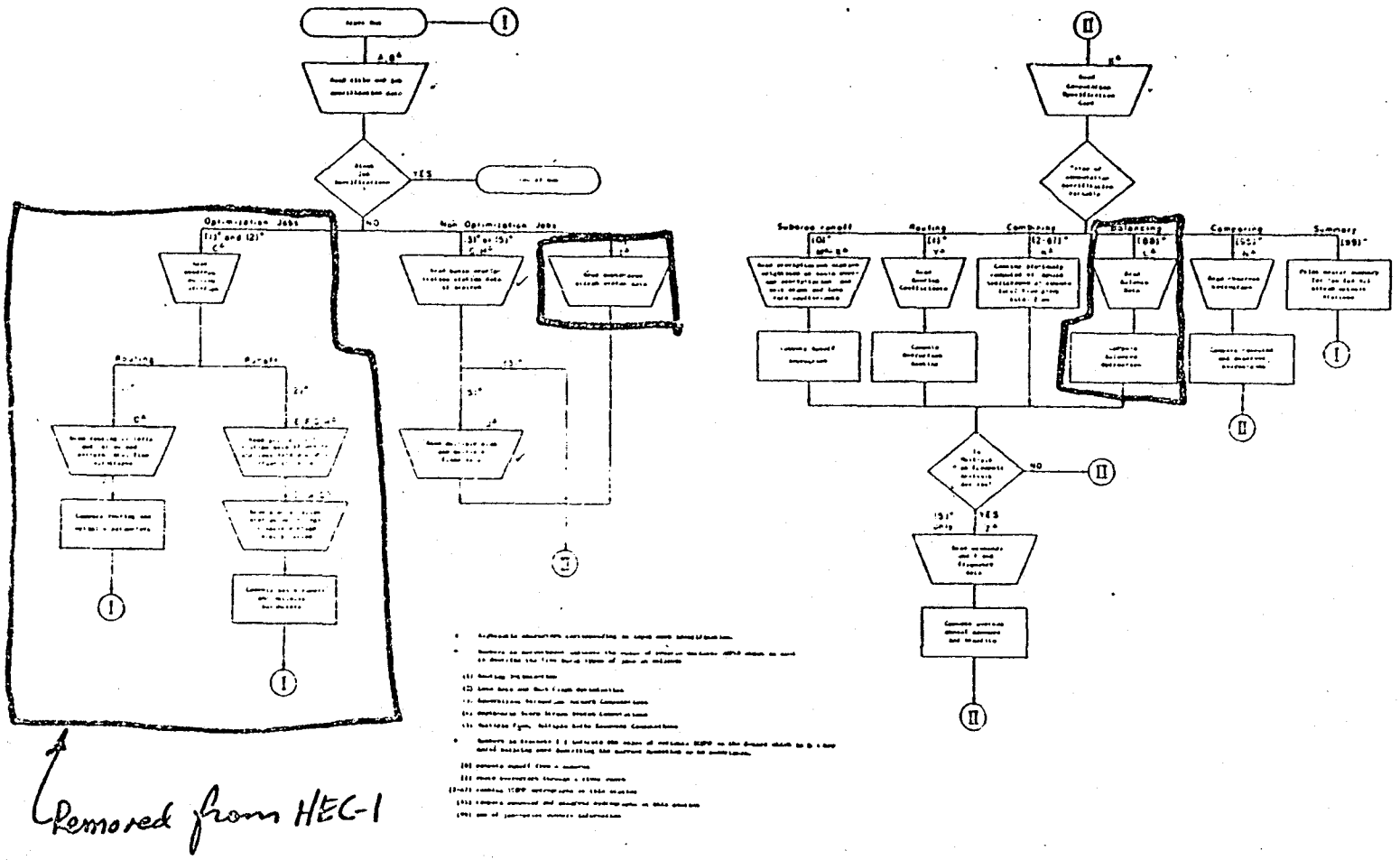
4. Complete and insert a new "S-curve" method (now being developed) to allow unit hydrographs to be read in and converted to a unit hydrograph equal to the execution consultation time period.

5. Develop a loose leaf type users handbook to explain in detail methods and procedures used in the FER program (this is being requested by the state of Nebraska).

VG #1 - Plate from HEC-1DD program showing the program structure on the removed segments.

Figure 1

HEC-1 MACRO-FLOW CHART



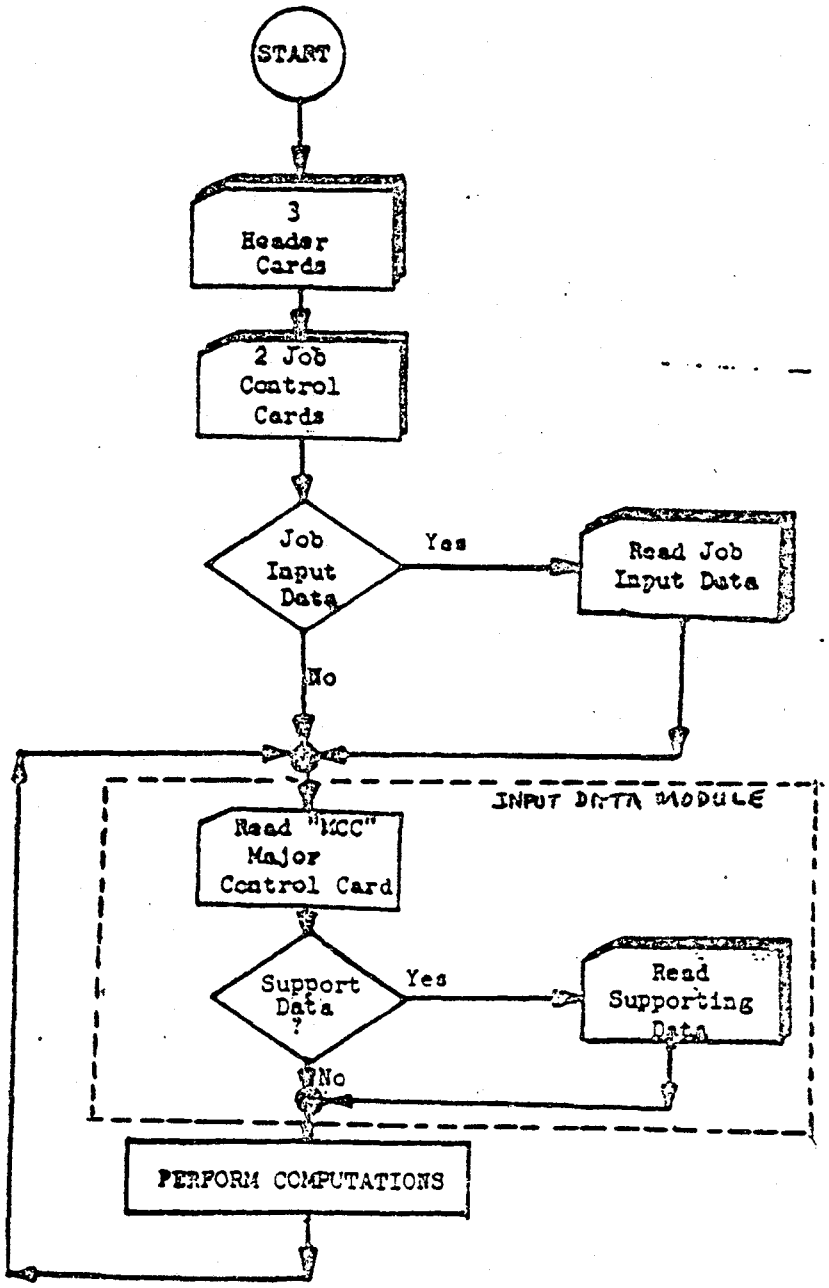
VG #2 - Diagram showing new "FER" program subroutine linkage.

```
1          MAP OF    FERKC                          *** PAGE  1
0
                   +-DATE*
                   I-TIME*
                   I-IOPRNT
                   I-XDATE
                   I-RAINGA
                   I-OUTSUM----SUMRY
                   I-ECONO
                   I-RUNTM
                   I-QMOVE
                   I-SUMRY
                   I
                   I          +-SUMRY
                   I-COMPAR--I
                   I          +-GRAPH
                   I
                   I          +-SUMRY
                   I-HYSTOR--I
                   I          +-GRAPH
                   I
                   I          +-TIMEX2
                   I-HYREAD--I
                   I          +-SUMRY
                   I
                   I-PMFDST
          FERKC---I
                   I-READPR
                   I-QCARDS
                   I-RNMASS
                   I-RAINLS----SCSCN
                   I-UGORTS
                   I-TMAREA
                   I-UGDATA
                   I-RAINFL----INTRN
                   I
                   I          +-UGSCS
                   I          I-UGCLRK
                   I          I-LOSSCS
                   I-RUNOF---I
                   I          I-LOSCON
                   I          I-LOSHLT
                   I          +-LOSHEC
                   I
                   I-GRAPH
                   I-ARATIO
                   I
                   I-HMR52              +-UGSCS
                   I                    I-UGCLRK
                   I                    I-LOSSCS
                   I          +-RUNOF---I
                   I          I         I-LOSCON
                   I          I         I-LOSHLT
                   I          I         +-LOSHEC
                   I          I-SUMRY
                   I-MULTPI--I
                   I          I-GRAPH
                   I          +-QMOVE
                   I
                   I-RTECRD----KCDXS
                   I
                   I          +-RTELAG
                   I          I-RTERAD--- RTEPUL----RTELAG
                   I          I-RTETMS
                   I          I-RTEMSK
                   +-ROUTE---I-RTETAT
                              I-RTESTS----RTELAG
                              I-RTEPUL----RTELAG
                              I
                              I         +-HLOGIC
                              I         I-KCDXS
                              I         I
                              I         I         +-TRIB
                              I         I-RCDATA--I-OX
                              I         I         I
                              I         I         I         +-INTER
                              I         I         +-BACWTR--I
                              I         I                   +-TERP
                              I         I
                              I         I
                              I-RTEDYM--I-TERP
                              I         I-TIMEX2
                              I         I-INTER
                              I         +-TRI
                              I
                              +-RTESCS----KCDXS
                              + RTEMASK
```

V.G. #2

VG #3 - Flow diagram of input data.

PROGRAM INPUT DATA



VG #4 - Data for "HEC" test No. 5 from "HEC" Manual.

```
A                                TEST NO. 5
A                            TEST OF SPFE, PRS
A                     AND ROUTING AND COMBINING ROUTINES
B    60      2                                          1       1       0
1     3
K     3      1       0       0       0
R     1      0     100       0     100   0.667       0       0
P           29.50   89.8   100.0   187.0   118.0
T                                            0.50     .15               0
Y  7.67    8.72
X     0      0    1.00
K     1      2       0       0       0
Y                             1
1                             2                     -1.
2     0  10000   25000   40000   55000   70000   85000  100000  250000
3     0    100    1000    5000   10000   20000   30000   40000   50000
K     0      2       0       0       0
R     0      0     177       0       0       0     0.8       0
O    43  19.95
1     0   .001    .002    .001    .001    .003    .009    .005    .006    .012
1   .021   .031    .051    .055    .045    .046    .040    .034    .040    .049
1   .056   .066    .079   .0135    .045    .022    .009    .009    .014    .013
1   .002   .005    .014    .008    .001    .000    .005    .007    .013    .021
1   .010   .005    .004    .003    .002
T  1.03    1.02     4.0     0.7
Y  3.68    2.94      11
1     3      3       8      17      23      33      50      66      79      94
1   100
X  1860    1770    1.49
K     2      2       0       0       0
K     1      4       0       0       0
Y   100     .2
1     3      2       0
K     0      3       0       0       0       0
A     1     -1     100       0     100   0.867       0       0
P  14.0
T                                            1.00      .2     2.1       0
U    18
1  1549   5135    8011    7775    4990    2444    1197     586     287     141
X  -1.0      0    1.00
K     1      4       0       0       0       0
Y    20     .1
1     1                     3.2     0.2
K     2      4
K     1      5       0       0       0
Y     1
1            3       2
K     0      5       0       0       0       0
R    -1      0     200       0     200       0       0
N     0                                                                170
N     0
N  1180   2190    2570    1550     788     390     195     525    2040    6460
N  0040  16500   39900   58600   58603   40800   23308   11600    5940    3000
N  1030    698     175      29       8
N     0
K     2      5       0       0       0
K     1      6
Y
1                     3
K     0      6
R                   700                                                     1
N  1063   1063    1063    1063    1063    1061    1052    1030     994     955
N  9150   5760    3270    2030    7670    7330    7290    5620   14370   27950
N  3063   7712   10277   13102   15675   17964   20576   24880   30961   37896
N  44534  50211   53593   55615   63316   83180  103214  100608   99196   87362
N  76377  62573   47452   34747   26097   20979   16184   16162   14062   11994
N  10134   6013    7942    7378    6969    6611    6271    5948    5643    5355
K     1      7
Y                             1
1                                     0.2              -1
1     0   1000   10000   50000   75000  125000  300000
3     0   1000    9000   30000   55000   78000  100000
X    95      7
N  1063   1063    1063    1063    1063    1061    1052    1030     996     955
N   915    876     839     803     769     735     729     862    1437    2795
N  3063   7712   10277   13102   15675   17964   20576   24880   30961   37896
N  44534  50211   53593   55615   63316   83180  103214  100608   99196   87362
N  76377  62573   47452   34747   26097   20979   16184   16162   14062   11994
N  10134   6313    7942    7378    6969    6611    6271    5948    5643    5355
K    99
```

VG #5 - Same data recoded to "FER" INPUT DATA.

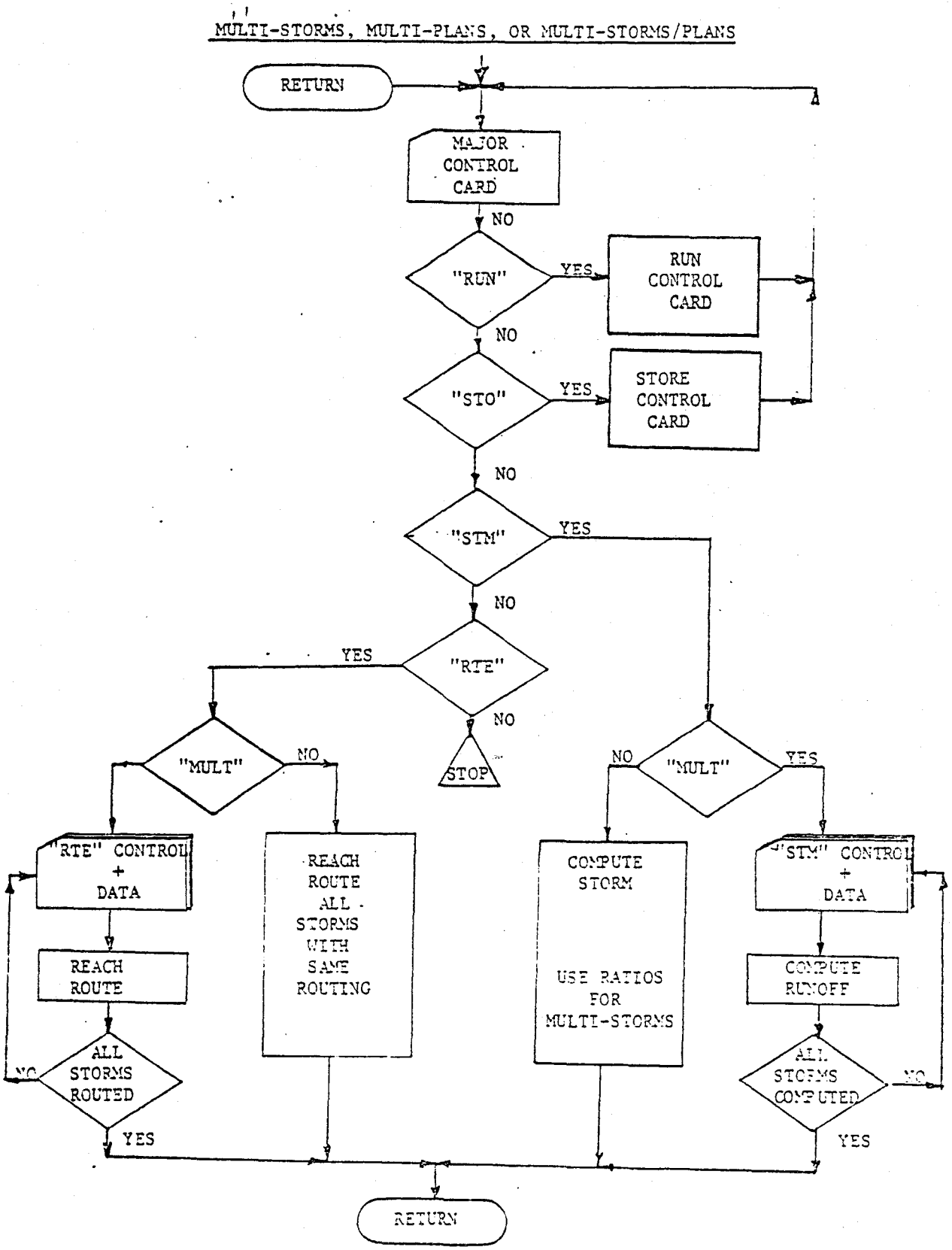
```
DFERB                    TEST NO. 5
          THIS TEST IS THE SAME AS HEC1-TEST 5 THRU STEP 16
          TEST OF SPFE, PMS AND ROUTING AND COMBINING ROUTINES
     50       2       0  1003.0       0       0       0     0.0     0.0       0
      0       0       0      10
STM PMP    AREA1 100.0  100.0   0.867   29.50   0.0
   89.0   100.0   107.0   118.0
LOS CNST0.50    0.15     0.0     0.0     0.0     0.0     0.0     0.0     0.0
UHG CLRK7.67    2.92     0.0     0.0     0.0     0.0     1.0     1.0     0.0
RTE PULS     IP2 0.0    -1.0     9.0     0.0     0.0     0.0     2.0
      0   10000   25000   40000   55000   70000   85000  100000  250000
      0     100    1000    5000   10000   20000   30000   40000   50000
STO HYD         1.0
STM PRCP   AREA2 177.0  45.0    19.55   0.0     0.0     0.8
      0    .001    .002    .001    .001    .003    .009    .005    .006    .012
   .021    .031    .051    .055    .043    .046    .040    .034    .040    .049
   .056    .066    .079   .0135    .045    .022    .009    .009    .014    .013
   .002    .005    .014    .008    .001    .000    .005    .007    .013    .021
   .018    .005    .004    .003    .002
LOS HEC 1.02    1.02     4.0     0.7     0.0     0.0     0.0     0.0     0.0
UHG CLRK 3.68   2.94    11.0     0.0    1860.   1770.    1.49    1.0     0.0
      0       3       8      17      23      33      50      66      79      94
    100
STO ADDR     IP2  1.0
RTE TATM     IP4 3.0                     0.0     100.0   0.2     0.0
STO HYD         1.0
STM SPF    AREA3 100.0  100.0   0.867   14.0    0.0
LOS CNST1.00    0.2      2.1     0.0     0.0     0.0     0.0     0.0     0.0
UHG READ 10.0   0.0      0.0     0.0    -1.0     0.0     1.0     1.0     0.0
   1549    5135    8011    7775    4990    2444    1197     586     287     141
RTE MUSK     IP4 3.2     0.2     1.0             20.0    0.1     0.0     0.0
STO TOT      IP4 1.0
RTE STST     IP5 3.0                             1.0     0.0     1.0     0.0
STO HYD         1.0
STM READ   AREA5 200.0  200.0   0.0
      0                                                                     170
   1180    2190    2370    1550     780     390     195     525    2040    4480
   8440   18500   39000   58600   58600   40000   22300   11600    5960    3000
   1490     690     175      80       8
      0
STO ADDR        1.0
RTE LAG      IP6 3.0     0.0             0.0     0.0
STO HYD         1.0
STM READ   AREA6 700.0
   1063    1063    1063    1063    1063    1061    1052    1030     996     955
    915     876     839     803     769     738     729     862    1437    2795
   5065    7712   10277   13102   15675   17964   20576   24880   30061   37896
  44556   50211   53593   55815   63316   83180  103214  108608   99196   87362
  76377   62573   47452   34747   26097   20079   18184   16162   14062   11944
  10134    8813    7942    7378    6969    6611    6271    5948    5643    5355
RTE RAD      IP7 -1.0     3.2     7.0     0.0     0.0     0.0     0.0     0.0
      0    1000   10000   50000   75000  125000  300000
      3    1000    5000   30000   55000   75000  100000
STO HYD         1.0
RUN PLOT
STM CMPR     IP7
   1063    1063    1063    1063    1063    1061    1052    1030     996     955
    915     876     839     803     769     738     729     862    1437    2795
   5065    7712   10277   13102   15675   17964   20576   24880   30061   37896
  44556   50211   53593   55815   63316   83180  103214  108608   99196   87362
  76377   62573   47452   34747   26097   20079   18184   16162   14062   11944
  10134    8813    7942    7378    6969    6611    6271    5948    5643    5355
RUN PLOF
RUN TERM
EOT
```

VG #6 - Listing of primary and secondary control words.

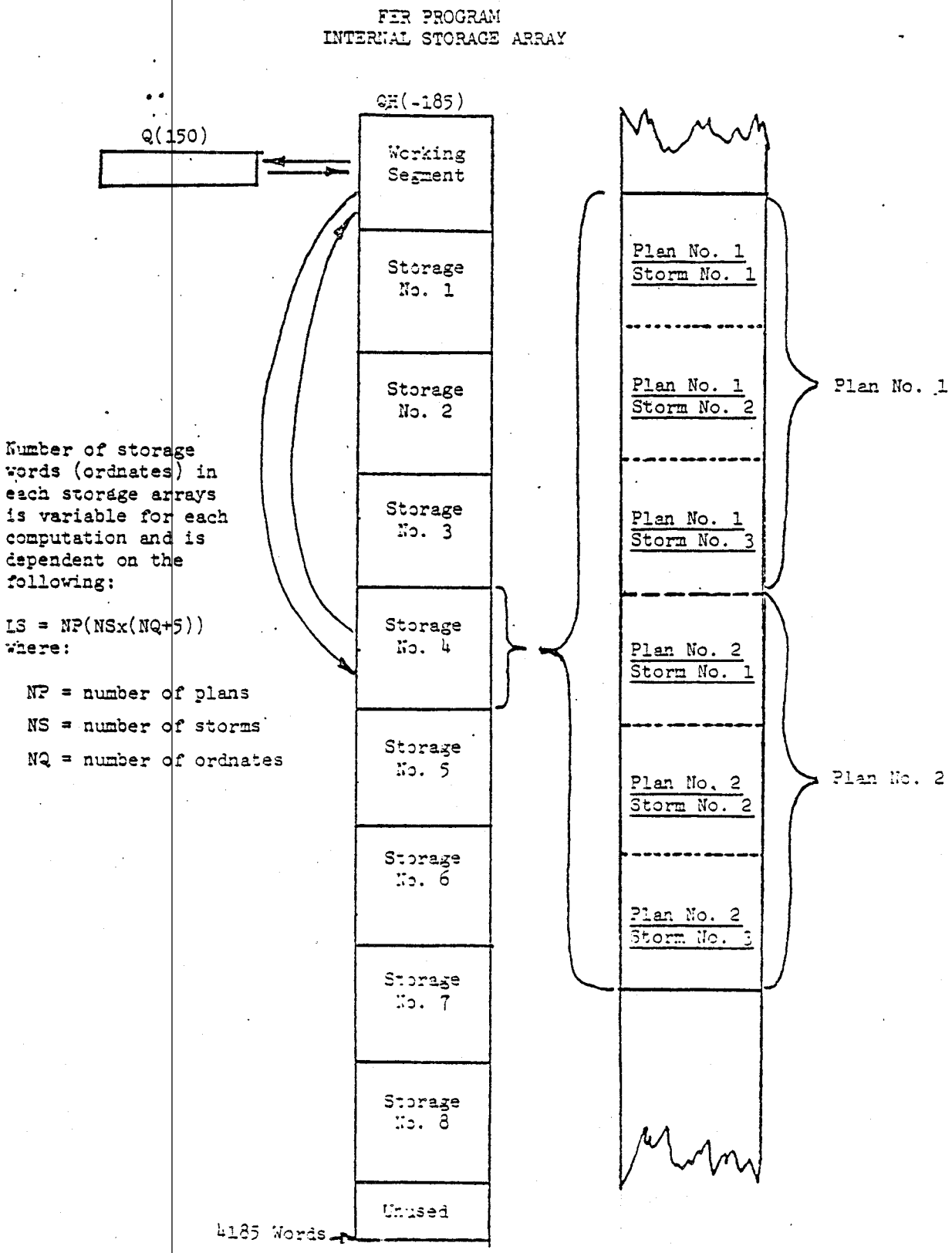
FER PROGRAM CONTROL WORDS				
MAJOR CONTROL CARDS				
CARD COLUMNS 1 THRU 3				
"HEM"	"STM"		"RTE"	"STO"
CARD COLUMNS 5 THRU 8				
"BEG " "END " "[illegible]" "PLOT" "PRNT" "TIME" "DOWN" "TDE" "TROF" "PLOF" "CMPR" "NQ.." "TAPE"	"MULT" "READ" "HYD " "PRCP" "PRNT" "SPF"" "TMP " "COPR" "MASS"	"TAPE" "HMS2" "RATO"	"MULT" "DONE" "SAME" "TATM" "MUSK" "STST" "LAG " "LOS " "TMST" "PULS" "MPUL" "RAD " "TRAD" "SCS " "DYAM" "MSKM" "NULL"	"HYD " "GET " "ADD " "ADDR" "SUB " "TOT " "ZERO" "ZAP "
Secondary Control Cards To Be Used With "STM" CARD COLUMNS 1 THRU 3				
"LOG"	"UBC"			
CARD COLUMNS 5 THRU 8				
"HOLT" "SAME" "DONE" "UBC " "CMST" "SCS "	"SAME" "READ" "CLEX" "SNTD" "SCS " "SCRY"			

Future

VG #7 - Flow diagram showing use of multistorm/multiplan control cards.



VG #8 - FER program storage areas.



BASIN RUNOFF MODEL

STRANGER CREEK

INPUT DATA LISTING

DEVELOPMENT OF UNIT HYDROGRAPHS

REACH ROUTING AND ACCUMULATIONS

HYDROLOGIC ANALYSES OF WATERSHED SYSTEMS

A. Need for hydrologic analyses of floods

flood plain information
water resources planning
design of water resource projects
operation of water resource projects
national dam safety program

B. Hydrologic investigations

1. Peak discharge and volume frequency analyses for:

existing watershed conditions
projected land use changes
alternate plans of development

2. Reservoir storage requirements for flood control

3. Design floods, peak and volume, for water resource projects

(a) Impoundments

top of embankment determination
type and size of emergency spillway
type and size of outlet works

(b) Levee local protection projects

(c) Channel improvement projects

4. Water surface profiles

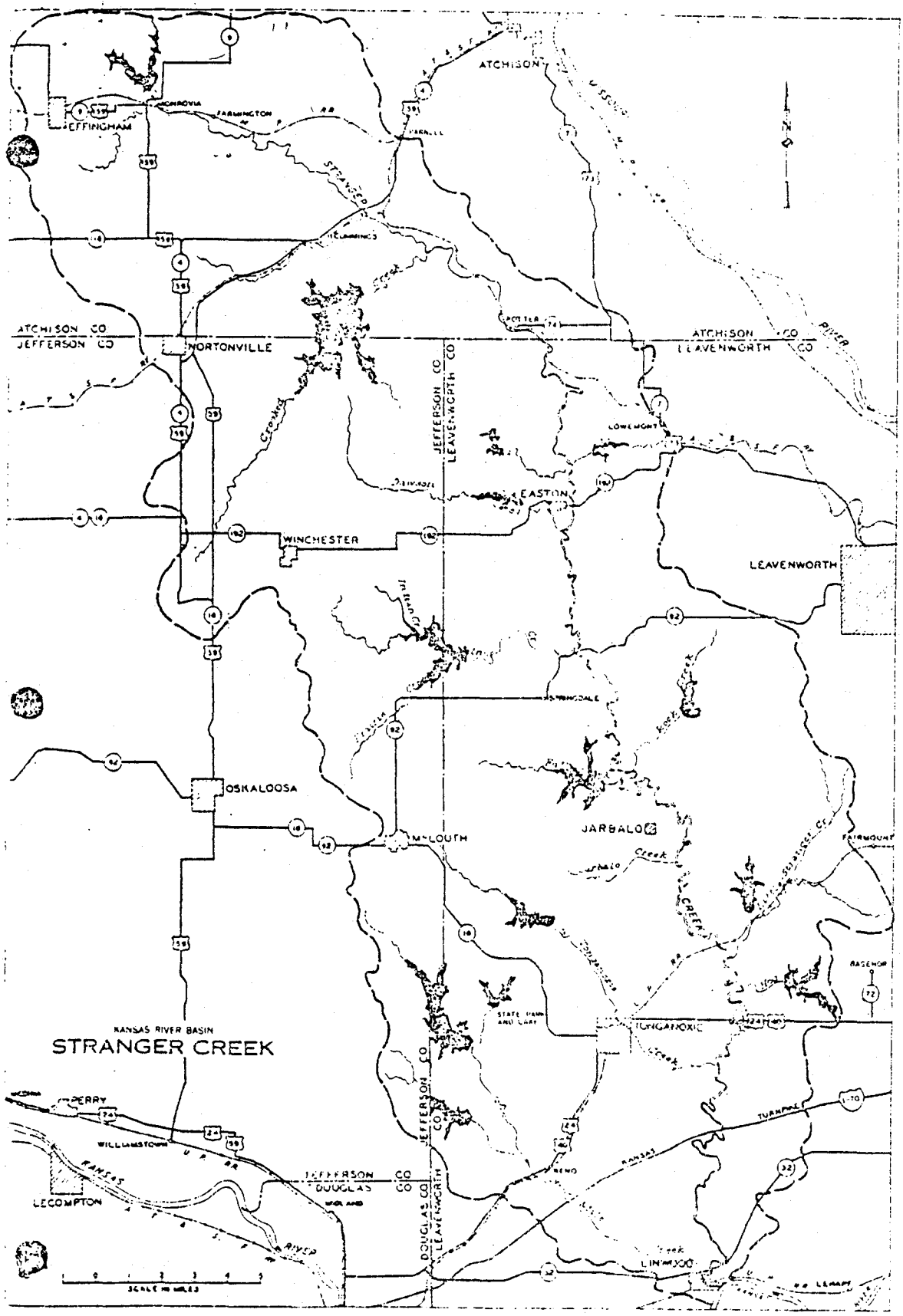
C. Information needs

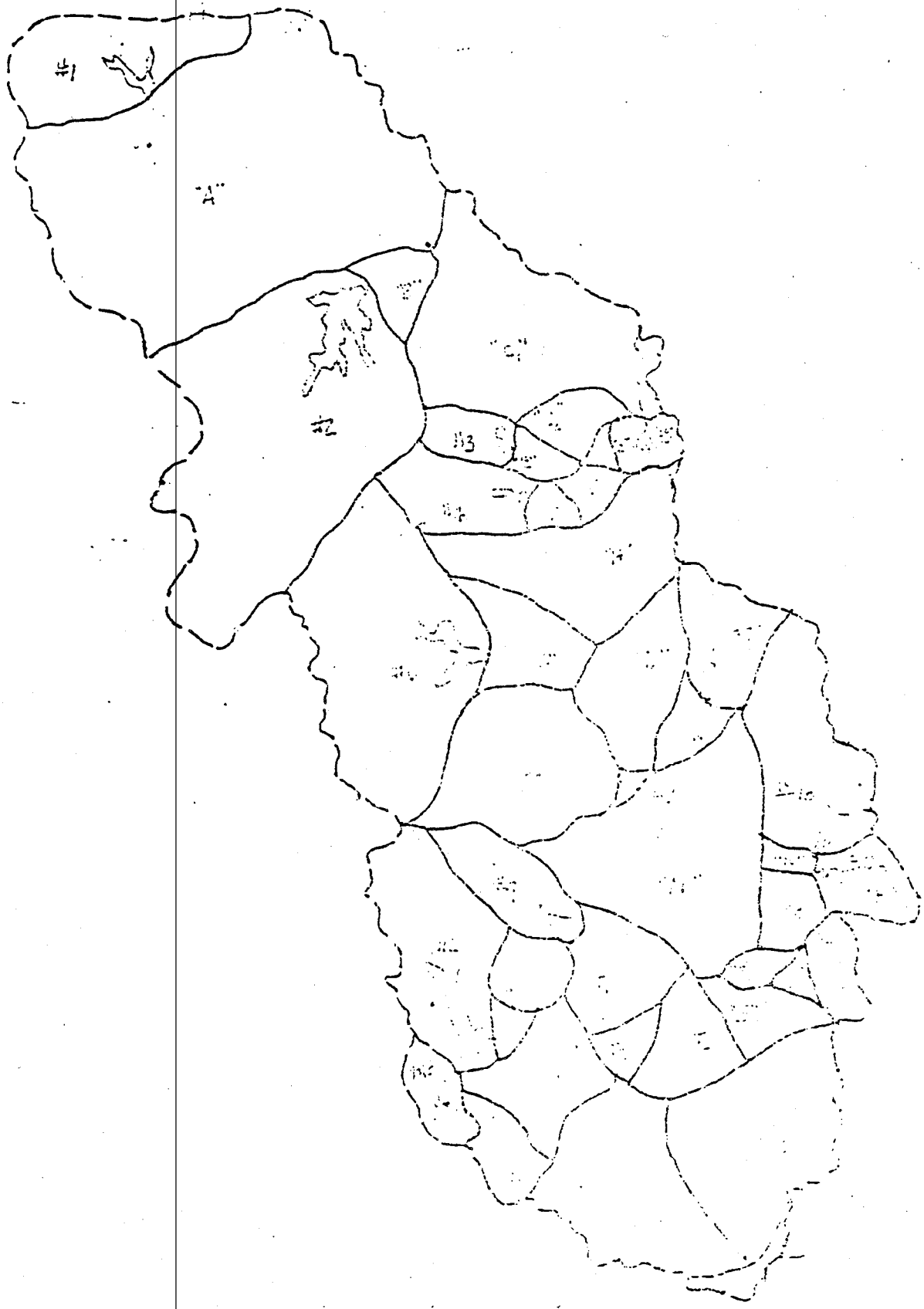
stream gaging records
precipitation data for storms
topographic maps
channel and valley cross-sections
precipitation data for hypothetical storms

D. Watershed Delineation

pertinent index locations
subarea boundaries
stream routing reaches

HYDROLOGIC MODELING FOR FLOOD HAZARDS • TR-20/HEC-1




#1
"A"
#2
#3

```
STRANGER CREEK STUDY - CALIBRATION RUN NO. 16
ATTEMPT TO RECONSTITUTE COMPUTED UNIT HYDROGRAPHS BY ACCUMULATION OF SUBAREAS
COMPARE ACUMG AND ACUMOR (ROUTED SIX PERIODS) WITH EASTON AND TONGANOXIE
150.    1.0
0.                                          ?
RUN PRNT        -25.
STM MASS  LAKE1 21.8    1.      1.      2.
0.0     1.0
LOS NONE
UHG CLRK3.4     5.1     -2.5    1.0
STO HYD   LAKE1 3.0
RUN PRNT        25.
RTE TATM  CONVRT6.
RUN PRNT        -25.
STO GET   LAKE1 3.0
RTE TATM  RECHA 12.
STO HYD   1RA   1.0
STM MASS  AREAA 84.6    1.
LOS NONE
UHG CLRK12.2    9.0     -2.0    0.5
STO ADD   1ARA  1.0
STM MASS  LAKE2 59.7    1.
LOS NONE
UHG CLRK7.4     3.7     -2.0    0.5
STO HYD   LAKE2 3.0
RUN PRNT        25.
RTE TATM  CONVRT6.
RUN PRNT        -25.
STO GET   LAKE2 3.0
RUN TIME        0.0     30.
RTE TATM  RECHB 8.
STO HYD   2RB   2.0
STM MASS  AREAB 5.4     1.
LOS NONE
UHG CLRK3.              -3.5    1.0
STO ADDR  2BRB  2.0
RUN TIME        1.0     0.0
STO ADDR  ACUMAB1.0
RTE TATM  RECHC17.
STO HYD   ABRC1 1.0
STM MASS  AREAC126.2    1.
LOS NONE
UHG CLRK6.      5.0     -2.5    1.0
STO ADDR  ACUMC11.0
RTE TATM  RCHC2G5.
STO HYD   1C1RG 1.0
RUN TIME        0.0     30.
STM MASS  AREAC28.5     1.
LOS NONE
UHG CLRK2.0     3.0     -2.5    1.0
STO HYD   C2    2.0
STM MASS  LAKE4 3.6     1.
LOS NONE
UHG CLRK3.      4.5     -2.5    1.0
```

DSCUHG1 page 1

```
STO HYD   LAKE4 3.0
RUN PRNT        25.
RTE TATM  CONVRT12.
RUN PRNT        -25.
STO GET   LAKE4 3.0
RTE TATM  RECHE 4.
STO ADD   4RE   2.0
STM MASS  AREAE 1.5     1.
LOS NONE
UHG CLRK1.5     2.25    -2.5    1.0
STO ADD   ACUM4E2.0
STM MASS  LAKE5 3.8     1.0
LOS NONE
UHG CLRK2.5     3.75    -2.5    1.0
STO HYD   LAKE5 3.0
RUN PRNT        25.
RTE TATM  CONVRT12.
RUN PRNT        -25.
STO GET   LAKE5 3.0
RTE TATM  RECHD 1.
STO ADD   ACM4E52.0
STM MASS  AREAD 0.5     1.
LOS NONE
UHG CLRK1.2     1.8     -3.5    1.0
STO ADDR  ACUM452.0
RTE TATM  RECHG 5.
STO HYD   45RG  2.0
STM MASS  AREAG 1.9     1.
LOS NONE
UHG CLRK1.5     2.25    -3.0    1.0
STO ADDR  ACMC2G2.0
RUN TIME        1.0     0.0
STO ADD   ACUMG 1.0
STO GET   ACUMG 1.0
RTE TATM  CONVRT6.
RUN TIME        0.0     30.
STM MASS  LAKE3 7.4     1.
LOS NONE
UHG CLRK4.      6.0     -2.5    1.0
STO HYD   LAKE3 3.0
RUN PRNT        25.
RTE TATM  CONVRT12.
RUN PRNT        -25.
STO GET   LAKE3 3.0
RTE TATM  RECHF 4.
STO HYD   ACUMG32.0
STM MASS  AREAF 2.0     1.
LOS NONE
UHG CLRK2.0     3.0     -3.0    1.0
STO ADDR  ACUMF 2.0
RUN TIME        1.0     0.0
STO ADDR  ACUMFG1.0
RTE TATM  RECHH 5.
STO HYD   1GRH  1.0
STM MASS  AREAH 13.5    1.
```

EASTON

#1

#2

DSCUHG1 #2

```
LOS NONE
UHG CLRK6.5             -3.5    1.0
STO ADD   ACUMH 1.0
STM MASS  LAKE6 35.1    1.
LOS NONE
UHG CLRK6.5             -2.5    1.0
STO HYD   LAKE6 3.0
RUN PRNT        25.
RTE TATM  CONVRT6.
RUN PRNT        -25.
STO GET   LAKE6 3.0
RTE TATM  RECHI 6.
STO HYD   6RH   2.0
STM MASS  AREAI 11.3    1.
LOS NONE
UHG CLRK4.0             -2.2    1.0
STO ADDR  6IRH  2.0
STO ADDR  ACUMI 1.0
RTE TATM  RECHJ 5.0
STO HYD   1HRJ  1.0
STM MASS  AREAJ 8.0     1.
LOS NONE
UHG CLRK4.5             -3.0    1.0
STO ADD   ACUMJ 1.0
RUN TIME        0.0     30.
STM MASS  LAKE7 4.6     1.
LOS NONE
UHG CLRK4.      6.0     -2.7    1.0
STO HYD   LAKE7 3.0
RUN PRNT        25.
RTE TATM  CONVRT12.
RUN PRNT        -25.
STO GET   LAKE7 3.0
RTE TATM  RECHK 8.0
STO HYD   7RJ   2.0
STM MASS  AREAK 3.3     1.
LOS NONE
UHG CLRK2.      3.0     -3.0    1.0
STO ADD   7K    2.0
STM MASS  LAKE8 20.8    1.
LOS NONE
UHG CLRK2.5             -2.7    1.0
STO HYD   LAKE8 3.0
RUN PRNT        25.
RTE TATM  CONVRT12.
RUN PRNT        -25.
STO GET   LAKE8 3.0
RTE TATM  RECHL 3.0
STO ADD   7KSRJ 2.0
STM MASS  AREAL 1.4     1.
LOS NONE
UHG CLRK2.5     3.75    -3.0    1.0
STO ADDR  7K8L  2.0
RUN TIME        1.0     0.0
STO ADDR  ACUMFL1.0
```

DSCUHG1 #3

```
RTE TATM  RECHM 15.
STO HYD   1LRM  1.0
STM MASS  AREAM 35.6    1.
LOS NONE
UHG CLRK8.0             -3.2    1.0
STO ADD   ACUMM 1.0
STM MASS  LAKE1014.2    1.
LOS NONE
UHG CLRK7.0             -2.5    0.5
STO HYD   LAKE103.0
RUN PRNT        25.
RTE TATM  CONVRT6.
RUN PRNT        -25.
STO GET   LAKE103.0
RUN TIME        0.0     30.
RTE TATM  RECHN 2.
STO HYD   10RN  2.0
STM MASS  AREAN 1.4     1.
LOS NONE
UHG CLRK1.0     1.5     -3.2    1.0
STO ADD   10N   2.0
STM MASS  LAK10A5.5     1.
LOS NONE
UHG CLRK2.5     3.75    -2.5    1.0
STO HYD   LAK10A3.0
RUN PRNT        25.
RTE TATM  CONVRT12.
RUN PRNT        -25.
STO GET   LAK10A3.0
RTE TATM  RECHP 2.
STO ADDR  10N10A2.0
STO HYD   10N10A2.0
STM MASS  AREAP 7.5     1.
LOS NONE
UHG CLRK2.5     3.75    -3.5    1.0
STO ADDR  ACUMNP2.0
RTE TATM  RECHO 8.
STO HYD   NPRO  2.0
STM MASS  AREAO 4.5     1.
LOS NONE
UHG CLRK2.5     3.75    -3.2    1.0
STO ADDR  ACUMO 2.0
RUN TIME        1.0     0.0
STO ADDR  ACUMMO1.0
RTE TATM  RECHQ 2.
STO HYD   MORQ  1.0
RUN TIME        0.0     30.
STM MASS  AREAQ 1.3     1.
LOS NONE
UHG CLRK1.0     1.5     -3.0    1.0
RUN TIME        1.0     0.0
STO ADD   ACUMO 1.0
STM MASS  LAKE1111.3    1.
LOS NONE
UHG CLRK4.5     6.75    -2.7    1.0
```

DSCUHS1 pg 4

```
STO HYD   LAKE113.0
RUN PRNT        25.
RTE TATM  CONVRT6.
RUN PRNT        -25.
STO GET   LAKE113.0
RTE TATM  RECHR 1.
STO HYD   11RR  2.0
STM MASS  AREAR 0.4     1.
LOS NONE
UHG CLRK0.5     0.75    -3.2    1.0
STO ADDR  11R   2.0
STO ADDR  ACUMOR1.0
STO GET   ACUMOR1.0
RTE TATM  CONVRT6.
STO GET   ACUMOR1.0
RTE TATM  RECH3 4.
STO HYD   ORPS  1.0
RUN TIME        0.0     30.
STM MASS  AREAS 4.8     1.
LOS NONE
UHG CLRK2.              -3.2    1.0
RUN TIME        1.0     0.0
STO ADD   ACUMS 1.0
RUN TIME        0.0     30.
STM MASS  LAKE9 8.5     1.
LOS NONE
UHG CLRK5.      2.5     -2.5    1.0
STO HYD   LAKE9 3.0
RUN PRNT        25.
RTE TATM  CONVRT12.
RUN PRNT        -25.
STO GET   LAKE9 3.0
RTE TATM  RECHT110.
STO HYD   9RT1  2.0
STM MASS  AREAT18.9     1.
LOS NONE
UHG CLRK3.0     1.7     -2.5    1.0
STO ADDR  ACUMT12.0
RTE TATM  RECHT22.
STO ADD   T1RT2 2.0
STM MASS  AREAT22.5     1.
LOS NONE
UHG CLRK1.5     1.0     -2.5    1.0
STO ADDR  ACUMT22.0
RTE TATM  RECHT37.
STO ADD   T2RT3 2.0
STM MASS  AREAT38.1     1.
LOS NONE
UHG CLRK3.              -3.2    1.0
STO ADDR  ACUMT32.0
RUN TIME        1.0     0.0
STO ADDR  ACUMST1.0
RTE TATM  RECHZ 13.
STO ADD   SRZ   1.0
STM MASS  AREAZ 24.6    1.
```

TONG CLAE

DSCUHS1 pg 5

```
LOS NONE
UHG CLRK8.              -3.2    1.0
STO ADD   ACUMZ 1.0
RUN TIME        0.0     30.
STM MASS  LAKEU 3.8     1.
LOS NONE.
UHG CLRK1.      1.5     -2.5    1.0
RTE PULS  LAKEU         3930.   16.
0.      40.     200.    700.    1500.   2800.   3930.   4095.   4273.   4456.
4646.   4844.   5050.   5408.   5791.   6200.
0.      0.      0.      0.      0.      0.      0.      337.    1214.   2244.
3453.   4834.   6345.   7993.   12395.  22264.
RTE TATM  RECHV 4.
STO HYD   URV   2.0
STM MASS  AREAV 1.5     1.
LOS NONE
UHG CLRK1.5     2.25    -3.2    1.0
STO ADDR  ACUMV 2.0
RUN TIME        1.0     0.0
STO HYD   ACUMV 2.0
STM MASS  LAKE1216.9    1.
LOS NONE
UHG CLRK5.              -3.0    1.0
STO HYD   LAKE123.0
RUN PRNT        25.
RTE TATM  CONVRT6.
RUN PRNT        -25.
STO GET   LAKE123.0
STO ADDR  ACUM122.0
RTE TATM  RECHW 6.
STO HYD   12RW  2.0
STM MASS  AREAW 10.8    1.
LOS NONE
UHG CLRK4.5             -3.0    1.0
STO ADD   ACUMW 2.0
RUN TIME        0.0     30.
STM MASS  LAKE135.2     1.
LOS NONE
UHG CLRK3.5     5.25    -2.2    1.0
STO HYD   LAKE133.0
RUN PRNT        25.
RTE TATM  CONVRT12.
RUN PRNT        -25.
STO GET   LAKE133.0
RTE TATM  RECHX 5.
RUN TIME        1.0
STO ADD   13RW  2.0
STM MASS  AREAX 3.5     1.
LOS NONE
UHG CLRK2.5     3.75    -2.7    1.0
STO ADDR  ACUMXW2.0
RTE TATM  RECHY 10.
STO ADD   XWRY  2.0
STM MASS  AREAY 17.2    1.
LOS NONE
```

DSCUHG1 #6

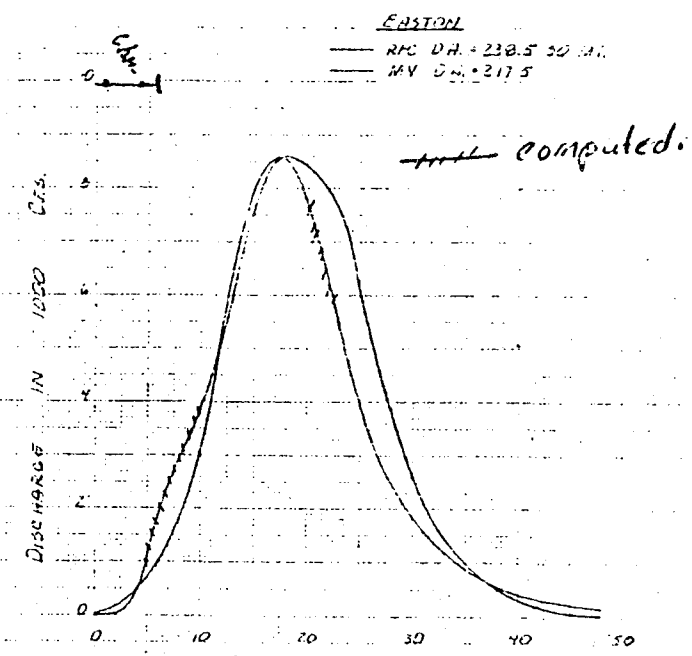
```
UHG CLRK6.5             -3.5    1.0
STO ADDR  ACUMY 2.0
STO ADDR  ACUMZ1.0
RTE TATM  RECHAA2.
STO HYD   YIRAA 1.0
RUN TIME        0.0     30.
STM MASS  AREAAA2.9     1.
LOS NONE
UHG CLRK2.              -3.5    1.0
RUN TIME        1.0     0.0
STO ADDR  BASIN 1.0
RTE TATM  CONVRT6.
RUN STOP
-END OF INFORMATION-
```

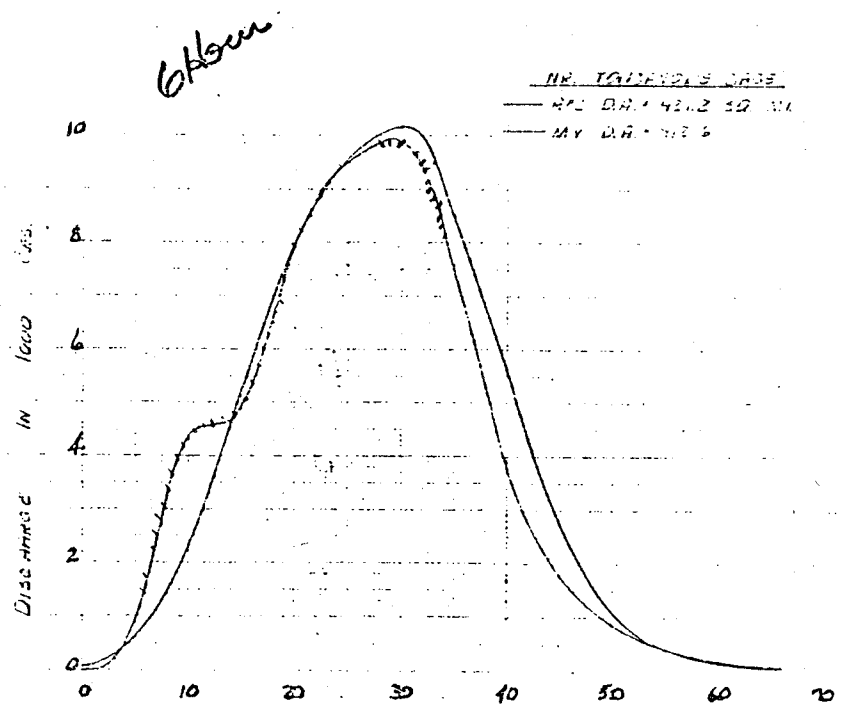
530 #5 40 SMA 1

DSCUHG1 #7

STRANGER CREEK STUDY

RECONSTITUTION OF 6-HR. UNIT HYDROGRAPHS PROVIDED BY THE KANSAS CITY RIVER FORECAST CENTER (RFC)





ILLUDAS

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Illinois State Water Survey
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ILLUDAS --- The Illinois Urban Drainage Area Simulator

Section 1 --- OVERVIEW

The ILLUDAS model is presented as an objective method for the hydrologic design of storm drainage systems in urban areas. The model may also be used for the hydrologic analysis of existing drainage systems. ILLUDAS uses either an observed or a theoretical rainfall pattern which is assumed to be uniformly distributed over the basin as the primary input. The basin is divided into sub-basins, one for each design point in the basin. Surface runoff hydrographs are produced for both the contributing paved and grassed areas in each sub-basin. These hydrographs are combined and routed downstream from one design point to the next until the basin outlet is reached. Pipe sizes are determined at each design point if so required. Detention storage may be included in any sub-basin either in the form of a volume to be maximized or as an actual operating rating curve.

The model was first introduced in 1974. It was the product of research performed at the Illinois State Water Survey which was aimed at finding a more reliable design tool for storm sewer design than the much-used Rational Formula. Since its introduction, ILLUDAS has undergone several updates and has gathered several hundred users both in this country and abroad. The latest update to the model was its implementation on the IBM PC. The algorithms for trial-and-error solutions and routing have been changed in the micro-computer version so as to yield smaller sums of errors in the routing techniques than are created by its mainframe predecessor.

The micro-computer version of the ILLUDAS package includes a data set tool for creation and modification of ILLUDAS input data sets, a data set translation utility to convert mainframe data sets to the micro format, and a post-processing graphics program that allows on-screen and/or printed hydrographs from simulation output files.

The mainframe version of the ILLUDAS package may be obtained directly from the Illinois State Water Survey at a cost of $200.00 for individuals and businesses outside of Illinois, and for $100.00 for Illinois businesses and residents.

The micro-computer version of the ILLUDAS package may be obtained directly from the Illinois State Water Survey at a cost of $200.00 for Illinois businesses and residents. For individuals and businesses outside of Illinois, the ILUDRAIN (ILLUDAS) package may be obtained from C.E. Software, at a cost of $375.00.

Correspondence or phone calls pertaining to ILLUDAS / ILUDRAIN may be addressed to either Michael L. Terstriep or Douglas C. Noel at the following addresses:

Mainframe ILLUDAS (all), or
Micro-ILLUDAS (IL only)

Illinois State Water Survey
P.O. Box 5050, Station A
Champaign, IL 61820

Phone: *(0800-1700 Central)*
MLT: (217) 333-4959
DCN: (217) 333-0545

Micro-ILLUDAS (ILUDRAIN)
(outside Illinois)

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REFERENCES

Terstriep, Michael L., and John B. Stall, The Illinois Urban Drainage Area Simulator, ILLUDAS, Illinois State Water Survey Bulletin 58, 90 p., 1974.

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Part 2. Theory and Development

Paved-Area Runoff

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

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

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

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

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

The second step is to apply Manning's equation to compute the velocity of flow in the gutters. With these velocities, travel times are computed at various points on the paved area in each sub-basin. These travel times are plotted on the paved area, and by connecting points of equal travel time a series of isochrones are drawn on the paved area, as shown in figure 1a. The directly connected paved area between these isochrones is measured and designated areas PA1, PA2, PA3, PA4, and PA5. These various areas are accumulated and plotted against travel time to the inlet as shown in figure 1b. This time-area curve shows the amount of paved area within the sub-basin that is contributing water at the storm drain inlet at any time after the beginning of runoff. In the computer program described later, the time-area curve was assumed to be a straight line connecting the origin and the end-point of the curve. The end-point, as illustrated in figure 1b, represents the travel time from the farthest point of the directly connected paved area, and the total amount of the directly connected paved area.

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

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

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

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

Grassed-Area Runoff

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

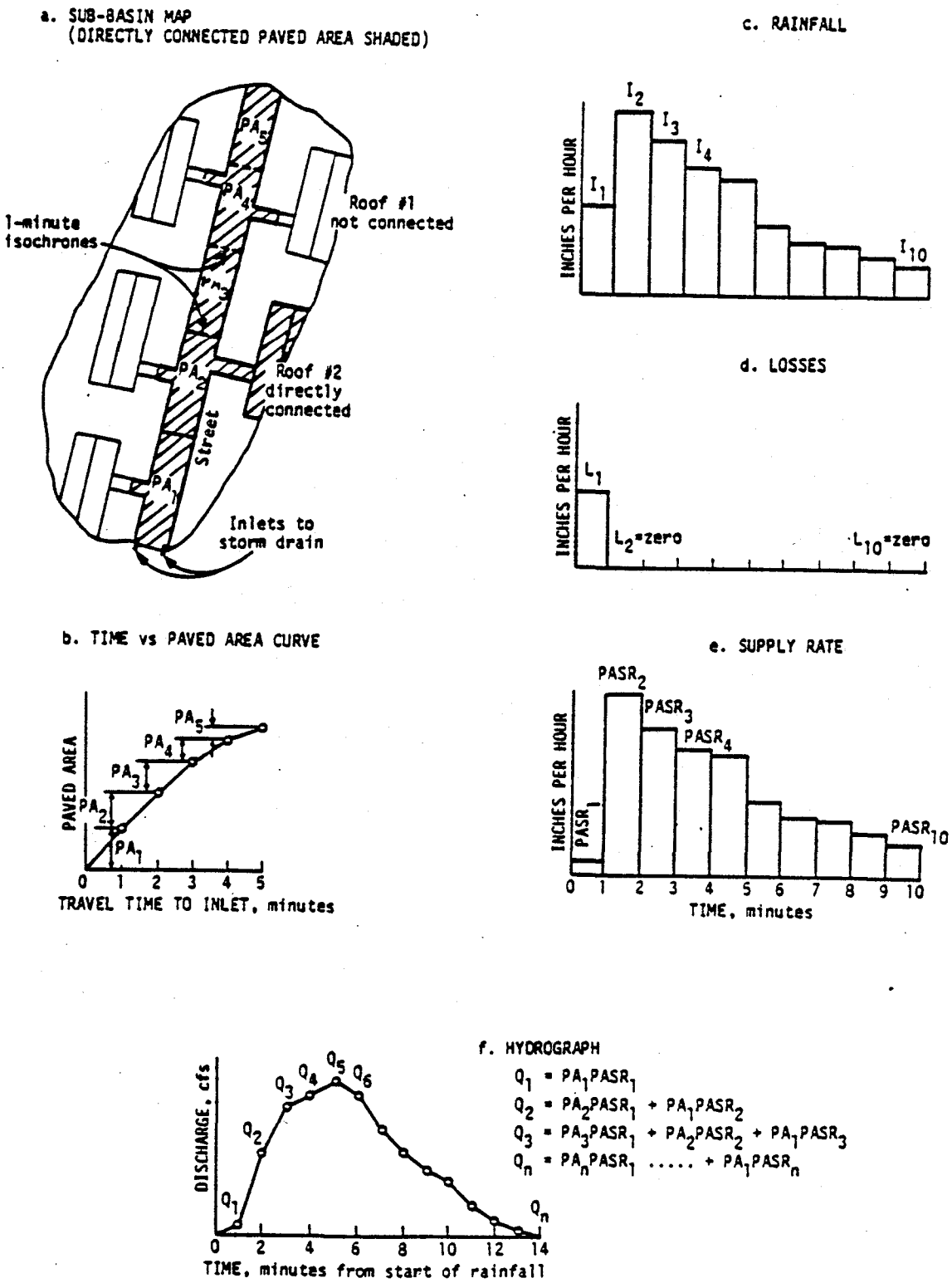


Figure 1. Elements in the development of the paved-area hydrograph

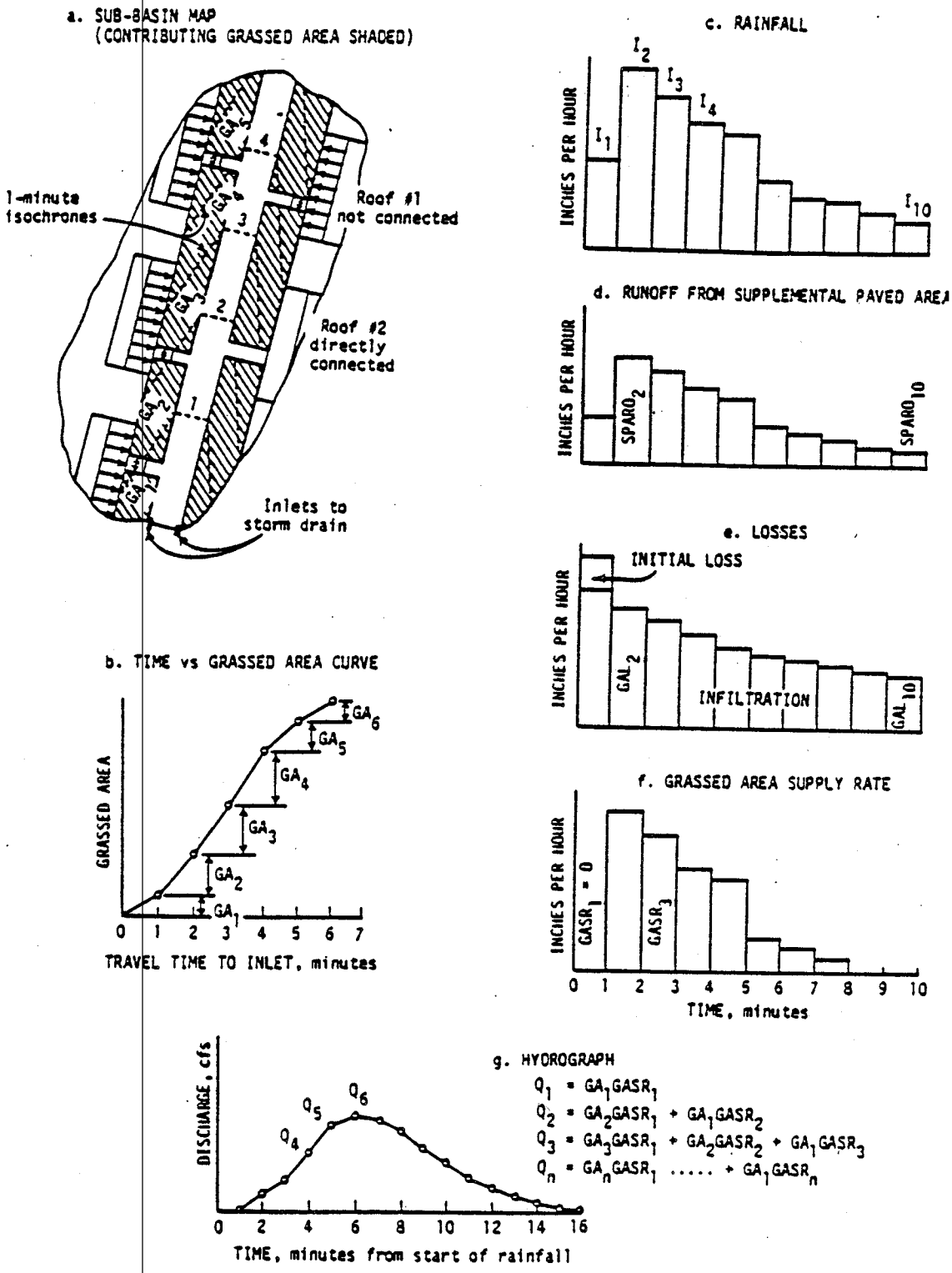


Figure 2. Elements in the development of the grassed-area hydrograph

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

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

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

where

q_e = discharge of overland flow, in cfs per foot of width, at equilibrium
I = supply rate in inches per hour assumed to be 1.0 in this study
L = length of overland flow in feet

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

where

t_e = time of equilibrium in minutes

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

S = surface slope in feet per foot
c = coefficient having a value of 0.046 for bluegrass turf

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

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

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

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

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

where

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

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

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

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

Infiltration

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

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

Water Storage in Soil

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

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

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

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

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

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

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

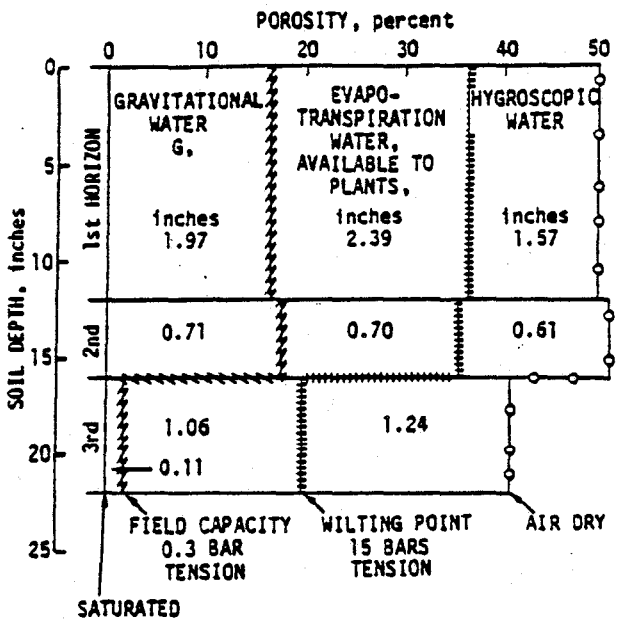


Figure 3. Water storages available within Alexis silt loam

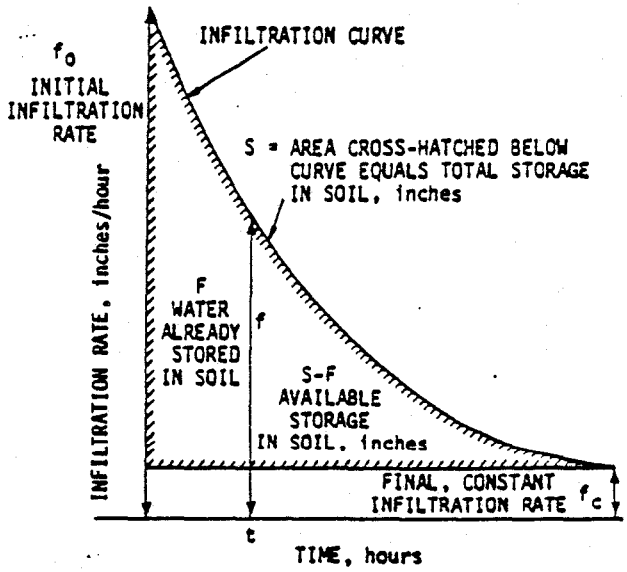


Figure 4. Diagram of infiltration curve and infiltration rates as related to storage in soil

Computed Infiltration

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

$$f = a(S - F)^n + f_c \qquad (4)$$

where

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

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

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

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

Table 2. Computation of Infiltration Curve for Alexis Silt Loam

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

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

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

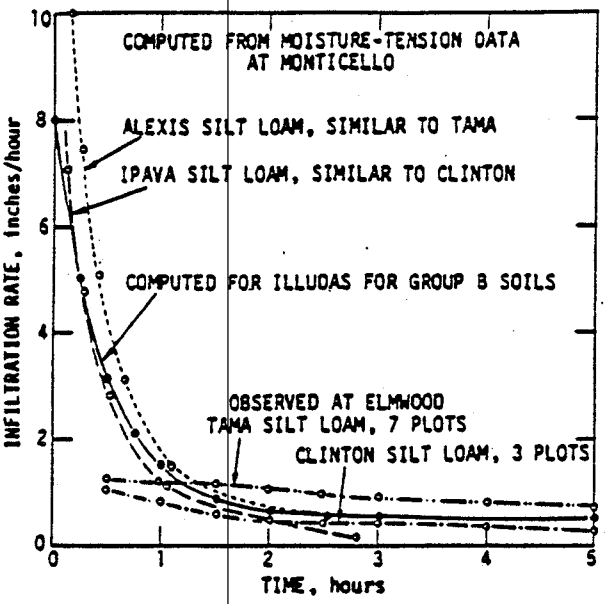


Figure 5. Infiltration curves for bluegrass turf

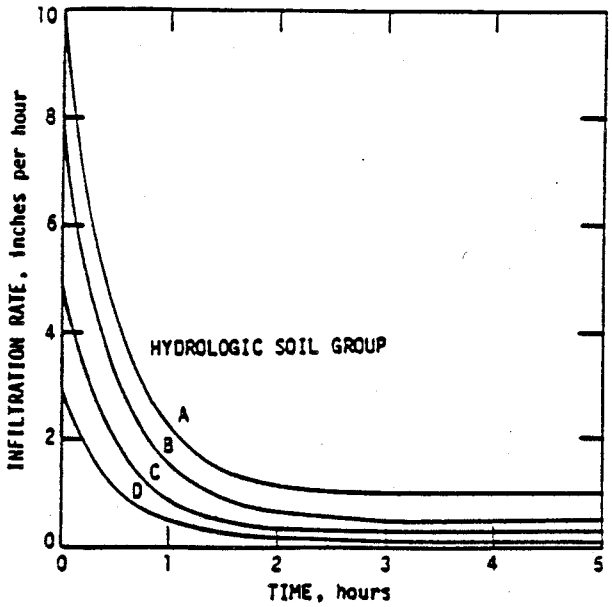


Figure 6. Standard infiltration curves for bluegrass turf used in ILLUDAS for soils of four hydrologic groups

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

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

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

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

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

where

f_0 = initial infiltration rate, inches per hour
e = base of natural logs
k = a shape factor selected as $k = 2$
t = time from start of rainfall

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

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

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

Item	Value			
Hydrologic soil group				
USDA designation	A	B	C	D
ILLUDAS designation	1	2	3	4
Final constant infiltration rate, f_c, *inches per hour*	1.0	0.50	0.25	0.10
Initial infiltration rate, f_0, *inches per hour*	10	8	5	3
Depression storage, *inches*	0.20	0.20	0.20	0.20
Shape factor, k, of infiltration curve	2	2	2	2
Available storage capacity, S, in soil mantle, *inches*, for four antecedent conditions				
Bone dry, condition 1	6	4	3	2
Rather dry, condition 2	4	2.5	2	1.3
Rather wet, condition 3	2	1	1	0.5
Saturated, condition 4	0	0	0	0
Infiltration accumulated, F, in soil mantle, *inches*, at start of rainfall				
Bone dry, condition 1	0	0	0	0
Rather dry, condition 2	2	1.5	1	0.7
Rather wet, condition 3	4	3	2	1.5
Saturated, condition 4	6	4	3	2

f_c and f_0 were selected arbitrarily. The shape factor k was selected as equal to 2 to provide the shape best reflecting natural conditions as shown in figure 5. Selected representative values for the available storage capacity, S, for four antecedent moisture conditions were used to provide values of infiltration accumulated, F, in inches for each condition.

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

Table 4. Antecedent Moisture Conditions for Bluegrass Lawns

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

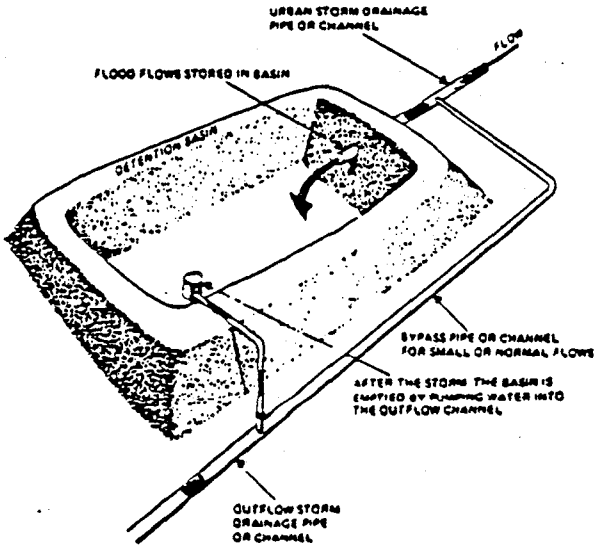


Figure 8. Schematic view of a detention basin for an urban basin

Flood Water Detention Basins

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

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

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

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

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

ROUTING OPTIONS FOR ILLUDAS

Included in this new version of ILLUDAS are two reach routing techniques which are able to route through circular or rectangular conduits, or trapezoidal open channels. The two types of routing include simple time shift routing and hydrologic routing utilizing an implicit solution of the continuity equation. In the implicit method, a linear approximation of the continuity equation is used:

$$\frac{\Delta S}{\Delta t} = \frac{I1+I2}{2} - \frac{O1+O2}{2} = \frac{L}{\Delta t}\left[\frac{FLAREA(I2)+FLAREA(O2)}{2} - \frac{FLAREA(I1)+FLAREA(O1)}{2}\right] \quad (1)$$

where I1 and O1 are the initial inflow and outflow; I2 and O2 are the final inflow and outflow; FLAREA is the flow area function in which the Kinematic wave assumption that $S_f = S_o$ and Manning's equation are used, where S_f is the friction slope and S_o is the reach slope; and L is the reach length. Equation (1) can be rearranged to obtain:

$$I1+I2-O1-O2 = [FLAREA(I2)+FLAREA(O2)-FLAREA(I1)-FLAREA(O1)]\frac{L}{\Delta t} \quad (2)$$

$$O2+\frac{L}{\Delta t}FLAREA(O2) = I1+I2-O1+[FLAREA(I1)+FLAREA(O1)-FLAREA(I2)]\frac{L}{\Delta t} \quad (3)$$

Equation (3) must be solved for O2 and thus requires an implicit solution.

The flow area function (FLAREA) is determined for each cross-section by a dimensionless flow area versus dimensionless discharge tabular look-up within the program. This non-dimensionalized function is computed for the channel geometry. For the case of trapezoidal open channels, a Newton-Raphson implicit solution of the Manning's equation must be performed to develop this dimensionless function. Parabolic interpolation between two percent intervals of the dimensionless area is performed by a function subroutine.

Equation (3) is solved by an accelerated bracketing technique by obtaining successive estimates of O2 by the equation

$$O2^1 = O2^0 + F^1/\alpha^1 \quad (4)$$

where $O2^1$ is the new estimate of O2 (the solution) and $O2^0$ is the initial estimate; also $F = I1+I2-O1-O2+[FLAREA(I1)+FLAREA(O1)-FLAREA(O2)-FLAREA(I2)]$

$\frac{L}{\Delta t}$ is the zeroed continuity equation from equation (3) and equals zero at the solution for O2 and α^1 is the acceleration/deceleration factor.

In the implicit solution to equation (3) certain conditions can cause unreasonable solutions which must be corrected.

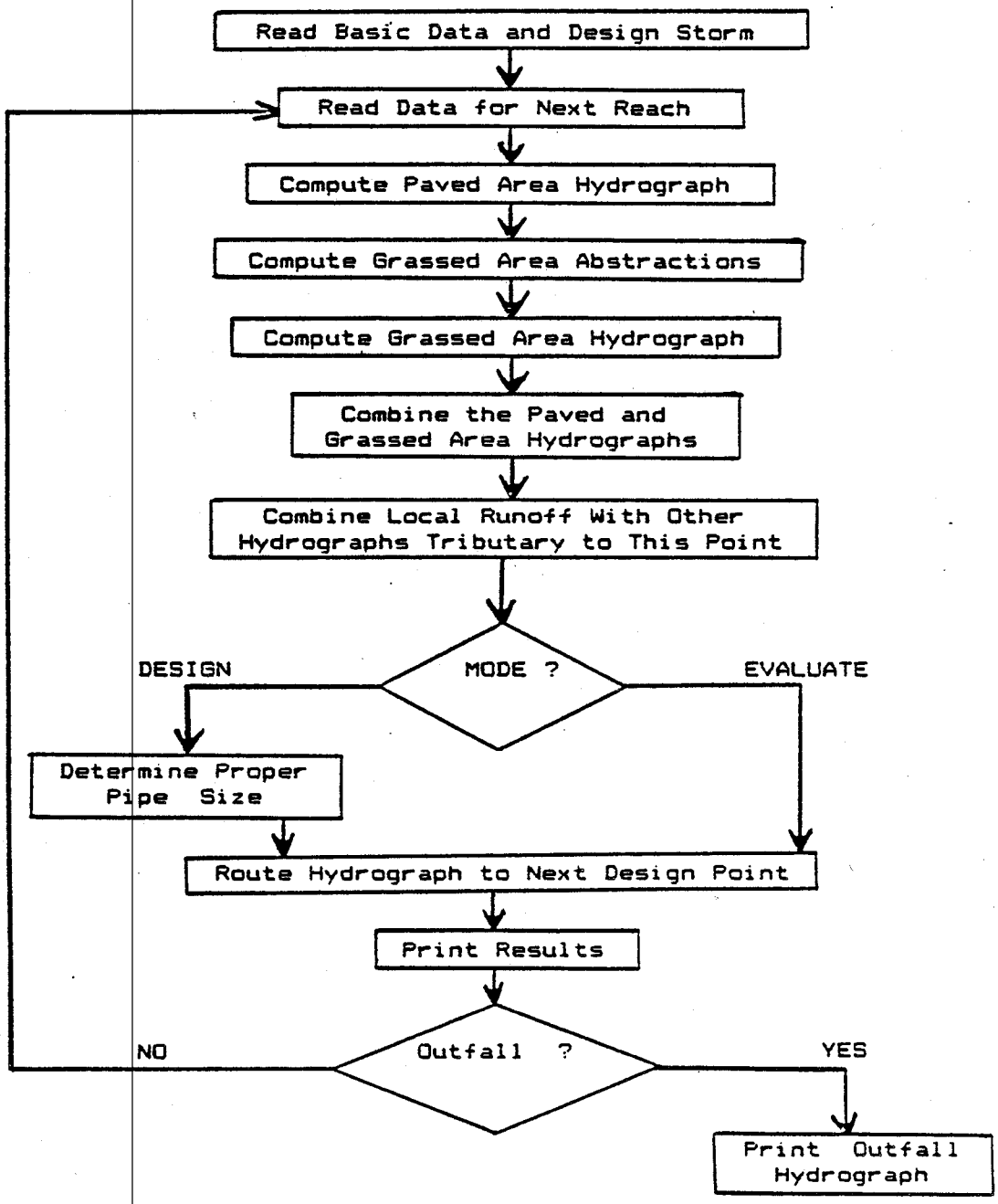
These include:

1) When O2 (solution to equation (3)) is greater than just-full pipe flow and I2 is equal to just-full pipe flow then continuity can only be maintained by increasing upstream detention storage thus decreasing the final inflow discharge (I2).
2) When O2 is less than zero then O2 must be set equal to zero, with a subsequent change in routed hydrograph volume.

The time shift routing of the hydrograph is performed for both the time shift and the hydrologic routing techniques. For time shift routing it is used to determine final outflows (O2) directly. While in hydrologic routing the time shift routing is used to obtain the initial estimates of O2 to be used in the implicit solution. Time shifting of the inflow hydrograph is performed by computing the velocity from the inflow hydrograph peak (after discharge limiting due to pipe surcharging and/or user-supplied discharge limiting for both pipes and open channels). The inflow hydrograph is interpolated at time increments equal to the time of travel of the inflow hydrograph peak discharge through the reach. This interpolation eliminates instabilities in the hydrologic routing and discharge "lumping" doesn't occur for large user-specified simulation time increments in the time shift routing. This interpolation is done unless the computed travel time for the reach of interest is greater than or equal to the user-specified time increment, in which case the user-specified time increment is used. After routing, the outflow hydrograph is back-interpolated to the user-specified time increment for subsequent computations by the program.

Section 3 --- PROGRAM STRUCTURE and IMPLEMENTATION

The basic processing hierarchy utilized by ILLUDAS can be seen in the accompanying flow chart.



Applying ILLUDAS

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

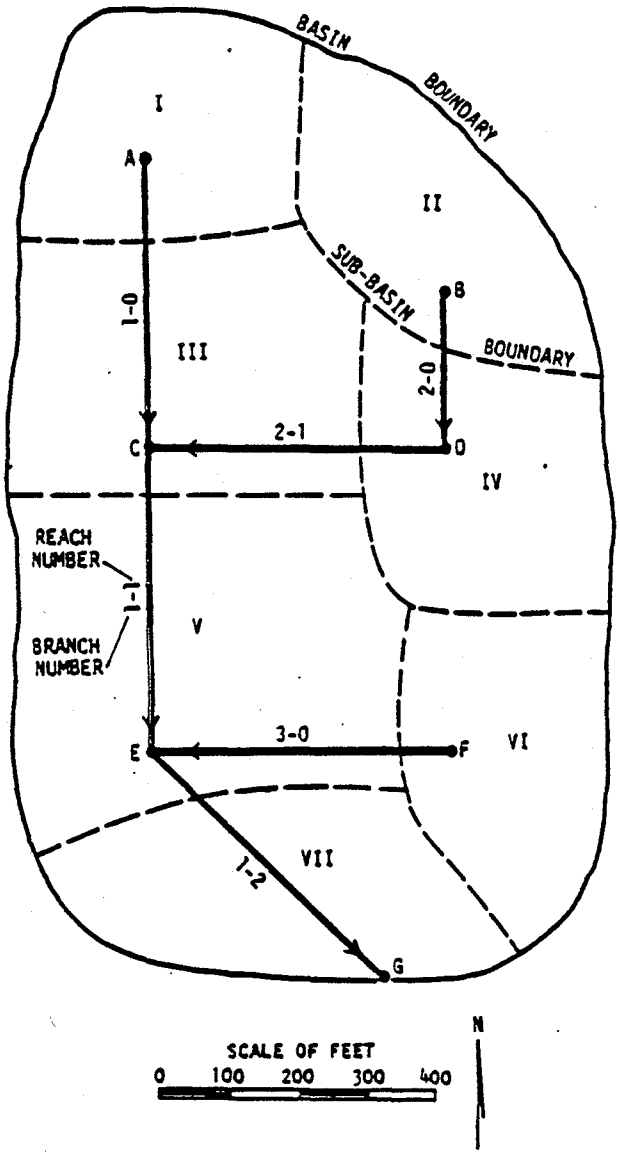


Figure 76. Sample basin showing sub-basins and reach numbering system

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

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

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

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

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

Table 29. Basic Data for Sample Basin

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

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

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

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

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

Determination of Parameters

Most of the parameters used in ILLUDAS are determined in a very straight-forward manner, namely by the inspection of maps and/or aerial photographs. Some parameters, such as initial abstractions and the time of concentration for local surface runoff require the use of either standard values or procedures, or some preliminary calculations using your favorite surface runoff algorithm.

ILLUDAS allows surface cover types to be input either as actual acerages, which will require extensive map evaluation, or as percentages of the local sub-basin areas, which can be a bit more general.

If a new drainage system is being designed, the expected pipe lengths and slopes between manholes, and the network connectivity are about all the data you can expect to supply for the drainage system itself. If you are evaluating an existing drainage system, you should describe the system using the actual field values collected. In some cases, such as multiple pipes or eliptical pipes, the determination of either an equivalent circular or equivalent rectangular section may be required.

Unless observed data are available, no calibration of the model is possible. If observed data are available for a nearby basin of similiar land use and hydraulic characteristics, some calibration based on the simulation of such a basin may be helpful. Otherwise, we suggest that you make several simulation runs in which you might vary the initial moisture for pervious areas and possibly vary the initial abstraction depths so that you tend to create an envelope for design hydrograph peaks from which to select pipe sizes or storage requirements.

If Observed data are available when evaluating an existing drainage basin, we suggest the following procedure:

I. Calibrate storms yielding *paved runoff* only
   A. Calibrate *hydrograph volume*
      1. vary depth of *paved abstraction*
      2. vary acreage of Contributing Paved Area *
   B. Calibrate *hydrograph shape and timing*
      1. vary *paved time of concentration*
      2. vary *Manning's roughness*

II. Calibrate storms yielding *both paved and grassed runoff*
   A. Calibrate *hydrograph volume*
      1. vary depth of *grassed abstraction*
      2. vary acreage of Supplemental *Paved Area* *
      3. vary acreage of *Contributing Grassed Area* *
      4. vary *Antecedent Moisture*
   B. Calibrate *hydrograph shape* by *grassed TOC*

The model also allows for the uniform increase or decrease of event rainfall ordinates once the above parameters have been determined.

Section 4 --- EXAMPLES

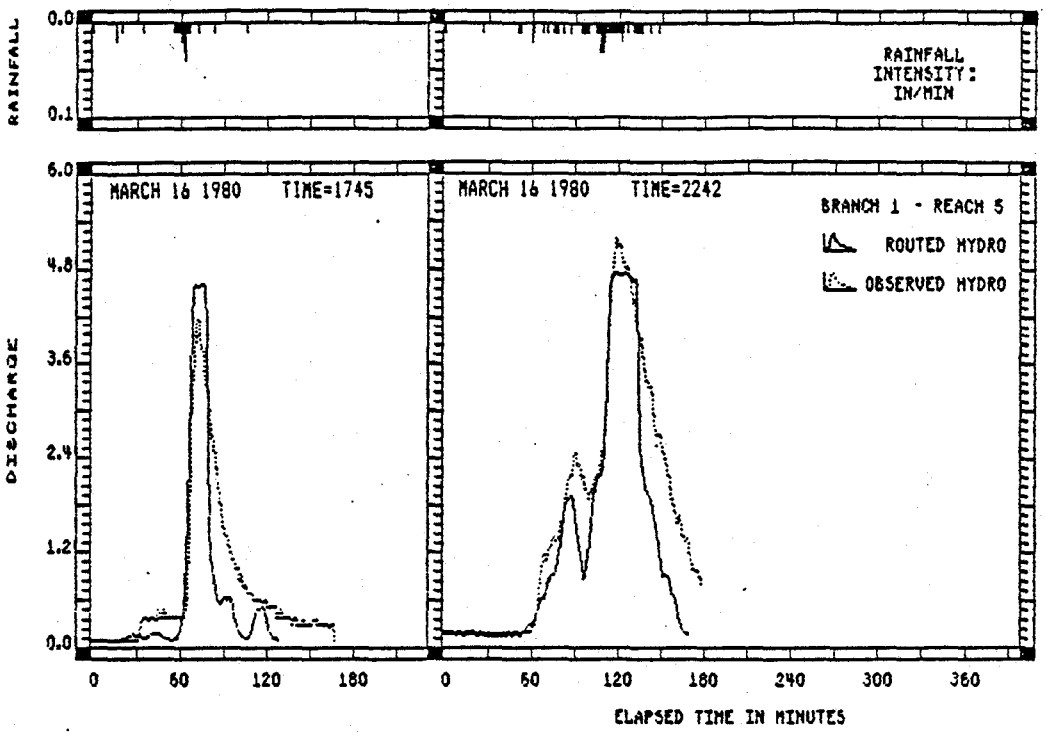
The examples presented on the following pages are for a basin studied during Phase One of the Champaign-Urbana (IL) part of the Nationwide Urban Runoff Program, sponsored by the United States Environmental Protection Agency. Both the rainfall and runoff data were collected by the staff of the Illinois State Water Survey. A tipping bucket raingage mounted atop an eight foot high steel pipe and an asymmetric flume installed in the storm sewer were used for the data collection. The information was electronically processed and written to diskette during the storm events.

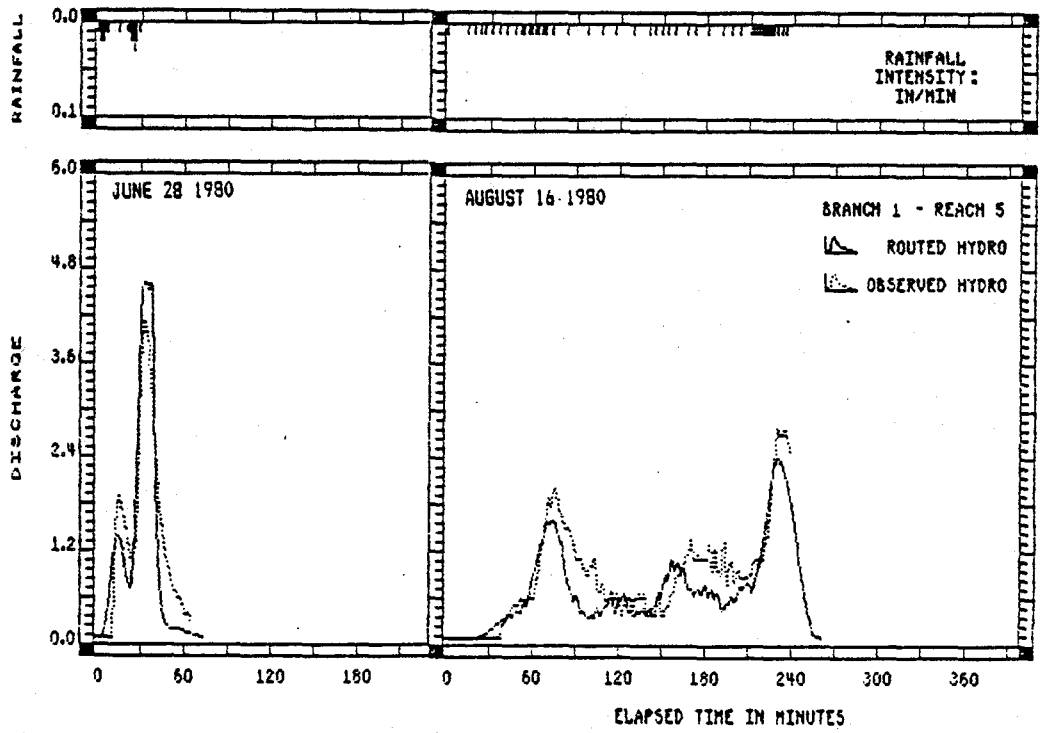
These data have been coded and run on the micro version of ILLUDAS. The plots shown were produced using the micro package.

The two events of March 16, 1980, involved rainfall on existing snowpack. Although the model does not simulate snowmelt, and therfore cannot properly generate the total runoff volume of such an event, these events were included so as to show how well we can simulate the peaks, shape, and timing of even these events.

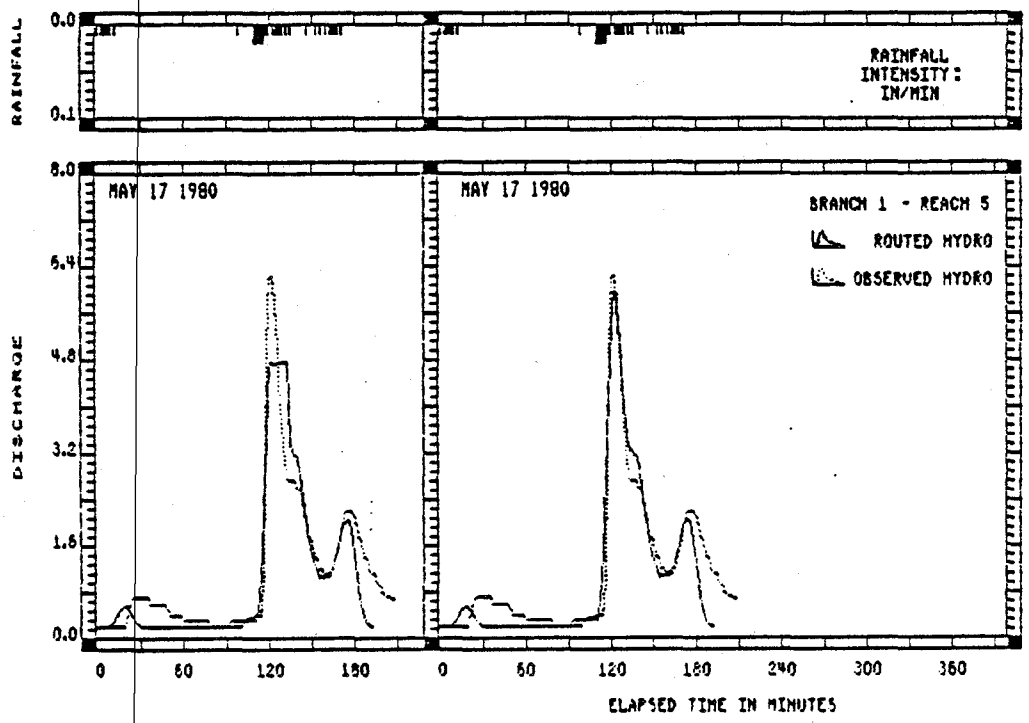
During the course of the project, it was noticed that at the inlet end of the reach immediately upstream of the outfall reach there is a considerable volume of storage generated. This pipe inlet receives discharge from three pipes at the manhole, all of which have significantly larger slopes than the pipe in question.

HYDROLOGIC MODELING FOR FLOOD HAZARDS • ILLUDAS

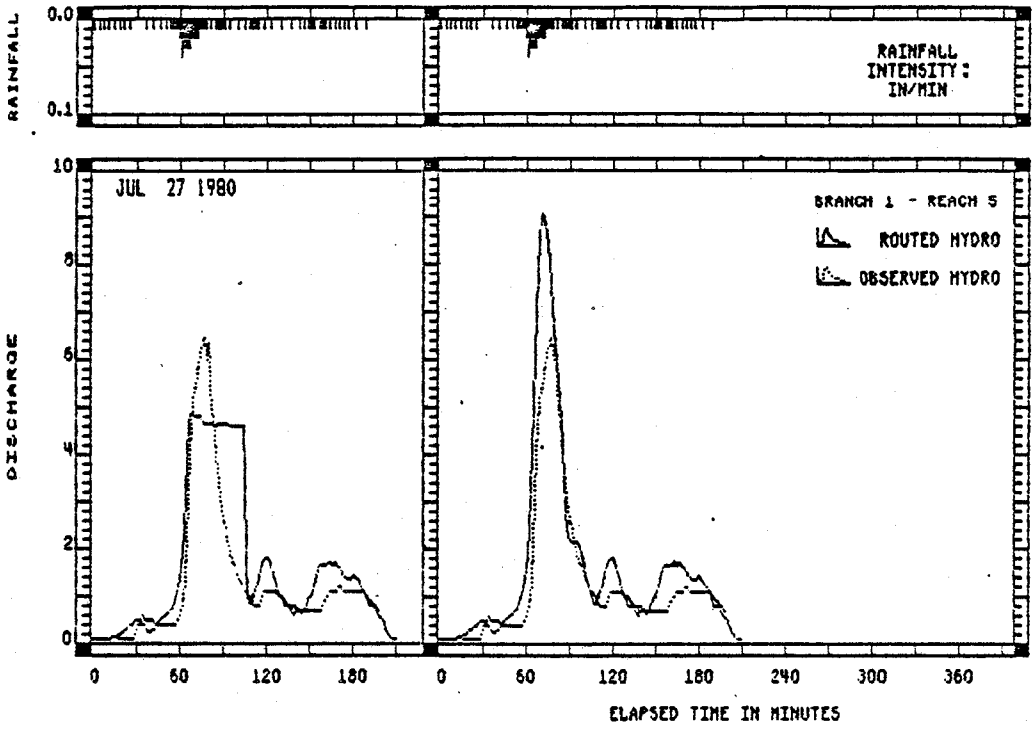


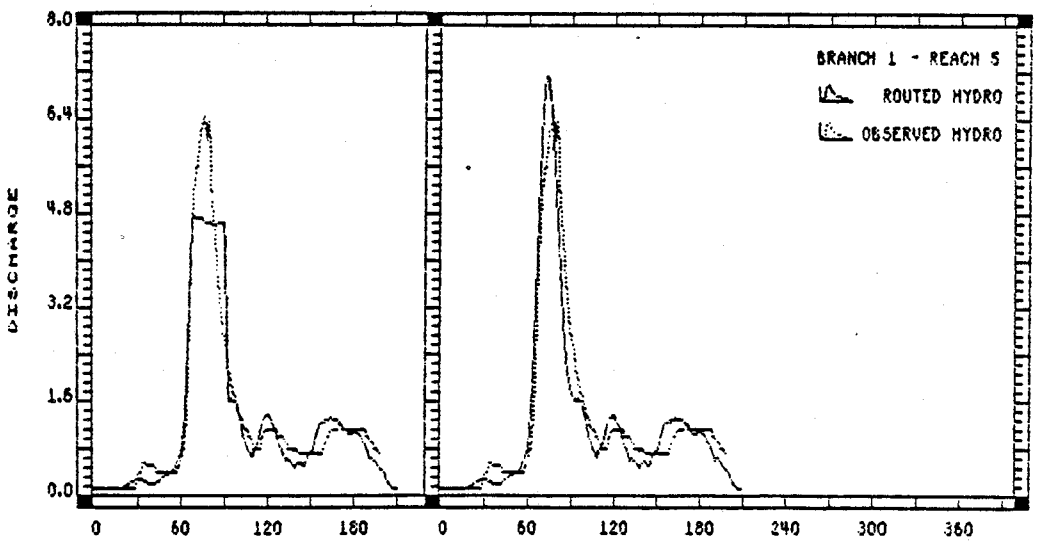


Inspection of the observed hydrographs for events with rainfall intensities of 0.4 inches/hour or greater showed that pressure flow developed under these conditions almost immediately (usually within 3 minutes). With this in mind, the event of May 17, 1980, was simulated both with the drainage network completely as defined by the field survey, and also with the above mentioned problem reach in design mode. The resulting plots are shown here. The first plot shows the effect of the limiting pipe capacity on the simulated routed hydrograph, which is essentially to "lop off" the peak of the hydrograph. The second plot shows that because pressure flow occurs so quickly, our assumption that we could allow the model to size a pipe large enough to pass the design peak discharge was, in this case, a fairly legitimate way to approximate the hydraulic bottleneck in this basin. This would not necessarily have been true if any or all of the incoming pipes at the manhole had had significantly lower slopes.



The plots on this page show the same solution to the hydraulic bottleneck. Notice however that the volume is grossly over-simulated in the upper two plots. A rainfall reduction of 25 percent yielded the plots at the bottom of the page. Winds related to this storm may have shook our raingage, generating more tips than the rainfall would have produced (100% paved area runoff).





Example Uses of ILLUDAS

Teaching ...	University classes Continuing education
New Designs ...	Residential drainage Commercial drainage Airport drainage Highway drainage
Evaluation ...	Hydrologic research Evaluate existing drainage Future planning / zoning Input to other models Detention storage requirements
Re-design ...	Airport drainage Petroleum storage area Residential drainage Commercial drainage Highway drainage

Use of ILLUDAS for Urban Flood Abatement Studies

1. Identify problem basin(s)
2. Site raingage(s) and flow meter(s)
3. Collect data on rainfall depth-durations and local flooding (streets, basements, etc.)
4. Develope relationship between rainfall depth-durations and flood potential
5. Develope relationship between rainfall depth-durations and ILLUDAS-generated surcharges for observed events
6. Using information developed in 4. & 5., run ILLUDAS to test alternative solutions to urban flooding problem

Section 5 --- ASSESSMENT

Common Pitfalls

Basin size

Although this model is capable of modeling an almost infinite number of reaches, in order that the assumption of uniform rainfall be realistic, the model's use should be limited to basins with less than 500 acres.

Basin cover

One of the most abused features of this model is it's ability to model runoff from grassed areas. Even though this feature can make a significant difference in the runoff volume for very intense rainfalls, it was developed for only one instance, that being *to simulate runoff from urban lawns and parkways.* Many users have attempted to use it to model wetlands or to estimate the runoff from a design storm on an undeveloped tract of land (to generate retention / detention volumes required by zoning). The algorithms are not applicable to these types of uses, and any such use would probably generate a biased parameter set.

Basin slope

The model is intended for use in areas with mild slopes. The channel and paved surface routing are based on the Manning Equation and an assumption of sub-critical flow. Steep slopes will generate hydrographs which underestimate both peaks and peak times. On the other extreme, too mild a slope (i.e. coastal plains) should require a model which considers outlet control.

Data

If modeling an existing drainage network using observed data, keep in mind the reliabilty of the data (source, when collected, where collected, how collected, method of reduction, etc.) while calibrating the model.

Level of detail

Another common problem with data is the level of detail needed for a good simulation. If you are using five minute rainfall data, there is no advantage to detailing your drainage system so as to produce one minute travel times in the reaches. The user should break up reaches if there is a significant change in slope or if the reach geometry is changing (pipe diameters increasing, etc.), but you do not need to model every set of inlets as being the start of a new reach.

Verification

ILLUDAS is intended to be used as a tool to provide users with *estimates* of conditions in an existing drainage network or to provide the *hydrologic* design of a pipe size(s). Too many users have blind faith in model results. When designing new drainage, hydraulic losses due to headwalls, transitions, and manholes, as well as silting, scouring and possible outlet control should be considered, even though they may not change your design.

Trap efficiency

ILLUDAS assumes that there is a 100 percent trap efficiency at all surface inlets in a sub-basin. Although this will not always be a terrible assumption, some over-estimation of surcharged volumes may result because for very intense storms there may be a considerable volume of surface flow which runs past an inlet and on down the surface flow path to the next inlet. In many cases, this may be seen as a compensating error for all but the headwater reaches of the drainage network.

Surcharged reaches

As seen in the Nationwide Urban Runoff Program examples, typical model runs that don't provide gimmicks for surcharged conditions produce severely attenuated hydrograph peaks when inflow exceeds reach capacity. In cases where the branches discharging to such a node have mild slopes, the user must consider the possibilty that even though pressure flow

might be negligible, the stored volume at the node may reach a stage where it begins to flow down the pavement to the next node.

When surcharged conditions exist when the upstream pipe slopes are mild, the user must determine whether outlet control is being imposed on these pipes or whether potential pressure flow away from this node is preventing the developement of outlet control.

Possible Improvements

Update grassed area infiltration / runoff algorithm, to allow wider range of grassed area applications.

Add a user-friendly interface to other models, allowing local surface hydrograph import from or export to other urban drainage models.

Replace internal time of concentration algorithms, the functions for determining paved and grassed surface entry times could be improved by replacing both these algorithms and the surface hydrograph convolution technique with a modified kinematic wave procedure.

Allow less than 100 % trap efficiencies.

REVISED SWMM

Jody Farhat
Hydraulic Engineer
Omaha District
Corps of Engineers
Department of the Army
6014 U.S. Post Office
Omaha, Nebraska 68102-4978

Revised Stormwater Management Model (SWMM)

I. Overview

A copy of the SWMM source program for CDC-Cybernet computer system, a sample input and output and a rough users manual is available by written request to:

Mr. William Todsen
Chief, Engineering Division
U.S. Army Corps of Engineers
Missouri River Division
P.O. Box 103 Downtown Station
Omaha, Nebraska 68101

Please specify the tape format including the number of tracks, density and tape label required. There is a $200.00 charge.

II. Model Theory and Structure

A. Theoretical Development.

The SWMM model simulates the runoff phenomena of a drainage basin for any given rainfall pattern. Since the only information which can reasonably be expected to be available is climatological and physical watershed characteristics, such as size, ground slope, and types of cover, SWMM represents the drainage basin by an aggregate of idealized subcatchments and gutters that are common denominators of the drainage plan and are readily quantifiable. Flexibility of the program to be adopted to any prototype conditions was of great concern and was considered in the model formulation and development.

Hydrographs are calculated by a step-by-step accounting of rainfall, infiltration, detention, overland flow, and gutter flow. The drainage basin is conceptually represented by a network of hydraulic elements, i.e., subcatchments, gutters, and pipes. Hydraulic properties of each element are then characterized by various parameters, such as size, slope, and roughness coefficient.

As was originally envisioned, a subcatchment is rectangular in shape with reasonably uniform watershed characteristics, such as surface cover and ground slope. The geometry of a subcatchment is defined by the area, width, and ground slope. The type of ground cover determines the detention depth requirements, the roughness factor (such as Manning's coefficient), and the coefficients describing the infiltration loss by Horton's exponential function (Ref. 5).

The subcatchments need not be the same size and the irregular shape can be approximated by an equivalent rectangle by computing the mean width. In principle, a series of subcatchments can be designed to cover the entire drainage basin.

The subcatchments form an aggregated system by gutters and pipes specifying the connectivity of flow. Hydraulically, gutters and pipes are described by the Manning's coefficient, length, invert slope, and geometric description of the shape. The latter may include the bottom width and side slopes for rectangles, trapezoids, and triangles and the diameter for circular pipes. Missouri River Division revisions allow the use of an overbank floodway section in conjunction with gutters and pipes. Diversions from one gutter to another, detention dams and inflow hydrographs may also be used.

Figure 1 presents a typical but idealized drainage system. As shown, subcatchment 2 drains into gutter 1. Both gutter 1 and subcatchment 1 discharge into gutter 2. Several subcatchments and gutters can be connected to a given gutter, depending on the topographic condition.

B. Relevant Equations

The basic equations used in the SWMM model are summarized below.

1. Mannings Equation:

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

for determining outflow rates from subcatchment areas and from gutters.

2. Continuity Equation:

for determining water depths in subcatchment areas and gutters.

3. Horton's Exponential Function:

$$I_t = fo + (fi - fo)e^{-\alpha t}$$

for computing infiltration.

C. Coefficients

Coefficients used in the SWMM model are summarized below.

1. Subcatchment physical characteristics

a. Area
b. Lateral outflow length
c. Percent impervious
d. Ground slope
e. Surface resistance factors for impervious and pervious areas (Manning's N)
f. Surface retention storage for impervious and pervious areas
g. Infiltration rates

2. Conveyance element dimensions

   a. Bottom width
   b. Length
   c. Invert slope
   d. Left and right bank side slopes
   e. Mannings N
   f. Depth
   g. Overbank dimensions

3. Rainfall hyetograph - intensity (in/hr)

D. Use in the Missouri River Division

The Missouri River Division has made several changes to the SWMM model. These changes are outlined below.

1. Overbank floodway sections may be used in conjunction with gutter and pipes.

2. A detention dam with a specified storage-outflow curve may occupy any gutter or pipe segment.

3. To aid in overland flow routing, the way infiltration losses are computed has been changed. The MRD modification limits the infiltration loss to no greater than the current rainfall except that the loss is always at least 5 percent of the computed infiltration rate. This revised algorithm is more in line with the practice of handling rainfall loss rates in the Missouri River Division.

4. An inflow hydrograph may be specified at any gutter or pipe segment. The inflow hydrograph is input as a table of time in hours versus inflow in cfs.

5. A diversion from a gutter or pipe out of the system or to another gutter or pipe may be used. A table of total flow in the gutter versus flow diverted in cfs is included in the input data.

6. Several changes were made which modified some details of what values are printed out and output format.

7. Modifications were made to allow watershed and pipe/gutter input data to be modified by a factor to avoid punching new cards.

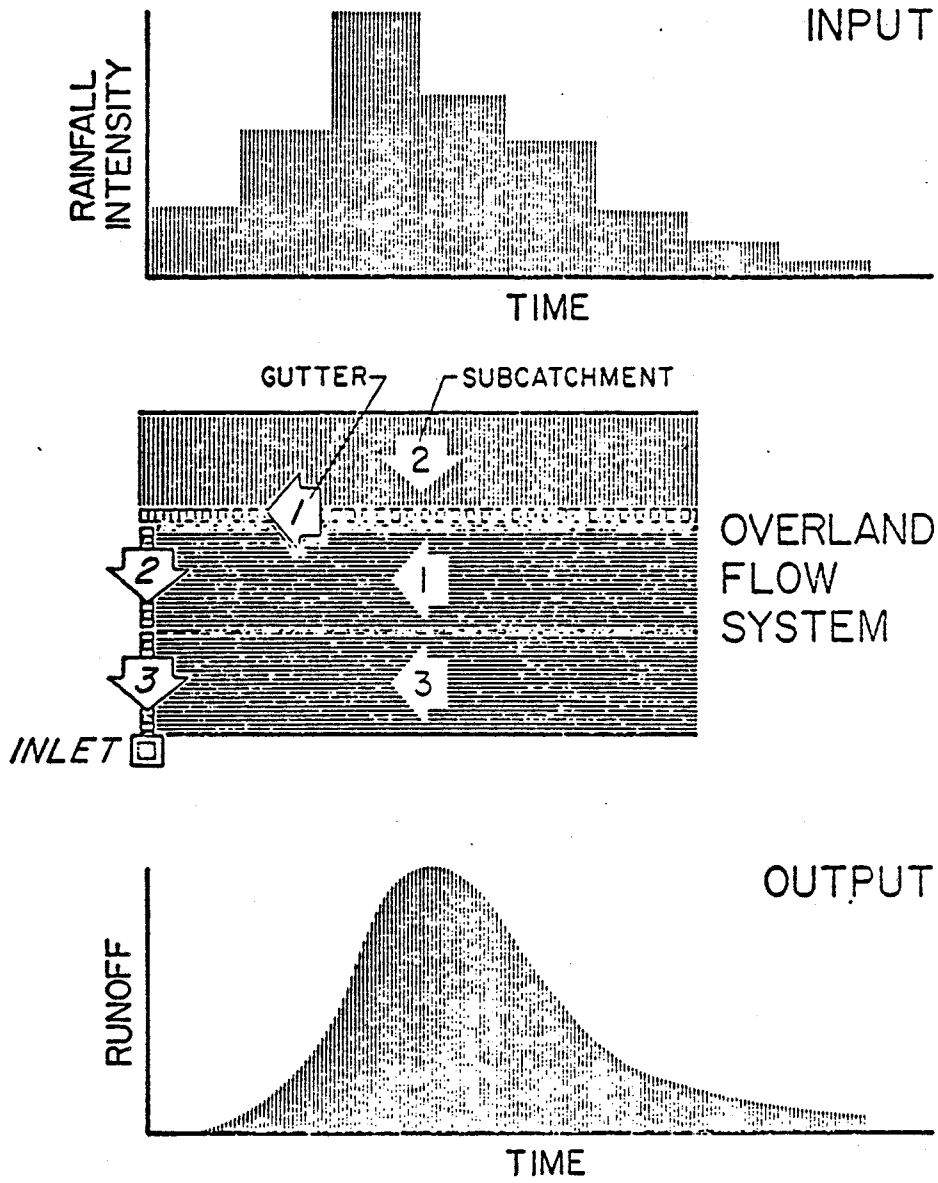


Figure 1. DEFINITION OF A DRAINAGE SYSTEM

III. Program Structure and Implementation

Once the properties of each hydraulic element are given, the computer can be instructed to make a step-by-step accounting of how much water comes in, how much is lost to infiltration, and what will be the outflow and the remaining water depth in each increment of time. This accounting procedure can best be described by the flow chart shown in Figure 5-2. First, the computer reads and edits data pertaining to the rainfall hyetograph, subcatchment, and gutter characteristics and their interconnections. Unit conversion is made for all parameters to conform in the pound-foot-second system.

Stepwise computation proceeds as follows (Figure 2):

1. Rainfall is added to the subcatchment according to the specified hyetograph,

$$D_1 = D_t + R_t \Delta t \qquad (1)$$

where D_1 = Water depth after rainfall

D_t = Water depth of the subcatchment at time, t

R_t = Intensity of rainfall in time interval, Δt

2. Infiltration is computed by Horton's exponential function and is subtracted from water depth existing on the subcatchment,

$$I_t = f_o + (f_i - f_o)\, e^{-\alpha t} \qquad (2)$$

and

$$D_2 = D_1 - I_t \Delta t \qquad (3)$$

where f_o, f_i, and α are coefficients in Horton's equation

D_2 is intermediate water depth after accounting for infiltration.

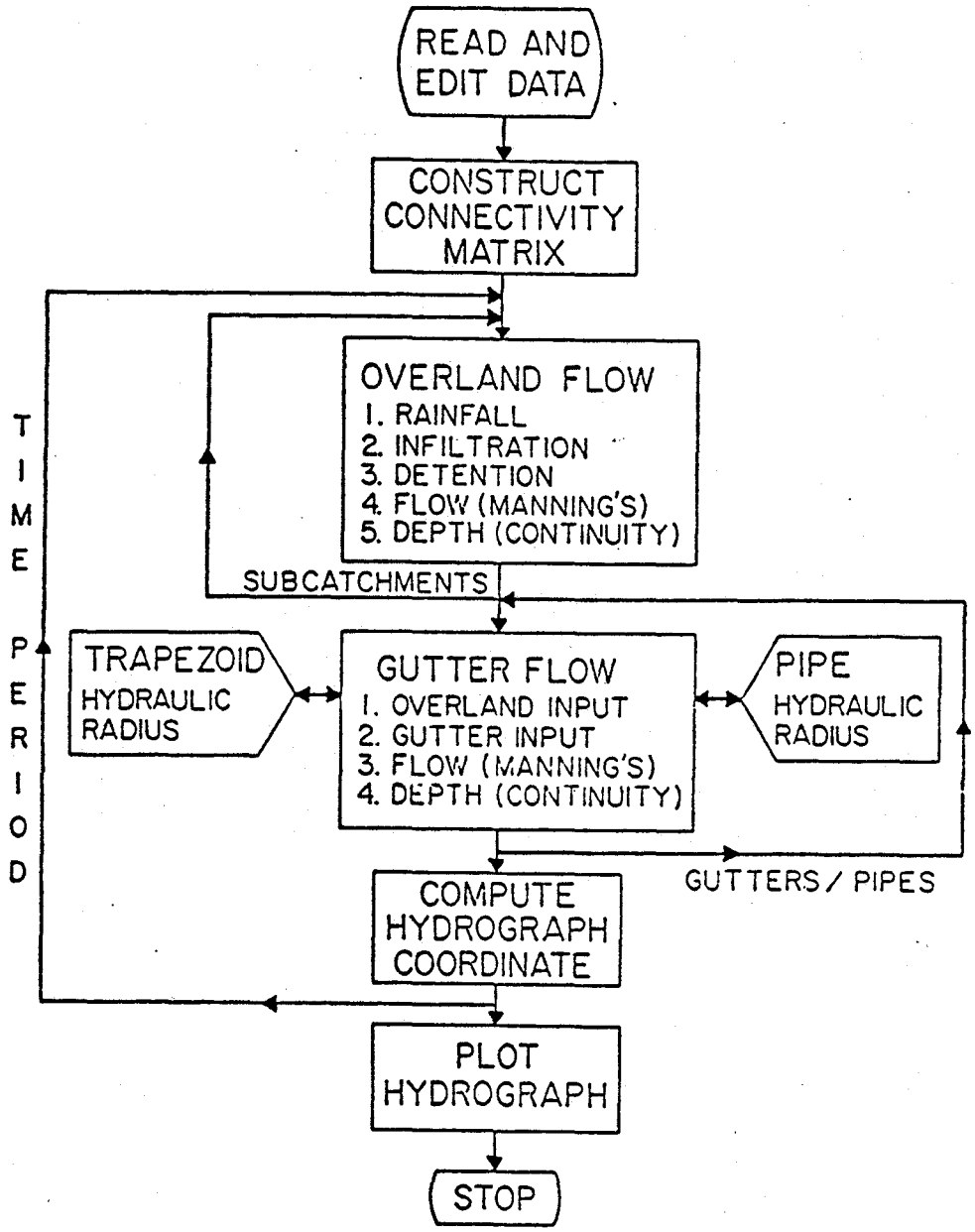


Figure 2. FLOW CHART, HYDROGRAPHIC COMPUTATION

3. If the resulting water depth of the subcatchment, D_2, is larger than the specified detention requirement, D_d, an outflow rate is computed using Manning's equation,

$$V = \frac{1.49}{n} (D_2 - D_d)^{2/3} s^{1/2} \quad (4)$$

and

$$Q_w = V W (D_2 - D_d) \quad (5)$$

where V = Velocity

n = Manning's coefficient

s = Ground slope

W = Width

Q_w = Outflow rate

4. The continuity equation is solved to determine the water depth of the subcatchments, resulting from the rainfall, infiltration, and outflow,

$$D_{t+\Delta t} = D_2 - (Q_w/A)\Delta t \quad (6)$$

where A is the surface area of the subcatchment

5. Steps 1 to 4 are repeated until computations for all subcatchments are completed.

6. The inflow (Q_{in}) to a gutter is computed as a summation of outflow from tributary subcatchments ($Q_{w,i}$) and flow rate of immediate upstream gutters ($Q_{g,i}$).

$$Q_{in} = \Sigma Q_{w,i} + \Sigma Q_{g,i} \quad (7)$$

7. The inflow is added to raise the existing water depth of the gutter according to its geometry,

$$Y_1 = Y_t + (Q_{in} / A_s)\ \Delta t \qquad (8)$$

where Y_1 and Y_t = Water depth of the gutter

A_s = Mean water surface area between Y_1 and Y_t

8. The outflow is calculated for the gutter using Manning's equation,

$$V = \frac{1.49}{n} R^{2/3} S_i^{\ 1/2} \qquad (9)$$

and

$$Q_g = V A_c \qquad (10)$$

where R = Hydraulic radius

S_i = Invert slope

A_c = Cross-sectional area at Y_1

9. The continuity equation is solved to determine the water depth of the gutter, resulting from the inflow and outflow.

$$Y_{t+\Delta t} = Y_1 + (Q_{in} - Q_g)\ \Delta t / A_s \qquad (11)$$

10. Steps 6 to 9 are repeated until all the gutters are finished.

11. The flows, reaching the point of concern, are added to produce a hydrograph coordinate along the time axis.

12. The processes from Steps 1 to 11 are repeated for succeeding time periods until the complete hydrograph is computed.

The step-by-step description of the solution procedure gives a physical picture of the processes being modeled. The integration of variables in each time increment was originally performed by the modified Euler's two-step method. This is now accomplished by the Newton-Raphson method (Ref. 7), which produces a smoother hydrograph and more stable solution.

As was noted, Manning's equation was used for computation during each time interval of integration. The "state" of the system, however, is being updated continuously. Thus, the dynamic behavior of the runoff phenomena was simulated by a stepwise and successive quasi-steady state approximation.

IV. Example of Model Application

HYDROLOGY
Middle Logan Creek
Flood Insurance Study
Randolph, Nebraska

1. General. The hydrology study for Middle Logan Creek Flood Insurance Study at Randolph, Nebraska is presented in the following paragraphs. A map of Middle Logan Creek is presented in Attachment 1.

2. Basin Description. Middle Logan Creeks is a tributary of Logan Creek, a left bank tributary of the Elkhorn River. Middle Logan Creek extends about 4 miles northwest of Randolph and flows in an easterly direction to the downstream study limit about 2 miles east of Randolph. Middle Logan Creek has several major tributaries both to the north and to the south. The drainage area above the downstream limit is 27.8 square miles. Land use in the basin is predominately agricultural. Land slopes range from 1 to 13 percent.

3. Existing Structures. One existing structure in the study area which would significantly attenuate the flow crosses the large northern tributary. The structure is located on the diagonal county road in Section 19, approximately 250 feet north of its intersection with Nebraska State Highway 20. The channel opening under this bridge is approximately 30 feet by 3.5 feet. The location of the structure is shown on Attachment 1.

4. Discharge-Probability.

a. Hydrologic Model. Discharge-probability relationships for Middle Logan Creek were based on a rainfall-runoff analysis using the Environmental Protection Agency's "Storm Water Management Model." The basin was divided into 92 subcatchments which averaged 195 acres.

b. Rainfall. The 10-, 50-, and 100-year one-hour point rainfall values were obtained from NOAA Technical Memorandum NWS HYDRO-35. The 500-year event was determined from an extrapolation of a line plotted through the 10-, 50-, and 100-year values. The maximum one-hour point rainfall values for the 500-, 100-, 50-, and 10-year events are 4.50, 3.63, 3.26, and 2.4 inches, respectively. These values were adjusted for basin size and shape prior to application in the SWMM model.

c. Friction and Infiltration Losses. The Manning "n" coefficient used for channel and overbank conditions ranged from .030 to .040. A soil infiltration rate of 0.60 inches per hour was used for the pervious areas in the hydrologic model. Surface storage values for the pervious and impervious areas were generally .050 and .150 inches, respectively. Larger values were used in several basins based on a site visit.

d. Routing. Storm runoff occurring in the Middle Logan Creek basin above the bridge on the diagonal county road described earlier was routed through the structure. The discharge rating curve through the bridge was developed using the Corps of Engineers' HEC-2 computer program and surveyed cross-sections downstream of and through the bridge. The results of the HEC-2 program were combined with a storage-elevation curve upstream of the

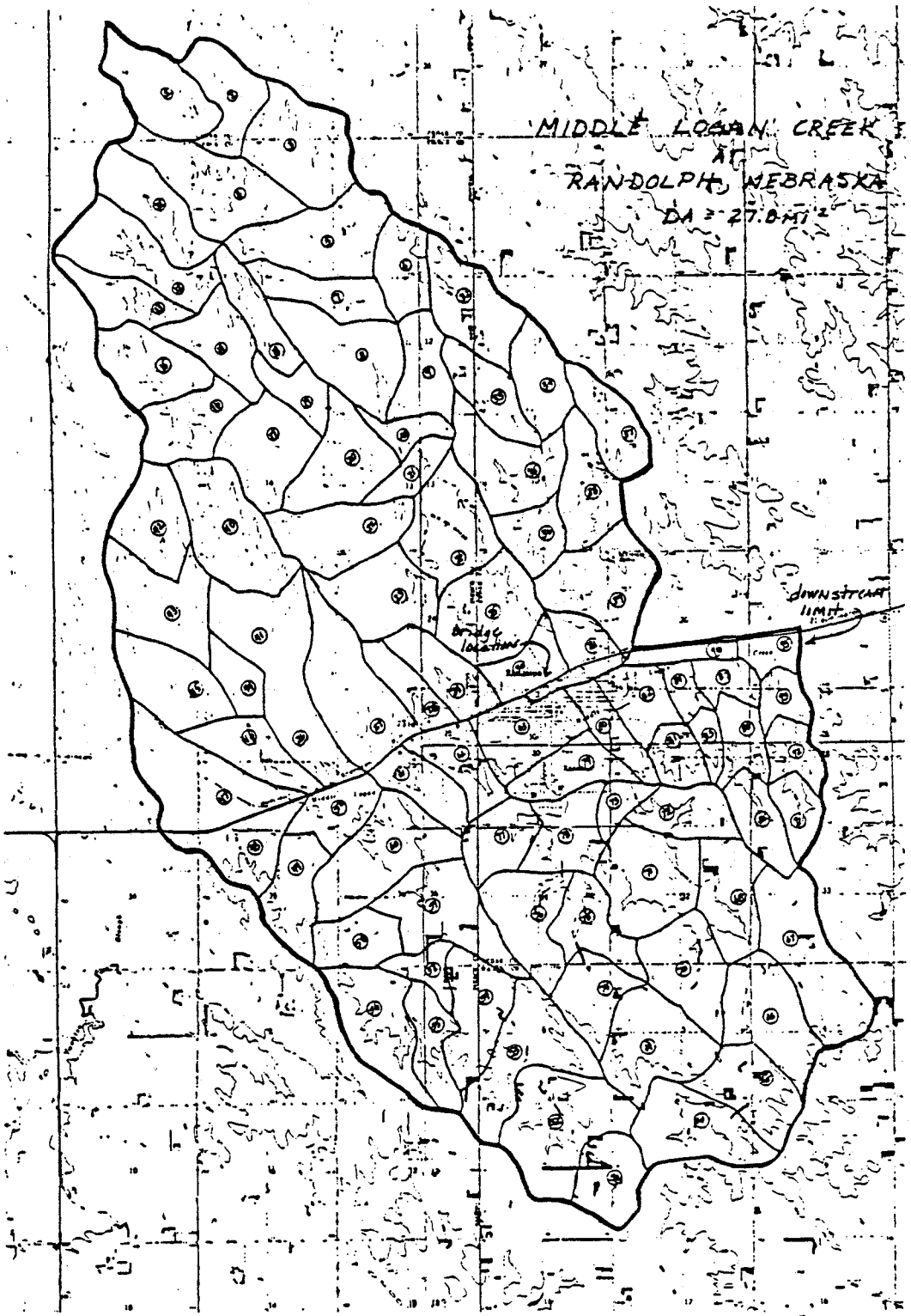
bridge developed using surveyed cross-sections and topographic maps to produce the storage-discharge relationship used in the SWMM model.

e. Calibration. The SWMM model was calibrated using a regional study of floods on small drainage areas in Nebraska. The results of the analysis for Middle Logan Creek at downstream study limit and at subcatchment 62 (as shown on Attachment 1) are shown on Attachments 2 and 3, respectively for comparison purposes.

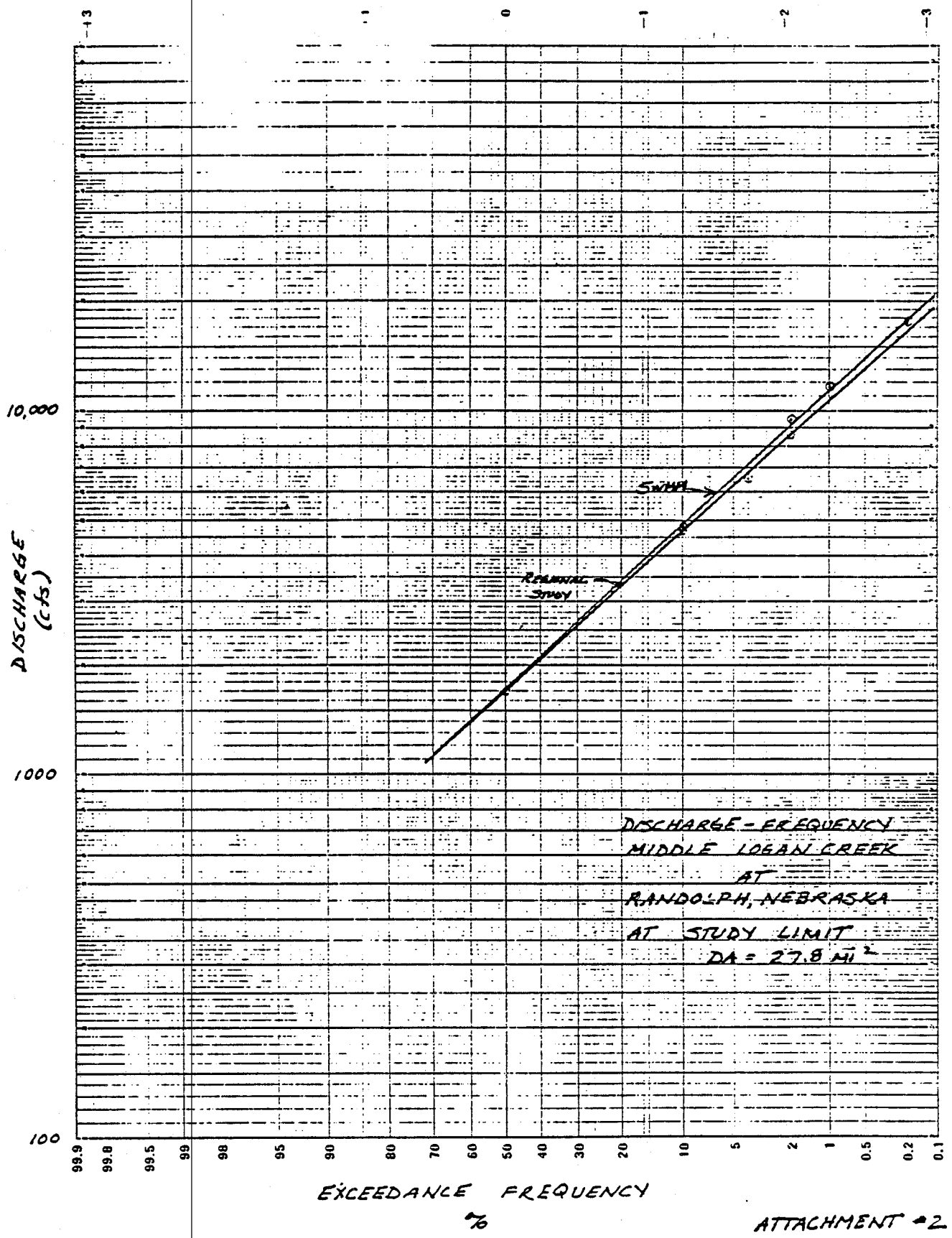
5. Summary. Discharge-probability relationships for the 500-, 100-, 50-, and 10-year events for Middle Logan Creek at selected locations are shown in Table 1.

Table 1
Summary of Peak Discharges
Middle Logan Creek at Randolph, Nebraska

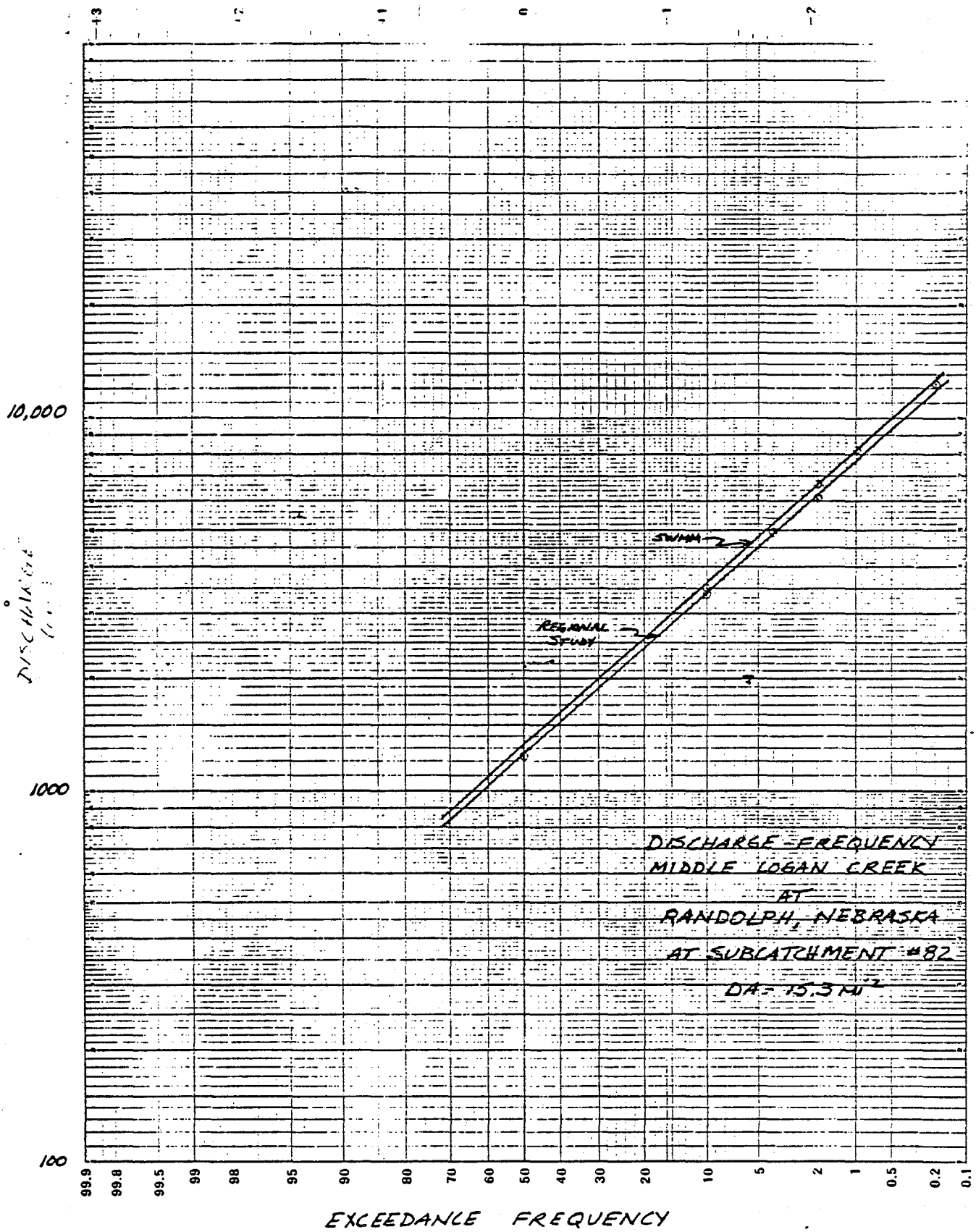
Subarea & Location	Drainage Area (Sq. mi)	Peak Discharge (Cubic feet per second) 10-Yr	50-Yr	100-Yr	500-Yr
95-Middle Logan Creek at Study Limit	27.8	4822	9432	11,677	17,333
67-Middle Logan Creek at junct. w/north tributary	16.0	3531	6785	8,342	12,403
62-Middle Logan Creek at Randolph	15.3	3491	6714	8,267	12,316
63-Middle Logan Creek at junct. w/south tributary	9.4	2419	4561	5,596	8,071
79-South tributary at junct. w/Middle Logan Creek	5.4	1540	2920	3,539	5,326
76-West branch of South Tributary	1.4	524	1082	1,363	2,033
73-East Branch of South Tributary	3.7	1103	2050	2,507	3,635
90-North Tributary at junct. w/Middle Logan Creek	11.3	2195	4135	5,142	7,371
31-West Branch of North Tributary	5.7	1765	3457	4,278	6,170
39-East Branch of North Tributary	2.5	608	1594	1,920	2,987



ATTACHMENT #1



ATTACHMENT #2



ATTACHMENT #3

Middle Logan Creek at Randolph, Nebraska
County Road Bridge across Northern Tributary
Elevation-Storage-Discharge Data

Elevation (ft, msl)	Storage (ac-ft)	Discharge (cfs)
1648.0	0.0	0
1649.6	0.4	50
1650.4	0.8	100
1651.3	1.5	200
1652.2	2.3	400
1652.9	3.1	600
1653.5	4.0	800
1654.0	4.8	1,000
1655.0	6.5	1,450
1656.0	9.0	1,850
1657.0	11.5	4,250
1658.0	14.5	11,150

V. Assessment

The SWMM model has been used on a wide variety of basins with good results. Application of the model is generally not difficult since the model uses physical parameters which can be measured easily. Calibration has been made with both historical hydrologic records and regional studies. SWMM is a kinetic model and therefore may not work well on very flat drainate basins. One pitfall of the model is that it does not consider backwater effects. This can make modeling difficult if there are many bridges or other obsticals to be routed through. In the past, SWMM has been combined with other models if this is a problem.

Examples of Subcatchment Descriptions

Middle Logan Creek, Randolph, Nebraska

SUBAREA NUMBER	GUTTER OR MANHOLE	WIDTH (FT)	AREA (AC)	PERCENT IMPERV.	SLOPE (FT/FT)	RESISTANCE FACTOR IMPERV.	RESISTANCE FACTOR PERV.	SURFACE STORAGE (IN) IMPERV.	SURFACE STORAGE (IN) PERV.	INFILTRATION RATE (IN/HR) MAXIMUM	INFILTRATION RATE (IN/HR) MINIMUM	INFILTRATION RATE (IN/HR) DECAY RATE	GAGE NO
-2	0	0.	0.0	1.0	.0500	.013	.250	.050	.150	.80	.80	.00115	
1	1	8400.	253.4	1.0	.0600	.013	.250	.050	.150	.80	.80	.00115	1
2	2	2000.	192.7	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
3	3	8400.	310.3	1.0	.0300	.013	.250	.050	.150	.80	.80	.00115	1
4	4	13200.	305.5	1.0	.0500	.013	.250	.050	.150	.80	.80	.00115	1
5	5	4200.	309.8	1.0	.0300	.013	.250	.050	.150	.80	.80	.00115	1
6	6	6000.	150.4	1.0	.0500	.013	.250	.050	.150	.80	.80	.00115	1
7	7	4400.	161.3	1.0	.0500	.013	.250	.050	.150	.80	.80	.00115	1
8	8	8800.	294.2	1.0	.0400	.013	.250	.050	.300	.80	.80	.00115	1
9	9	8000.	158.8	1.0	.0600	.013	.250	.050	.150	.80	.80	.00115	1
10	10	3200.	68.1	1.0	.0600	.013	.250	.050	.300	.80	.80	.00115	1
11	11	5200.	215.7	1.0	.0800	.013	.250	.050	.150	.80	.80	.00115	1
12	12	3000.	270.4	1.0	.1000	.013	.250	.050	.150	.80	.80	.00115	1
13	13	7600.	152.0	1.0	.1000	.013	.250	.050	.150	.80	.80	.00115	1
14	14	8400.	185.1	1.0	.0900	.013	.250	.050	.150	.80	.80	.00115	1
15	15	8400.	211.6	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
16	16	1600.	186.0	1.0	.0900	.013	.250	.050	.150	.80	.80	.00115	1
17	17	2400.	242.6	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
18	18	13600.	132.7	1.0	.0800	.013	.250	.050	.150	.80	.80	.00115	1
19	19	8100.	101.3	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
20	20	6000.	242.1	1.0	.0400	.013	.250	.050	.300	.80	.80	.00115	1
21	21	2800.	85.2	1.0	.0600	.013	.250	.050	.300	.80	.80	.00115	1
22	22	10000.	253.4	1.0	.0400	.013	.250	.050	.150	.80	.80	.00115	1
23	23	7200.	258.7	1.0	.0500	.013	.250	.050	.500	.80	.80	.00115	1
24	24	8000.	268.3	1.0	.0500	.013	.250	.050	.300	.80	.80	.00115	1
25	25	11200.	288.6	1.0	.0500	.013	.250	.050	.300	.80	.80	.00115	1
26	26	8000.	189.0	1.0	.0700	.013	.250	.050	.300	.80	.80	.00115	1
27	27	10000.	334.8	3.0	.0800	.013	.250	.050	.300	.80	.80	.00115	1
28	28	6800.	105.0	3.0	.1100	.013	.250	.050	.300	.80	.80	.00115	1
29	29	6400.	156.6	5.0	.0700	.013	.250	.050	.300	.80	.80	.00115	1
30	30	4800.	117.1	5.0	.0300	.013	.250	.050	.300	.80	.80	.00115	1
31	31	3300.	129.9	5.0	.0500	.013	.250	.050	.300	.80	.80	.00115	1
32	32	7600.	241.8	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
33	33	8000.	161.8	1.0	.1000	.013	.250	.050	.150	.80	.80	.00115	1
34	34	8400.	161.6	1.0	.0800	.013	.250	.050	.150	.80	.80	.00115	1
35	35	5200.	254.5	1.0	.0800	.013	.250	.050	.150	.80	.80	.00115	1
36	36	3000.	119.5	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
37	37	8000.	163.2	1.0	.0600	.013	.250	.050	.150	.80	.80	.00115	1
38	38	7600.	196.1	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
39	39	10400.	299.5	2.0	.0600	.013	.250	.050	.300	.80	.80	.00115	1
40	40	12800.	312.7	1.0	.0400	.013	.250	.050	.150	.80	.80	.00115	1
41	41	6000.	239.9	1.0	.0400	.013	.250	.050	.150	.80	.80	.00115	1
42	42	7600.	201.6	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
43	43	6400.	273.5	1.0	.0500	.013	.250	.050	.150	.80	.80	.00115	1
44	44	4400.	96.1	1.0	.0800	.013	.250	.050	.150	.80	.80	.00115	1
45	45	8000.	238.0	1.0	.0500	.013	.250	.050	.150	.80	.80	.00115	1
46	46	8000.	167.4	1.0	.0500	.013	.250	.050	.150	.80	.80	.00115	1
47	47	12000.	333.9	1.0	.0500	.013	.250	.050	.150	.80	.80	.00115	1
48	48	4600.	194.0	1.0	.0600	.013	.250	.050	.150	.80	.80	.00115	1
49	49	7600.	179.2	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
50	50	2600.	141.9	1.0	.0200	.013	.250	.050	.150	.80	.80	.00115	1
51	51	5600.	173.4	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
52	52	8000.	372.1	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
53	53	4400.	316.6	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1
54	54	7600.	223.5	1.0	.0600	.013	.250	.050	.150	.80	.80	.00115	1
55	55	4200.	138.9	1.0	.0700	.013	.250	.050	.150	.80	.80	.00115	1

Examples of Gutter Descriptions

Middle Logan Creek, Randolph, Nebraska

Gutter number	Gutter connection	JUN	NP		WIDTH OR DIAM (FT)	LENGTH (FT)	INVERT SLOPE (FT/FT)	SIDE SLOPES HORIZ TO VERT L	R	OVERBANK/SURCHARGE MANNINGS N	DEPTH (FT)	JK
1	2	0	4	CHANNEL	4.0	4200.	.0250	1.0	1.0	.035	2.00	0
				OVERFLOW	50.0	4000.	.0263	25.0	25.0	.040	10.00	
2	3	0	4	CHANNEL	12.0	1000.	.0100	1.0	1.0	.035	3.00	0
				OVERFLOW	50.0	900.	.0111	25.0	25.0	.040	10.00	
3	5	0	4	CHANNEL	15.0	4200.	.0070	1.0	1.0	.035	4.00	0
				OVERFLOW	100.0	4000.	.0074	50.0	50.0	.040	10.00	
4	5	0	1	CHANNEL	100.0	6600.	.0200	25.0	25.0	.040	10.00	0
5	7	0	4	CHANNEL	10.0	4600.	.0050	1.0	1.0	.035	2.00	0
				OVERFLOW	100.0	4200.	.0055	50.0	50.0	.040	10.00	
6	7	0	1	CHANNEL	100.0	3000.	.0300	25.0	25.0	.040	10.00	0
7	8	0	4	CHANNEL	4.0	2200.	.0050	1.0	1.0	.035	3.00	0
				OVERFLOW	100.0	2000.	.0055	100.0	10.0	.040	10.00	
8	10	0	4	CHANNEL	8.0	4400.	.0050	1.0	1.0	.035	3.00	0
				OVERFLOW	100.0	4000.	.0055	50.0	50.0	.040	10.00	
9	10	0	1	CHANNEL	100.0	4000.	.0150	25.0	25.0	.040	10.00	0
10	21	0	4	CHANNEL	10.0	1600.	.0050	1.0	1.0	.035	4.00	0
				OVERFLOW	100.0	1500.	.0053	75.0	75.0	.040	10.00	
11	12	0	4	CHANNEL	3.0	4600.	.0200	1.0	1.0	.035	2.00	0
				OVERFLOW	100.0	4200.	.0219	50.0	50.0	.040	10.00	
12	14	0	4	CHANNEL	3.0	2000.	.0080	1.0	1.0	.035	2.00	0
				OVERFLOW	100.0	1900.	.0084	50.0	50.0	.040	10.00	
13	14	0	4	CHANNEL	3.0	3800.	.0300	1.0	1.0	.035	2.00	0
				OVERFLOW	100.0	3600.	.0317	50.0	50.0	.040	10.00	
14	17	0	4	CHANNEL	5.0	4400.	.0080	1.0	1.0	.035	2.00	0
				OVERFLOW	100.0	4000.	.0088	50.0	50.0	.040	10.00	
15	16	0	4	CHANNEL	8.0	4200.	.0200	1.0	1.0	.035	3.00	0
				OVERFLOW	100.0	4000.	.0210	25.0	25.0	.040	10.00	
16	17	0	1	CHANNEL	100.0	1600.	.0100	25.0	25.0	.040	10.00	0
17	20	0	4	CHANNEL	10.0	2800.	.0060	1.5	1.5	.035	7.00	0
				OVERFLOW	100.0	2600.	.0065	50.0	50.0	.040	10.00	
18	19	0	4	CHANNEL	2.0	6800.	.0200	1.0	1.0	.035	1.00	0
				OVERFLOW	50.0	6000.	.0227	25.0	25.0	.040	10.00	
19	20	0	4	CHANNEL	2.0	5400.	.0100	1.0	1.0	.035	1.00	0
				OVERFLOW	50.0	5000.	.0108	50.0	50.0	.040	10.00	
20	21	0	4	CHANNEL	10.0	3000.	.0060	1.5	1.5	.035	7.00	0
				OVERFLOW	100.0	2800.	.0064	100.0	100.0	.040	10.00	
21	24	0	4	CHANNEL	12.0	1400.	.0050	1.0	1.0	.035	5.00	0
				OVERFLOW	100.0	1300.	.0054	75.0	75.0	.040	10.00	
22	24	0	4	CHANNEL	3.0	5000.	.0250	1.0	1.0	.035	1.00	0
				OVERFLOW	200.0	4500.	.0278	25.0	25.0	.040	10.00	
23	24	0	1	CHANNEL	100.0	3600.	.0250	50.0	50.0	.040	10.00	0
24	25	0	4	CHANNEL	17.0	4000.	.0030	1.0	1.0	.035	5.00	0
				OVERFLOW	0.0	3800.	.0032	100.0	100.0	.040	10.00	
25	30	0	4	CHANNEL	8.0	5600.	.0030	2.0	2.0	.035	8.00	0
				OVERFLOW	0.0	5400.	.0031	20.0	100.0	.040	16.00	
26	50	0	1	CHANNEL	100.0	4000.	.0100	50.0	50.0	.040	10.00	0
27	61	0	4	CHANNEL	5.0	5000.	.0150	1.0	1.0	.035	2.00	0
				OVERFLOW	100.0	4800.	.0156	50.0	50.0	.040	10.00	
28	62	0	1	CHANNEL	20.0	3400.	.0250	25.0	25.0	.040	10.00	0
29	63	0	1	CHANNEL	100.0	3200.	.0250	25.0	25.0	.040	10.00	0
30	201	0	4	CHANNEL	6.0	2400.	.0030	1.5	2.5	.035	6.00	0
				OVERFLOW	0.0	2300.	.0031	85.0	100.0	.040	13.00	
201	31	12	2	PIPE	.5	30.	.0100	0.0	0.0	.040	.50	0

RESERVOIR STORAGE IN ACRE-FEET VS SPILLWAY OUTFLOW

		0.0	0.0		.4	50.0	.6	100.0		1.5	200.0		2.3	400.0		3.1	600.0
	4.0	800.0	4.8	1000.0		6.5	1450.0	9.0	1850.0	11.5	4250.0		14.5	11150.0			
31	40		0		4		CHANNEL	8.0	2200.	.0030		6.0	2.0		.035	7.00	0
							OVERFLOW	0.0	2100.	.0031		35.0	35.0		.040	12.00	

Examples of Outflow Hydrograph Tables for 10 Locations

Middle Logan Creek, Randolph, Nebraska

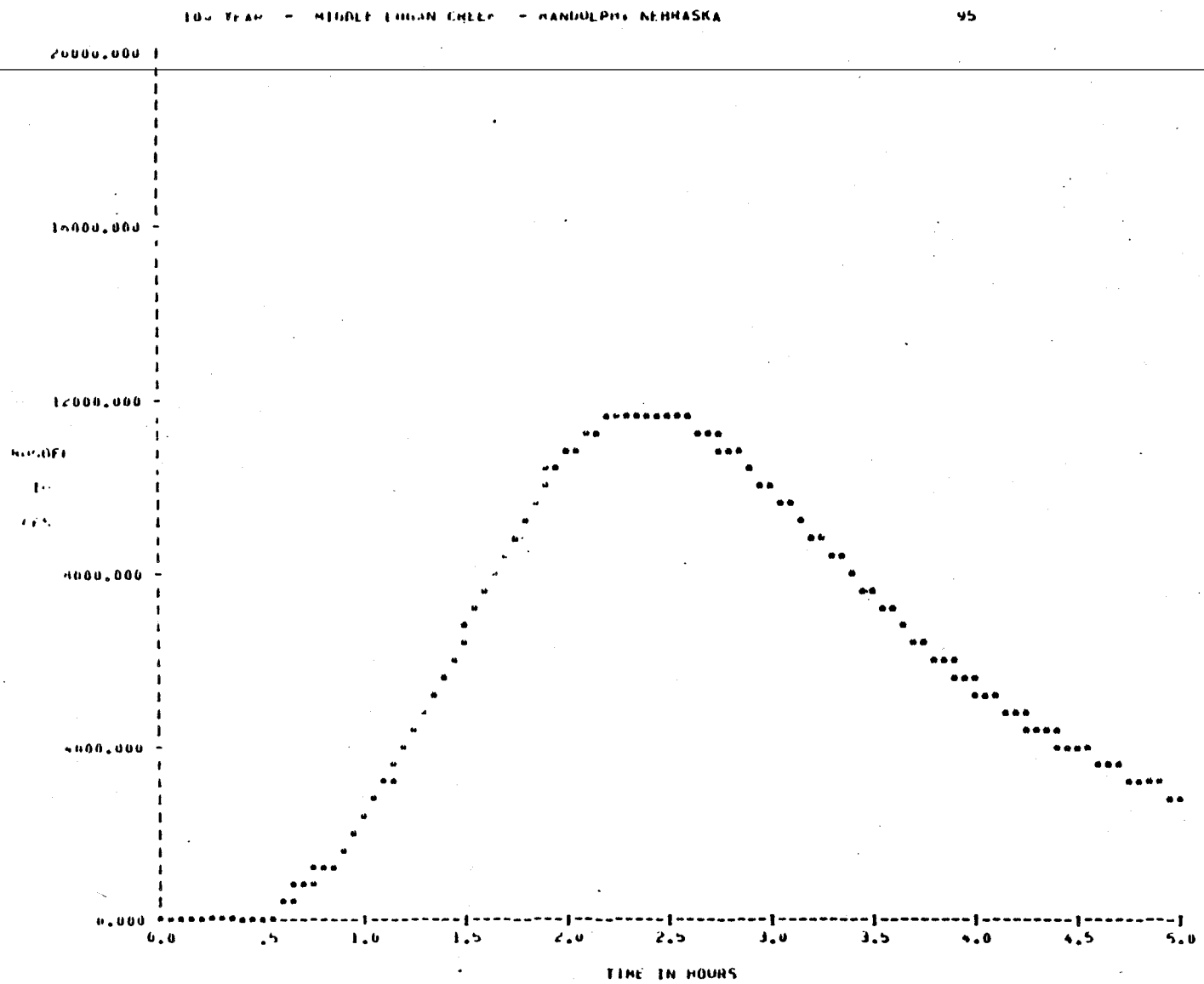
HYDROGRAPHS ARE LISTED FOR THE FOLLOWING 10 POINTS

THE UPPER NUMBER IS DISCHARGE IN CFS
THE LOWER NUMBER IS DEPTH ABOVE INVERT IN FEET
THE LETTERS (S), DENOTES STORAGE IN ACRE-FT FOR SURCHARGED PIPE OR DAM. DISCHARGE INCLUDES SPILLWAY OUTFLOW.
THE LETTER (I), DENOTES GUTTER INFLOW IN CFS FROM SPECIFIED INFLOW HYDROGRAPH
THE LETTER (D), DENOTES DISCHARGE IN CFS DIVERTED FROM THIS GUTTER

TIME	40	34	63	73	78	79	82	87	90	95
0 5.00	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
	.00	.00	.00	.00	.00	.00	.00	.00	.00	.00
0 10.00	.33	.29	1.86	.03	.28	1.63	.75	.13	.08	.16
	.11	.06	.25	.01	.08	.27	.15	.06	.04	.05
0 15.00	2.07	.98	20.32	.13	1.11	15.08	9.79	.89	.51	1.27
	.31	.16	.98	.02	.16	.88	.66	.18	.12	.18
0 20.00	4.43	2.25	58.23	.50	2.45	40.64	39.10	3.24	1.64	3.82
	.45	.26	1.73	.05	.23	1.40	1.35	.35	.25	.35
0 25.00	9.24	6.43	121.45	3.01	7.07	80.69	110.30	13.25	5.56	13.03
	.75	.47	2.53	.15	.35	1.91	2.22	.69	.51	.73
0 30.00	24.42	30.22	251.12	18.39	24.78	164.13	271.89	53.21	20.60	53.93
	1.30	1.11	3.61	.39	.59	2.60	3.34	1.28	1.07	1.65
0 35.00	75.99	130.58	516.78	90.51	87.93	317.41	620.50	191.36	76.42	221.68
	2.33	2.35	5.07	.84	.98	3.32	4.77	2.18	2.16	3.55
0 40.00	170.04	328.18	786.05	274.55	219.92	456.40	1082.44	441.18	206.31	619.55
	3.47	3.64	6.13	1.37	1.40	3.64	6.02	3.06	3.53	5.95
0 45.00	315.02	489.74	1012.94	553.56	405.21	653.55	1267.37	763.09	399.84	969.69
	4.64	4.18	6.86	1.84	1.77	3.94	6.26	3.80	4.82	7.20
0 50.00	526.97	592.19	1344.53	887.49	609.76	968.81	1537.26	1152.31	628.83	1280.85
	5.87	4.40	7.77	2.20	2.07	4.27	6.50	4.47	5.92	7.57
0 55.00	703.95	736.86	1776.38	1185.35	789.27	1375.96	1927.98	1569.73	894.25	1773.50
	6.49	4.61	8.77	2.43	2.29	4.58	6.77	5.05	6.92	7.96
1 -.00	861.98	890.21	2284.31	1452.31	920.76	1813.20	2465.82	2020.00	993.18	2373.04
	6.85	4.78	9.77	2.60	2.43	4.84	7.05	5.57	7.15	8.31
1 5.00	1043.45	1054.98	2808.68	1670.65	991.87	2211.54	3085.82	2509.19	1119.09	3007.36
	7.11	4.93	10.66	2.72	2.50	5.04	7.30	6.05	7.32	8.60
1 10.00	1234.78	1159.40	3272.09	1831.82	1006.91	2528.47	3740.69	2943.59	1264.02	3655.04
	7.38	5.06	11.29	2.80	2.51	5.18	7.53	6.33	7.49	8.85
1 15.00	1429.69	1319.88	3571.69	1946.93	987.12	2756.52	4433.22	3450.18	1482.03	4318.31
	7.58	5.15	11.59	2.86	2.49	5.27	7.73	6.61	7.66	9.07
1 20.00	1631.92	1469.91	3853.72	2021.96	941.00	2900.24	5109.29	4018.59	1705.29	5028.26
	7.77	5.21	11.83	2.89	2.45	5.32	7.91	6.88	7.81	9.28
1 25.00	1844.46	1607.33	4117.42	2062.32	881.85	2972.14	5723.33	4625.83	1944.32	5795.59
	7.94	5.25	12.02	2.91	2.39	5.35	8.05	7.12	7.95	9.49
1 30.00	2067.26	1693.57	4347.88	2075.73	817.41	2987.63	6253.89	5236.87	2188.48	6605.62
	8.10	5.27	12.17	2.92	2.32	5.36	8.17	7.34	8.08	9.69
1 35.00	2297.47	1692.47	4531.63	2067.44	752.56	2961.45	6691.66	5815.48	2436.82	7411.24
	8.25	5.27	12.28	2.91	2.25	5.35	8.26	7.52	8.20	9.87
1 40.00	2529.15	1669.02	4660.36	2040.10	696.11	2905.14	7033.58	6332.79	2690.95	8188.27
	8.39	5.25	12.35	2.90	2.17	5.33	8.33	7.67	8.31	10.03
1 45.00	2754.55	1628.45	4731.40	1996.32	631.56	2826.93	7280.53	6770.02	2940.52	8931.94
	8.51	5.22	12.39	2.88	2.10	5.30	8.38	7.79	8.40	10.18
1 50.00	2965.28	1575.59	4746.73	1938.84	577.56	2732.81	7436.29	7117.53	3178.55	9613.66
	8.62	5.19	12.40	2.85	2.03	5.26	8.41	7.88	8.49	10.31
1 55.00	3153.41	1519.61	4711.55	1870.42	528.24	2627.31	7506.99	7372.62	3402.55	10209.81
	8.72	5.14	12.38	2.82	1.96	5.22	8.42	7.94	8.57	10.41
2 -.00	3312.42	1298.89	4632.41	1793.59	483.50	2513.93	7500.68	7537.58	3609.31	10706.83
	8.79	5.09	12.34	2.78	1.90	5.17	8.42	7.98	8.63	10.50

Example of Outflow Hydrograph

Middle Logan Creek, Randolph, NE



MITCAT

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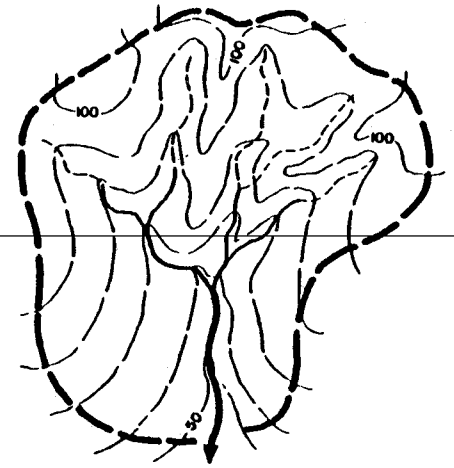


Figure 6.14. Drainage network of typical catchment. (Reproduced from Ref. 31.)

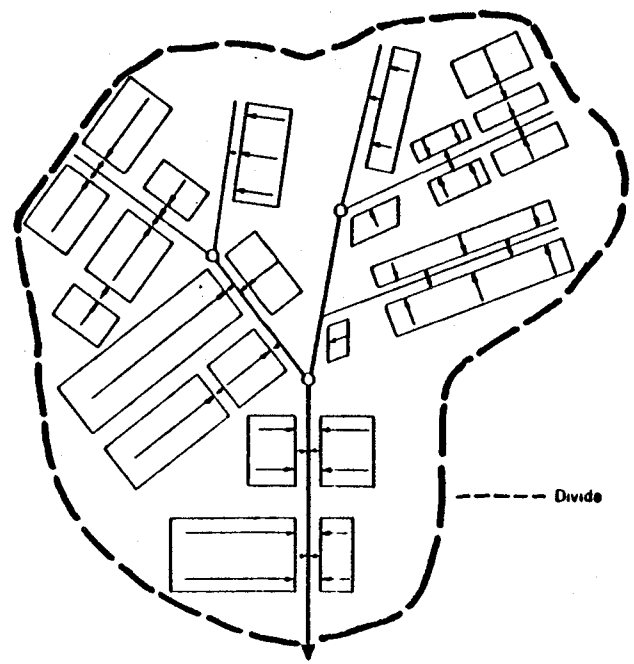


Figure 6.15. Equivalent "block" diagram of the catchment. (Reproduced from Ref. 31.)

6.5 MASSACHUSETTS INSTITUTE OF TECHNOLOGY CATCHMENT MODEL

One of the computer models mentioned in the previous chapter, the Massachusetts Institute of Technology Catchment Model (MITCAT) [31], is useful in evaluating drainage management programs.

Philosophy of the Catchment Model

MITCAT represents the physical movement of water over the catchment surface and through the channel network of a river basin [32]. Recognizing that surface geometry is extremely irregular and impossible (and uncessary) to represent in complete detail in either a physical or a mathematical model, MITCAT follows a reductionist approach. The model replaces the natural complexities with a number of simple elements such as overland flow planes, stream segments, pipe segments, etc. A suitable combination of an appropriate number of these simple elements is assumed sufficient to model the behavior of an entire catchment.

A sample catchment is illustrated in Fig. 6.14. A possible combination of overland flow and streamflow elements appears as a detailed model of this catchment in Fig. 6.15. Less detailed representations may be used in most practical applications.

Some of the basic considerations governing the development of the model were:

The model was based on sound physical reasoning.

The parameters were directly related to the physical characteristics of the catchment and were directly measurable from map or field data whenever possible.

"Fudge factors," which proliferate in many hydrologic models, were avoided.

The model was made as simple as possible, and minimizes the amount of field data required before reasonable results could be derived. The results are, in general, not very sensitive to uncertainty in the parameter estimates.

The model eliminates or minimizes the requirements for historical rainfall or runoff data for calibration or estimation of parameters.

The model handles catchments of any size or shape, urban or rural, and does not require complex input data to achieve this capability.

The model has been designed principally to simulate storm runoff events.

Numerical approximations, such as finite grid mesh sizes, optimum time step, linearization procedures, etc., are controlled internally to make the model as self-sufficient as possible and to make the model useful to engineers not familiar with the numerical analysis or numerical solution of different equations.

With these considerations in mind, certain basic runoff elements were chosen. The elements considered were:

Flow distributed over the surface of the catchment is modeled by planes of overland flow. An overland flow plane is subject to spatially uniform lateral inflow from rainfall, lateral outflow to infiltration, and upstream inflow from adjacent overland flow segments.

Flow from the overland flow planes would be 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 presently includes both open channel flow reaches and closed pipe elements.

Storage is simulated by reservoir elements.

Each of these elements can be linked together to simulate a prototype basin. The elements are first linked into *segments* to define the model of a single subbasin. These segments are in turn linked together to define the total basin network. The choice and arrangement of elements in any particular situation depends upon several factors, which are presented in subsequent sections.

Model Overview

The present MITCAT model can input several forms of rainfall data, which are then reduced to effective rainfall by one of several infiltration algorithms. Holtan's, Horton's, an Antecendent Precipitation Index Method, SCS, or direct input methods are immediately available for use; other algorithms have been programmed but are not directly on line.

Runoff as overland flow and as stream is simulated by use of the kinematic wave equation. The kinematic wave model was adopted as the prime routing method due to several factors. As the simplest of the nonlinear models available, it yields the benefits of a nonlinear response without needing an unduly complicated solution procedure. The use of any equation of hydraulic unsteady flow in modeling the phenomenon of overland flow is subject to considerable parameter adjustment to account for the peculiar flow regime involved. To this end, the two parameters of the kinematic model yield sufficient range to model the runoff phenomenon.

The kinematic wave equation for an overland flow segment is:

$$\frac{\partial y}{\partial t} + \frac{\partial q}{\partial x} = i - f \qquad (6.44)$$

$$q = \alpha_c y^{m_c} \qquad (6.45)$$

where y is the depth of flow, q is the rate of flow, t is time, x is distance along the segment, i is the rainfall intensity, and f is the infiltration rate. In Eq. (6.44), both i and f may vary with x and t. The difference i - f may be treated as an effective rainfall rate (which, by convention in hydrology, is never negative). The fact that f may vary with x causes the model to simulate runoff only from those locations where i exceeds f.

The corresponding equation for the stream segments is:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad (6.46)$$

$$Q = \alpha_s A^{m_s} \qquad (6.47)$$

where A is the cross-sectional area of flow, Q is the discharge rate, and q is the lateral inflow rate of overland flow.

The above equations contain the so-called kinematic wave parameters (α, m) for both overland flow and stream segments. The estimation of these parameters is performed internally in the model from readily derived properties of the element. These properties generally include:

Cross section shape
Slope
Roughness factor, generally Manning's n

The extraction of the appropriate values of α and m are presented in the principal reference [31].

The model solves the kinematic wave equations by numerical techniques. The details of these techniques have been carefully developed over a period of many years to the point where reliable procedures have been programmed to automatically assure the most economical solution of these equations [31].

The data input needs can generally be classified as follows:

Rainfall
Infiltration parameters
Area of basin or subbasin
Length of typical overland pattern
Slope of typical overland runoff pattern
Roughness of typical overland runoff pattern
Impervious cover of basin
Length of stream through basin
Slope of stream through basin
Roughness of stream through basin
Typical section of stream through basin

The analysis and reduction of a basin to a representative model is a reasonable process and is the final determination as to the extent of data necessary to fulfill the above classes.

The full MITCAT model, as currently operational, requires approximately 350K bytes of core to run. A small nonproprietary version, referred to as MINICAT, is available which has basic capabilities of watershed representation, routing elements, and two infiltration methods (SCS or Horton's); MINICAT requires 140K bytes to operate, though this requirement can be reduced to 64K bytes.

MITCAT Structure and File System

The key elements of MITCAT are its modular structure and extensive file system. These allow flexibility for the user to decide how to model a basin and how to analyze the problem under study.

Structure of the Program

MITCAT follows certain basic steps during a simulation run. These are:

Read and check input data: The user's commands and the associated data are read in and checked for errors, and if any are present, messages are printed out.

Display input data: All of the input data, as well as data picked up from the file system or estimated by MITCAT, are printed out. This allows easy verification of the input data by the user.

Routing and output: Each active element in the system is then simulated and its results are printed/plotted/stored.

MITCAT operates on the basin configuration in a sequential manner. This means that the whole time history of interest is processed through each of the simulating modules in sequence. This mode of operation should be contrasted with the parallel modes where the state (i.e., water stage, storage, etc., rather than discharges) of the total system is computed at each time step.

Use of a sequential model has the advantage that each individual element can be operated at different time steps. The size of the time increment may be dictated by model stability and/or convergence criteria or simply by the user's idea of what may be an "optimum" time step for such an element.

A disadvantage of this mode of operation is that decision making (where it is required) at each time step is rather difficult since the "state" of the system at all the downstream points is not known. On the other hand, a parallel model of operation permits ready decision making at each time step.

DATSYS File System

A feature of MITCAT is its associated file system (DATSYS). The data file handling system is designed to allow the user to access, process, etc. data which has been stored in the data bank system during the operation of a simulation (or other program). The outputs from these runs are usually time histories of data which are frequently necessary to plot (separately or as multiple plots), compute the moments of, and/or copy to other files. It is also possible to load historic data into the file system so that they may be used by a main line program for a simulation run.

Figure 6.16 shows the basic structure of the MITCAT System. Most important for the purpose of this book are the connections between MITCAT, the data bank, and the land use files. The efficient use of this file system can reduce the cost of a simulation run. For example, a run can be restarted at an intermediate point in a catchment if a new test must be made where only a change in the lower part of the basin is to be made.

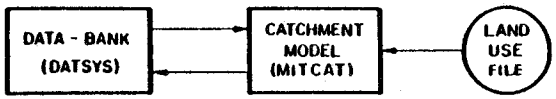


Figure 6.16. MITCAT system structure. (Reproduced from Ref. 31.)

Modeling Strategy

The general philosophy behind the catchment model was presented earlier in this section. As was emphasized, the model follows a reductionist approach using simple elements to model the natural complexities of a drainage basin. Presented here are the approach to be used in modeling the basin and the available methods of linking the various individual elements. Also presented are the strategies available to the user in modeling the behavior of complex hydrologic systems.

MITCAT is primarily a model to permit simulation of individual storms. It is thus more suitable for specific flood event simulation than for long-term simulation which might be required for low-flow determination or storage yield analysis. Another model, which may be based on the MITCAT data base, should be used if long-term simulation is desired.

MITCAT Elements

MITCAT is designed to operate as an interconnected series of simple routing elements. The operation of each of these elements is simulated by a relatively simple set of nonlinear techniques. These techniques were selected so that the data required for each could generally be derived from readily available sources such as topographic maps, soil surveys, and stream cross sections. At the same time, a determined effort was made to limit the number of black-box techniques and the resulting factors which would be required. The resulting MITCAT model is a deterministic simulation model with little calibration required to model the behavior of a given hydrologic system. Although the operation of each element within the model can be readily and easily simulated, the user must ensure that the chosen network of segments and elements does, in fact, represent the prototype hydrologic system in the basin. This section is designed to present some of the points considered germane to the correct choice of modeling elements and to present operational techniques developed in the first years of applied use of MITCAT.

Initial examination of the basin to be simulated will indicate the occurrence of a number of specific locations, both natural and those commonly referred to as "design points," which serve to define specific points of interest along the various stream reaches within the network. The natural points usually occur at locations such as stream junctions, highway crossings, and reservoir sites. The first step in the application of a limited-element model is to subdivide the basin into a number of subbasins. These are shown in Fig. 6.17, and are chosen so that subbasin boundaries occur at each of the following locations:

Primary junctions
Design points
Stream flow gage locations (if available)
Reservoir sites

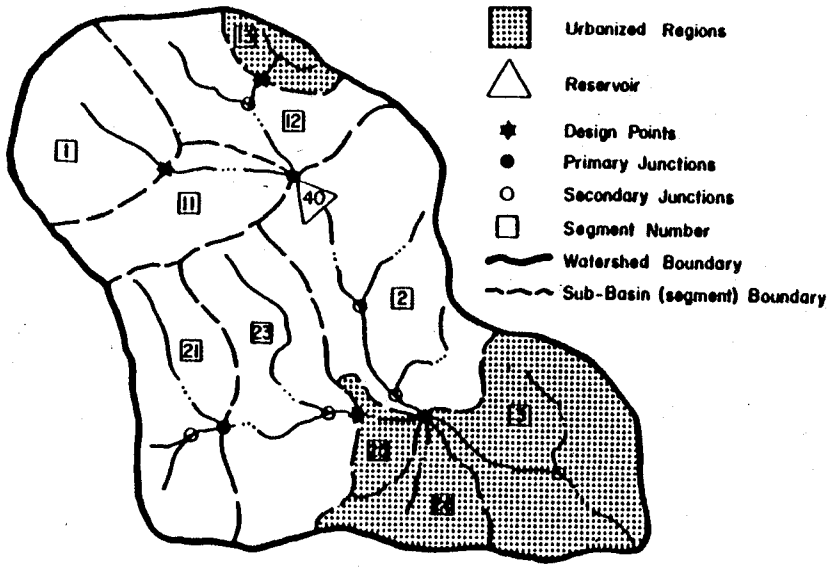


Figure 6.17. Watershed example. (Reproduced from Ref. 31.)

These initial subbasins may need to be further subdivided to account for major nonhomogeneities within them. For example, extreme rainfall variations due to orographic effects or development of part of a subbasin should encourage the modeler to subdivide further until a reasonably homogeneous condition exists. It is difficult to define what specific size limitations might be applied to the selection of the subbasins. Operational use of the model has indicated that subbasin sizes ranging up to 10 km^2 may be adequately modeled using a single model segment. A *segment* is the MITCAT model equivalent of a subbasin. In areas with relatively steep slopes and limited urbanization, segment sizes of up to 26 km^2 have been used. It is normally considered good practice, however, in situations where such large segments are involved to select the subbasins so that at least three subbasins are modeled upstream of any design points. This prevents the "lumping" inherent in the model application from significantly distorting the outflow hydrograph at the desired location.

The basic network of subbasins indicated in Fig. 6.17 can be visualized as in Fig. 6.18. This figure represents the MITCAT network connectivity. It is noted that such a schematic closely follows the prototype system with one notable exception. MITCAT programming considerations limit, to not more than two, the number of segments which may be linked as upstream inputs to a segment. In order to model the situation where segments 25, 26, and 2 are linked to segment 3, it would be necessary to insert a "dummy" segment, such as number 31, in the network. Such segments serve no routing purpose but are simply used to permit simulation of networks which have junctions with numerous streams tributary at the junction.

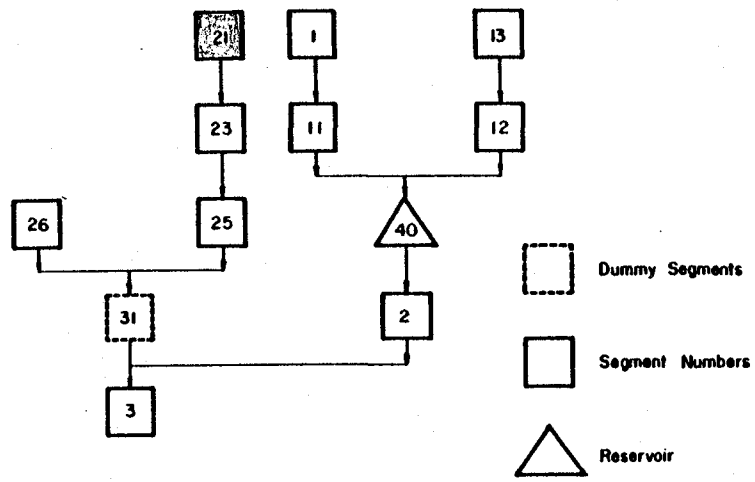


Figure 6.18. Segment connectivity of watershed example. (Reproduced from Ref. 31.)

The selection of the model *elements* required to simulate the hydrologic behavior within each subbasin is the next area where modeling strategy is involved. There are four such elements available within MITCAT, namely:

The *catchment* element, which is used to simulate overland flow
The *stream* element, which is used to simulate the behavior of a stream reach
The *reservoir* element, which is used to simulate the operation of on-stream regulation facilities, and smaller storage systems such as reservoirs for site detention control
The *pipe* element, which may be used to model the behavior of closed conduits

Typical schematics of each of these elements are presented in Fig. 6.19.

Most subbasins (segments) are modeled using a simple combination of catchment and stream elements as illustrated in Fig. 6.20. The typical V-sloped segment shown has been found to be an extremely efficient and accurate way of modeling the behavior of far more complex prototype systems.

Catchment Element

In the V-sloped segment rainfall input is applied to each of the catchment elements. The operation of each of these elements is simulated by modeling the processes of infiltration and overland flow which occur on a unit strip of each element. In order to apply simple modeling techniques, all the parameters for each of these elements are considered to be spatially homogeneous. The parameter that is of greatest interest to the model at this stage is the overland flow length, designated by Lc_1 or Lc_2 in Fig. 6.20. Inspection of the topographic map of each subbasin will reveal some variations of this value

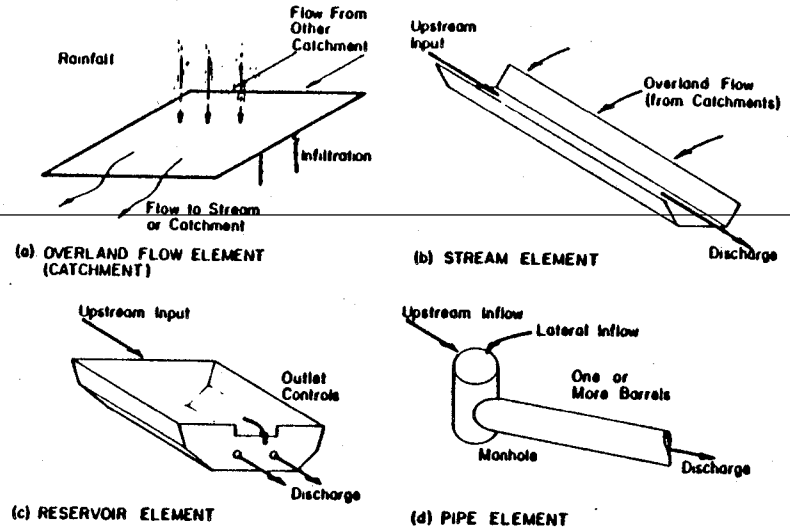


Figure 6.19. Typical MITCAT elements. (Reproduced from Ref. 31.)

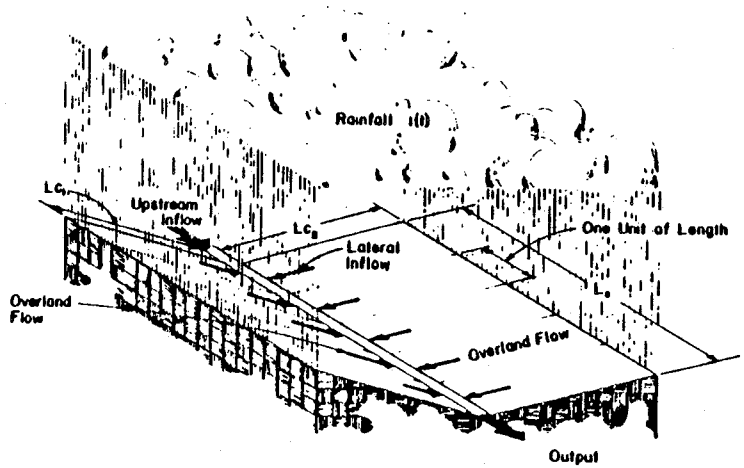


Figure 6.20. Simple catchment-stream elements used to model a segment. (Reproduced from Ref. 31.)

over the subbasin. It will be found, however, that for most natural basins, a surprisingly constant value of Lc can be found when one has determined what the full drainage network within the basin looks like. This network includes not only the main stem streams, but also the multitude of smaller tributaries and rivulets which act as channels under significant rainfall events. Such a schematic is shown in Fig. 6.21. The correct selection of the value(s) of this overland flow length Lc is essential for the adequate modeling of the basin response. This parameter is by far the most important variable for determining the response characteristic of the overland flow segment. Since these elements in turn act as the primary high frequency and limited bandwidth filters in the simulation network, their accurate representation is crucial to the ability of the chosen schematic to simulate the prototype system.

Table 6.11 gives Manning's n for overland flow roughness effects. Since the selection of the correct overland flow length is so critical, various internal arrangements of the segment structure will be required in order to adequately include the range of overland flow lengths which may occur in the prototype basin. In certain cases, a very simple schematic as shown in Fig. 6.22a will suffice. The more general case is shown in Fig. 6.22b, where an overland flow element is modeled in each side of the main stem. In this case, the overland flow lengths for the elements may be the same or, as more generally found, somewhat different. This schematic is also able to handle the normal variations of the other parameters such as slope and roughness as they occur.

A more complex configuration is illustrated in Fig. 6.22c. This would be used to model an urban segment where a number of different flow situations exist (Fig. 6.23). Elements would include:

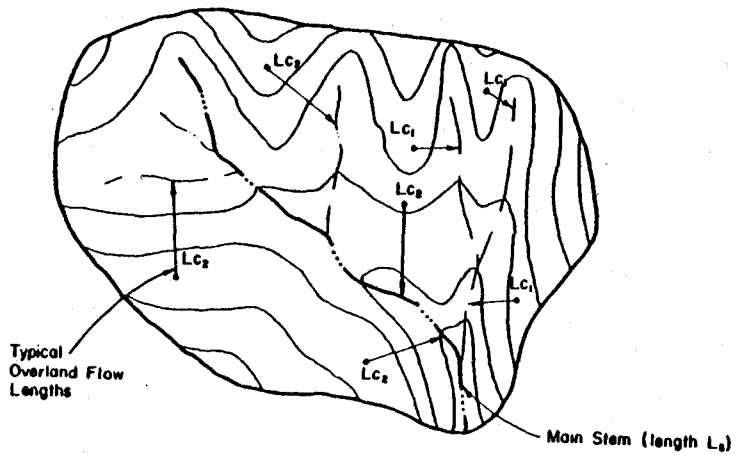
Flow from impervious roadway and roof surfaces
Flow from a pervious lawn accepting runoff from commercial areas
Flow from impervious commercial/parking areas

Roof drains normally discharge directly into pipe systems in Melbourne, and so roof and street areas are combined into one element. Figure 6.23c is

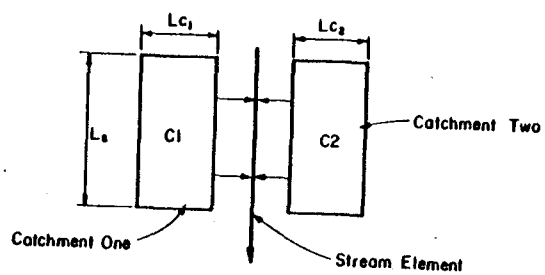
Table 6.11 Manning's n for Overland Flow

Surface	n
Dense growth	0.4 -0.5
Pasture	0.3 -0.4
Lawns	0.2 -0.3
Concrete/asphalt:	
Shallow depths	0.10-0.15
Small depths	0.05-0.10[a]

[a]Significant flow occurring over the surface.
Source: U.S. Forest Service [13].



(a) PROTOTYPE WATERSHED



(b) MODEL SCHEMATIC

Figure 6.21. (a) Prototype watershed; (b) typical model schematic for natural watershed. (Reproduced from Ref. 31.)

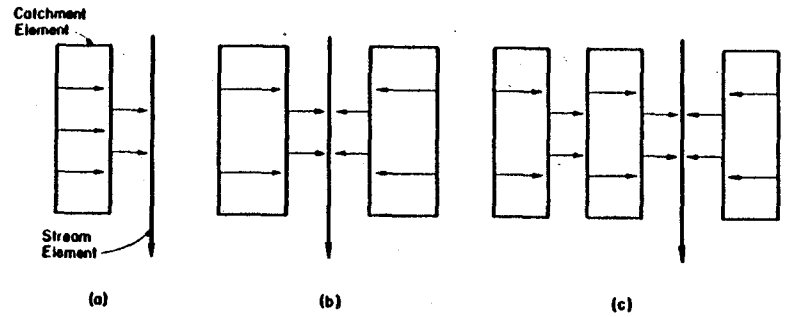
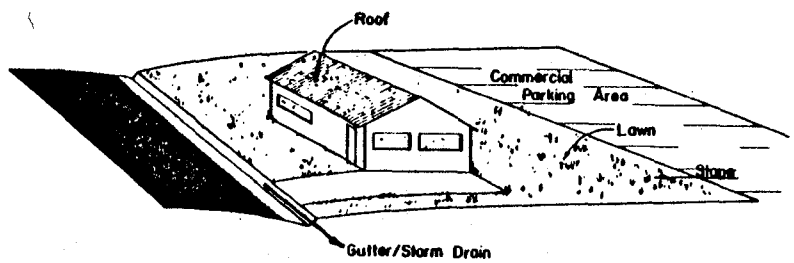
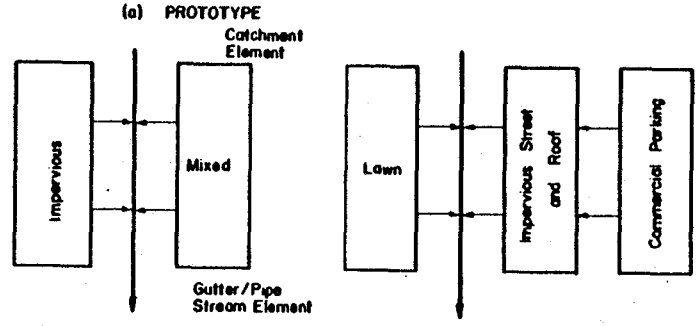


Figure 6.22. Typical stream segment structures. (Reproduced from Ref. 31.)



(a) PROTOTYPE



(b) SCHEMATIC ONE (c) SCHEMATIC TWO

Figure 6.23. Typical model of urban area. (Reproduced from Ref. 31.)

presented to illustrate a more complex model for some applications. Segments shown in Fig. 6.23b and c have been successfully used for subbasins up to 400 hectares (1000 acres) in size.

Stream Element

In each of the schematics, a *stream* element is considered to be a stream reach of uniform cross section and to have a length of L.s.

The cross section is usually considered to be a simple shape as shown in Fig. 6.24 where the basic shapes currently available for simulation with MITCAT are illustrated. The model is not especially sensitive to the cross section chosen where discharge simulation is involved; however, stage, velocity, and other derived data are sensitive to the shape chosen. From operational use of MITCAT, some general rules have emerged for the appropriate section shapes to use.

It is found that most upstream reaches are best simulated by use of simple triangular shapes. Many of these channels are extremely small, and any flood flow uses the overbank area extensively. Such overbank areas are frequently triangular in cross section–see Fig. 6.25a.

In further downstream reaches, simulation with basically trapezoidal channel sections is found to be most appropriate–see Fig. 6.25b. In such cases, care must be taken to fit the most appropriate cross-section shape for the range of flood stages of interest.

Urban areas sometimes have gutter sections as their primary drain element. In these segments, use of a gutter shape as in Fig. 6.25c will be desirable. However, for the simulation of larger urban areas, where storm drains are used, a stream segment with a circular shape (Fig. 6.24e) should be used. This particular shape actually simulates the behavior of a regular conduit up to the 90 percent full position. Flows greater than that are modeled by assuming that the capacity of the element increases as required, i.e., no pressure flow and/or surcharging occurs. Use of a circular cross section is appropriate since the area being modeled does not normally drain to a single storm drain; rather, it consists of a whole series of storm drains connected in a relatively complex network. However, it has been found possible to duplicate the whole network by a *single* stream/pipe element as shown in Fig. 6.23b or c.

As an area is urbanized, its drainage network simulation will also change. A typical staged development of a segment schematic is shown in Fig. 6.26. In this situation, the undeveloped (natural) basin can be typically modeled using a single segment, with a simple catchment-stream-catchment structure of the segment. As the basin is urbanized, more and more of the area is provided with storm drains (including gutters) and a different set of drainage controls exists. The easiest way to simulate the runoff from such urbanizing areas is by using a segment in parallel to the segment modeling the runoff from the natural system–see Fig. 6.26b. The discharges from both of these segments are combined to estimate the total flow from the basin. In the fully developed case, the runoff from the total basin may well be simulated by the urban segment and only the upstream flow routed through the main stem. The latter may, of course, also be modified in order to handle the increased discharge levels, and so forth.

As discussed above, a range of relatively simple cross-section shapes are used to model prototype stream channels. These shapes are normally sufficient to enable adequate determination of the discharge rate, and were

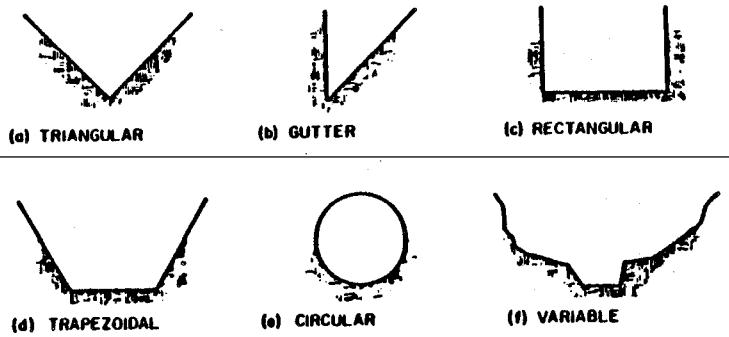


Figure 6.24. Stream shapes. (Reproduced from Ref. 31.)

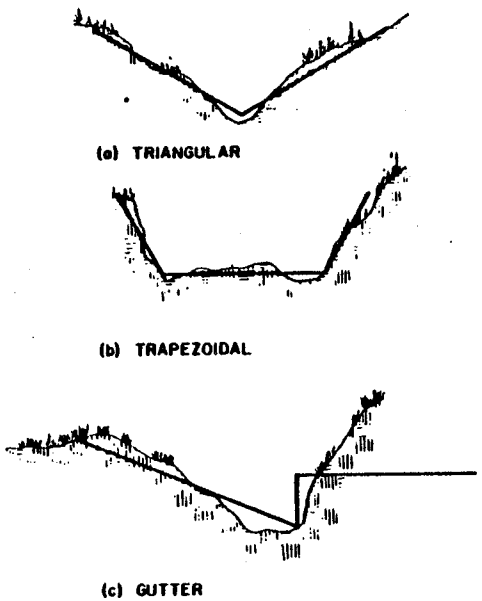


Figure 6.25. Selection of stream cross section. (Reproduced from Ref. 31.)

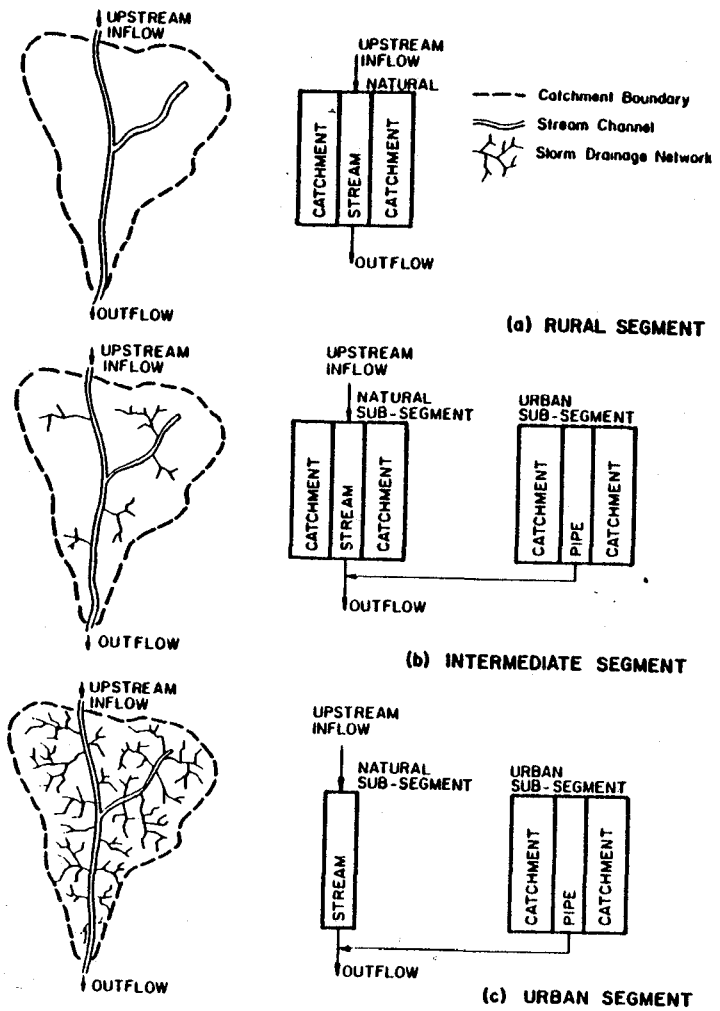


Figure 6.26. Typical urbanization of a segment. (Reproduced from Ref. 31.)

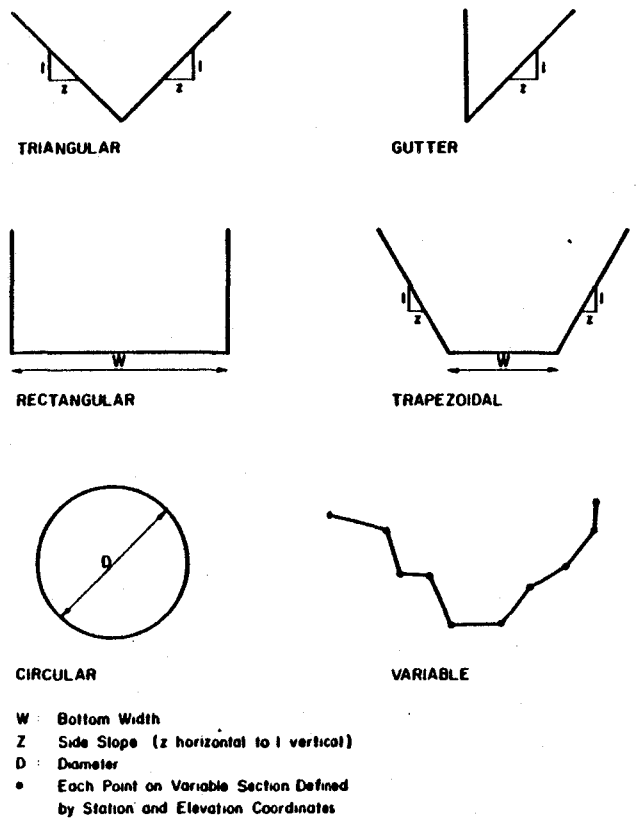
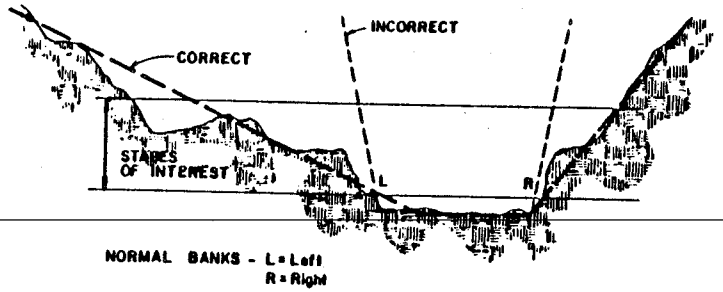


Figure 6.27. Stream cross-section definitions. (Reproduced from Ref. 31.)

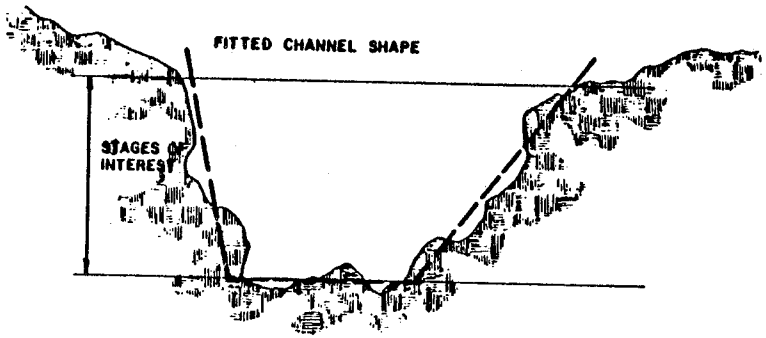
chosen because simulation with such simple elements is extremely fast. They will not, of course, be sufficiently detailed to allow accurate determination of the equivalent flow stages. In this instance, other techniques are available within MITCAT to permit adequate stage determination.

Variable. Available cross sections are illustrated in Fig. 6.27 where their significant parameters are noted. It should be noted that the parameters used include the cross-section shapes as well as the segment slope S and the effective Manning n. The value of n appropriate for stream segments can be estimated from field inspection or from standard hydraulic handbooks.

Triangular Sections. These sections will be very useful for simulating small natural streams, i.e., where permanent flow is very small compared to flood flows, and streams that flow through unimproved valleys. Certain man-made channels may also fit this category.



(a) SIGNIFICANT OUT-OF-BANKS FLOW



(b) CONSTRICTED SECTIONS

Figure 6.28. Fitting trapezoidal cross section to natural channels. (Reproduced from Ref. 31.)

Gutter Sections. These sections are of use in urban areas where significant flow in the gutters is to be modeled.

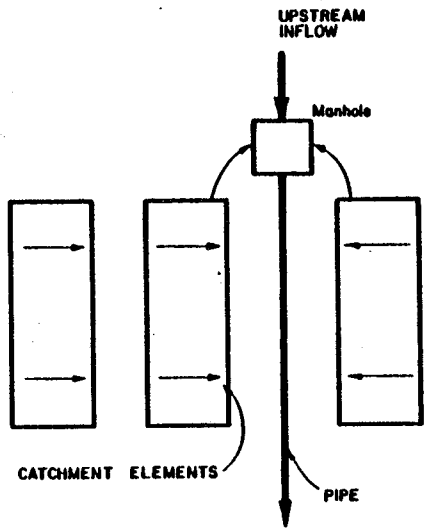
Rectangular Section. This section shape is of most use in simulating man-made stream segments where the flow is expected to stay in banks for the event being considered. It is preferable to use either a triangular or a trapezoidal section to model most prototype conditions.

Trapezoidal Sections. This section shape is of interest in modeling the stream channels occurring in the downstream part of river basins. It is very important in this instance to choose the correct side slopes to adequately handle the full range of flow stages. Figure 6.28 illustrates the choices available.

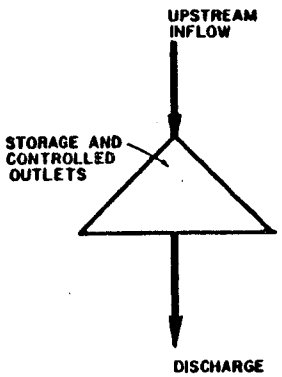
Circular Sections. Modeling the storm drainage system in urban areas requires the capability of simulating the response characteristics of stream segments with a circular cross-section slope. In many cases, the pipe will never fill completely during a storm event and the routing through the pipe will resemble normal stream routing. The method allows the pipe (or circular cross-section stream segment) to continue to respond even under surcharge conditions; i.e., there is no upper limit to the capacity of the stream segment. This method has proved very useful in simulating the response of urban basins without the need to specify the complete internal drainage system. Instead, a "typical" segment is specified and allowed to handle as much flow as required. To some extent, this duplicates the behavior of the prototype where runoff that is not accepted by catch basins usually flows over the surface until it either enters the storm drain system or is discharged to a natural watercourse.

Reservoir and Pipe Elements

The other simulating elements considered are the *reservoir* and *pipe* elements for which simple schematics are shown in Fig. 6.29a and b. Both elements allow modeling of on-line storage. The storage is considered explicitly in the reservoir elements. In pipe elements, each element (which can consist of a number of individual barrels) has an upper limit on its discharge



(a) PIPE SEGMENT WITH PIPE AND CATCHMENTS



(b) RESERVOIR SEGMENT

Figure 6.29. Typical reservoir and pipe segments. (Reproduced from Ref. 31.)

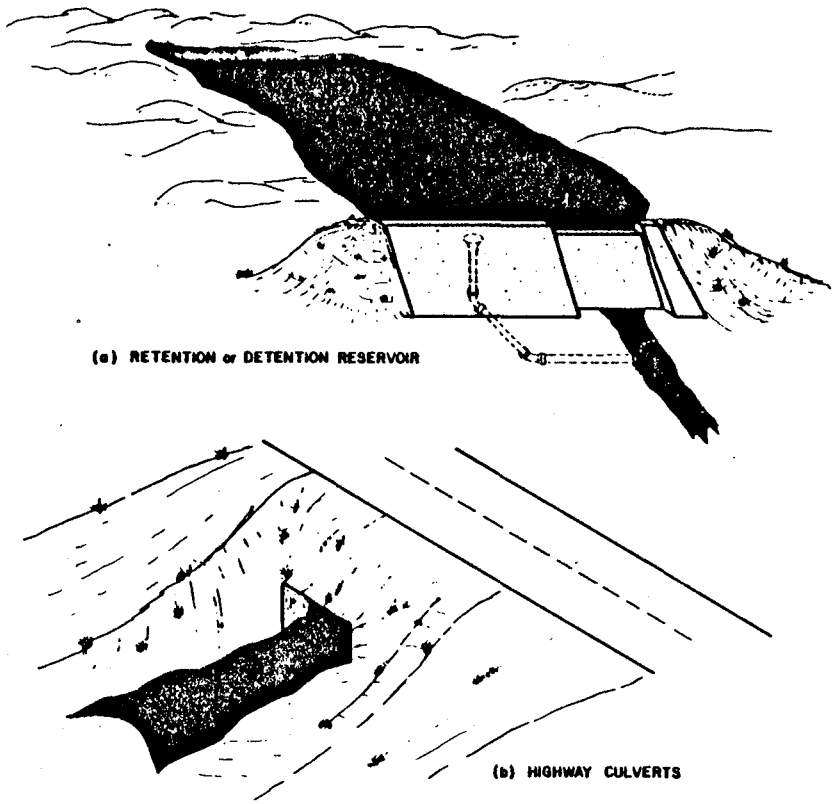


Figure 6.30. Uses of reservoir elements. (Reproduced from Ref. 31.)

capacity. The model thus allocates a "manhole" storage volume upstream of the element to regulate flow into the pipe element. All flows in excess of the pipe's capacity are stored and released to the pipe as capacity becomes available. The pipe element will be discussed in detail later.

Reservoir elements are used to model a number of commonly occurring situations in the basin–see Fig. 6.30. These normally include:

Retention or detention reservoir
Highway culverts, with the embankment acting to form a storage volume under flood conditions

The operation of each of these systems can normally be simulated through the use of relatively simple rules.

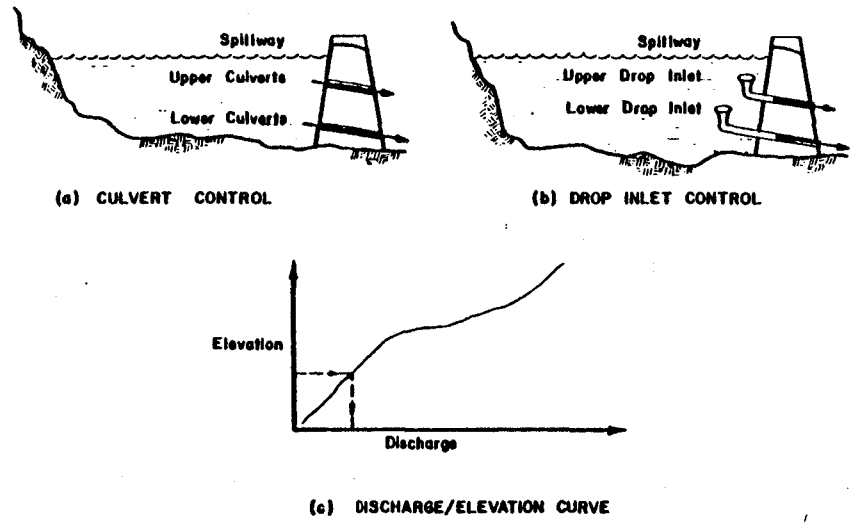


Figure 6.31. Reservoir outlet controls. (Reproduced from Ref. 31.)

The outlet control conditions which can currently be simulated are shown in Fig. 6.31. Basically, the controls can consist of up to two outlets, each of which is either a simple pipe or a drop-inlet (glory hole) type. Combined with these is an emergency spillway. The operation of these outlets is solely a function of the water surface elevation in the reservoir at any given time. In order to provide the ability to simulate more complex conditions, the user can specify a discharge vs. elevation relation which the model is to follow. The above set of controls has been found adequate to simulate the operation of all the major on-stream reservoirs found in urban drainage systems. Routing through such reservoirs is based on a normal input/output/storage relationship, i.e., the discharge at a given time is solely a function of the available "head" in the reservoir and the characteristics of the outlet controls.

A pipe element could be useful for the detailed modeling of small urbanized areas–see Fig. 6.32. In these situations, the user may need to know the flow and grade line elevation at each manhole within the system–the pipe element provides this ability. The pipe element can consist of a number of parallel barrels of pipe section, each with a similar cross section.

The schematic shown in Fig. 6.32b shows that a pipe element can have lateral inflow from adjacent catchment elements. This lateral flow is not uniformly distributed along the pipe's length, as would be the case with a stream segment, but the total flow is input to the manhole which is assumed to be at the upstream end of each pipe. The manhole itself has a number of parameters (such as area and elevation) associated with it so that the model can adequately simulate the processes such as surcharging and spill to adjacent areas.

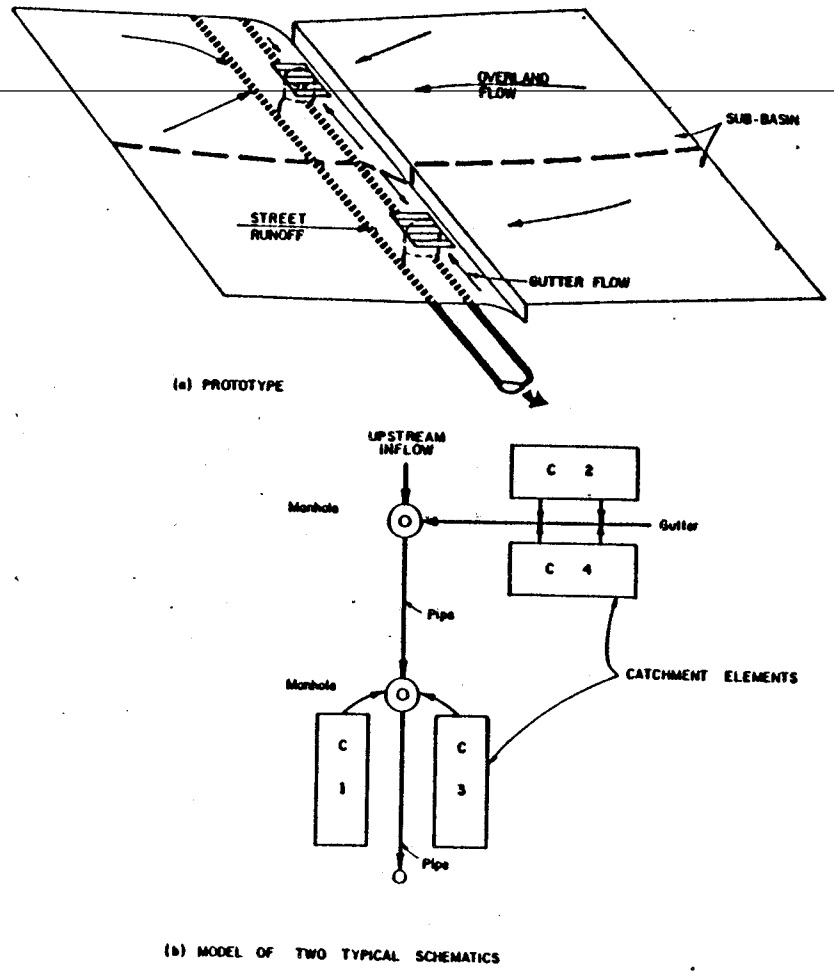


Figure 6.32. Use of pipe elements. (Reproduced from Ref. 31.)

Other Capabilities

MITCAT has other algorithms which are useful in various situations. These include situations such as:

- Off-stream storage
- Splitting flow
- Backwater situations
- Many small streams entering a major stream
- Stage discharge checks

These and other unusual situations are covered in the user's manual [31].

Some Other Useful Guidelines

by

William C. Taggart

1. Natural Overland flow lengths do not often exceed 1000 feet and almost never exceed 2000 feet. The fundamental reason is that the sheet flow is often collected into small confined flows where the Manning's n must change.

2. Urban Overland flow lengths do not often exceed 300 feet for mixed (house and lot - 75 to 200 is normal) catchments or parking lots, or 50 feet for streets (25-40 is normal). For streets, measure from crown diagonally along fall line to gutter. For lots, measure from high points, typically in the middle of back yard depending on grading.

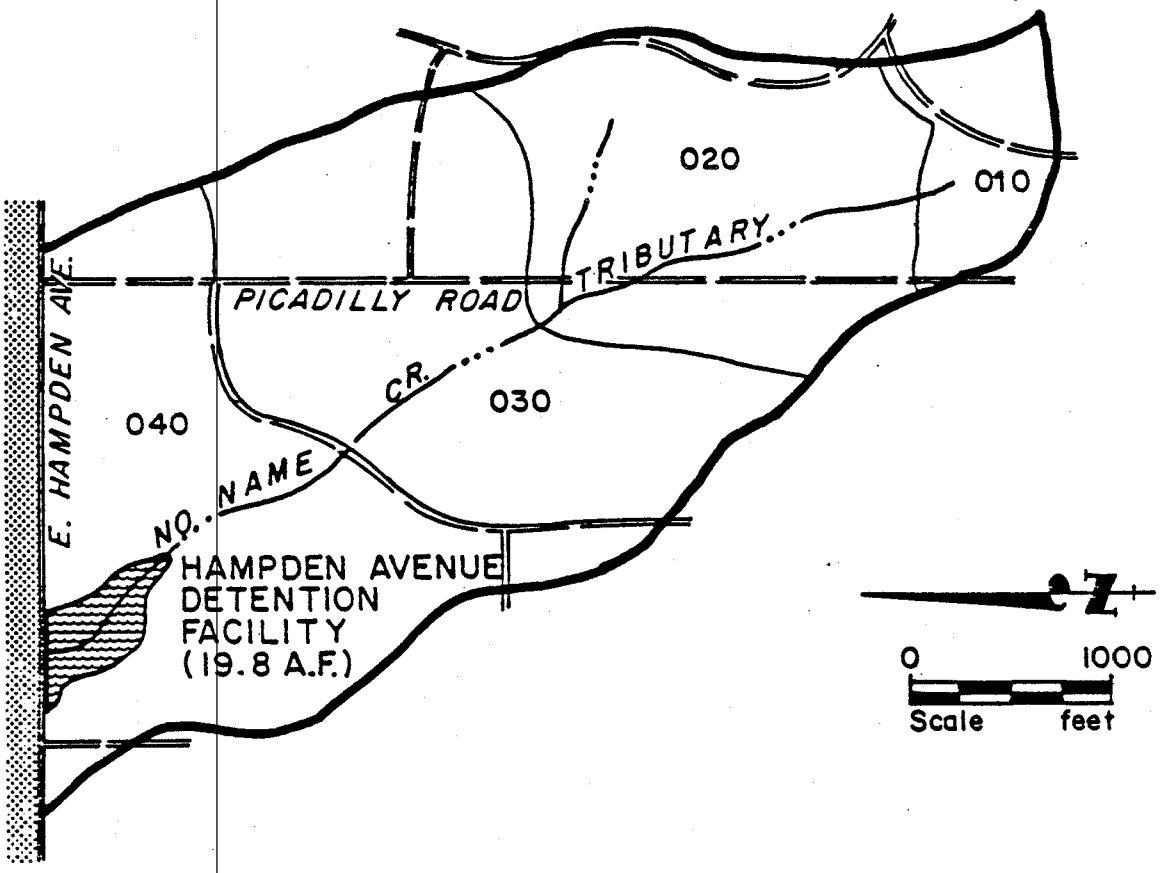
3. Measure overland flow lengths from divides, or in the case of urban areas, other relative high points.

4. Watch relationship of ratio of effective lengths to true lengths. Should not exceed 3:1 or be less than 1/3:1 for catchments, and 4:1 or 1/4:1 for streams.

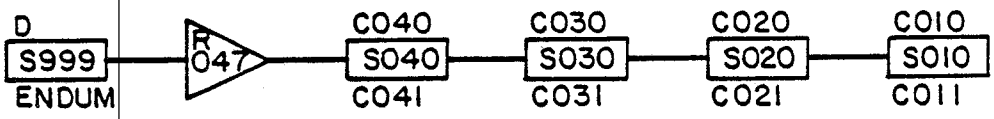
5. Use check runs and **check areas are correct.**

6. **Check, Check, Check** Volumes. This will reveal 90% of the modeling errors.

7. When in doubt about modeling strategy, prepare a more detailed test model and compare its result against the base model (applicable portion thereof) for appropriate frequencies. Generally, only a few segments are required to a given point of interest and thus simple models can be used. However, if the test model gives significantly different results, the more detailed approach should probably be used.



NO NAME CREEK TRIBUTARY BASIN



DEVELOPED CONDITION

MITCAT MODEL FLOWCHART

LEGEND

C030 —— *LOT CATCHMENT*

S030 —— *STREAM ELEMENT*

C031 —— *STREET CATCHMENT*

D
S031 —— *DUMMY STREAM ELEMENT*

****

C A T C H M E N T A N A L Y S I S

BRENDAN M. HARLEY MASSACHUSETTS INSTITUTE OF TECHNOLOGY 1972

NO NAME CREEK TRIB 100-YEAR DEVELOPED CONDITION W/ R047 HYDROLOGY (MAY 24, 1985)

D A T E : 07/08/85 VERSION 9 B 7/79 T I M E : 15.17.27

****

RESOURCE ANALYSIS, INC.

235 WYMAN STREET

WALTHAM, MASS. 02154, USA

```
STREAM NUMB 010 NAME MAINS010 A-LENG 680  LENG 2631.75 SLOP .042 GUTT Z 30
                    N .025        FINISH
STREAM NUMB 020 NAME MAINS020 A-LENG 1910 LENG 7869.84 SLOP .032 GUTT Z 30
                    N .025        FINISH
STREAM NUMB 030 NAME MAINS030 A-LENG 1310 LENG 9379.92 SLOP .024 TRAP Z 14
          W 65      N .035        FINISH
STREAM NUMB 040 NAME MAINS040 A-LENG 1920 LENG 10759.32 SLOP .016 TRIA Z 15
                    N .035        FINISH
RESERV NUMB 047 NAME REGIN047 ALPHA .00762 M .532 ZP1 5688.0 ZSPILL 5696.1
      D1 33 SLENG 13.8 ZREF 5688.0 CSPILL 3. Z0 5688.0 NB1 1 OUTLET INLET
      CD1 .578    FINISH
STREAM NUMB 999 NAME ENDUM999 DUMMY FINISH
CATCH NUMB 010 NAME LOTS010 A-LENG 170 LENG 240 SLOP .04 N .3 INFIL SCS
                    CURVE 74 ZIMP 74.3      FINISH
CATCH NUMB 011 NAME STRET011 A-LENG 25 LENG 43.03 SLOP .03 N .15 INFIL SCS
                    CURVE 74 ZIMP 90        FINISH
CATCH NUMB 020 NAME LOTS020 A-LENG 150 LENG 300 SLOP .04 N .3 INFIL SCS
                    CURVE 74 ZIMP 35        FINISH
CATCH NUMB 021 NAME STRET021 A-LENG 25 LENG 74.72 SLOP .03 N .15 INFIL SCS
                    CURVE 74 ZIMP 90        FINISH
CATCH NUMB 030 NAME LOTS030 A-LENG 150 LENG 300 SLOP .04 N .3 INFIL SCS
                    CURVE 74 ZIMP 35        FINISH
CATCH NUMB 031 NAME STRET031 A-LENG 25 LENG 75.23 SLOP .03 N .15 INFIL SCS
                    CURVE 74 ZIMP 90        FINISH
CATCH NUMB 040 NAME LOTS040 A-LENG 150 LENG 300 SLOP .04 N .3 INFIL SCS
                    CURVE 74 ZIMP 35        FINISH
CATCH NUMB 041 NAME STRET041 A-LENG 25 LENG 74.90 SLOP .03 N .15 INFIL SCS
                    CURVE 74 ZIMP 90        FINISH
NODE  NODE 999 S 999  NODE 047 R 047  NODE 040 C 040 S 040 C 041
     NODE 030 C 030 S 030 C 031      NODE 020 C 020 S 020 C 021
     NODE 010 C 010 S 010 C 011 FINISH
CONNECTIVITY NODE 999 LINK 047
            NODE 047 LINK 040  NODE 040 LINK 030  NODE 030 LINK 020
            NODE 020 LINK 010  FINISH
HYDROGRAPH PRINT RESERV OUTFLOW 047  RESERV UPSTREAM 047 FINISH
HYDROGRAPH HOUR FINISH
INFLOW TYPE 5 BLOCK DELT 5 INTENSITY  0. 0.36 0.6 1.44 2.52 4.2 2.16 0.96
0.72 0.6 0.6 0.48 0.48 0.48 0.48 0.36 0.36 0.36 0.36 0.36 0.24 0.24 0.24 0.24
0.24 0. FINISH
INFLOW TYPE 10 BLOCK DELT 5 INTENSITY 0. 0.36 0.72 1.68 3. 4.92 2.4 1.08 0.84
0.72 0.72 0.72 0.6 0.6 0.6 0.6 0.48 0.48 0.36 0.36 0.36 0.36 0.36 0.36 0.24 0.
       FINISH
INFLOW TYPE 100 BLOCK DELT 5 INTENSITY 0. 0.36 0.96 1.44 2.52 4.44 8.04 4.44
2.52 1.92 1.56 1.32 1.32 1.32 0.6 0.6 0.48 0.48 0.36 0.36 0.36 0.36 0.36 0.36
0.36 0. FINISH
CONTROL DTFIX 5.  TMAX 180.  FINISH
CHANGE CATCH INFLOW 100 FINISH
DISPLAY ALL FINISH
60

*** CONNEC *** AUTO SET OF TERMINATING ELEMENT..
              RUN WILL NOW TERMINATE AT STREAM  NO.   999 IN NODE   999 ...

*** CONNEC *** OUTPUT FROM FINAL ELEMENT — (( STREAM     999 )) — RESET TO DISPLAY OUTPUT....
```

SEGMENT CONNECTIVITY :

```
**********
*MAINS010*
**********
   \
   \
   V
**********
*MAINS020*
**********
   \
   \
   V
**********
*MAINS030*
**********
   \
   \
   V
**********
*MAINS040*
**********
   \
   \
   V   H
*********
*REGIN047*
*********
   \   H
   \
   V
**********
*ENDUM999*
**********
   \   H
   \
   V
 /FINISH/
```

NODE INFRA-STRUCTURE :

NAME	NODE NO	SCHEMATIC	TOTAL AREA (ACRES)	IMPERVIOUS AREA (ACRES)	PERVIOUS AREA (ACRES)
ENDUM999	999	9999	.00	.00	.00
MAINS010	10	C 10 — S 10 — C 11	17.10	13.11	3.99
MAINS020	20	C 20 — S 20 — C 21	67.70	31.12	36.58
MAINS030	30	C 30 — S 30 — C 31	80.80	37.19	43.61
MAINS040	40	C 40 — S 40 — C 41	92.60	42.59	50.02
REGIN047	47	R 47	.00	.00	.00
			???????	???????	???????
	TOTALS FOR THIS RUN :	AREAS IN ACRES :	258.20	124.01	134.19
		AREAS IN SQ. MILES :	.4034	.1938	.2097

STREAM GEOMETRY DATA :

NUMBER	NAME	COORDINATES X	Y	Z	ACT.LENGTH (FT)	SLOPE	GRADIENT (1 FT. IN)	X-SECTION SHAPE	WIDTH/DIAM (FT)	SIDE SLOPE (1 FT. IN)
999	ENDUM999	-	-	-		— DUMMY —		TRIANGULAR	-	25.0
10	MAINS010	-	-	-	680.00	.04200	23.81	GUTTER	-	30.0
20	MAINS020	-	-	-	1910.00	.03200	31.25	GUTTER	-	30.0
30	MAINS030	-	-	-	1310.00	.02400	41.67	TRAPEZOID	65.00	14.0
40	MAINS040	-	-	-	1920.00	.01600	62.50	TRIANGULAR	-	15.0

STREAM DATA - KINEMATIC :

NUMBER	NAME	EFF.LENGTH (FT)	MANNING (N)	ALPHA (ACTUAL)	ALPHA (EFFECT.)	M	ROUTING SCHEME	CRITICAL CONDITIONS VEL(FT/S)	DISCH(CFS)	STAGE(FT)
999	ENDUM999	-	.015	.303	.303	1.333	— DUMMY —	-	-	-
10	MAINS010	2631.750	.025	3.050	11.803	1.333	F.DIFF KINEMATIC	-	-	-
20	MAINS020	7869.840	.025	2.662	10.968	1.333	F.DIFF KINEMATIC	-	-	-
30	MAINS030	9379.920	.035	.780	5.584	1.450	F.DIFF KINEMATIC	-	-	-
40	MAINS040	10759.320	.035	1.375	7.707	1.333	F.DIFF KINEMATIC	-	-	-

STREAM DATA - EROSION PARAMETERS :

NUMBER	NAME	PLASTIC INDEX	GRAIN SIZE (MM)	VEGETATED (%)	LIMITING TRACTIVE FORCES (LB/SQ.FT) WET	NO-VEG	100%-VEG	ACTUAL	INDICATOR CHECKED
999	ENDUM999	12.0	-	.0	-	-	-	-	VELOCITY
10	MAINS010	12.0	-	.0	-	-	-	-	VELOCITY
20	MAINS020	12.0	-	.0	-	-	-	-	VELOCITY
30	MAINS030	12.0	-	.0	-	-	-	-	VELOCITY
40	MAINS040	12.0	-	.0	-	-	-	-	VELOCITY

CATCHMENT GEOMETRY DATA :

NUMBER	NAME	COORDINATES X	Y	Z	ACT.LENGTH (FT)	SLOPE	GRADIENT (1 FT. IN)	% IMPERVIOUS	%SEWERED
10	LOTS010	-	-	-	170.00	.04000	25.00	74.30	-
20	LOTS020	-	-	-	150.00	.04000	25.00	35.00	-
30	LOTS030	-	-	-	150.00	.04000	25.00	35.00	-
40	LOTS040	-	-	-	150.00	.04000	25.00	35.00	-
11	STRET011	-	-	-	25.00	.03000	33.33	90.00	-
21	STRET021	-	-	-	25.00	.03000	33.33	90.00	-
31	STRET031	-	-	-	25.00	.03000	33.33	90.00	-
41	STRET041	-	-	-	25.00	.03000	33.33	90.00	-

CATCHMENT DATA - KINEMATIC :

NUMBER	NAME	EFF.LENGTH (FT.)	MANNING (N)	ALPHA (ACTUAL)	ALPHA (EFFECT.)	M	ROUTING SCHEME	EVAPORATION PARAMETERS ROUGHNESS	STOMATAL RESIS.
10	LOTS010	240.000	.300	.993	1.402	1.667	F.DIFF KINEMATIC	-	-
20	LOTS020	300.000	.300	.993	1.987	1.667	F.DIFF KINEMATIC	-	-
30	LOTS030	300.000	.300	.993	1.987	1.667	F.DIFF KINEMATIC	-	-
40	LOTS040	300.000	.300	.993	1.987	1.667	F.DIFF KINEMATIC	-	-
11	STRET011	43.030	.150	1.721	2.961	1.667	F.DIFF KINEMATIC	-	-
21	STRET021	74.720	.150	1.721	5.142	1.667	F.DIFF KINEMATIC	-	-
31	STRET031	75.230	.150	1.721	5.177	1.667	F.DIFF KINEMATIC	-	-
41	STRET041	74.900	.150	1.721	5.155	1.667	F.DIFF KINEMATIC	-	-

CATCHMENT DATA - INFLOWS :

NUMBER	NAME	LATERAL INFLOW GAUGE	STORM	FILE	TIME	INFLOW SCALING TIME	INTEN.	INITIAL LAG (MIN)	INFILTRATION PARAMETERS TYPE	1	2	3	DETN	
10	LOTS010	0	100	-	-	-	-	-	SCS	74.00	3.51	-	.70	AMC 2
20	LOTS020	0	100	-	-	-	-	-	SCS	74.00	3.51	-	.70	AMC 2
30	LOTS030	0	100	-	-	-	-	-	SCS	74.00	3.51	-	.70	AMC 2
40	LOTS040	0	100	-	-	-	-	-	SCS	74.00	3.51	-	.70	AMC 2
11	STRET011	0	100	-	-	-	-	-	SCS	74.00	3.51	-	.70	AMC 2
21	STRET021	0	100	-	-	-	-	-	SCS	74.00	3.51	-	.70	AMC 2
31	STRET031	0	100	-	-	-	-	-	SCS	74.00	3.51	-	.70	AMC 2
41	STRET041	0	100	-	-	-	-	-	SCS	74.00	3.51	-	.70	AMC 2

RESERVOIR GEOMETRY DATA :

NUMBER	NAME	STORAGE/ELEVATION			SPILLWAY			LOWER OUTLETS			UPPER OUTLETS		
		ALPHA	M	REF.ELEVN (FT)	ELEVN (FT)	LENGTH (FT)	CD	ELEVN (FT)	DIMENSIONS (FT)	NO	ELEVN (FT)	DIMENSIONS (FT)	NO
47	REGIN047	.008	.5320	5688.00	5696.10	13.80	3.00	5688.00	2.75	1	-	-	-

RESERVOIR OPERATION :

NUMBER	NAME	ROUTING	OUTLET TYPE	DISCHARGES ADDITIVE ?	INITIAL WSE (FT)	CONSTANT DISCHARGES LOWER (CFS)	CONSTANT DISCHARGES UPPER (CFS)	LOWER OUTLET LEN/ELEVN* (FT)	LOWER OUTLET SLPE/DIAM* -/(IN)	UPPER OUTLET LEN/ELEVN* (FT)	UPPER OUTLET SLPE/DIAM* -/(IN)
47	REGIN047	STANDARD	INLET CONTRL	YES	5688.00	-	-	-	-	-	-

RESERVOIR - INFLOWS :

NUMBER	NAME	UPSTREAM INFLOW					INFLOW SCALING		OUTLETS		DISCHARGE COEFFS	
		SEGMENT	ELEMENT	NUMBER	FILE	TIME	TIME	INTEN.	KE	MANNING N	LOWER	UPPER
47	REGIN047	-	-	-	-	-	-	-	-	-	.58	.00

```
*****************************************************************************
*                                                                           *
*     R E S E R V O I R:  47 - - REBIN047 - - UPSTREAM INFLOW               *
*                                                                           *
*****************************************************************************
```

TIME (HOUR)	INFLOW (CUSECS)
.000	.000000
.083	.000000
.167	1.970631
.250	17.312646
.333	56.212445
.417	132.329567
.500	280.083199
.583	611.483894
.667	857.131248
.750	755.691673
.833	568.872551
.917	432.174427
1.000	345.304165
1.083	296.766396
1.167	276.480171
1.250	240.917103
1.333	189.210209
1.417	150.570267
1.500	123.916122
1.583	105.914030
1.667	92.373105
1.750	83.974640
1.833	79.078863
1.917	76.668296
2.000	75.967502
2.083	75.897806
2.167	65.945757
2.250	46.880061
2.333	33.733066
2.417	22.998391
2.500	13.123954
2.583	7.923126
2.667	5.200561
2.750	3.701913
2.833	2.803871
2.917	2.214685
3.000	1.798662

```
*****************************************************************************
```

```
*****************************************************************************
*                                                                           *
*            R E S E R V. :  47 - - REGIN047 - - OUTPUT                     *
*                                                                           *
*****************************************************************************
```

TIME (HOUR)	DISCHARGE (CUSECS)
.000	.00
.083	.00
.167	.52
.250	3.42
.333	10.32
.417	23.33
.500	39.60
.583	60.02
.667	79.43
.750	170.74
.833	237.82
.917	272.36
1.000	285.56
1.083	287.59
1.167	285.58
1.250	277.48
1.333	261.62
1.417	241.94
1.500	221.38
1.583	201.71
1.667	183.52
1.750	167.40
1.833	153.51
1.917	141.76
2.000	132.02
2.083	123.94
2.167	115.88
2.250	106.68
2.333	97.52
2.417	88.90
2.500	81.13
2.583	74.97
2.667	71.47
2.750	70.41
2.833	69.42
2.917	68.40
3.000	67.36

```
*****************************************************************************
```

END OF RUN FOR 07/08/85 AT 15.17.27

SYSTEM TERMINATED AT STREAM NUMBER 999 IN NODE 999

(REQUESTED TERMINATION WAS AT STREAM NUMBER 999)

TOTAL RUN VOLUMES :

INIT. SURF. VOLUME :	.0000		AC.FT.
UPSTREAM INFLOW :	.0000		AC.FT.
LATERAL INFLOW :	66.0501		AC.FT.
TOTAL INFLOW :		66.0501	AC.FT.
ROUTED OUTFLOW :	32.6524		AC.FT.
VOLUME INFILTRATED :	23.6723		AC.FT.
VOLUME SPILL DUMPED:	.0000		AC.FT.
TOTAL OUTFLOW :		56.3247	AC.FT.
REM. SURF. VOLUME :	9.4656		AC.FT.
VOLUME EVAPORATED :	.0000		AC.FT.
TOT. ACCOUNTED FOR :		65.7903	AC.FT.

RUN LOSSES :

VOLUME LOST ... :	.2597	AC.FT.
	.3933	PER CENT.

— RUN TERMINATED ON 07/08/85 AT 15.17.34 —

FINAL SUMMARY OF VOLUMES:

NUMBER	NAME	TIME STEP (MINS)	INFLOW VOLUMES SURFACE (AC.FT)	INFLOW VOLUMES UPSTREAM (AC.FT)	INFLOW VOLUMES LATERAL (AC.FT)	INFILTRATION VOLUME (AC.FT)	INFILTRATION PERCENT (INFLOW)	OUTFLOW VOLUMES RUNOFF (AC.FT)	OUTFLOW VOLUMES PERCENT (INFLOW)	OUTFLOW VOLUMES SURFACE (AC.FT)	OUTFLOW VOLUMES SPILLED (AC.FT)
CATCHMENTS											
10	LOTS010	5.00	.00	.000000	.001410	.000250	17.72	.001151	81.67	.000008	
20	LOTS020	5.00	.00	.000000	.001762	.000790	44.82	.000969	54.98	.000002	
30	LOTS030	5.00	.00	.000000	.001762	.000790	44.82	.000969	54.98	.000002	
40	LOTS040	5.00	.00	.000000	.001762	.000790	44.82	.000969	54.98	.000002	
11	STRET011	5.00	.00	.000000	.000253	.000017	6.90	.000235	93.13	.000000	
21	STRET021	5.00	.00	.000000	.000439	.000030	6.90	.000409	93.13	.000000	
31	STRET031	5.00	.00	.000000	.000442	.000030	6.90	.000411	93.13	.000000	
41	STRET041	5.00	.00	.000000	.000440	.000030	6.90	.000410	93.13	.000000	
	TOTALS :		.00	.000000	.008269	.002728	32.99	.005522	66.79	.000014	
STREAMS											
999	ENDUM999	.00	.00	32.65	.00	.00	.00	32.65	100.00	.00	.00
10	MAINS010	5.00	.00	.00	3.65	.00	.00	3.65	99.98	.00	.00
20	MAINS020	5.00	.00	3.65	10.84	.00	.00	14.48	99.97	.00	.00
30	MAINS030	5.00	.00	14.48	12.94	.00	.00	27.42	99.97	.01	.00
40	MAINS040	5.00	.00	27.42	14.83	.00	.00	42.23	99.97	.01	.00
	TOTALS :		.00	78.20	42.26	.00	.00	120.43	99.98	.03	
RESERVOIRS											
47	REGIN047	5.00	.00	42.23	.00	.00	.00	32.65	77.32	9.35	.00
	TOTALS :		.00	42.23	.00	.00	.00	32.65	77.32	9.35	

CATCHMENTS:

NUMBER	NAME	MAXIMUM VALUES OUTFLOW (CFS)	MAXIMUM VALUES STAGE (FT)	TIME TO PEAK (MINS)
10	LOTS010	.02619	.0918	35.00
11	STRET011	.00703	.0266	35.00
20	LOTS020	.02006	.0635	40.00
21	STRET021	.01246	.0270	35.00
30	LOTS030	.02006	.0635	40.00
31	STRET031	.01255	.0270	35.00
40	LOTS040	.02006	.0635	40.00
41	STRET041	.01249	.0270	35.00

STREAMS:

NUMBER	NAME	MAXIMUM VALUES				TIME TO	FLAG/CHECK VALUES							
		OUTFLOW	STAGE	VEL.	TR.FORCE	PEAK	OUTFLOW	T.E.	DEPTH	T.E.	VELOCITY	T.E.	TR.FORCE	T.E.
		(CFS)	(FT)	(FPS)	(LB/SQ.FT)	(MINS)	(CFS)	(MINS)	(FT.)	(MINS)	(FPS)	(MINS)	(LB/SQ.FT)	(MINS)
999	ENDUM999	287.59	2.62	N.A.	N.A.	65.00	N.A.	—	N.A.	—	N.A.	—	N.A.	—
10	MAINS010	81.34	.88	N.A.	N.A.	40.00	N.A.	—	N.A.	—	N.A.	—	N.A.	—
20	MAINS020	308.01	1.53	N.A.	N.A.	40.00	N.A.	—	N.A.	—	N.A.	—	N.A.	—
30	MAINS030	574.17	1.17	N.A.	N.A.	40.00	N.A.	—	N.A.	—	N.A.	—	N.A.	—
40	MAINS040	857.13	2.88	N.A.	N.A.	40.00	N.A.	—	N.A.	—	N.A.	—	N.A.	—

KEY:

T.E. = TIME IN EXCESS
N.A. = NOT APPLICABLE
(W) = WET CHANNEL LIMIT
(O) = OVERBANK LIMIT

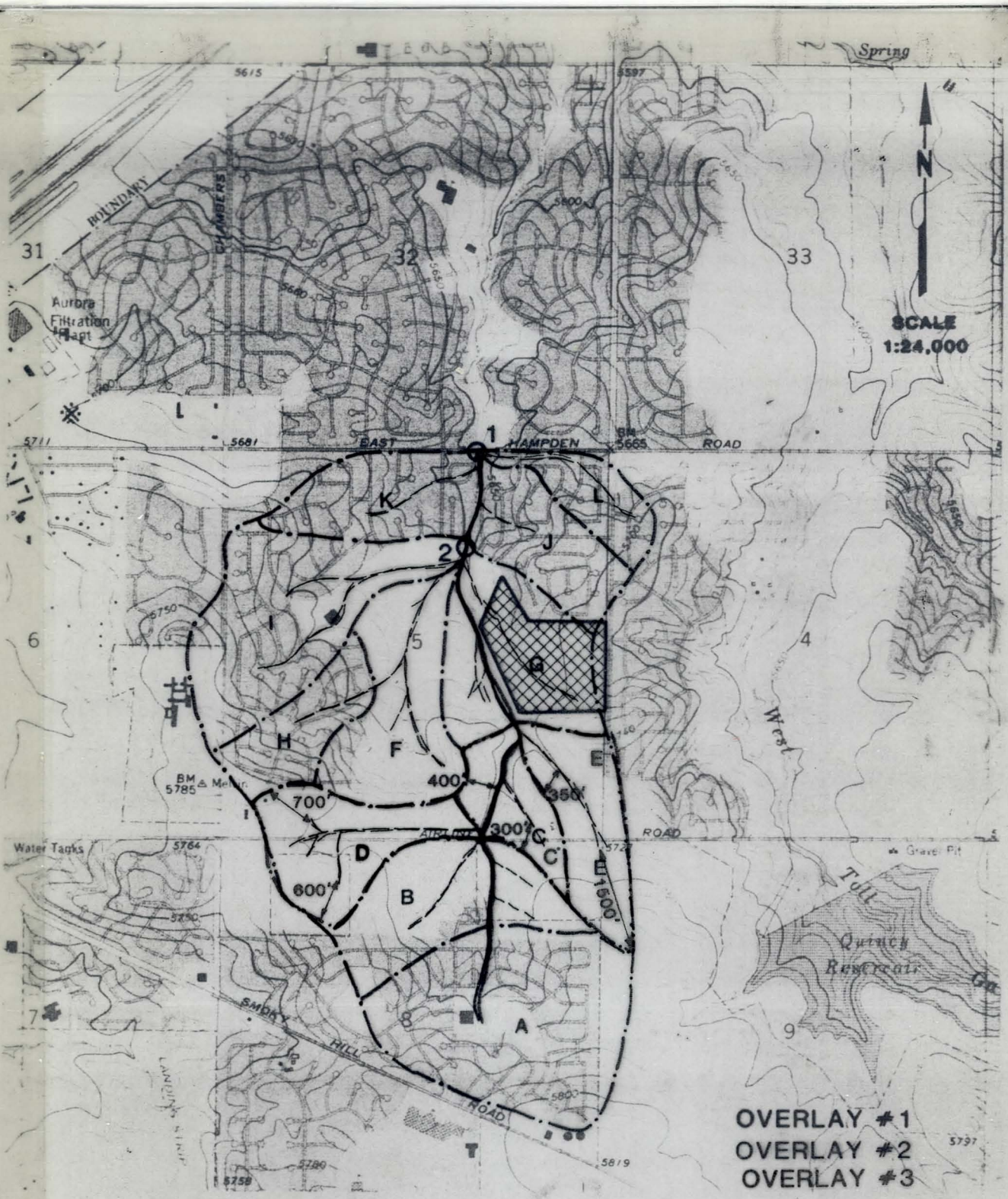
RESERVOIRS:

NUMBER	NAME	MAXIMUM VALUES				TIME TO	FLAG/CHECK VALUES			
		OUTFLOW	STAGE	VOLUME	SPILL/HEAD	PEAK	OUTFLOW	T.E.	HEAD	T.E.
		(CFS)	(FT)	(AC.FT)	(FT)	(MINS)	(CFS)	(MINS)	(FT)	(MINS)
47	REGIN047	287.59	5698.98	19.87	2.88	65.00	N.A.	—	N.A.	—

KEY:

T.E. = TIME IN EXCESS
N.A. = NOT APPLICABLE

@FIN



OVERLAY #1
OVERLAY #2
OVERLAY #3

MITCAT APPLICATION
HYDROLOGIC MONITORING FOR FLOOD HAZARDS AND MANAGEMENT

LOCATION OF DESIGN POINTS AND NEW DEVELOPMENT
SUB-BASIN DEFINITION AND DRAINAGE NETWORK
OVERLAND FLOW LENGTHS AND SLOPES

CDM
environmental engineers, scientists, planners & management consultants

WORKSHOP ON HYDROLOGIC MODELING FOR FLOOD HAZARDS
AND MANAGEMENT - EXAMPLE MITCAT APPLICATION

1.0 INTRODUCTION

This sample problem illustrates the application of the MITCAT model to a typical problem - determination of the increase in runoff due to development of open areas within an urban watershed. Plate 1 is a reproduction of a topographic map (USGS 1:2400 quadrangle) which shows the watershed under consideration. Your firm has been hired to determine if improvement to the existing culverts under East Hampden Road and Loop Road (Locations 1 and 2 shown on overlay #1) will be necessary to handle the increased runoff from a one hundred-year storm due to development of the parcel shown on overlay #1.

Application of MITCAT to such a problem would require:

- Rainfall data to define a design storm.
- Soils data to define infiltration characteristics.
- Information from field inspection or as-built plans describing stream channel characteristics (typical cross-section and roughness) and pipe networks.
- Information describing land use characteristics sufficient to determine the percentage of impervious area within the watershed or sub-basin areas.
- Topographic mapping of the study area.

This information is supplied in Tables 1-4; these tables are keyed to the sub-basins and stream network shown on overlay #2. Development of key model parameters is discussed in the following sections. The participants are encouraged to develop a MITCAT model for the middle section (sub basins F-I) and to use the available computer facilities to run the model.

TABLE 1

SUB-BASIN CHARACTERISTICS

Sub-Basin I.D.	SCS Curve No.	Area (ft^2)
A	80	6.8×10^6
B	65	2.5×10^6
C	65	1.7×10^6
D	65	3.0×10^6
E	65	2.6×10^6
F	65	4.1×10^6
G	65	4.3×10^6
H	80	2.0×10^6
I	80, 65*	5.3×10^6
J	80	2.7×10^6
K	80	2.8×10^6
L	80	1.8×10^6

* 80 for developed portion, 65 for undeveloped

TABLE 2

TYPICAL MANNINGS "n" FOR OVERLAND FLOW

Surface	n
Dense growth	0.4 - 0.5
Pasture	0.3 - 0.4
Lawns	0.2 - 0.3
Concrete/Asphalt	0.05 - 0.15

TABLE 3

STREAM CHANNEL CHARACTERISTICS

Sub-Basin	Channel Description
F	Bottom width increases in downwstream direction from 1 foot to 4 feet; banks are 2-3 feet high, side slopes approximately 10:1; vegetation growing out of banks and channel.
G	Bottom width varies from 2 to 4 feet; banks are approximately 3 feet high with side slopes from 5:1 to 15:1; some vegetation overhangs banks.
H	Storm drain network has 24-inch diameter concrete pipe as trunk line with 15-inch diameter laterals. Maximum length of tributary and branch line is 1200 feet. Slope decreases in downstream direction, varies from .050 to .035.
I	Storm system has 36-inch trunk line; maximum length of tributary and any branch line is 1,000 feet, slope varies from .025 to .040. Pipe system outfalls into stream channel with bottom width ranging from 2 to 4 feet. Banks are relatively steep with side slopes approximately 3:1. Sandy, "clean" channel.
Reach between design points 1 and 2 (see overlay #1)	Bottom width 5-6 feet, side slopes approximately 3:1; banks 3-4 feet high. Channel bottom sandy and free of vegetation.

TABLE 4

Time (Min.)	Rainfall Intensity (In./Hr)
0	0.0
5	0.36
10	0.96
15	1.44
20	2.52
25	4.44
30	8.04
35	4.44
40	2.92
45	1.92
50	1.56
55	1.32
60	1.32
65	1.32
70	0.60
75	0.60
80	0.48
85	0.48
90	0.36
95	0.36
100	0.36
105	0.36
110	0.36
115	0.36
120	0.36
125	0.0

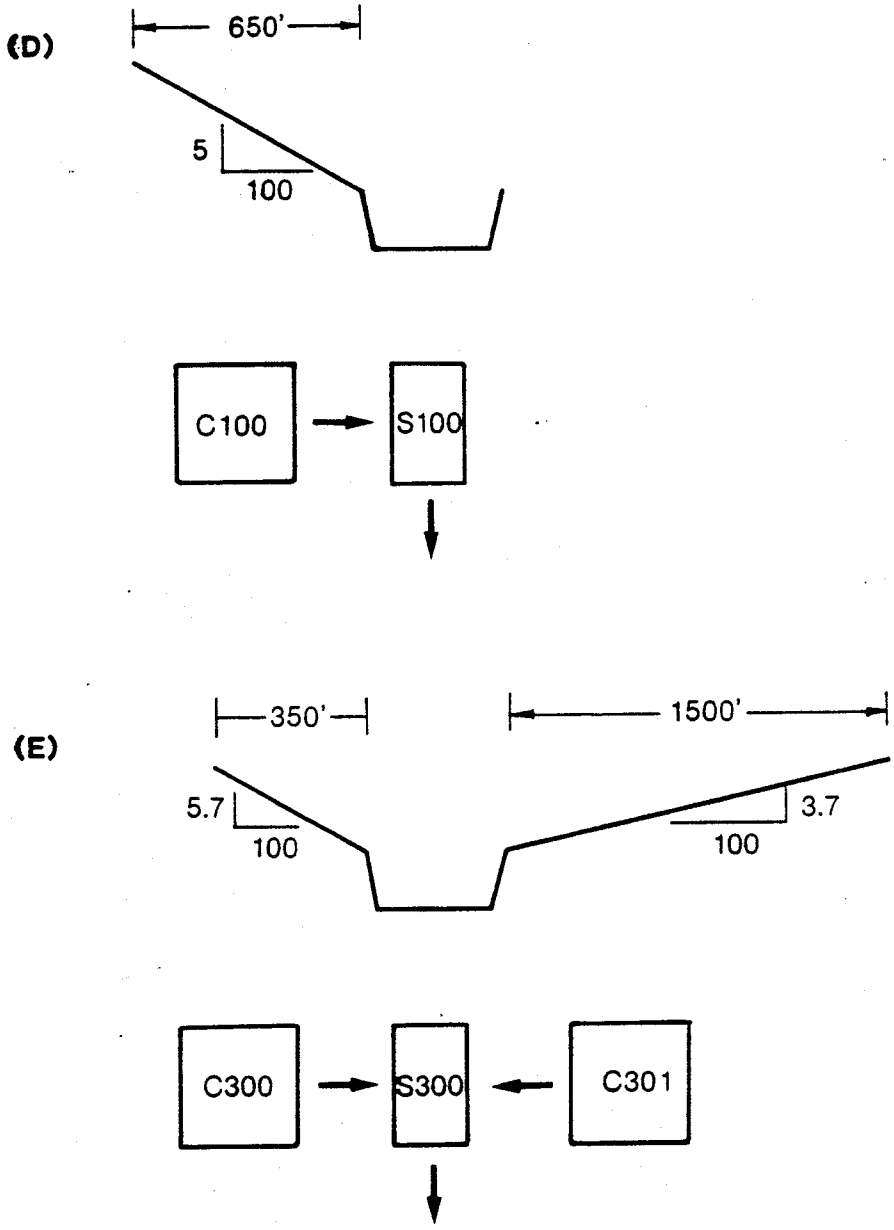


Figure 1 Mitcat Representation of Sub-Basins D-E

2.0 SUB-BASIN DEFINITION

The first step in development of model parameters is to sub-divide the study area into sub-basins. It is recommended that 3-5 sub-basins be established upstream from any "design point" (i.e. a location where hydrograph information is desired) to reflect variations in watershed characteristics. Other factors which should be considered when determining sub-basin boundaries include:

a. Study Objectives

MITCAT can define discharge hydrographs and related information only at the end points of sub-watersheds. It is therefore necessary to obtain reasonable coincidence of sub-basin end points and locations where hydrographs are desired; for example, at stream gauges, major highway culverts or proposed locations of future structural improvements.

b. Definition of the Drainage Network

MITCAT is a physically-based model and the relationship of the simulating elements should closely parallel the actual conditions. Design points should therefore include the junctions of major tributaries, locations of stream diversions or streamflow augmentation, etc.

c. Major Surface Water Features

Reservoirs, detention ponds, and undersized culverts or significant channel constrictions which provide flood storage should be represented. These features may greatly alter the timing and magnitude of watershed response to storm events.

d. Change in Hydrologic/Hydraulic Character

Sub-basins should be selected and defined to represent fairly homogeneous hydraulic or hydrologic characteristics. For example, individual sub-basins should be established to represent:

1. Major changes in the slope, roughness coefficients or cross-section of the principal stream or drainage channel.

2. Significant changes in land use patterns.

3. Marked variation in overland runoff or soil infiltration characteristics.

Overlay #2 illustrates the sub-basin definition for this example.

3.0 CATCHMENT (OVERLAND FLOW) PARAMETERS

The required model input for catchment parameters is as follows:

NUMB	ID number for defining drainage network.
NAME	User designated name for reference purposes.
A-LENG	typical maximum distance of overland flow travel.
LENGTH	Effective catchment overland flow distance used in calculating area.
SLOPE	Typical topographic slope of catchment area.
N	Roughness co-efficient for overland flow.
INFILTRATION DATA	Parameters for one of the following methods - SCS, Holtan, Horton, and API.
% IMPERVIOUS	Portion of catchment area that may be considered as impervious.

Overlay #3 illustrates typical overland flow lengths for sub-basins C-E.

One technique for evaluating the SLOPE and A-LEN parameters is illustrated using overlay #3. In this method, the user first delineates (from the best information available), the streams, swales, gullies, etc. that function as channels during a storm event. The next step is to draw lines representing maximum overland flow lengths from local high points to these channels. These lines should be characteristic of significant portions of the segment. The A-LENG parameter may be computed as a weighted average of the length of these lines. As few or as many such lines may be drawn as the user deems necessary; as experience increases the user is generally satisfied with one or two such lines. The slope parameter is computed from available topographic data.

4.0 STREAM (CONVEYANCE SECTION) PARAMETERS

The required model input for simulating streamflow is:

	NUMB	ID number for definition of drainage network.
	NAME	Arbitrary ID name specified by user.
	A-LENG	Length of main channel within the segment boundary.
	LENGTH	Effective channel length used to compute sub-basin areas.
	SLOPE	Average SLOPE of channel bottom over the reach described by "A-LENG" above.
	CROSS SECTIONAL DATA	Parameters to define cross-sectional shape of main channel, including Manning 'n' roughness coefficient.
*	SPILL DATA	Parameters required to define stream diversions.
*	DISTRIBUTE	Used to reflect the impact of non-uniform spacing of tributary inflow to a stream, primarily used in handling urban areas.

For stream elements the "A-LENG" and "SLOPE" parameters may be determined from available topographic maps. It is necessary only to define the main drainage channel, measure stream length and calculate the channel slope.

Cross-sectional data and roughness co-efficients may be determined from available information such as USGS field surveys or field inspections by the project team.

Definition of stream elements for urban areas is somewhat less precise. At the level of detail in this example, the urban stream element is being chosen to represent the aggregate affects of a number of individual storm

---

*OPTIONAL PARAMETERS

sewer systems rather than a unique pipe length or portion of a unique system. The "A-LEN" parameter is thus an approximate measure of the maximum length of travel within a typical storm drain system and the "SLOPE" is an average value reflecting the slope of individual pipes within the network. In Figure 2 for example, the longest path within the network is from "A" to "P", a distance of 500 ft. at the indicated scale. Over a given reach the drainage pipes slope from .008 ft/ft to .025 ft/ft. A reasonable slope estimate would be .010 given the lesser variation in slope as pipe sizes increase. In choosing a typical cross-section the user should emphasize the larger portions of the network; a 36" diameter circular section would provide an adequate representation of the system in Figure 2 for example.

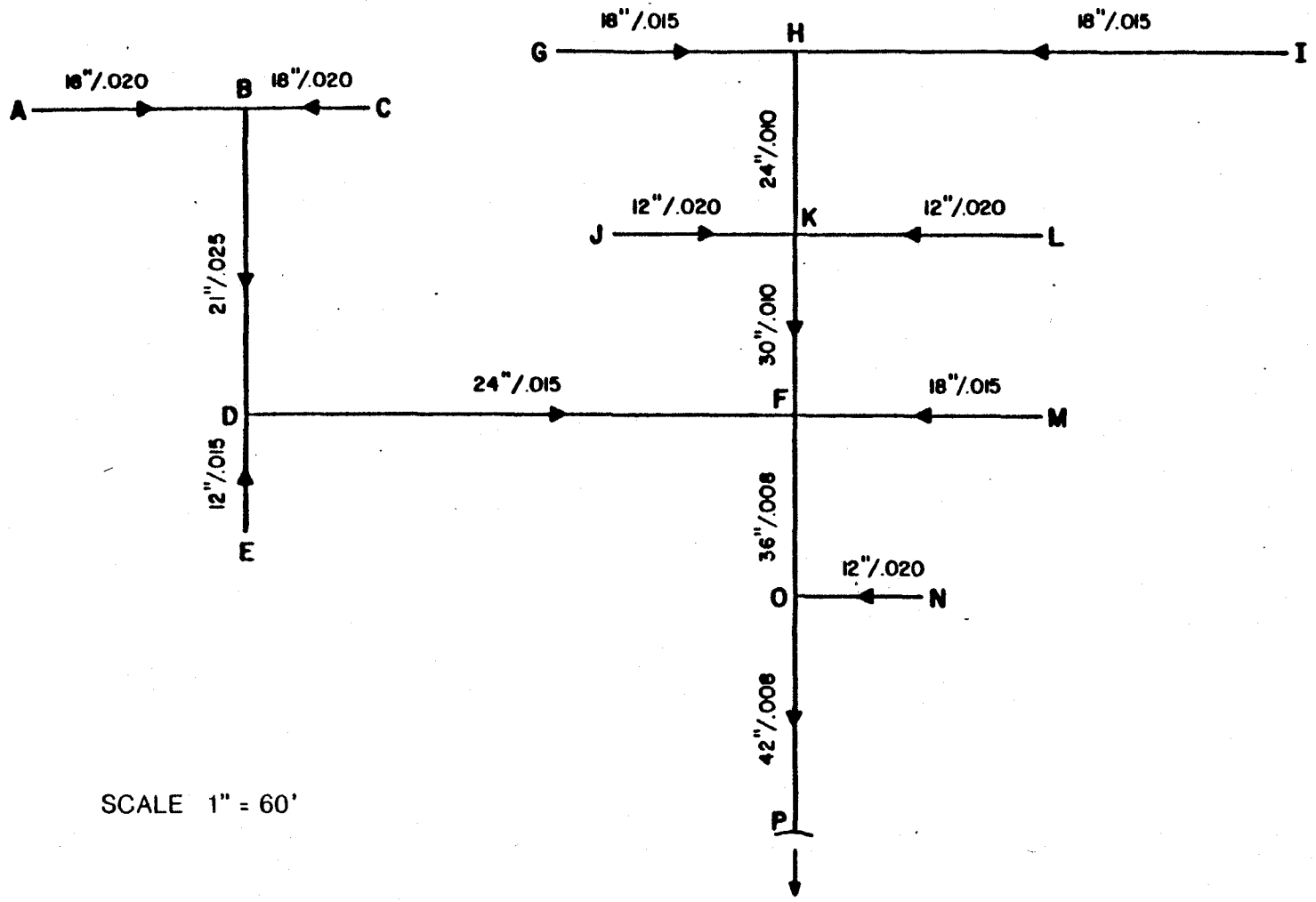


Figure 2 Storm Drainage System

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5.0 AREA CALCULATIONS

In MITCAT, sub-basins are represented as rectangular areas whose width is equal to the overland flow length (specified as the A-LENG parameter). Sub-basin area would then be equal to the product of the rectangle width (A-LEN) and length. In an "ideal" situation, the length of the rectangle would be equal to the length of the stream which drains the catchment and the area would be equal to the product of the stream A-LENG parameter and the sum of the A-LENG's for all catchment draining to the stream.

In practice, the model representation does not explicitly represent each minor tributary or drainage swale within a sub-basin. Consequently, the calculation of catchment areas based solely on A-LENG parameters would invariably be in error. An "effective length" for each catchment and stream is required (i.e., "LENGTH" parameter). MITCAT uses the distance specified in A-LENG to perform channel or overland flow routing and maintains mass balance by scaling catchment or stream input by the ratio of actual to effective length. Experience has shown that simulation accuracy in computing catchment runoff is particularly sensitive to this ratio; streamflow routing is somewhat less sensitive, especially in urban areas where stream or pipe elements represent manmade channels or pipe networks. For catchment, the ratio of actual to effective length should fall within the range of 3:1 to 1/3:1. If this cannot be achieved the sub-basin should be re-segmented. For natural streams, it is recommended that ratios be maintained between 4:1 and 1/4:1. In urban areas, however, it is permissible to use ratios up to 100:1 or 1:100. Examples are presented below.

In sub-basin D, overland flow is represented by a single catchment of A-LENG 650 as shown in Figure 1. The area was planimetered as 3,057,000 ft2 and the stream A-LENG is 2000 feet. A 1:1 ratio is most desirable so the catchment LENGTH is set equal to 650 feet and the resultant stream LENGTH is calculated as:

$$\text{STREAM LENGTH} = 3{,}057{,}000\ Ft^2/650\ FT = 4703.1\ Ft.$$

The ratio of A-LENG to LENGTH is then checked for the stream and is satisfactory at 0.42.

A more complicated situation was encountered in sub-basin E. Overland runoff in the upper part of that area is estimated to travel 1,500 feet before reaching a stream while the remainder travels only 350 feet. Two catchments (see Figure 1) are used to represent this situation. The total sub-basin area is 2,630,000 Ft^2; approximately 1,578,000 Ft^2 is represented by the "short" catchment (i.e. 60% of area E as estimated "by eye") and 1,052,000 ft^2 (i.e. 40% of area E) by the "long" catchment. If A-LENG/LENGTH for each catchment is set to 1.0, the area attributed to the long catchment would be approximately 81% of the area:

$$\left(81\% = \frac{1{,}500}{1{,}500 + 350} \times 100\right)$$

rather than 40% as desired. Consequently, the ratio of catchment effective lengths should be set equal to the ratio of catchment areas before calculating stream length. In this case, an effective length of 900 for the short catchment and 600 for the long catchment maintains the correct areas (60% and 40% of total area, respectively) and satisfies the 3:1 to 1/3:1 range for each catchment. Stream LENGTH is then calculated as:

$$\text{STREAM LENGTH} = 2{,}630{,}000/(900 + 600) = 1753.3$$

Since A-LENG for the stream is 2100, this representation should give accurate results.

6.0 DRAINAGE NETWORK & MODEL INPUT SPECIFICATIONS

6.1 Input Conventions

A significant problem in a complex simulation model is the specification of the data required to define the various model parameters. These parameters are relatively numerous and may refer either to individual routing elements or to the basin as a whole. The approach adopted in MITCAT has been to provide a user-oriented free-format command structure where each data value being entered is identified by an alphameric 'tag' and thus assigned to its desired location. In addition, default values are assigned for all non-essential data so that relatively little input data is required to run a simple model, while the ability to operate under more complex situations is readily available.

The input data is assumed to consist of two levels of commands referred to hereafter as primary and secondary commands. The PRIMARY level commands are used to specify the type of data which follow, i.e., does the data refer to a routing element?, the basin connectivity?, etc. Each primary command sequence begins with a single alphameric word, contains a set of secondary commands and terminates with a FINISH command. SECONDARY level commands normally consist of an alphameric command tag followed by the required numeric data value. On occasion, a secondary command will consist solely of a single alphameric command.

In order to enhance user interaction with the MITCAT system, the data is read in a free format fashion. Using such a format, each command tag and/or data has no fixed location in a data card (or terminal input line) but is simply separated from the other data by one or more blanks acting as delimiters of the data string. Such a format also permits a variable sequence of secondary commands within a primary command block rather than at fixed locations dictated by programming considerations. A typical command sequence in a MITCAT data input file might look as follows:

CATCHMENT	NUMBER	1	NAME	TEST-1	LENGTH	100.3	FINISH
	Tag	Data[1]	Tag	Data[2]	Tag	Data[3]	
Primary Command	Secondary Command Block		Secondary Command Block		Secondary Command Block		Terminates Primary Command Block

1 - Secondary data is integer value

2 - Secondary data is alphameric

3 - Secondary data is non-integer

The above commands serve to define an overland flow element (CATCHMENT) with the following parameters:

Number : 1

Name : TEST-1

Length : 100.3 (feet)

The input editor allows the above data to be input on a single input record (card or line) or specified over a number of records. For example the following sequence is functionally equivalent to the example presented above.

Card 1: CATCHMENT NUMBER 1 LENGTH

Card 2: 100.3 NAME TEST-1

Note that in this case the primary command block (CATCHMENT to FINISH) has been spread over two records. In addition, the secondary command block (LENGTH 100.3) has, itself, been split between records. The only exception to such splitting is that it is not permissible to split either a single command tag (e.g. LENGTH) or data value (e.g. 100.3) over two such records.

It will also be noted in the above example that the relative locations of the NAME and LENGTH secondary command blocks within the CATCHMENT primary command block have been reversed. Such changes are permitted in MITCAT. This ability to sort secondary command data is extremely useful in adding

data to the record(s) already describing a primary command block. For example, slope data for the CATCHMENT described above could be added to card #2 as follows:

```
Card 1:  CATCHMENT  NUMBER  1  LENGTH
Card 2:  100.3  NAME  TEST-1  SLOPE .05  FINISH
```

Alternatively, a new card #3 could have been added:

```
Card 1:  CATCHMENT  NUMBER  1  LENGTH
Card 2:  100.3  NAME  TEST-1  FINISH
Card 3:  CATCH  NUMB  1  SLOPE  .05  LENGTH  200. FINISH
```

The command editor would have assigned all secondary command blocks following the identification of the specific element (NUMB command) to the same CATCHMENT data set as referenced by Cards 1 and 2. Card 3 need not be the next card immediately following Card 2 in a complex input deck. The user may use the above capability to add and/or update data specified in a preceding primary command block. For example, following input of Card 3, the length of CATCHMENT NUMBER 1 would be 200 feet rather than 100.3 feet.

6.2 Drainage Network

Any conveyance element is referred to as a "NODE" in MITCAT. The stream network is specified through use of the "STRUCTURE" and "CONNECTIVITY" primary commands. The STRUCTURE command defines the relationship of catchments and streams through use of the identification numbers assigned by the user. For example, the command:

```
STRUC  NODE  300  C 300  S 300  C 301  FINIS
```

defines sub-basin E as shown in Figure 2. The basin drainage network is then defined by specifying the relationship of one sub-basin to another through the CONNECT command. For example, the command:

```
CONNEC  NODE  300  LINK  200  100  FINIS
```

specifies that the sub-basin defined by nodes 200 and 100 drain into sub-basin E of Figure 2.

7.0 SUMMARY

Figure 3 presents a comparison of key model capabilities for a variety of hydrologic models, including MITCAT; this Figure illustrates the wide range of capabilities included in the MITCAT model. In choosing which model to use for a particular problem, one should remember two limitations on the use of MITCAT:

1. The kinematic wave formulation doesn't include backwater impacts.
2. Micro-based models may be more cost-effective for application to very detailed studies of small watersheds, such as design of sub-division drainage systems.

For most applications, however, use of MITCAT offers several advantages:

1. User-friendly with respect to input and output formats
2. Will accept variable cross-section geometry and roughness co-efficients for streamflow routing in addition to "ideal" shapes (e.g., rectangles, trapezoids, etc.)
3. Based on physical properties of watershed so it can be applied with reasonable confidence in areas where streamflow data are non-existent or out of date
4. Can simulate most hydrology/hydraulic situations one might encounter - rural or urban watersheds, streamflow diversions to other basins or other parts of same watershed, detention ponds or other storage facilities, etc

These advantages have been illustrated in developing solutions to drainage problems in numerous instances throughout the U.S. and overseas.

	CATCHMENT HYDROLOGY							SEWER HYDRAULICS								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	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	●				●	●		●									●	●		●	

Source: Adapted from Evaluation of Methematical Models for the Simulation of Time-Varying Runoff and Water Quality in Storm and Combined Sewerage Systems, A. Brandstetter, R. Field, and H. Torno, paper presented at the conference on Environmental Modeling and Simulation, sponsored by the U.S. Environmental Protection Agency, Edison, N.J., April 1976.

Figure 3 COMPARISON OF MAJOR MODEL CATEGORIES

SACRAMENTO WATERSHED MODEL

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THE SACRAMENTO WATERSHED MODEL

BY

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Conceptual hydrological models can be a fundamental tool for providing improved hydrologic services. However, such models can be truly effective only if they are utilized in a manner which allows them to reasonably reflect those processes in the soil mantle which are significant to the runoff process. The Sacramento Watershed Model was developed as a headwater forecasting system which was based on a conceptualization of those processes which control headwater flows. In the United States where the Sacramento Watershed Model has been applied, there have been many examples of the inherent flexibility of such conceptual models. This flexibility has made it possible to assess the time trends in rapidly urbanizing basins - to modify the rainfall runoff process on the basis of calamities such as wildfires - to assess the potential for future streamflow during drought conditions, to extend the range of forecast services to much smaller river basins, and to provide ancillary information on runoff and soil moisture conditions for a variety of purposes. These capabilities have been obtained from a streamflow simulation model which has provided an extremely effective tool for the National Weather Service flood forecasting program but which, due to its conceptual structure has contributed to many other applications.

The important element in the application of conceptual models such as the Sacramento Watershed Model is that the fitted model must be representative of those conditions and processes which are consistent with moisture storages and transfers within the soil mantle and the resulting streamflow at headwater points. Regardless of a conceptual hydrologic model's inherent capability, the number of physically-related parameters is necessarily so large that if relatively simple fitting rules are not followed, a black box solution to the modeling problem can inadvertently take place. A black box application of a conceptual model is one which seriously distorts those physical processes which the model is attempting to describe. Such an analysis can limit the appropriateness of the model and severely restrict its potential applications. For this reason, the proper starting point for using a conceptual model is with an understanding of the basic concepts upon which the model is based. Once these concepts are understood, proper application of the models representations of the physical processes is made possible and a broad scale of applications can be reasonably made. The most fundamental conceptual rainfall runoff model - and one which effectively expresses the basic rainfall runoff processes over an extended period of time is:

Runoff = Rainfall - Evapotranspiration

In some areas, a portion of the runoff may leave the basin by subterranean routes, but for most basins this simple expression is a very reasonable summary of the rainfall runoff process. Strangely enough, this simple relationship is frequently neglected by conceptual modelers who hurry to work with detailed modelling systems without gaining the sense of balance which a simple system can provide. Many simulation analyses have been undertaken without reviewing the appropriateness of the data set. A review of the basic relationship (Runoff = Rainfall - Evapotranspiration) can assist the modeler in verifying the appropriateness of the available data. A mistaken estimate of one or more of the key components in this natural triangle can frustrate all efforts to properly utilize a conceptual model.

The first truth which must be observed before attempting to apply a conceptual model is that the water balance distribution between rainfall, runoff, and evapotranspiration must make sense. If the rainfall, runoff, and evapotranspiration data do not provide reasonable representations of the actual dimensions of the processes involved, then there will be distortions of the natural processes which will permeate subsequent analyses. In order to allow a more effective evaluation of runoff, an effective conceptual model most be capable of emulating the natural water balance in space and time. If the basic water balance is distorted through a biased evaluation of rainfall, runoff, or evapotranspiration, then the conceptual model will be forced into the role of a black box procedure. Such a distortion restricts the systems ability to properly emulate runoff conditions and can seriously affect analyses which are not encompassed in the original data set.

Unfortunately, our basic hydrologic measurement systems have serious biases which make it difficult to respect the simple water balance maintained by nature. Throughout the world, precipitation suffers from a significant under-measurement bias. Runoff suffers from substantial measurement problems during extreme events; high flow determinations are frequently obtained by rating extrapolations which can test the acumen of any hydrologist. Low flows are subject to major mis-analysis from debris on controls or relatively minor changes in the condition of the stream bed. Evapotranspiration, a major component of the hydrologic cycle, is not measured at all. The closest phenomena to evapotranspiration which is measured is evaporation, and available evaporation measurements leave much to be desired. Even so, evaporation and evapotranspiration are two remarkably different processes; processes which do not regularly compare with one another for there are conditions which can cause one to increase substantially while the other is diminishing. An effective water balance can not be defined without consideration of these problems. The simple water balance equation and the Sacramento Watershed Model, which is an expansion of the same

basic logic, require an appreciation of measurement problems associated with the hydrologic cycle.

RAINFALL

The first element in defining runoff is rainfall. Careful analyses of rainfall data indicate that there is a world-wide problem in precipitation analysis which generates an aggregate undermeasurement between 10% and 15% of the actual rainfall. Unfortunately, rainfall is not systematically undermeasured but is dominantly influenced by the problem of wind effects over the orifice of the precipitation gage. As wind speed increases, the ability of the gage to measure rainfall decreases (see fig. 1) Unfortunately, major precipitation periods are generally the result of strong winds providing rapid advection of the atmospheric moisture needed to support heavy rainfall rates. Under rapidly fluctuating wind conditions, rainfall runoff relations which attempt to define a loss curve from unadjusted rainfall data can produce major mis-analysis of the potential for future runoff.

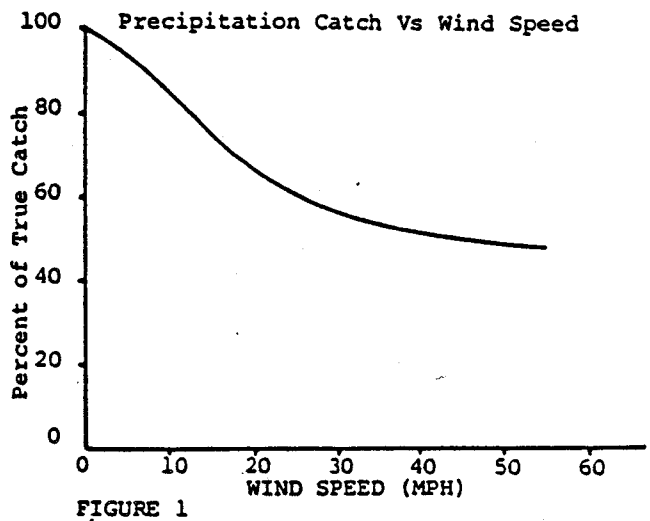


FIGURE 1

Data is generally unavailable to adequately define the degree of rain gage catch deficiency during past storms. The basic rainfall measurement problem is such that due to data deficiencies, a properly tuned hydrologic model should have an under-forecasting bias for historical runoff events when wind was an important factor in the storm.

If the rainfall measurement problem is not considered during the tuning of a model, a serious problem can result. An operational hydrologist applying the model to rainfall information with a different bias than the initial data set will produce an invalid answer. This problem will occur even if the aggregate rainfall volumes at the rainfall sites used in development is comparable to the total volume of rainfall measured at raingage sites used in operational decision making. Where the rainfall gages used for operational decisions measure rainfall at a different scale than the original development sites, the change in input scale can dramatically alter the tuned models apparent ability to predict runoff.

If consistent runoff analyses are to be produced, then the scale of precipitation input must be consistently maintained. Unfortunately, many historic records of rainfall are remarkably inconsistent. This may be due to a change in gage location, a change in gage characteristics, modifications at or near the site which alters the historic wind patterns or variations in the observation techniques of the observors. For these reasons and others, a careful application of the Sacramento Watershed Model, or for that matter the basic water balance equation, requires a continuous comparative analysis of rainfall and runoff records to insure that a distortion is not being introduced in an attempt to describe an unusual pattern which may be a result of data inconsistencies rather than a true event.

EVAPOTRANSPIRATION

Detailed studies of evapotranspiration have demonstrated significant differences between measured evaporation at pan sites and the field use of moisture by plants. As a result, any attempt to utilize pan evaporation as an index to basin evapotranspiration requires a significant transformation of the pan data. Inasmuch as evapotranspiration may be the dominant use of the moisture supplied by precipitation, and since it is one of the most difficult processes to evaluate in hydrologic analysis, evapotranspiration estimates can provide a principal source of error in streamflow simulation. Basin evapotranspiration, as used in the Sacramento Watershed Model, includes transpiration from leaf surfaces of water taken from the soil by plant roots, direct evaporation from the soil and from leaf and other surfaces wetted by rain, and direct evaporation from the surfaces of lakes, ponds and streams. In most basins, evapotranspiration is the dominant process in removing water from the soil.

The estimation of transpiration from evaporation pan data can introduce several significant problems in attempting to evaluate the transfer of moisture by plants from the soil to the atmosphere. The processes of evaporation from a pan and transpi-

ration from the surfaces of a plant leaf differ in several important aspects. An evaporation pan has considerable heat storage while a plant leaf has very little. Therefore, the temperature of a plant leaf responds more quickly to changes in air temperature and radiation. Soil temperatures, which influence both the growth of vegetation and the ability of roots to absorb soil moisture, change rather quickly at the soil surface but much more slowly with depth. The growth of annual plants and the leaf development of deciduous perennials varies with the time of the year and with past temperature and moisture conditions. Even for mature leaves, their condition and ability to transpire varies with season and past weather. The processes of water-surface evaporation also differs from transpiration in that transpiration may be restricted by nocturnal or other closures of stomata, the small openings in leaf surfaces through which water vapor and other gases diffuse. Stomatal closure may also be induced by excessive moisture stress in the leaf. Such stress tends to limit transpiration when atmospheric conditions favor rapidly increasing evaporation rates.

In many areas, more moisture is lost through evapotranspiration than is discharged by the streams. Potential evapotranspiration, the evapotranspiration which would take place if an adequate supply of moisture were available, often exceeds rainfall on an annual basis. Under these conditions, streamflow may occur only during and shortly after concentrations of precipitation. Due to the significance of evapotranspiration to the runoff processes, it is necessary to utilize a physically representative estimate of evapotranspiration. Inasmuch as the characteristics of the evaporation pan do vary significantly from those of vegetation, we can anticipate that any estimation of evapotranspiration demand which is based upon pan observations will naturally reflect the differences between pan and basin characteristics. Nonetheless, evaporation pan data despite its definite deficiencies is generally the only available measured index to the evapotranspiration process. Evaporation pan data is generally modified through the use of monthly or seasonal pan coefficients in an attempt to approximate the evapotranspiration process.

Basin evapotranspiration demand is defined as the evapotranspiration which would occur if there were no deficiencies in the availability of moisture for evapotranspiration. The demand is significantly affected by plant regimes which exist in the basin. When the basin warms after a seasonal cold period where vegetative cover is largely annual or deciduous, pan evaporation may increase much more rapidly than basin evapotranspiration demand. This is because evapotranspiration demand must wait for the growth of leaves and roots.

Under these circumstances, seasonal coefficients applied to pan data, in an attempt to modify evaporation data to the scale of basin evapotranspiration, are likely to be quite small during those periods when observations of pan evaporation show a rapid increase. When the basin is undergoing seasonal cooling, pan evaporation may decrease at a more rapid rate than is appropriate to evapotranspiration from a fully developed leaf and root systems. As a consequence, coefficients applied to the pan data to transform it for soil moisture accounting purposes may be larger during the annual cooling cycle than during a similar pan evaporation period in the spring.

In areas where early frosts dramatically change the moisture use of plants, pan evaporation data may show only a slight alteration of the evaporation rate even though the vegetation has discontinued the transpiration process. Seasonal coefficients applied to pan data in order to approximate basin evapotranspiration demand can produce very unsatisfactory results during abnormal weather conditions. Even so, the product of seasonal coefficients and pan observations frequently provide the only available data for approximating a seasonal evapotranspiration demand curve.

Due to the inappropriateness of pan evaporation measurements, direct estimates of seasonal evapotranspiration values can be more efficacious than attempting to re-dimension pan data. It has been the author's observation that a seasonal evapotranspiraation demand curve will peak in mid-summer at a level of approximately 4mm per day in cool climates and close to 7mm per day in warm climates. The low point on the evapotranspiration demand curve generally persists about three months during the winter period, and will have a magnitude of about 1mm per day in cold climates. In warm climates, winter evapotranspiration demand may continue at relatively high values. In any case, the average monthly evapotranspiration demand will generally be dimensionally similar to the average monthly measured evaporation from a carefully maintained pan during the summer and winter extremes.

The differences between the evapotranspiration process and the evaporation process are such that the use of evaporation to represent evapotranspiration is an approximation which can help define soil moisture under some conditions, but which is totally inappropriate to others. Inasmuch as the evapotranspiration process is a major component of the hydrologic cycle and since it is relatively poorly defined, it is not difficult to understand why many investigators have turned away from conceptual models which require evapotranspiration information and pursued the development of stochastic representations of the rainfall-runoff process.

The measurement problems inherent in natural data bases is severe. Applying physically based models under these conditions takes skillful judgement. The use of physically based conceptual

models can, however, lead to a better appreciation of the physical problems involved, and through such knowledge to improvements in hydrologic measurement and analysis systems. Thus, physically based conceptual models can lead to a more secure basis for the critical analysis and forecast decisions which are crucial to operational hydrology. With this very brief overview of some of the issues involved in a conceptual approach to rainfall-runoff modelling, it is now appropriate to consider the mechanisms which are basic to a conceptual headwater forecasting model.

SOIL MOISTURE

The start of any headwater modeling system is the rainfall-runoff process. For a given soil, the primary condition which affects the rainfall runoff process is soil moisture. Any reasonably constructed representation of the rainfall-runoff process must provide a simulation of the dominant soil moisture conditions. To better understand these conditions, a brief review of some of the basic characteristics of soil moisture may be helpful.

Analyses of the movement and storage of moisture within the soil have demonstrated that there are three dominate forms of moisture in any given volume of soil. These forms are: hygroscopic, tension, and free water. Hygroscopic water is that water which is tightly bound to soil particles - so tightly bound that it can only be removed by extreme heating. This volume of water is approximately 5% of the weight of an apparently dry soil. Hygroscopic moisture is essentially locked to the soil particles. Only in warm, dry climates will the sun's heating remove small volumes of hygroscopic moisture which are near the surface of the ground. For this reason, hygroscopic moisture is a nearly constant part of the soil-moisture system and can be considered as relatively invariant in the modeling process.

The second form of water in the soil is that additional volume of water which can be held in close juxtaposition to the soil particles by the attraction of the soil particles for moisture, but which can be removed by the evapotranspiration process. This type of water is frequently called tension water. The attraction of tension water to soil particles is greater than the attraction of gravity pulling the water toward deeper depths. However, forces exerted by plant systems and/or vapor pressure differences between very shallow soils and the atmosphere can overcome the moisture attraction of the soil particles As a result, a small portion of the tension water can evaporate. The dominant use of this moisture is however, to supply water for transpiration by plants. Where plant roots are adequately developed to draw on this moisture, the entire tension volume can be lost from the soil through evapotranspiration. The volume of water involved in tension uses can add approximately 10% by weight to the soil.

The third form of water, which is significant to a moisture budget of the soil system, is free water. Free water is that water which moves through the soil. The first function met by free water is to transfer water to deeper levels and to resupply tension water deficiencies. When the tension deficiencies are met, at those depths to which the free water has percolated, the free water may drain horizontally or it may be temporarily stored in the interstices between soil particles. The volume of free water which can maintain temporary residence in the soil is highly dependent upon soil types and depths but, when it is present it can add approximately 45% to the weight of the soil. These different forms of moisture within the soil - hygroscopic, tension, and free - can be visualized as adding weight to a dry soil mass approximately as indicated in Figure 2. The actual weights added to a dry soil by the application of moisture are very dependent upon soil types. The values used in this discussion are illustrative of the general dimension of moisture within the soil but considerable variance may occur in various soils.

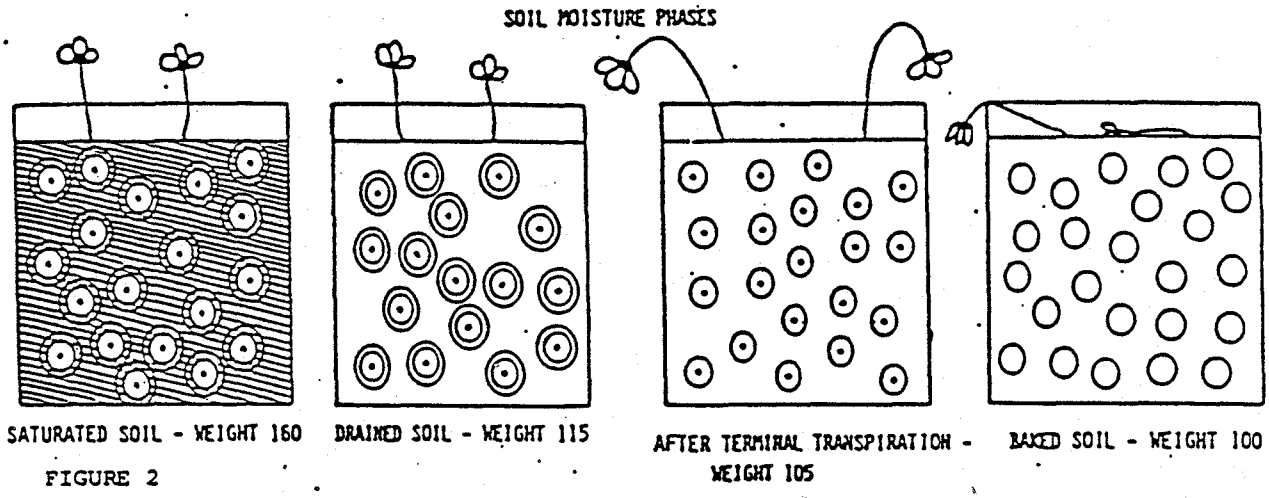


FIGURE 2

Any moisture accounting procedure must consider the amount and distribution of the two regularly variable phases of moisture within the soil - the tension water and the free water. For purposes of simplification, the relatively invariant volume of hygroscopic water can be ignored. The downward transfer of water in the soil takes place when rainfall wets the upper soil particles beyond their ability to hold the available moisture supply against the force of gravity (i.e., the surface tension water demand is exceeded). As gravity draws the water to greater depths, any unmet tension water requirements in that soil horizon are resupplied by the descending free water. This vertical transport of water is accomplished through a wetting front which gradually descends to deeper soils. This process follows the general form shown in Figure 3.

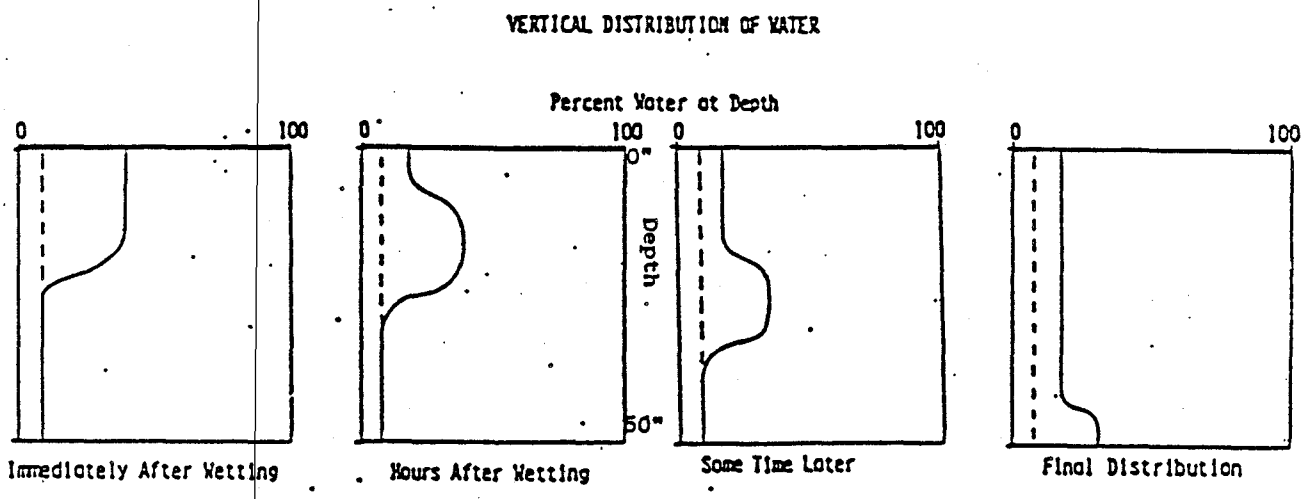


FIGURE 3

At this point, there may be a tendency to question the relevance of these phenomena when the basic question with which we are concerned is "what volume of rainfall will run off?" However, without identifying how soil moisture conditions change, we would not be able to evaluate the effect of rainfall on the basin and we would not be able to develop a rainfall-runoff model that described the dominate runoff forms which influence streamflow. The definition of the volumes of moisture involved in the soil moisture processes and the determination of the time-dependent functions which govern how these conditions change with time, are essential components of a conceptual rainfall-runoff relation.

THE SACRAMENTO WATERSHED MODEL

The Sacramento Watershed Model is based on the concepts which we have discussed. A principal problem in organizing the model was to link these concepts in a manner which provided a reasonable representation of subsequent runoff conditions but, included only that level of detail which could be demonstrated as being necessary for an effective determination of streamflow volumes. In a natural river basin, there are many characteristics in addition to those we have already described. However, at this point we have already identified the basic soil moisture storages which had to be considered in order that the Sacramento Watershed Model could be organized with a conceptual structure.

Within every river basin, the same moisture storages and the same moisture transport mechanisms are active. The question is, how can we construct a generalized representation of these processes which can be applied over a relatively large area, and still reflect the soil moisture mechanisms which have been identified in controlled tests of soil moisture characteristics. It should be recognized that only those elements of the system which vary with time need to be modeled. For this reason, the relatively negligible changes in hygroscopic soil moisture storage can be included in the modelling of the tension water system.

A small quantity of rainfall may be held in the vegetative cover of a basin and never enter the soil system. This process, called interception, causes moisture to be stored and lost from the system in a manner which parallels changes in tension water storage. Lumping the interception moisture component with tension water provides a convenient means of approximating all upper level non-free water storages. Thus, tension water and free water, their volumes, changing vertical distribution, and the transfer mechanisms between these major storages present the major components of the conceptual modelling process.

If we visualize a river basin as having rigid boundaries both horizontally and vertically, then we can quickly deduce that during the drying cycle once moisture stops leaving the basin we need not account for further moisture losses. Therefore, within the scope of what we have previously analyzed, once losses from basin drainage and evapotranspiration deplete the available water supply, the soil moisture condition is invariant until some type of moisture resupply occurs.

In order to provide for the moisture uses we have discussed earlier, it should be apparent that all river flows which do not drain across the soil surface to various streambeds, but instead pass temporarily through the soil, must spend some time as free water in the soil system. Many years ago, Darcy determined that water passes through the soil as a function of the soil's porosity and the head which is driving the water. The outflow from a volume of water draining through a uniform soil would depict a straight declining line if the discharge were plotted against time with semi-logarithmic coordinates. Barnes shed additional light on the relationship between Darcy's proposition, frequently called Darcy's Law, and the characterization of gaged streamflow recessions. Barnes demonstrated that normal recession curves appeared to be composed of drainage from three separate aquifers (see Fig. 4). Each aquifer drained independently and with significantly different characteristics. These aquifers could be seen most clearly on a continuous recession following a major rise. The composite of flow from these three aquifers actually produced a curvilinear recession until that time when the aquifer with the slowest drainage characteristic became sufficiently dominant.

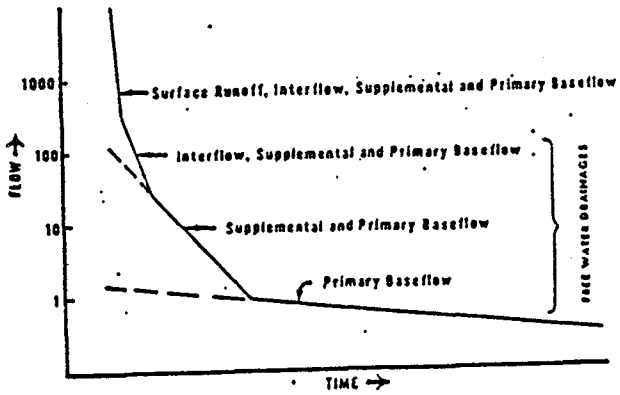


FIG. 4

In order to represent these three aquifers in a conceptual model, it became apparent that three separate free water storages each with its own drainage characteristics would be needed. Such a representation would allow an effective integration of the work of Darcy and Barnes and provide a conceptual linking of the works of these earlier hydrologists. The characteristics of the different free waters made it apparent that one of these drainages had a very brief residence in the soil - as a consequence, the authors of the Sacramento Watershed Model associated the quickly draining storage with the most rapidly moving free water, that which was temporarily resident in the upper soil. In an attempt to keep the storages necessary for accounting purposes to a minimum, they also conceived this fast-draining upper soil storage as providing the head function necessary for moving the wetting front to deeper soil horizons. The other two free water drainages identified through Barnes' recession analysis exhibited longer soil residence periods. The drainage of the next most rapidly draining aquifer generally took several weeks, while the slowest draining aquifer could take many months or even years to drain. In the Sacramento Watershed Model, the two slower draining free water storages were assigned to a lower zone of the soil while the rapidly-draining free water storage was assigned to the upper zone of the soil.

Our earlier analysis of soil moisture has, however, made it clear that where free water exists in the soil, the tension water requirements of the co-located soil particles must be met. As a consequence, it was necessary to consider tension water storages which paralleled the free water storages. Thus, an upper zone tension water storage and a lower zone tension water storage were required to parallel the free water moisture storages we have discussed earlier.

At this point, a series of soil moisture storages have been considered which provide a parallel to the earlier works of Barnes on recession analysis, Darcy on flow through porous media, and Hanks and Klute on the moisture phases of water and their vertical movement. To this depiction, the authors of the Sacramento Watershed Model found it necessary to add several concepts. Where soils normally contained sufficient moisture such that root systems of plants could regularly obtain necessary moisture without reaching the bottom of the aquifers, the deepest portion of the lower free water storage was exempted from having to resupply tension water. Provision for this capability (which was called Reserve) was included on the assumption that some forms of climax vegetation - the necessary mechanism for effective evapotranspiration at those depths - might not develop root systems which were sufficient to reach the deepest depths of the aquifer system. Thus, a provision for reserving a percentage of the lower zone free water against the claims of evapotranspiration was included on the basis that tension water at those depths would never be utilized by the plant root system.

Another feature involving the lower zone was the recognition that soil depths varied over different portions of the basin. As a result, the lower zone free water aquifers might begin to be recharged in some areas before the aggregate tension water requirement of the deeper soils was satisfied. As a consequence,

a feature was provided which made it possible to direct a portion of the water percolating to the lower zone directly to the free water storages. This parameter, identified as PFREE defined that portion of the percolated water which was distributed to free water aquifers during the period when deep tension water deficiencies were not fully met.

The upper zone of the Sacramento Watershed Model included the free water and tension water storages which were earlier found to be necessary components for a conceptual analysis. However, two additional features were necessary to complete the conceptual description of the model's upper zone. The first of these stemmed from the fact that the area covered by any running stream and the impervious areas immediately adjoining that stream will convert rainfall to runoff with little perceptible loss of volume. Those areas which regularly produce such runoff are considered to be permanently impervious areas. That percent of the basin which exhibits this permanently impervious characteristic is identified in the Sacramento Watershed Model as the percent impervious or PCTIM. No soil moisture accounting is performed for that portion of the area which is identified as permanently impervious.

The other upper zone feature which was necessary, stemmed from the observation that as a basin approached saturation, the area covered by the river expands and an increasing portion of the soil mantle exhibits runoff characteristics similar to the permanently impervious areas. In order to represent this phenomena, the authors of the Sacramento Watershed Model included a variable which allowed the impervious area to swell with increasing basin wetness. This additional impervious area was identified in the model structure as ADIMP.

The goal of the Sacramento Watershed Model is to forecast streamflow from rainfall, utilizing a conceptual representation of the physical processes and storages which influence the rainfall-runoff process. As a consequence, it is necessary to assemble the model so that it depicts discharge in a river system. The use of a watershed's gaged recession curve to describe the characteristic of the free water aquifers is a clear example of the necessary linkage between streamflow and soil moisture storage.

Another feature of the model, particularly necessary for stream flow evaluation, is the impact of evaporation from the stream surface and the evapotranspiration from riparian vegetation adjoining the stream. These conditions modify all flows but become particularly apparent during warm, dry recession periods. The impact of these factors upon the base flow recessions frequently confuse analysts who are attempting to evaluate recession characteristics. This is because recorded streamflow is the result of all of the factors influencing the basin hydrology and does not just represent the total discharge from base flow aquifers. The impact of moisture losses from the

stream and from riparian vegetation areas can be seen most clearly from their influence on recessions during dry periods. These losses of potential streamflow are represented in the Sacramento Watershed Model through the parameter SARVA. SARVA defines the percent of the basin covered by the stream and or vegetation which regularly utilize water for evaporation or evapotranspiration which would otherwise produce base flow. Moisture losses from the SARVA area are subtracted from the volume of water which would otherwise be available to produce streamflow.

In order to utilize the concepts we have discussed, it should be apparent that the Sacramento Watershed Model necessarily includes more than a soil moisture accounting process. In order to achieve the basic water balance with which we initiated our discussion of conceptual models, it was necessary for the Sacramento Watershed Model to complete the total production of a streamflow hydrograph at an outflow point. There have been applications of portions of the Sacramento Watershed Model which attempted to utilize the outputs from the soil moisture aquifers described in the Sacramento Watershed Model with separate linkages to the stream channel. Many such approaches reviewed by the author have included features which were inconsistent with the conceptual processes involved in the model's development. In order to avoid such inconsistencies, the Sacramento Watershed Model was developed as a complete headwater forecasting system. Application of portions of the logic must be carefully constructed to avoid inconsistencies with the conceptual formulation in the basic model.

Descriptions of the principal components of the Sacramento Watershed Model are now nearly complete. However, two additional features must be added which allow modifications of the base flow to conform to those basins where subsurface losses from the free water aquifer do not contribute to observed streamflow. In some areas, the works of nature have resulted in the lower portions of the streambed becoming filled with porous material. In areas of extreme glaciation, the amount of porous material in the bed of a stream can be quite large. Where such conditions exist, the amount of water which can be measured in the stream does not include the flow through the porous bed of the stream. Yet, the voids in the porous area must be filled with water or the stream will immediately sink to fill these voids. The water needed to fill this area is a fixed volume of flow which takes place below the apparent surface of the streambed. The volume of flow necessary to fill this aquifer has been called SSOUT as a representation of the sub-surface outflow it depicts. Recession analysis provides a useful key to the magnitude of SSOUT. Whenever a constant volume can be added to a given base flow recession, and a semi-logarithmic plot of the recession can be transformed to a valid representation of Darcy's Law (i.e., an apparently uniform base flow aquifer), the evidence for SSOUT becomes overwhelming. (See Figure 5.) The magnitude of the flow addition which linearizes the semi-logarithmic plot provides a definition of SSOUT.

APPROXIMATION OF SSOUT

The Constant Streamflow Loss Term

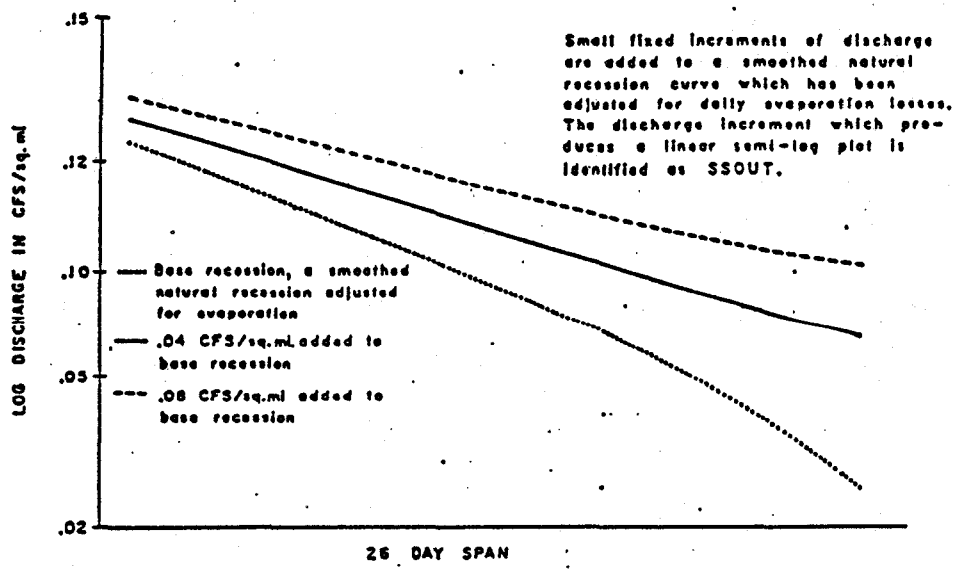


FIG. 5

One other element completes the base flow picture for the conceptual hydrologic model we have been describing - potential base flow which leaves the basin without affecting base flow conditions in the river. This additional element to the base flow depiction represents that base flow, which flows outside the channel. This component is identified as SIDE in the model. It is represented as a decimal fraction of the base flow which can be perceived in the stream. A side value of 1 would mean that as much base flow exits the basin outside of the river channel as is observed within the channel. With the description of these two base flow characteristics, SSOUT and SIDE which modify the basin's watertight boundaries, we have completed the description of the model components.

Before developing the functions linking the concepts in the Sacramento Watershed Model, let's review the organization of the storages and processes we have discussed. Figure 6 is a depiction of these concepts in a manner which graphically suggests their relative positions and their contributions to the streamflow process. To the descriptive names for the components identified in Figure 6, Figure 7 adds the neumonics which have been adopted for use as computer designations in the Sacramento Watershed Model. These neumonics and what they represent are:

PCTIM - A decimal fraction expressing the percent of the basin which is regularly impervious and which is effectively linked to this stream channel such that it contributes to instantaneous runoff.This water bypasses the storages in the upper zone.

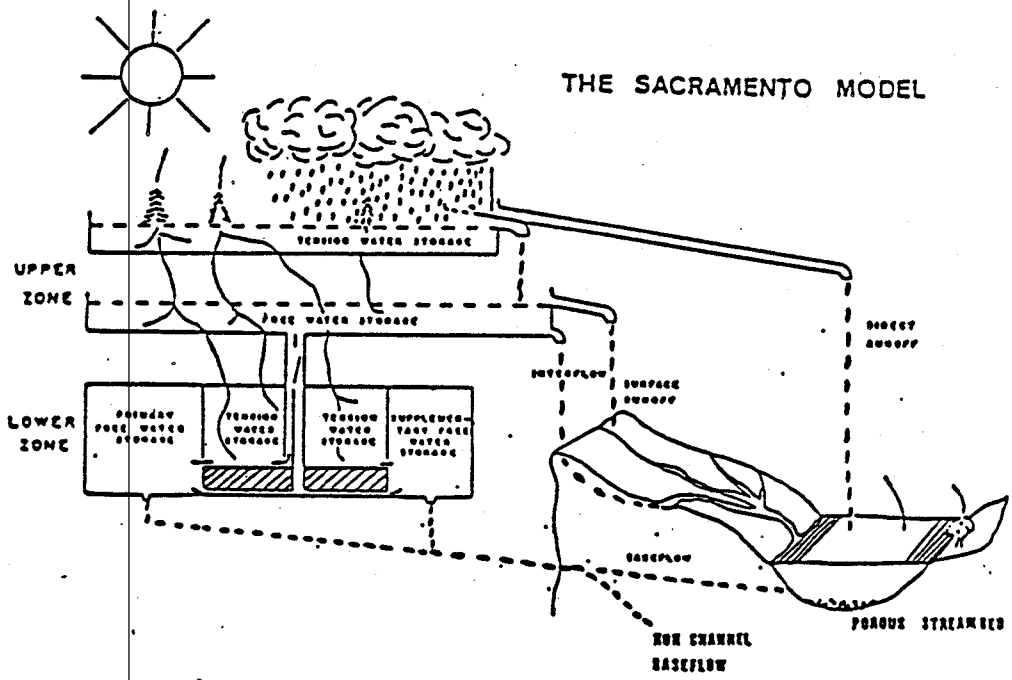

THE SACRAMENTO MODEL
UPPER ZONE
LOWER ZONE
NON CHANNEL BASEFLOW
POROUS STREAMBED

FIG. 6

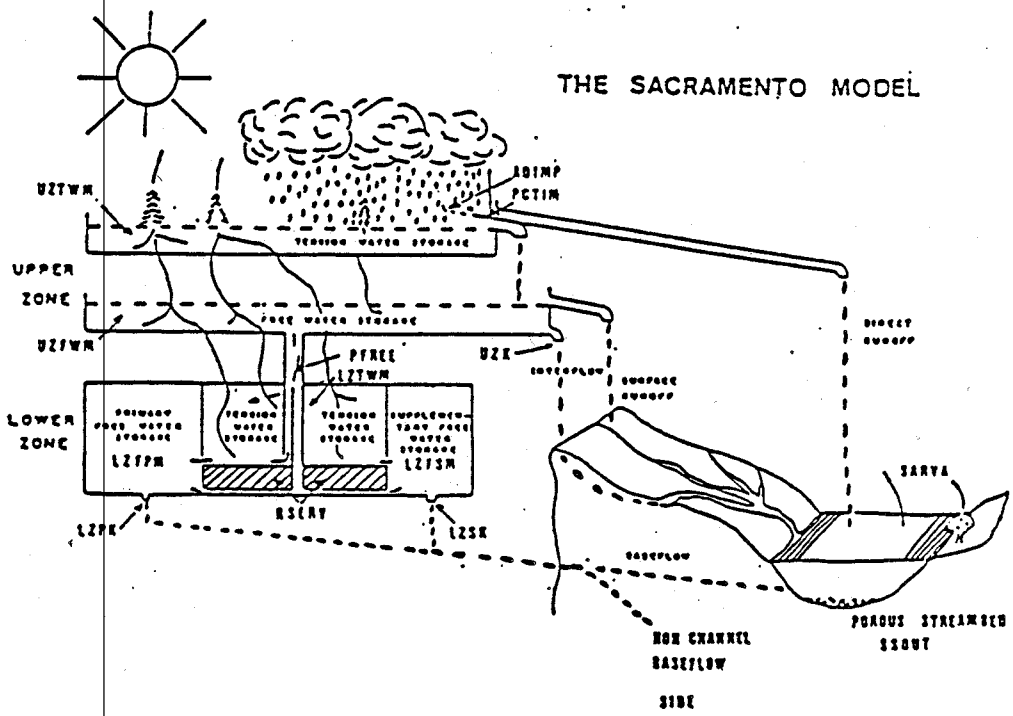

THE SACRAMENTO MODEL
UZTWM
ADIMP
PCTIM
UPPER ZONE
UZFWM
PFREE
LZTWM
UZK
LOWER ZONE
LZFPM
LZFSM
RSERV
LZPK
LZSK
SARVA
NON CHANNEL BASEFLOW
POROUS STREAMBED
SSOUT
SIDE

FIG. 7

ADIMP - That additional fraction of the basin area which develops impervious characteristics when the soil mantles field capacity is fully met.

UZTWM - Upper Zone Tension Water Maximum, the maximum volume of water which can be stored by the upper zone between the field capacity and the wilting point. This moisture capacity applies to that volume of soil moisture which can be lost by direct evaporation from the soil surface or through the evapotranspiration process of shallow rooted vegetation. This storage has a strong attraction for water and must be filled before any water in the upper zone is available for transfer to other storages.

UZFWM - Upper Zone Free Water Maximum, the limiting capacity of upper zone free water. Upper zone free water provides the source of water for interflow and the driving force for transferring water to deeper depths.

UZK - Defines the daily rate at which water stored as Upper Zone Free Water can drain as interflow.

PFREE - Establishes the minimum proportion of water percolating from the upper zone to the lower zone which is available for recharging the lower zone free water storages.

LZTWM - Lower Zone Tension Water Maximum - the maximum capacity of lower zone tension water. This water can only be removed from the soil through evapotranspiration.

LZFSM - Lower Zone Free Water Supplemental Maximum - the maximum capacity of lower zone supplemental free water; i.e., the maximum volume of free water storage from which supplemental baseflow can be drawn.

LZSK - Defines the daily rate at which water stored in Lower Zone Free Water Supplemental can drain as supplemental baseflow.

LZFPM - Lower Zone Free Water Primary Maximum - the maximum capacity of lower zone primary free water, i.e., the maximum capacity from which primary baseflow may be drawn.

LZPK - Defines the daily rate at which water stored in Lower Zone Free Water Primary can drain as primary baseflow.

RSERV - The decimal fraction of lower zone free water which due to its depth below the root system is unavailable to resupply lower zone tension water.

SIDE - The decimal fraction of observed base flow which leaves the basin by non-channel underground routes.

SSOUT - The volume of flow which can be conveyed by porous material in the bed of the stream. This volume is generally so small that it can be set to zero. However, if the riverbed is quite permeable, it can assume a significant magnitude.

SARVA - A decimal fraction representing that portion of the basin normally covered by streams, lakes and riparian vegetation which can deplete stream flow by evaporation and/or evapotranspiration.

PERCOLATION

The storages UZTWM, UZFWM, LZTWM, LZFSM, and LZFPM represent the maximum moisture-holding capacity of those portions of the soil mantle which influence the rainfall-runoff process. In order to maintain an accounting of the volume of water in each of these storages, the moisture contents of each storage are identified respectively as UZTWC, UZFWC, LZTWC, LZFSC, and LZFPC.

Over the permeable portion of every catchment, the Sacramento Watershed Model visualizes the components we have assembled as interacting in the following way. The initial soil-moisture storage identified as Upper Zone Tension must be totally filled before moisture becomes available to enter other storages. Upper Zone Tension represents that volume of precipitation which would be required under dry conditions to meet all interception requirements and to provide sufficient moisture to the upper soil mantle so that additional water becomes available for percolation to deeper zones and, if sufficient water is available so that horizontal drainage can begin. When the Upper Zone Tension volume has been filled, excess moisture in the upper soil becomes available for distribution from Upper Zone Free Water. The Upper Zone Free Water is free to transfer water to deeper portions of the soil mantle or to move laterally through the soil in response to gravitational and pressure forces. The Upper Zone Free Water storage supplies water for percolation to lower zones and for interflow. The demands imposed upon the Upper Zone Free Water vary with the amount of water available in the upper zone. Once the Upper Zone Free Water meets the immediate percolation requirement, the residual volume is available to generate interflow. Thus, interflow is proportional to the available free water volume after percolation, i.e.,

INTERFLOW = UZK*UZFWC

where

UZK is the Upper Zone Free Water daily storage depletion coefficient,

and

USFWC is the volume of free water temporarily stored in the upper zone.

The rate of vertical drainage, the percolation to deeper soils, is controlled by the contents of the Upper Zone Free Water and the deficiency of lower zone moisture volumes. The preferred path for moisture in Upper Zone Free Water is considered to be downward as percolation. Horizontal flow in the form of interflow occurs only when the volume stored in the Upper Zone Free Water exceeds the rate at which percolation can occur from the Upper Zone Free Water. Thus, interflow requires that the

precipitation rate must be in excess of the percolation rate. When the precipitation rate exceeds both the percolation rate and the interflow drainage capacity, then the Upper Zone Free Water capacity can be filled completely. Excess precipitation will then result in surface runoff. Surface runoff produced by these mechanics is a highly rate-dependent volume. Depending upon soil porosity, a rainfall volume which falls in a few hours can produce substantial runoff while the same rainfall volume spread over a day may have little effect upon streamflow. The rate of surface runoff is determined by the rate of precipitation application, the degree of dryness of the different zones and the permeability of the soils.

Lower Zone Tension Water Maximum is that depth of water held by the lower zone soil after wetting and drainage which is generally available for evapotranspiration. The two lower zone free water storages, Primary and Supplemental, represent those volumes which are available for drainage as baseflow or sub-surface outflow not appearing in the channel. These free water storages fill simultaneously from percolated water and drain independently at different rates, giving a variable ground water recession.

The percolation mechanics of the Sacramento Watershed Model have been designed to correspond with the observed characteristics of the motion of moisture through the soil mantle. This has been accomplished by utilizing the storages we have analyzed to drive a moisture transfer system. The mechanics of transfer from upper zone to lower zone volumes is based upon the computation of a lower zone percolation demand. When the lower zone is totally saturated, then the percolation into the lower zone must be limited to a value equal to that value which is draining out of the lower zone. This limiting drainage rate is computed as the sum of the products of each of the two lower zone free water storages and its respective drainage rate. This limiting rate of drainage from the combined lower zone storages is defined by the system as PBASE. PBASE, the percolation base rate, represents the drainage from the soil moisture system when the soil moisture system is saturated. Mathematically, PBASE can be expressed as:

PBASE = (LZFSM*LZSK) + (LZFPM*LZPK),

where

LZFSM = lower zone free water maximum supplementary storage, which is the maximum storage capacity for faster drainage baseflow.

LZSK = lower zone supplementary daily storage depletion coefficient,

LZFPM = lower zone free water maximum primary storage, which is the maximum storage capacity for slower draining baseflow, and

LZPK = lower zone primary daily storage depletion coefficient.

Although PBASE defines the maximum percolation into a saturated soil, much higher rates of percolation will occur when the soil is dry. Regardless of the magnitude of percolation under dry conditions, that magnitude can be defined as a multiple of the PBASE rate such that:

MAX PERCOLATION RATE = PBASE*(1+ZPERC)

Where ZPERC, a percolation maximizing constant, provides the necessary multiple to increase percolation from the saturated rate, PBASE, to the maximum rate under dry conditions. Inasmuch as the maximum percolation occurs when the upper zone is saturated and the lower zone is dry, then the percolation demanded by the lower zone can be represented as:

$$\text{LOWER ZONE PERCOLATION DEMAND} = \text{PBASE}*(1+\text{ZPERC}*f(\frac{\text{Lower Zone Deficiency}}{\text{Lower Zone Capacity}}))$$

Following the work of many experimenters with infiltration, it is reasonable to assume that the change in lower zone percolation demand is exponentially related to the ratio of (lower zone deficiency/lower zone capacity). This allows us to express the equation for percolation demand with varying soil moisture as:

$$\text{Percolation Demand}=\text{PBASE}*(1+\text{ZPERC}(\frac{\Sigma(\text{Lower zone capacities less contents})}{\Sigma(\text{Lower zone capacities})})^{\text{REXP}})$$

The actual percolation must, however, also be controlled by the supply of available water. Consequently, the effective percolation must be obtained by modifying the percolation demand on the basis of the available supply. This leads us to the relationship:

$$\text{Percolation}=\text{Percolation Demand}*(\frac{\text{Upper zone free water content}}{\text{Upper zone free water capacity}}).$$

Assuming a continuously full upper zone free water, this relationship of percolation demand versus lower zone soil-moisture is illustrated in Figure 8 for several values of REXP. Percolation curves from this relationship allow a very close parallel to the infiltration curves observed in field tests. Total moisture movement into the soil has a potential equal to the volume necessary to replace the upper zone moisture losses which resulted from transpiration, horizontal drainage and vertical percolation. It is the sum of these moisture losses

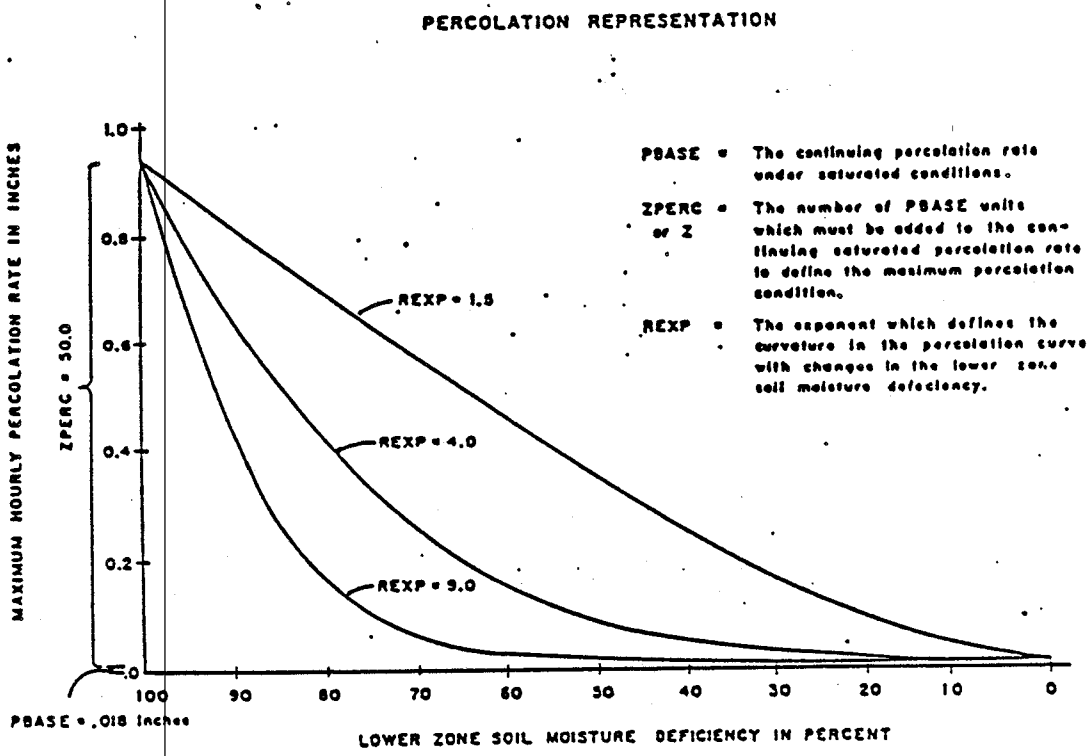
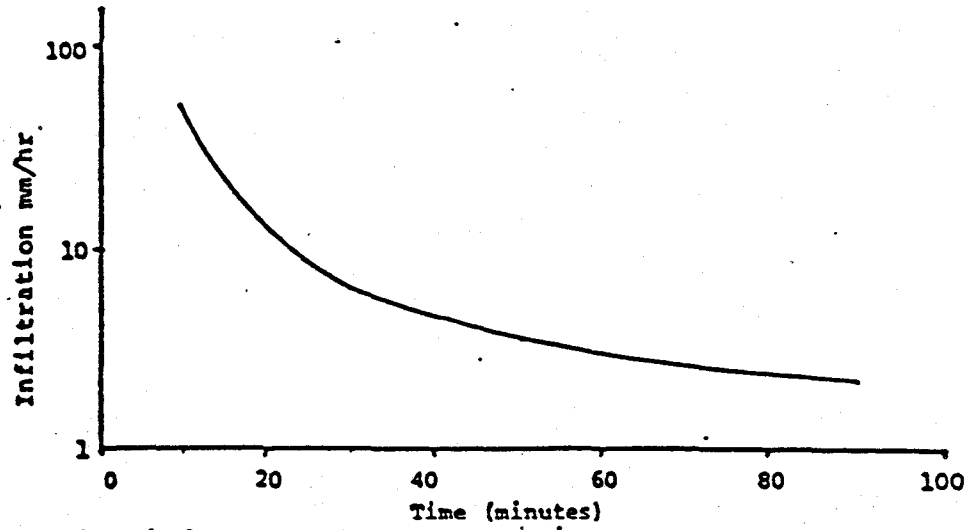
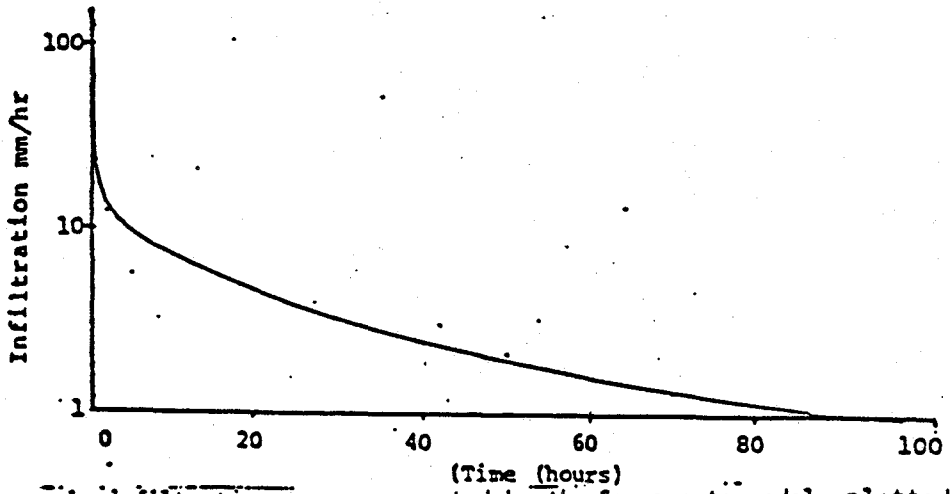


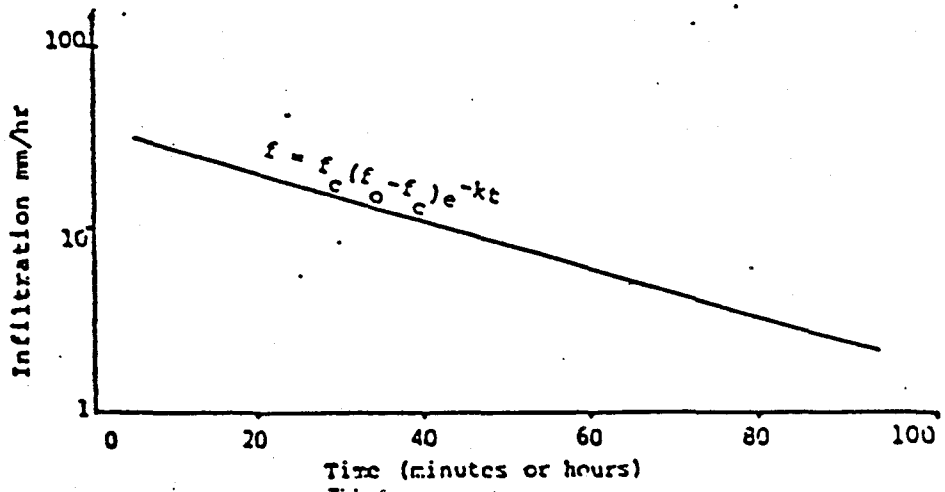
Figure 8



A typical observed infiltration curve plotted as log f vs time. (Replotted from Linsley, Kohler and Paulhus, Houston black loam.)



An infiltration curve generated by the Sacramento model, plotted as log f vs time. (Parameters fitted to Arroyo Seco, California. Excess water applied after wetting and prolonged drying.)



Horton's equation plotted as log f vs time.

FIG. 9

which determines the maximum infiltration rate at any given time. Continuous filling of all the storages will generate infiltration curves which are directly analogous to the work on infiltration by Horton. While interrupted infiltration is described in a manner which effectively parallels the combined effects induced by percolation and evapotranspiration (see Fig. 9). Thus, percolation is defined by an interrelationship between soil drainage characteristics, time variant soil-moisture conditions and the rate at which rainfall is applied to the soil. These features produce a highly time-dependent percolation capability which produces runoff in response to the rainfall volume and the rainfall rate.

The volume of rainfall which is percolated to the lower zone is divided among three significant soil-moisture storages. The first of these, lower zone tension, represents that volume of moisture in the lower zone soil which will be claimed by dry soil particles when moisture from a wetting front reaches that depth. Tension water deficiencies are associated with a suction force, so they tend to absorb all percolating water until these deficiencies are satisfied. However, variations in soil conditions and rainfall amounts over a drainage basin cause variations from the average condition. The effect of these variations is approximated in the model by diverting a fraction of the percolated water into lower zone free water storages before lower zone tension water deficiencies are fully satisfied (PFREE). An examination of Figure 7 indicates that water percolating from the upper zone free water to the lower zone may go totally to tension water, or some fraction of the percolated water, PFREE, may be made available to the primary and supplementary storages. At any time that the lower zone tension storage becomes totally filled, continued percolation is divided between the two lower zone free water storages. At all times, water made available to primary and supplementary storages is distributed between those lower zone storages in response to their relative deficiencies.

During our discussion of the percolation process, we have introduced three new terms:

PBASE - The constraining percolation rate when the soil system is saturated, PBASE is defined as being equal to LZFSM*LZSK+LSFPM*LZPK.

REXP - The exponent in the percolation relationship which determines how percolation varies with different degrees of soil wetness.

ZPERC - The multiple which must be applied to PBASE so that the maximum percolation rate (ZPERC+1)*PBASE can be defined.

We have discussed the motion of water into and through the soil at some length but, up to this point, we have paid little attention to the formulation of that portion of the hydrologic cycle which we have identified as evapotranspiration.

EVAPOTRANSPIRATION

The loss of moisture from a river basin takes place from drainage and from a combination of evaporation and evapotranspiration. The Sacramento Model uses a single relationship to express both of these phenomena - where soil moisture losses are primarily from vegetative uses, this provides an effective simplification. However, under those conditions where a river basin has a significant percentage of water surface and the surrounding soils could become quite dry, then separate evaporation and evapotranspiration would be desirable. Thus far, the authors have analyzed many hundreds of basins without encountering a problem associated with the use of a single relationship which lumps evaporation with evapotranspiration. An analysis of the reason this is true, suggests that it stems from evapotranspiration generally being an order of magnitude more significant to catchment runoff than is evaporation.

In the Sacramento Watershed Model, water is removed from the stream surface and from phreatophyte vegetation areas at the potentional rate. Over other portions of the soil mantle, evapotranspiration uses varies with both evapotranspiration demand and the volume and distribution of tension water storage. As the soil mantle dries from evapotranspiration, moisture is withdrawn from the upper zone tension water at the potential rate multiplied by the proportional loading of the upper zone tension water storage. In the lower zone, evapotranspiration takes place at a rate determined by the unmet potential evapotranspiration times the ratio of the lower zone tension water content to total tension water capacity.

If evapotranspiration should occur at such a rate that the ratio of contents to capacity of tension water, then water is transferred from available free water to tension water and the relative loadings are balanced. This makes it possible to maintain a moisture profile that is logically consistent. Depending upon basin conditions, some fraction of the lower zone free water is considered to be below the root zone and therefore unavailable for such transfers. Starting with a saturated soil, and exposing it to a constant evapotranspiration demand would produce an effective evapotranspiration use curve of the type illustrated in Figure 10.

Various algorithms have been utilized to compute evapotranspiration demand. Hounam (1971) has reported on an approach suitable for such a computation. The authors of the Sacramento Watershed Model have, however, developed a distinct preference for utilizing a normal evapotranspiration demand curve which is

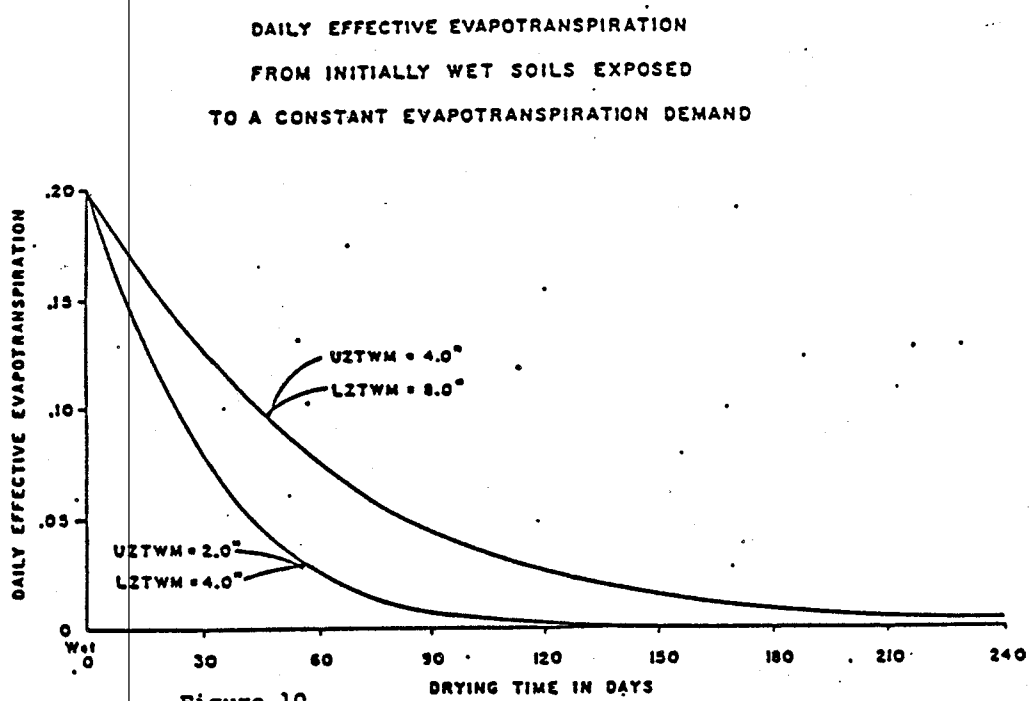


Figure 10

APPROXIMATE
AVERAGE DAILY EVAPOTRANSPIRATION DEMAND
IN MM

	SPRING		SUMMER		FALL		WINTER	
	WET	DRY	WET	DRY	WET	DRY	WET	DRY
HOT	3	4	6	7	3	4	2	2
WARM	2	3	5	6	2	3	1	1
COOL	1	1	3	4	1	2	0	1
COLD	1	1	2	2	1	1	0	0

Figure 11

established from the mean daily value for the mid-point of each month.

In a number of basins an attempt has been made to redimension daily pan evaporation data to provide an estimate of the daily evapotranspiration demand. In a few events within any given record, the results of using daily estimates of evapotranspiration generally show improved reconstitution of the catchments streamflow. Unfortunately, these improvements have been negated by other events whose reconstitution have been penalized by the differences between the evapotranspiration process and the evaporation process. As a result, the authors have concluded that although average evaporation data can assist in evaluating an average evapotranspiration demand, the use of discrete daily evaporation data is as likely to be misleading as it is to be helpful. A very rough guide to daily evapotranspiration which approximates the values of ET the authors have experienced is tabulated in Figure 11.

RUNOFF TIME DISTRIBUTION

At this point, we have examined most of the concepts on which the Sacramento Watershed Model is based. However, as the simulation process is applied to and verified by actual streamflow conditions, the need for some additional considerations becomes apparent. It should be remembered that streamflow recessions are simulated by drainage from groundwater aquifers. These recessions are influenced by subterranean losses to ancillary channels - either porous aquifers below the headwater channel, or sub-surface outflow outside of the normal surface channel. In addition, flows at the point of reconstitution are affected by evaporation from the water surface and from the phreatophytes which take moisture from the channel or from that water which is enroute to the channel. Although these features will define a large portion of the flow regime, that water which flows rapidly toward the river system must be treated with some form of runoff distribution function. Runoff distribution functions reshape the momentary sources of runoff into a shape which conforms to the timing of the outflow hydrograph of the watershed. For this purpose, two tools are utilized. The first tool is a histogram of flows vs. time. The histogram is a first cousin to the well-known unitgraph concept. The second tool affecting runoff distribution is a routing process, which is applied after the histogram to provide a nonlinear flow vs. time relationship.

The combined processes provide for a wide variety of flow characteristics which closely parallel channel conditions. In combination, the two techniques overcame many of the linearity problems which limit the application of unitgraph concepts. It should be pointed out that the time histogram which is utilized for the initial time vs flow distribution is based upon those flows which reach the outflow point with the greatest

efficiency. If varying volumes of flow exhibit different characteristics, then the utilization of the sharpest time vs. flow distribution allows the channel storage feature to provide the necessary change in channel efficiency to maintain a close parallel between forecast and observed flows.

The routing mechanics utilized to reshape channel flows are based upon a layered Muskingum concept. Four levels of flow are provided within the Sacramento Model to allow variable degrees of attenuation based upon modifying the discharge hydrograph produced by the histogram distribution. When the channel storage feature is utilized, the volumetric capacity of each of the discharge layers must be determined as well as the routing coefficient appropriate to each layer. As an example, period outflow volumes might be developed from a histogram distribution which produced the discharges sequence:

1.0 2.0 5.0 20.0 10.0 4.0 2.0

If the river rose to levels which reflected different channel storages than those described by the histogram application, the histogram produced volumes might be transformed with layer widths of 5.0, 10.0, and 5.0 with routing coefficients of 1.0, .8, and .5 in the following manner:

LAYER 1

Width 5.0 Flow Component up to 5.0 with coefficient 1.0
Layer 1, Input and Output

1.0 2.0 5.0 5.0 5.0 4.0 2.0

LAYER 2

Width 10.0, Flow Component> 5.0 but less than or equal to 15.0 with coefficient .8

Layer 2, Input

0.0 0.0 0.0 10.0 5.0 0.0 0.0

Routing

8.0 4.0
1.6 0.8
0.32 0.16
0.06

Layer 2, Output

0.0 0.0 0.0 8.0 5.6 1.12 0.22

LAYER 3

Width 5.0 Flow Component> 15.0 but less than or equal to 20.00 with coefficient .5

Layer 3, Input

0.0 0.0 0.0 5.0 0.0 0.0 0.0

Routing

2.5
1.25
0.62
0.31

Layer 3, Output

0.0 0.0 0.0 2.5 1.25 0.62 0.31

Layers 1-3

1.0 2.0 5.0 15.5 11.85 5.74 2.53

NOTE: The summation of the layers must include the entire range of possible flows, or truncation of high flow values will occur.

The channel storage feature will generally be appropriate where significant changes in channel characteristics take place with changing depth of the flood wave. In addition, routing will frequently be required for very small flows which wend their way so tortuously through the river system that they exhibit pronounced channel storage. It should be emphasized that the routing feature which produces a nonlinear runoff response is an option which can be bypassed unless the simpler time distribution of the histogram is clearly inadequate.

MODEL FITTING

At this point, we have assembled all of the concepts which are necessary to generate an effective simulation of headwater flows. These components and the logic which links them are the Sacramento Watershed Model. It should be emphasized that all of these components are not necessary in every basin, but all of the components are necessary to allow the Sacramento Watershed Model to be applied to a wide variety of highly diverse river conditions. The integration of these conceptual processes provides a remarkably flexible capability which, if properly applied, can substantially enhance many forms of hydrologic analysis and forecasting. The attempt to provide a physically realistic but parametrically simple model has provided a very powerful tool for hydrologic analysis. The flexibility of the resulting formulation and the number of descriptive parameters is necessary for general application; however, care must be taken to assure that the model features are used in a hydrologically rational manner. The maximum number of parameters should be established by a hydrologic reading of the streamflow record, not by attempting to match a streamflow record by a random juggling of parameters.

The very first decision which must be made should be determined by resorting to the primitive accounting process which should never be neglected.

Runoff = Rainfall - Evapotranspiration

In most areas, runoff is the most effectively measured of these three processes. If the analyst quickly notes the annual volumes reported for runoff and rainfall, and makes an estimate of the annual evapotranspiration which must have taken place to support the basin vegetation, he can generally spot any substantial problems with rainfall interpretation or subsurface aquifer systems which are likely to compromise the simulation. It should be noted that the simple water balance does not have to

be met in a single year. Changes in soil moisture storage are likely to prevent that condition from occurring, but over a period of a decade soil moisture changes will generally be obscured and the average rainfall-runoff and evapotranspiration should emerge. It should be emphasized that just as the rainfall and runoff represent the value which occurred and not the potential, the evapotranspiration in the simple runoff equation will be that which occurred. Unless the basin is quite wet, actual evapotranspiration can differ remarkably from the evapotranspiration potential. Figure 12 will provide a rough guide to the annual effective evapotranspiration in an area. This graph is only intended as an approximation of annual effective ET. Variability in plant systems, latitude, atmospheric humidity, winds, the seasonality of precipitation, and other factors all influence the actual evapotranspiration and will result in deviation from the indicated values. Even so, the relationship provides a first estimate which should help the analyst.

The primary reason for reviewing the water balance is to attempt to place the precipitation input in the proper scale for successful modeling of the streamflow. Due to wind influence, gage shape, and exposure problems, measured rainfall is frequently about 85% of that which actually reaches the land surface. In addition, there are areas where orographic features of a basin influence the rainfall pattern and can affect the ratio of the recorded rainfall to basin rainfall. In areas where convective precipitation occurs, the recorded precipitation may have a nearly random relationship to basin precipitation. As a result of these and other problems, careful attention must be paid to utilizing the available precipitation data so that it provides the most useful approximation of actual basin rainfall. The challenge of dimensioning recorded precipitation so that it reflects the necessary water balance appropriate to model fitting has been neglected by many modelers. The resulting distortions to model parameters and the systems simulation capability can be severe. Such analyses will generally have little application to events which are outside the scale of the original fitting.

Another common pitfall which confuses the simulation is the lack of consistency in measurement techniques at the rain gage site. All too often, changes in measuring equipment, gage location or site conditions are unrecorded and a nonuniform record of rainfall measurements is identified as a single continuous record. The relative error introduced to the simulation process by precipitation measurement vagaries can be compared against the relative error introduced by variations in a simulation parameter (Figure 13).

INCREMENTAL EFFECT OF 10% CHANGES IN BASIC INPUT
OR PARAMETER VALUES AS EVALUATED WITH
THE SACRAMENTO MODEL

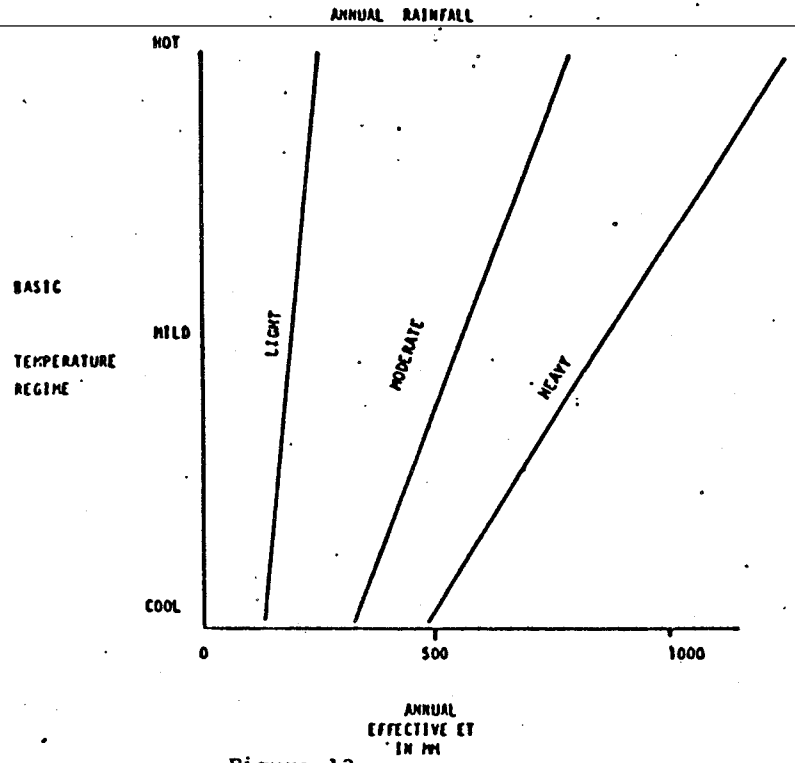


Figure 12

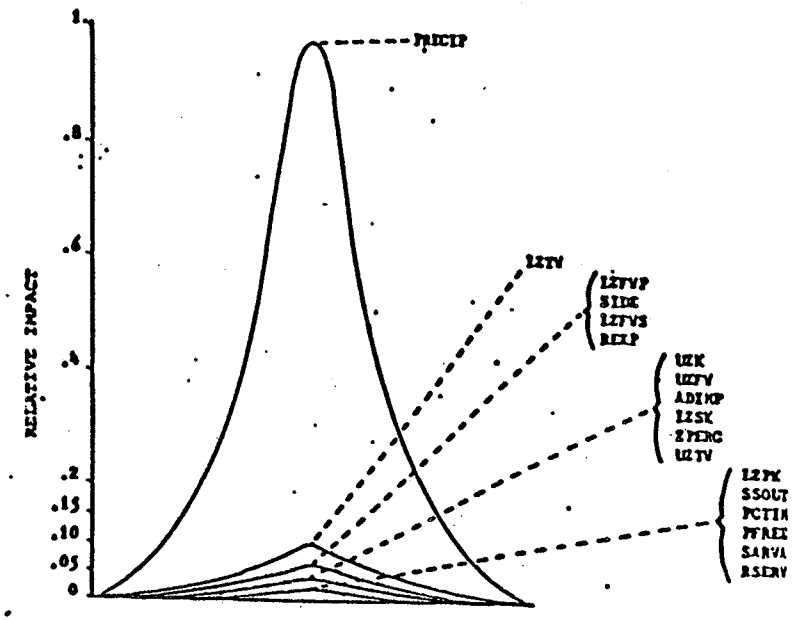


Figure 13

Any change in rainfall measurement can produce an error in runoff evaluation which is an order of magnitude greater than a similar percentage change in any model parameter. The important factor to gain from such a comparison is that precipitation is the driving component of the hydrologic system unless precipitation is consistently measured and properly dimensioned, the modeling effort will be compromised.

During the course of this presentation, we have raised caution concerning the precipitation data, the evapotranspiration estimate, and the runoff record. Hopefully, with a somewhat better appreciation of some of the basic flaws in hydrologic data sets, we may be better prepared to consider calibration of the model itself. Remember, that the model is a mathematical simplification of a sequence of natural processes. Where the records provide irrational inputs (and many records do), it is necessary to make logical decisions as to why certain phenomena have been reported and to subjectively evaluate the simulation capability accordingly. Inasmuch as automated optimization systems applied to model parameters can not make such evaluations, data irregularities create one of the principal constraints in using automated fitting procedures.

The complexities of natural data sets, such as changing a discharge rating at an arbitrary time, rainfall being lower than runoff due to strong winds reducing rainfall catch, lack of suitability of the rain gage locations, rises in streamflow from unreported rainfall, and other problems which are implicit in the data sets, should always be kept in mind. Indeed, applying a streamflow simulation model in an intelligent manner requires a high level of hydrologic acumen and familiarity with all the measurement problems which can mislead the unwary. With these words of caution, it is time to "read" the daily record which a properly dimensioned rainfall input and a concurrent streamflow record provide.

Through "reading" the historical record, we are attempting to approximate as many of the parametric representations in the simulation model as can be deduced from the historical records of precipitation and streamflow. Generally, the features which can be closely determined from historical records will include:

PCTIM - the percent of the basin which is impervious.

ADIMP - the additional impervious under saturated conditions.

UZTWM - the upper zone tension water maximum.

UZK - the upper zone free water daily interflow depletion rate.

LZSK - the lower zone daily depletion rate of the more rapidly draining base flow.

LZPK - the lower zone daily depletion rate of the slowest draining base flow.

In addition, we should be able to make a reasonable estimate of LZTWM, the minimum size of LZFSM, the minimum size of LZFPM, and the magnitude of PFREE. In order to "read" the streamflow record, a preliminary run of the Sacramento Watershed Model with a standard set of system values can be helpful. Assuming that the rainfall and evapotranspiration have been reasonably dimensioned on the basis of a water balance analysis, an initial simulation based upon the following values can simplify parameter evaluation.

UZTWM	100 mm
UZFWM	40 mm
LZTWM	150 mm
LZFSM	30 mm
LZFPM	100 mm
ADIMP	.02
PCTIM	.01
SARVA	.01
SIDE	0
SSOUT	0

RSERV	.2
PFREE	.2
UZK	.4
LZSK	.08
LZPK	.004
ZPERC	35.
REXP	1.5

These 17 parameters are the basis of the Sacramento Watershed Model. Although it is also necessary to define the precipitation field, the annual ET curve, and the time distribution of runoff, the 17 values listed are the descriptive parameters associated with the Sacramento Model. If we review a period of record by inserting these parameters into the model, we can gain some useful insight into many of the values which are appropriate for a particular watershed.

The first parameter to be defined by inspection is PCTIM - the percent of the basin which produces impervious runoff. Scan the record for small runoff events which occur when the UZTW is not full- that is, there has been enough intervening evapotranspiration since the last storm so that the rainfall event has the potential for being completely stored within the Upper Zone Tension Water Storage. Runoff under these conditions must be from the impervious area. The ratio of rainfall to storm runoff accumulated for a number of events provides an effective estimate of PCTIM. If we have taken an adequate sample, our fitting of PCTIM is complete.

ADIMP, the additional percent of the basin which becomes impervious when the basin is quite wet, can be estimated by finding a small storm following soon after a major runoff event which filled the tension water storages. The precipitation which occurs should be less than the evapotranspiration which has occurred since the previous rain. The ratio of the runoff volume generated by the light rain to the rainfall volume which produced the additional increment of runoff defines the total area of the basin which has impervious characteristics. By subtracting the PCTIM area from this computation, the ADIMP can be evaluated. As with all determinations of parameter values, the more cases which can be used to support the computation, the sounder the analysis. Generally, the value of ADIMP will not exceed 10% of the basin. Frequently, it is so small that a zero value can be used. If a large value of ADIMP is computed, a review of the basin's topography should show extensive marshes, overflow areas, or other water surfaces becoming an increasing feature of the basin during wet periods. In many areas, particularly rough terrain,

or areas with relatively deeply-incised streams, the value of ADIMP may be sufficiently close to zero so that ADIMP is effectively non-functional.

Inasmuch as UZTW must be filled before any runoff in excess of the impervious runoff can occur, very dry periods should be examined. The amount of rainfall which must be applied after a dry period before any runoff in excess of the impervious fraction can occur is a depiction of UZTWM. This value of rain is a measure of the rainfall deficit which must be met to replace water lost from UZTW. If the dry period is long enough, the rainfall deficit will be the actual value of UZTWM. This value will generally be relatively small in areas of sparse vegetation and generally relatively large in areas with heavy growths of plant materials.

LZTWM, the lower zone tension water maximum storage, can be closely evaluated in areas where the rainfall follows a marked seasonal pattern. When a computer run over several years of data is performed, the error in the forecast runoff volumes associated with that period of the runoff year when the Lower Zone Tension Water should be filling, provides a quick way of modifying the initial estimate of LZTWM size. As additional parameters are fitted and the simulation converges on the observed runoff, the error in runoff volumes during that period when excess water is directed primarily to Lower Zone Tension Water will continue to be a guide to the volumetric adjustment of LZTWM. A properly sized LZTWM is needed to obtain an optimum water balance. Inconsistencies in the annual precipitation catch will have a major impact upon the ability to define LZTWM.

LZPK - this value is based upon the rate at which base flow runoff changes per day. Conceptually, it is one of the simplest parameters to determine. Unfortunately, the effects of evaporation from the stream surface and from surrounding riparian vegetation can somewhat cloud this determination.

The best procedure is to take an extended period from a few weeks after a major storm to just prior to an event which produces runoff in excess of the impervious fraction. Utilizing a period which is not marred by other than impervious area runoff, will allow a reasonable determination of LZPK. This equation which allows the determination of LZPK is:

$$LZPK = 1 - (QP_t/QP_o)^{1/t}$$

This equation has been utilized to produce Figure 14. Use of Figure 14 helps in appreciating the consistency of LZPK even though there may be minor problems in assessing the recession length.

SSOUT - If the periods utilized for determining LZPK are plotted on a semi-logarithmic coordinate system, some interesting attributes of the stream may be observable. If the actual

recession does not approximate a straight line but is convex during a period when evapotranspiration is relatively low, then the addition of a constant volume to the observed points may result in a more linear appearance to the semi-logarithmic representation of the recession. The constant which must be added to the recession points is an indication of the base flow which is being claimed from the flowing stream in order to fill the porous aquifers in the base of the streambed. That volume of flow which translates a convex representation of the recession to a linear recession (on a semi-logarithmic plot) is the volume which should be assigned to SSOUT (see Figure 5).

SARVA - A semi-logarithmic plot of a recession which takes place during warm periods may show a concave shape. Provided the recession curve shows the greatest departure from a linear condition when the evapotranspiration demand is highest, the loss from the channel provides a measure of SARVA the stream and riparian vegetation area. If, during summer heat, the streamflow volume is .03 mm of runoff per day less than a rational recession, and the estimated daily evapotranspiration potential is 6 mm per day, then the area affected by SARVA is .03/6 or .005. That is, five thousandths of the basin is withdrawing water directly from the stream or its adjoining underground aquifers.

LZSK determines the rate of the faster draining base flow recession which is superimposed upon the primary base flow. This recession value and the volume it drains from LZFSM can be estimated by following the process illustrated in Fig. 15.

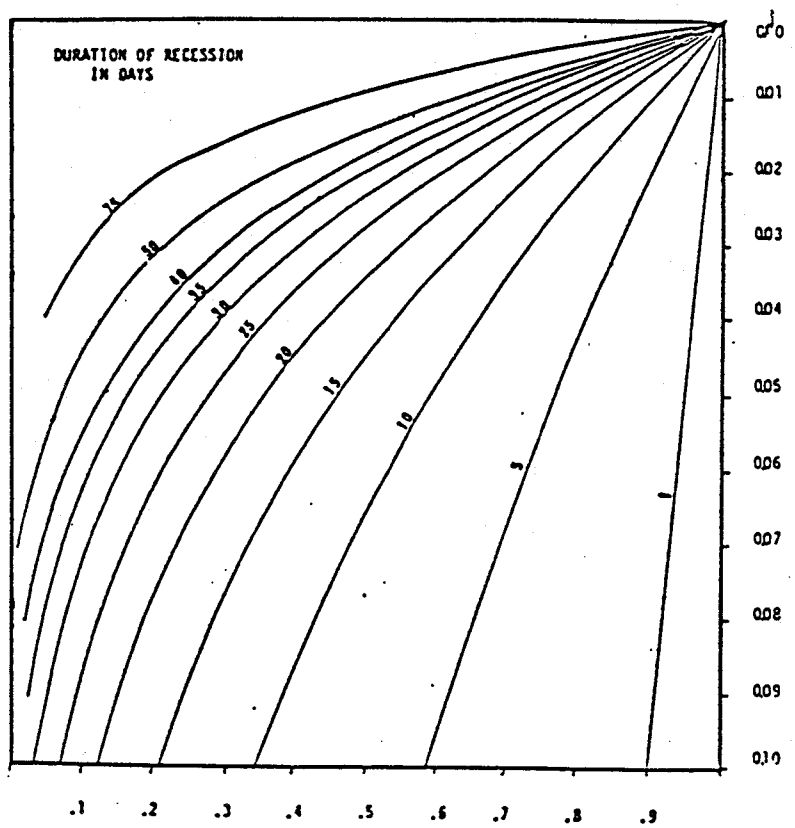


Figure 14

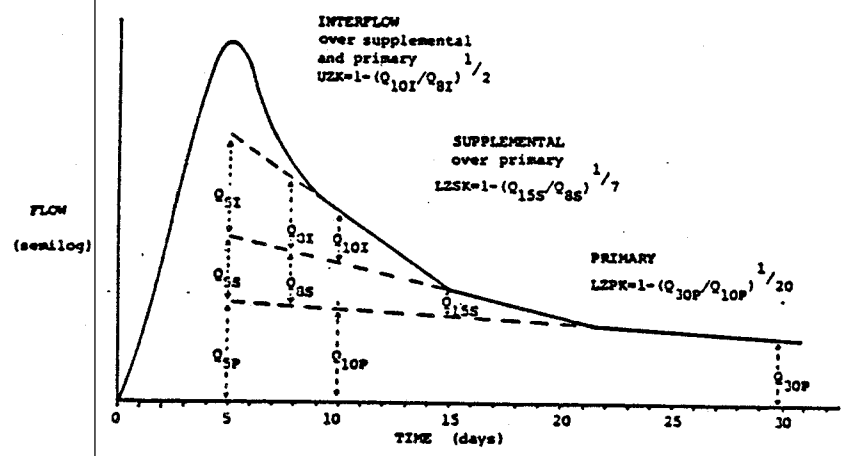


Figure 15

maximum observable volumes are

INTERFLOW = Q_{5I}/UZK

SUPPLEMENTAL = Q_{5S}/LZSK

PRIMARY = Q_{5P}/LZPK

LZFSM - Under very wet conditions, the computed value of LZFSM may be quite close to the actual value of LZFSM. However, it has been the authors experience that rarely is the soil sufficiently wet so that the full value of LZFSM can be computed from a hydrograph. Generally, a truer value of LZFSM will be about 60% higher than the computed value.

LZFPM - The problem of finding a wet enough condition to establish LZFPM is similar to the problem encountered in establishing LZFSM. The mathematical approach of determining LZFPM is a direct parallel to that utilized in estimating LZFSM. That is, LZFPM is equal to the highest value of lower zone free water outflow which can be found in the record divided by the rate at which the LZPK drains lower zone primary free water. The maximum size of LZFPM will however generally require less of an expansion from its computed value than will LZFSM.

PFREE - the parameter PFREE has a particularly unique feature, one which an analyst can readily read from a hydrograph of the streamflow record, but one which is virtually impossible to find with any of the automated optimization approaches which have been suggested for conceptual models. Following a relatively dry period, when there is a significant deficiency in the Lower Zone Tension Water, a storm should be examined which refills a portion of the lower zone tension water but does not come close to filling the Lower Zone Tension Water. Under these conditions, any increase in the base flow must be attributed to water which bypasses the lower zone tension water and flows directly to the lower zone free water aquifers. Since our test run utilized a value of .2 for PFREE, the actual rate of base flow increase compared to the projected rate of base flow increase during the special conditions cited allows a simple

modification of the value of PFREE. Although it may be desirable to adjust the value of PFREE after the size of the various storages are better known and the basic recession characteristics are well identified, the PFREE term will only be significant under the conditions we have described. It is a parameter with a very limited visibility. If the conditions do not exist to "read" PFREE from the observed hydrographs, its effect will be too small to define with optimization.

Of the seventeen parameters we started out with, UZFW, RSERV, UZK, ZPERC, SIDE, and REXP remain to be evaluated. Of these, RESRV, the portion of lower zone free water not subject to tension water transfers, is a rather insensitive component of the model. The initial assumption of .2 will generally function quite well. A change of less than .1 from the starting value will encompass virtually all of the basins I have encountered. This parameter is totally masked in wet areas and only has a real application in extremely dry areas.

UZK, the rate at which interflow is withdrawn from the UZFW storage, can be approximated from an extension of the process previously used for LZSK. However, since the UZFW storage the computed value of UZK (refer to Fig. 15) will be substantially higher than the actual value. The maximum interflow component which can be read or projected from available recession data should be considered as an upper bound of UZK. Due to percolation effects a more realistic value is likely to be about 1/2 the value described by the hydrograph.

UZFWM can be approximated by extending the logic utilized in determining LZFSM. The problem of percolation from UZFWM will, however, cause an underestimate of the size of this storage. As a result, the computed value should be considered as a minimum and generally will need to be increased by a factor of 2. The final value of UZFWM will have a major impact upon surface runoff computations and must necessarily be sized to provide an interflow regime appropriate to the dominant soil types in the watershed.

SIDE, the base flow which takes place outside the stream channel, has a unique impact on the hydrograph. Following a period of extensive wetting, when the base flow aquifers are adequately filled to produce the maximum observed recessions, and the Lower Zone Tension Water has been fully wetted due to its preferential treatment in distributing water, a significant over-forecast of storm runoff volumes may occur. This over-forecast response, if systematically repeated, is a key signal that the minimum percolation rate has been underestimated. If the forecast recession is an adequate dimension, then enlarging SIDE will provide the percolation potential during saturated and near-saturated periods without requiring an excessive flow in the measured portion of base flow.

ZPERC, the term which increases the percolation rate to its maximum value during dry periods, has a feature which makes it relatively simple to establish. ZPERC demonstrates its greatest sensitivity when the basin is relatively dry - a storm which follows an initial wetting of the upper soil is ideal for focusing on establishing ZPERC. Remember, (ZPERC+1)*PBASE is the high point on the percolation curve. By focusing on those storms which occur with a wet upper soil but a relatively dry lower soil, a reasonable value of ZPERC can be evaluated.

REXP defines the rate at which the percolation rate changes from (1+ZPERC)*PBASE under dry conditions to PBASE under wet conditions. Those storms which most clearly demonstrate a problem with REXP are those which occur when the lower zone storages are approaching a half-filled condition. Any systematic error in such storms can be ascribed to REXP. If more water needs to be percolated in order to reduce the volume of immediate storm runoff, then REXP must be decreased. If, however, the simulated storm runoff is inadequate to match the observed, then REXP must be increased.

Following these guides becomes a matter of habit for the trained analyst. However, it should be recognized that the hydrologist performing a simulation is attempting to solve a puzzle which has many parts missing and many other parts of questionable size. He must always be aware of the room for error in his own efforts and in the efforts of those who provided the data he is attempting to evaluate. These problems have led many investigators into stochastic models rather than conceptual models. A difficulty with such an approach is that limitations become buried in the procedure and the opportunities for identifying those areas which can contribute to rational improvements is constrained. Simultaneously, the validity of applying a stochastic model outside its range of experience can be much more hazardous than the similar application of a conceptual model which is based on well-known physical laws.

There is another consideration which should be regularly reviewed when performing a simulation with the Sacramento Watershed Model - the monthly error summaries. The original estimate of the evapotranspiration curve is necessarily based on modifying evaporation data. After a number of simulations, it is appropriate to review the monthly error volumes. An examination of the error in flow values as related to the surface flow forecast, is generally related to one of two error sources.

The first error source is a mis-evaluation of the potential ET for that month. A modification of the presumed ET value may help to alleviate the monthly bias. It should be remembered, however, that the ET curve is a continuous function rising to a maximum in summer and declining to a minimum in winter. No change in the monthly ET which distorts this shape should be used unless there is external evidence supporting an irregular seasonal ET demand. (Such as a seasonal plowing or burning schedule over a major portion of the basin)

A second source of bias in the monthly runoff volume may come from a periodic precipitation bias associated with seasonal wind maximums or minimums. Measured rain is merely an index to true basin rain, and if the index is unstable due to a seasonal measurement problem, a bias pattern will be evident in the simulation. This bias may be associated with a seasonal condition and it may be prevalent in many major storms.

Another feature of the monthly simulation error table may be an extended period of time with a consistent, shifting forecast bias. Such a condition will occur if the representativeness of the rain gage catch has undergone a change during the simulation period.

It has been our experience that those hydrologists who carefully apply the Sacramento Watershed Model gain a rewarding insight into hydrologic processes. Their appreciation of the physical concepts involved and the impacts of problems in hydrologic measurements can make a substantial contribution to their hydrologic knowledge. Nonetheless, once the processing logic for any system is placed in a computer, there are many who will utilize the system as a black box. It is our hope that the effort we have gone to in preparing this material will help to avoid black box applications and will contribute to the intelligent use of a highly descriptive system.

Errata

SACRAMENTO WATERSHED MODEL

by

Robert J. C. Burnash

Page 7, para 3, line 2

should read <u>dominant</u> not dominate

Page 12, para 3, line 4, last word

should read <u>these</u> not this

Page 14, para 2, lines 7 and 9

should read <u>mnemonics</u> not neumonics

Page 24, para 4

should start as:

If evapotranspiration should occur at such a rate that the ratio of tension water contents to tension water capacity is less than that of a similiar ratio for free water, then water is (etc.)

Figure 2

SACRAMENTO MODEL

Base Flow continuing to deplete
Rain re-supplying upper zone tension
Direct runoff occurring

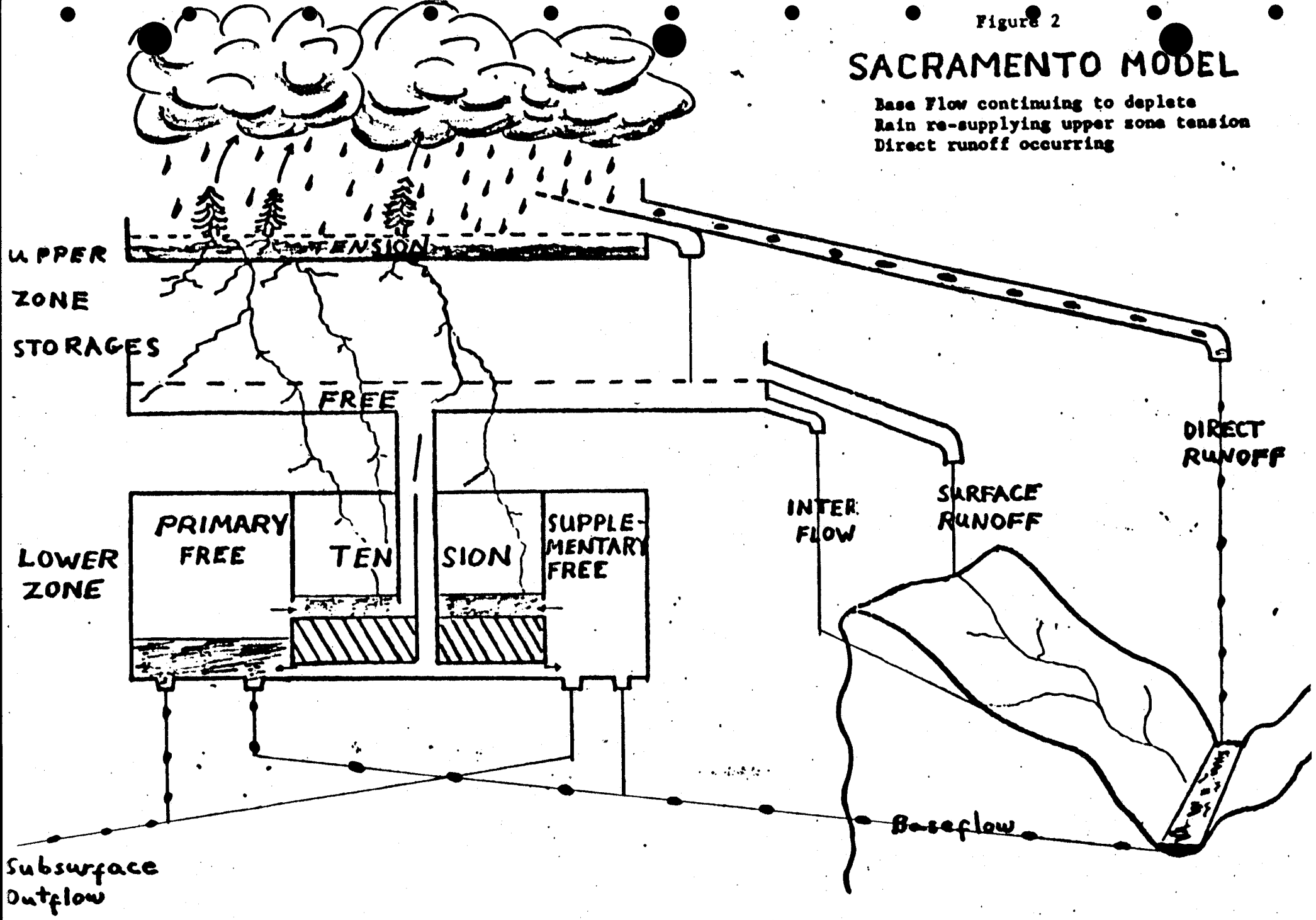
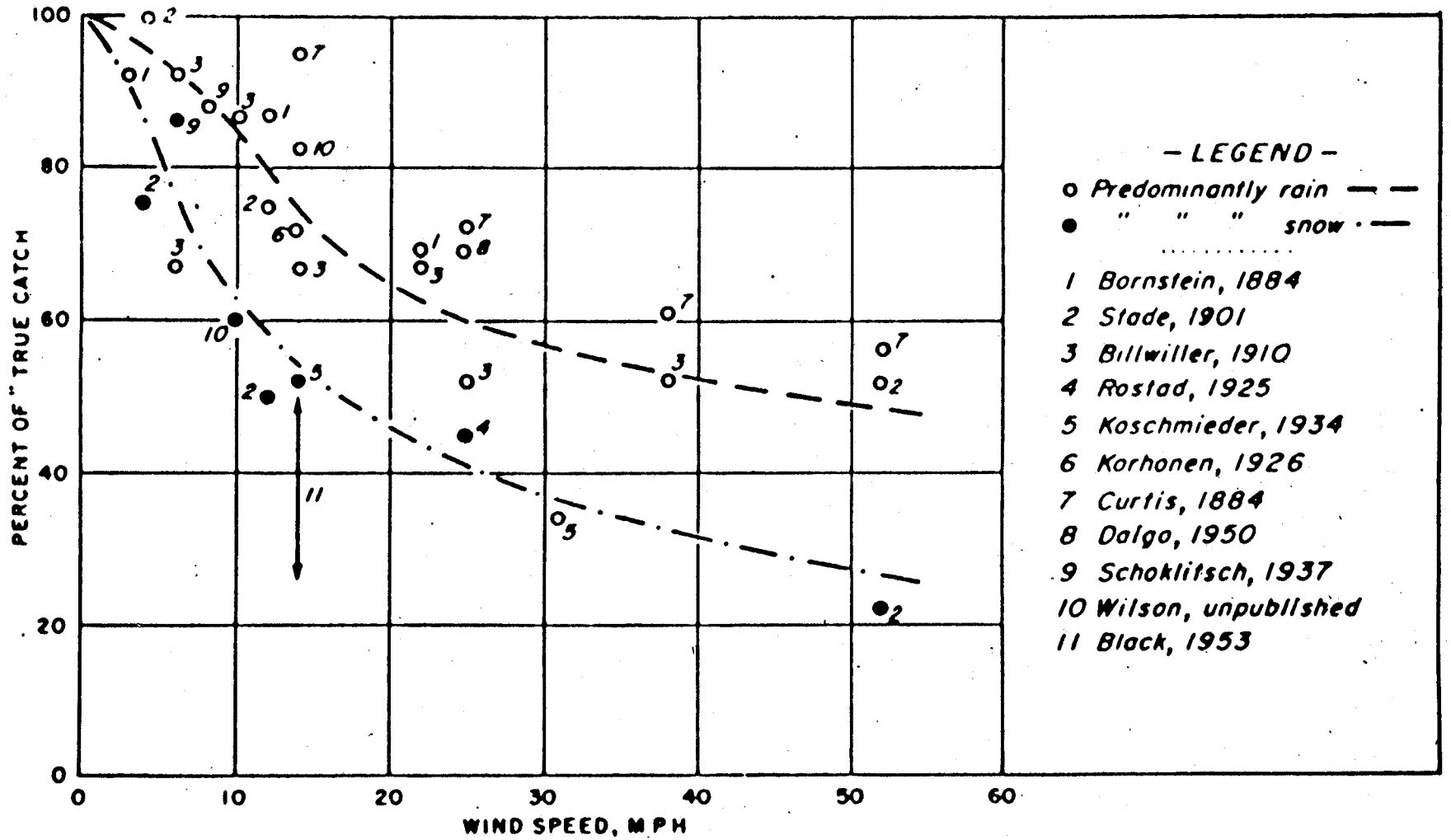
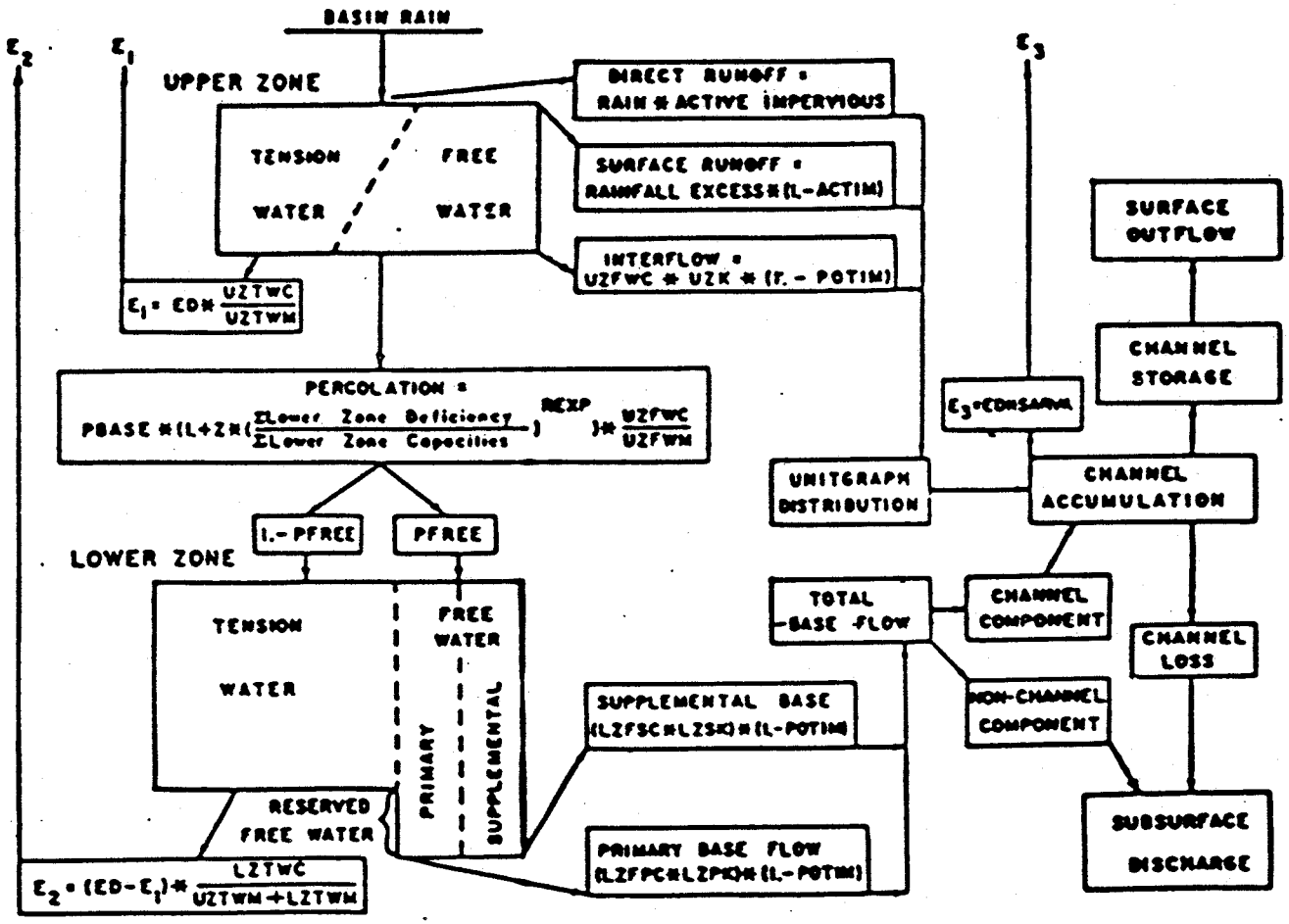


FIG I (AFTER WILSON)



-General relation of precipitation catch to wind speed

FIGURE 2.
Components of a Generalized Hydrologic Model



FLOOD FORECAST

EXTENDED ANALYSIS
WATER SUPPLY

DROUGHT EVALUATION

FORAGE PRODUCTION

RESERVOIR DESIGN

RESERVOIR OPERATION

CHANGES IN BASIN CONDITIONS

EFFECT OF BURNS

LANDSLIDE EVALUATION

COLORADO URBAN HYDROGRAPH PROCEDURE (CUHP)

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Control District
Suite 156-B
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Denver, Colorado 80211
&
Dr. Young Yoon, P.E.
Director, Hydrologic Services
Boyle Engineering Corporation
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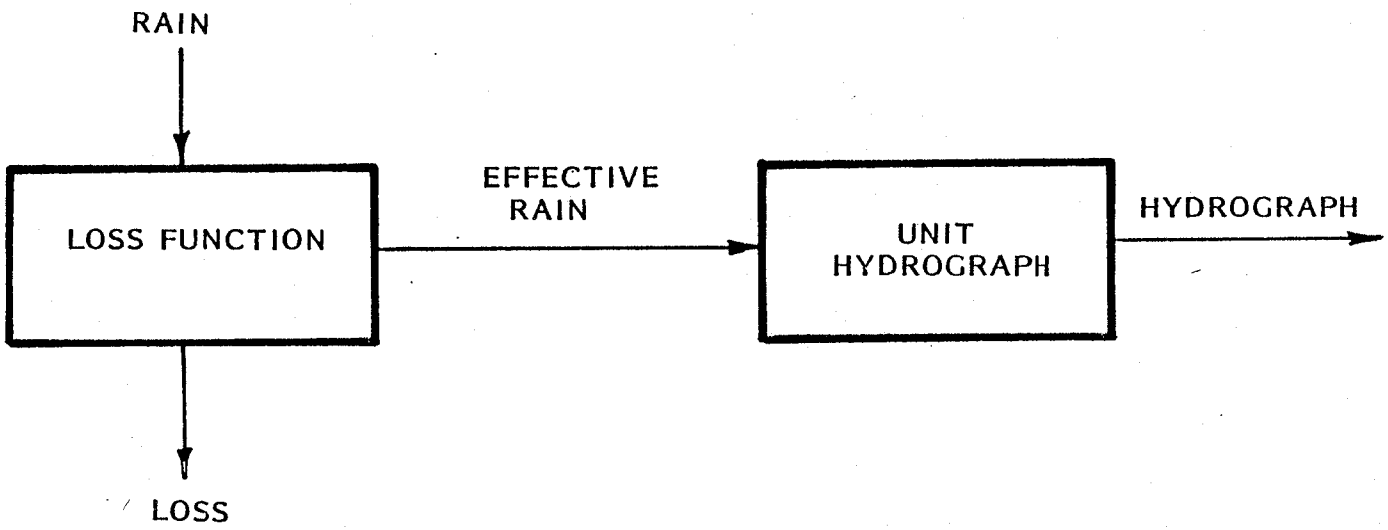
Lakewood, Colorado 80228

CUHP

COLORADO URBAN HYDROGRAPH PROCEDURE

CUHP


RAIN
LOSS FUNCTION
EFFECTIVE RAIN
UNIT HYDROGRAPH
HYDROGRAPH
LOSS

LOSSES

DERESSION LOSS - PERVIOUS AREA
IMPERVIOUS AREA

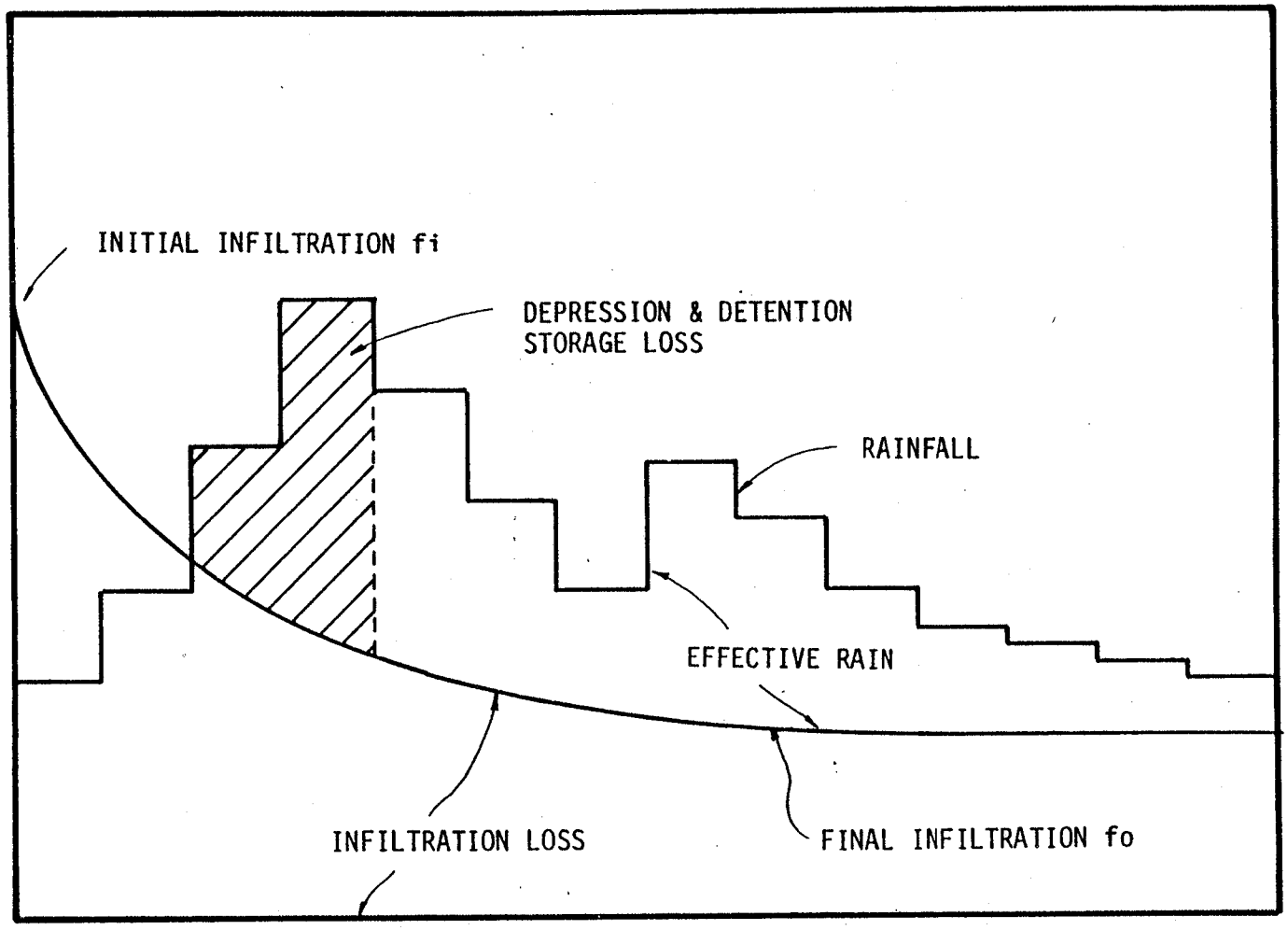
INFILTRATION LOSS - PERVIOUS AREA

HORTON's EQUATION

SCS SOIL CLASSIFICATION

TYPICAL DEPRESSION RETENTION FOR VARIOUS LAND COVERS
(all values in inches)
(For use with CUHP Method)

Land Cover	Depression & Detention	Recommended
Impervious		
Large Paved Areas	0.05 - 0.15	0.1
Roofs - Flat	0.1 - 0.3	0.1
Roofs - Sloped	0.05 - 0.1	0.05
Pervious		
Lawn Grass	0.2 - 0.5	0.35
Wooded Area and Open Fields	0.2 - 0.6	0.4



REPRESENTATION OF HORTON'S EQUATION

a. <u>Soil Infiltration Loss</u>

HORTON'S INFILTRATION EQUATION

$$f = f_o + (f_i - f_o) e^{-at}$$

In which f = infiltration rate (inches/hour)
f_i = initial infiltration rate (inches/hour)
f_o = final infiltration rate (inches/hour)
e = natural logarithm base
a = decay coefficient (1/second)
t = time in seconds

RECOMMENDED HORTON'S EQUATION PARAMETERS

SCS Hydrologic Soil Group	Infiltration (in/hr) Initial - f_i	Final - f_o	Decay Coefficient
A	5.0	1.0	0.0007
B	4.5	0.6	0.0018
C	3.0	0.5	0.0018
D	3.0	0.5	0.0018

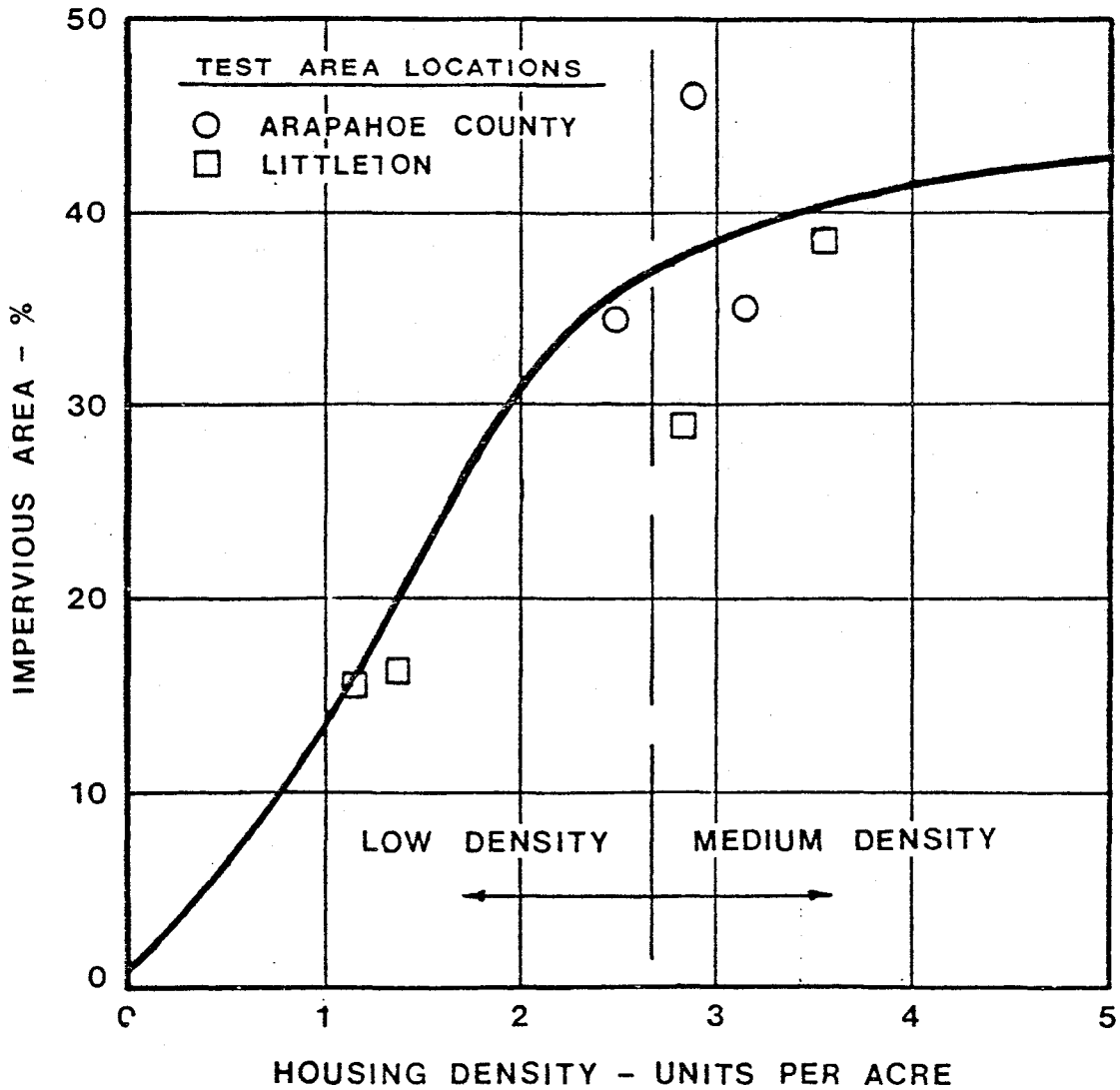
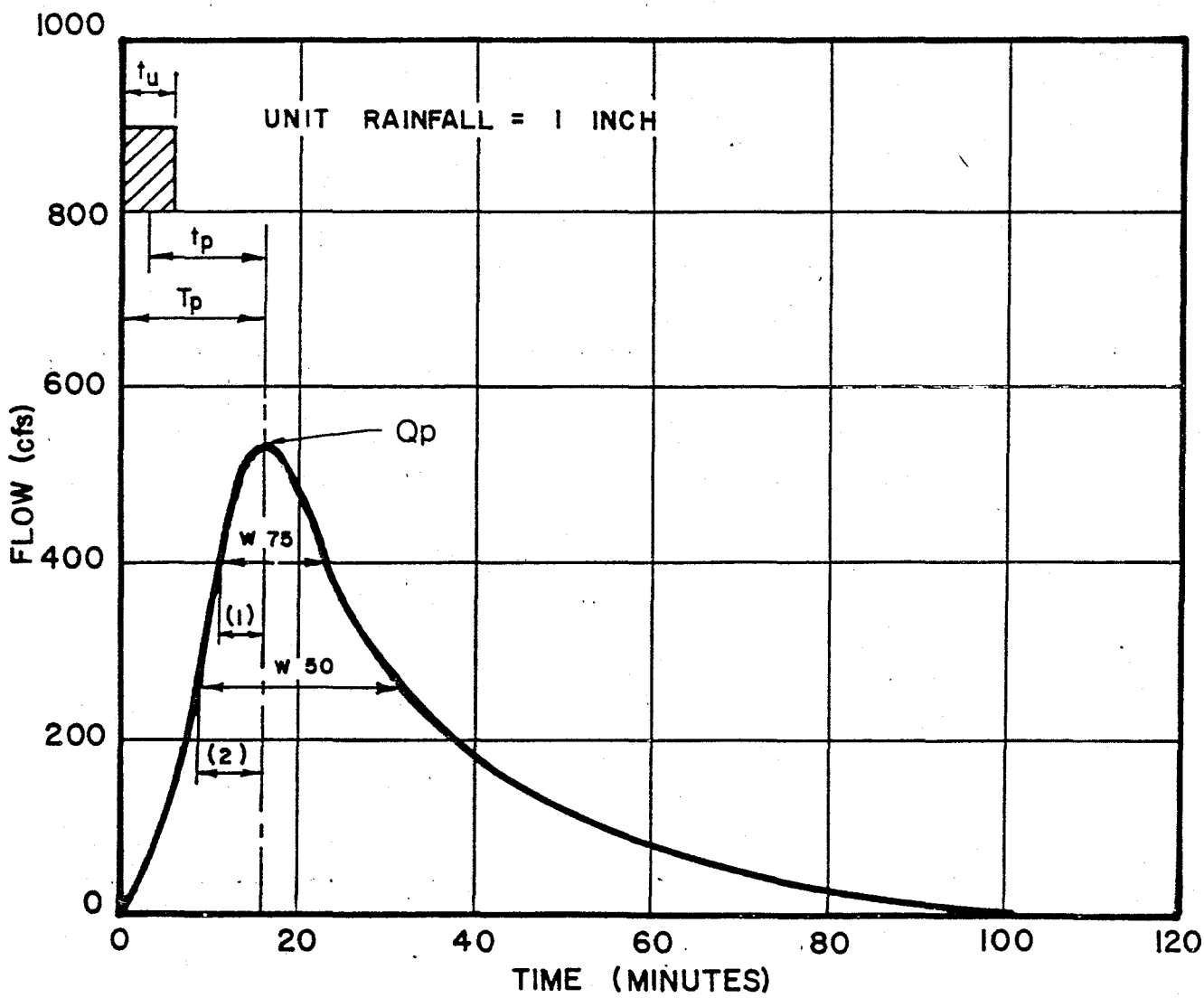
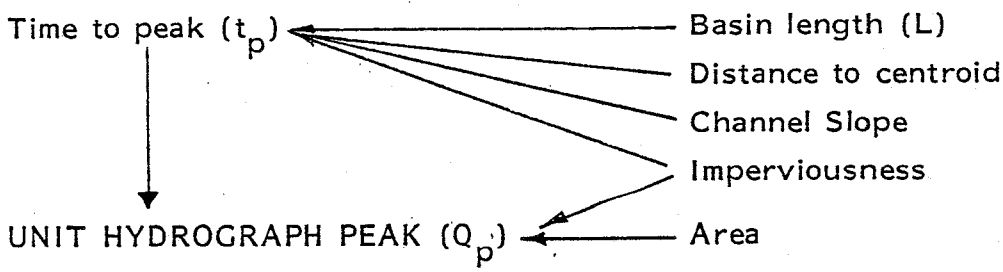


FIGURE 2-1. RESIDENTIAL HOUSING DENSITY
vs.
IMPERVIOUS AREA



UNIT HYDROGRAPH

RELATIONSHIP BETWEEN
UNIT HYDROGRAPH AND BASIN PARAMETERS



BASIC UNIT HYDROGRAPH EQUATIONS

$$t_p = C_t \left[(L\, L_{ca}/\sqrt{S})\right]^{0.48} \qquad T_p = 60t_p + 0.5t_u$$

In which t_p = time to peak of the unit hydrograph from midpoint of unit rainfall in hours.

L = length along stream from study point to upstream limits of the basin in miles

L_{ca} = length along stream from study point to a point along stream adjacent to the centroid of the basin in miles.

S = weighted average slope of basin along the stream to upstream limits of the basin in feet per foot.

C_t = coefficient reflecting time to peak.

T_p = time from beginning of unit rainfall to peak of hydrograph in minutes.

t_u = time of unit rainfall duration in minutes.

$$q_p = \frac{640\ C_p}{t_p} \qquad Q_p = q_p\ A$$

In Which q_p = peak rate of runoff in cfs per square mile.

C_p = coefficient related to peak rate of runoff.

Q_p = peak of the unit hydrograph in cfs.

A = area of basin is square miles.

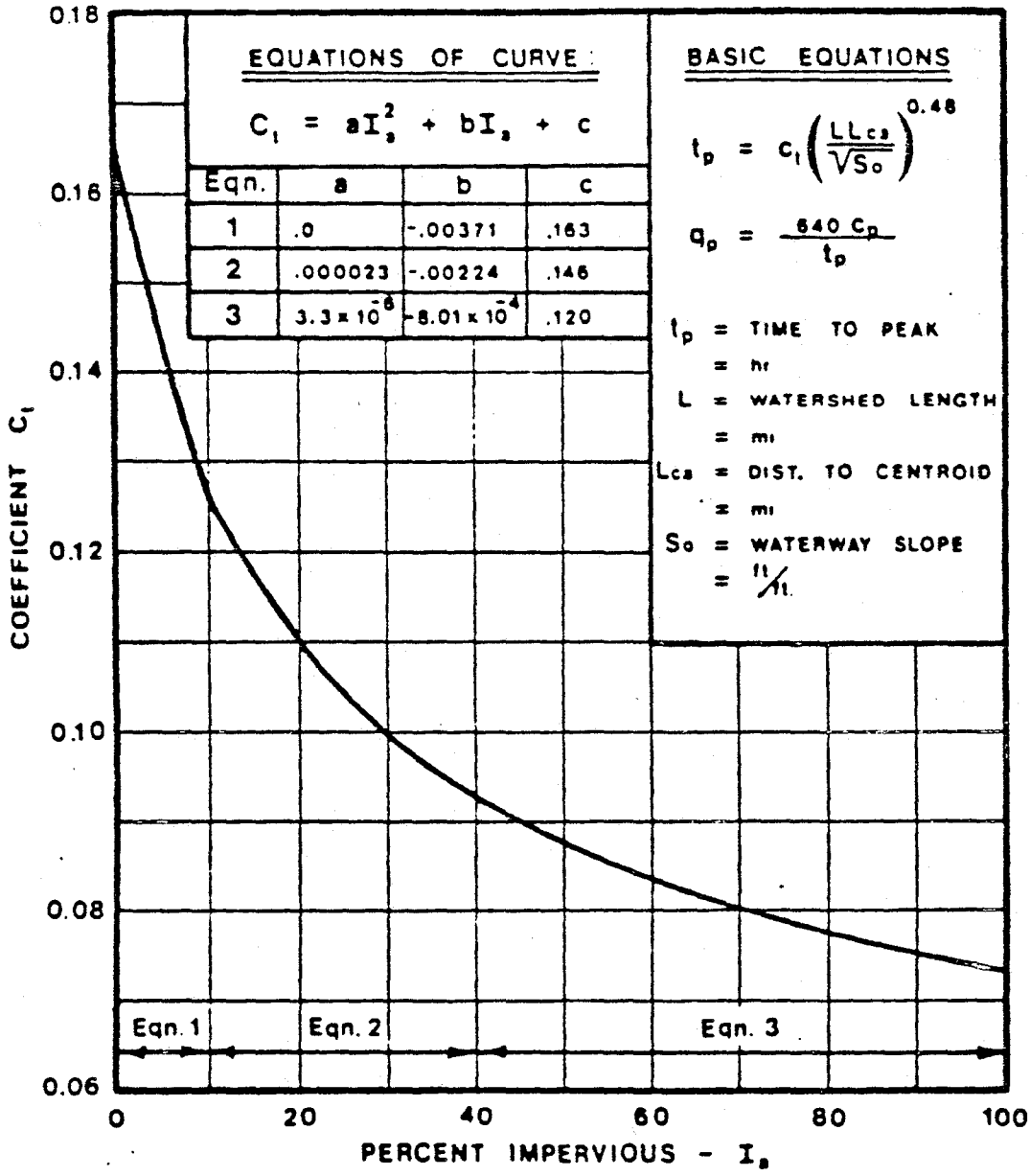


FIGURE 4-2. RELATIONSHIP BETWEEN C_t & IMPERVIOUSNESS

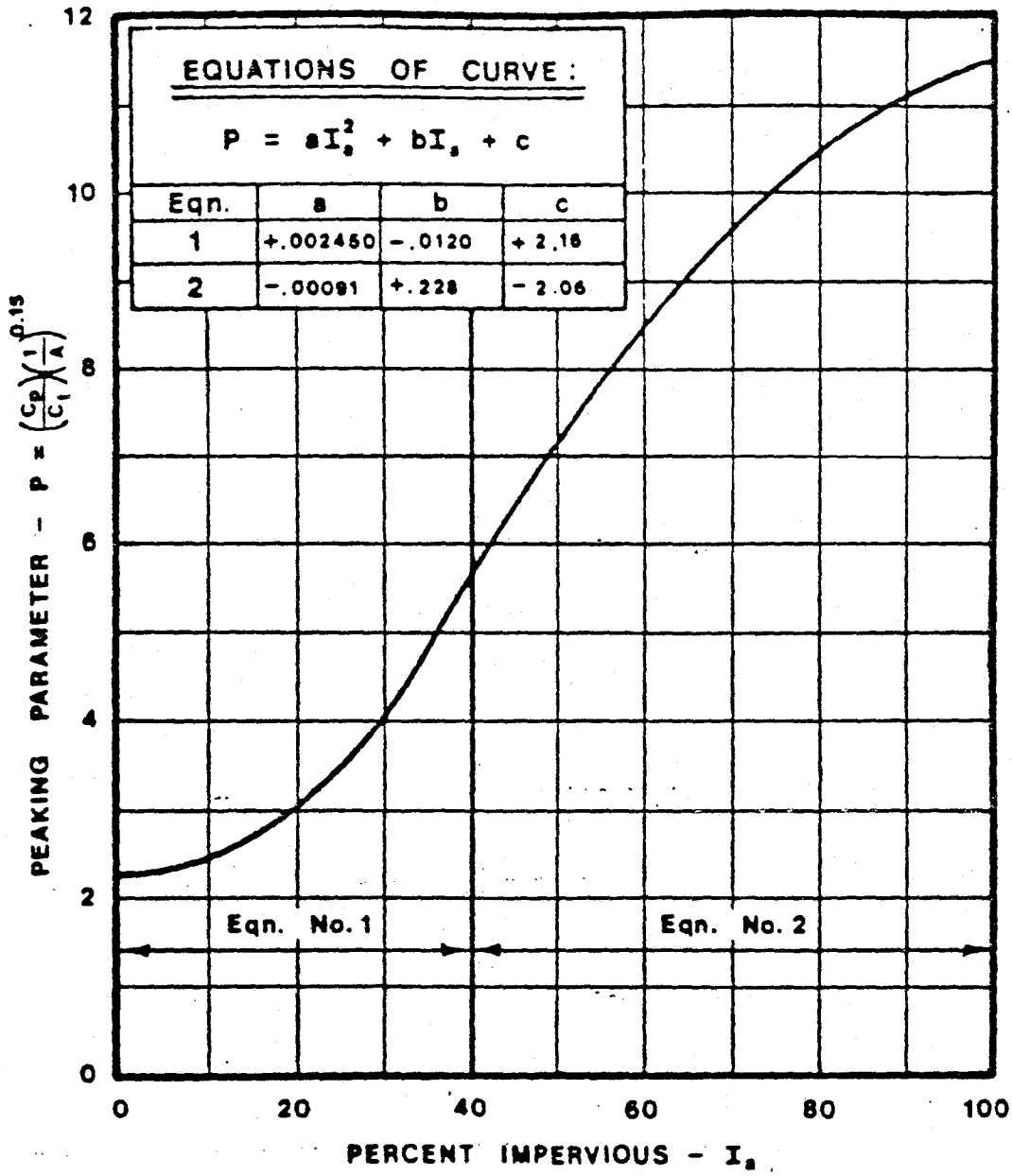
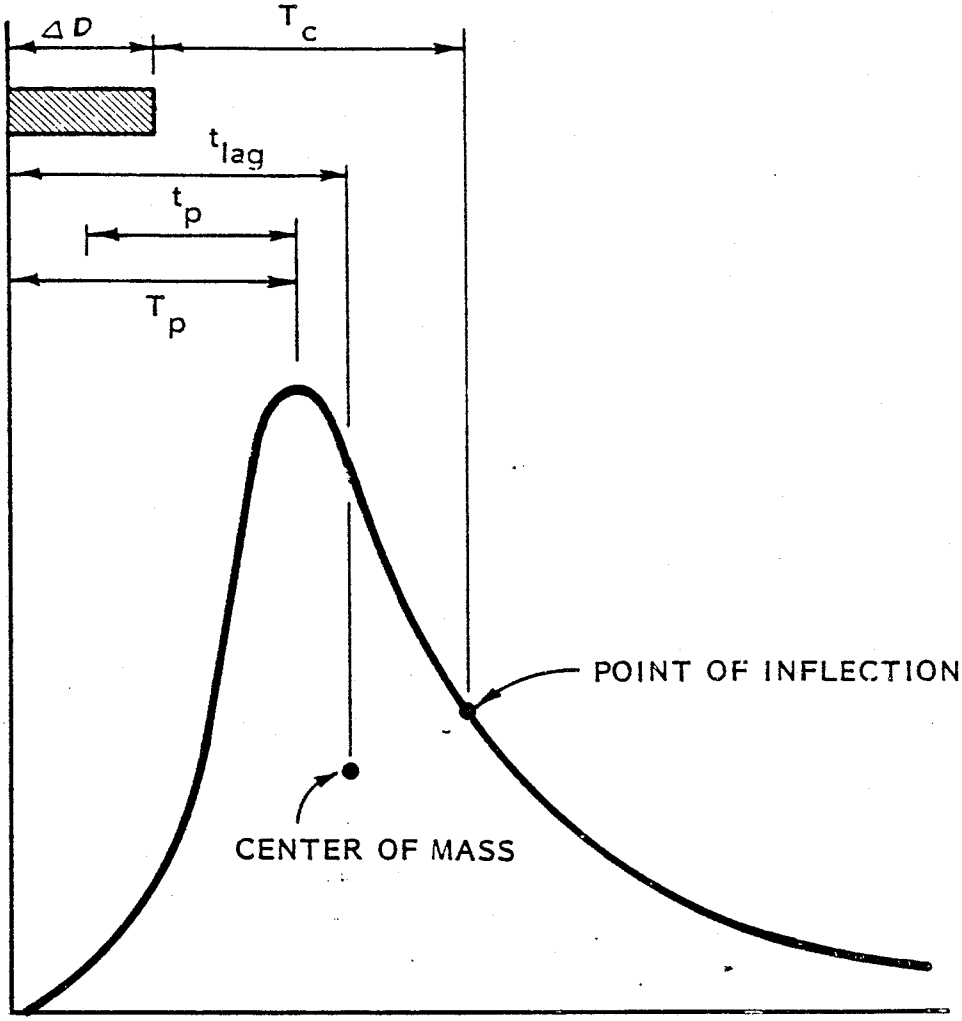


FIGURE 4-3. RELATIONSHIP BETWEEN PEAKING PARAMETER AND IMPERVIOUSNESS

UNIT HYDROGRAPH



	TIME VARIABLE
SCS	$t_p = f(L, S, CN)$
SNYDER's	$t_p = C_t (LL_{ca})^{0.3}$
CUHP	$t_p = C_t \left(\frac{LL_{ca}}{\sqrt{S}}\right)^{0.48}$
COE - LA DISTRICT	$t_{lag} = 24.n\left(\frac{LL_{ca}}{\sqrt{S}}\right)^{0.38}$

WEIGHTED SLOPE

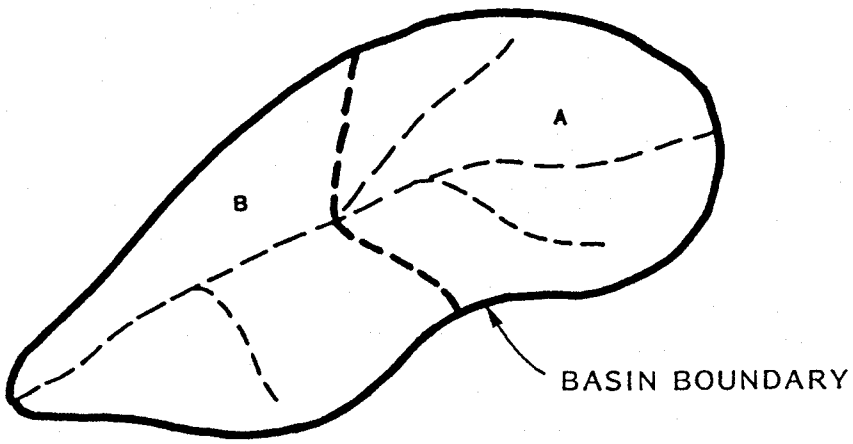
$$S = \left[\frac{L_1 S_1^{0.24} + L_2 S_2^{0.24} + \ldots + L_n S_n^{0.24}}{L_1 + L_2 + L_3 + \ldots L_n} \right]^{4.17}$$

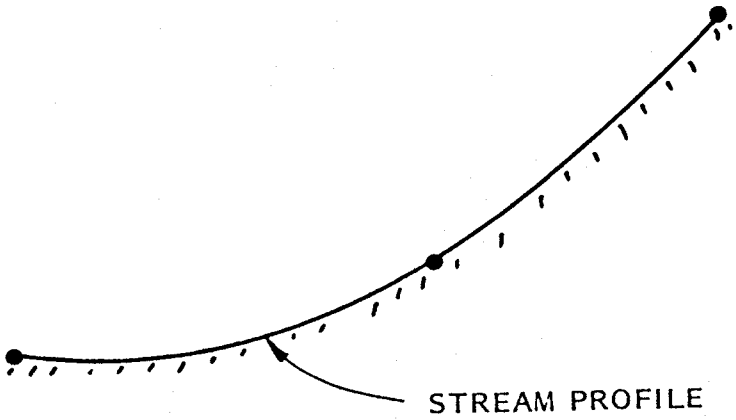
In Which, S = weighted basin waterway slope in ft/ft

$S_1, S_2, \ldots S_n$ = slopes of individual reaches in ft/ft (after adjustments using Figure 4-1)

$L_1, L_2, \ldots L_n$ = lengths of corresponding reaches.

SLOPE





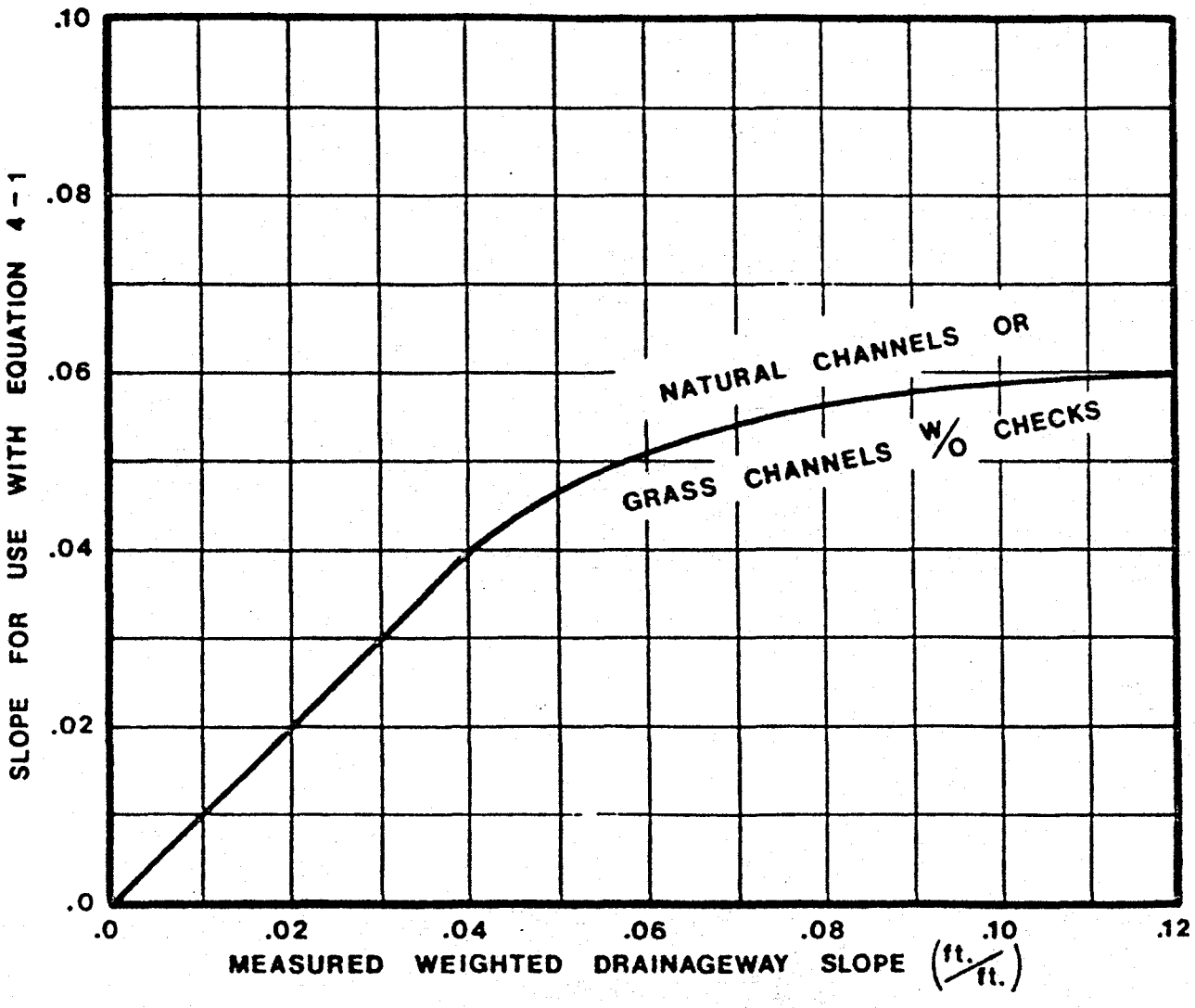
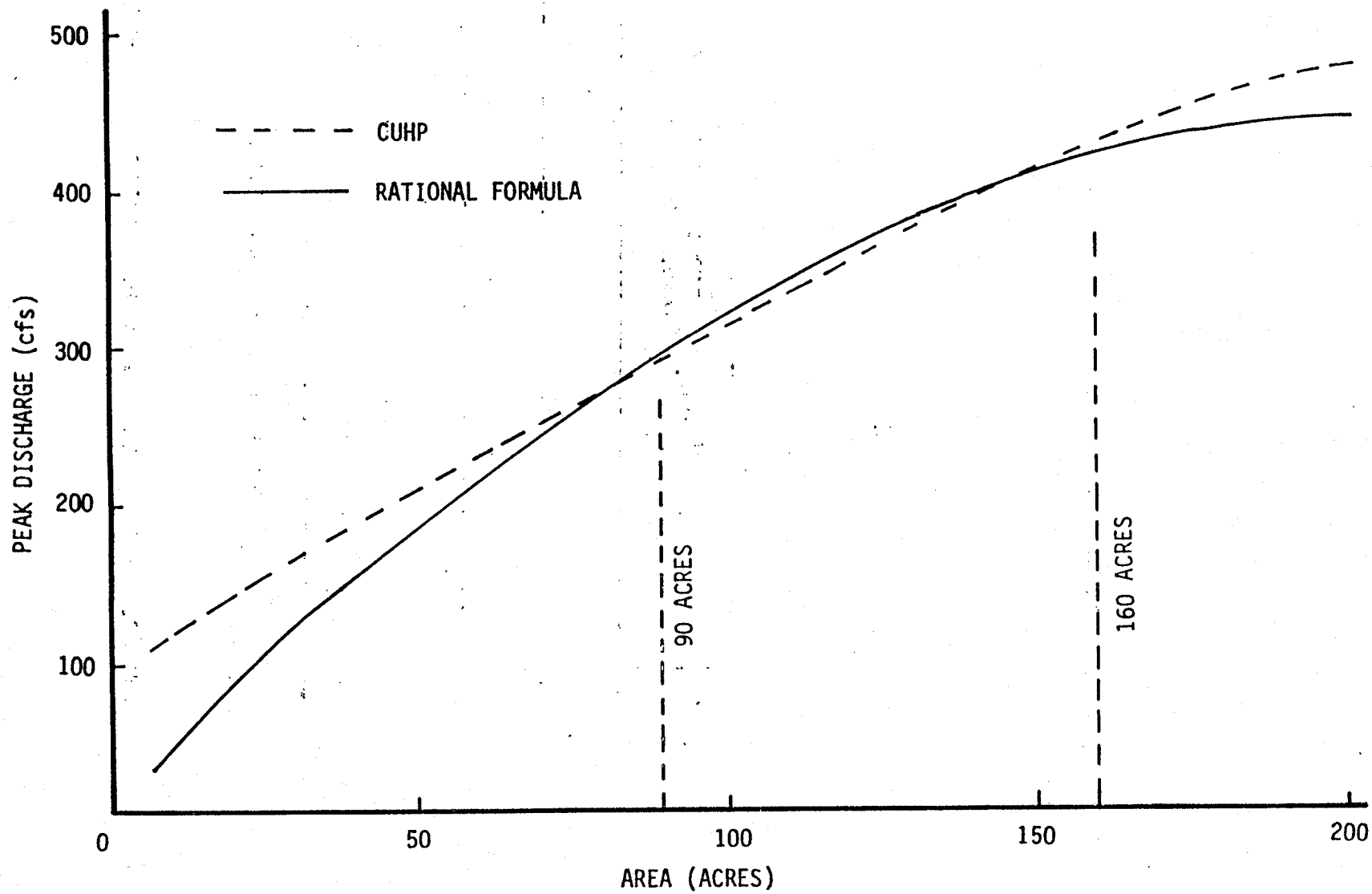


FIGURE 4 - 1. SLOPE CORRECTION FOR NATURAL DRAINAGEWAYS

COMPARISON OF CUHP AND RATIONAL FORMULA RESULTS



MODIFICATION OF THE CUHP PROCEDURES FOR SMALL AREAS LESS THAN 90 ACRES

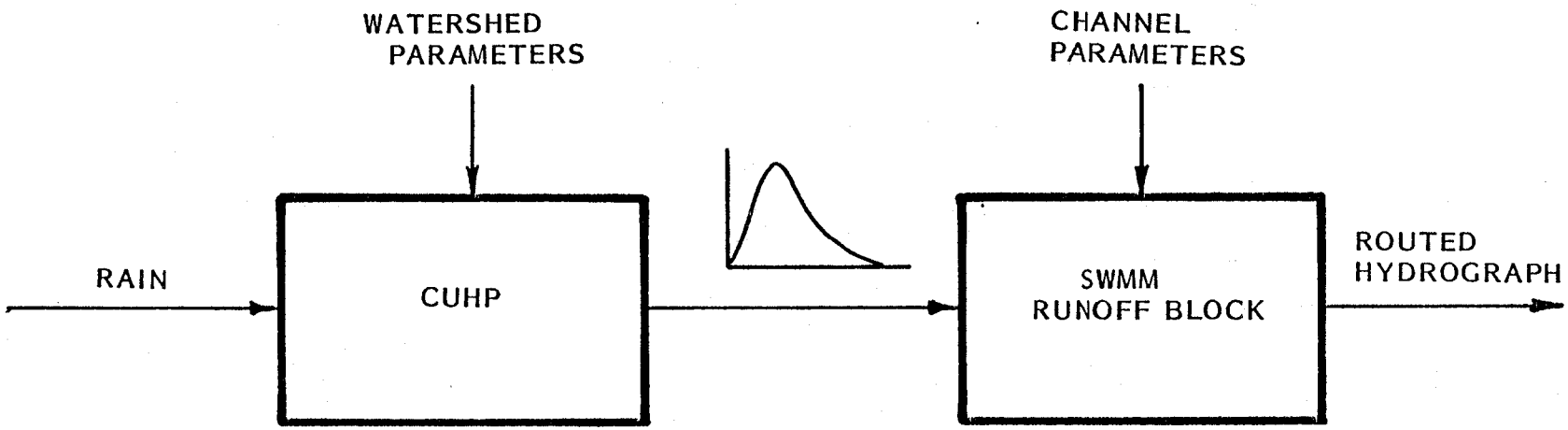
- The time to peak (t_p) estimated by the CUHP procedure may not be accurate.

- The time to peak (t_p) may be estimated by the following formula using the time of concentration as estimated by the rational formula method.

$$t_p = 0.39\left(\frac{F_i^2-0.36F_i+0.07}{F_i^2-0.49F_i+0.14}\right)(T_c-6F_i)$$

In which, t_p is the time to peak in minutes, T_c is the time of concentration in minutes, and F_i is the impervious area, as a fraction.

A COMBINATION
OF
CUHP and SWMM



MODIFIED RUNOFF BLOCK (UDSWM2)

BACKGROUND

- Original Version
  - Version II Runoff block by University of Florida

- Modifications
  - Battelle Institute
  - Corps of Engineers, Hydrologic Engineering Center
  - Corps of Engineers, Missouri River Division
  - Boyle Engineering Corporation

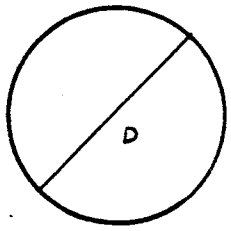
Program Structure

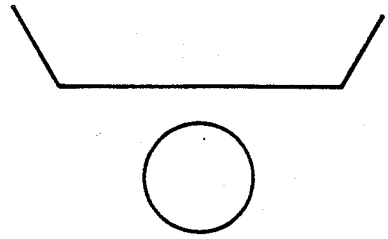
- Simulates Quantity Runoff Only

- Can Be Linked With The CUHP

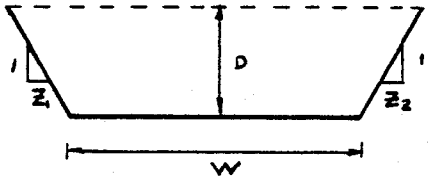
CONVEYANCE ELEMENT

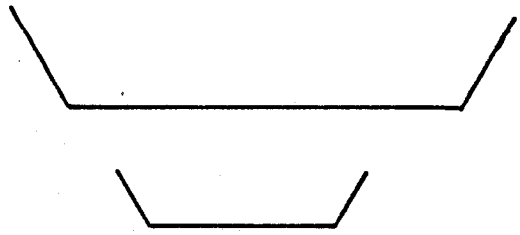
1) CIRCULAR PIPE



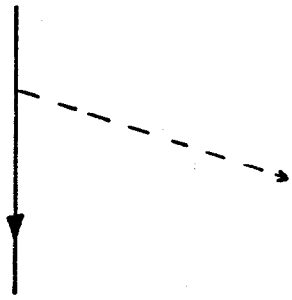


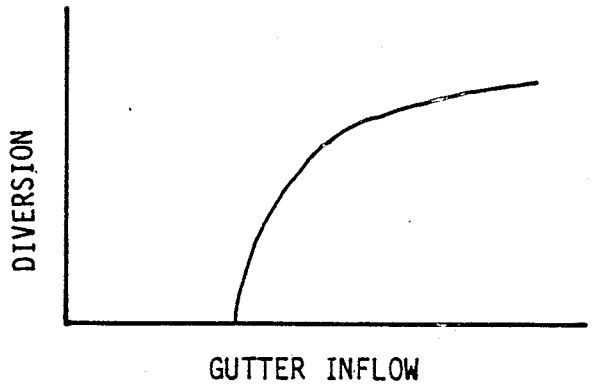
2) TRAP



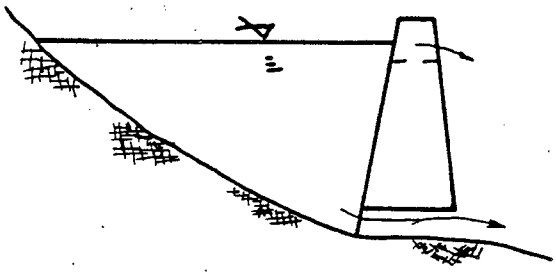


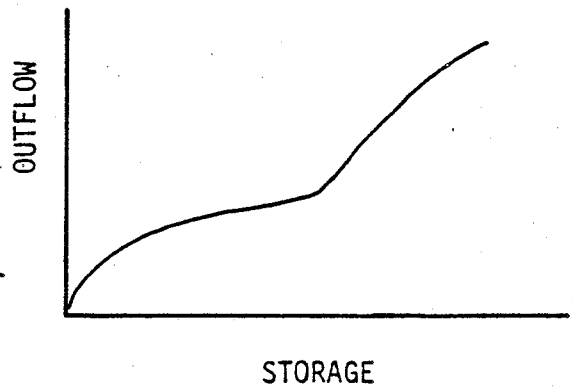
3) DIVERSIONS

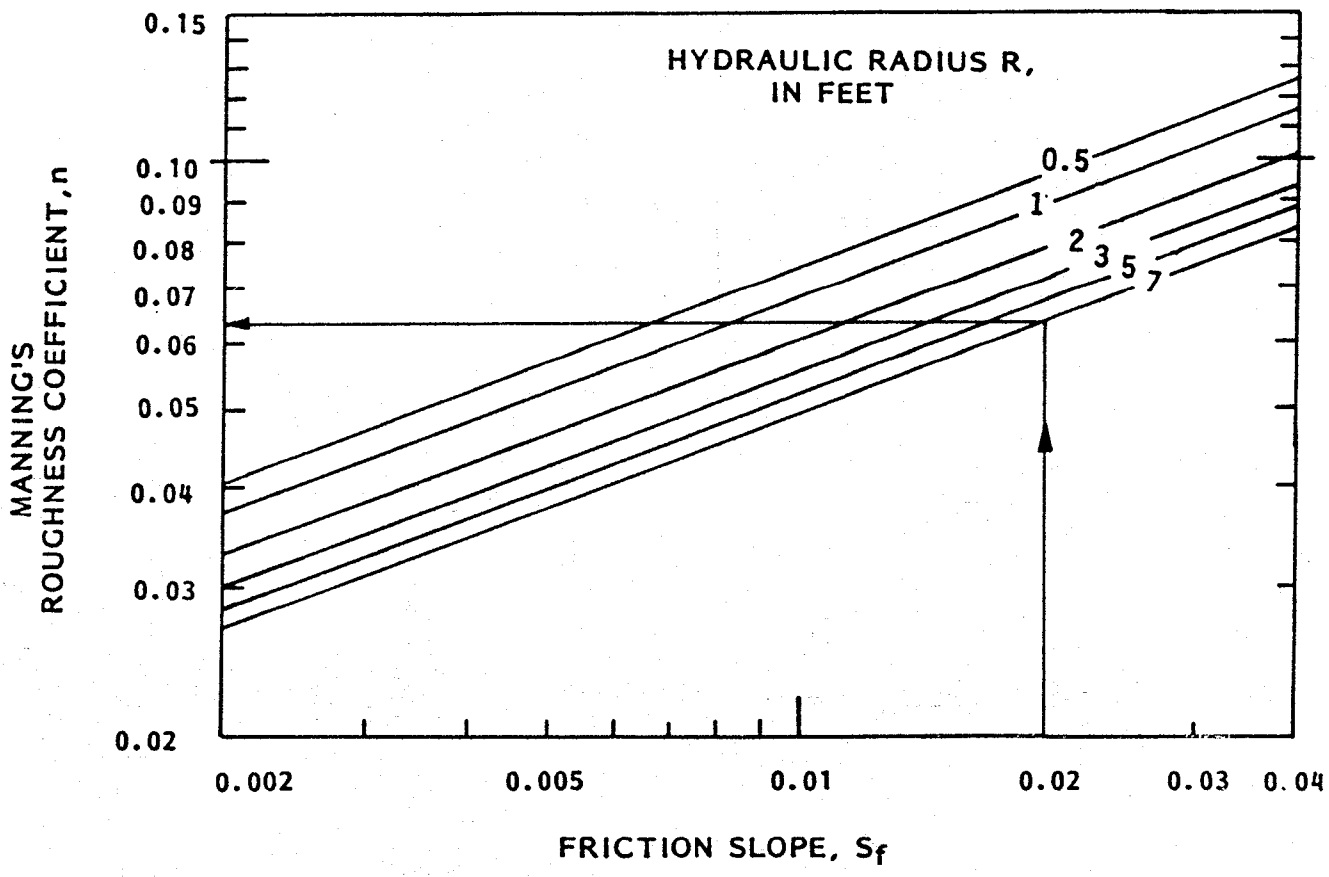




4) DETENTION BASIN







$$n=0.39S_f^{0.38}R^{-0.16}$$

RELATION BETWEEN MANNING'S ROUGHNESS COEFFICIENT, FRICTION SLOPE AND HYDRAULIC RADIUS

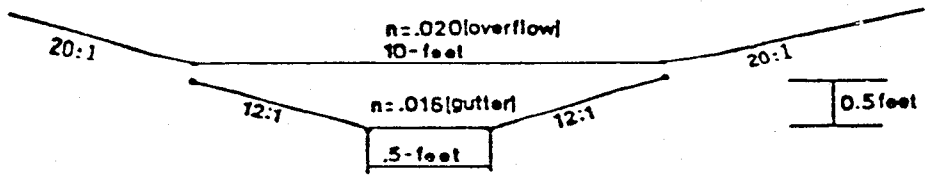
REF. DRAFT U.S.G.S. REPORT BY ROBERT O. JARRETT "DETERMINATION OF ROUGHNESS COEFFICIENTS FOR STREAMS IN COLORADO." DRAFT DATE JULY 1984

ROUGHNESS COEFFICIENT

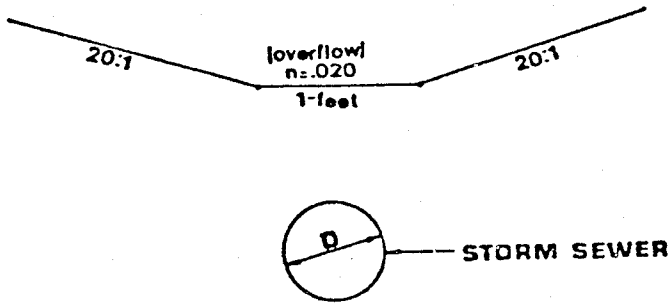
	Hydrologic Routing		Hydraulic Calculation
	n	←	1.25 x n
GUTTER or PIPE	0.016		0.013

Figure 2

SUGGESTED TRAPEZOIDAL APPROXIMATIONS FOR STREETS

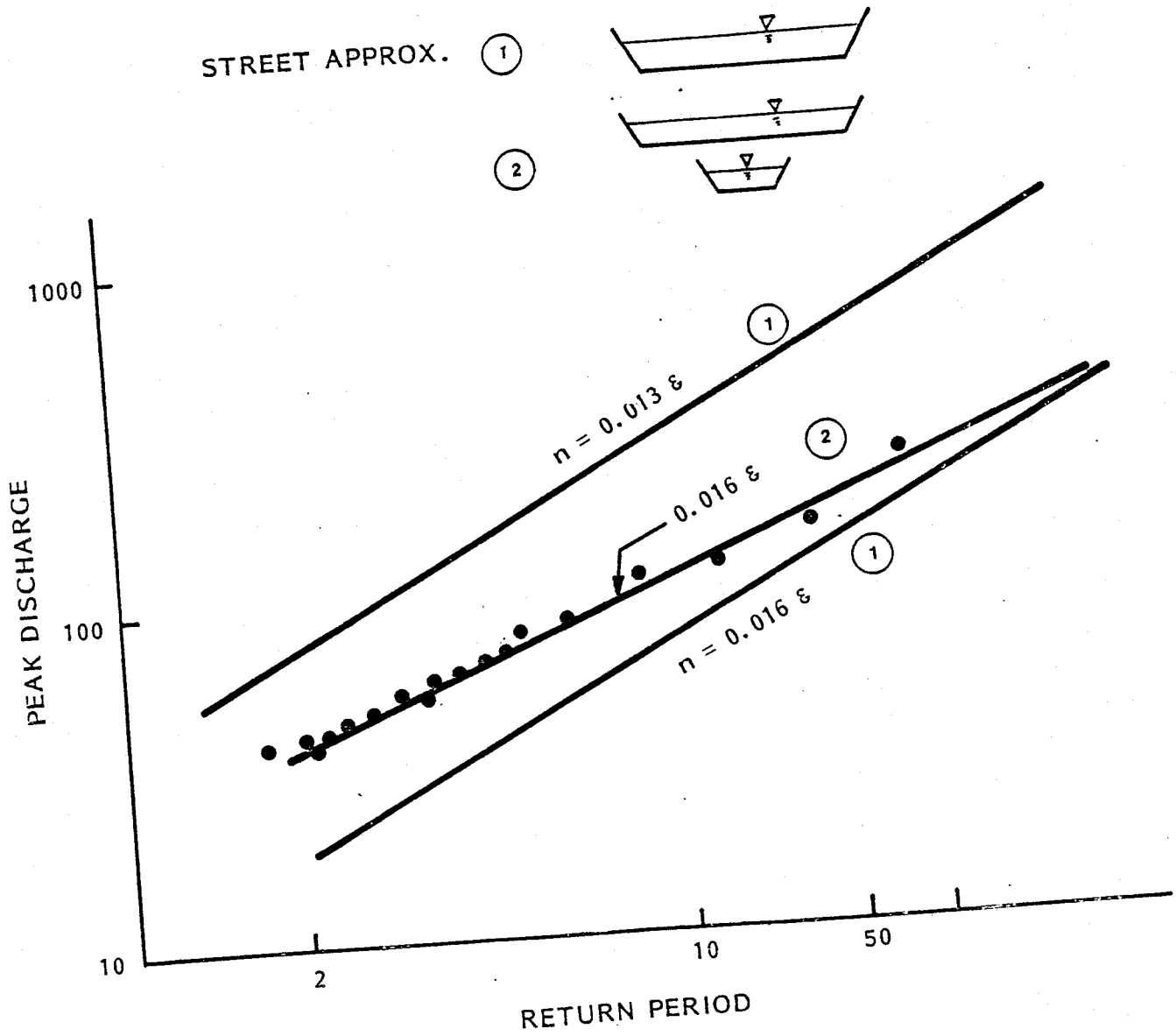


FULL STREET (TWO GUTTER) W/O STORM SEWER
(FOR ONE GUTTER USE ONE-HALF OF SECTION)



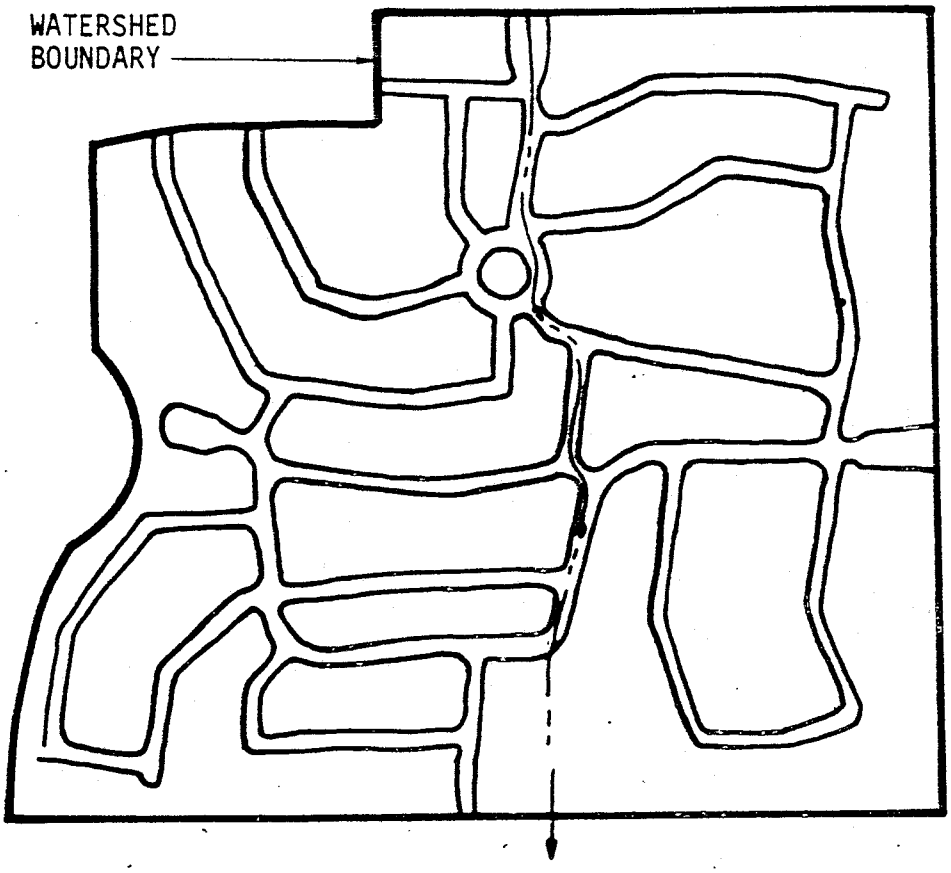
FULL STREET WITH A STORM SEWER

COMPARISON OF
MEASURED FLOW & CUHP RESULTS



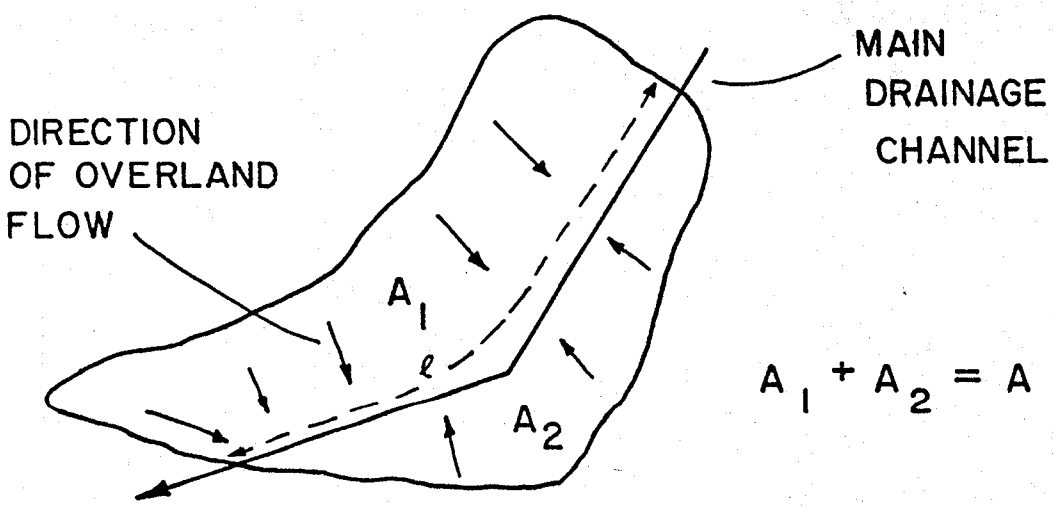
COMPARISON OF SWMM & CUHP WATERSHED PARAMETERS

EXAMPLE 1



	CUHP	SWMM
Drainage Area	39 Ac	39 Ac
Basin Length or Width	1370 Ft.(L)	2740 Ft. (W)
Distance to Centroid	740 Ft.	-
Slope	0.039	0.02
Total Imperviousness	70 %	70 %
Imperviousness w/o Depression	-	10 %
Manning's n		
Pervious area	-	0.25
Impervious area	-	0.016
Depression Loss		
Pervious area	0.35	0.35 inches
Impervious area	0.05	0.06 inches
Infiltration Coefficient		
Maximum (f_i)	4.5 in/hr	4.5 in/hr
Minimum (f_o)	0.6 in/hr	0.6 in/hr
Decay Rate (a)	0.0018	0.0018

BASIN WIDTH

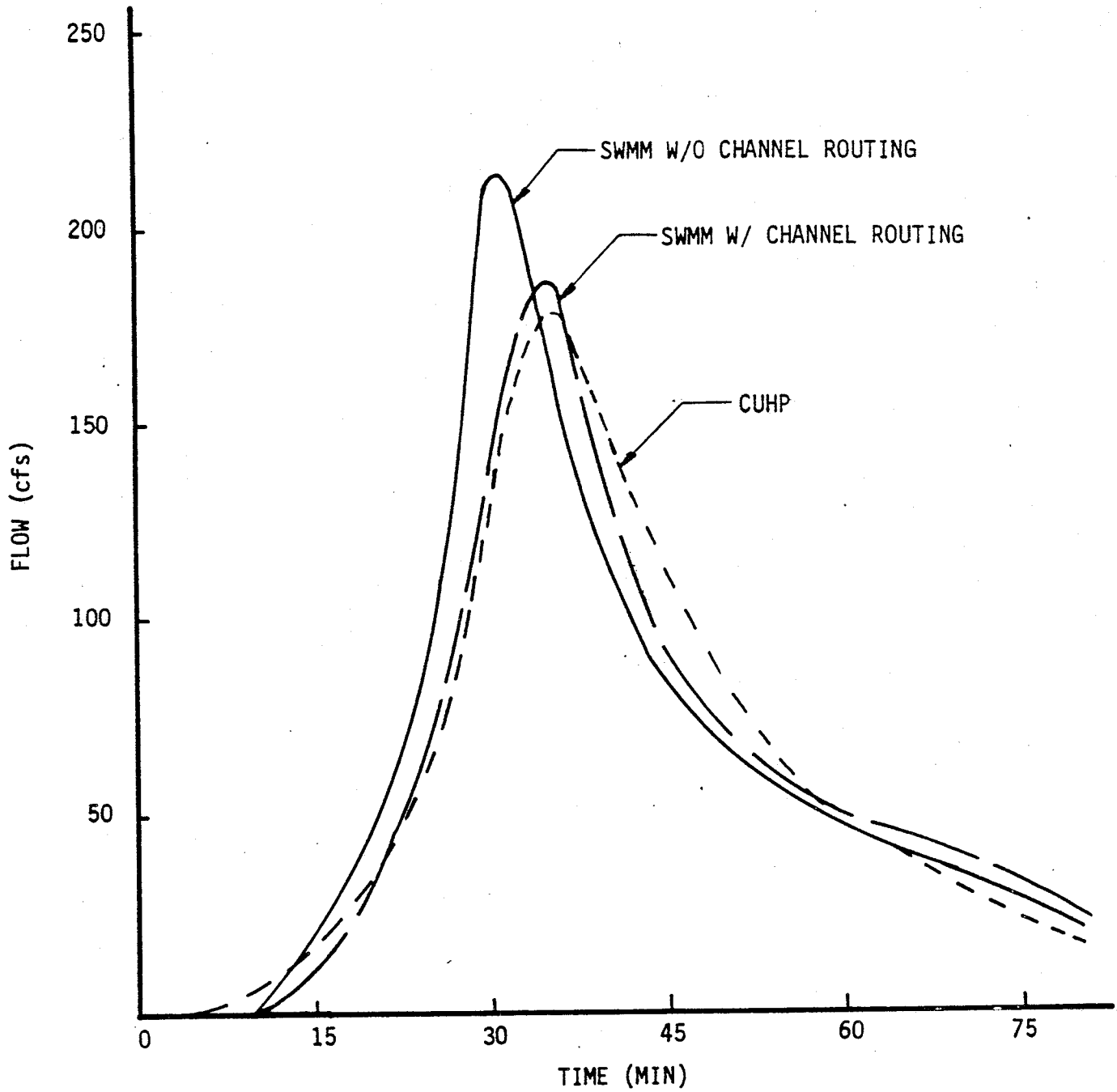


$$A_1 + A_2 = A$$

$$W = (2 - \gamma)\ell \qquad \gamma = \frac{A_2 - A_1}{A}$$

Where W = subcatchment width, ft, and
ℓ = length of main drainage channel, ft.
γ = skew factor, $0 \leq \gamma \leq 1.0$,
A_1 = area to one side of channel, ft.^{2},
A_2 = area to other side of channel, ft.^{2}, and
A = total area, ft.^{2}.

COMPARISON OF CUHP AND SWMM RESULTS
EXAMPLE NO. 1



NOMOGRAPHS TO APPROXIMATE BIG DISCHARGES

June, 1985

BOYLE ENGINEERING CORPORATION

The attached computation worksheet and nomographs are intended to provide a guick and easy peak flow computation based on the methods of the Colorado Urban Hydrograph Procedure. The procedure to make use of the nomographs are as follows:

1) Estimate the following parameters;

   A, area of the basin in square miles

   - soil type (A,B,C, or D)

   I_a, imperviousness in percent

   L, length along the stream from study point to upstream limits of the basin in miles

   L_{ca}, length along the stream from study point to a point along stream adjacent to the centroid of the basin in miles

   S, weighted average slope of the basin along the stream to upstream limits of the basin in feet per foot

   I, 1-hour rainfall in inches

2) Estimate 2-hour effective rainfall (R_e) using the effective rainfall chart with given imperviousness, soil type, and 1-hour rainfall

3) Compute a value for $LL_{ca}/\sqrt{S}$

4) Enter the peak flow chart with A, $LL_{ca}/\sqrt{S}$, I_a to determine a coefficient, C.

5) The peak flow, Q, will then be

$$Q = C * R_e \times A$$

Where Q = peak flow rate in cfs

C = coefficient determined using the peak flow chart

R_e = 2-hour effective rainfall as determined by the effective rainfall chart

A = area in acres

The limitations and accuracy involved in using the nomographs are as follows:

1) The basin area should be greater than 100 acres and less than 4000 acres, having homogeneous characteristics and a regular shape.

2) In developing the effective rainfall chart, the depression losses are assumed to be 0.3 inches for pervious areas and 0.05 inches for impervious areas.

3) The error in the peak flow estimation increases as the value of "C" determined by the peak flow chart increases. The expected error ranges for infrequent storm events greater than a 10-year return period are

   if $C < 1$, $\pm 5\%$

   if $C \geq 1$, $\pm 10\%$

   If a storm event is less than a 10-year return period, a more error may be expected

<u>It is cautioned that the nomographs are only used for checking the results produced by the CUHP computer model or in the case of needs for the very rough estimation. The nomographs should never be used for reporting, planning,or design purposes.</u>

PEAK FLOW COMPUTATION WORKSHEET

(For checking purpose only - NOT to be used for design)

Basin Name _____

Geographic location of center of basin: Long. _____ Lat. _____

Basin Area _____ square miles. x 640 = acres

Land Use _____ Impervious Area (I_a) _____%

Watershed Parameters

Basin length	L	=	_____ mi
Distance to Centroid	L_{ca}	=	_____ mi
Drainageway slope	S	=	_____ ft/ft

$$\frac{LL_{ca}}{\sqrt{S}} = _____$$

Storm Frequency _____ Year, 1-Hr Rain _____ inches

Estimate effective rain for each soil type

Soil Type	Percent		Effective Rain*		
A	_____%	x	[ ]	/100 =	_____
B	_____%	x	[ ]	/100 =	_____
C & D	_____%	x	[ ]	/100 =	_____

average effective rain (Re) _____ inches

* To determine effective rainfall, enter effective rainfall charts with "Ia" and "1-hour rainfall".

Runoff volume = Re (inches) x A (acres) /12

= _____ x _____ /12

= _____ ac.ft.

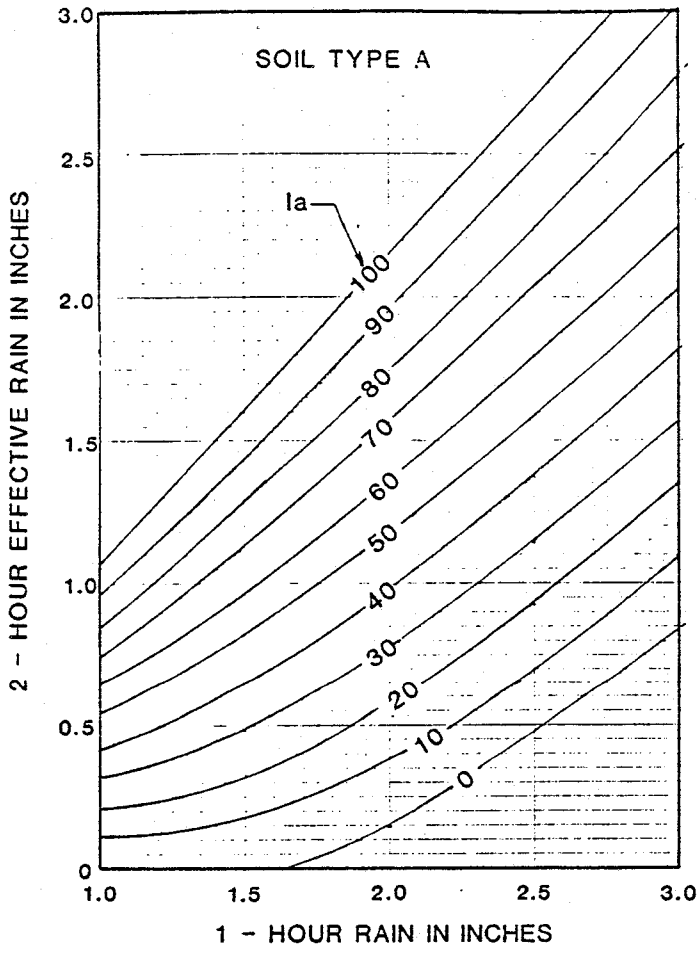
Peak Flow = C* x Re (inches) x A (acres)

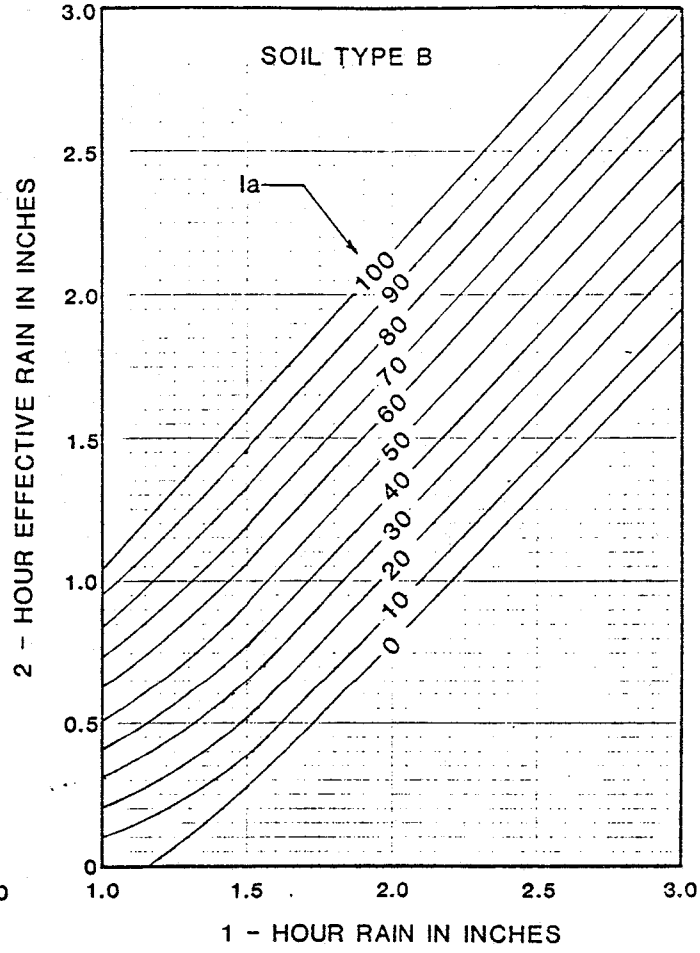
= [ ] x _____ x _____

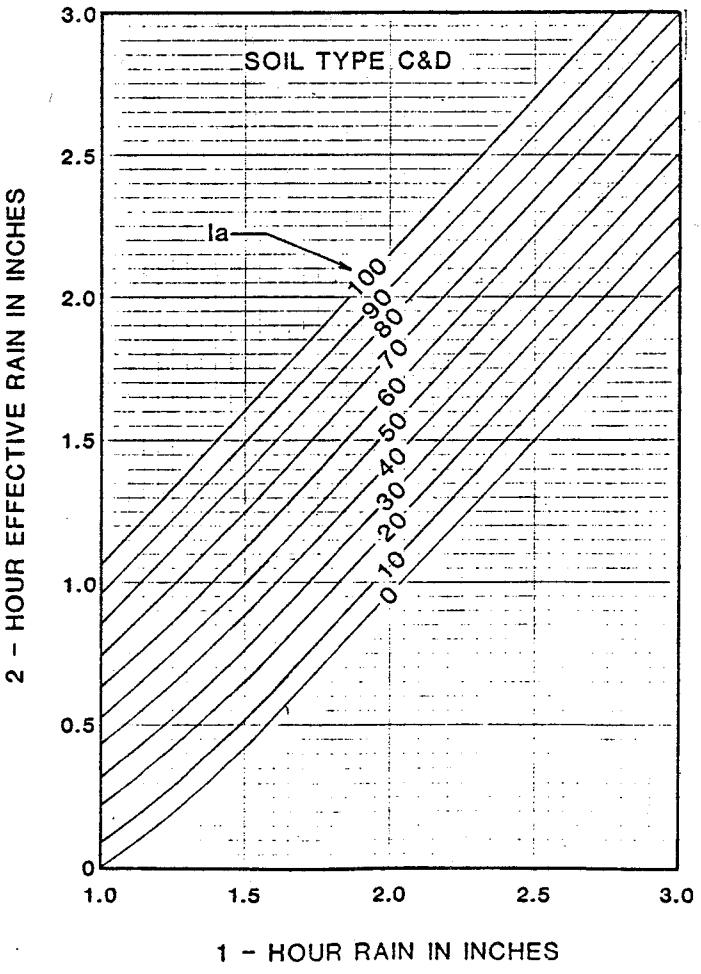
= _____ cfs

* To determine C, enter peak flow chart with "A", "$LL_{ca}/\sqrt{S}$", "Ia".

Summary: Frequency _____ Year, Peak flow _____ cfs, Vol _____ ac.ft







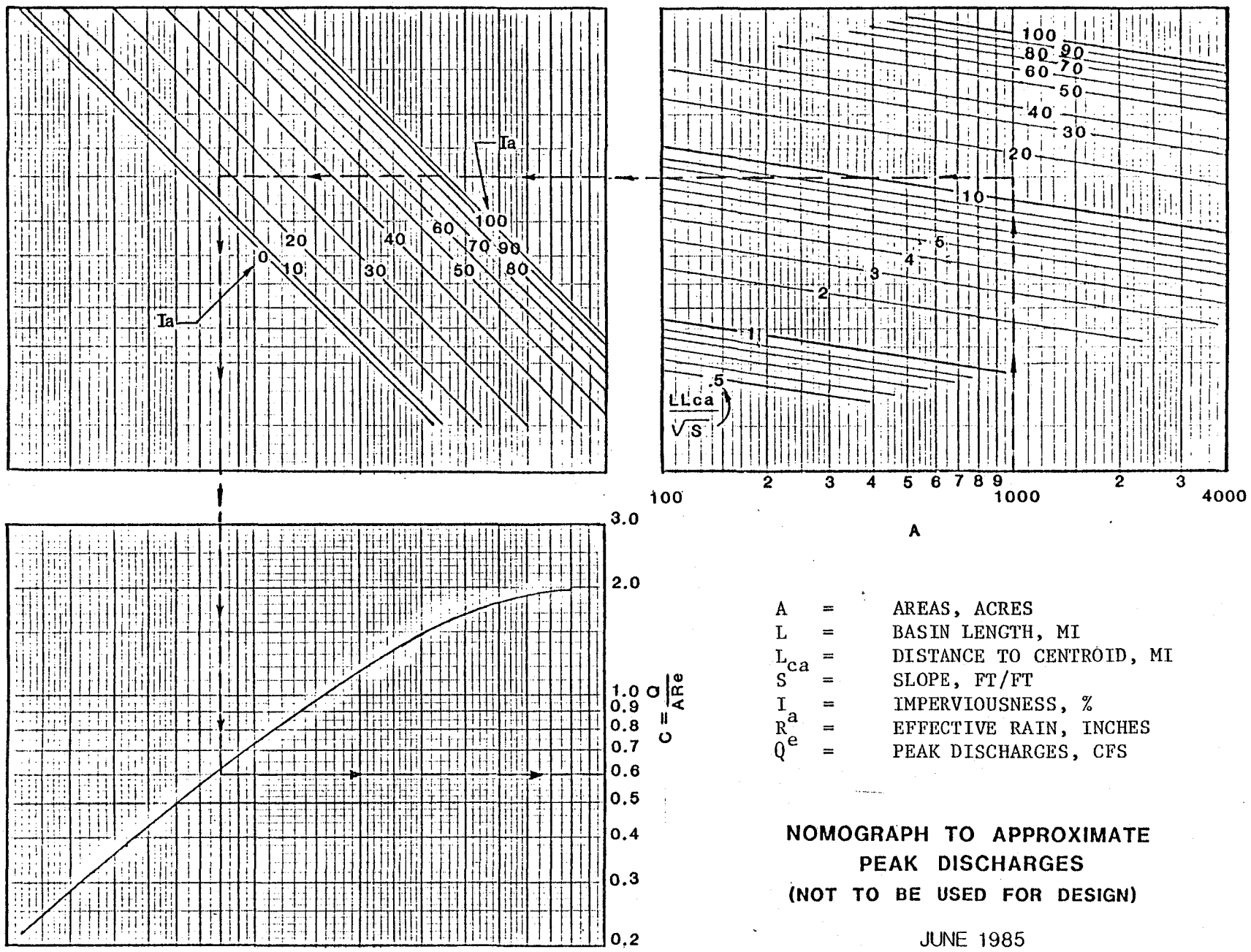
Ia = IMPERVIOUSNESS

NOMOGRAPH TO APPROXIMATE 2 - HOUR EFFECTIVE RAINFALL FROM 1 - HOUR RAINFALL

(NOT TO BE USED FOR DESIGN)

JUNE 1985

Boyle Engineering Corporation



A	=	AREAS, ACRES
L	=	BASIN LENGTH, MI
L_{ca}	=	DISTANCE TO CENTROID, MI
S	=	SLOPE, FT/FT
I_a	=	IMPERVIOUSNESS, %
R_e	=	EFFECTIVE RAIN, INCHES
Q	=	PEAK DISCHARGES, CFS

NOMOGRAPH TO APPROXIMATE PEAK DISCHARGES

(NOT TO BE USED FOR DESIGN)

JUNE 1985

Boyle Engineering Corporation

$$t_p = C_t\,[(L\,L_{ca}/\,S)]^{0.48} \qquad (4\text{-}1)$$

In which t_p = time to peak of the unit hydrograph from midpoint of unit rainfall in hours.

L = length along stream from study point to upstream limits of the basin in miles

L_{ca} = length along stream from study point to a point along stream adjacent to the centroid of the basin in miles.

S = weighted average slope of basin along the stream to upstream limits of the basin in feet per foot.

C_t = coefficient reflecting time to peak.

The time from the beginning of unit rainfall to the peak of the unit hydrograph is determined by:

$$T_p = 60t_p + 0.5t_u \qquad (4\text{-}2)$$

In which T_p = time from beginning of unit rainfall to peak of hydrograph in minutes.

t_u = time of unit rainfall duration in minutes.

The unit peak of the unit hydrograph is defined by:

$$q_p = \frac{640\,C_p}{t_p} \qquad (4\text{-}3)$$

In Which q_p = peak rate of runoff in cfs per square mile.

C_p = coefficient related to peak rate of runoff.

Once q_p is determined, the peak of the unit hydrograph for the basin is computed by:

$$Q_p = q_p\,A \qquad (4\text{-}4)$$

In Which Q_p = peak of the unit hydrograph in cfs.

A = area of basin is square miles.

4.4 Unit Storm Duration

For most urban studies the unit storm duration, t_u, should be 5-minutes. However, the unit duration may be increased for larger basins. It is convenient to have the unit duration incremented in multiples of 5-minutes (i.e. 10- or 15-minutes) with the maximum unit duration recommended at 15-minutes for the 1982 version of the CUHP. An acceptable unit storm duration, whenever it is larger than 5-minutes, should not exceed one-third of t_p. As an example, if the basin has a t_p = 35-minutes then an appropriate unit storm duration would be 5-minutes or 10-minutes (i.e. less than or equal to 1/3 t_p).

4.5 Basin Size Limits

The rainfall/runoff data used in the development of the 1982 version of the CUHP was obtained primarily from small basins. Basin sizes ranged from 0.15 square miles to 3.08 square miles. Although some extrapolation is justified, unlimited extrapolation of how the basin responds to rainfall is not justified. It is recommended that the maximum size of a basin to be analyzed with a single unit hydrograph be limited to 5-square miles. Whenever a larger basin needs to be studied, it is suggested it be subdivided into sub-basins of 5-square miles or less and individual sub-basin storm hydrograph be routed downstream using appropriate channel routing procedures. The routed hydrographs then need to be added to develop a single composite storm hydrograph.

4.6 Basin Shape Limits

The basin shape can have a profound effect on the final results and, in some instances, can result in underestimates of peak flows. Experience with the 1982 version of the CUHP has shown that whenever basin length is increased faster than basin area, the storm hydrograph peak will tend to decrease. Although hydrologic routing is an integral part of runoff analysis, the data used to develop the 1982 version of the CUHP is insufficient to say that the observed CUHP response with increasing basin length is valid. For this reason, it is important to subdivide irregularly shaped or very long basins (i.e., basin length to width ration of 4 or more) into more regularly

shaped sub-basins. A composite basin storm hydrograph can be developed using appropriate routing and by adding of the individual sub-basin storm hydrographs.

4.7 Basin Slope Limits and Considerations

The 1982 version of the CUHP was developed using data from basins having a range of major drainageway slopes between 0.005 ft/ft and 0.037 ft/ft. Caution needs to be used when extrapolating beyond this range. In natural and grass lined drainageways, channels become unstable when a Froude Number of 1.0 is approached. As a result, there are natural processes at work that limit the time to peak of a unit hydrograph as the drainageway becomes steeper. To account for this phenomena, it is recommended that the slope used in Equation 4-1 for natural drainageways and existing man made grass-lined channels be adjusted using Figure 4-1.

Grass lined channels designed and built using Urban Drainage and Flood Control District criteria, have a slope that limits maximum control velocities. A typical range of slopes for such channels is 0.004 ft/ft to 0.007 ft/ft. It is recommended that for preliminary estimating purposes a slope of 0.005 ft/ft be used along grass lined channels that are to be designed using UDFCD criteria.

The UDFCD criteria also limits the Froude Number to 0.8 for riprap lined channels. For this reason it is suggested that for preliminary estimating purposes where riprap channels are contemplated, a slope of ~~0.10~~ 0.01 be used with Equation 4-1. When a riprap channel is in existence, use the measured average channel profile slope.

In concrete lined channels and buried conduits, the velocities can be very high. For this reason, it is recommended the average <u>ground slope</u> (i.e. not flow line slope) be used where concrete lined channels and/or storm sewers dominate the basin drainageways. There is no upper limit recommended to the slope for such basins Where the flow line slope varies along the channel, calculate a weighted basin slope for use with Equation 4-1. Do this by first segmenting the major drainageway into reaches having similar longitudinal slopes. Then calculate the weighted slope using Equation 4-5.

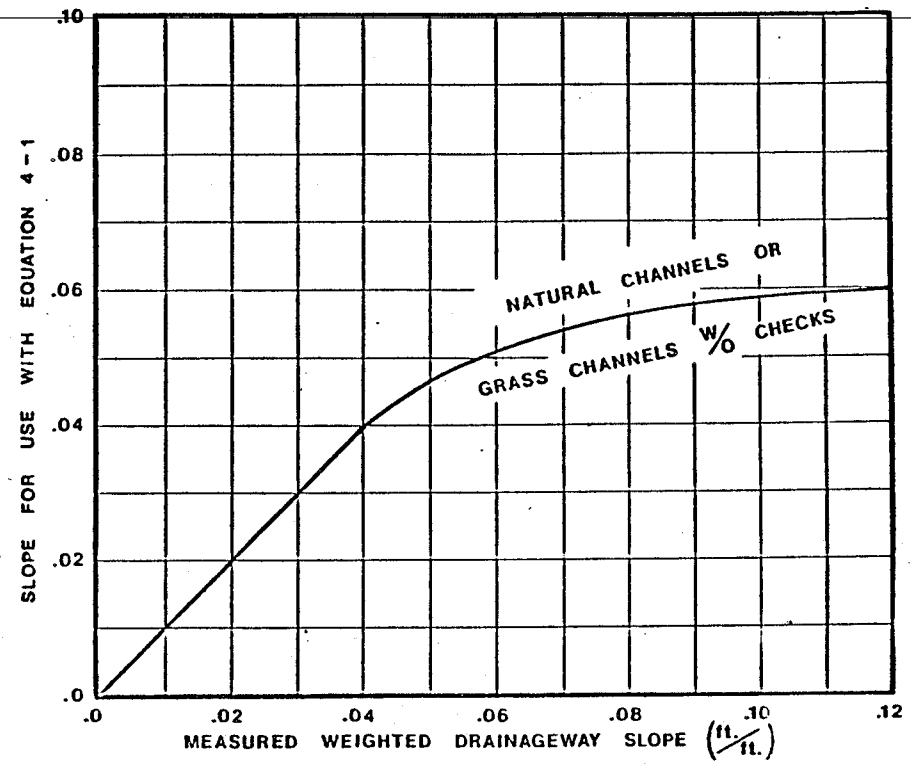


FIGURE 4 - 1. SLOPE CORRECTION FOR NATURAL DRAINAGEWAYS

$$S = \left[\frac{L_1 S_1^{0.24} + L_2 S_2^{0.24} + \ldots + L_n S_n^{0.24}}{L_1 + L_2 + L_3 + \ldots L_n} \right]^{4.17} \qquad (4\text{-}5)$$

In Which, S = weighted basin waterway slope in ft/ft

$S_1, S_2, \ldots S_n$ = slopes of individual reaches in ft/ft (after adjusting using Figure 4-1)

$L_1, S_2, \ldots L_n$ = lengths of corresponding reaches.

4.8 Basin Land Use Consideration

A lumped parameter model such as a CUHP relies on data from basins having relatively uniform land use. It is recommended that basins having zones of differing land use be subdivided into sub-basins having relatively uniform land use. As an example, if a lower half of a watershed has been urbanized and the upper half is to remain as open space, it is best to develop two distinct hydrographs. The upper sub-basin hydrograph will be based on the coefficients for undeveloped land and the lower sub-basin hydrograph will be the result of coefficients for the developed area.

4.9 Determination of C_t and C_p Coefficients

The value of C_t in Equation 4-1 may be determined using Figure 4-2. Note that the curve in Figure 4-1 can be represented using parabolic equations having the percent impervious (I_a) as an independent variable. Three sets of coefficients that are unique for each of the three ranges of imperviousness describe the curve. The mathematical description of the I_a vs C_t curve was developed so that the CUHP procedure could be computerized.

The value of C_p to be used in Equation 4-3 may be determined using Figure 4-3. The curve in Figure 4-2 is also represented with a parabolic equation.

To determine C_p, first obtain the value of the Peaking Parameter P, from Figure 4-3. Then calculate C_p using Equation 4-6:

$$C_p = P\, C_t\, A^{0.15} \qquad (4\text{-}6)$$

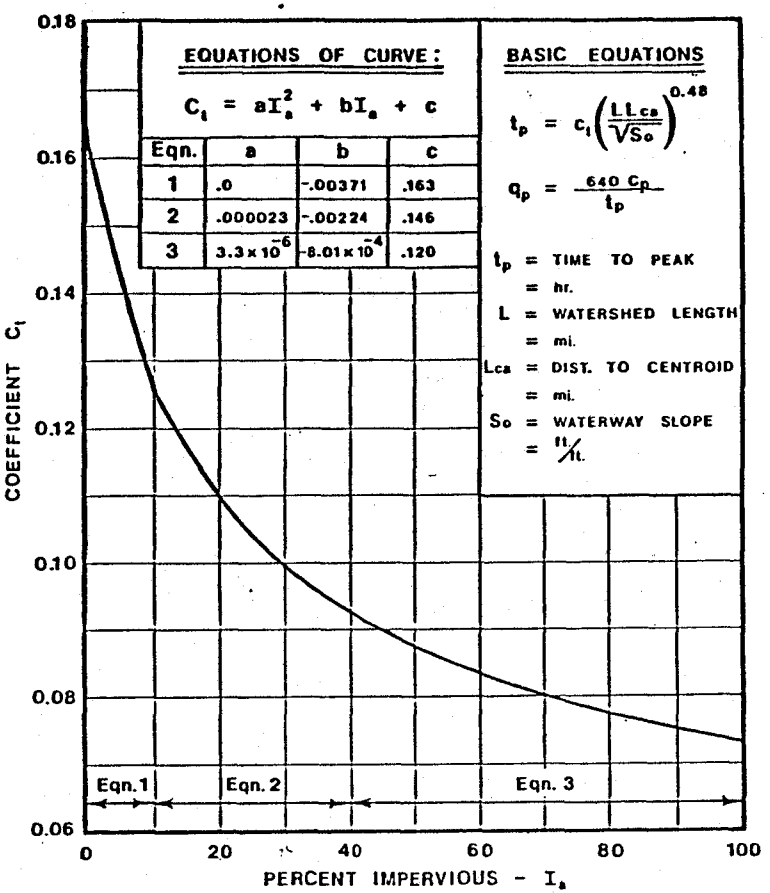


FIGURE 4-2. RELATIONSHIP BETWEEN C_t & IMPERVIOUSNESS

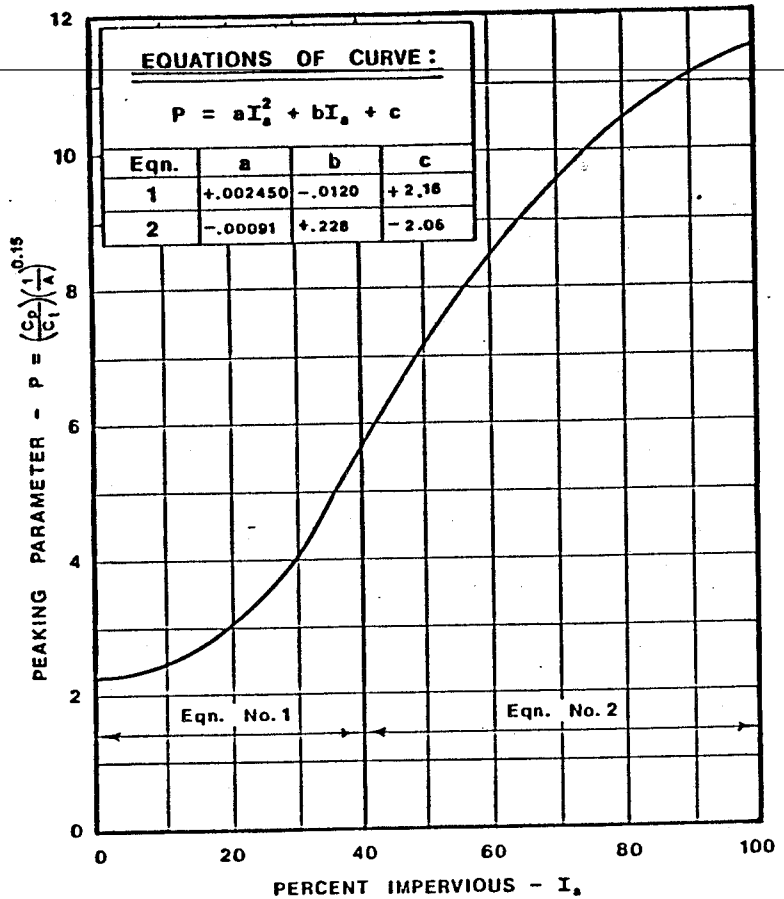


FIGURE 4 - 3. RELATIONSHIP BETWEEN PEAKING PARAMETER AND IMPERVIOUSNESS

In Which, C_t = coefficient from Figure 4-2
P = peaking parameter from Figure 4-3
A = basin area in square miles.

4.10 Unit Hydrograph Shape

The shape of the unit hydrograph is a function of the physical characteristics of the watershed. It incorporates the effects of watershed size, shape, degree of development, slope, type and size of drainage system, soils, and many other watershed factors. The shape of the unit hydrograph is also dependent on the temporal and spatial distribution of rainstorms and will vary with each storm event. As a result, a unit hydrograph based on rainfall/runoff data is an approximation that provides the engineer or hydrologist with a reasonable unit hydrograph shape for a given hydrologic region and land development practices.

Equations 4-1 through 4-6 are used to define the peak discharge and its location for the unit hydrograph. The widths of the unit hydrograph at 50 and 75 percent of the peak can be estimated using Figure 4-4. Note that the unit hydrograph width at 50 and 75 percent of the peak are given in hours. The two equations shown on Figure 4-4 mathematically describe the two lines on the figure.

In addition to knowing the location of the unit hydrograph peak and its width at two points on its ordinate, it also helps to know how to distribute the two widths around the peak. A study of many unit hydrographs generated using recorded rainfall and runoff events indicates that, as a general rule, 0.35 of the width at 50 percent of peak is to the left of the peak and 0.65 of the width is to the right of the peak. At 75 percent of the peak, 0.45 of the width is left of the peak and 0.55 of the width is to the right of the peak. However, on some hydrographs this rule needs to be modified. Whenever the above rule results in the hydrograph at 50 percent of peak being to the left of the peak by more than 0.6 T_p (T_p = the distance from zero to the peak of the unit hydrograph); the x-coordinate at 50 percent of peak should be placed at 0.6 T_p and at 75 percent of the peak it should be placed at 0.424 T_p. Figure 4-5 shows how a typical unit

hydrograph may be shaped to best approximate the trends found in the rainfall/runoff data.

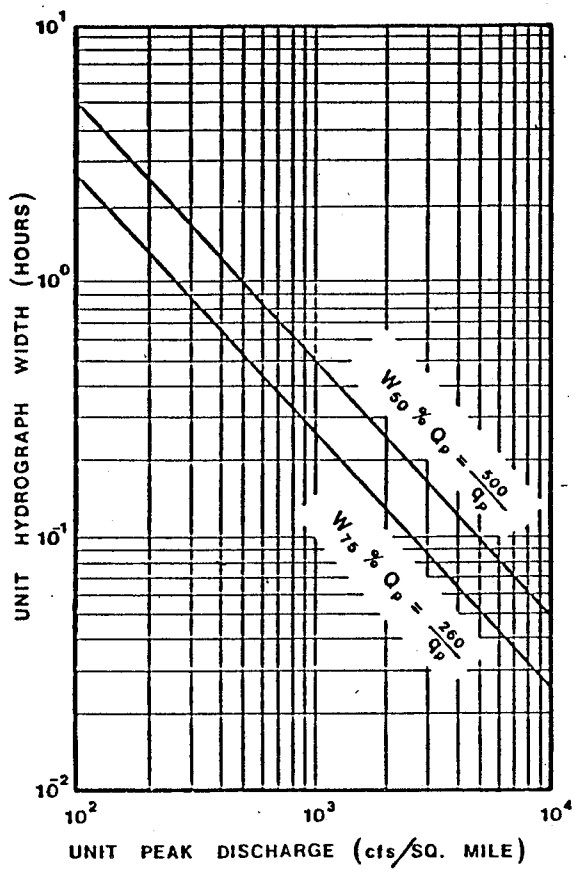


FIGURE 4-4. UNIT HYDROGRAPH WIDTHS

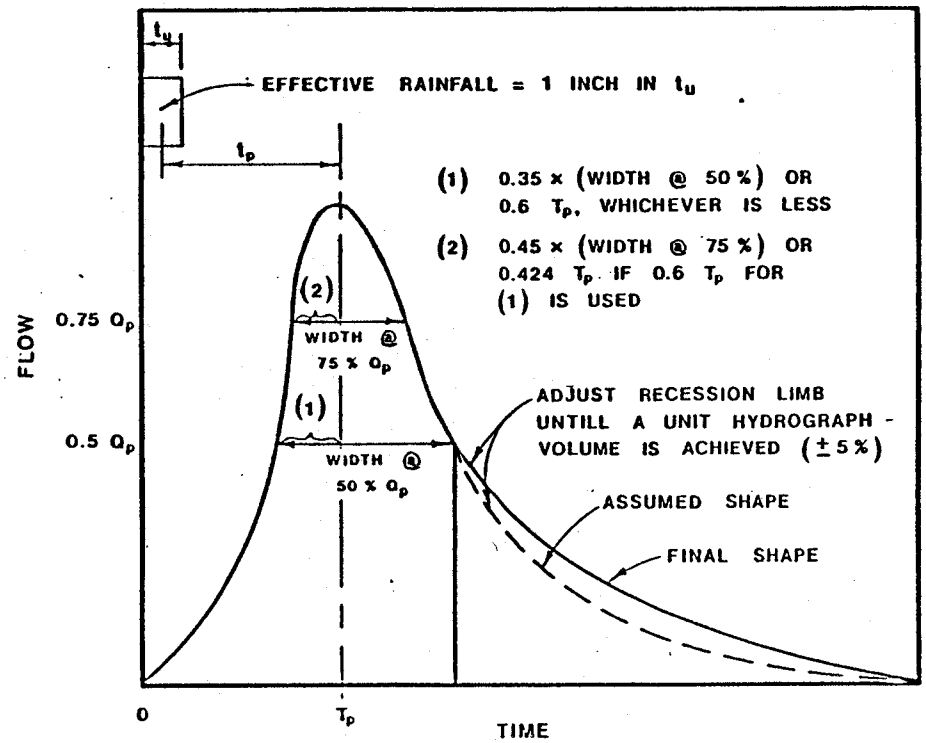


FIGURE 4-5. UNIT HYDROGRAPH

RELIABILITY OF DESIGN STORMS IN MODELLING

by

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Denver, Colorado

Abstract. Stormwater runoff is generally simulated using a design storm as input. The use of design storms significantly reduces the complexity of stormwater runoff analysis. As a result, the use of design storms is popular among engineers. The basic premise behind their use is that a design storm of a given return frequency will produce a simulated runoff peak and volume having the same return frequency. As an example, using a 5-year design storm will result in a 5-year runoff peak and volume.

Typically, design storms are developed by statistical analysis of rainfall records. The resultant temperal distributions of these design storms may be quite unlike the rainstorms occuring in nature. Worse, the use of design storms may not be resulting in runoff peaks and volumes having the same return frequency. As a result, drainage and flood control facilities designed using design storms may be oversized or undersized. Unfortunately, design storms are, by and large, not tested against long-term rainfall/runoff record simulation to determine if they will reasonably simulate runoff for a given return frequency.

A research effort is underway by the Urban Drainage and Flood Control District in the Denver, Colorado area to develop more reliable runoff simulation techniques. As a part of this effort, the design storm concept is being analyzed. Although the District's research effort is broad in scope and is exhaustive in nature, this paper discusses only the findings and the work to date relative to the design storm concept. The findings are based on applying 73 years of rainfall to simulate runoff with computer models. These computer models were calibrated for each basin using rainfall/runoff data collected since 1969. In addition, this paper examines the principles of design storm analysis and offers suggestions on how to develop and test design storms.

Introduction

In 1969 the Urban Drainage and Flood Control District (District) serving the metropolitan Denver, Colorado, area, became a cooperative sponsor with the U.S. Geological Survey (USGS) to collect rainfall/runoff data from urban catchments. In 1977 the District started its own data analysis and research effort to develop more reliable urban runoff simulation techniques. As a part of this effort, the design storm concent was analyzed. Although the District's research efforts are broad in scope, this paper presents only the recent findings related to the design storm concept.

Background

The use of design storms is very popular among drainage and flood control engineers and has achieved almost universal acceptance. In practice, it is assumed that a design storm of a given recurrence frequency will simulate a runoff peak and volume having the same frequency. Several techniques to develop the design storm from rainfall records have evolved or have been proposed over the past 20 years, including Chicago[1], ISWS[2], CUHP[3], and Urban Storm Runoff Inlet Hydrograph Study[4]. All of these techniques are based on the statistical analysis of rainfall data with very little, if any, verification of results through the investigation of the resultant runoff. Since only rainfall data are analyzed, independent of the total rainfall/runoff process, the validity of the design storm concept has been questioned. McPherson [5] has pointed out the fallacy of assigning identical frequencies of occurrence to rainfall and runoff when in reality both processes can exhibit statistical nonhomogeniety. The use of the design storm, according to McPherson, may be acceptable when only gross differences in level of protection from flooding are sought, but actual rainfall histories need to be used for the final design of operating facilities.[6]

Recent investigations have reported differences in predicted recurrence probabilities of peak flows using design storms when compared to simulated peaks using actual rainfall histories[7,8,9]. Marsalek observed that for the conditions he investigated in Ontario, Canada, only half of the linear variation in runoff peaks could be explained by the linear variation in rainfall intensity[8]. This led him to conclude that parameters of rainfall distribution are important in the generation of realistic runoff peak flow. Wenzel and Voorhees concluded that the design storm hyetograph and antecedent soil moisture conditions are very important parameters[9]. Both Marsalek and Wenzel expressed concern about the validity of the computer models used when extrapolating to less frequent large storm events. Both reported their models operated in a surcharge mode during the larger events and, as a result, they felt the accuracy of the predicted runoff peaks during very large rainstorms was suspect.

Rainfall Analysis

Local governments in the Denver area have adopted the use of the Urban Storm Drainage Criteria Manual (USDCM)[3] for planning and design of drainage and flood control facilities. The manual was published in 1969 by the Denver Regional Council of Governments (DRCOG) and contains rainfall isopluvial maps for the Denver metropolitan area for a variety of storm durations and return periods. The USDCM also contains a step by step procedure for reducing the isopluvial information to design storms. Subsequent to the adoption of the USDCM by local governments, another set of isopluvial maps was published by the National Oceanic and Atmospheric Administration (NOAA)[10]. The two sets of maps did not agree. The local governments continued to use the previously adopted USDCM rainfall information, while Federal agencies used the NOAA Atlas. Occasionally, disagreements occurred between various parties that were based solely on the argument that one set of design storms was better than the other. None of the arguments were backed by runoff data.

To examine the validity of the published isopluvial maps, the 73 maximum 30-minute rainfall depths recorded at the Denver gage from 1898 through 1971 were reduced to a Weibull probability plotting position. The rainfall data plotted on log normal probability paper are shown on Figure 1. The two lines shown on Figure 1 represent the 30-minute rainfall depths obtained from the DRCOG and NOAA isopluvial maps. It is interesting that neither of them fit the rainfall data well, yet the isopluvial maps information was the basis of disagreements between local and Federal officials. If one thorougly investigated the procedures used to develop the two sets of isopluvial maps, the reasons for these differences can probably be discovered. However, that is not the point. What is important to recognize is that published rainfall depth-frequency-duration maps are often used as the basis for development of design storms. Besides being statistically nonhomogeneous with runoff, the design storms in themselves may originate from information that may not be totally consistent with the rainfall data collected locally.

Also shown on Figure 1 are the 7-day antecedent precipitation data corresponding to each of the rainstorms used. Examining this data reveals that the antecedent precipitation is random in nature and it is not possible to draw any conclusions as to how it may affect the statistical distribution of runoff. To identify the potential effects of antecedent precipitation, it is necessary to examine the runoff peaks simulated while accounting and not accounting for antecedent moisture conditions. Such dual runoff simulation was performed and the results are reported later in this paper.

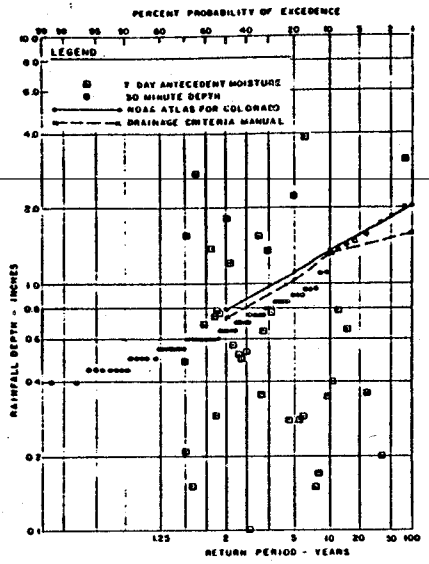

Figure 1. Probability Distribution Of 30-Minute Rainfall Depths at Denver Raingage, Period 1898 through 1971.

Runoff Analysis

Gaging Program

Shortly after its inception in 1969, the cooperative District and USGS rainfall-runoff data collection program set up approximately 30 gaging stations in the Denver metropolitan area. In 1977 the District started its own data analysis and interpretation effort. After a review of the data collected, approximately one-half of the gaging sites were abandoned and the data collected at these sites were dropped from the records because of problems ranging from variable basin boundaries to undefinable flow gage rating curves. It is difficult to interpret field data obtained even under perfect gaging conditions and the additional difficulties associated with the interpretation of questionable data was sufficient reason to limit data acquisition and analysis to sites where problems could be identified and resolved.

Seven of the stations remaining in the data collection program have been analyzed in detail to date. Of these seven, data from four of the basins

Table 1. Observed Characteristics of Gaged Basins

Site	Area (mi^2)	Length (mi.)	Slope (ft/ft)	Impervious (Percent)	Drainage System
1. Northglenn	0.56	1.17	0.034	35	Streets and Grass Channel
2. Denver	0.29	.84	0.005	40	Streets and Concrete Channel
3. Englewood	0.43	1.52	0.010	45	Streets and Pipes
4. Airport	0.15	.97	0.005	97	Large Pipes

were chosen for use in the investigation of the design storm concept. The four basins represent a variety of urban land uses, including a mobile home park (Northglenn site), single family residential (Denver and Englewood sites), and an airport terminal. Site maps for each basin are shown in Figures 2 through 5. Basin Characteristics for all four are summarized in Table 1.

The first three basins were selected for this study because the stormwater runoff is not subject to detention storage routing of any kind within the basins. There is a possibility of flow surcharge storage at the airport site during rainstorms having a recurrence interval in excess of 10-years; however, none of the gaged runoff events indicated a surcharged condition. Although the simulated runoff from the airport site may not accurately reflect the true probability for recurrence intervals greater than 10-years, all four sites can be considered to indicate runoff trends from sites that have no on-site detention storage.

Model Calibration

Runoff simulation was performed with the District's computer model that uses a linear unit hydrograph in combination with Horton's exponential decay infiltration function[11]. This model was selected for its simplicity and low cost of operation, and because it was found to reproduce the gaged runoff peaks and volumes with reasonable consistency. The model was calibrated using the rainfall and runoff data recorded at the four gaged basins. Both runoff peak and volume were calibrated. Two kinematic wave models were also calibrated for the Denver site, but they produced less consistent duplication of runoff peaks and were considerably more difficult to use in processing large numbers of storm events. As a result, neither kinematic wave model was used for this investigation.

Table 2 summarizes the calibrated infiltration and other rainfall losses used in the runoff simulations. The data for the four sites were analyzed in detail to gain insight into the effects of antecedent precipitation. Based on what was learned, the initial infiltration and depression storage values were adjusted for each storm to compensate for the effects of the recorded antecedent precipitation data. A comparison of the observed and simulated flow peaks at all four gaging sites is shown in Figure 6, which also demonstrates the validity of the computer model calibration.

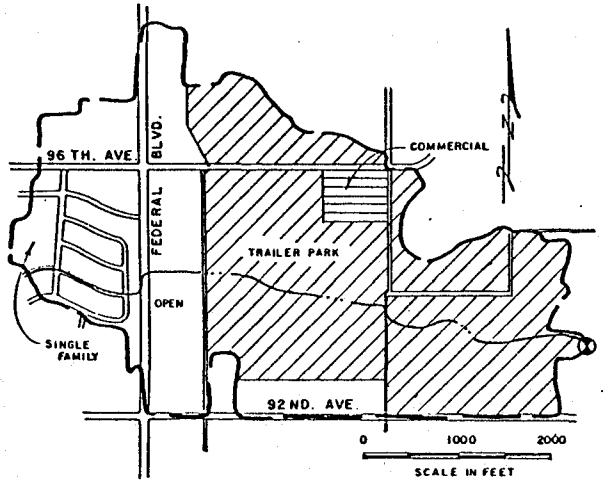

Figure 2. Map of Northglenn Gaging Site

Table 2. Calibrated Rainfall Loss Parameters

Site	Initial Infiltration (in/hr)	Final Infiltration (in/hr)	Exponential Coefficient (1/sec.)	Pervious Storage (in.)	Impervious Storage (in.)
1. Northglenn	3.00	0.50	0.0018	0.50	0.10
2. Denver	4.50	1.10	0.0007	0.40	0.10
3. Englewood	4.00	0.50	0.0018	0.40	0.10
4. Airport	3.00	0.50	0.0018	0.40	0.10

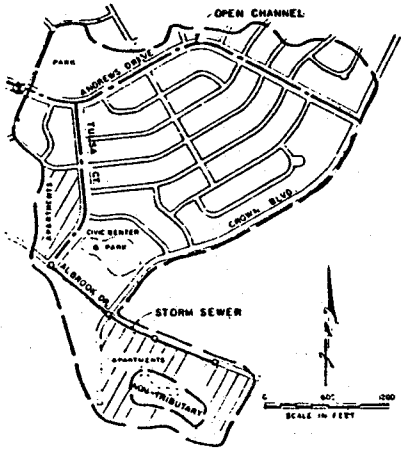

Figure 3. Map of Denver Gaging Site

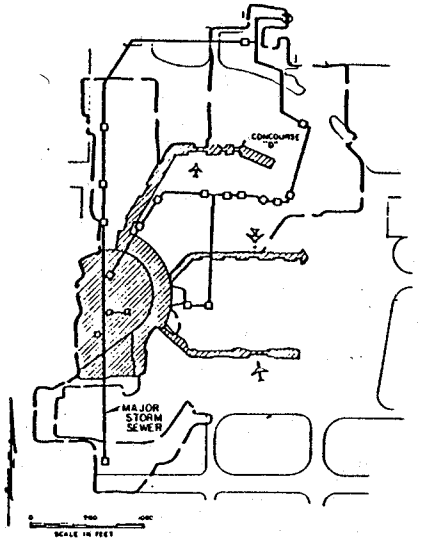

Figure 5. Map of Airport Gaging Site

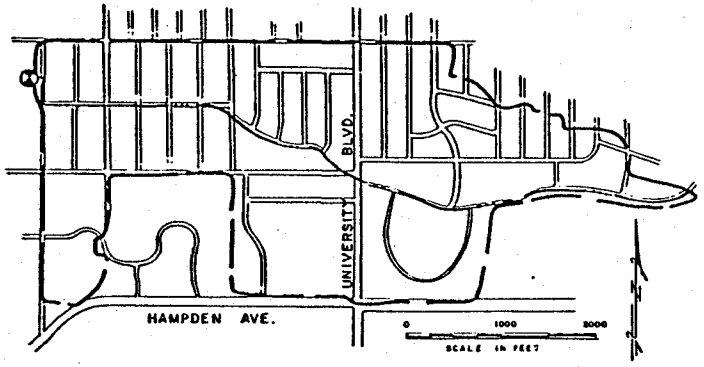

Figure 4. Map of Englewood Gaging Site

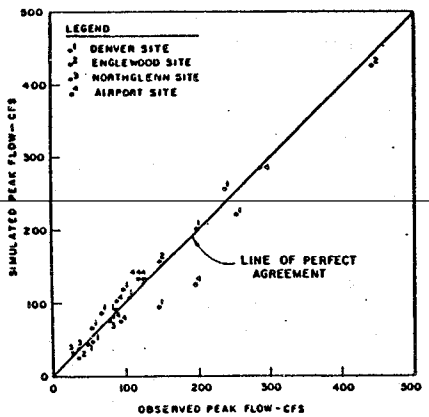

Figure 6. Comparison of Simulated and Observed Peak Flows

Runoff Probabilities and Design Storms

Having a calibrated computer model for each of the four gaged basins, it was possible to simulate the runoff that would result from a series of large rainstorms recorded at the Denver rain gage. Digitized rainfall data for a 73-year period (1898-1971) at the Denver rain gage were obtained by USGS from the National Weather Service. The 73 rainstorms having the largest recorded one-hour rainfall accumulation were selected to represent a partial duration series for the 73-year period of record. Because the digitized rainfall data was reported

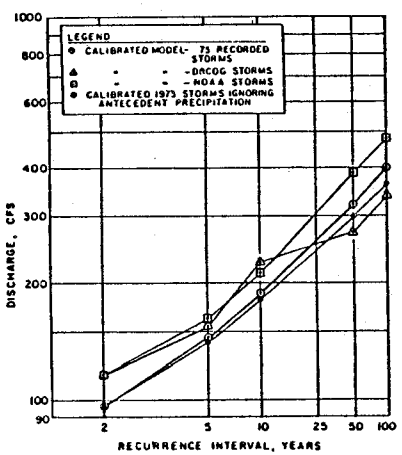

Figure 8. Peak Flow Probability Distribution for Denver Site

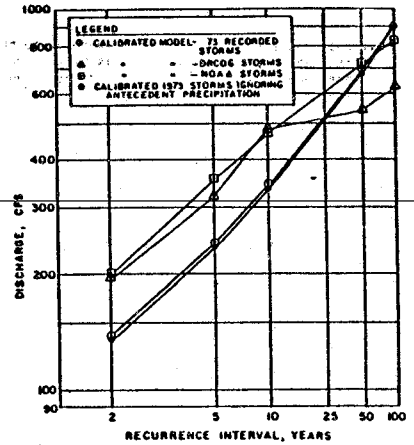

Figure 7. Peak Flow Probability Distribution for Northglenn Site

in 5-minute time intervals, and because the study basins were relatively small, a 5-minute unit hydrograph was selected for use in this study.

All 73 storms were used to simulate peak stormwater runoff from the four sites. Before the long term simulation was started, the antecedent precipitation for each of the 73 storms was quantified and initial rainfall abstractions were adjusted using the trends observed from the rainfall-runoff data at the gaged basins. A total of 73

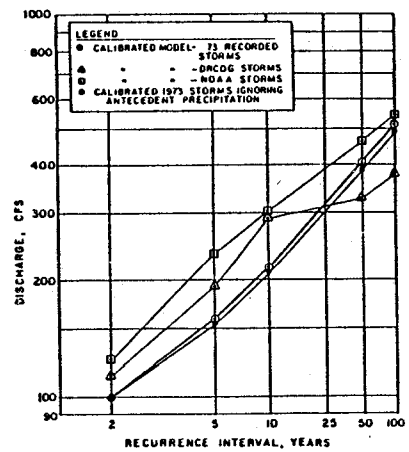

Figure 9. Peak Flow Probability Distribution for Englewood Site

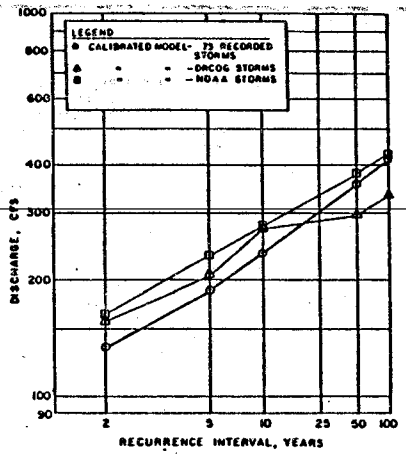

Figure 10. Peak Flow Probability Distribution for Airport Site

runoff peak flows were then simulated and analyzed using the Log Pearson Type III statistical analysis recommended by the U.S. Water Resources Council.[12] An identical statistical analysis was performed using peak flows simulated while ignoring the effects of antecedent precipitation. The resultant distributions of simulated peak flows vs. the recurrence interval are presented in Figures 7 through 10.

Antecedent precipitation appears to have a relatively small effect in the four test basins. It is possible that the algorithm used in correcting for antecedent precipitation underestimated its effects; however, there are other possible explanations. Denver is located in a semi-arid region of the United States and has an average annual precipitation of only 15 inches, with approximately one-half of that being rainfall. Referring to Figure 1, one can see that only ten of the 73 rainstorms shown had a 7-day antecedent rainfall that exceeded 0.8 inches. The lack of precipitation in this semi-arid region also results in the lack of antecedent precipitation and can explain why it has only a minor effect on the statistical distribution of runoff peaks.

The District's studies also indicate that the runoff from impervious surfaces in the Denver area tends to overshadow the runoff from the pervious areas. Runoff from impervious surfaces is very quick to occur and concentrate and is primarily responsible for the peak flow on small urban basins. At the same time, runoff from pervious areas occurs later in the storm and contributes little to the peak flow during the storms that are common to the semi-arid climate. High antecedent moisture may result in increased volumes, but it appears to have very little impact on peak flows from urbanized basins in the Denver area and possibly in other communities located in the semi-arid regions of the country.

To illustrate the relative accuracy of the two types of design storms being used in the Denver area, the peak flows estimated using the design storms are also shown in Figures 7 through 10. These design storms were developed in accordance with the USDCM procedures and the DRCOG and NOAA published isopluvials discussed earlier. It appears that using design storms developed solely from rainfall data can result in significant variances in the peak flows when compared to the statistical distribution of simulated peaks.

The predominant trend is for the tested design storm to overestimate the peak flow. This is not surprising since the statistical analysis commonly used in the development of design storms tends to maximize rainfall depths for all time increments. However, the author believes that the temporal distribution of the design storm can also affect runoff peak calculations. When a leading or advanced type of design storm distribution is used, the largest rainfall intensities occur at the time when rainfall losses are large and the runoff is reduced. If, however, a lagging storm pattern is used, the reverse is true and runoff is increased.

Because of the random nature of the temporal distribution of rainfall during rainstorms, it is naive to believe that a design storm can be developed to represent a real storm of a known recurrence interval. Design storms do not represent typical rainstorms and are a conglomeration of many storms that have occurred in the past. However, the concept of a design storm should not be abandoned just because a design storm does not represent a typically occurring rainstorm. A properly conceived design storm can still be a very valuable planning tool for use in estimating rainstorm runoff, provided its shortcomings are understood and it is used only when appropriate.

Alternatives To Design Storm

What alternatives are there to a design storm? One obvious alternative is to perform a long term simulation using a calibrated computer model and recorded rainfall data. This approach would be similar to the one taken during the District's investigation and may be more accurate. Such an approach takes substantial expertise, time and budget. Another alternative is to identify a number of recorded rainstorms[6], say five to ten, as being representative of a desired recurrence event which can be used to test final design of drainage facilities. The advantages of this approach include the use of recorded rainfall data, which accounts for a number of temporal distributions found in nature, and the user is provided with the argument that an arbitrary design storm is not being used. The use of selected recorded storms has disadvantages that are similar to the ones stated for the long term simulation method, without the advantages of using a complete rainfall history. Regardless of how the historic rainfall record is used, it is important to recognize that it is a historic record and is not an absolute predictor of the future.

When the planning effort has substantial potential economic impact, is regional in nature, and a high level of expertise and adequate budget can be committed, then long term simulation is justified and needs to be considered. For instance,

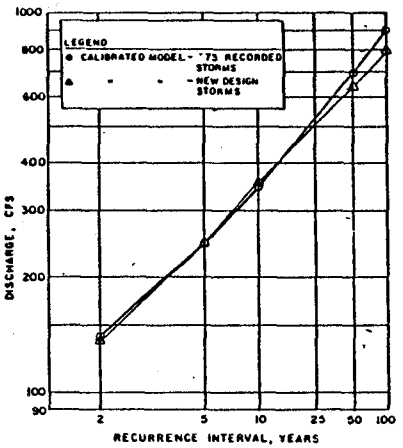

Figure 11. Peak Flow Distribution Using New Design Storms for Northglenn Site

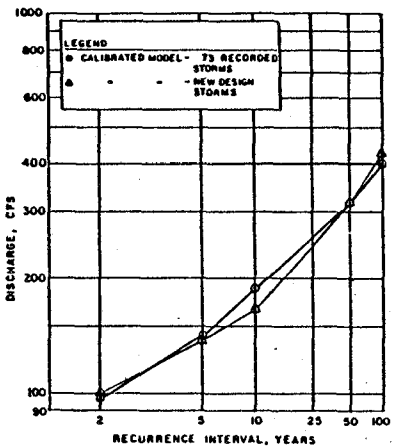

Figure 12. Peak Flow Distribution Using New Design Storms for Denver Site

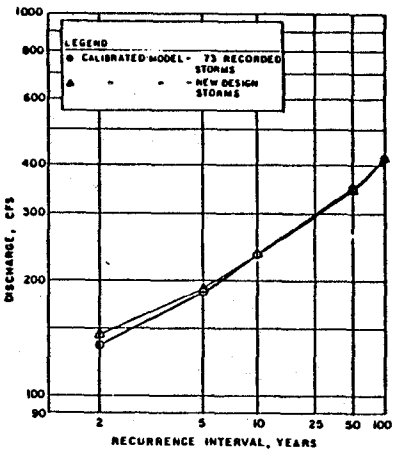

Figure 14. Peak Flow Distribution Using New Design Storms for Airport Site

regional non-point source water quality planning is an area where the design storm approach has very little merit. The non-homogeneous statistical characteristics of rainfall, antecedent precipitation, pollutant buildup rates, best management practices, and other phenomena, some of which are not well understood at this time, demand that a rainfall record of temporal and spacial distribution be used. Similar arguments can be made for regional flood control planning. However, when the problem shifts to smaller drainage subbasins and individual storm sewer or detention pond design, there is a need for simplified approaches to the problem. In such instances the design storm is state-of-the-art to many of the professionals.

Design Of A Design Storm

The need for simple, straightforward approaches in urban drainage and flood control is recognized by the Urban Drainage and Flood Control District. This need prompted the District to pursue the development of design storms that would simulate peak flows to fit the runoff probabilities. The approach required the use of readily available published rainfall information that had a broad base of acceptance. Because the NOAA Rainfall Atlas[10] was used by the State of Colorado outside the District and was exclusively in use by Federal agencies, it was selected as the base source of rainfall information. The one-hour rainfall depths for the various recurrence intervals were taken from the Atlas at the Denver Rain Gage location. A temporal rainfall distribution was then developed for each recurrence interval storm and was converted to a percentage of the NOAA Atlas one-hour rainfall depth. After several runoff simulation trials a series of temporal rainstorm distributions related to the NOAA Atlas information were found to reasonably reconstitute the peak at each recurrence interval for all four test basins. The results can be seen by comparing the peak flows obtained using the new design storms against the

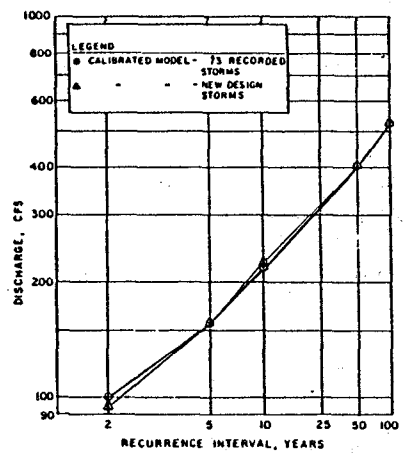

Figure 13. Peak Flow Distribution Using New Design Storms for Englewood Site

distribution curves of the peak flows obtained using long term simulation. The comparisons are made in Figures 11 through 14.

The results to date look encouraging; however the District is not yet prepared to revise the USDCM. Before permanent policy revisions are made to the design procedures used in the Denver area, the new design storms will undergo further testing using other gaged basins. The volumetric integrity of the hydrographs simulated using the new design storms also needs to be verified. Flood routing of the hydrographs obtained using the long term rainfall record will be performed using a variety of detention pond designs. The routed peaks will then be statistically analyzed and compared against routed peaks obtained using the new design storms. It is hoped that the new design storms will have realistic flood routing characteristics. If they don't, it may even be necessary to develop another series of design storms for use in the design of detention storage facilities. The ultimate goal of the District is to develop design storms that will consistently result in a reasonable prediction of the peak flows and volumes for storm runoff in the Denver area. These design storms will then be available to the engineering profession for use in the design of local drainage and flood control facilities. These new design storms are not intended to be used in regional non-point water quality studies, nor professed to be the only ones to be used in regional flood control projects.

Summary and Conclusion

The Urban Drainage and Flood Control District staff is involved in a research program to develop more reliable urban stormwater runoff simulation tools. As part of this effort, the District is investigating the design storm concepts being used in the Denver area. As a result of this investigation the following observations and conclusions have been made:

1. Design storms are developed using information contained in published isopluvial maps that may not be totally consistent with the long-term rainfall data collected locally.

2. Design storms developed using published isopluvial maps result in runoff peaks that can vary significantly from the peak flows obtained through statistical analysis of long-term simulation of runoff using recorded rainfall.

3. Antecedent precipitation in the semi-arid Denver area appears to have very little effect on the probability distribution of runoff from small urban basins.

4. It is possible to develop design storms that reasonably duplicate the peak flows from small urban basins at various recurrence intervals. However, this requires substantial rainfall-runoff data to permit calibration of computer models, long term simulation of runoff using recorded rainstorms and statistical analysis of simulated flow peaks and volumes.

5. Design storms developed using long term runoff simulation as a point of reference are useful in the planning of storm sewers, detention ponds and other flood control facilities. Recorded rainfall records that include temporal and spacial rainfall distributions need to be developed and cannot, at this time, be short cut through the use of a design storm whenever water quality studies are performed and/or stormwater management operational systems are designed. The use of design storms for these purposes fails to recognize the non-homogeneous statistical distribution of a large number of variables affecting the results.

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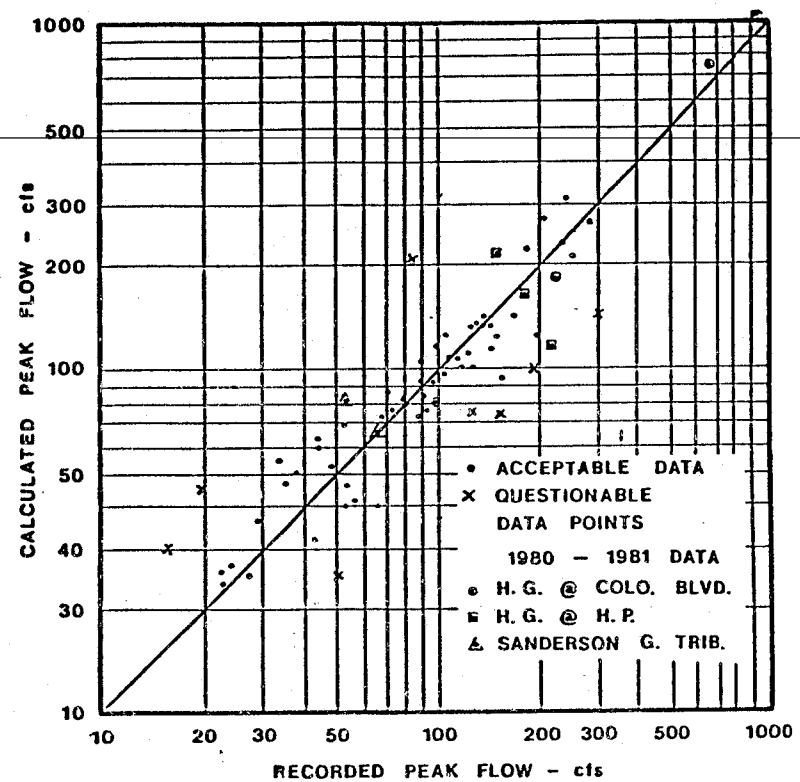

FIGURE 4-8. COMPARISON OF RECORDED PEAKS WITH CALCULATED PEAKS USING NEW CUHP

(5 - MINUTE UNIT HYDROGRAPHS)

COLORADO URBAN HYDROGRAPH PROCEDURE
COMPUTER PROGRAM - PC VERSION
(CUHPE/PC)

January, 1985

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INTRODUCTION

The Colorado Urban Hydrograph Procedure Computer Program (CUHPD) was originally developed in January, 1978 and updated in March, 1983. This personal computer version (CUHPE/PC) includes additional modifications made by Boyle Engineering Corporation under the direction of Ben Urbonas of Urban Drainage and Flood Control District. Major modifications incorporated into this PC version are as follows:

1. The equation for calculating time to peak has been modified to be consistent with the 1984 revision of the Urban Storm Drainage Criteria Manual.

2. Dimensional and common statements and some of the logic in the earlier version were modified to be adoptable on an IBM PC-XT.

3. Plotting routines included in the earlier version are eliminated.

4. Some input data requirements are simplified to facilitate data preparation effort. It is no longer necessary to define entire hyetographs or to define basin parameters Ct and Cp.

5. The program has, as an option, a capability of estimating hydrographs for small drainage areas (generally less than 90 acres) for which the original CUHP method is not applicable. The resulting flood peaks in many cases are generally comparable, but not identical to those estimated by the Rational Formula as specified in the Urban Storm Drainage Criteria Manual. To exercise this option, input the time of concentration (Tc) as computed by the rational formula procedure described in the Urban Storm Drainage Criteria Manual.

6. The program can, as an option, create an input data file containing storm hydrograph values for the purpose of combining and routing these hydrographs through the application of UDSWM2-PC, a version of the runoff block of the Storm Water Management Model (SWMM) originally developed by the U.S. Environmental Protection Agency and modified by the U.S. Army Corps of Engineers. A personal computer version of this program and a user manual is being distributed by Urban Drainage and Flood Control District as a separate package.

BASIC RELATIONSHIPS

A. Rainfall

The program requires the input of the design storm either as a detailed hyetograph or, as an option, as a 1-hour depth from which a detailed distribution is computed. The user is referred to the Urban Storm Drainage Criteria Manual for the recommended methodology of developing a design storm. Once the design storm is input or computed, rainfall losses are calculated by the program in accordance with the Drainage Criteria Manual and account for retention and depression storage and for infiltration. See Section 2 in the "Runoff" chapter of the Criteria Manual for the excess precipitation algorithm used by this program.

1. The infiltration rate can be handled in two ways for each basin:
   a) A constant infiltration rate throughout the storm can be specified. To use a constant infiltration rate, the user needs to enter only the initial infiltration rate (INFL).
   b) A variable infiltration rate based on Horton's equation can be calculated.

   Horton's equation is $f = f_o + (f_i - f_o)^{-at}$

   Where f = infiltration rate in inches/hour

   f_i = initial infiltration rate in inches/hour

   f_o = final infiltration rate in inches/hour

   a = decay coefficient in per second units

   t = time in seconds

2. Infiltration and depression losses can be varied from storm to storm. To do so, the user inputs multiplying factors with rainfall data for each storm to change the initial and final infiltration decay rate and depression storage values entered with basin data. For example, if the user wishes to use Horton's equation with f_i = 3.0 inches/hour, f_o = 0.5 inches/hour and a = 0.0018 per second for the 2- and 5-year storm and then wants to reduce the infiltration to a constant 1/2 inch/hour rate for the 10- through 100-year storms, let CINFL = 0.17 on the 10- through 100-year rainfall data card B-2 and leave all other correction factors blank. This will multiply f_i = 3.0 in/hr by 0.17 and reduce the initial infiltration rate to 0.51 in/hr, which will then decay rapidly to the final infiltration rate of 0.50 in/hr. If the user has entered rainfall loss adjustment factors with the rainfall data and does not wish to use them for a given basin during a computer run, he can nullify these correction factors by entering any positive number for XNUL on the basin data card D. A positive entry for XNUL will cause the computer to use only the rainfall loss factors entered with basin data.
3. Depression storage is entered independently for the pervious and impervious areas. A single correction factor is permitted with each rainstorm input if the user wishes to vary depression storage between storms.
4. If the user does not enter any rainfall loss adjustment factors (leave blanks in the adjustment factor columns) for a given storm, the infiltration and depression storage parameters entered with basin data will be used. Adjustment factors are provided to permit the user to account for the effects of antecedent precipitation on rainfall abstractions.

5. Up to 72 rainfall increments may be input for each storm hyetograph. If a detailed storm hyetograph is to be input, the user needs to identify the storm and then input incremental rainfall depths for each time increment of the hyetograph. If the 1-hour depth input option is to be used, the user needs to identify the design storm and then enter the 1-hour depth and the return period (i.e. 2-, 5-, 10-, 50-, or 100-year). The design hyetograph will then be computed by the program using the 2-hour storm distribution recommended in the District's Urban Storm Drainage Criteria Manual. The unit time specified under basin data (use only 5-minute or 10-minute unit time with the 1-hour option) has to be consistent with the time increment used for the tabulation of the design storm. Namely, if a 5-minute unit hydrograph is to be used, then the rainfall data should also be entered using 5-minute time increments.
6. A user may input up to 99 rainstorms. Any consecutive block of rainstorms may be called for processing by the basin cards. It is possible to input several sets of rainstorms developed for different parts of the metropolitan area and then to specify the specific rainstorms to be used by a given basin. It is also possible to input 5-minute and 10-minute rainstorms and then call the appropriate ones, depending on the unit hydrograph used, for a given basin.

B. Unit and Storm Hydrograph

The computer program will generate a unit hydrograph column matrix for each basin. It will then set up an excess precipitation row matrix for each storm called by the basin card which it then cross-multiplies with the unit hydrograph matrix. The resultant storm hydrographs are then printed out. Unit hydrographs are developed using basin parameters of imperviousness and area and the equations in the Urban Storm Drainage Criteria Manual as revised in 1984. As an option, the values of C_p and C_t, if known or predetermined, can be input directly to override the values computed using the Criteria Manual procedure.

Unlike the earlier version (i.e. CUHPD), CUHPE/PC utilizes the following equation for t_p as found in the 1984 revision to the Criteria Manual:

$$t_p = C_t \left(LLca/\sqrt{S}\right)^{0.48}.$$

It is no longer necessary to modify the C_t coefficient as was the case with CUHPD.

Because of the speed of the computer, it is not necessary to vary the unit duration of the unit hydrograph in order to save manual calculation time. It is recommended that a 5-minute unit hydrograph be used for all work; however, the user has the option of using any unit time provided the rainfall data used has the same unit time and a hyetograph is defined by the user.

If the 1-hour rainfall input option is used, the user may choose either 5- or 10-minute unit duration only; however the 5-minute duration is recommended for all 2-hour storms.

The user needs to input the following basin parameters to generate a unit and storm hydrograph:

1. Input Parameters. The following basin parameters are needed for the program to generate a unit and storm hydrograph.

a) Area - Basin area in square miles

b) Basin Length - The length in miles from the downstream design point of the basin or sub-basin along the main drainageway to the high point on its respective basin or sub-basin. When a basin is subdivided into a series of sub-basins, the sub-basin length used shall include the distance required for runoff to reach the major channel from the farthest point in the sub-basin.

c) Centroid Distance - Distance in miles from the design point of the basin or sub-basin along the main drainageway to its respective basin or sub-basin centroid.

d) Percent Impervious - The portion of the basin surface which is impervious in percent.

e) Basin Slope - The weighted average slope of the basin in feet per foot. See instructions in the Urban Storm Drainage Criteria Manual to determine the weighted slope.

f) As an option for small urbanized areas, (e.g. less than 90 acres), you may enter time of concentration in minutes. Procedure for estimating time of concentration is given in the Criteria Manual. By specifying the time of concentration, hydrograph peaks will be computed and displayed using both the CUHP method and the Rational Formula. This option will be exercised only if the 1-hour rainfall option is used for Card B2.

g) Pervious retention - Maximum depression storage on pervious surfaces in inches.

h) Impervious retention - Maximum depression storage on impervious surfaces in inches.

i) Infiltration rate - Initial infiltration rate for pervious surfaces in the basin in inches per hour. If this entry is used by itself, it will be used as a constant infiltration rate throughout the storm. If the next two entries are made, then this value will be used as the initial infiltration rate in Horton's equation.

j) Decay - Exponential decay coefficient in Horton's equation in "per second" units.

k) Final infiltration - Final infiltration rate in Horton's equation in inches per hour.

l) Nullifier of Rainfall Loss Adjustment Factors Entered with Rainfall - To be used if rainfall loss adjustment factors entered with rainfall data are to be nullified (optional).

2. Optional Input Parameters For Hydrograph Shaping. The unit hydrograph is developed by the computer using the algorithm used in the 1984 revision of the Urban Storm Drainage Criteria Manual. As noted before, the coefficients C_t and C_p are computed by the program; however, they can be specified by the user as an option.

The shaping of the unit hydrograph also relies on proportioning the widths at 50% and 75% of the unit hydrograph peak. The proportioning is based on 0.35 of the width at 50% of peak being ahead of the "time to peak" and 0.45 of the width at 75% of peak

being ahead of the "time to peak". These proportioning factors were selected after observing a number of unit hydrographs derived from the rainfall/runoff data collected by the U.S.G.S. for the District. It is possible for the user to overridge the unit hydrograph widths and the proportioning of these widths built into the program. For drainage and flood studies within the District, the program values shall be used. If the user has derived unit hydrographs from reliable rainfall-runoff data for a study basin and can develop a "calibrated" unit hydrograph for this basin, this option permits reshaping the unit hydrograph accordingly.

C. Rational Formula Option For Small Catchments

The Colorado Urban Hydrograph Procedure (CUHP) is applicable for basins that are generally larger than 90 acres. When the procedure is applied to smaller basins, it has a tendency to overestimate or underestimate peak runoff rates. The Colorado Urban Drainage and Flood Control District's "Urban Storm Drainage Criteria Manual" suggests the use of the Rational Method for such small basins. In the CUHPE/PC version, an optional routine is incorporated which computes peak runoff rates based on the Rational Method. In addition to basin parameters required for the CUHP, the user may specify the time of concentration for the basin of interest and the CUHPE/PC will then compute and display both peak runoff rates as computed by the CUHP and the Rational Method for comparison purposes. The program also contains a routine which modifies the CUHP procedure for generating hydrographs in such a way that their peaks become compatible with those by the Rational Method for small drainage areas.

1. Calculation of Peak Runoff Rates Based on the Rational Method.

The Rational Method is based on the following formula:

$$Q = CIA$$

In which, Q is defined as the maximum rate of runoff in cubic feet per second; C is a runoff coefficient; I is the average intensity of rainfall in inches per hour for a duration equal to the time of concentration; and A is the drainage area in acres. To estimate peak runoff rates using the above rational formula, the following relationships were developed and incorporated in the CUHPE/PC:

The rainfall intensity-duration relationship specified in the Urban Storm Drainage Criteria Manual is as follows:

DURATION (Minutes)	RATIO TO 1-Hour Rain Depth
5	0.28
10	0.45
15	0.57
30	0.79
60	1.00

Based on the above relationship, the average intensity for any given duration can be estimated by:

$$I_t = 28.5 I_{60}/(t+10)^{0.786}$$

In which, I_t is the average intensity in inches per hour for a duration of t minutes; t is the rain duration in minutes; and I_{60} is the rain depth in inches for a 1-hour duration.

The Urban Storm Drainage Criteria Manual provides a table specifying values of the runoff coefficient, C, to be used for various land use and storm frequency conditions. In the CUHPE/PC, these values are approximated by

$$C = R_e/R_t$$

In which, R_e is the effective rainfall and R_t is the total rainfall for a 2-hour storm as used in the CUHP method.

A value of the runoff coefficient as determined by the above equation is dependent upon rainfall magnitude, imperviousness, depression loss, and the Horton's infiltration equation parameters. On the other hand, the runoff coefficents specified in the Urban Storm Drainage Criteria Manual only vary with imperviousness and storm frequency. However, when typical values of rainfall intensity for a given frequency and depression loss are used, the runoff coefficients generated by the program generally agree with the values of C specified in the Urban Storm Drainage Criteria Manual.

When the user specifices the time of concentration and the 1-hour rain depth, the CUHPE/PC will automatically calculate and display a peak runoff rate based on the above Rational procedure. However, when this procedure is used for a drainage area larger than 160 acres, which is not recommended by District, a warning statement will appear in the output along with the estimated peak.

2. <u>Modification of the CUHP Procedure</u>

As mentioned earlier, the peak runoff rates developed by the CUHP may not be consistent with those by the Rational Method for smaller drainage areas. This inconsistency results from different procedures used to calculate the time to peak used for the CUHP and the time of concentration for the Rational formula. The differences, however, become small or almost negligible when the following relationship is used to relate the two variables.

$$t_p = 0.39\left(\frac{F_i^2-0.36F_i+0.07}{F_i^2-0.49F_i+0.14}\right)(T_c-6F_i)$$

In which, t_p is the time to peak in minutes, T_c is the time of concentration in minutes, and F_i is the impervious area, as a fraction.

In the CUHPE/PC, the above relationship is incorporated to estimate the time to peak for a given value of the time of concentration provided by the user, replacing the CUHP for estimating the time to peak. This optional procedure should

be only utilized for small drainage areas (the recommended threshold drainage area is 90 acres). When the procedure is utilized for larger drainage areas greater than 90 acres, a warning statement will appear in the output.

3. Rational Method Options.

The CUHPE/PC program has two options under which it will generate peak flows using the Rational Method. Under the first option (i.e. ICIA = 0 on Card C), if the user does not input a time of concentration (T_C), the program will only generate a hydrograph using the Urban Storm Drainage Criteria Manual (USDCM) procedure; however, if the user inputs a value of TC and the 1-hour rainfall depth, the program will generate a hydrograph using the USDCM procedure and will also calculate the peak flow using the Rational Method as explained earlier. Under the second option (i.e. ICIA = 1 on Card C) the user is required to provide an input value for TC and the 1-hour rainfall depth. The program will then generate a storm hydrograph that has been modified using the input TC and will also calculate the peak flow using the Rational Method. This second option is recommended only for areas smaller than 90 acres and should not be used at all for areas larger than 160 acres. Also, this second option has only been tested for areas larger than 10 acres in size and should not be used for smaller sub-catchments.

USE OF CUHPD WITH HEC-1

The Urban Drainage and Flood Control District's CUHPD program was modified in February 1983 to provide a user-selected option of writing storm hydrographs to an output disk file for subsequent use with the multi-plan method of river routing available with HEC-1 (flood hydrograph program of the U.S. Army Corps of Engineers). AT that time, the acronym for the District's program was changed to CUHPD to distinguish this version from previous versions of the program. The user of the HEC-1 output of CUHPD is advised to carefully review the HEC-1 Users Manual (HEC-1 Flood Hydrograph Package Users Manual, U. S. Army Corps of Engineers Water Resources Support Center, 1981) before proceeding with such use.

In converting the program to CUHPE/PC (i.e. to run on a PC type equipment), the HEC-1 output option was not fully tested and may require further work to generate output for use with mainframe or PC compatible HEC-1 program. The text that follows is, for the most part, found in the CUHPD User Manual.