West Regional
Dam Safety
Technical Seminar

DETERMINATION OF THE PROBABLE MAXIMUM FLOOD

Volume I

November 14-16, 1994
Phoenix, AZ

REMARKS

- Hydrograph Method is the preferred approach for estimating T_c

- This is the way how T_c is defined in unit hydrograph applications

- Regression Method must be used with caution. The origins and application limitations of these equations should be clearly understood. Often, the units and definition of the various parameters in the equations vary from one to the other

- The use of hydraulic calculations appears to be well founded. However, it may not be necessarily consistent with the way T_c is defined in unit hydrograph applications

DETERMINATION OF THE PROBABLE MAXIMUM FLOOD

November 14-16, 1994

INSTRUCTORS: Dr. John J. Cassidy, Bechtel Corporation
Terry L. Hampton, Mead & Hunt, Inc.
Dr. Arthur C. Miller, Penn State University

SEMINAR AGENDA

DETERMINATION OF PROBABLE MAXIMUM FLOOD

ASSOCIATION OF STATE DAM SAFETY OFFICIALS

1. Seminar Agenda
2. General Information
3. Instructor Resumes

Monday

	Time	Description
4.	8:00	Welcome - Introduction - Announcements An Overview Objective - To provide a reasonable approach for uniform application in determining the probable maximum flood (PMF) hydrograph PMF Guidelines Section 8-1
5.	8:30	Introduction to Runoff Analysis - The Hydrologic Cycle (1) Nature of Runoff Hydrographs (2) Basin Rainfall (3) Effective Rainfall (4) Loss Analysis
6.	9:00	Preliminary Review of Hydrologic Data PMF Guidelines Section 8-2
7.	9:20	Development of Hydrologic Criteria of the PMF PMF Guidelines Section 8-3
	9:40	COFFEE BREAK
8.	10:00	Data Acquisition PMF Guidelines Section 8-4

	Time	Description
9.	10:20	Review and Assessment of Data PMF Guidelines Section 8-5
10.	10:40	Subdivision and Drainage Area PMF Guidelines Section 8-6
11.	11:00	Approach to Tasks for PMF Development PMF Guidelines Section 8-7
12.	11:20	Unit Hydrograph Theory - Theory of Unit Hydrograph for Gaged Watersheds - Assumptions and Limitations PMF Guidelines Section 8-8 (1) Definition (2) Base Flow Separation (3) Duration of the Unit Hydrograph (4) Computation Time Increments - How Important
	12:00	LUNCH
13.	1:00	Methods of Calculating Infiltration (1) Uniform Loss Function - Time Index (2) Soil Conservation Service's Curve Number Method (3) Horton Equation (4) Green and Ampt Infiltration Equation (5) Physically Based Methodology
14.	1:45	Time of Concentration (1) Regression Methods (2) Hydraulic Methods (3) Hydrograph Method
15.	2:05	Clark Method for Deriving Unit Hydrographs (1) Conceptual Models of the Unit Hydrographs (2) Concept of the Instantaneous Unit Hydrograph (IUH)
	2:45	COFFEE BREAK
16.	3:05	Flood Hydrograph for Gaged Watershed - Sabrina Example
	5:00	Adjourn

Tuesday

	Time	Description
17.	8:00	Review and Questions
18.	8:30	Synthetic Unit Hydrography Theory for Ungaged Watersheds (1) Snyder Synthetic Unit Hydrograph (2) Soil Conservation Service Dimensionless Unit Hydrograph Developing Watershed Parameters for Ungaged Watersheds (1) Clark Instantaneous Unit Hydrograph (2) Snyder's Synthetic Unit Hydrograph (3) SCS Dimensionless Unit Graph
	9:30	COFFEE BREAK
	9:45	Continuation of Previous Lecture
19.	10:30	Unit Hydrographs for Sites with Limited Data PMF Guidelines Section 8-9 (1) Search for Applicable Unit Hydrographs (2) Regional Analysis (3) Data Required (4) Rainfall Analysis (5) Development of Generalized Regional Relationships
20.	11:00	Introduction to Flood Routing PMF Guidelines Section 8-11 (1) Hydraulic (2) Hydrologic (a) Muskingum (b) Muskingum Cunge (c) Reservoir Routing
	12:00	LUNCH
21.	1:00	Probable Maximum Flood (PMF) Development PMF Guidelines Section 8-10
22.	1:30	Data Collection for Ungaged Watersheds - Sensitivity
23.	2:00	Example: Corsorona Rapids
	3:00	COFFEE BREAK
	3:20	Continuation of Example Discussion
	5:00	Adjourn

Wednesday

	Time	Description
24.	8:00	Review and Questions
25.	8:30	Ungaged Watersheds - No Data (Bishopsville Example)
	9:30	COFFEE BREAK
26.	9:50	Glossary, Terms, and Report Formats
27.	10:10	Review and Questions
28.	11:00	Limitations of Unit Hydrograph Theory
29.	11:20	Hydrology (1) Future Models (2) GIS Databases (3) Kinematic Wave (4) New Research Being Developed
	12:00	LUNCH
30.	1:00	Example: Austen
	3:00	COFFEE BREAK
31.	3:20	Special Considerations (1) Dam Break Parameters (2) Antecedent Conditions (3) Start Q at Beginning of Flow (4) Reservoir Levels (5) Gate Operations (6) Sediment
	4:00	Summary
	4:30	Evaluations
	5:00	Adjourn

Chapter VIII

Determination of the Probable Maximum Flood

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Figure 8-8.2 Estimation of Clark Unit-Hydrograph Parameters

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Table 8-10.1 Minimum Infiltration Rates for Hydrologic Soil Groups

RESUME OF

DR. JOHN J. CASSIDY

JOHN J. CASSIDY MANAGER, HYDRAULICS/HYDROLOGY

EDUCATION:

B.S., Civil Engineering, 1952
M.S., Civil Engineering, 1960
Montana State University
Ph.D., Hydraulic Engineering, 1964
University of Iowa

SUMMARY:

38 Years: Management, direction and supervision of hydraulic and hydrologic engineering studies for hydroelectric projects, thermal power plants, and water-resource development projects for industrial and mining facilities; teaching and research in fluid mechanics, hydraulic engineering and hydrology.

EXPERIENCE:

1985 - Present: As Manager of Bechtel's Hydraulics/Hydrology Group, he manages and directs the analyses and studies performed by a 40-member team of highly-trained and experienced hydraulic and hydrologic engineering specialists. The Group performs a wide variety of tasks related to water resources development, the design of hydroelectric and thermal power plants, fisheries, and waste isolation. Capabilities of the Group include numerical simulation of fluid flow, development of flood hydrographs, water-resource assessment, and conceptual design and hydraulic analysis for intake and outlet works, penstocks, spillways, power tunnels, gates, valves, and surge chambers, and thermal modeling for streams, rivers, estuaries, and reservoirs. The Group performs field studies to collect data required for modeling and installs and monitors instrumentation. Because of their association with field studies, the Group maintains information files on instrumentation and recommends state-of-the-art equipment for use on Bechtel monitoring or data-collection projects. The Group performs general surface water studies to screen sites for potential developments and assesses the availability and adequacy of hydrologic data for site screening studies. During the past three years, Dr. Cassidy has served as a member of the National Academy of Science Committee on Potential Effects of Global Climate Change on Water Resources.

1975-1979 and 1981-1985: As Chief Hydrologic Engineer, he was responsible for the conduct of hydrologic studies for water resource hydroelectric and industrial projects. He directed a group of hydrologic and hydraulic engineers in field work and analyses related to water yield, flooding potential, and water conveyance. Studies frequently included surface runoff, modeling, prediction of erosion as well as sediment transport and deposition, reservoir operation simulation, development of design floods, and the analysis and use of basic rainfall and runoff data. Operation studies were conducted for both small and large hydroelectric facilities to determine capacity, average annual energy generation, and the probability of various levels of energy generation.

Major projects included: the Rio Grande Rositas Project (Bolivia); Sultan Hydroelectric Project (Washington); Ok Tedi Project (Papua, New Guinea); Setif Project (Algeria); Quartz Hill Project (Alaska); the PG&E Geyser 21 and 22 Geothermal Project and the Dinkey Creek Hydroelectric Project (California). He directed hydrologic, operation, and hydraulic studies for small hydro development at Lake Redding, California, New Martinsville, Ohio and Barrent River, Alabama. He monitored, coordinated and reviewed the field data collection efforts for a water balance study for three large storage ponds in the Weldon Springs waste-isolation site near St. Louis.

1979-1981: As Director of the State of Washington Water Research Center, he had technical and administrative responsibility for all water-related research studies funded through that center. Studies included methodology for assessment of undeveloped hydroelectric potential and the design and layout of small hydroelectric plants. He was a consultant with the National Rural Electric Cooperative Association on the design of small hydroelectric plants and gave several organized short courses on design and development of mini hydro plants. One of these short courses was presented to hydroelectric engineers and planners in Quito, Ecuador in 1980. He also managed several field studies for assessing the impact of the Mt. St. Helens eruption on water resources.

1974-1975: As Assistant Chief Hydraulic Engineer at Bechtel, he had supervising responsibility for the daily conduct, conceptual design and hydraulic analyses for spillways, outlet works, and other structures for water conveyance systems. He developed and supervised model studies for the spillway design, pump intakes, and intake structures for hydroelectric plants.

1972-1974: Dr. Cassidy was Chairman of the Civil Engineering Department of the University of Missouri and had direct responsibility for the academic and research programs in the department.

1962-1972: During this period, Dr. Cassidy was responsible for teaching and research programs at the University of Missouri in Hydraulics and Hydrology. Research and development projects included model studies and analytical studies for flows over spillways, flow in turbine draft tubes, resistance to flow in pipes subject to aquatic growth, and local flooding as a parameter for site planning. He spent one year with the U. S. Bureau of Reclamation in Denver, Colorado, developing methodology to analyze draft-tube surging in the Grand Coulee III draft tubes.

1958-1962: He served as an instructor in Civil Engineering and in Hydraulics at Montana State University and the University of Iowa, respectively, while pursuing graduate studies in hydraulics and hydrology.

1955-1958: As a Design Engineer with the Montana State Water Conservation Board, he performed hydraulic and structural designs associated with dams and canals for irrigation and performed hydrologic planning studies for irrigation projects. He analyzed several irrigation-district-owned storage reservoirs for the addition of mini-hydroelectric plants.

PROFESSIONAL REGISTRATIONS:

Registered Professional Engineer -
Montana, California, Washington, Idaho

MEMBERSHIPS:

Fellow, ASCE
Member, USCOLD, IAHR, ASDSO

AWARDS:

Named a Bechtel Fellow in 1985
ASCE Hunter Rouse Hydraulic Engineering Lecture, 1989
Elected to U. S. National Academy of Engineering, 1994

HOME ADDRESS: 4400 Capitol Court
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BORN: Gebo, Wyoming, June 21, 1930

EDUCATION: B.S.C.E. Montana State University 1952
M.S.C.E. Montana State University 1960
Ph.D. University of Iowa (Hydraulics) 1964

HONORS: Named Bechtel Fellow 1985
ASCE, Hunter Rouse Hydraulic
Engineering Lecture 1988

REGISTERED PROFESSIONAL
ENGINEER: California, Montana, Idaho, Washington

EXPERIENCE:

Dec. 1985 - Present: Manager, Hydraulics/Hydrology
Bechtel Corporation, San Francisco

July 1981 - Dec. 1985: Chief Hydrologic Engineer, Bechtel Civil and Minerals, San Francisco, California

Jan. 1979 - July 1981: Director, Washington Water Research Center and Professor of Civil Engineering, Washington State University, Pullman, Washington

Oct. 1975 - Dec. 1978: Chief Hydrologic Engineer
Bechtel, Inc., San Francisco, California

June 1974 - Oct. 1975: Assistant Chief Hydraulic Engineer,
Bechtel, Inc., San Francisco, California

Sept. 1963 - June 1974: Civil Engineering Faculty, University of Missouri, Department Chairman, 1972 - 1974

Summer 1967: NSF Short Course On Electronics, University of North Carolina, Chapel Hill, North Carolina

Summer 1970: Short Course on Analysis of Water-Resources Systems, University of Nebraska, Lincoln, Nebraska

1968 - 1969 Ford Foundation Engineering Resident,U.S. Bureau of Reclamation, Hydraulics Branch, Denver, Colorado

1960 - 1963 Half-time Instructor and Graduate Student, Department of Mechanics and Hydraulics, University of Iowa, Iowa City, Iowa

Summer 1960 NSF Fellow, Summer Fluid Mechanics Institute, Civil Engineering Department, Colorado State University, Fort Collins, Colorado

1958 - 1960 Instructor and Graduate Student in Civil Engineering, Montana State University, Bozeman, Montana

1955 - 1958 Design Engineer, Montana State Water Conservation Board, Helena, Montana

1953 - 1955 Enlisted Man, U.S. Army, Construction Inspector, Okinawa and Japan

1952 - 1953 Highway Engineer, U.S. Bureau of Public Roads, Missoula, Montana

1952 Deck Officer, U. S. Coast and Geodetic Survey, Seattle, Washington and Alaska

Duties at University of Missouri:

Jan. 1972 - June 1974 Professor and Chairman of Civil Engineering, and Member of Doctoral Faculty

1963 - 1974 In charge of Hydraulics, Fluid Mechanics, and Water Resources Programs in Civil Engineering (graduate and undergraduate).

1972 - 1974 Elected Member, Faculty Council on University Policy

1971 - 1972 Initiated teaching of Mechanics of Statics as a self-paced course.

1970 - 1972 As Chairman of Engineering Library Committee, was in charge of planning complete remodeling of Engineering Library. Wrote and submitted successful proposal for remodeling funds.

1969 - 1972 Member of University-wide Water Resources Committee.

1968 - 1971 In charge of organizing and instruction in new senior design course. This course utilized services of consulting firms in Missouri to provide design problems and subsequent periodic critiques of student designs during each semester.

1964 - 1966 In charge of remodeling and redesigning of Hydraulics Laboratory. Wrote and submitted successful NSF Equipment Proposal. Designed and supervised construction of new flumes, water tunnel, wind tunnel, and miscellaneous other equipment.

Duties with Bechtel, Inc.

1974 - 1975 Assistant Chief Hydraulic Engineer, Bechtel, Inc., San Francisco. Responsible for hydraulic analysis, including model studies, for complex hydraulic systems for large power plants, hydropower projects, irrigation projects, and other projects relating to mineral and petroleum processing.

1975 - 1978, 1981 - 1985 Chief Hydrologic Engineer, Bechtel, Inc. Responsible for hydrologic studies on engineering projects in the United States and foreign countries. Studies included a wide range of waste-isolation, hydroelectric and water-resource dams, mine developments and tailings dams, thermal power plants, flood control, and industrial projects.

1985 - Present Manager, Hydraulics/Hydrology Group. Responsible for direction of all studies related to hydraulics and hydrology for Bechtel projects in all offices.

Duties with Washington State University:

1979 - 1981 Director, Washington Water Research Center and Professor of Civil Engineering. General development and administration of water-research program. Teaching graduate courses in hydraulic engineering and hydrology.

Professional Activities:

1957 - 1958 Secretary-Treasurer, Helena Branch, Montana Section, ASCE

1967 - 1970 Member of Control Group of Task Committee on Wind Loads in Structural Division of ASCE

Oct. 1969 Manager, Hydraulics/Hydrology Group. Responsible for direction of all Delegate to ASCE Conference on Quality Teaching, Oklahoma State University

1969 - 1970 Second Vice President, Mid-Missouri Section, ASCE

1970 - 1971 First Vice President, Mid-Missouri Section, ASCE

1971 - 1972 President, Mid-Missouri Section, ASCE

1972 - 1974 Mid-Missouri Section ASCE, Delegate to and Secretary of District 16 Council

Jan. 1972 Mid-Missouri Delegate to ASCE Local Sections Conference, Dallas,Texas

Mar. 1972 Mid-Missouri Delegate to Zone III Caucus ASCE, Kansas City, Missouri

Oct. 1972 Moderator for Session on hydraulics and hydrology at St. Louis, National Meeting of American Water Resources Association

1971 - 1975 Member, Hydraulic Structures Committee, Hydraulics Division ASCE. In charge of Program Arrangements for two sessions at the 1973 National ASCE Water Resources Convention in Washington, D.C.

1970 - 1974 Member ASEE Fluid Mechanics Area Committee

1970 - 1974 Member of Education Committee, Consulting Engineers Council, Missouri

1973 - 1974 Chairman, Missouri Society of Professional Engineers Committee on Professional Registration

1974 - 1975 Chairman of ASCE Hydraulics Division Committee on Hydraulic Structures. Organized sessions on hydraulic structures for Specialty Conference at Knoxville, Tennessee in August, 1974

1975 - 1978 ASCE Hydraulics Division representative for organizing committee of joint IAHR-ASCE Symposium on Hydraulic Machinery held at Fort Collins, Colorado in September, 1978

1976 - 1980 Member of ASCE Hydraulics Division Executive Committee, Chairman 1978-1979

1976 - Present Member of USCOLD Committee on Hydraulics for Large Dams

1978 - Chairman and organizer of special short course on the Hydraulics Design of Cooling Water Systems for Large Thermal Power Plants, Colorado State University, Fort Collins, Colorado, June 6, 1978

1978 - 1980 Member of ASCE Technical Council on Computer Practices, Committee on Coordination within ASCE

1979 - Chairman and Organizer of session on Analytical Techniques in Hydrology, Symposium on Improved Hydrologic Forecasting, Asilomar, California, March 19, 1979

1979 - 1981 Director, Washington State Section of American Water Resources Association

1980 - 1983 Member of ASCE National Task Force on Dam Safety

1980 - Organized and chaired special symposium on environmental and physical considerations related to Salmon-spawning Gravel, Seattle, Washington

1981 - Organized and chaired special symposium on groundwater considerations in the Pacific Northwest, Spokane, Washington.

1981 - Organized and directed symposium on Indian Water Rights, Spokane, Washington

1980 - 1984 ASCE Hydraulics Division Representative to Management Group D, Chairman 1983

1982 - 1984 Co-Technical Chairman and Proceedings Editor, American Water Resources Association, Symposium on Hydrologic Forecasting, Seattle, Washington 1984

1983 - 1984 Member, ASCE Technical Activities Committee

1983 - 1985 Member, ASCE National Water Policy Committee, Chairman 1985

1983 - 1985 Member, ASCE Special Task Committee on International Relationships

1983 - 1986 Member, ASCE Technical Activities Planning Committee

1983 - 1993 Member, San Francisco Section of ASCE, Committee on Water Policy, Chairman, 1987-88, 1988-89

1984 - Member, Organizing Committee for Waterpower 1985

1985 - Chairman, Committee on Hydraulic Engineering for Dam Safety, EPRI/FEMA Workshop on Dam Safety Research, Denver, Colorado

1985 - 1988	Chairman, Local Tour Committee for 16th International Congress on Large Dams
1985 - 1988	Member, Public Relations Committee for 16th International Congress on Large Dams
1986 - 1987	Member, Organizing Committee for Waterpower 87.
1986-1988	Chairman, Finance, Steering Committee to Organize the 16th Congress of the International Commission on Large Dams
1987 - Present	Chairman, Committee on Hydraulics for Dams, International Commission on Large Dams
1987 - Present	Chairman, Subcommittee on Vibration of Structures, Committee on Hydraulics for Large Dams, International Commission on Large Dams
1987 - Present	Member, Civil Engineering Department Advisory Committee, California Polytechnic University, San Luis Obispo, California
1988 - Present	Member, Advisory Committee, St. Anthony Falls Hydraulics Laboratory, University of Minnesota, Minneapolis, Minnesota
1988 -	Secretary for Question 63, Design Floods, 16th Congress, International Commission on Large Dams, San Francisco, California, June, 1988
1988 - Present	Adjunct Professor of Civil Engineering, Department of Civil Engineering, University of California, Berkeley, California
1988-1991	Chairman, Task Committee to Develop a Position Paper on Dredging and Disposal of Dredged Material, San Francisco Section of ASCE, Water Policy Committee
1988-1992	Elected Member of Board of Directors, U. S. Committee on Large Dams
1990 -	Elected Vice President, San Francisco Section, ASCE
1990 -	Convener, FEMA Sponsored Workshop on Probable Maximum Precipitation and Probable Maximum Flood, Berkeley Springs, Virginia
1991 -	Chairman, Panel on Water Resources Infrastructure, ASCE Civil Engineering Research Foundation, Forum on Research Needs, Washington, D. C.

1991-1993 Member, Technical Program Committee, Waterpower 1993. Chairman, Sessions on Research and Development, Geotechnical Engineering and Hydraulic Design, Nashville, Tennessee

1992-1993 Member, Technical Program Committee, ASCE National Conference on Hydraulic Engineering, San Francisco, California

Societies:

Fellow, American Society of Civil Engineers
Honor Member, Chi Epsilon (Elected by students of University of Missouri Chapter, 1968)
Member, American Water Resources Association
Member, American Geophysical Union
Member, U.S. Committee on Large Dams
Member, International Association for Hydraulic Research

Principal Investigator for Following Sponsored Research Projects:

1964 - 1965 Flow Through a Spillway Flip Bucket, U.S. Army Engineers Waterways Experiment Station, Vicksburg, Mississippi

1964 - 1965 National Science Foundation Undergraduate Equipment Grant, Washington, D. C.

1965 - 1966 Hydraulic Efficiencies of Grate Inlets, Rowland Engineering Company, St. Louis, Missouri

1965 - 1966 Analytical Formulation of Flow Over a Spillway Toe Curve, U. S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi

1966 - 1968 River Basin Development, Office of Water Resources Research, Washington, D. C.

1967 - 1968 Effects of Algae Growth on Flow in Pipes, Missouri Clean Water Commission, Jefferson City, Missouri

1967 - 1968 Unsteady Flow in Low-Pressure Pipelines, U.S. Bureau of Reclamation, Denver, Colorado

1967 - 1968 Motion of Spheres in a Highly Turbulent Field, NSF, Washington, D.C.

1968 - 1969 Draft-Tube Surging, U.S. Bureau of Reclamation, Denver, Colorado

1969 - 1971 Motion of Solid Particles in a Turbulent Field, National Science Foundation, Washington, D. C.

1971 - 1972 Reaeration of Water with Turbine Draft Tubes, Office of Water Resources Research, Washington, D. C.

1972 - 1973 Environmental Problems Due to Urban Development, Department of Community Affairs, Jefferson City, Missouri

1973 - 1974 Two-Dimensional Model Study of American Falls Dam Spillway, Bechtel, Inc., San Francisco, California

1979 - 1980 Assessment of Small Hydroelectric Potential in the State of Washington, U.S. Department of Energy, Washington, D. C.

1980 - 1982 Coordination and Review of Research Related to Effects of Mt. St. Helens Eruption on Water Resources, Office of Water Resources Research, Washington, D. C.

Publications:

"Irrotational Flow Over Spillways of Finite Height", ASCE Journal of the Engineering Mechanics Division, V. 4591, EM6, December, 1965.

"Aerodynamic Characteristics of Farm Grains", Transactions of ASAE, Vol. 9, No. 1, 1966.

"Hydraulic Characteristics of Grate Inlets", National Research Council, Highway Research Record, No. 123, Publication 1365, January, 1966.

"Aerodynamic Properties of Black Walnuts; Application in Separating Good from Bad Walnuts", Transactions of ASAE, Vol. 10, No. 1, 1967.

"Flow Through a Spillway Flip Bucket", ASCE Journal of the Hydraulics Division, Vol. 6487, No. HY2, March, 1969 (with C. W. Lenau).

"Experimental Study and Analysis of Draft-Tube Surging", Report No. HYD-591, U.S. Bureau of Reclamation, May,1969.

"Designing Spillways for Negative Pressures", ASCE, Journal of the Hydraulics Division, Vol. 96, No. HY3, March 1970.

"Frequency and Amplitude of Pressure Surges Generated by Swirling Flow", Paper El, Transactions of the 1970 Symposium on Hydraulic Machinery and Cavitation, International Association for Hydraulic Research, Stockholm, Sweden, August 1970.

"Realistic Civil Engineering Design", Journal of ASEE, Vol. 61, No. 2, November, 1970, Part 1.

"Observations of Unsteady Flow Arising After Vortex Breakdown", Journal of Fluid Mechanics, Vol. 41, Part 4, 1970.

"Five Years of Teaching Civil Engineering Design with the Help of Practicing Engineers", Proceedings of the ASCE Conference on Civil Engineering Education, Ohio State University, March, 1974.

(Textbook)
Hydrology for Engineers and Planners, Iowa State University Press, Ames, Iowa, January, 1975 (with A. T. Hjelmfelt, Jr.).

"What the Design Engineer Needs from the Hydrometeorologist", Proceedings of the Annual Conference, American Meteorological Society, Toronto, Canada, October, 1977 (with W. P. Henry).

"What the Design Engineer Needs from the Hydrometeorologist, II", Journal of Applied Meteorology, Vol. 17, No. 10, October 1978 (with W. P. Henry).

"Hydrologic Study for Larona Hydroelectric Development", Proceedings of the ASCE Hydraulics Division Specialty Conference on Conservation and Utilization of Water and Energy Resources, San Francisco, August, 1979.

"Site Development and Hydraulic Design for Small Hydroelectric Developments", Proceedings of the International Symposium on Small Hydroelectric Development, National Rural Electric Cooperative Association, Washington, D. C., August, 1980.

"State of the Development of Small Hydroelectric Plants", Proceedings of the 2nd Bonneville Power Administration Conference on Energy Supply, BPA, Portland, Oregon, October 27, 1980.

"Hydraulic and Hydrologic Computer Applications", Journal of Technical Councils of ASCE, Vol. 108, No. TC2, November, 1982.

"Spillways of High Dams", Chapter 4, <u>Developments of High Dams</u>, Elsevier Applied Science Publishers, Ltd., London, England, 1984 (with R. A. Elder).

"Labyrinth-Crest Spillway, Planning, Design, and Construction", Proceedings of the International Conference on Hydraulic Aspects of Flood Control, BHRA, London, England, September, 1983 (with C. A. Gardner and R. T. Peacock).

"Site Development and Hydraulic Analysis", Chapter 4, Small and Mini Hydropower Systems, McGraw-Hill Book Company, New York, NY, 1984.

"Dams and Reservoirs", Chapter 5, Small and Mini Hydropower Systems, McGraw-Hill Book Company, New York, NY, 1984.

Editor, Critical Assessment of Forecasting in Western Water Resource Management, Proceedings of the June 11-13, 1984 AWRA Symposium, American Water Resources Association, Bethesda, MD., 1985 (with D. P. Lettenmaier).

"Sultan River Hydroelectric Project, Hydrologic Analysis", Proceedings of the Symposium, Waterpower 1985, Las Vegas, Nevada, September 1985 (with S. L. Hui).

"Boardman Labyrinth-Crest Spillway", ASCE Journal of Hydraulic Engineering, Vol. III, No. 3, March, 1985 (with C. A. Gardner and R. T. Peacock).

"Hydrologic Studies for Dinkey Creek Hydroelectric Power Project", Proceedings of the Symposium, Waterpower 87, August 1987 (with S. L. Hui).

(Textbook)
Hydraulic Engineering, Houghton Mifflin Company, Boston Mass., 1988 (with J. A. Roberson and M. H. Chaudry).

"Consideration with Regard to the Choice of Recurrence Interval for a Design Flood", proceedings of the 16th Congress, International Commission on Large Dams, June 1988, San Francisco (with D. B. Cherry, S. L. Hui, and J. E. Welton).

"Flood Criteria and the Safety of Tailings Dams", proceedings of the International Symposium on Safety and Rehabilitation of Tailings Dams, ICOLD Executive Meeting, Sydney, Australia, May 1990.

"Impact of Artificial Reservoirs on Hydrological Equilibrium", Keynote Address, Proceedings International Conference on Water Resources in Mountainous Regions, Lausanne, Switzerland, September 1990.

"Design Flood for Harriman Dam, a Site Specific PMP and PMF Study", proceedings of the Symposium on Dam Rehabilitation, Annual Meeting of the U.S. Committee on Large Dams, Forth Worth, Texas, April 1992.

"Dams and Extreme Floods - Operation", invited, "General Report for Session B, International Symposium on Dams and Extreme Floods", Executive Meeting of the International Commission on Large Dams, Granada, Spain, September 1992.

"Flood Data and the Effect on Dam Safety", invited, Keynote Address, International Conference on Dam Safety, Grundewald, Switzerland, April 1993.

"Design Floods and Flood Calculations," invited, Keynote Address, First Algerian Congress on Large Dams, Algiers, Algeria, May 1993.

"Hydraulic Design for Replacement of Floor Blocks at Pit 6 Stilling Basin," First USCOLD Technical Conference, Denver, Colorado, August 24, 1993.

"A Guideline for the Determination of Probable Maximum Flood for Civil Works," Proceedings of Waterpower 93, Nashville, Tennessee, August 13, 1993.

Oral Presentations:

"Spillway Characteristics as a Function of Crest Shape", ASCE National Environmental Engineering Conference, Chattanooga, Tennessee, May 1965.

"Analysis of Flow Through Spillway Flip Buckets", ASCE Hydraulics Division Specialty Conference, Madison, Wisconsin, August 1967.

"Proper Hydraulic Design of Sanitary Sewers", Annual Conference Missouri Water Pollution Control Federation, St. Louis, Missouri, May 1968.

"Design of Grate Inlets for Highway Storm Drains", National Highway Research Board Meeting, Washington, DC, January 1969.

"Frequency and Amplitude of Pressure Surges Generated by Swirling Flow", International Association of Hydraulic Research, 5th Symposium on Hydraulic Machinery and Equipment, Stockholm, Sweden, August 1970.

"Faculty Preparation for Teaching Students to Better Meet Employers' Needs - Viewpoint of the Teaching Faculty Member", Invited Paper, National Conference ASEE, Lubbock, Texas, June 1972.

"Design Considerations for Reservoirs and Spillways", Short Course on Fundamental Hydraulics and Hydrology of Dam Design. University of Missouri - Rolla, October 11, 1976; May 15, 1977; June 1, 1978; May 23, 1979; May 23, 1980.

"Hydraulic Design in High Velocity Flow", A Series of invited lectures presented to the Engineering and Research staff of the U.S. Bureau of Reclamation, Denver, Colorado, June 18-19, 1979.

"Site Development and Hydraulic Design for Small Hydroelectric Developments", Invited Lecture, AID Symposium on Small-Hydroelectric Development, Quito, Ecuador, August 18-20, 1980.

"State of Development of Small Hydroelectric Plants", 2nd Annual Bonneville Power Administration Conference on Energy Supply, Portland, Oregon, October 27, 1980.

"Practical Considerations of the Probable Maximum Flood Concept in Relation to Dam-Hazard Classification", ASCE Annual Convention, St. Louis, Missouri, October, 1981.

"Labyrinth-Crest Spillway, Planning, Design, and Construction", International Conference on Hydraulic Aspects of Flood Control, London, England, September 1984.

"Hydraulic Studies for Dinkey Creek Hydroelectric Power Project", Waterpower 87, Portland, Oregon, August 1987.

"Hydrologic Engineering Calculations", a series of lectures and workshops on Hydrologic Engineering for Hydroelectric Project Design, presented to the Ministry of Energy (PLN), Jakarta, Indonesia, November-December 1989.

"Problems and Challenges in the Design of Hydroelectric Projects", Deere Memorial Lecture, College of Engineering, University of Iowa, Iowa City, Iowa, April 1990.

"Impact of Artificial Reservoirs on Hydrological Equilibrium", Keynote Address, International Conference on Water Resources in Mountainous Regions, Lausanne, Switzerland, September 1990.

"Engineering Hydrology From a Project Standpoint," invited, Keynote Address, ASCE International Symposium on Engineering Hydrology, San Francisco, California, July 27, 1993.

Representative Project Experience:

1973 - Conducted hydraulic model study for American Falls Dam. Study considered the effect on spillway, sluice, and penstock discharge of the old submerged dam upstream, Bechtel, Inc.

1974 - Discharge measurements and hydraulic analysis of the cooling water system for Jim Bridger Power Plant, Bechtel Power Corp.

1974 - Hydraulic model study and hydraulic design of modifications for Pit 6 and Pit 7 Spillways, two 60-m high concrete dams in northern California, Pacific Gas and Electric Co., Bechtel, Inc.

1975 - Hydraulic model study and Field Performance Tests for pump intakes for Jim Bridger Power Plant in South Wyoming, Bechtel Power Corp.

1975 - Hydrology, hydraulic design, and power study for the Rio Grande-Rositas project near Santa Cruz, Bolivia. A 200-m high multi-purpose earthfill dam, Bechtel, Inc.

1975 - Hydrology and hydraulics of river diversion for the Tara Mine project, Tara, Ireland, Canadian Bechtel, Ltd.

1976 - Hydrology, hydraulic design, and spillway model study for the Boardman Cooling Water Reservoir, a 35-m high earthfill dam in north central Oregon, Bechtel, Inc.

1976 - Hydrology and spillway design for Coronado Cooling-Water Reservoir, a 40-m high dam in western Arizona, Bechtel, Inc.

1977 - Hydrology for water supply for a 10-million ton/year steel mill in Algeria, Bechtel, Inc.

1977 - Hydrology and hydraulics for Lornex Tailings Disposal Dam, an 80-m high tailings dam in south central British Columbia, Bechtel, Inc.

1977 - Hydrology and hydraulic design for two storage dams on the Samarco Iron Ore Project in Brazil, Bechtel, Inc.

1977 - Hydrology for freshwater supply for a large nickel development on Gag Island, a small nearly uninhabited island in Indonesia, Bechtel, Inc.

1978 - Hydrology, yield study, and hydraulic analysis for the Setif project in eastern Algeria; a large irrigation project involving four new dams up to 100-m in height, and a transmountain pumping scheme, Bechtel, Inc.

1978 - Hydrology and yield study for Zayante project near Santa Cruz, California, a 100-m high earth and rock fill dam for municipal water supply, Bechtel, Inc.

1978 - Hydrology for a large uranium project in Australia involving seven small dams to trap and hold possibly contaminated runoff and mill tailings, Bechtel, Inc.

1978 - Hydrology for Black Mountain Tailings Dam, a 60-m high dam, in western South Africa, Bechtel, Inc.

1979 - Analysis of Record Floods in Nevada, city of North Las Vegas.

1980 - Hydraulic Engineering Review of a Diversion and Closure Scheme for the Yacyreta Project between Argentina and Paraguay, Morrison Knudsen Co.

1980 - Development of emergency operation plans for dams on the Lewis River which might be subject to mudflows generated by volcanic eruptions. Pacific Power and Light Company.

1980 - Pre-feasibility hydrologic study for three major dam sites in the Catatumbo River Basin in Columbia, South America, Cajiao and Gomez Engineers.

1981 - Hydraulic engineering for Sultan River Hydroelectric Project in Washington, Bechtel, Inc.

1982 - Hydrology and hydraulic engineering for the OK Tedi Hydroelectric Project in New Guinea, Bechtel, Inc.

1983 - Hydrology for water supply, flood analysis, instream flow needs for Salmon spawning, Quartz Hill Molybdenum project near Ketchikan, Alaska, Bechtel Civil & Minerals, Inc.

1983 - Flood analysis for Hope Creek Nuclear Project including analysis of wave effects, Bechtel Power Corporation.

1984 - Hydrologic analysis for several small hydroelectric developments including Sheldon Springs in New York and Lake Redding in California, Bechtel, Inc.

1984 - Hydrologic analysis and site characterization for several waste-isolation projects including the FUSRAP sites at Niagara Falls, New York and Weldon Springs, Missouri, Bechtel National, Inc.

1983 - 1985 Hydrologic and hydraulic calculations for dam safety studies for several dams in Oregon, Washington, California, and Montana, Bechtel, Inc.

1985 - Hydrologic analysis and water balance study for Stringfellow Acid Pits, Bechtel National, Inc.

1986 - Hydrology, Sediment and Hydraulic Design, Karnali Hydroelectric Project, Karnali River, Nepal, Bechtel Civil, Inc.

1987 - Hydraulic Design, Hydrologic Studies, Dinkey Creek Hydroelectric Project, King's River Drainage, California, Bechtel Civil, Inc.

1988 - Hydraulic Design, Hydrologic Studies, Sediment Studies for Cowlitz Falls Hydroelectric Project on the Cowlitz River in Washington, Bechtel Civil, Inc.

1989 - Investigation of severe cavitation problems with the circulating water pumps, Shoubrah El-Kheima Power Project, Cairo, Egypt, Overseas Bechtel, Inc.

1990 - Study of methods for removal of sediment from Rock Creek and Cresta Reservoirs for Pacific Gas and Electric Company, Bechtel Corp.

1990 - Spillway Design for Lower Larona River Dams, Larona River Hydroelectric Project, Sulawesi, Indonesia, Bechtel Civil Company.

1991 - Design of Bank Protection and Drop Structures for a 19-mile Reach of the Whitewater River near Indio, California.

1992 - Operation Studies for the Grayrocks Reservoir on the Laramie River in Wyoming

1993 - Design of New Floor Blocks for Pit 6 Dam to Resist Failure by Vibration.

RESUME OF

TERRY L. HAMPTON

Terry L. Hampton, P.E.

Manager of Water Resources - Hydrology & Hydraulics

Employment
Mead & Hunt, Inc.
Wisconsin Department of Natural Resources

Education
B.S. in Civil Engineering - University of Wisconsin - 1971

Registration
Professional Engineer in Wisconsin and Michigan

Memberships
American Society of Civil Engineers
Wisconsin & National Society of Professional Engineers
Wisconsin Association of Consulting Engineers, Past President
American Consulting Engineers Council
Association of State Dam Safety Officials, Advisory Committee
United States Committee on Large Dams
Hydro Users Group PMP Study, Executive Committee
EPRI PMP Study, Review Committee
FERC-EPRI PMF Standards Review Committee
Committee of Hydroelectric Dam Owners

Experience
Over 20 years of experience in the design, analysis, and development of dams and hydroelectric facilities, field inspections of structures, and hydraulic and hydrologic analyses for feasibility studies and dam safety assessments. Developed and renovated dams and dikes, built new hydroelectric facilities ranging from 400 to 14,000 kW, and reactivated decommissioned facilities. Conducted economic evaluations and sensitivity analyses, directed engineering design, coordinated financing, and acted as owners' agent to approve pay requests for proposed hydroelectric developments.

Involved in developing new hydrologic approaches to determining the Probable Maximum Flood (PMF), with involvement in satellite imagery and geographic information system projects, loss function investigations, and overland flow modeling. In the past five years, managed over 30 PMF studies, several of which have been completed in 1993 using the Wisconsin-Michigan PMP study data and complying with Chapter VIII of the October 1993 FERC *Engineering Guidelines*.

Approved by the FERC as an Independent Inspector. Conducted more than 60 FERC Part 12 dam safety inspections and many inspections of non-federal dams. FERC-regulated dam inspections included physical condition field inspections, concrete and earthen structural stability evaluations, and spillway adequacy studies. Developed watershed models to determine the PMF and 100-year flood hydrographs using HEC-1 and HMR52 computer programs. Performed dam breach and flood hydrograph routings using HEC-1, HEC-2, NWS DAMBRK, and HEC Gradually Varied Unsteady Flow Profiles programs.

Terry L. Hampton, P.E.

Manager of Water Resources - Hydrology & Hydraulics

Hydrology--Serves on the Review Committee of the Electric Power Research Institute (EPRI) Probable Maximum Precipitation (PMP) Study. Has managed many PMF and PMP studies at projects in the upper Midwest regulated by the Federal Energy Regulatory Commission (FERC). Hydrologic investigations included rainfall/runoff analyses, hydrograph determinations, PMF and flood flow-frequency analyses, urban hydrology, unsteady and steady-state water surface profiles, and dam break analyses. Has conducted flood routings on major rivers, including the Mississippi, Illinois, and Wisconsin and on many other rivers and watersheds in Wisconsin, Minnesota, Illinois, Michigan, Virginia, West Virginia, Alabama, and Missouri. Has developed many watershed models to determine the PMF and other design floods using the U.S. Army Corps of Engineers (COE) HEC-1, and HMR52 models. Is a member of the Wisconsin-Michigan Hydro Users Group, which is currently sponsoring a re-evaluation of the PMP in the region. Recently attended the Berkeley Springs FEMA workshop on the PMP and PMF and was invited to make a presentation there on the project owners' perspective of the PMF. Is currently helping to develop new approaches to determining the PMF, such as using geographic information and image processing systems.

1993 and 1994 PMF Studies--Has actively participated in recent programs to update techniques and data sources used to calculate the Probable Maximum Flood (PMF), including serving on the review committees for the Wisconsin-Michigan Probable Maximum Precipitation (PMP) study and the FERC *Engineering Guidelines* for PMF determination.

In March 1994, finished preparing four example studies for FERC and the Electric Power Research Institute (EPRI), to be included as an addendum to Chapter VIII of the FERC October 1993 *Engineering Guidelines*. These examples are adaptations of actual studies, expanded and modified to show a range of traditional and innovative approaches to PMF determinations. Each study was prepared with FERC and EPRI staff review. Will also be traveling throughout the United States teaching a seminar on application of the guidelines. During and after development of the guidelines, performed several PMF analyses that complied with, or anticipated, the recommended procedures.

Hydraulics--Has performed many dam breach studies and flood hydrograph routings using the COE computer models HEC-1 and HEC-2 and spillway adequacy studies using the National Weather Service (NWS) DAMBRK model. Has also taught short courses in NWS DAMBRK computer modeling, which were sponsored by the Association of State Dam Safety Officials and the UW-Madison Extension. Has prepared dam break inundation maps and Emergency Action Plans for many FERC-regulated and non-federal dams. Projects involved floodplain modeling for flood insurance studies; floodplain ordinance amendments using the modeling; efficiency testing; spillway design; gate hydraulics; and turbine intake and outlet hydraulics--including trashracks, flumes, and draft tubes.

Design--Has analyzed and designed a diverse range of water resources developments, including shorewalls, erosion protection measures, fish barriers and ladders, dams, dikes, and channel improvements. Has directed engineering design and served as project manager/engineer for multimillion-dollar hydroelectric development programs in Wisconsin and Michigan and developed operating schemes for the feasibility and licensing phases, as needed. Applied principals of littoral drift and wave reflections and refraction in design of breakwater areas. Developed and managed construction of a fish barrier, trap-and-harvest facility. Managed the development of a hydroelectric plant using a siphon penstock (with a lift near the theoretical practical limit of 25 feet) for water conveyance. While with the WDNR, developed the concept and design of an experimental earth dike using geotextiles for support, oversaw construction, and monitored the facility.

Turbine Efficiency Testing--Has performed and managed many efficiency tests, which included performance testing on new and existing turbines taking flow measurements using either fiber optic or ''Price AA'' current meters. Calculated efficiencies of units as well as discharge tables to monitor flow through the plants. Conducted many feasibility studies based on the efficiency tests, taking hydrologic and hydraulic aspects into account as well as economics and resource utilization.

Preliminary Design--Conducted flow duration and power analyses for 16 hydro feasibility studies.

Geotechnical Work--Has conducted and managed projects involving dike stability analysis, dike repair and foundation evaluation.

Structural Repair--Has conducted and managed projects involving concrete structure assessments and repair, including structural stability and stress analyses, rehabilitation of abutment walls, and other concrete structure renovations.

Dam Licensing--Has managed the preparation of seven post-ECPA (Electric Consumers Protection Act) applications to the FERC, including one original and four relicensing applications, one amendment, and one additional information report. Projects involved negotiation with resource agencies for special studies, dispute resolution with the FERC, subconsultant tracking, and preparing draft and final reports. Applications included studies of historical and archaeological resources; low flows; water use and quality--including sediment sampling and analysis; fish, wildlife, and botanical resources; threatened and endangered species; fish populations, entrainment, and mortality; recreational resources; and land management and aesthetics.

Preliminary Design--Conducted flow duration and power analyses for 16 hydro feasibility studies.

Geotechnical Work--Has conducted and managed projects involving dike stability analysis, dike repair and foundation evaluation.

Structural Repair--Has conducted and managed projects involving concrete structure assessments and repair, including structural stability and stress analyses, rehabilitation of abutment walls, and other concrete structure renovations.

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Credentials

Presenter at HydroVision '94 in Phoenix, August 1994. *Rainfall-RunoffModeling--Extrapolator or Predictor?*

American Society of Civil Engineers Conference, *Determination of PMF Ongoing Research*, Milwaukee, 1991.

Short courses in *NWS DAMBRK Computer Modeling*, Asheville, North Carolina, and Marlborough, Massachusetts, sponsored by Association of State Dam Safety Officials, 1990; in Madison, Wisconsin, sponsored by the UW-Madison Extension, 1991.

Harvest Facility Protects & Improves Fishery and Collapsible Gate Improves Recreation on the Wisconsin River, presented at Waterpower, International Conference on Hydropower, Denver, 1991.

Speaker at FEMA workshop on PMP and PMF, Berkeley Springs, West Virginia, 1990.

Siphon Penstock Operations and Design of Siphon Penstocks for Low-Head Hydros, presented at Waterpower, International Conference on Hydropower, Portland, 1987.

Fishery Issues Slow Boardman River Redevelopment, presented at Waterpower, International Conference on Hydropower, Las Vegas, 1985.

Inspection and Evaluation of Safety of Non-Federal Dams, Instructor, Baltimore, Cincinnati, Kansas City, Sacramento, Albuquerque, Atlanta, and Fort Mitchell, Kentucky, 1980-81.

Second Tri-State Conference on Dam Safety, keynote address: *Inspection of Dams*, Atlanta, 1981.

Bridge Hydraulics, WDNR staff training seminar, instructor, 1975.

Floodway Workshop and *Floodplain Hydraulics*, UW-Madison Extension seminars, instructor, 1979.

Drainage Basin Flood Hydrology, HEC-1 and HEC-2 seminar, UW-Madison Extension, instructor, 1980.

Engineering Foundation Conference, presentation on *Floodproofing*, 1977.

Seminars/Conferences Attended

Designing and Conducting Studies Using Instream Flow Incremental Methodology, National Ecology Center, U.S. Fish & Wildlife Service, Fort Collins, Colorado, 1988.

Special Topics in Hydraulics–Flooding Routing, semester seminar at UW-Madison, 1984.

National Weather Service seminar on *DWOPER and DAMBRK Hydraulic Computer Programs*, NWS, Minneapolis, 1981.

ASCE Conference on *Mathematical and Physical Models in Hydraulic Engineering*, 1978.

U.S. Geological Survey course on *Statistical Hydrology* (Bulletin 17), 1976.

Floodplain Hydrology (HEC-1) and Hydraulics (HEC-2), Department of the Army, 1974.

Other

ASDSO Conference Planning Committee on Dam Safety, 1990 and 1991

Upper Mississippi River Basin Commission Task Force on Flood Plain Management, while with WDNR, 1976-78.

Great River Environmental Action Team, Mississippi River Floodplain Management and Dredging Requirement Work Groups, while with WDNR, 1976-77.

Related Projects

Wisconsin River Probable Maximum Flood Study (PMF).

This study, now being completed for five owners of fourteen dams on the Wisconsin River and tributaries, uses the following elements of the October 1993, Chapter VIII PMF Guideline:

- 23 subbasins, totalling 7,000 square miles
- regional study of unit hydrograph parameters
- dynamic (UNET) routing of flood flows through river valley and floodplains
- optimization of Probable Maximum Storm for each project site
- detailed (STATSGO) method of estimating losses
- use of Wisconsin-Michigan PMP and WMPMS model
- snowpack and snowmelt analysis

Au Sable River PMF Study.

This study, completed in 1992, represented a significant deviation from accepted procedures for estimating loss functions. The unique approach used a detailed model study of a small subbasin of the 1800-square mile Au Sable River drainage to investigate the spatially distributed losses occurring in very sandy soils with poorly developed drainage. The runoff model used (the Agricultural Research Service's KINEMAT model) had never been used before for a FERC-regulated project. The study supported the use of very high equivalent average basin loss rates, reducing the FERC-accepted estimate of the PMF by approximately 50 percent. The study was accepted by the FERC in September, 1993.

Apple River PMF Study.

This study, currently in progress, is a detailed model analysis of an ungaged, 300-square mile drainage in western Wisconsin. The Apple River basin was previously estimated, by the U. S. Geological Survey and others, to contain a drainage area greater than 500 square miles. A first step in the present Apple River study was to develop a detailed map of contributing and non-contributing areas as indicated by standard 7.5-minute quadrangle maps. This analysis has shown that contributing areas are much smaller than previously estimated. A supplemental pilot study uses aerial photography to construct a detailed digital elevation model of a portion of the basin; this model will then be used to develop a more precise description of drainage patterns. In addition, the Apple River study uses the Corps of Engineers UNET model to route flows from small subbasins to the basin outlet. This approach has at least two advantages: first, the dynamic routing model accounts for the extensive floodplain and wetland storage; and second, the UNET models account for the great majority of the basin timing with physically-based hydraulic equations instead of empirically derived unit hydrographs. This study is still in progress and will be completed in March 1993.

Related Projects

Cooke Dam
AuSable River, Michigan.
Managed a structural stability and stress analysis of an upstream slab of the spillway. The goal was to better define material properties of the concrete structure that might have directly affected results of an earlier ultimate strength analysis, which had indicated that the concrete structure was overstressed in bending and shear. Involved an extensive literature search and laboratory testing of the concrete and reinforcement steel. The study concluded that the concrete and steel reinforcement strengths had been understated in the initial analysis. The integrity of the structure was found to be adequate, and no remedial action was needed.

Prairie du Sac Dam
Wisconsin River
Managed a project to assess the condition of the pile foundation under the buttresses and piers of the spillway to determine if the piles were sufficient to withstand the vertical and lateral forces that could cause spillway instability. Excavations were made and cores were taken of various pile and wood species in each section. Determined stability of the powerhouse, lock, and spillway under four different loading conditions based on dam segment foundation condition. The structures were found to be stable under the conditions analyzed and no remedial action was recommended. Made recommendations for regular monitoring of the horizontal and vertical control and the phreatic surface under the spillway and aprons.

Rhinelander Paper Company
Dike repair; seepage problem, minor material removal.

Sabin Dam
Traverse City, Michigan
Dike repair; unstable section improvement.

Alcona, Loud, Five Channels--Cooke & Foote Hydroelectric Plants
Au Sable River, Michigan
FERC-approved inspector for the five projects. As project manager, supervised independent consultant's inspections on these hydro projects, which included physical condition inspections, stability analyses, and spillway adequacy studies.

La Crosse River Study
La Crosse County, Wisconsin
Performed hydraulic and hydrologic analyses to delineate floodplain. Studied the effect of highway, bridge, and levee construction and channel realignment.

Related Projects

LCO Hydro
Winter, Wisconsin
As project manager, managed the feasibility studies and environmental assessments and the design and construction engineering for a hydroelectric plant.

Brown Bridge Dam
Boardman River, Michigan
Rehabilitation of abutment wall.

Boardman & Sabin Dams
Traverse City, Michigan
Concrete structure renovations.

Rock River Hydro Plant Relicensing
Janesville and Beloit, Wisconsin, and Rockton, Illinois
Principal-in-charge for the post-ECPA relicensing of three minor hydro projects with licenses expiring December 1993. The project included special environmental studies, preparing Exhibit E of the license application, and a minimum-flow study to determine effects of different flow regimes on stream habitat.

Rothschild Hydro Project Relicensing
Wausau, Wisconsin
Principal-in-charge managing activities for post-ECPA relicensing of a major (less than 5 MW) project on the Wisconsin River in Marathon County. Activities included preparing the Initial Consultation Package of the draft and final new license applications. Also involved in preparing Exhibit E and technical exhibits, working through a three-stage agency consultation process, and developing special study scopes. Participated in the FERC dispute resolution process regarding fish entrainment/mortality and in the negotiation of a joint study scope for investigations now being conducted with two other river users.

Mead & Hunt PMF Studies, 1993-1994

Name of Basin	Drainage Area (square miles)	Number of Subbasins	Other model features	Status
Peshtigo River (Wisconsin)	530	2	UNET routing through linear reservoir; STATSGO loss rate estimates; unit hydrograph transfer from adjacent basin; unlimiting snowpack	Submitted
Flambeau River (Wisconsin)	1,880	4	Mixed gaged/ungaged unit hydrograph analysis; UNET routing; STATSGO loss rates; 100-year snowpack	In progress
Sturgeon River (Michigan)	305	1	Gaged basin unit hydrograph analysis; STATSGO loss rate; unlimiting snowpack	Submitted
Oconto River (Wisconsin)	739	1	Gaged basin unit hydrograph analysis; STATSGO loss rate; unlimiting snowpack	Submitted
Black River (Wisconsin)	1,278	1	Gaged basin unit hydrograph analysis; STATSGO loss rate; unlimiting snowpack	Submitted
Grand River (Michigan)	1,750	6	Mixed gaged/ungaged hydrograph analysis; one urban subbasin; UNET routing; STATSGO losses; 100-year snowpack	In progress
Dead River (Michigan)	150	8	Ungaged basin with SCS unit hydrograph analyses; STATSGO losses; UNET routing through linear reservoir system; unlimiting snowpack	Submitted
Boardman River (Michigan)	180	3	Partially gaged basin; STATSGO loss rates; UNET routing; unlimiting snowpack	In progress

RESUME OF

DR. ARTHUR C. MILLER

Arthur C. Miller
Professor of Civil Engineering
The Pennsylvania State University

Office Address

220 Sackett Building
University Park, PA 16802
(814) 865-1521

Home Address

1825 Woodledge Drive
State College, PA 16803
(814) 867-8660

Personnel Information:

Birth Date: September 1, 1942

Professional Registration:

1. Registered Professional Engineer
   Pennsylvania, PE-022486

2. Registered Professional Land Surveyor
   Pennsylvania, SU-000236-A

Education:

B.S.	University of Massachusetts Civil Engineering	1965
M.S.	Colorado State University Hydraulics and Water Resources	1967
Ph.D.	Colorado State University Hydraulics	1972

Professional History:

Professor	Penn State University	1985 - present
Visiting Scientist	U.S. Army Corps of Engineers Waterways Experiment Station	1984
Associate Professor	Penn State Univ.	1978 - 1985
Assistant Professor	Penn State Univ.	1972 - 1978
Instructor	Colorado State Univ.	1969 - 1972
NSF Trainee	Colorado State Univ.	1965 - 1969

Honors and Awards:

1. National Science Foundation Traineeship for Graduate Studies (1965-1969)

2. American Society of Civil Engineers Award as an Outstanding Faculty Advisor to the Student Chapter of ASCE for 1975-1976, 1976-77, 1991-92, 1992-93.

3. Penn State College of Engineering Outstanding Teaching Award (1989)

4. American Society of Civil Engineers Award for Best Technical Note Given to the Task Committee Members for writing the paper "Hydrologic Design Criteria for Dams Safety."

5. Faculty Service Award of Conference and Institutes Division, National Education Association for Outstanding Contributions to Conferences and Institutes or Residential Continuing Education Programs, 1992.

Professional Activities:

1. American Water Resources Association
2. American Geophysical Union
3. Sigma Xi
4. Water Pollution Control Federation
5. State Coordinating Committee for the State Rural Clean Water Program (1980-81)
6. Pennsylvania Board of Professional Registration
   Developed problems and Graded Transportation, Environmental, and Water Resources for the Professional Engineering Examination in Pennsylvania (1978-1980)
7. American Society of Civil Engineers

University:

(1) Faculty Advisor Student Chapter 1973-1978
(2) Faculty Advisor Student Chapter 1988-present

Local Section: (Central Pennsylvania Section of ASCE)

(1)	Board of Directors	(1975-1979)
(2)	Newsletter Editor	(1979)
(3)	Vice President	(1980)
(4)	President Elect	(1981)
(5)	President	(1982)

7. American Society of Civil Engineers (con't)

National:

(1) Task Committee on Outlet Controls for Stormwater Detention Facilities, Member of Hydraulic Structures Task Committee 1983-1985

(2) Task Committee on Spillway Design Criteria for the Hydrologic Safety of Dams - final report submitted 1988

(3) Task Committee on Risk Base Analysis for Decision Making - Inactive committee (1985-88)

(4) Task Committee on Bridge Scour - Committee September 1990-present.

8. Energy Power Research Institute (EPRI)

(1) Chairman Task Committee on Standards for PMF for Dam Safety Analysis (1991-present)

<u>Teaching Assignments:</u>

Courses Taught:

Elementary Surveying
Route Surveying
Highway Engineering
Geometric Design
Statics
Dynamics
Strength of Materials
Wastewater Management
Fluid Mechanics
Applied Hydraulics
Hydrology
Advanced Hydrology
Stormwater Management
Open Channel Hydraulics
Unsteady Flow in Open Channels
Hydraulic Structures
Erosion and Sedimentation

Research Activities:

Research Projects Completed:

1974-76 -- Atmospheric Inputs to the Upper Great Lakes by Dry Deposition Processes; EPA - $78,000/15 months; Co-Principal Investigators -- W.J. Moroz, R. Kabel, A.C. Miller.

1974-76 -- Winter Ice Production for Power Plant Waste Heat Disposal in Summer in Mid and High Latitudes; Pennsylvania Power and Light Company; $36,500/18 months; Principal Investigator.

1974-76 -- Quantitative and Qualitative Implications of Urban Storm Runoff Abatement Measures; Office of Water Research Technology and City of Philadelphia; $72,000/24 months; Co-Principal Investigators -- G. Aron, D.A. Long, A.C. Miller.

1975-75 -- Resistance to Flow: College of Engineering Central Fund for Research; $1500/12 months; Principal Investigator -- A.C. Miller.

1975-77 -- Implementing a Procedure to Predict Sediment Flow From Highway Construction Sites; PennDOT; $118,802/36 months; Principal Investigator -- A.C. Miller

1977-77 -- Operational Manual for Systematic River Basin Flood Plain Study; U.S. Army Corps of Engineers; Vicksburg, MS., $7,600/summer; Co-Principal Investigators -- G. Aron, A.C. Miller.

1977-78 -- Implementing a Procedure to Predict Sediment Flow From Highway Construction Sites; PennDOT; $20,000/6 months; Increase in Scope of Ongoing Study; Principal Investigator -- A.C. Miller.

1977-78 -- Hydraulic Model Study of Loyalsock Creek; $62,951/18 months; Principal Investigator -- A.C. Miller.

1979-80 -- Gross Allotment and Source Allocation Policy and Procedures; PennDER; $30,481/18 months; Co-Principal Investigators -- R.A. Chadderton and A.C. Miller.

1980-81 -- Improving Curve Number Based Runoff Prediction by Revamping Antecedent Moisture Classification; Agricultural Research Service; $9,600/12 months; Principal Investigator -- A.C. Miller.

1980-81 -- Effect of Best Management Practices on Nonpoint Sources in the Conestoga River Above Lancaster, PA, PennDER and EPA; $58,000/6 months; Principal Investigator -- A.C. Miller.

1980-81 -- Loyalsock Movie a Hydraulic Model Study; Federal Highway Administration; $6,000/12 months; Principal Investigator -- A.C. Miller.

1980-83 -- Flow Control Structures for Highway Crossings; Surtron Corporation - A Hydraulic Model Study of Spur Dikes Configurations; $134,000/36 months; Principal Investigator -- A.C. Miller.

1981-82 -- Antecedent Moisture Content - An Improvement on Soil Conservation Services Runoff Procedure; Agricultural Service; $11,600/12 months; Principal Investigator -- A.C. Miller.

1983-84 -- Floodways Produced from Breached Dams; U.S. Army Corps of Engineers Waterways Experiment Station; Vicksburg, MS; $42,937/9 months; Principal Investigator -- A.C. Miller.

1984-86 -- Groundwater Modeling an Analysis of Existing Two Dimensional Models; Agricultural Research Service; $20,000 for Student Support for two years; Co-Principal Investigators -- W. Gburek and A.C. Miller

1984-85 -- Military Hydrology; U.S. Army Corps of Engineers Waterways Experiment Station; Vicksburg, MS; $72,000/12 months; Principal Investigator -- A.C. Miller.

1984-86 -- Microcomputer Program for Culvert Design and Analysis (HY-8); Federal Highway Administration, U.S. Department of Transportation; $82,382/24 months; Co-Principal Investigators G. Aron and A.C. Miller

1986-87 -- Data Generation and Analysis; Institute for Land and Water Resources; $17,639/12 months; Principal Investigator -- A.C. Miller.

1987-88 -- Research Fellowship Grant for Arthur C. Parola; U.S. Department of Transportation/Federal Highway Administration; $13,698/15 months; Principal Investigator - A.C. Miller

1987-88 -- Microcomputer Program for Culvert Design and Analysis (HY-8) - Hydrology and Energy Dissipators; Federal Highway Administration, U.S. Department of Transportation; $55,000/18 months; Principal Investigator -- A.C. Miller.

1987-89 -- Stability of Riprap Streambed Protection Around the Base of Bridge Piers; Federal Highway Administration, U.S. Department of Transforation; $25,000/18 months; Co-Principal Investigators -- A.C. Parola and A.C. Miller.

1987-90 -- Surface Hydrology, Sediment Transport Dynamics and Remote Sensing of Disturbed Watersheds in a Humid, Temperate Region; Department of Energy; $211,843/36 months; Co-Principal Investigators -- T. Gardner and A.C. Miller.

1988-88 -- Development of Video for Underwater Bridge Safety Inspection. PennDOT; $29,933/6 months; Principal Investigator - A.C. Miller

1988-89 -- Development of Methodology for Flood Plain Delineation; Federal Emergency Management Agency; U.S. Army Corps of Engineers; $12,991/6 months; Principal Investigator -- A.C. Miller.

1988-89 -- A Mathematical Procedure for Interpolating Cross Sections for Gradually Varied Flow Computer Models; U.S. Army Corps of Engineers Hydrologic Engineering Center; Davis, CA; Principal Investigator -- A.C. Miller.

1988-90 -- Modeling Fluid Dynamics and Dissolved Gas Concentrations in Flowing Water; U.S. Fish and Wildlife Service, Wellsboro National Fish Laboratory; $25,000/18 months; Co-Principal Investigator -- D.A. Shellman and A.C. Miller.

1988-90 -- Hydraulic Design Techniques for Bridges, Culverts, and Low Water Crossings on Low Volume Roads; RTAP Project #67, PennDOT; $96,000/18 months; Principal Investigator -- A.C. Miller.

1989-90 -- Model Study of Wildcat and Tinklepaugh Creeks, Supercritical Flow Transitions; Pennsylvania Department of General Services and Department of Environmental Resources; $49,000/12 months; Principal Investigator -- A.C. Miller.

1990-92 -- Prediction of Bridge Scour Due to Piers; PennDOT; $149,000/24 months; Principal Investigator -- A.C. Miller.

1990-95 -- The Global Water Cycle; NASA; $6 Million/5 years; Co-Principal Investigator 10% of Salary for first two years and then 20% for the next three years. Purpose is to Develop a Center of Excellence for Hydrologic Research.

Publications:

Miller, A.C., Richardson, E.V., "Diffusion and Dispersion in Rough Flow," ASCE Journal of Hydraulics, pp. 159-171, January 1974.

Miller, A.C., "Highway Embankments," International Correspondence School Text, Scranton, PA, 78 pages, 1974.

Miller, A.C., "Highway Drainage," International Correspondence School Text, Scranton, PA, 97 pages, 1975.

Miller, A.C., "Culvert Hydraulics," International Correspondence School Text, PA, 90 pages, 1977.

Aron, G., Miller, A.C., "Adaptation of Peak Flows and Design Hydrographs from Gaged to Nearby Gaged Watersheds," American Water Resources Bulletin, 1977.

Aron, G., Miller, A.C., Lakotos, D.A., "An Irrigation Formula Based on SCS Curve Number," ASCE Journal of Irrigation and Drainage, Vol. 103. No. IR4, December 1977.

Miller, A.C., Daily, D.A., "Rainfall Factors that Affect Erosion," Transportation Research Board Record 642, 1978.

Miller, A.C., Veon, W.J., "Soil Properties that Affect Erosion," Transportation Research Board Record 642, 1978.

Miller, A.C., Chadderton, R.A., Discussion, "Water Rights, Eminent Domain, and Public Trust," American Water Resource Bulletin, 1978.

Miller, A.C., Chadderton, R.A., and Brown, S., "Loyalsock Creek Model Study," Verification of Mathematical and Physical Models in Hydraulic Engineering, Proceedings of the 26th Annual Hydraulics Division Specialty Conference, University of Maryland, August 1978.

Miller, A.C., Chadderton, R.A., and Brown, S., "Loyalsock Creek Model Study - Phase II," Conservation and Utilization of Water and Energy Resources, Proceedings of the 27th Annual Hydraulics Division Specialty Conference, San Francisco, California, August 1979.

Chadderton, R.A. and Miller, A.C., "Friction Slope Models for M2 Profiles," American Water Resource Bulletin, April 1980.

Chadderton, R.A. and Miller, A.C., Discussion, "Analysis of Uncertainty in Flood Plain Mapping," American Water Resource Bulletin, May 1980.

Chadderton, R.A. and Miller, A.C., "An Analysis of Total Maximum Daily Load and Waste Load Allocation Procedures," Proceedings of the 16th American Water Resources Conference, Minneapolis, Minnesota, October 1980.

Chadderton, R.A., Miller, A.C., and McDonnell, A.J., "Analysis of Total Maximum Daily Load and Waste Load Allocation Procedures," American Water Resource Association, Vol. 17, No. 5, October 1981.

Miller, A.C., Veon, W.J., and Chadderton, R.A., "Comparison of Prediction Methods for Soil Erosion From Highway Construction Sites, "Transportation Research Board Record 896, Hydrology and Hydraulics: Water, Noise, and Air Quality, 1982.

Miller, A.C., "Hydraulic Effects on Streams," Proceedings Stormwater Detention Facilities Planning, Design, Operation and Maintenance," Henniker, NH, pp. 94-104, American Society of Civil Engineers, August 1982.

Chadderton, R.A., Miller, A.C., and McDonnell, A.J., "Uncertainty Analysis of Dissolved Oxygen Model," ASCE Journal of Environmental Engineering Division, Vol. 108, No. 5, October, 1982 pp 1003-1014.

Khanbilvardi, R.M., Rogowski, A.S., and Miller, A.C., "Modeling Upland Erosion," American Water Resources Bulletin, January 1983, pp 29-35.

Miller, A.C., Kerr, S.K., and Speader, D.J., "Calibration of Snyder Coefficients for Pennsylvania," American Water Resources Bulletin, August 1983, pp 625-630.

Khanbilvardi, R.M., Rogowski, A.S., and Miller, A.C., "Predicting Erosion and Deposition on Strip-mined and Reclaimed Area," American Water Resources Bulletin, August 1983, pp 585-593.

Miller, A.C., Pena, J.A., Urbanski, J.A., Kerr, S.N., "Ice Pond Cooling System for Power Plants," ASCE Journal of Energy Engineering Division, September 1983, pp 201-206.

Miller, A.C., Gilbert, D.K., Nesbitt, J.B., and Kerr, S.N., "Sanitary Sewer Design," Proceedings of 1st National Conference on Microcomputers in Civil Engineering, University of Central Florida, November 1983.

Miller, A.C., Kerr, S.N., and Sartor, J., "Physical Modeling of Spurs," River Meandering, pp 996-1007, American Society of Civil Engineers, 1984.

Miller, A.C., Kerr, S.N., and Sartor, J., "Channel Stabilization Using Multiple Spurs," Water Resource Development: Proceedings, ASCE Hydraulics Specialty Conference, pp 291-295, August 1984.

Brulo, A.T., Kibler, D.F., and Miller, A.C., "Evaluation of Two-Stage Outlet Hydraulics," Water Resource Development: Proceedings ASCE Hydraulics Specialty Conference, pp 345-350, August 1984.

Miller, A.C., Kerr, S.N., James, W., Jourdan, M., "MILHY-A Microcomputer Model for Hydrologic and Hydraulic Analysis of Streamflow Forecasting," Proceedings of 2nd National Conference on Microcomputers in Civil Engineering, University of Central Florida, October 1984.

Urbanas, B.R., et. al (Task Committee Members) "Stormwater Detention Outlet Control Structures." Task committee on the Design of Outlet Control Structures, separate publication, pp. 1-35, American Society of Civil Engineers (1985).

Davis, J.F., Miller, A.C., "A Microcomputer Program for Sediment Transport in Open Channels," Proceedings ASCE Hydraulics Specialty Conference, August 1985.

Miller, A.C., "Criteria for Spillway Design Floods," Proceedings of the American Society of Civil Engineers Hydraulics Division National Conference on Hydraulic Engineering and Engineering Hydrology Symposium, Williamsburg, VA, pp 514-519 (August 1987).

Traver, R.G., Miller, A.C. "Modeling Unsteady One Dimensional Open Channel Flow Using the Slope Friction Form of the Saint-Venant Equations," Proceedings of the American Society of Civil Engineers Hydraulics Division National Conference on Hydraulic Engineering and Engineering Hydrology Symposium, Williamsburg, VA pp 770-775 (August 1987).

Task Committee of ASCE. "Design Criteria for the Hydrologic Safety of Dams." 91p separate printing 1989.

Baron, E., et. al. Global Water Cycle. Text book Written by a consortium of authors funded under the EOS program by NASA at Penn State University (1990).

Aron, G. and Miller, A., "Stability Problems in Stream Water Profile Computations," ASCE Hydraulics Specialty Conference, Baltimore, MD, (August 1992).

Nale, D. and Miller, A., "Digital Elevation Models Utilized in Flood Plain Hydraulics Using HEC-2," Surveying and Mapping, U.S. Army Corps of Engineers, (September 1992).

Johnson, D., and Miller, A.C., "Bridge Scour Prediction Methods Applicable to Streams in Pennsylvania, ASCE Hydraulics Specialty Conference, San Francisco, CA, (July 1993).

Johnson, D., and Miller, A.C., "Formulation of a Hydrologic Model, For Use With Remotely Sensed Data," ASCE Engineering Hydrology Symposium Proceedings, San Francisco, CA, (July 1993).

Traver, R.G., Miller, A.C., "Open Channel Cross Section Interpolation by Geometric Property," American Water Resource Association. Accepted for Publication. 1993.

Papers Presented at Technical and Professional Meetings:

Miller, A.C. "Prediction of Erosion from Construction Sites." Presented at the Annual Transportation Research Board Meeting, Washington, D.C. (January 1976).

Miller, A.C., "Capabilities, of HEC-2." Presented to the Central Pennsylvania Section of American Society of Civil Engineers, Harrisburg, PA. (April 1977).

Miller, A.C., "Sediment Erosion from Highway Construction Sites." Presented at the Federal Highway Review, Ohio State University (September 1977).

Miller, A.C., "Future Highway Research Needs in Hydraulics." Presented at the Federal Highway Research Review, Ohio State University (September 1977).

Miller, A.C., "Background Sediment Yields from Construction Sites." Presented at the Annual Transportation research Board Meeting, Washington, D.C. (January 1979).

Miller, A.C., "Loyalsock Model Study." Presented at the American Highway Association Meeting, Williamsport, PA (February 1979).

Chadderton, R.A., Miller, A.C. "An Analysis of Total Maximum Daily Load and Waste Allocation Procedures." Presented at the 16th America Water Resources Association Conference, Water Resources Issues of the Eighties, Minneapolis, MN (October 1980). Presented by R.A. Chadderton).

Miller, A.C. "Sediment and Erosion Control Aspects of Stormwater Detention." Presented at the Third Annual Symposium on Stormwater Management, Penn State University (May 1982).

Miller, A.C., "Calibration of SCS Curve Numbers." Presented at the Third Annual Symposium on Stormwater Management, Penn State University (May 1982).

Khanbilvardi, R.M., Rogowski, A.S., Miller, A.C. "Modeling Erosion and Sediment Transport in Rills." Presented at the American Society of Agricultural Engineers, Chicago, IL (December 1982). Presented by R.M. Khanbilvardi).

Miller, A.C., Wall, D.J. "Dual-Purpose Detention Facilities." Presented at the 1983 Advances in Stormwater Management Symposium at Penn State University (May 1983).

Miller, A.C., Kerr, S.N., Nesbitt, J.B. "A Microcomputer Program for the Design and Analysis of Sanitary Sewers." Water Pollution Control Federation, Annual Meeting, New Orleans, LA (October 1984).

Miller, A.C. "Application of Risk Analysis to Dam Safety." Presented at the Engineering Foundation Conference on Decision Making Using Risk Analysis, Santa Barbara, CA (November 1985).

Miller, A.C., Parola, A.C. "Culvert Analysis." Presented at the Central Pennsylvania Section of American Society of Civil Engineers Meeting, Harrisburg, PA (March 1987).

Miller, A.C., Mazid, M.M. "Using Remotely Sensed Information for Hydrologic Modeling." Presented at the 14th Annual Conference on Water Resources Planning and Management Division of American Society of Civil Engineers Meeting, Kansas City, MO (March 1987).

Miller, A.C. "Utilizing Risk Analysis for Spillway Design Criteria." Presented at the 14th Annual Conference Water Resources Planning and Management Division of American Society of Civil Engineers Meeting, Kansas City, MO (March 1987).

Miller, A.C. "Information Needs for Dam Safety Analysis and Evaluation Categories." Presented at the ASCE Hydraulics Division National Conference on Hydraulic Engineering and Engineering

Miller, A.C. "Criteria for Spillway Design Floods." American Society of Civil Engineers National Conference on Hydraulic Engineering and Engineering Hydrology Symposium, Williamsburg, VA (August 1987).

Traver, R.G., Miller, A.C. "Modeling Unsteady One-Dimensional Open Channel Flow Using the Slope Friction form of the Saint-Venant Equations." American Society of Civil Engineers National Conference on Hydraulic Engineering and Engineering Hydrology Symposium, Williamsburg, VA (August 1987).

Miller, A.C. "Limitations of One-Dimensional Models for Developing Water Surface Profiles." Presented to the Hydrologic Engineering Center, Davis, CA (September 1987).

Miller, A.C. "American Society of Civil Engineers Spillway Design Flood Report: An Academic Perspective." Fourth Annual Conference of The Association of State Dam Safety Officials, Columbus, OH (September 1987).

Miller, A.C. "Milton Bridge Failure - A Critical Review." Presented to the Pennsylvania Department of Transportation, Harrisburg, PA (October 1987).

Miller, A.C. "Hydrology-Yesterday Today and Tomorrow." Central Pennsylvania Section of the American Society of Civil Engineers, Harrisburg, PA (January 1988).

Miller, A.C. "Bridge Scour Prediction Methods Applicable to Pennsylvania Streams." FHWA Region 1 Bridge Conference, Baltimore, MD., (July 1991)

Aron, G., Miller, A.C., "Convergence Improvements in Water Surface Profile Computations, ASCE Hydraulics Specialty Conference, Baltimore, MD., (July, 1992).

Miller, A.C., "Terrestrial Hydrologic Model Using Remote Sensed Data" presented to the Hydroelectric User Group, Madison, WI., (January 1993).

Johnson, D., and Miller, A.C., "Bridge Scour Prediction Methods Applicable to Streams in Pennsylvania, ASCE Hydraulics Specialty Conference, San Francisco, CA, (July 1993).

Johnson, D., and Miller, A.C., "Formulation of a Hydrologic Model, For Use With Remotely Sensed Data," ASCE Engineering Hydrology Symposium Proceedings, San Francisco, CA, (July 1993).

Research Reports to Sponsor

Miller, A.C., Rainfall Index for Erosion. Report submitted to PennDOT (January 1977)

Moroz, W.J., Kabel, R.L, Taheri, M., Miller, A.C., Hoffman, H.J., Brtko, W.J., Custino, T.A. Atmospheric Inputs to the Upper Great Lakes by Dry Deposition Processes. Report to U.S. Environmental Protection Agency. CAES 439-76: 98 pp. (February 1976)

Aron, G., Long, D.A., Miller, A.C. Quantitative and Qualitative Implications of Urban Storm Runoff Abatement Devices. Final report to City of Philadelphia and to U.S. Department of Interior, Office of Water Resources (December 1976).

Miller, A.C., Geissler, G., Penna, J.A., Aron, G., Ballestero, T.B., Urbanski, J.A. Winter Ice Production for Power Plant Waste Heat Disposal in Summer in Mid and High Latitudes. Final report to Pennsylvania Power and Light Company, GPU Service Corporation and Philadelphia Electric Company, 180 pp. Institute for Research on Land and Water Resources (1977).

Miller, A.C., Chadderton, R.A., Brown, S. Loyalsock Model Study. Final report to PennDOT, 70 pp (May 1979).

Miller, A.C., White, E.L., Veon, W.J. A Procedure to Predict Sediment Flow from Highway Construction Sites. Design Procedures. Final report to PennDOT, 45 pp (June 1979).

Chadderton, R.,A., Miller, A.C., McDonnell, A.J. Wasteload Allocation for Pennsylvania: Evaluation of Procedures. Final report to Bureau of Water Quality Management, Pennsylvania Department of Environmental Resources, 230 pp. plus Appendices (June 1980).

Miller, A.C., White, E.L., Veon, W.J. A Procedure to Predict Sediment Flow from Highway Construction Sites. Design Procedure: Final report to PennDOT, 126 pp. (January 1981)

Miller, A.C., White, E.L., Veon, W.J. A Procedure to Predict Sediment Flow from Highway Construction Sites. Data report: Final report to PennDOT, 40 pp. (January 1981)

Miller, A.C., White, E.L., Veon, W.J. A Procedure to Predict Sediment Flow from Highway Construction Sites. Literature Review: Final report to PennDOT, 340 pp. (January 1981)

Brown, S.A., McQuivey, R.S., Miller, A.C. Flow Control Structures for Highways in River Environments. Final report to Federal Highway Administration, Washington, D.C., 154 pp (June 1981).

Miller, A.C., Dinkel, R. Improving Curve Number-Based Runoff Prediction by Revising Antecedent Moisture Classification. Final report to the Agricultural Research Service, State College, PA. 42 pp (August 1981).

Miller, A.C. Effect of Best Management Practices on Nonpoint Sources in the Conestoga River Above Lancaster, PA. final report to Pennsylvania Department of Environmental Resources and U.S. Environmental Protection Agency, Washington, D.C. 30 pp (September 1981).

Miller, A.C., Kerr, S.N., Reams, H., Pysher, T., Sartor, J. Flow Control Structures for Highway Stream Crossings. Final report to Sutron Corporation, Fairfax, VA. 150 pp plus 400 pages of Appendices (August 1983).

Khanbilvardi, M.R., Rogowski, A.S., Miller, A.C. Rill-Interrill Erosion and Deposition Model of Stripmine Hydrology. Final report to U.S. Environmental Protection Agency, Washington, D.C., 175 pp (March 1984).

Miller, A.C., Kerr, S.N., Jourdan, M., Collins, J. Student Workbook on Streamflow Forecasting, Report to U.S. Army Corps of Engineers Waterways Experiment Station, Vicksbury, MS., 324 pp (October 1984).

Miller, A.C., Cross Section Spacing for Water Surface Profile

Computations. Final report to U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS., 38 pp (October 1984).

Kerr, S.N., Miller, A.C. MILHY-Hydrology Manual. Final report to U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS., (December 1985).

Kerr, S.N., Miller, A.C., Microcomputer Program for Military Hydrology. Computer Model submitted to U.S. Army Corps of Engineers Waterways Experiment Station, Vicksburg, MS., (December 1985).

Miller, A.C., Parola, A.C. HY-8 Culvert Hydraulics. A microcomputer program to compute the flow through culverts. Computer program submitted to Federal Highway Administration, (December 1986).

Miller, A.C., Parola, A.C. Final report for Culvert Hydraulics Microcomputer Program Grant No. HY-8, Version 1.1. Submitted to Federal Highway Administration (January 1987).

Miller, A.C. Milton Bridge Failure. Final report to PennDOT 25 pp (May 1987).

Miller, A.C. Cross Section Location for Water Surface Profile Computations. Final report to U.S. Army Corps of Engineers Hydrologic Engineering Center, Davis, CA., 22 pp (November 1988).

Miller, A.C. Underwater Bridge Inspection - PennDOT Video for Training (1989).

Miller, A.C., Glenn, J., Methodology for Flood Plain Delineation. Final Report submitted to U.S. Army Corps of Engineers Philadelphia District - Plus Computer Model (1989). Gardner, T.A., Miller, A.C., Peterson, G., Surface Hydrology, Sediment Transport Dynamics and Remote Sensing of Disturbed Watersheds in Humid, Temperate Region. Final Report submitted to Department of Energy (1990).

Miller, A.C., Means, F., Hydraulic Design Techniques for Bridges, Culverts, and Low Water Crossings on Low Volume Roads. Final Report submitted to PennDOT, Harrisburg, PA, 85 pp plus computer code (1991).

Gardner, T. and Miller, A., Surface Hydrology, Sediment Transport Dynamics, and Remote Sensing of Disturbed Watersheds in a Humid Temperature Region, (Final Report to Sponsor, The U.S. Department of Energy, Agreement No. DE-FG02-87ER60594), October 1991.

Shellman, A., and Miller., Modeling Fluid Dynamics and Dissolved Oxygen Gas Concentrations in Flowing Water, (Final Report to Sponsor, The U.S. Fish Commission) 1991.

Miller, A.C., Johnson, D., Steinhart, R., Bridge Scour Prediction Methods Applicable to Streams in Pennsylvania. (Final Report Submitted to the Pennsylvania Department of Transportation, 1992.

Graduate Thesis Supervised

Student Name	Type of Degree	Date Degree Granted (Mo/Yr)	Title of Thesis
Haktanir, Tefaruk	M.S.	11/75	A comparative Study of Three Contemporary Methods about Hydraulics of Bridge Waterways
Grindall, E.J.	M.Eng.	06/76	Communication Skills Training for Engineers
Ballestero, T.	M.S.	03/77	Winter Ice Production in Mid and High Latitudes for Cooling Warm Water Effluent from Power Plant Discharges
Daily, D.A.	M.S.	08/77	Rainfall-Runoff Factors Associated with Accelerated Soil Erosion on Highway Construction Sites
Greiner, T.	M.S.	11/78	Varied Flow Friction Slope Averaging in Nonprismatic Channels
Spaeder, D.J.	M.S.	03/79	Calibration of Snyder Unit Hydrograph Coefficients for Pennsylvania Watersheds
Brown, S.A.	M.S.	08/79	Investigation of Meander Migration and Control on the Loyalsock Creek Using a Physical Model
Crouse, J.M.	M.S.	08/79	A Hydraulic and Economic Evaluation of Urban Storm Runoff Detention and Retention Devices for Small Catchments
Martin, C.D.	M.S.	11/79	Critique of the Soil Conservation Service Runoff Model
Hood, W.K.	M.S.	08/81	An Examination of Four Waste Load Allocation Strategies Applicable to River Systems
Veon, W.J.	M.S.	11/81	Estimation of Sediment Yields Resulting from Rainfall on Highway Construction Areas in Pennsylvania

Dinkel, R.S.	M.Eng.	05/82	An Evaluation of Curve Number and Antecedent Moisture Class
Sullivan, J.T.	M.S.	05/82	Effects of Impoundments on Dissolved Oxygen Concentrations in the Assabet River
Shetwan, M.A.	M.S.	11/82	Estimating Upland Erosion Using Unsteady Flow Model
Khanbilvardi, M.R.	Ph.D.	03/83	Soil Erosion from Upland Areas
Rihtnour, T.A.	MSEPC	05/83	Performance Evaluation of a Controlled Decant Dewatering System for Surface Mine Sedimentation Ponds
Pysher, T.R.	M.Eng.	05/83	A Laboratory Study on Spur Dike Design for Meandering Streams
Mazid, M.M.	M.S.	05/83	Unsteady Free Surface Flow Computations
Reams, H.E.	M.Eng.	08/83	A Laboratory Study of the Relationships Between Spur Design Parameters
Potter, S.T.	M.S.	03/84	Groundwater Modeling with Two Phase Flow
Sartor, J.P.	M.S.	12/84	Laboratory Study of Spur Dikes for Flow Control and Bank Erosion Protection Along Straight and Meandering Channels
Parola, A.C.	M.S.	05/87	Culvert Hydraulics The Development of a Computer Model
Mazid, M.M.	Ph.D.	05/87	Modeling Upland Erosion Utilizing Partial Area Techniques
Folmar, G.	M.S.	05/89	Hydraulics of Energy Dissipators:
Traver, R.G.	Ph.D.	05/89	Unsteady Flow Using the Friction Form of the Saint-Venant Equations
Potter, S.T.	Ph.D.	12/89	Groundwater Modeling in Three-Dimensional Flow

Parola, A.C.	Ph.D.	12/89	Rip-Rap Protection for Bridge Scour Around Piers
Means, F.	M.S.	05/91	Hydraulics of Bridge Waterway Openings
Ceislak, J.	M.S.	08/91	Theoretical Approach to Minor Loss Coefficients in Open Channel Flow
Shellman, A.D.	M.S.	12/91	Modeling Fluid Dynamics and Dissolved Gas Concentrations in Flowing Water
Susan Long	M.S.	05/92	Finite Element Method Applied to a One-Dimensional Model to Predict Sub and Supercritical Flow
Johnson, D	M.S.	08/92	Scour in Constricted Cross Sections Using Conveyance Tubes
Kurt Kuralik	M.S.	012/92	Bridge and Waterway Openings for Low Volume Roadways
Lewis, J.	Ph.D.	12/93	An Unsteady Sediment Transport Flow Model for Flood Forecasting on Large River Systems
Steinhart, R.	M.S.	12/93	Bridge Pier Scour-A Realistic Estimate
Ken Reuther	M.S.	12/93	
Ceislak, J.	Ph.D.	12/93	Hydrology - Partial Area Concept Utilizing the Theory of Fractal Analysis
Glenn, J.	M.Eng.	12/93	Solution of Sub and Supercritical Flow in Gradually Varied Steady Flow
Lukhele. D.	M.Eng.	06/94	Soil Moisture Infiltration Utilizing GIS Data Bases

Membership on Graduate Degree Candidates' Committees

Student Name	Type of Degree
Anderson, Paul	M.Eng.
Brulo, Albert	M.S.
Brunner, Greg	M.S.
Carrizo, Hermes	M.Eng.
Cavacas, Allan	M.S.
Craig, Robert	M.S.
Davis, John	Ph.D.
Dunn, Christopher	M.S.
Dymond, Randy	Ph.D.
Englot, Mary	M.S.
Engman, Edwin	Ph.D.
Ericksen, Mark	Ph.D. (Geology)
Etzel, Ronald	M.S.
Fredric, Pirkle	Ph.D. (Geology)
Garvey, David	M.S.
Gotzmer, Jerrold	M.S.
Gustenhoven, Carl	M.S.
Haktanir, Tefaruk	Ph.D.
Hartman, Richard	M.S.
Hermansen, Knud	Ph.D.
Heuer, Dennis	M.S.
Huebner, Richard	Ph.D.
Johnson, Robert	M.S.
Jones, Debie	M.S.
Kotz, David	M.S.
Lakatos, David	M.S.
Moore, Charles	M.S.
Morales, Leonardo	Ph.D.
Morton, Micheal	M.S.
Nelson, Kim	MEPC
O'Brien, Michael	M.Eng.
Pierce, Greg	M.S. (Fish Biology)
Ritter, John	Ph.D. (Geology)
Sanchez, Gerardo	M.Eng.
Slingerland, Rudy	M.S. (Geology)
Slingerland, Rudy	Ph.D. (Geology)
Snow, Scott	M.S. (Geology)
Snow, Scott	Ph.D. (Geology)
Sobashinski, Daniel	M.S. (Forest Hydrology)
Stong, Blaine	Ph.D.
Strand, Stuart	Ph.D.
Sutton, John III	Ph.D.
Swanson, John	M.S.
Tagliati, Caesar	M.S.
Touysinhthiphonexay, Kim	M.S. (Geology)
Touysinhthiphonexay, Kim	Ph.D. (Geology)
Warwick, John	Ph.D.
Weidmann, Suzan	M.S. (Geology)
Weigel, Linda	M.S.
Wertz, William	Ph.D. (Geology)

Wong, Steven	M.Eng.
Yoo, Chung-Sik	M.Eng.

Continuing Education:

Dr. Arthur C. Miller has been involved in the Pennsylvania State University Continuing Education Program since 1973. Over that period of time he has been the chairman or co-chairman of over 80 one-week short courses. Twelve different water resource courses have been developed to enhance the technical ability of the practicing professional engineer. The courses have been a very successful segment of the Continuing Education Program at Penn State as evidenced by their longevity and the faculty service award of conference and institutes division, which Dr. Miller received in 1992.

Dr. Miller has also been involved in teaching courses throughout the country for the National Highway Institute (NHI). He has taught over 80 of these courses on topics ranging from fundamental hydraulics, open channel flow, to hydrologic processes. The courses contents vary from fundamental theory to hands on computer application.

COURSE TITLES AND BRIEF DESCRIPTION

1. Hydrologic and Hydraulic Analysis for Small Watersheds.
   The course covers the fundamentals of hydrology and hydraulics used in the analysis of small watersheds. Topics discussed include: rainfall estimates, hydrologic abstractions, unit hydrograph principles, synthetic unit hydrographs, detention basin design, hydrologic routing, and culvert analysis.

2. Urban Hydrology
   This course covers several computer models. The models discussed are STORM, TR-55, PSRM, and SWMM. Hands on application of each of the computer models is accomplished through workshops where the participants have an opportunity to compare the differences of each computer model.

3. HEC-1 Flood Plain Hydrology
Course topics include basic principles and practical applications of hydrology that relate to flood plain studies. A major emphasis is placed on the U.S. Army Corps of Engineers computer program HEC-1, which was developed at the Hydrologic Engineering Center in Davis, California. The course covers: runoff analysis, unit hydrograph theory, kinematic wave, loss rate analysis, design storm criteria, hydrologic flood routing, and basin modeling. The course is presented in a series of lectures and workshops.

4. HEC-2 Flood Plain Hydraulics
Basic principles and practical applications of river hydraulics that are related to flood plain studies are presented in this 5-day course. A major emphasis is placed on the U.S. Army Corps of Engineers Computer Program HEC-2 that was developed at the Hydrologic Engineering Center in Davis, California. The course covers: water surface profile computations, data requirements for computer modeling, sensitivity analysis, hydraulics of bridge waterways and flood plain determination.

5. Advanced Hydraulics for Flood Plain Hydraulics with Application to HEC-2
This course deals with the advanced problems encountered when modeling flood plains with a one-dimensional computer model. Particular emphasis is placed on simulating two-dimensional flow problems using a one-dimensional model. Topics covered in the course include: sensitivity analysis of data input, computational procedures used in flood plain determinations, flow through bridges, and errors associated with flood plain delineations.

6. Flood Flow Frequency Analysis
Topics covered in this course include: review of statistics and probability, frequency analysis of gaged watersheds, statistical treatment of data, expected probability adjustment, refinements to frequency curve, development of frequency curves using regional information, and frequency analysis for ungaged watersheds.

7. <u>Advances in Stormwater Management</u>
This symposium offers a series of technical and nontechnical lectures, discussion groups, and workshops focused on problems facing the planner and engineer. The emphasis is on recent advances in the technical and planning aspects of stormwater management, with discussion of computational methods and environmental impacts. Workshop sessions deal with: computer methods utilized in computing runoff and planning aspects of developing a basin-wide stormwater program.

8. <u>Computer-Aided Design of Sanitary Sewer Systems</u>
The objective of the course is to introduce the practicing engineer to the usefulness of a microcomputer in the design of gravity sanitary sewers. At the conclusion of the workshops the participants should be able to use the microcomputer, understand the basis for the theory used in the program, and be able to modify the program for special problems they may encounter.

9. <u>Comparison of HEC-1, TR-20, and PSRM</u>
The objective of this course is to compare the analytical techniques used in the three programs. The workshops are set up to develop an outflow hydrograph using the three models and to relate the differences in the results to the fundamental theory to the three models.

10. <u>Analytical Techniques for Dam-Break Analysis</u>
The course is intended to provide participants with a workable knowledge of computational techniques for simulating the creation and movement of dam-break flood waves. The major focus of the course is the use of computer programs HEC-1 and DAMBRK. The course requires that the participants have a background in open channel hydraulics.

11. <u>Unsteady Flow in Open Channels</u>
This course presents the basic theory and application of one-dimensional unsteady flow in open channels. The civil engineering profession is on the verge of sweeping changes regarding the computational procedures used in everyday practice for determining the hydraulics of open channels. Microcomputers and the minicomputers are replacing the traditional computational procedures for solving problems and with more accuracy. It is important for engineers to keep abreast of the newest advances in their profession and to understand and use these methods in their daily operations. This course benefits practicing civil engineers involved in the hydraulics of open channel flow.

12. <u>Water Surface Profiles (WSPRO)</u>
The WSPRO computer model was developed to be used to model water surface profile computations for one-dimensional, gradually varied, steady flow in open channels. WSPRO also can be utilized to analyze flow through bridge and culverts, embankment overflow, and multiple-opening stream crossings.

Consulting Activities

Kimball Engineering, Inc., Ebensburg, PA - Urban Stormwater Runoff (1973-74).

University Area Joint Authority - In Place Calibration of Five Flow Metering Installations using Lithium Chloride. (1975).

Gwin, Dobson and Foreman, Altoona, PA - Flood Plain Hydraulic Studies (1974-76).

Penn State Engineering, State College, PA - River Sedimentation Problems in Moshanon Creek (1975-76).

Trout Unlimited, Erie, PA - Reservoir Analysis Study for Flood Control (1975).

International Correspondence School, Scranton, PA - Develop Three Texts on Culvert Hydraulics (1974-77).

Centre Citizens Committee, State College, PA -Environmental Inpact Statement for the 322 Bypass Around State College (1975).

Penn State Engineering, State College, PA - Flood Plain Hydraulics Study of Allegheny River near Pittsburgh, PA (1975-76).

Buchart-Horn Consulting Engineers and Planners, York, PA - River Modeling (1977).

Meade Corporation, Dayton, OH - Stormwater Runoff for Industrial Park (1977).

New Jersey Department of Environmental Protection and NJIT - Erosion and Sedimentation Problems in the South Branch of Rockway Creek (1977-79).

R and R Construction Company, State College, PA - Stormwater Drainage for Site Layout (1977).

Parsons, Brinckerhoff, Quade and Douglas, Inc., New York, NY - Critical Review of Hydrologic Study on the Niagara River (1978).

Owens/Corning Fiberglass, Columbus, OH - Stormwater Drainage on Site (1978).

Pennsylvania Professional Engineering Examination - Responsible for Water Resources problems on the Pennsylvania Professional Examination (1978-1980).

GAI Consultants, Pittsburgh, PA - Special Training of the Hydrology Specialty Group (1978).

Commissioner Brown, Lock Haven, PA - Analysis of Bridge Failure Across Bald Eagle Creek (1979).

The City of Utica, New York - Critique of Stormwater Management Plan for an Urban Mall (1979).

Wiley Publications, New York, NY - Book Review, "Hydrology and Water Resources Technology" (1979).

Skelly and Loy Consultants, Harrisburg, PA - Position Paper on Water Quality Standards for Suspended Sediments Downstream of Sedimentation Basins (1979).

The Master Group, Ambler, PA -Expert Witness on Flood Plain Zoning (1979).

Gwin, Dobson and Foreman, Altoona, PA - Stormwater Quality Modeling (1979).

Echert, Seamans, Cherin & Mellot, Pittsburgh, PA - Expert Witness Dam Safety Analysis (1980-83).

Maryland Department of Transportation - Position Paper on the Accuracy of Computer Modeling for Flood Plain Delineation (1980).

Stetson and Dale Engineering, Utica, NY - Erosion and Sediment Problems in Red House Lake, New York (1980).

The Ministry of Natural Resources, Ontario, Canada - Training Program for Hydrologic and Hydraulic Engineers (1980).

C. Raymond Weir Associated, Ambler, PA - Flood Plain Hydraulics Study (1980).

International Coal Refining Company, Allentown, PA - Critical Review of Synthetic Fuel Plant Site Location in Owensboro, KY (1980).

Geo-Science, Inc., Harrisburg, PA - Project Engineer on Developing a Consulting Proposal for Dam Safety Analysis (1981).

EADS, Inc., Altoona, PA - Dam Breach Analysis (1981).

Kerry Uhler and Associates, Bellefonte, PA - Bridge Hydraulics Problem (1981).

Whitman, Requardt and Associates, Baltimore, MD - Dam Breach Analysis (1981).

Robbin and Associates, Harrisburg, PA - Hydrologic Studies for Mining Areas (1981).

Whitman, Requardt and Associates, Baltimore, MD - Bridge Hydraulics Study (1981).

Schoenagel and Schoenagel Inc., Wallenwaupack, PA - Hydrologic Study for Dam Breach Analysis (1982; 1984; 1989; 1990).

Todd Giddings and Associates, State College, PA - Hydrologic Study for Stripmine Area (1982).

Sanders, Wall and Wrye, State College, PA - Hydrologic and Hydraulic Study for Tyrone Reservoir No. 2 (1983).

University Area Joint Authority, State College, PA - Calibration of Flow Metering Installations (1983).

Novak and Associates, Attorney-at-Law, State College, PA - Stormwater Management (1983).

Cox and Cox, Attorney-at-Law, Wellsboro, PA - Expert Witness for Hydraulic Bridge Failure (1983-84).

Todd Giddings and Associates, State College, PA - Dam Breach Analysis (1984).

University Area Joint Authority, State College, PA - Low Flow Analysis of Spring Creek a Statistical Study (1984).

U.S. Department of Transportation, Washington, D.C. - Training in Hydraulics of Rivers and Bridge Waterway Computations (1984).

CIA - Dam Breach Analysis for the Kumgangsan Dam on the North Han River in North Korea (1986-87).

FHWA - Instructor in a course entitled Flood Plain Hydraulics KY, CT, other states.

Schoenagel & Schoenagel, Inc., Dam Safety Evaluation, Scranton, PA (1987).

Federal Highway Administration Hydrologic and Hydraulic Problems Associated with Drainage Problems for Highway Engineers, Washington, D.C. (1985-present).

Kidde Consultants, In., - I-195 Bridge over the Patapsco River, Hydrualic Analysis (1986-88).

Maryland State Highway Administration - Consulting on hydraulics bridge waterways, Baltimore, Md (1986-present).

Pennsylvania Attorney General's Office. Expert Witness on

Drainage From an Interstate Highway System, Scranton, PA (1987-88).

U.S. Army Corps of Engineers. Numerical Modeling of One-Dimensional Flow, Hydrologic Engineering Center, Davis, CA (1987-88).

Thethys Consultants Inc. Expert Witness, Stormwater Drainage System Evaluation Upstream of an Interstate Highway (1990).

U.S. Army Corps of Engineers. Flood Plain Delineation - A One-Dimensional Model, Philadelphia District, Philadelphia, PA (1990-91).

U.S. Army Corps of Engineers. Bridge Hydraulics for Flood Plains, Philadelphia District, Philadelphia, PA (1992-present).

Colorado Springs Utilities Water Authority, Independent Review Board for the Location of A Water Treatment Facility in the City of Colorado Springs, CO., (1991-1993)

Appleton Paper Company, Establishing Firm Yield for the Roaring Spring a Water Supply Source. (1992-present)

Bloomsburg County, Developing a Drainage Plan for the Bloomsburg Fair Grounds, (1992-present)

Personal Resume

Samuel L. Hui

Home Address:	328 Trestle Glen Court Walnut Creek, CA U.S.A. 94598
Telephone:	Res. (510) 939-6186 Bus. (415) 768-7734
Birth Place & Date:	Zhanjiang, Guangdong, China 27 November 1942
Marital Status:	Married with two daughters
Nationality:	American
Education:	B.Sc. (Eng.) Civil Engineering, with Distinction Queen's University, Kingston, Ontario, Canada 1966
	M.Sc. (Eng.) Civil Engineering, Hydrology/Hydraulics Queen's University, Kingston, Ontario, Canada 1969
	Short Course on Statistical Hydrology Colorado State University, 1978
	Business Management Program Bechtel/Golden Gate University 1979
	A course on "Modeling of Contaminant Fate Transport in Ground Water", University of California Extension, Berkeley, 1985
	Short Course on "Modeling of Water Quality using the "QUAL2E" Code, U.S. Environmental Protection Agency, Athens, Georgia, U.S.A. 1990
Honors:	National Research Council (NRC) of Canada Scholar 1968 - 1969

Registered Professional Engineer:

Province of Ontario, Canada
State of California, U.S.A.

Experience:

November 1988 - Present	Chief Hydrologic Engineer Bechtel Corporation, San Francisco, CA, U.S.A.
July 1978 - November 1988	Assistant Chief Hydrologic Engineer Bechtel Civil Inc., San Francisco, CA, U.S.A.
November 1975 - July 1978	Engineering Supervisor, Hydrology Bechtel Inc., San Francisco, CA, U.S.A.
December 1973 - November 1975	Senior Engineer, Hydraulics/Hydrology Group Bechtel Inc., San Francisco, CA, U.S.A.
June 1971 - December 1973	Senior Hydraulic Engineer Acres Consulting Service Limited Niagara Falls, Ontario, Canada
May 1969 - June 1971	Intermediate Hydraulic Engineer Acres Consulting Service Limited Niagara Falls, Ontario, Canada
September 1967 - May 1969	Graduate Student/Research Assistant Department of Civil Engineering, Queen's University Kingston, Ontario, Canada
May 1966 - August 1967	Junior Civil Engineer Montreal Engineering Company, Limited Montreal, Quebec, Canada
Summer 1965	Summer Student/Engineer Ontario Hydro Toronto, Ontario, Canada
Summer 1964	Summer Student/Research Assistant National Research Council (NRC) of Canada Ottawa, Ontario, Canada

Duties with Bechtel:

1973 - 1974	Senior Engineer, Hydraulics/Hydrology Group; Performed various hydrologic studies and hydraulic analysis for nuclear power plants and flood control projects.
1975 - 1978	Engineering Supervisor, Hydrology; Supervised a group of hydrologic engineers, as many as eleven, in performing hydrologic studies and hydraulic analyses for hydroelectric developments, nuclear and coal-fired power projects, flood control and airport drainage works, reservoir sedimentation and energy production studies for water resources projects, water supply and hydrology for tailings dams for mining developments, wave studies for reservoir design and coastal structures and the preparation of licensing documents for nuclear power projects, hydroelectric and mining developments, hazardous waste isolation and other industrial projects.
1978 - 1988	Assistant Chief Hydrologic Engineer; Responsible for the daily conduct of the Hydrology Group in San Francisco. This includes planning and scoping of tasks, work assignments, and technical supervision and review of all hydrologic analyses and water resources studies. Studies included a wide range of chemical and nuclear waste-isolation projects, hydroelectric and water resources developments, mine drainage and tailings dams, flood control, airport projects, coal-fired and nuclear power plants, and industrial projects.
1988 - present	Chief Hydrologic Engineer; Responsible for all hydrologic analyses and water resources studies on engineering projects in United States and in foreign countries, including work assignments performed in all Bechtel Regional Offices and area offices. Assist the Manager of the Hydraulics/Hydrology Group in the business development for the Group, which includes the identification of potential target clients, the marketing of services, and the

preparation and the presentation of business proposals to potential clients.

Professional Societies:

Member , American Society of Civil Engineers
Member, Canadian Society for Civil Engineering
Member , Engineering Institute of Canada
Member, U.S. Committee on Large Dams
Member, International Water Resources Association
Certified Professional Hydrologist, American Institute of Hydrology

Professional Activities:

1981	Member of Control Group, American Society of Civil Engineers (ASCE) Task Committee on Urban Stormwater Management in Coastal Areas
	Member, ASCE Task Committee on Application of Risk and Reliability Analysis on Design of Hydraulic Structures
1984 - present	Member, ASCE Technical Committee on Surface Water Hydrology
1985 - 1989	Member of Control Group, ASCE Technical Committee on Surface Water Hydrology; Chairperson 1987 - 1988
1989 - present	Secretary and Member of Control Group, ASCE Management Group D (MGD) Task Committee on Preparation of the Handbook of Hydrology
1991 - present	Member of the American National Standard Institute (ANSI) Committee 2.13 - Evaluation of Surface Water Supplies for Nuclear Power Sites
1994	Member of the International Program Committee for METEOHYTEC 21 - International Conference on Meteorological and Hydrological Technology and its Management to be held in Geneva, Switzerland in May 1995

1994	Member of the Technical Program Committee for Water Power '95 to be held in San Francisco, U.S.A. in July 1995

Representative Project Experience:

1964 -	Heat budget calculations using recorded hydro-meteorological data to determine the feasibility of keeping open the St Lawrence River for all year-round navigation; National Research Council of Canada
1964 -	Wave refraction diagrams and breakwater model study for the Codroy Harbor in Newfoundland, Canada; National Research Council of Canada
1965 -	Preliminary structural design of the intake channel for the Pickering Nuclear Power Station near Toronto, Ontario, Canada; Ontario Hydro
1966 -	Feasibility studies of several hydroelectric developments in the Province of Alberta, Canada and foreign countries; Montreal Engineering Company, Limited
1966 -	Statistical analysis of the water level of the St. Lawrence River near Seven Island for the design of the Gentilly Nuclear Power Station, Quebec, Canada; Montreal Engineering Company, Limited
1966 -	Water supply system for the Dawson City, Yukon Territories, Canada; Montreal Engineering Company, limited
1967 -	Preliminary design of the anchored blocks of a penstock for a hydroelectric power plant in Bolivia; Montreal Engineering Company, Limited
1967 -	Cost estimates of the third unit for the Trenton Coal-

	fired Power Station, Nova Scotia, Canada; Montreal Engineering Company, Limited
1967 - 1969	Research work in the field of snow hydrology and the development of a mathematical model of snowmelt runoff for a typical intermediate drainage basin in Southern Ontario, Canada; Department of Civil Engineering, Queen's University
1969 - 1971	Study, planning and formulation of a comprehensive mathematical flood model for the State of Bangladesh and the development a computer program for reservoir operations using dynamic programming techniques for the Karnafuli Reservoir in Bangladesh; Acres Consulting Services Limited
1969 -	Hydrologic data analyses for the St John River Water Quality simulation model in New Brunswick, Canada; Acres Consulting Services Limited
1970 -	Developments of a hydro-meteorological data collection network and the reservoir operating rules, and the flood forecasting for the Churchill Falls Hydroelectric Project in Newfoundland, Canada; Acres Consulting Services Limited
1971 -	Derivation of the long-term streamflows for the feasibility study of the Lower Churchill River Developments in Newfoundland, Canada; Acres Consulting Services Limited
1971 - 1972	Development of the long-term streamflows and flood analyses for the preliminary design of the James Bay Hydroelectric Project in Quebec, Canada; Acres Consulting Services Limited
1972 -	Hydrologic studies for a flood control project in the Ellicott Creek Basin, near Buffalo, New York, U.S.A.; Acres Consulting Services Limited
1972 -	Water resources assessment of the Lost River Basin for the development of the water supply system for a mining project near Nome, Alaska, U.S.A.; Acres Consulting Services Limited

1973 -	Conceptual development of the experimental equipment to measure the possible mercury fluxes and migration in an artificial mercury waste-isolation island in St. Clair River in the Canada/U.S.A. border; Acres Consulting Services Limited
1973 -	Water resources review of the Restigouche County, New Brunswick, Canada; Acres Consulting Services Limited
1973 - 1977	Preparation of the Safety Analysis Report (SAR) and the Environmental Report (ER) for the Boardman, Pebble Springs, Skagit , Susquenhanna and Limerick Nuclear Power Projects in Oregon, Oregon, Washington, Pennsylvania and Pennsylvania, U.S.A., respectively; Bechtel Power Corporation
1974 -	Flood analysis for the Iron Gate Hydroelectric Project , California, U.S.A. as part of the dam safety investigation; Bechtel Incorporated
1974 -1975	Hydro-meteorological data collection and analysis, flood studies and water supply assessment for the Arun Liquefied Natural Gas Project on the Island of Sumatra, Indonesia; Bechtel Incorporated
1974 - 1991	Flood analyses, sediment transport studies and debris production estimates for the La Quinta Stormwater Project, Deep Canyon Stormwater Project, Palm Valley Flood Control Project, Mid-Valley Flood Control Project, Thousand Palms Stormwater Project in the Coachella Valley area, south of Palm Springs, California, U.S.A.; Bechtel Corporation
1975 - 1976	Long-term streamflow reconstitution, probable maximum precipitation (PMP) and probable maximum flood (PMF) determinations, reservoir sedimentation evaluation and energy production studies, reservoir freeboard allowance and riprap estimates for the Rio Grande-Rositas Multi-purpose Water Resources Project near Santa Cruz, Bolivia; Overseas Bechtel Incorporated

1975 - 1977	Hydro-meteorological data collection and analyses, and water supply studies for a large nickel development on Gag Island, a small uninhabited island in Indonesia; Behctel Incorporated
1976 - 1977	Flood studies for the Samarco Iron Ore Project near Belo Horizonte, Brazil; Bechtel Incorporated
1976 - 1978	Stormwater management studies and flood hydrology for the access highways, the airport terminal and the infield areas for the King Khaled International Airport, Riyadh, Saudi Arabia; Bechtel Civil & Minerals Incorporated
1977 -	Hydrology for water supply for a 10-million ton/year steel mill in Western Algeria; Bechtel Incorporated
1977 -	Preparation of the Safety Analysis Report (SAR) and the Environmental Report (ER) for the Waste Isolation Pilot Project (WIPP) in New Mexico, U.S.A.; Bechtel National Incorporated
1977 - 1978	Flood hydrology, water balance studies and the design of riprap of the flue-gas desulphurization ponds of the Jim Bridger Power Project, in Wyoming, U.S.A.; Bechtel Power Corporation
1977 -1979	Flood hydrology and sediment transport studies for the design of major river crossings of the water supply pipeline for the Palo Verde Nuclear Power Project near Phoenix, Arizona, U.S.A.; Bechtel Power Corporation
1978 - 1980	Energy production and power studies for the Wanapum and Priest Rapids Hydroelectric Projects in the Mid-Columbia River, Washington, U.S.A.; Bechtel Civil & Minerals Incorporated
1978 - 1981	Probable maximum precipitation (PMP) and probable maximum flood (PMF) determinations, reservoir sedimentation evaluation, reservoir yield studies and

	cross drainage analysis for the Setif Project in the interior of Algeria; a large irrigation development involving four dams and a transmountain water conveyance scheme; Bechtel Civil & Minerals Incorporated
1978 - 1981	Long-term streamflow reconstitution, energy production studies and hydrologic analyses for environmental permit applications for the Henry Jackson (Sultan River) Hydroelectric Project in Washington, U.S.A.; Bechtel Civil & Minerals Incorporated
1979 -	Water supply assessment for the long-term planning study of the Chuquicamata Copper Mining Project near Calama, Chile; Bechtel Civil & Minerals Incorporated
1979 -	Water supply analysis for the Matam Perimeter Study in Senegal, Africa; Bechtel National Incorporated
1979 -	Preparation of hydrologic design criteria for the Chevron Kosti Oil Development Project in Sudan, Africa; Bechtel Incorporated
1980 -	Water supply studies and stormwater management evaluation for the Creston Coal-fired Power Project near Spokane, Washington, U.S.A.; Bechtel Power Corporation
1980 - 1982	Flood studies and water supply evaluation of the Chase Creek drainage basin for the Morenci Mines in Morenci, Arizona, U.S.A.; Bechtel Civil & Minerals Incorporated
1981 -	Stormwater management evaluation of the coal ash disposal area for the Kaiser Aluminum & Chemical Plant near Gramercy, Louisiana, U.S.A; Bechtel Power Corporation
1981 -	Energy production studies for the Lake Chukachumna Hydroelectric Development in Alaska, U.S.A.; Bechtel Civil & Minerals Incorporated

1981 -	Flood and drainage analyses for the Chuitna Coal Project in Alaska, U.S.A.; Bechtel Civil & MInerals Incorporated
1981 -	Energy production studies for Milner hydroelectric development on the Snake River, Idaho, U.S.A. for Idaho Power Company; Bechtel Civil & Minerals Incorporated
1981 - 1983	Hydrology for hydroelectric development, water supply and tailings disposal for the OK Tedi Mining Project in Papua New Guinea; Bechtel Civil & Minerals Incorporated
1982 -	Flood and drainage evaluation for the development of Zhungeer Coal Field in Inner Mongolia, China; Bechtel Civil & MInerals Incorporated
1982 - 1983	Hydrology for flood analysis, water supply, site drainage and instream flow requirements for environmental conservation for the Quartz Hill Molybdenum Project near Ketchikan, Alaska, U.S.A.; Bechtel Civil & Minerals Incorporated
1982 - 1984	Flood studies and hydrologic characterization of the U..S. Department of Energy (DOE) Formerly Used Sites Remedial Action Prpgram (FUSRAP) sites at Niagara Falls, New York and Weldon Springs, Missouri, U.S.A.; Bechtel National Incorporated
1983 -	Flood analysis for the Hope Creek Generation Station, New Jersey, U.S.A., including wave effects; Bechtel Power corporation
1983 -	Hydrology for environmental impact evaluations of the Jamsboro Power Project in Pakistan; Bechtel Environmental Incorporated
1983 -	Hydrology for the Parachute Creek Oil Shale Development Project in Colorado, U.S.A.; Bechtel Incorporated

1983 -	Modeling of surface runoff, erosion and contaminant transport for the FMC Richmond site in California, U.S.A.; Bechtel Environmental Incorporated
1983 -1984	Rainfall analysis, field infiltration tests and site drainage design for the King Fahd International Airport near Damman, Saudi Arabia; Bechtel Civil & Minerals Incorporated
1983 - 1986	Environmental assessment and hydrologic characterization of candidate sites for the U.S. Department of Energy (DOE) Office of Nuclear Waste Isolation (ONWI) salt repository program; Bechtel National Incorporated
1983 - 1986	Developments of Probable maximum flood and dam break analyses for dam safety studies for several dams in the States of Montana and Washington for Montana Power Company; Bechtel Civil & Minerals Incorporated
1984 -1986	Flood hydrology and water balance study for the Stringfellow Acid Pits Hazardous Waste Remedial Project near Riverside, California, U.S.A.; Bechtel Environmental Incorporated
1985 -	Water supply studies for the Chisumbanje Irrigation Project in Zimbabwe, Africa; Overseas Bechtel Incorporated
1985 -	Energy production study for the New Martinsville Hydroelectric Project on the Ohio River, West Virginia, U.S.A.; Bechtel Civil & Minerals Incorporated
1985 -	Modeling of surface runoff, erosion and contaminant transport for the FMC Dublin Road site in New York, U.S.A.; Bechtel Environmental Incorporated
1985 -1986	Surface water hydrologic analyses for the California Low-level Radioactive Waste Disposal Project;

	Bechtel National Incorporated
1985 -1986	Flood analysis and energy production studies for the Sheldon Springs Hydroelectric Project in Vermont, U.S.A.; Bechtel Civil & Minerals Incorporated
1985 - 1986	Flood analysis and water balance study for a landfill containing hazardous wastes near Monterey Park, California, U.S.A.; Bechtel Environmental Incorporated
1986 -	Energy production studies for the feasibility investigation of small hydroelectric developments on the Friant-Kern canal in Central Valley of California, U.S.A.; Bechtel Civil & Minerals Incorporated
1986 -	Energy production studies for the Fieldcrest Hydroelectric Project on the Chattahoochee River in the Alabama/Georgia border, U.S.A.; Bechtel Civil & Minerals Incorporated
1986 -1987	Development of the probable maximum flood for the Skookumchuck River Basin, Washington, U.S.A., as part of the dam safety investigation; Bechtel Civil & Minerals Incorporated
1986 - 1989	Hydrologic studies for the Cowlitz Falls Hydroelectric Project in Washington, U.S.A., including development of probable maximum flood, sediment routing computations and detailed operation study to estimate annual energy; Bechtel Civil Incorporated
1987 -	Flood hydrology and energy production studies for the Dinkey Hydroelectric Project in the Kings River Basin in Central California, U.S.A.; Bechtel Civil Incorporated
1987 -	Hydrology for the feasibility study of the Mount Hope Pumped Storage Project, New Jersey, U.S.A.; Bechtel Civil Incorporated

1987 -1988	Drainage design for the Ground-Based Laser Facility in New Mexico, U.S.A.; Bechtel National Incorporated
1987 -1988	Modeling of storm runoff for the Union Oil refinery at Rodeo, California, U.S.A Bechtel Environmental Incorporated
1987 -1988	Hydrologic analysis and dam break studies for the Peace Dam Project on the Han River in South Korea; Overseas Bechtel Incorporated
1987 -1988	Flood hydrology and reservoir system operations and energy production studies for the Naranjito Hydroelectric Project in Honduras, Central America; Overseas Bechtel Incorporated
1987 - 1991	Site selection, and hydrologic characterization of and permit application for the selected Butte site for the Central Interstate Compact Low-level Radioactive Waste Disposal Project in Nebraska, U.S.A.; Bechtel National Incorporated
1988 -	Water management study for the entire Larona River Basin in Indonesia for energy generation enhancement; Bechtel Civil Incorporated
1988 -	Hydrology and energy production studies for the Rodgers Crossings Hydroelectric Project on the Kings River near Fresno, California, U.S.A.; Bechtel Civil Incorporated
1988 -	Review authority for the drainage design for the McCarran International Airport in Las Vegas, Nevada, U.S.A.; Bechtel Civil Incorporated
1988 - present	Flood hydrology for the Bay Area Rapid Transit (BART) Extension Project in the San Francisco Bay Area, California, U.S.A.; Bechtel Civil Incorporated
1989 -	Hydrology for the Chevron Chemical Company Pond Closure Project, Richmond, California U.S.A.;

	Bechtel Environmental Incorporated
1989 -	Review of the flood hydrology for the Santa Ana River Project for the U.S. Army Corps of Engineers; Bechtel Corporation
1989 -	Modeling of the storm runoff for the Chevron Chemical Fertilizer plant in Richmond, California, U.S.A; Bechtel Environmental Incorporated
1989 -1990	Study of methods for the removal of the accumulated sediment from the Rock Creek and Cresta Reservoirs and the analysis of their potential environmental impacts for Pacific Gas & Electric Company; Bechtel Civil Incorporated
1989 -1990	Review of the water quality modeling of the Santa Clara Valley Area for the Santa Clara Valley Water District; Bechtel Corporation
1989 - present	Consultant to the State of Wyoming on the development of the North Platte River Simulation Model; Bechtel Corporation
1990 -	Modeling of storm runoff for the Shell Oil Company refinery in Martinez, California, U.S.A.; Bechtel Environmental Incorporated
1990 -1991	Flood hydrology and dam break analyses for PAR Pond and L Lake Dams in the U.S. Department of Energy (DOE) Savannah River Site, South Carolina, U.S.A.; Bechtel Savannah River Incorporated
1991 -	Drainage design for the Scrugrass Co-generation Project in Pennsylvania, U.S.A.; Bechtel Power Corporation
1991 -	Modeling of storm runoff for the Tosco Oil Company refinery in Avon, California, U.S.A.; Bechtel Petroleum, Chemical & Industrial Incorporated
1991 -	Drainage study for the Vasco Road Relocation Project in Contra Costa County, California, U.S.A.; Bechtel Civil Incorporated

1991 -	Hydrologic characterization study for the storm water outfalls in the U.S. Department of Energy (DOE) Savannah River Site, South Carolina, U.S.A.; Bechtel Savannah River Incorporated
1991 -	Preparation of engineering manual on hydrologic engineering requirements for flood damage reduction projects for the U.S. Army of Corps of Engineers; Bechtel Corporation
1991 - present	Design of bank protection works for a 30-mile reach of the Whitewater River in Coachella Valley, Southern California; Bechtel Civil Company
1992 - 1993	Development of guidelines for the determination Probable Maximum Floods sponsored by Federal Energy Regulatory Commission, managed by Electrci Power Reseach Institute; Bechtel National, Inc.
1992 - 1994	Development of a drainage master plan for a 4-sq. mi. area near the proposed Dublin BART station; Bechtel Civil Company
1992 - present	Operations studies for the Grayrocks Reservoir on the Laramie River in Wyoming; Bechtel Civil Company
1992 - 1993	Review authority for hydrologic analyses for two flood control dams near Copiapo, Chile; Bechtel Corporation
1993	Drainage studies for an aluminum refinery near Corpus Christi, Texas; Bechtel Corporation
1994	Dam Break analyses for six dams on the Mokelumne River in California for Pacific Gas & Electric Company; Bechtel National, Inc.
1994 -	Hydrologic analyses for the feasibility study of the Maheshwar Hydro Project in India;

	Bechtel Civil Company
1994 -	Hydrolgic studies for the design of a 50-mile toll road near Shenzhen, China; Bechtel Civil Company
1994 -	Review authority for hydrologic studies for flood control works for the Southern Peru Copper Project near Toquepala, Peru; Bechtel Corporation

Publications:

"Sultan River Hydroelectric Project, Reservoir operations studies", Proceedings of the Symposium, Waterpower '85, Las Vegas, Nevada, U.S.A., September 1985 (with J.J. Cassidy)

"Hydrologic Studies for the Dinkey Creek Hydroelectric Power Project", Proceedings of the Symposium, Waterpower '87, Portland, Oregon, U.S.A., August 1987, (with J.J. Cassidy)

"Consideration with Regard to the Choice of Recurrence Interval for a Design Flood", Proceedings of the 16th Congress, International Commission on Large Dams, San Francisco, California, U.S.A., June 1988 (with J.J. Cassidy, D.B. Cherry and J.E. Welton)

"Hydrologic Simulation of the Rio Grande, Bolivia", Proceedings of the Symposium on Hydrology and Water Management of the Amazon Basin, Manus, Brazil, August 1990, (with V. Yucel)

"Design Floods and Flood Calculations" published in Proceedings of the Algerian National Congress on Large Dams, Algiers, Algeria, May 1993 (with J.J. Cassidy)

"Dam Break Study for Large Floodplain Area - A Case Study" published in the Proceedings of the ASCE Second International Symposium on Engineering Hydrology, San Francisco, California, July, 1993 (with J.S. Wang, R. Baysinger)

"A Guideline for the Determination of Probable Maximum Flood for Civil Works" published in the Proceedings of Waterpower '93, Nashville, Tennessee, August 1993, (with J.J. Cassidy, J.W. Gotzmer and D.I. Morris)

"Development of Rainfall Intensity-Duration-Frequency Data for Eastern Province, Saudi Arabia" published in the Proceedings of ACSE Saudi Arabia Section First Regional Civil Engineering Conference, Bahrain, September 1994. (with J.J. Cassidy and V.Yucel)

"Problems Encountered and Solved in the Application of SWMM" published in the Proceedings of Urban Hydrology Workshop sponsored by the U.S. Army Corps of Engineers, Davis, California, September 1994. (with J.S. Wang and K.Y. Ng)

Many other job related technical reports on hydrologic studies and water resources evaluations on feasibility investigations, preliminary and final designs.

Oral Presentations:

Expert witness on surface water hydrology testifying on behalf of the Puget Sound Power & Light Company before the U.S. Atomic Safety Licensing Board for the Skagit Nuclear Power Project, 1975

Lectures on "Hydrologic and Water Resources Engineering", presented as part of the Civil Engineering Review Course to the Candidates of the California Professional Civil Engineers Examination; University of California Extension, Berkeley; 1979 - 1991

Lectures on "Hydrologic and Water Resources Engineering", presented as part of the Civil Engineering Review Course to the Candidates of the California Professional Civil Engineers Examination; Bechtel Corporation In-house Education Program in San Francisco; 1979 - present

Expert witness on surface water hydrology tesfying on behalf of the Washington Water Power Company before the Washington State Thermal Power Siting Evaluation Council for the Cresta Power Project, 1980

"Sultan River Hydroelectric Project, Reservoir Operations Studies", Waterpower '85, Las Vegas, Nevada, U.S.A., September 1985

"Review of the Flood Hydrology for the Santa Ana River Basin", Hydraulics/Hydrology Seminar, 1988

"Hydrologic Simulation of the Rio Grande, Bolivia", Symposium on Hydrology and Water Management of the Amazon Basin, Manus, Brazil, August 1990

"A Guideline for the Determination of Probable Maxium Flood for Civil Works" presented in Waterpower '93, Nashville, Tennessee, August 1993

"Development of Rainfall Intensity-Duration-Frequency Data for Eastern Province, Saudi Arabia", ASCE Saudi Arabia Section First Regional Civil Engineering Conference, Bahrain, September 1994.

WELCOME - INTRODUCTION - ANNOUNCEMENTS

PMF GUIDELINES SECTION 8-1

Monday 8:00 a.m.

Chapter VIII

Determination of the Probable Maximum Flood

8-1 Introduction[1]

This chapter of the Engineering Guidelines is primarily intended to provide procedures for the development of the Probable Maximum Flood (PMF) for use in the analyses of proposed and existing dams and other water impoundment civil works structures. However, the procedures may also be used for other facilities requiring a PMF determination.

For about the last 40 years, the PMF has received general acceptance as the design flood for dams in the United States whose failure would pose a threat to public safety [Myers 1967]. More recently, the PMF has received acceptance as the design flood for large dams in many other countries as well [ICOLD 1991].

Precisely when the concept of the PMF was first proposed to the engineering community is somewhat obscure. Daniel Mead referred to a "probable maximum flood-flow" in his 1908 book on hydroelectric design [Mead 1908]. In 1914, A.E. Morgan referred to a "maximum possible event" in a discussion of regional peak discharges [Morgan 1914]. The details of using storm transposition to determine a "maximum probable precipitation" were discussed by G. Hathaway in 1939 [Hathaway 1939]. Bailey and Schneider proposed the PMF to determine spillway capacity in 1939 [Bailey and Schneider 1939]. Initially it was referred to as the "maximum possible flood," although the technical meaning was the same. Many variations of the definition of the PMF exist, but the definition in Chapter 2 of the FERC Engineering Guidelines is:

> ...the flood that may be expected from the most severe combination of critical meteorological and hydrologic conditions that are reasonably possible in the drainage basin under study.

A PMF is generated by the PMP, which has been defined in Chapter 2 of the FERC Guidelines as:

> ...theoretically, the greatest depth of precipitation for a given duration that is physically possible for a given size storm area at a particular geographic location at a certain time of year.

---

[1] This chapter of the Engineering Guidelines was developed under a contract with the Electric Power Research Institute (EPRI) and co-funded by the Federal Energy Regulatory Commission (FERC).

Caution: The intent of these guidelines is to provide consistency in PMF determinations. They are not a substitute for good engineering judgment and experience when available data clearly call for a departure from recommended procedures. Therefore, the recommended procedures should not be rigidly applied in place of other, justifiable analytical solutions.

Developing a PMF hydrograph for a dam safety evaluation generally involves two steps, which are respectively hydrologic and hydraulic in nature:

- Modeling of runoff through the project basin to produce an inflow PMF at the project site.

- Routing of the inflow PMF through the project reservoir to obtain the outflow PMF and the maximum reservoir elevation.

These steps involve considering several coincident or sequential events, each of which may have a strong effect on the resulting PMF. This chapter attempts to address those events and yet avoid an unreasonable compounding of probabilities that would make the resulting PMF hydrograph excessively conservative.

8-1.1 Objectives

There is little chance that hydrology will ever become the precise science that designers, owners, and regulators would like to see. So many parameters define the basin characteristics and hydraulics of runoff that the hydrologic engineer will always need to rely on experience and good judgment. This chapter is intended to provide systematic procedures that will consistently produce a realistic PMF hydrograph and appropriate flood levels for project safety.

While keeping the inherent uncertainty of hydrologic calculations in mind, the overall objectives of this chapter of the Guidelines are:

- To recommend a preferred method for developing PMF hydrographs.

- To present procedures which, if implemented by two or more qualified and experienced hydrologic engineers, would result in close estimates of the PMF.

- To make recommendations regarding the assumptions that must normally be made in developing a PMF hydrograph for gaged and ungaged sites.

- To produce an approach that will minimize the total effort and cost of required studies, while ensuring that the developed hydrograph is reasonable and prudent for use in the design or safety analysis of civil works.

- To provide guidelines for choosing appropriate hydrologic and hydraulic parameters.

- To provide greater consistency nationally for procedures used in PMF developments, while recognizing the wide variety of hydrologic conditions that exist across the United States.

8-1.2 Overview

The PMF is the reasonable upper limit of floods that can occur; however, its average annual exceedance probability is not known. Because it is the responsibility of owners of non-FERC-licensed dams to ensure the safety of their projects using the best available technology, the procedures recommended in this chapter to determine the PMF for FERC projects assume that all non-FERC-jurisdictional dams in the basin upstream of the project will not fail during floods up to the PMF. In other words, the PMF at the project site will not be a combination of the naturally occurring flood and the flood resulting from a failure of a dam not within the FERC's jurisdiction.

Instead, the PMF at the site is the result of routing the PMF through upstream dams assuming they remain in place. Of course, the procedure described above does not preclude FERC dam owners from considering the failure effect of upstream non-FERC dams in PMF evaluations.

Previously accepted PMF studies are not required to be reevaluated in accordance with the new Guidelines, unless it is determined that a reanalysis is warranted. However, all new PMF studies are to comply with the requirements of the Guidelines.

As PMF determinations are completed using the new Guidelines for a project with non-FERC-jurisdictional upstream dams, the FERC will advise the appropriate State Dam Safety Office (State) of the PMF study. The State will be informed that a new PMF study has been done for the FERC-jurisdictional dam, assuming all upstream dams do not fail and that the PMF study is available to the State for its review and information at the FERC Regional Office. The State will also be advised that reports on previous PMF studies for other dams under FERC jurisdiction might also be available in the Regional Office. States ensure the safety and adequacy of such dams under their own criteria and regulatory authority.

This chapter proposes the use of <u>unit-hydrograph theory</u> as the runoff model in the <u>preferred method</u> for developing an inflow PMF hydrograph. The development of the unit hydrograph is of primary importance in the ultimate development of the PMF hydrograph, because its use will determine both the temporal distribution and peak rate of runoff. <u>The use of the U.S. Army Corps of Engineers (COE) computer program HEC-1 Flood Hydrograph Package [HEC 1990] is recommended because of the widespread use and experience with that program.</u> However, many United States water resource agencies have developed material for their own regional use in developing hydrographs for gaged and ungaged basins, including dimensionless unit hydrographs, expressions for lag times, parameters for shaping unit hydrographs, and runoff models. This chapter recommends that any applicable special methods be located and evaluated.

- **Appendix VIII-A** includes a flow chart that summarizes this chapter.

- **Section 8-13** contains a glossary that defines technical terms used herein, which are part of the professional language of hydrologic engineering but may have slightly different meanings depending on the user.

- Cautionary statements have been provided throughout the text where care should be taken in the use of the recommended procedures, or where there are limitations to their application. These statements appear throughout the text in *italics*.

- A "gaged" basin is defined herein as one for which available hydrologic data, recorded at stations in the basin, are adequate to enable accurate computation of an inflow PMF hydrograph.

- **Appendix VIII-B** includes Probable Maximum Flood Study Report Outlines for gaged and ungaged basins.

  An "ungaged" basin is defined herein as one for which no hydrologic data have been recorded within the basin, or the available data is insufficient to enable accurate computation of an inflow PMF hydrograph.

  Development of unit hydrographs for both gaged and ungaged basins is discussed in this chapter. The inherent uncertainties in developing PMF hydrographs are significant even for locations where quality data are available; ungaged basins, of course, involve even more uncertainties. Final review of a PMF hydrograph should include a sensitivity analysis for parameters having significant effect on the inflow hydrograph.

- **Appendix VIII-C** presents a detailed explanation of the application of STATSGO (State Soil Geographic Database) data as recommended in Section 8-10.3.2 for use in determining infiltration rates.

- **Appendix VIII-D** presents reports on four hypothetical PMF studies:

  - **Example 1, Sabrina Dam** - A gaged basin with "detailed method" loss rate calculations.

  - **Example 2, Bishopsville Hydro** - An ungaged basin using synthetic unit hydrographs (SCS method) in the absence of local or regional streamflow information.

  - **Example 3, Corsorona Rapids** - A multi-subbasin, ungaged basin with a regional unit hydrograph study.

  - **Example 4, Austen Hydro** - A basin with unusual hydrology and data limitations, justifying a deviation from the procedures recommended in these guidelines.

8-1.3 Limitations

Drainage Basin Size. This chapter of the Guidelines covers drainage basins up to 10,000 square miles, although the size of the drainage area is not necessarily a limiting factor. Consideration has also been given to PMFs produced by local storms that would cover only part of a large basin, or all of a small one, and to general storms that could cover an entire 10,000-square-mile basin. The upper limit to basin size is arbitrary but was made to cover conditions applicable to many dams while still including basins requiring subdivision for analysis. This chapter applies to most basins with multiple FERC-licensed projects; however, procedures for very large drainage basins—such as the lower Missouri or the Columbia Rivers—cannot be easily generalized, since even general storms may not cover the entire basin.

Probable Maximum Precipitation. In writing this chapter, the preparers assumed that complete details of depth-area and duration of the PMP are available and no attention has been given to development of the PMP. However, references are made to developing the isohyetal pattern of the PMP and its use. Often this information can be obtained from the National Weather Service (NWS) Hydrometeorological Reports (HMRs). Because of the storm and flood data they include, the HMR series are important references, but other site-specific PMP studies may also be available. Figure 8-1.1 shows the geographic regions to which each HMR applies. Also included is a site-specific regional PMP study for the Wisconsin/Michigan area.

Hydrograph Development. Ranges of recommended values of parameters that must be assumed for developing hydrographs are given throughout the text. These parameters were taken from available material developed by government agencies and other organizations. However, the material cited or quoted does not represent an exhaustive search of the literature, and each section suggests potential sources of additional data, such as unit hydrographs or infiltration rates. The methods recommended were chosen from those widely recognized and accepted by the hydrologic engineering profession and for which considerable information is available.

Because the state-of-the-art in hydrology is constantly changing, the procedures suggested herein may require future changes. Therefore, this is a "dynamic" document—one subject to review and change as the state of hydrologic engineering is refined or improved.

Where there are limitations to the recommended procedures, or where care should be taken in their use, cautionary statements are provided throughout the text.

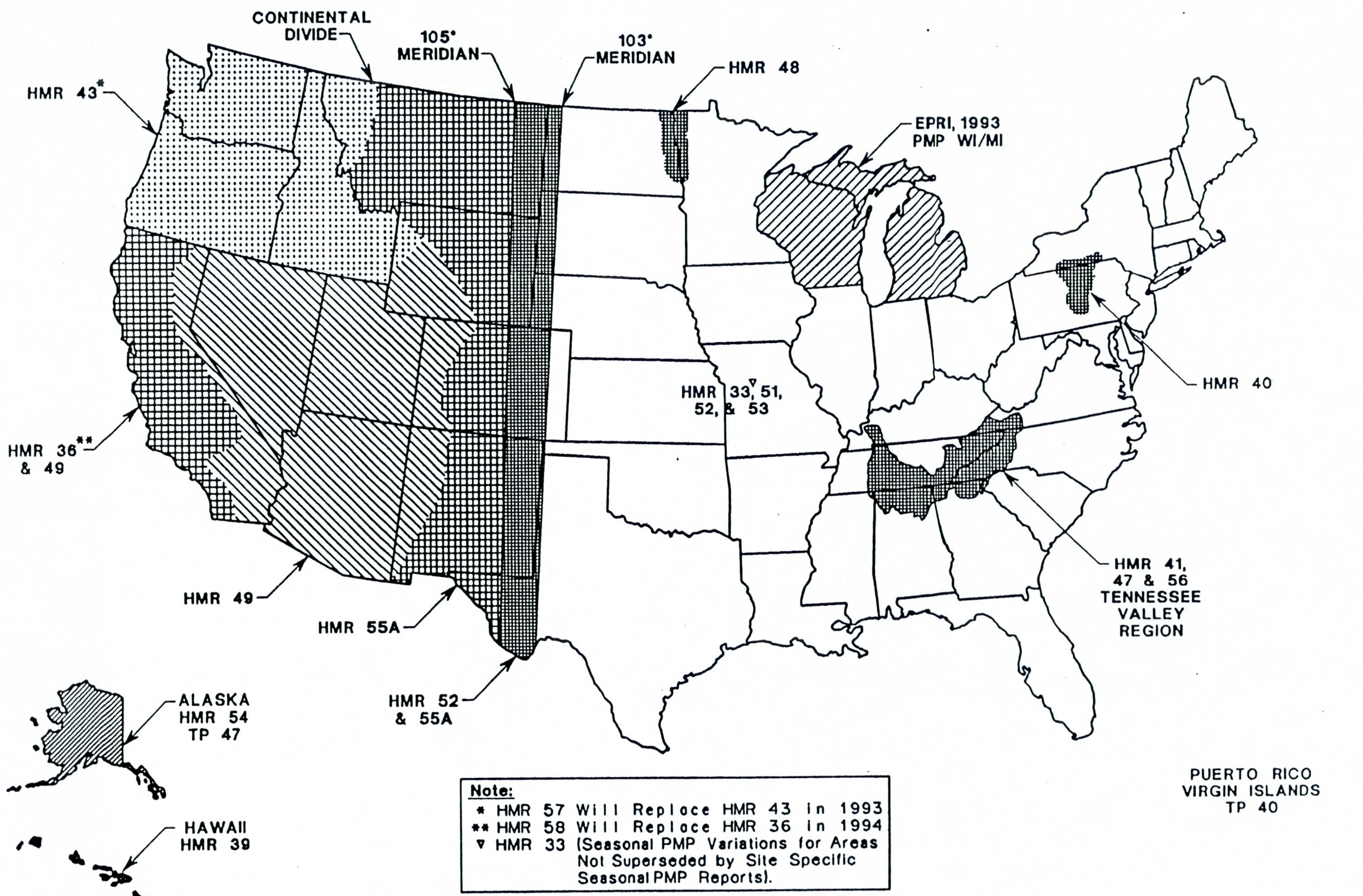


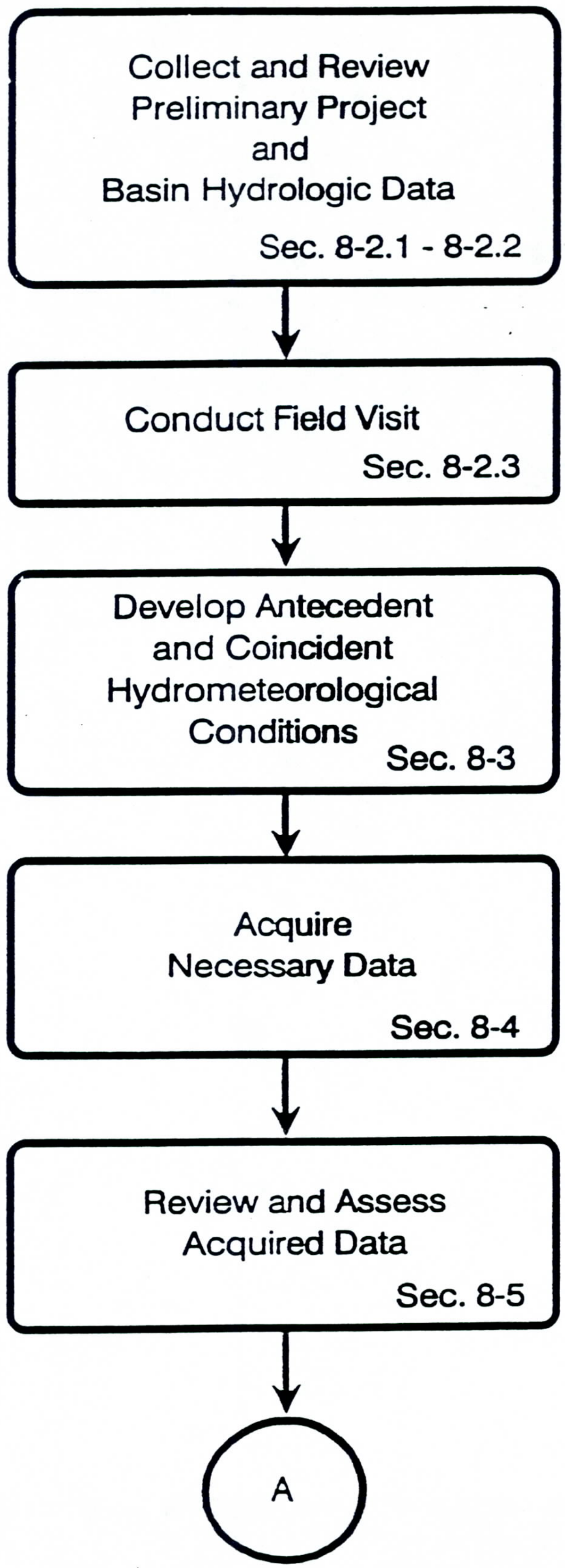
Fig. 8-1.1 Regions Covered by PMP Studies.

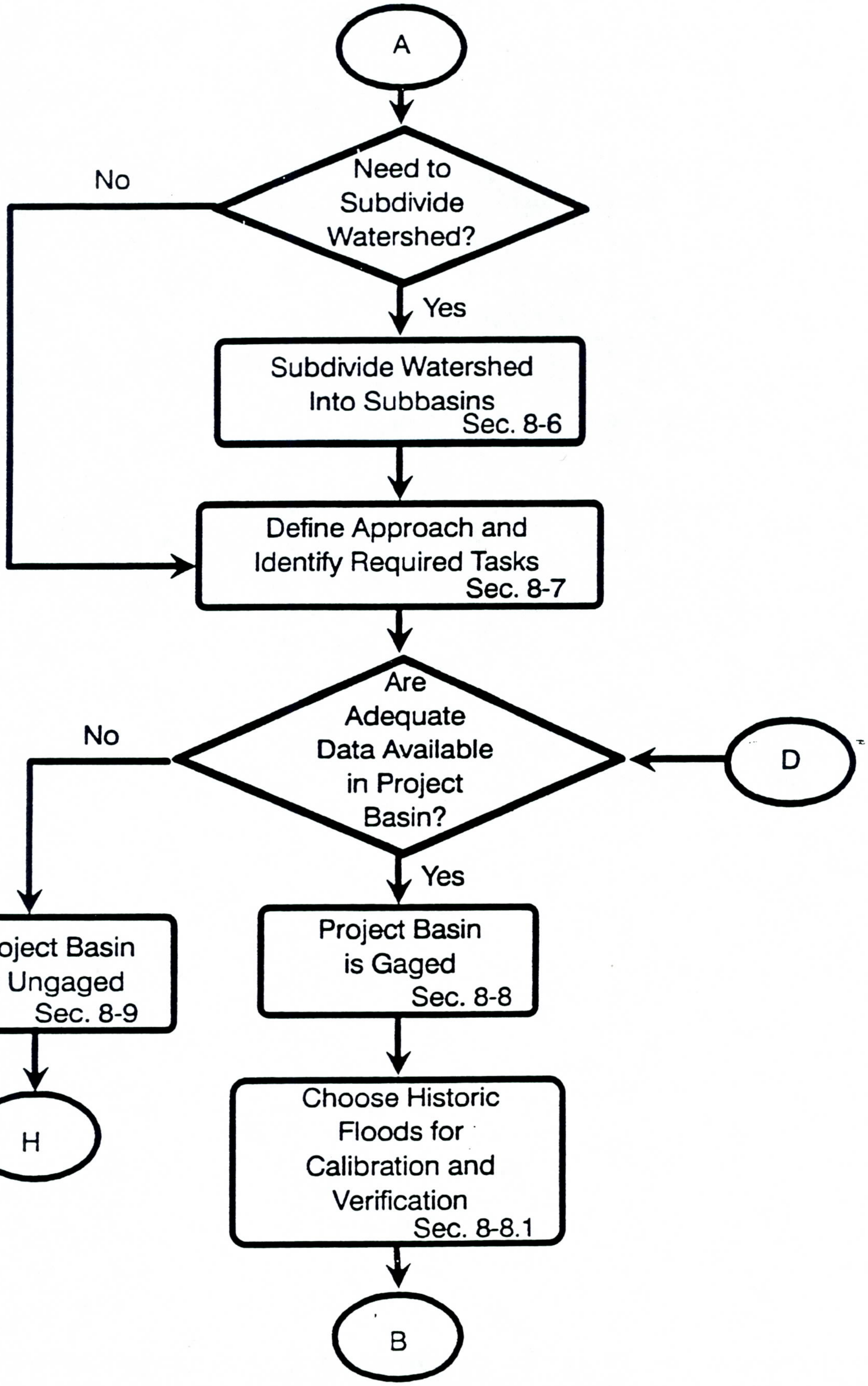
Appendix VIII-A

Determining the Probable Maximum Flood for Civil Works Flow Chart

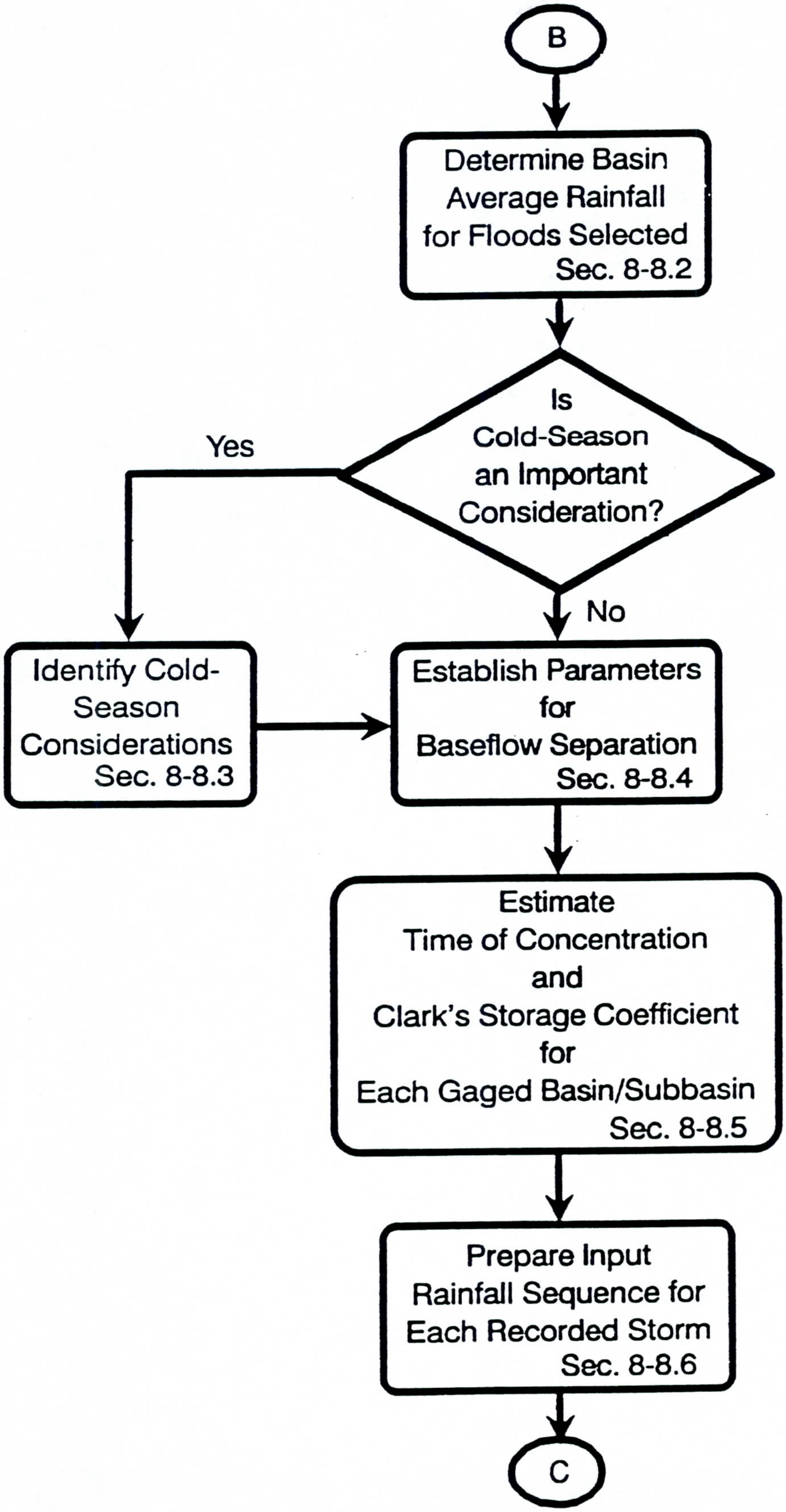
The flow chart shows the sequence of decisions and analyses required in determining the PMF for gaged and ungaged basins. PMF studies should follow the procedures specified in the flow chart, unless departures are justified in the study report. Chapter and section references are shown for each flow chart element.

DETERMINING THE PROBABLE MAXIMUM FLOOD FOR CIVIL WORKS
FLOW CHART

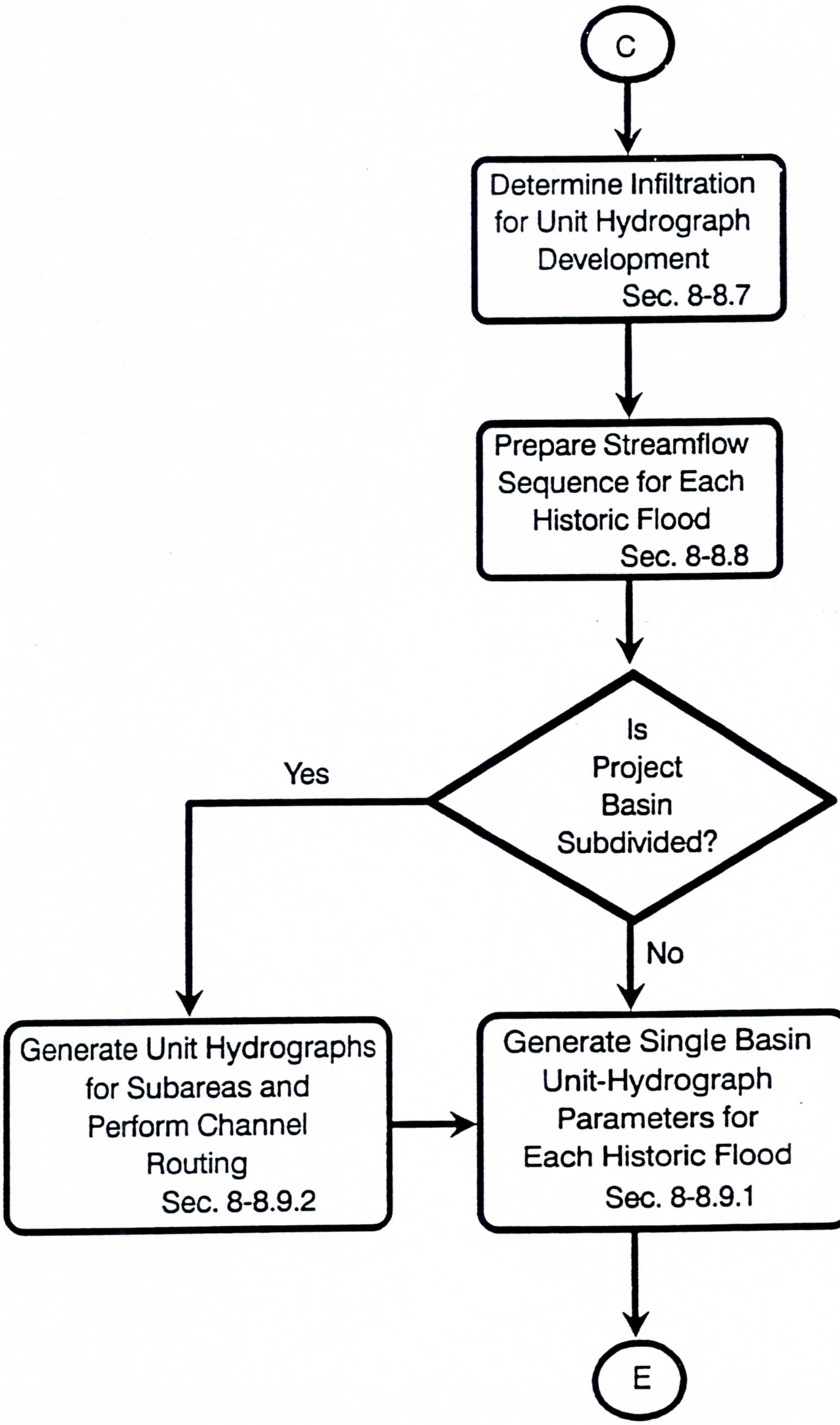



A
Need to Subdivide Watershed?
No
Yes
Subdivide Watershed Into Subbasins
Sec. 8-6
Define Approach and Identify Required Tasks
Sec. 8-7
Are Adequate Data Available in Project Basin?
D
No
Yes
Project Basin is Ungaged
Sec. 8-9
H
Project Basin is Gaged
Sec. 8-8
Choose Historic Floods for Calibration and Verification
Sec. 8-8.1
B

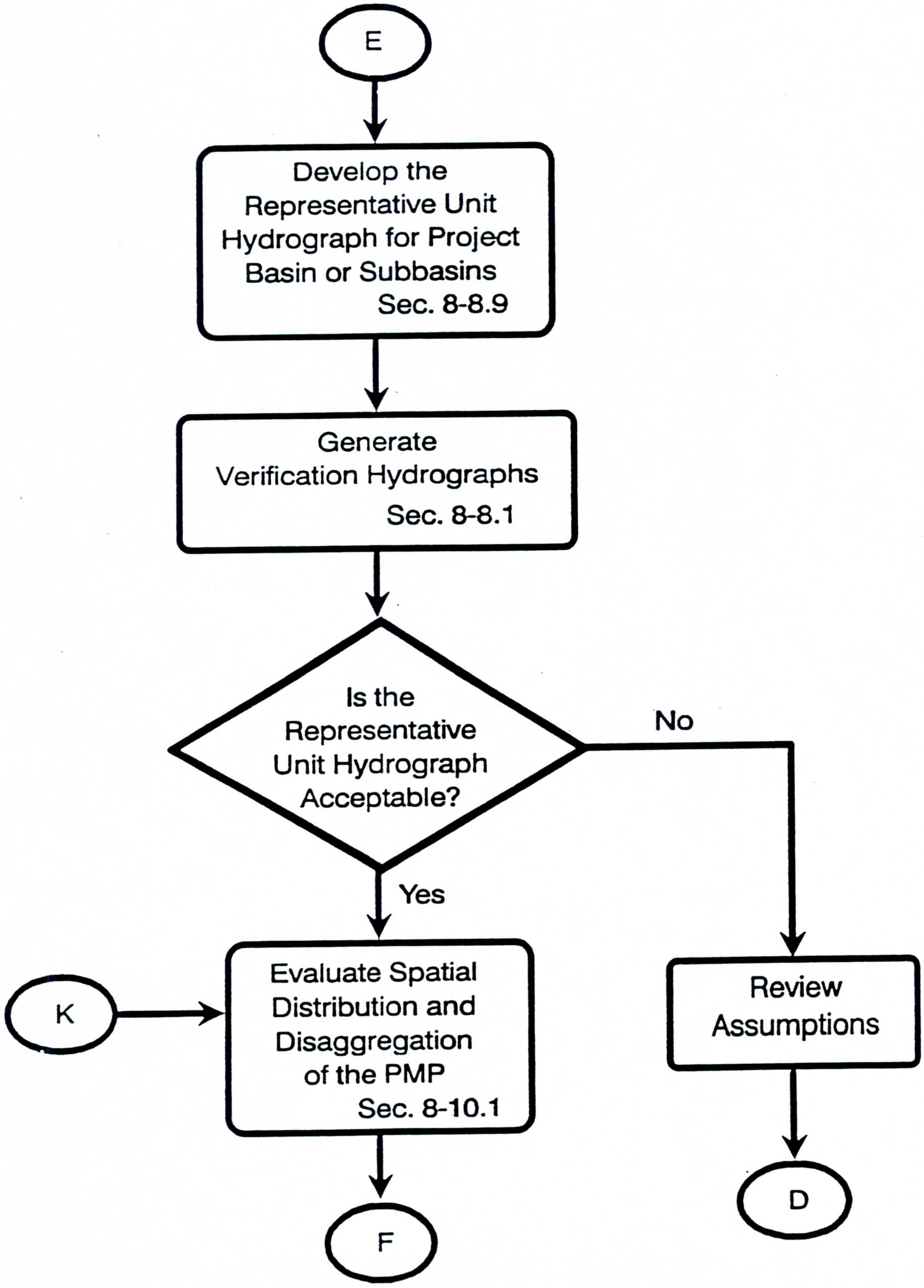
PMF DEVELOPMENT
(Gaged Basins)

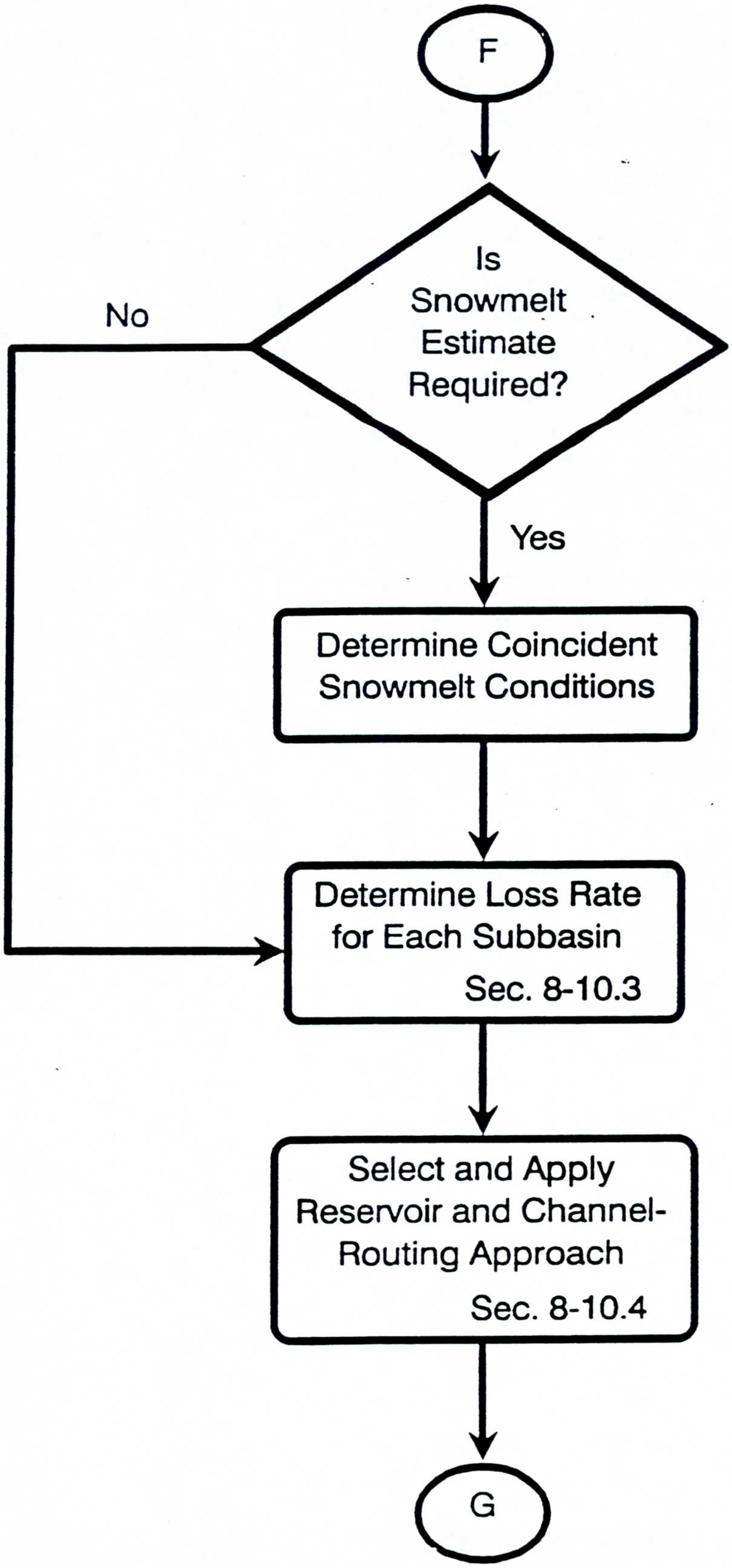


PMF DEVELOPMENT
(Gaged Basins)

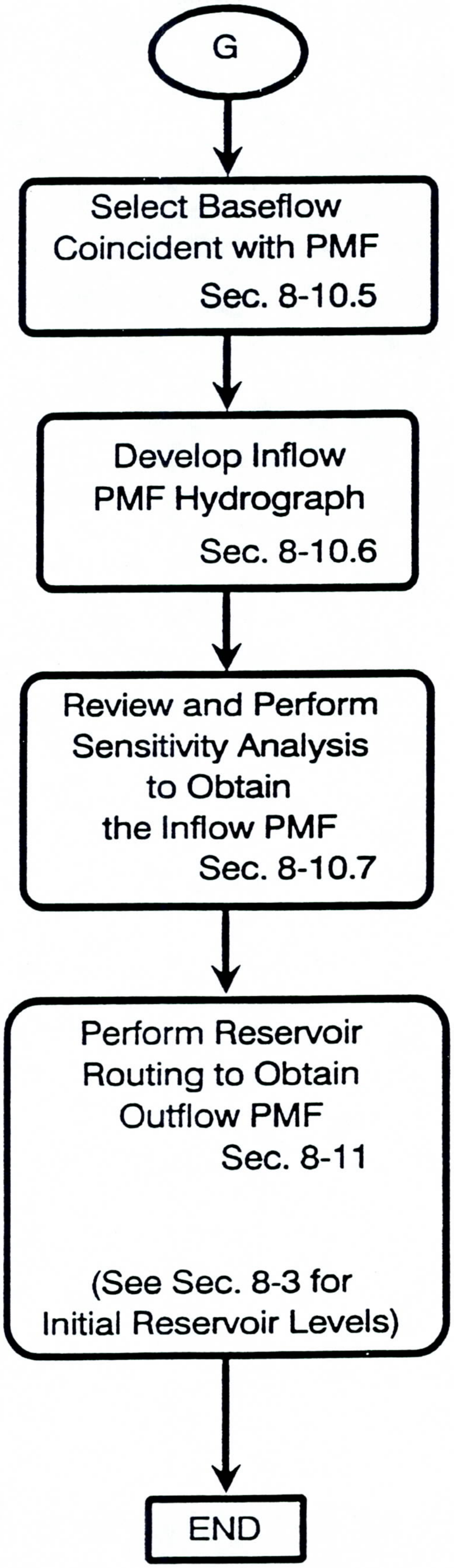


PMF DEVELOPMENT
(Gaged Basins)

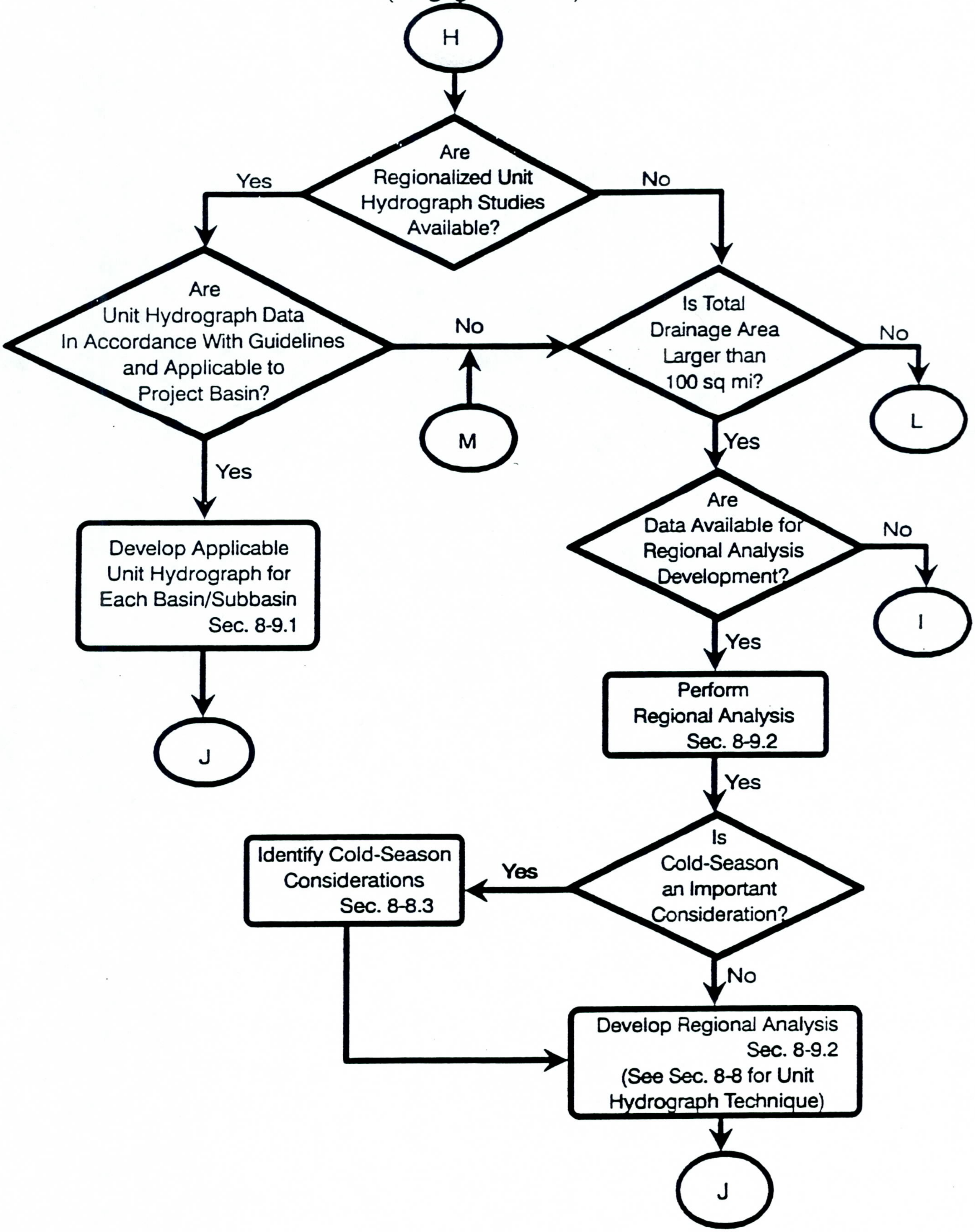



F
Is Snowmelt Estimate Required?
No
Yes
Determine Coincident Snowmelt Conditions
Determine Loss Rate for Each Subbasin
Sec. 8-10.3
Select and Apply Reservoir and Channel-Routing Approach
Sec. 8-10.4
G

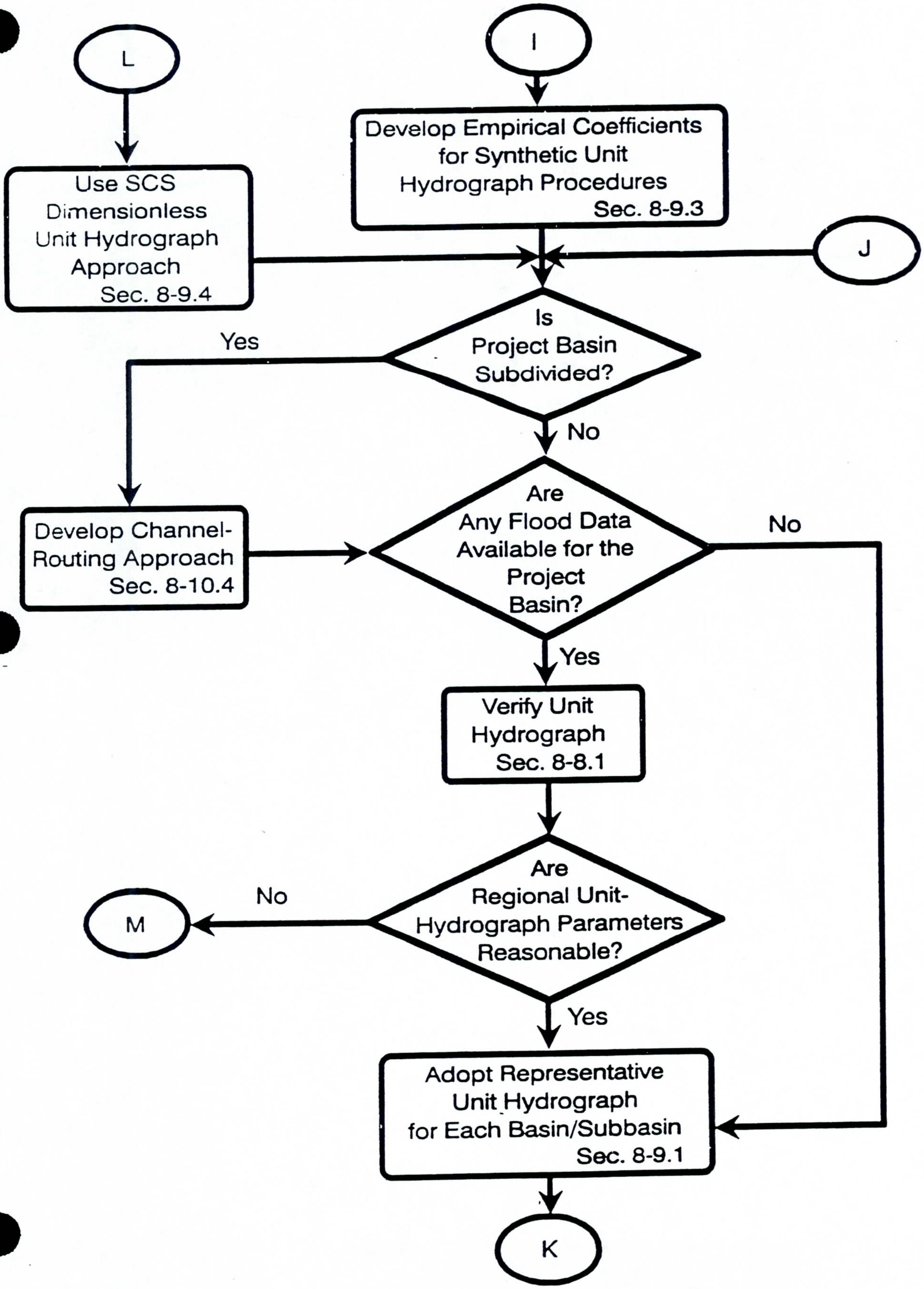
PMF DEVELOPMENT
(Gaged Basins)



PMF DEVELOPMENT
(Ungaged Basins)



PMF DEVELOPMENT
(Ungaged Basins)



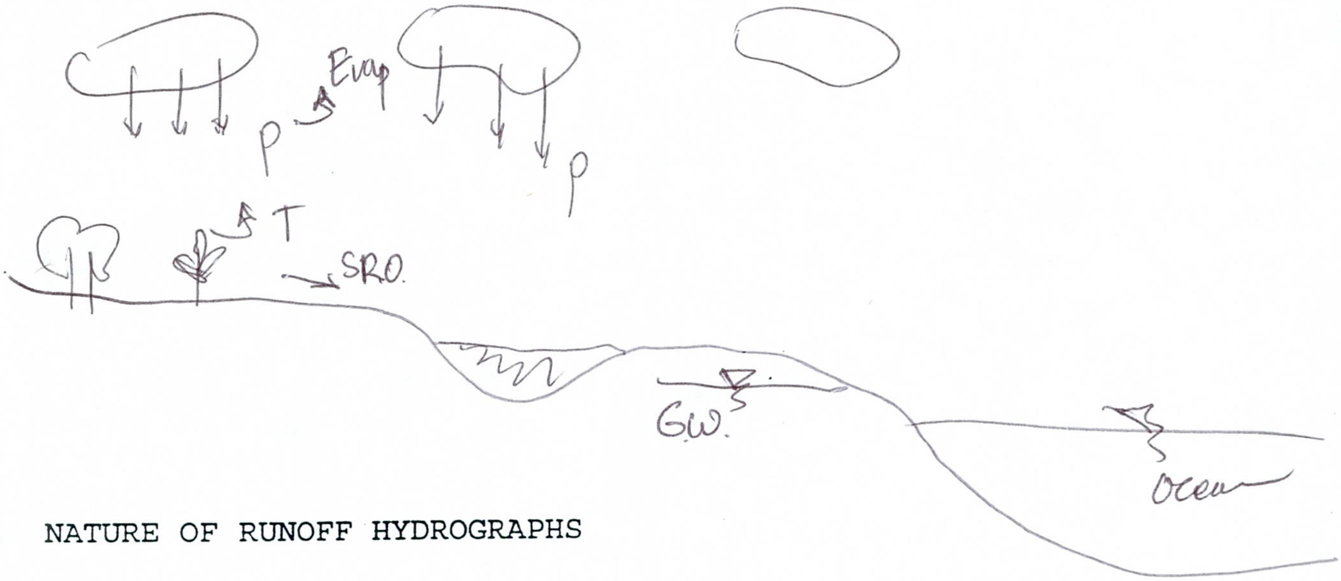
INTRODUCTION TO RUNOFF ANALYSIS
THE HYDROLOGIC CIRCLE

Monday 8:30 a.m.

11/14/94

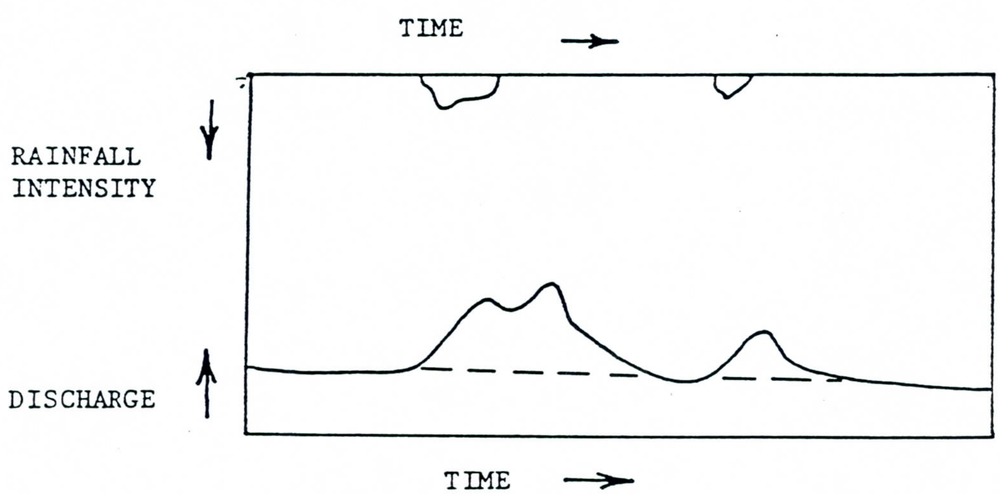
INTRODUCTION TO RUNOFF ANALYSIS

I. HYDROLOGIC CYCLE



II. NATURE OF RUNOFF HYDROGRAPHS

A. Unit hydrograph approach for modeling the rainfall-runoff process

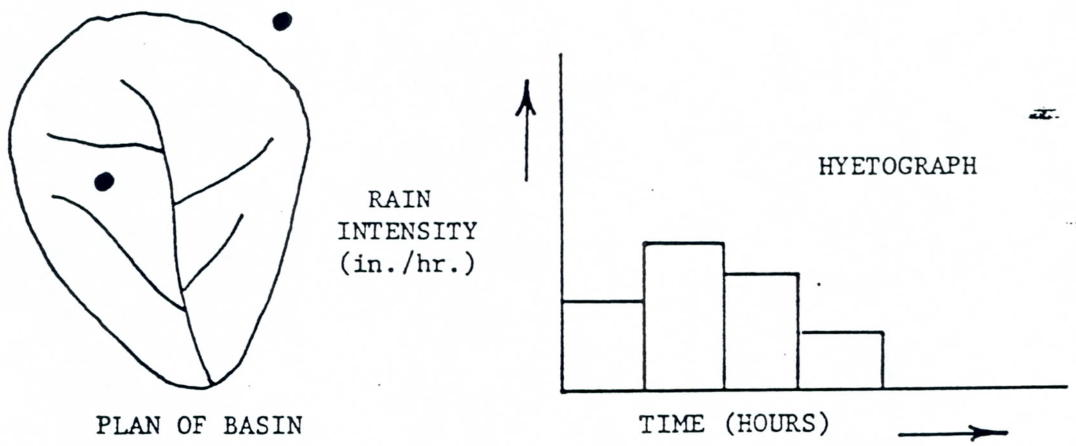


Runoff hydrograph usually consists of a fairly regular lower portion that changes slowly throughout the year and a rapidly fluctuating component that represents the immediate response to rainfall.

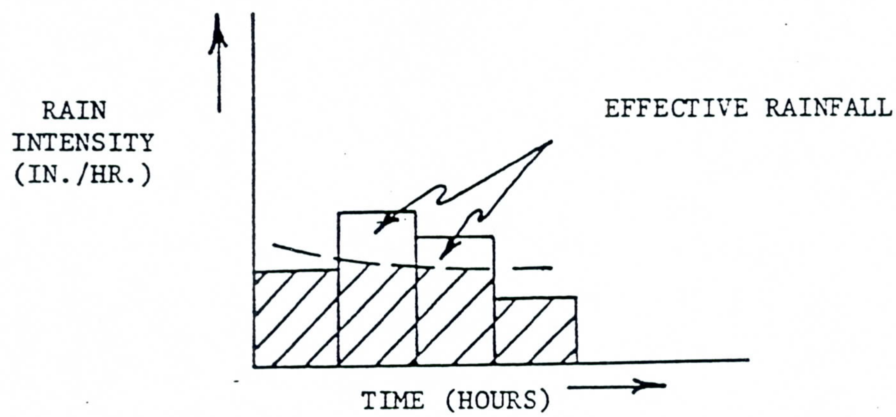
The lower, slowly changing portion of runoff is termed base flow. The rapidly fluctuating component is called direct runoff. This distinction is made because the unit hydrograph is essentially a tool for determining the direct runoff response to rainfall.

3. Overview of Unit Hydrograph Approach

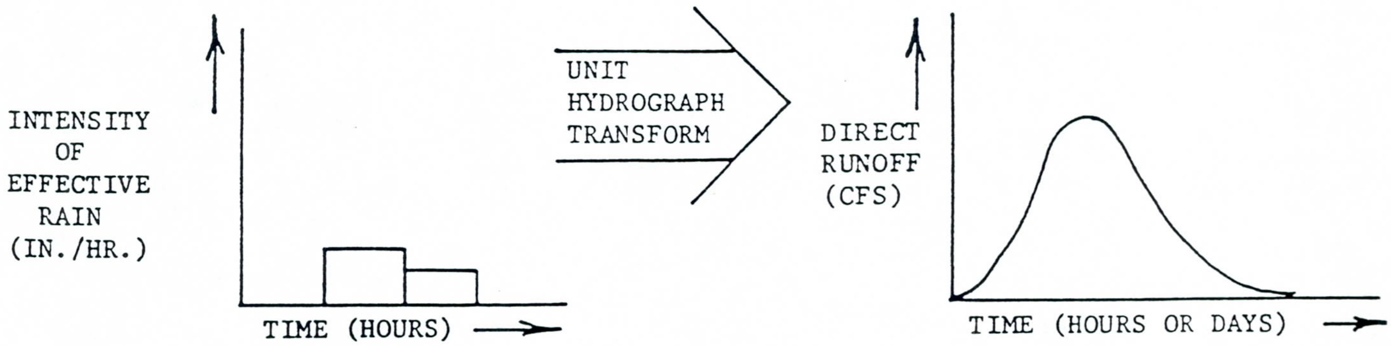
a. Describe rainfall in terms of basin average rainfall, the time-distribution of which is represented by a hyetograph.



b. Estimate "losses" and subtract these from basin average precipitation. The remainder is called effective rainfall.



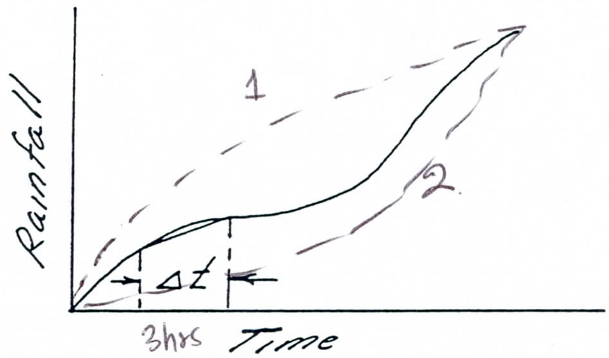
c. Transform the rainfall excess to direct runoff: The transform mechanism is a unit hydrograph.

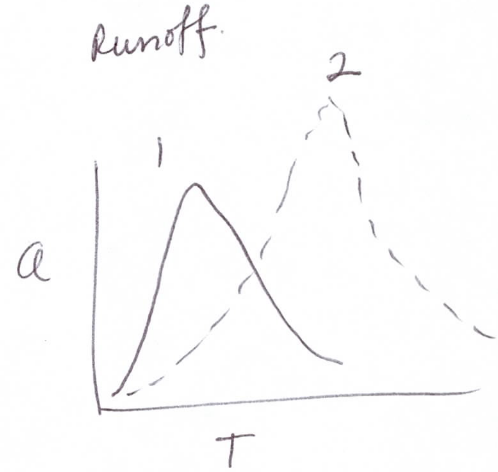


d. Add base flow to direct runoff hydrograph to obtain the desired total runoff hydrograph.

B. Precipitation

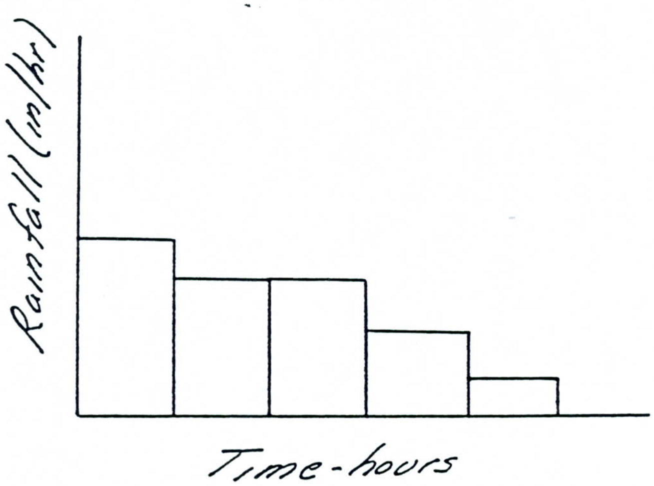
1. Intensity - in/hr (rate)



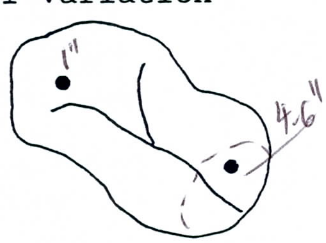


2. Duration -time (usually in hours)

3. Time Distribution



4. Spatial Variation



5. Frequency (e.g. 50-year return period)

BASIN RAINFALL

1. Techniques for Precipitation Analysis - Single Event

a. Two major elements - storm rainfall (termed basin average precipitation) and time distribution (hyetograph)

b. Basin Average Rainfall

(1) Point Sample for Small Basins (0-200 acres)

(2) Arithmetic Average

(a) Rainfall Variation Small

(b) Gage Distribution Near Uniform

(3) Thiessen Method

(a) Fairly Even Terrain

(b) Intermediate Size Basins (200-2000 acres)

(c) Polygon Areas Provide Station Weight

(4) Isohyetal Method

(a) Large Basins (2000 acres and over)

(b) Good on Variable Terrain

(c) Harder to Compute Averages

(5) Combination of Thiessen and Isohyetal Methods

c. Time Distribution

(1) Estimating rain distribution with recording data

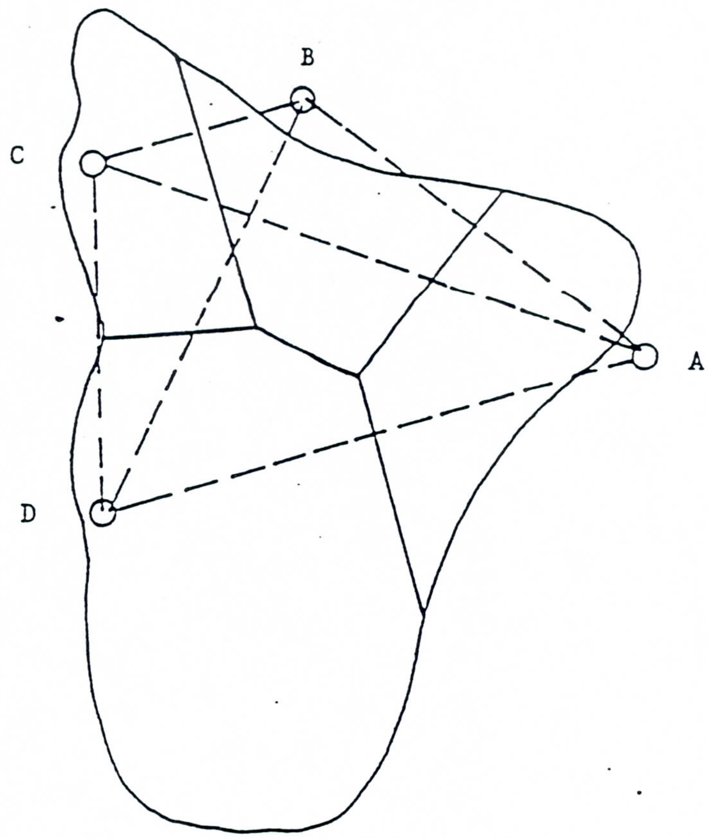
(a) Accumulate recording measurement

(b) Plot recording data (mass curve)

(c) Plot total storm precipitation gage data

(d) Distribute total storm precipitation gage data (hyetograph)

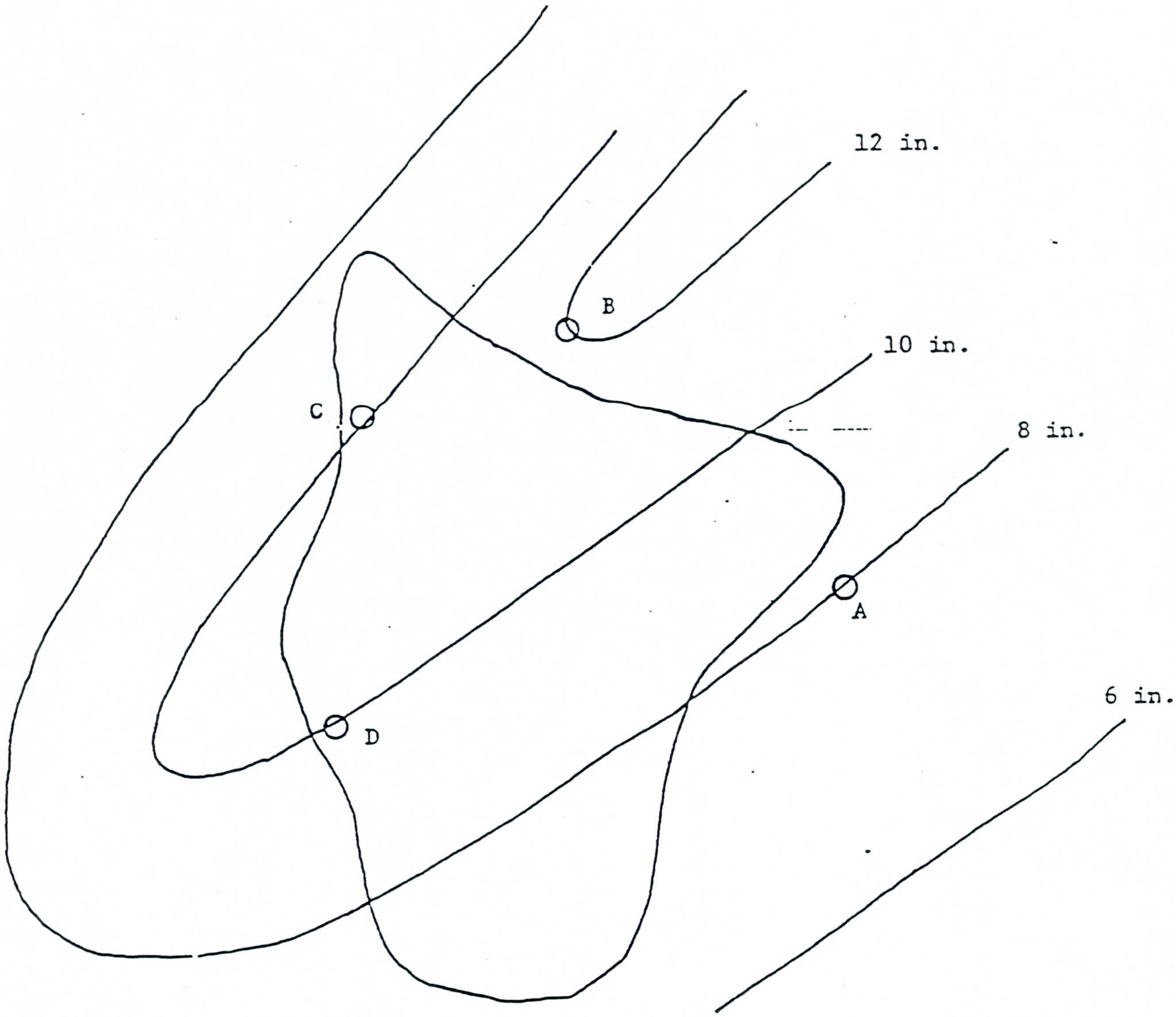
THIESSEN POLYGON METHOD



Station	Rain (in)	Polygon Area (acres)	Volume (acre-in)
A	8	100	800
B	12	88	1056
C	10	64	640
D	10	192	1920
Total		444	4416

$$\text{Basin Mean Rain} = \frac{4416}{444} = 9.94 \text{ in.}$$

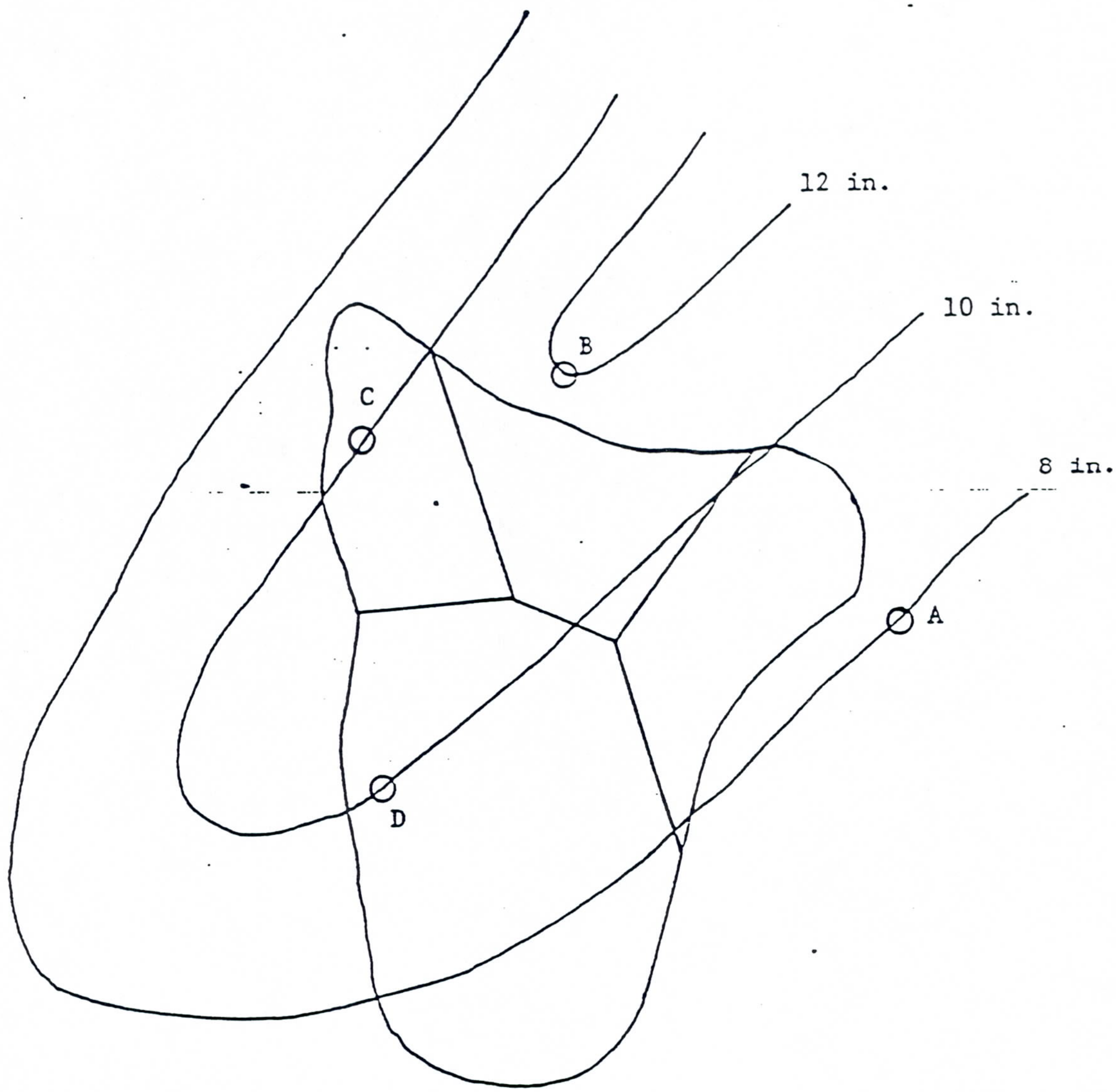
ISOHYETAL METHOD



Isohyet Range (in)	Mean Depth (in)	Effective Area (acres)	Volume (acre-in)
6- 8	7.3	80	584
8-10	9.0	164	1476
10-12	11.0	200	2200
Total		444	4260

$$\text{Basin Mean Rain} = \frac{4260}{444} = 9.59 \text{ in.}$$

COMBINATION OF THIESSEN
AND ISOHYETAL METHODS



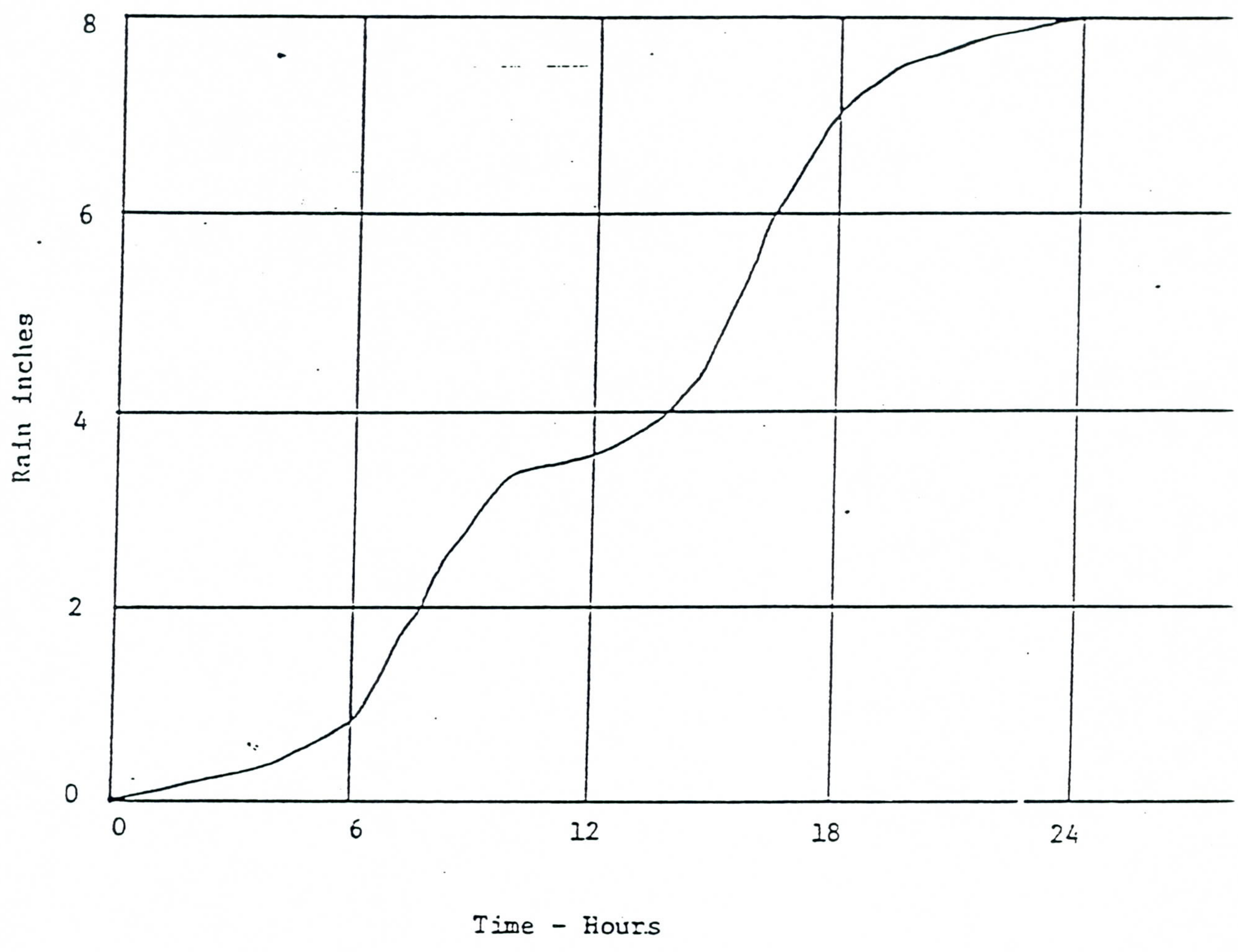
Station	Mean Depth from Isohyets (in)	Area within Polygon (acres)	Volume (acre-in)
A	9.0	100	900
B	11.0	88	968
C	10.5	64	672
D	9.0	192	1728
Total		444	4268

Basin Mean Rain $= \frac{4268}{444} = 9.61$ in.

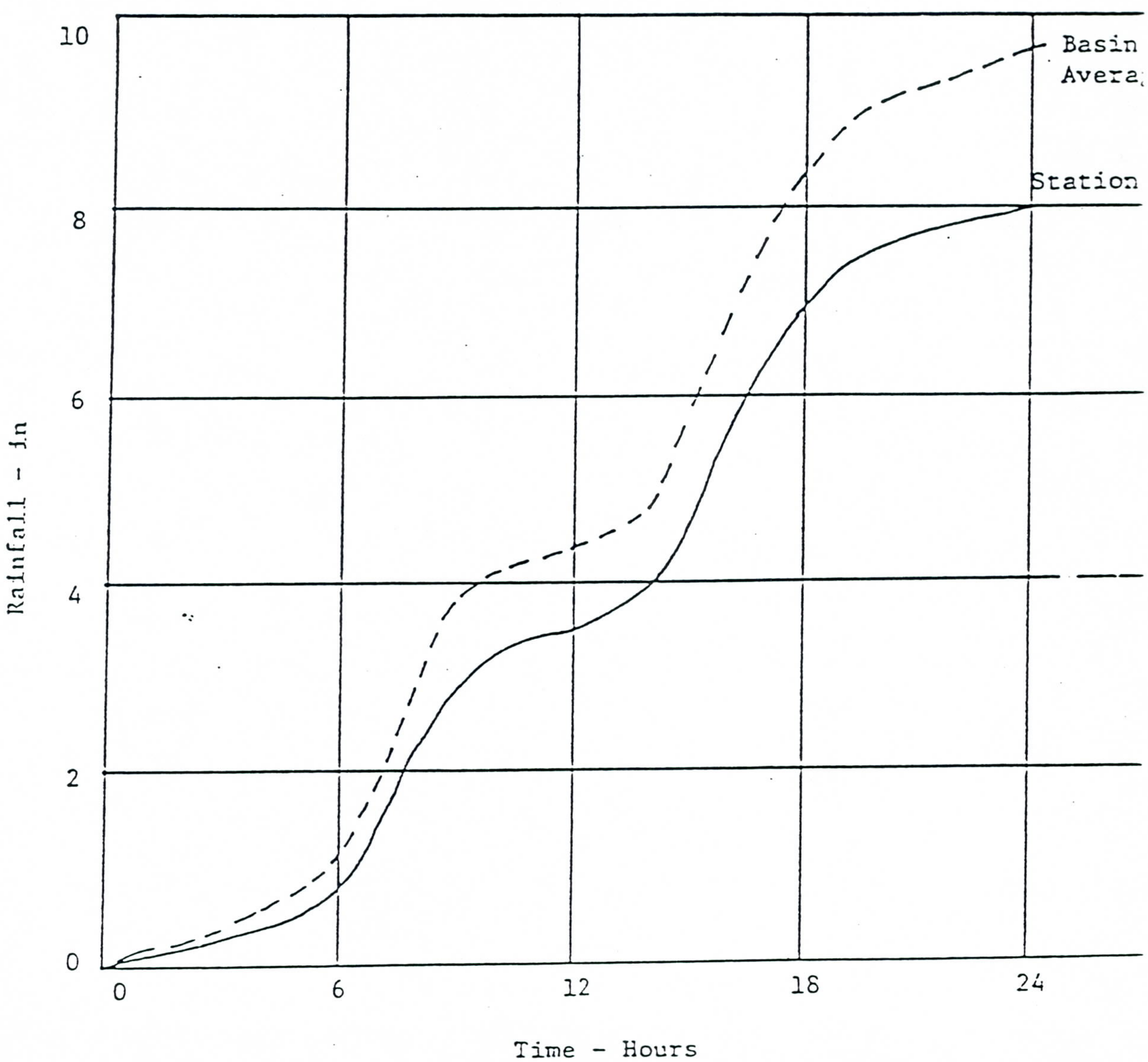
STATION A - Bihourly Rainfall in inches

Time - Hours	0	2	4	6	8	10	12	14	16	18	20
Rain - in	0	.2	.4	.8	2.4	3.4	3.6	4.0	5.6	7.0	7.6

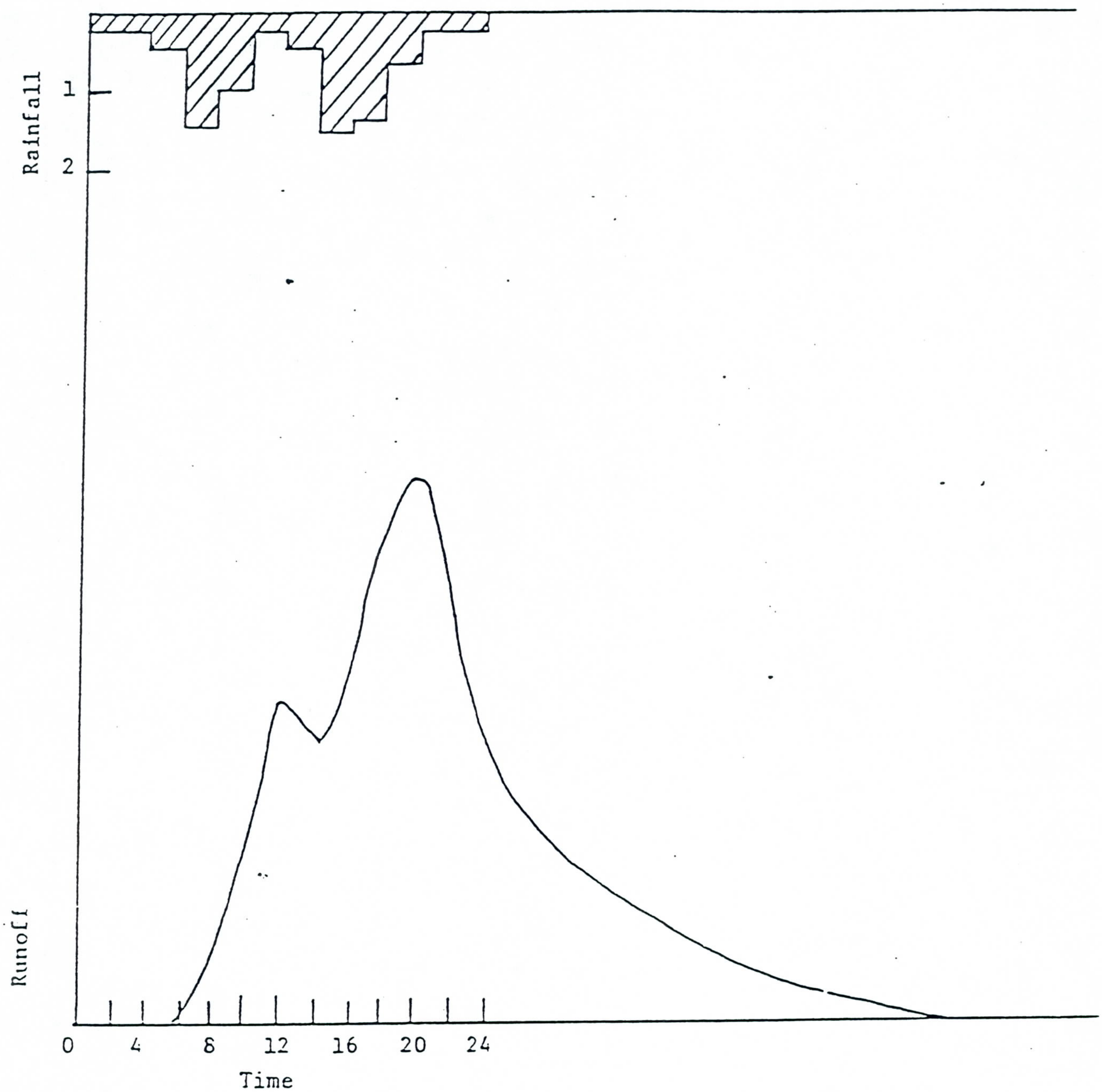
Time - Hours	22	24
Rain - in	7.8	8.0



Time	Gage A		Basin Average	
Hours	ΣRain	Ratio	ΣRain	Inc. Rain
0	0	0	0	0
2	0.2	0.025	0.24	0.24
4	0.4	0.050	0.48	0.24
6	0.8	0.100	0.96	0.48
8	2.4	0.300	2.88	1.92
10	3.4	0.425	4.08	1.20
12	3.6	0.450	4.32	0.24
14	4.0	0.500	4.80	0.48
16	5.6	0.700	6.72	1.92
18	7.0	0.875	8.40	1.68
20	7.6	0.950	9.12	0.72
22	7.8	0.975	9.36	0.24
24	8.0	1.000	9.60	0.24



BASIN AVERAGE - STORM HYETOGRAPH



C. HYDROLOGIC ABSTRACTIONS

1. INTERCEPTION

Part of the storm precipitation that occurs is intercepted by vegetation and other forms of cover on the drainage area. The amount of water is a function of (1) the storm character; (2) the species, age, and density of prevailing plants and trees; and (3) the season of the year.

The importance of interception in hydrologic modeling is tied to the purpose of the model. Estimates of loss to gross precipitation through interception can be significant in annual or long-term models, but for heavy rainfalls during individual storm events, accounting for interception may be unnecessary. It is important for the modeler to assess carefully both the time frame of the model and the volume of precipitation with which one must deal.

If adequate experimental data are available, the nature of the variance of interception versus time might be inferred. Otherwise, common practice is to deduct the estimated volume entirely from the initial period of the storm (initial abstraction).

Percent Interception by Forest Cover	
Spruce-Fir	Birch
32	12

2. DEPRESSION STORAGE

Precipitation that reaches the ground may infiltrate, flow over the surface, or become trapped in numerous small depressions from which the only escape is evaporation or infiltration. The nature of depressions, as well as their size, is largely a function of the original land form and local land-use practices. Because of extreme variability in the nature of depressions and the paucity of sufficient measurements, no generalized relation with enough specified parameters for all cases is feasible.

3. DETENTION STORAGE

The required depth of water to infiltrate the flow of water.

4. EVAPORATION

The moisture that is evaporated during and after a rainfall event. Usually for single event storms, evaporation is neglected.

5. TRANSPIRATION

The water that the vegetation transpires to the atmosphere. Usually for single event storms, transpiration is neglected.

6. INFILTRATION

Infiltration is the process whereby water enters the surface strata of the soil and moves downward toward the water table. The maximum rate at which a soil in any given condition is capable of absorbing water is called its infiltration capacity.

a. Measuring Infiltration - Commonly used methods for determining infiltration capacity are hydrograph analysis and infiltrometer studies.

b. Calculation of Infiltration - Capillary potential is the hydraulic head due to capillary forces. Capillary conductivity is the volume rate of flow of water through the soil under a gradient of unity (dependent on soil moisture content). Until saturation is reached at the surface, the infiltration rate is constant and equal to the rainfall application rate at the surface. As time goes on, the infiltration capacity continues to decline until it becomes equal to the saturated conductivity of the soil, the capillary conductivity when the soil is saturated.

Example of Precipitation
Analysis

FLOOD PLAIN HYDROLOGY

WORKSHOP I
PROBLEM STATEMENT

BASIN PRECIPITATION ANALYSIS

BASIN PRECIPITATION ANALYSIS

PROBLEM DESCRIPTION

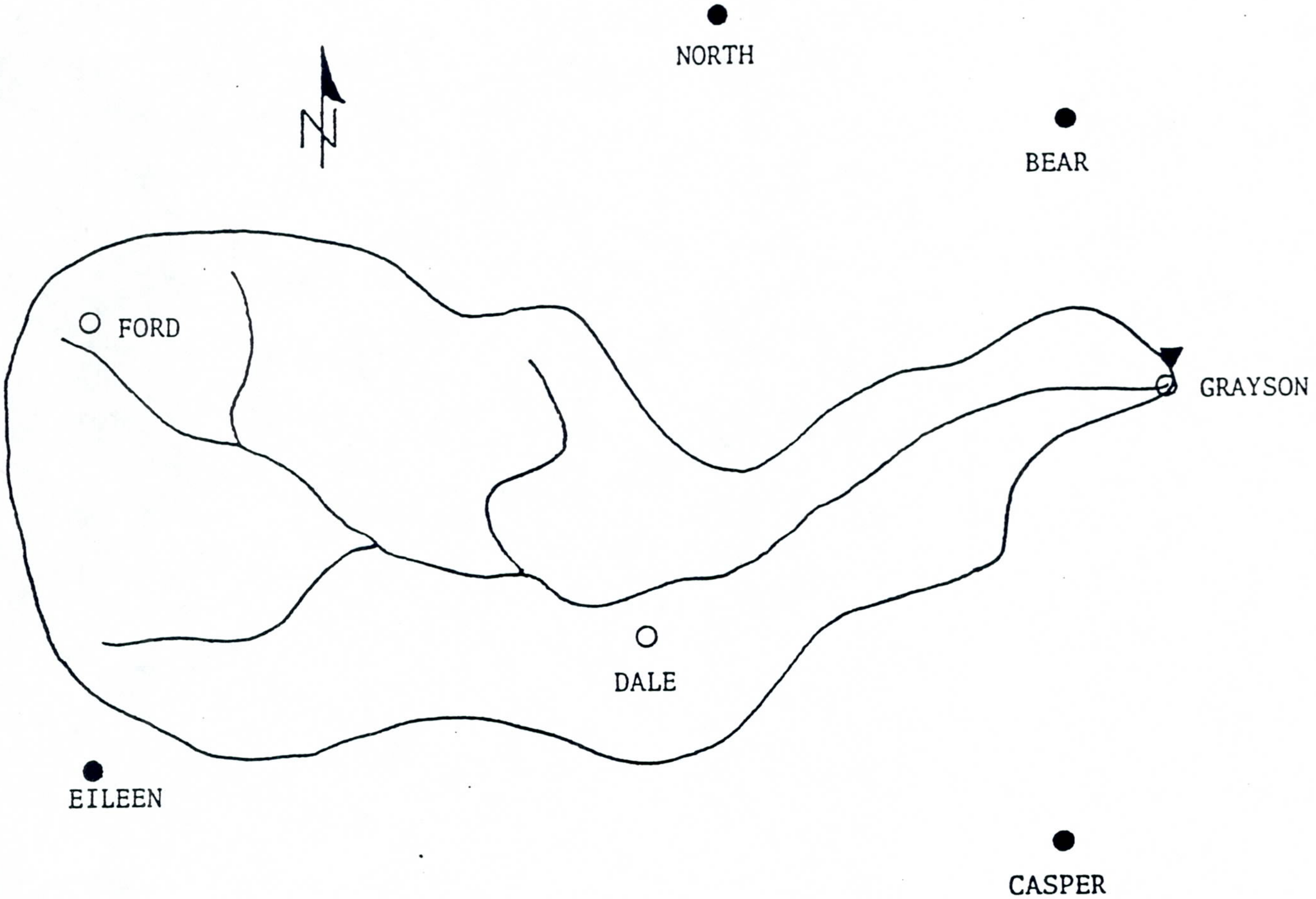
A map of the Clark Creek basin identifying the location of the rainfall gages in or near the basin is shown on page 2. The rainfall data collected at these gages for the October 2, 1962 storm are given on page 3.

PROBLEM

1. Using the total storm rainfall depths, draw the storm isohyetal pattern on the basin map. Use an isohyet interval of 0.5 inch.
2. Construct a network of Thiessen Polygons for the basin.
3. Derive the basin mean rainfall by the following methods:
   a. Thiessen polygon method. (Use the areas tabulated in Table I.)
   b. Combination method. (Use the areas tabulated in Table I.)

TABLE I

Station	Polygon Area
Grayson	10 sq. miles
Dale	45 " "
Eileen	14 " "
Ford	31 " "
	100 sq. miles = Total



CLARK CREEK BASIN

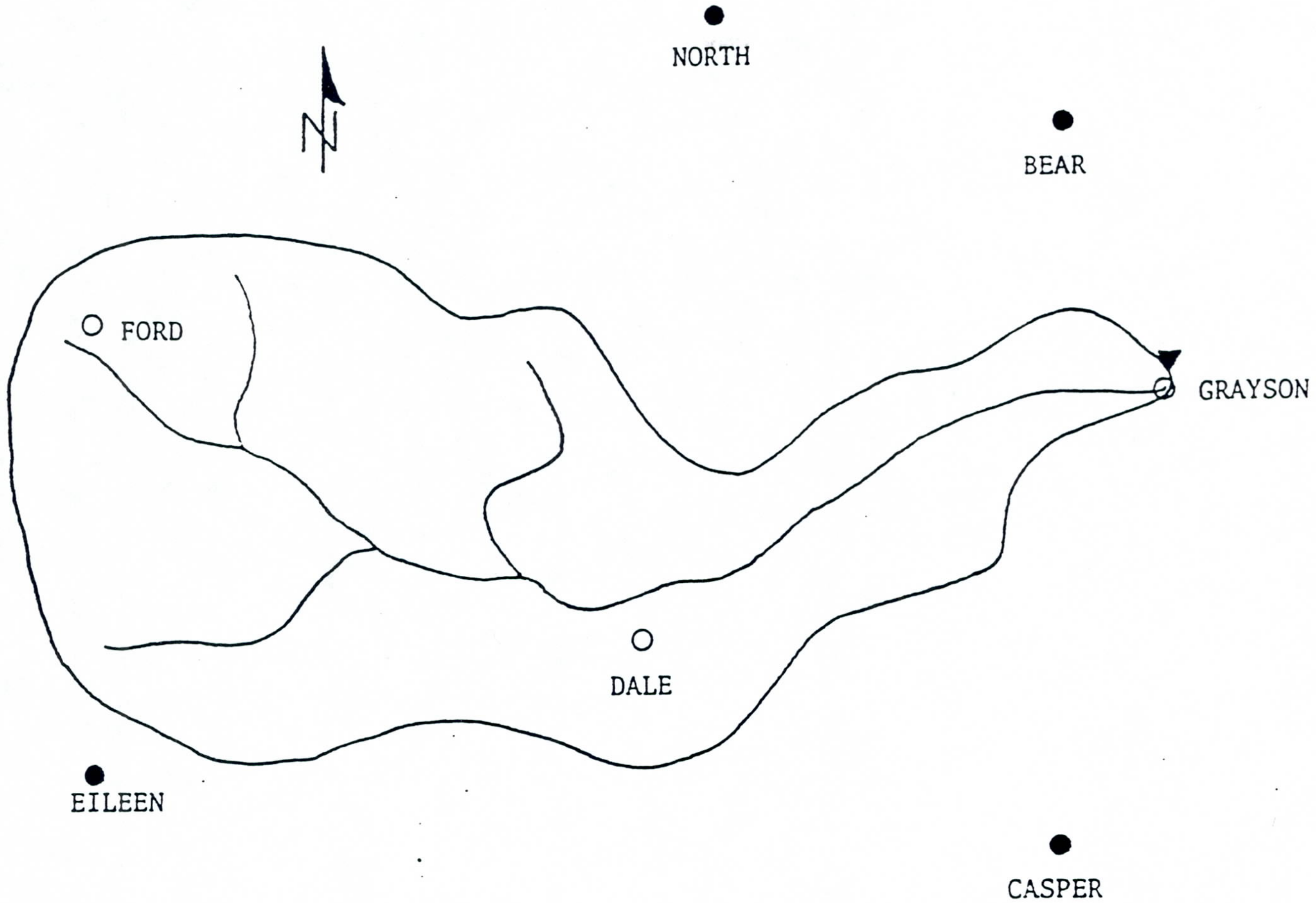
Scale 1:20,000

LEGEND

▼ Stream Gage

● Daily Rainfall Gage

○ Hourly Recording Rainfall Gage



CLARK CREEK BASIN

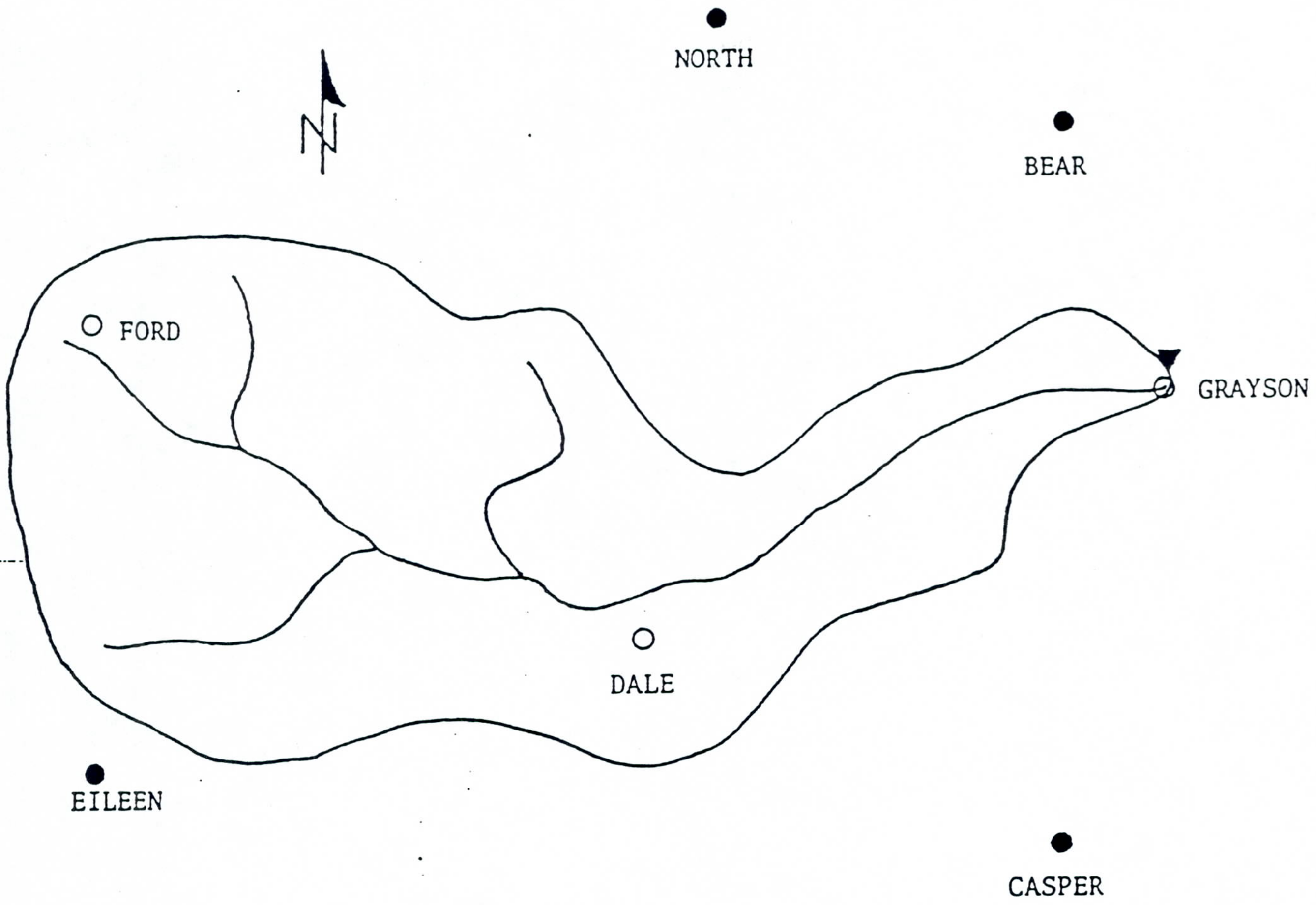
Scale 1:20,000

LEGEND

▼ Stream Gage

● Daily Rainfall Gage

○ Hourly Recording Rainfall Gage



CLARK CREEK BASIN

Scale 1:20,000

LEGEND

▼ Stream Gage

● Daily Rainfall Gage

○ Hourly Recording Rainfall Gage

CLARK CREEK BASIN

RAINFALL DATA

OCTOBER 2, 1962

BIHOURLY RAINFALL CHART READINGS - INCHES

TIME (Hour)	2	4	6	8	10	12	14	16	18	20	22	24
STATION												
DALE	0	0	.25	.87	1.70	2.52	2.65	2.65	2.65	2.65	2.65	2.65
FORD	0	0	.45	1.40	2.68	3.89	4.50	4.50	4.50	4.50	4.50	4.50

DAILY RAINFALL IN INCHES

DATE: OCT	1	2
STATION		
GRAYSON	0	1.40
BEAR	0	2.50
CASPER	0	1.90
EILEEN	0	3.30
NORTH	0	4.00

Solution of Precipitation Analysis

FLOOD PLAIN HYDROLOGY

WORKSHOP I

PROBLEM SOLUTION

BASIN PRECIPITATION ANALYSIS

BASIN PRECIPITATION ANALYSIS

Problem Solution

1 and 2. Total rainfall at each station:

Dale	= 2.65
Ford	= 4.50
Grayson	= 1.40
Bear	= 2.50
Casper	= 1.90
Eileen	= 3.30
North	= 4.00

The storm isohyetal pattern and the Thiessen polygons are shown on page 2.

3. a. Thiessen Polygon Method

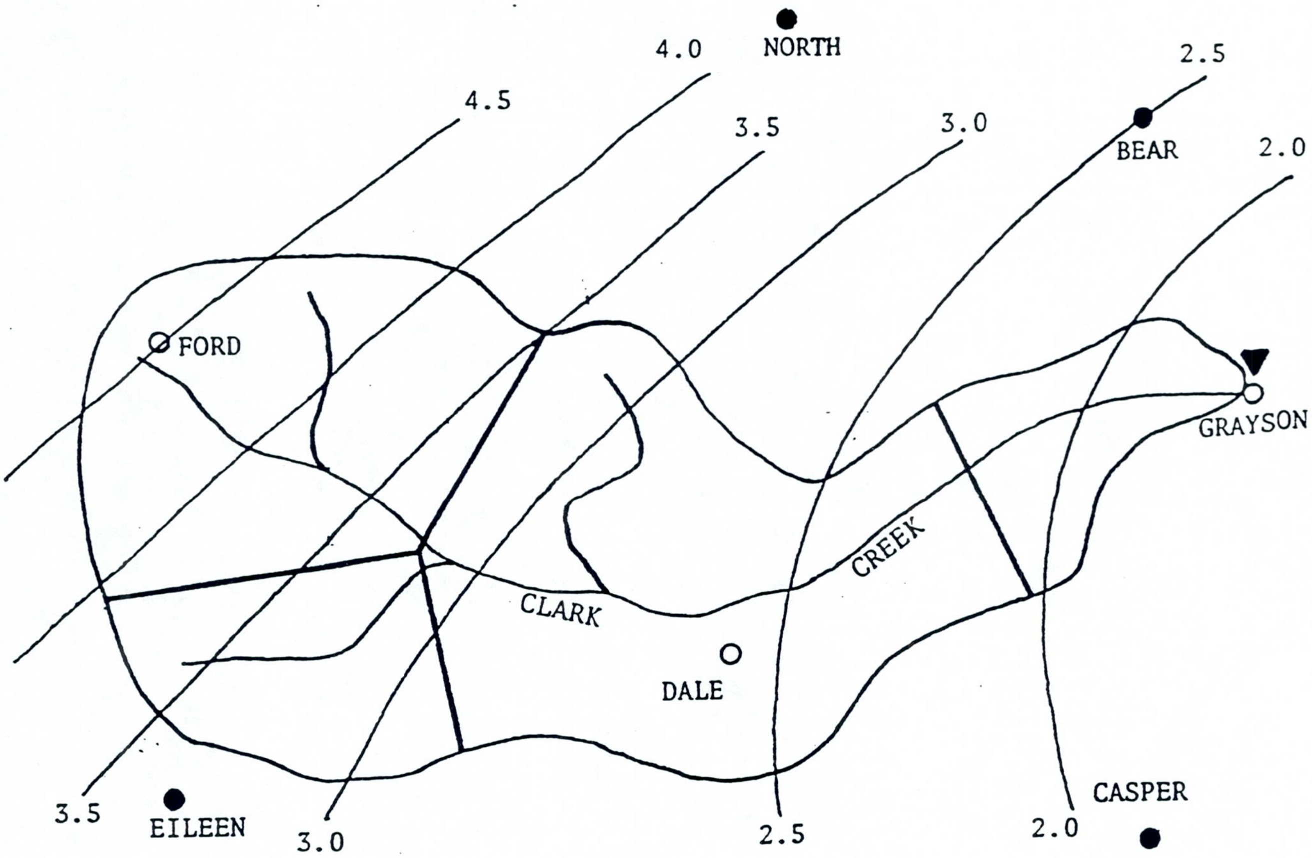
Station	Depth at Station	Area within Polygon	Volume
Grayson	1.40	10	14.0
Dale	2.65	45	119.2
Eileen	3.30	14	46.2
Ford	4.50	31	139.5
			318.9

Basin mean rainfall = 3.19 inches

b. Combination Method

Station	Mean Depth From Isohyets	Area within Polygon	Volume
Grayson	2.0	10	20.0
Dale	2.8	45	126.0
Eileen	3.3	14	46.2
Ford	4.1	31	127.0
			319.2

Basin mean rainfall = 3.19 inches



CLARK CREEK BASIN

Scale 1:20,000

LEGEND

▼ Stream Gage

● Daily Rainfall

○ Hourly Recording Rainfall Gage

PRELIMINARY REVIEW OF HYDROLOGIC DATA

PMF GUIDELINES SECTION 8-2

Monday 9:00

8-2 Preliminary Review of Project and Hydrologic Data

Prior to a site visit, the hydrologic engineer should become familiar with the project and the pertinent hydrology, which will help identify special features that should be observed and the types of data that should be pursued in the field. This section is intended to be an aid in obtaining and reviewing preliminary data.

8-2.1 Identify and Obtain Preliminary Data

General information about the project should be acquired to identify items that should be checked or obtained in the field. Generally, the greater the body of available data, the more confident one can be in the reliability of the final PMF hydrograph. Each project will dictate the level of required data acquisition. Information should include but not be limited to:

- <u>Topographic or site-specific maps</u>. The maps should show the project location, access roads, layout, and drainage area. Topographic quadrangle maps can be obtained from the United States Geological Survey (USGS) as well as from private vendors. In some areas, topographic maps are also available in digitized form from USGS Earth Science Information Centers (ESIC). Special topographic maps, used during dam design and construction or for other studies, are often available from the dam owner. Satellite imagery, available through the National Aeronautics and Space Administration (NASA) can be useful in addressing conditions within the drainage basin.

- <u>Aerial photographs of the drainage area</u>. These are sometimes available from the dam owner, the district offices of the United States Forest Service (USFS) and the Agricultural Stabilization and Conservation Service (ASCS), or local or state transportation agencies.

- <u>Drainage basin soil types for estimating infiltration rates</u>. If the basins have been covered by soil surveys, a State Soil Geographic Database (STATSGO)—which provides soil association maps and related data—will be available in digital form from state offices of the Soil Conservation Service (SCS). These data are also available from the SCS National Cartographic and GIS Center in Fort Worth, Texas.

- <u>Location and history of all stream gages within and near the drainage area</u>. Gage locations are often shown on USGS topographic maps. However, because of location changes, as well as gage closure or renewal in operation, this information should be checked with the district office of the USGS, the COE, state agencies, or the dam owner.

- <u>Stream gage history and readings</u>. These data are available from the USGS Water Supply Papers and Water Resource Data Reports for the state in which the project is located. The USGS also operates the National Water Data Exchange (NAWDEX), which compiles information on availability and source of water data not obtained by or filed by the USGS. Daily streamflow data are obtainable through authorized direct computer access to WATSTORE, the USGS computerized water data storage system, and NWIS-II (National Water Information System will replace WATSTORE and portions of NAWDEX in the near future). Continuous flood hydrographs can be obtained from district offices of the USGS. Data for all stream gages are usually reported to the USGS even though the gage may be operated and maintained by another federal, state, or local agency; the dam owner; or another private party. However, if the historical data for the gages are not obtainable from the USGS, the owner of the gage should be contacted directly. Historic ratings for the gages will be needed and can be obtained from the USGS district office, since they are usually not contained in the annual Water Resource Data Report. Privately owned firms market compact disks containing streamflow data from USGS records. In addition, the dam owner may have streamflow data not available through other sources.

- Location and history of all rain gages that are, or have been, operated in or near the drainage area. This information is available from the NWS National Climatic Data Center (NCDC) in Asheville, North Carolina, and from state climatological agencies, and regional climate centers.

- Rainfall data for rain gages that are, or have been, operated in or near the basin. These data are generally obtainable from the NCDC. Rainfall data are often also available from state water-resource agencies and may be available for major historic storms in special flood studies done by the COE, NWS, Tennessee Valley Authority (TVA), Federal Emergency Management Agency (FEMA), United States Bureau of Reclamation (USBR), SCS, USFS, or other federal, state, or local flood-control agencies. Much of the NWS-stored climatological data are available on compact disks from private vendors. In addition, the dam owner may have streamflow and rainfall data not reported elsewhere.

- Hydrologic data for historic storms and associated floods. A search should be made for this information, which will include the rain gage data (particularly from recording gages) within and near the drainage basin and corresponding flood hydrographs. Offices that may have performed special flood studies for severe floods and have such data on file include the COE; USGS; FEMA; NWS district offices as well as SCS state offices; TVA; and state or local flood-control agencies. Local newspapers and other media sources can sometimes provide useful information, but any such data must be verified before being used.

- Engineering reports that provide information on dam height and type, reservoir capacity-elevation, spillway type and rating, outlet type and capacity, and power-intake capacity. The dam owner is usually the best source for this information; much of it is generally contained in past safety-analysis reports (in the case of existing projects), which will be available from the dam owner or from state or federal dam-safety agencies.

- Information on project operation during past extreme floods. This information can be obtained from the project owner. Obtaining the information may require reviewing project records and interviewing project operators.

- Cross sections for the channels through which the PMF hydrograph may need to be routed. These may be available from FEMA or a local flood-control agency if flood studies have been made for the area. In some cases, cross sections of sufficient accuracy can be taken from 7½-minute USGS maps, but field surveys may be appropriate for some cases.

Caution: Accurate hydraulic-routing computations may require surveyed cross sections. Accuracy requirements are discussed in Section 8-10 under sensitivity analysis.

- Information on land use. Such information be may obtained from USGS topographic maps and local land use maps. Aerial photos are also very helpful for this purpose and are sometimes available from the USFS district offices, local and state transportation agencies, or the SCS state offices. The National Aerial Photography Program (NAPP) is available from the USGS office in Reston, Virginia. Satellite image analysis should be given consideration for cost-effective selection of these data. Field observations are also desirable.

- Information on geologic conditions within the drainage area. Geologic maps are frequently available from the USGS district offices, the SCS state offices, and state departments of natural resources.

8-2.2 Information About Upstream Dams

Any existing upstream dams must be identified and information must be obtained to determine whether or not they create sufficient storage to have an effect on PMF timing and peak flow. Up-to-date topographic maps of the drainage basin will generally show the location of any upstream dams large enough to require consideration. The National Dam Inventory (NATDAM)—available through the FEMA Dam Safety Branch in Washington D.C.—lists height, length, dam type, reservoir volume, date of construction, and ownership for dams in each state. Information desired for each dam includes:

- Type and height of dam, outlet works and spillway type and rating curves, and a cross section and crest profile of the dam. These data may be necessary for routing of a PMF hydrograph.

- Area and capacity versus elevation for the reservoir.

- Interviews with operators of upstream dams, who could possibly affect the timing and peak flow of the PMF hydrograph. Operators should be interviewed for information pertaining to the operation of those dams. Historical and proposed information on operation of the reservoir, spillway, outlet works, and power plants during extreme floods should be obtained.

All information should be reviewed to determine how flood operations have been performed in the past. This information may also be of interest in identifying historic floods for development of unit hydrographs.

8-2.3 Field Visit

Once the preliminary information has been obtained and reviewed, an experienced hydrologic engineer should visit the dam, spillway, outlet works, power plant, and the drainage area to check or confirm information developed in the preliminary review and to obtain firsthand information about the dam, its facilities, and the drainage area.

8-2.3.1 Dam, Spillway, Outlet Works, and Power Plant

The dam, spillway, outlet works, and power plant should be visited to obtain information not available in reports dealing with the site. Such information includes:

- Characteristics of spillways, outlet works, and power intakes.

- Discharge rating curves for each structure. Rating curves should be checked in the field to ensure that they take into account limitations in gate opening, such as orifice flow occurring because a radial gate cannot be opened wide enough to clear the water surface during passage of the PMF.

- Pertinent elevations on spillway and outlet works rating curves. Elevations provided should be confirmed.

- Gate operation. It is particularly important to ascertain that the gates are operable and have been operated under full head in the recent past.

- Flashboards. If an uncontrolled spillway is equipped with flashboards, information should be obtained on their height and the dates on which the flashboards are placed on the spillway and removed. It is also desirable to determine whether or not the flashboards will fail or can be readily released when a flood is imminent.

- Available power and backup systems. Availability of power and the existence of backup systems for operating spillway gates should be ascertained.

- Remote or local operation. It will be necessary to determine if the dam is operated remotely or by local operators and to obtain details and schedules for operation during extreme floods, including access to spillway and outlet facilities.

- Physical features of the dam and its appurtenances. Such information will be necessary for routing the PMF inflow flood through the reservoir and possibly for reverse-reservoir routing of releases to obtain inflow hydrographs for historic floods.

8-2.3.2 Operating Personnel Interviews

Operating personnel should be interviewed. Items of particular interest include:

- Procedures and operation rules for normal and emergency gate operation during extreme floods. An assessment should be made of the reliability inherent in the operation of spillway gates and flashboards, particularly if the project is remotely operated.

- Rule curves for seasonal operation of the reservoir.

- Information on historic floods. Such information includes flood-flow peaks and hydrographs, reservoir levels, lag time between occurrence of storms and arrival of flood peaks at the reservoir, maximum rates of reservoir rise, and rainfall depths and timing.

- High-water marks and eyewitness accounts of operations and events occurring during past floods.

- Procedures and results of spillway and outlet-works gate testing.

8-2.3.3 Drainage Area Assessment

The primary purpose of this assessment is to obtain quantitative information on the drainage area, with special emphasis on identifying all portions that contribute to runoff. To the extent possible, the drainage area should be observed by road. Photographs should be taken to establish a record to aid in later recollection. Previously obtained topographic, soil, and geologic maps; aerial photographs; and satellite imagery should be taken to the field for reference. If there are no roads, or if the drainage area is very large, it may be desirable to fly over the area. Drainage area observations should include confirming or identifying the following:

- Location of rain gages and stream gages.

- Existing upstream dams.

- Special features within the drainage basin such as marshes, lakes, and closed basins that may delay or reduce runoff.

- Manning's "n" and general hydrologic characteristics of stream channel.

- Areas where soil or geologic features could result in locally different rates of infiltration. These include large exposures of rock; areas of high permeability such as karst formations, deep sand, or fractured basalt; cultivated areas; areas of dense forest or managed forest cover; and high-altitude meadows.

- Large natural constrictions that could act as dams.

- Any changes in urbanization, hydrologic use, or land use and cover that may have occurred since surveys for the available topographic maps were conducted, or since the historic floods occurred.

The following may be necessary if peak flow data from the historic floods are incomplete:

- High-water marks along the streams on bridge piers or abutments or along banks. These may be useful in computing a peak flood flow.

- Eyewitness accounts of long-time residents. These will be helpful to obtain information on historic flooding. Verify the accuracy of accounts, if possible.

- Visits to local newspapers and television and radio stations. News reports on historic flooding may be available.

- Visits to the pertinent stream gages and rain gages. These may be necessary if the accuracy of the recorded data is in question.

DEVELOPMENT OF HYDROLOGIC CRITERIA OF THE PMF

PMF GUIDELINES SECTION 8-3

Monday 9:20 a.m.

8-3 Antecedent and Coincident Hydrometeorological Conditions

The inflow PMF hydrograph that produces the critical conditions within the reservoir and at the dam may depend on either the peak inflow rate or the timing and volume of PMF inflow, depending on spillway capacity and reservoir storage available at the beginning of the flood. Thus, the inflow PMF hydrograph could result from a high-intensity local storm, a general storm with a long duration, or a winter storm. This section discusses these considerations and their influence on PMF development procedures.

Caution: Although it may be possible to assess in advance whether the peak outflow and/or the maximum reservoir water-surface elevation will be produced by a local or a general storm, flood inflow hydrographs should be generated for each storm and then routed through the reservoir to clearly establish the PMF event.

8-3.1 Antecedent Conditions

The question has been raised as to whether a PMP flood hydrograph based solely on runoff from the PMP provides sufficiently small risk of exceedance for consideration of dam safety. In general, it does. Severe storms may be preceded by lesser ones; the real question of interest is: What reservoir level is reasonable as the starting elevation when routing the inflow PMF through the reservoir, considering the possibility of antecedent storms? It is advisable to determine if a water resources agency has conducted regional special studies related to antecedent storms. If so, the results should be considered for application. In the absence of antecedent storm information, the following four approaches are recommended as acceptable alternatives:

(1) Consider that the reservoir surface is at a predefined annual maximum level at the start of PMF inflow. It will be necessary to determine the annual maximum reservoir level for each dam, depending on the characteristics of the dam, its spillway and outlet works, and the historic and specified operation plans. If flashboards are normally used on the dam during the time of the PMF, they should be assumed to be in place for the determination of the annual maximum reservoir level. Routing of the PMF through the reservoir should assume that flashboards fail or collapse at their design level.

- For hydroelectric projects, the annual maximum reservoir level should be defined as the annual maximum normal operating level.

(2) Use an operating rule curve, when available, to identify the reservoir surface corresponding to the maximum storage level for the season of the controlling PMP. A 100-year, 24-hour storm—using the percentages of the 24-hour maximum temporal distribution developed for the PMP—should be assumed to end three days prior to the PMP. The runoff hydrograph from this 100-year storm should be routed through the reservoir using established project operating rules, with the beginning reservoir level at the normal maximum storage level for the season. The reservoir level at the beginning of inflow from PMP runoff should be taken as the level produced by the

routed inflow from the 100-year storm, but it need not be greater than the annual maximum reservoir level.

(3) Use or develop a wet-year rule curve to establish the reservoir level that would exist at the start of the inflow PMF. To develop this rule curve, assume that the reservoir level at the beginning of the inflow PMF is at the average of the five consecutive, highest wet-year reservoir levels occurring during the season of the critical PMP. The assumed starting level need not be higher than the annual maximum reservoir level.

(4) Analyze historical extreme floods and antecedent storms for the region. A possible procedure can be found in HMR 56 [NWS 1986]. If the analysis shows it is probable that antecedent storms do occur in the region and could significantly influence the maximum reservoir level and the magnitude of the routed PMF outflow, develop a storm that could reasonably be expected to occur antecedent to the PMP as follows:

   (a) Prepare an arithmetic plot of the antecedent storm rainfall expressed as a percentage of the principal storm versus the principal storm rainfall in inches. Draw an envelope line of the maximum values and extrapolate to the estimated PMP depth.

   (b) Determine the average time between the beginning of the antecedent storm and the following one.

   (c) Read a total rainfall depth for the antecedent storm from plot obtained in step (a) by the total PMP depth.

   (d) Set the time between the antecedent storm and the PMP equal to the average time interval determined in step (b).

   (e) Use both the antecedent storm and the PMP to develop an inflow PMF hydrograph.

Average monthly flow should be obtained for the months during the season when the critical PMP would occur. Tabulated monthly average data are available in USGS water data reports. The average monthly flow for the month of the critical PMP should be added to the inflow PMF hydrograph before routing through the reservoir. When using HEC-1 this initial flow is the parameter STRTQ. For the particular case when the basin has been subdivided, the initial flow will already have been added as described in Section 8-10.5. For "ungaged" basins, the average monthly flow per square mile of drainage area, obtained from records for nearby "gaged" basins, should be used to compute the initial flow.

A reservoir cannot be drawn down at the beginning of the PMF storm when flood routing, unless a drawdown is documented as the normal operating procedure for the reservoir during an impending storm.

8-3.2 Coincident Hydrometeorological Conditions

Assume the pertinent physical conditions of soil-moisture content, frozen ground (see Section 8-10.3.3), and snowpack water equivalent that could reasonably be expected to occur antecedent to the PMP. If snowpack is apt to exist in at least part of the drainage area in the season when the critical PMP would occur, an antecedent 100-year snowpack (covering the area that could be subject to snowpack) should be assumed to exist at the time when the PMP occurs (see Section 8-10.2.1).

DATA ACQUISITION

PMF GUIDELINES SECTION 8-4

Monday 10:00 a.m.

8-4 Data Acquisition

Hydrologic and meteorologic data are necessary to develop unit hydrographs. Primary objectives of data collection are as follows:

- To obtain basic precipitation and streamflow data to use in subsequent analysis.

- To enable the engineer to understand the hydrologic response of the basin to properly simulate the runoff process for the season when the critical PMF would occur.

In general, four types of data are recommended to develop a unit hydrograph, as follows:

- Streamflow records for major historic floods.

- Precipitation records for the storms that produced the historic floods.

- Physical characteristics of the watershed including topography, soil types, and land use.

- Snowpack and temperature records in the basin if snowmelt was a factor in historic floods.

In addition, it is necessary to understand the project's physical features, as well as those of upstream dams, to properly route flood hydrographs through the reservoir. This section describes the specific data needs.

Caution: Delays may be experienced in data collection. These can take the form of extended periods to retrieve data in storage and seasonal weather delays for field data collection. Appropriate time should be allotted (i.e., four to six months) for data collection.

8-4.1 Information from Previous Studies

As stated earlier, unit-hydrograph theory is recommended to develop the PMF inflow hydrograph. Since unit hydrographs are commonly developed and used in flood-control studies, local, state, or federal agencies with flood-control responsibilities may have already developed one for the basin of interest. If available and applicable, the use of such unit hydrographs can save considerable time and cost to develop the inflow PMF. This is particularly true for basins where the available streamflow or rainfall records may be less than desirable—in which case, it may be necessary to develop a new unit hydrograph with more recent data. Thus, it is necessary to search for previous flood studies for nearby dams and to inquire about the availability of relevant information. Sources of information about regional flood studies include:

- Local flood control districts
- COE district and division offices
- USBR regional offices
- TVA

- SCS state and district offices
- USGS district offices
- NWS River Forecast Centers
- State water resources agencies
- State departments of dam safety
- State departments of transportation
- Regional planning commissions or agencies
- Dam owners
- FEMA

All information obtained must be reviewed for quality and applicability; review, assessment, and justification procedures are described in Section 8-5.

8-4.2 Streamflow Data

8-4.2.1 Continuous Streamflow Hydrographs

The location of USGS stream gages, along with average daily flows for the water year, are given in the annual Water Resource Data Report issued for each state by the USGS. The USGS NAWDEX system catalogs sources and types of streamflow data that may not be listed in the Water Resource Data Reports. The search for streamflow data varies depending on whether the basin is gaged or ungaged. For gaged sites, collection is concentrated on the gages within the basin of interest; for ungaged sites, the collection effort is extended to gaged basins in the region. Daily flow records and maximum flows of record for gages in and near the basin can be obtained from USGS annual Water Resource Data Reports. Such data are needed to identify historic floods, which should be considered when developing a unit hydrograph.

To develop a unit hydrograph, streamflow hydrographs will be necessary for the identified major historic floods. Continuous streamflow hydrographs can be obtained from USGS district offices, where stage records for the historic floods and rating curves for the pertinent stream gages can also be obtained if questions about the accuracy of the historic flood records arise during the data review.

- <u>The continuous inflow hydrograph at a project needed for unit-hydrograph determination can be developed by reverse-reservoir routing</u>. This requires knowledge of project outflow and headwater elevations during one or more major floods. Project outflow can be estimated from downstream stream gage records or project discharges (gate operations and power releases).

- Unit hydrographs for a project developed from continuous flood inflow hydrographs developed by reverse-reservoir routing are most accurate for PMF determinations. This is because the effect of the reservoir impoundment on flood flows is directly taken into account [Maidment 1993, Newton 1983].

8-4.2.2 Peak Flow and Volume Data

As discussed in Section 8-4.1, the effort in collecting streamflow data will be greatly reduced if a previously developed unit hydrograph is available for the project basin that satisfies the guidelines in this chapter. In that case, the only streamflow data required will be those necessary to identify the occurrence of antecedent floods. Development of antecedent floods (Section 8-3.1) could require data on both annual-flood peak-flow rates and flood-hydrograph volumes. The necessary streamflow data and flood-peak frequency curves can be obtained from the USGS. In constructing or checking flood-peak frequency curves, flood peaks should be segregated according to cause (e.g., thunderstorm, hurricane, snowmelt, or rain-on-snow). It is particularly important to exclude floods caused by ice jams or dam breaks.

Information about peak rates of flow and the time of peak of past large floods is often helpful when evaluating the reliability of a unit hydrograph. Such information can be obtained from staff gages or crest stage recorders, or from flood marks and other informal flood records often available in special reports about major floods.

8-4.2.3 Monthly Streamflow Data

Monthly streamflow data should be assembled for the season when the PMF would be expected to occur. These data are available from USGS surface water records.

8-4.3 Precipitation Data

To develop the unit hydrograph, it is necessary to obtain precipitation data for the storms that caused the identified historic floods. Precipitation data for rain gages within and near the project basin can be obtained from NCDC or on compact disks from private vendors. Data from continuous recording gages (both within and near the basin) are particularly important in assessing the temporal distribution of rainfall within the basin in the process of developing the unit hydrograph. The altitude and the period of record for all rain gages should be noted. An isohyetal map of annual precipitation should be obtained, if available.

- Single extreme rainfall events should be used to develop the unit hydrograph. A unit hydrograph developed from a complex storm hydrograph (i.e., multiple events occurring back-to-back) can be in error and is difficult to compute, primarily because of baseflow separation.

Data for the periods preceding the historic floods will be required if a special study is made to assess antecedent conditions. Special flood and PMP studies—which may have been performed by the COE, USBR, NWS, or other federal or state agencies—usually contain precipitation data that is more detailed and, in general, more thoroughly reviewed and analyzed than that available from the NWS NCDC or private vendors. It is important to search for information from such studies.

8-4.4 Applicable Hydrometeorological Reports

Knowledge of the hydrometeorology of the basin and its surrounding areas is necessary to calculate the PMF. Applicable HMRs providing PMP estimates for the region often include useful information on record storms and the resulting floods. Sources of these data include:

- NWS
- FEMA
- State water resources agencies
- Local flood-control districts
- Privately funded regional or site-specific studies may have been done for some nearby dams. The results of such studies must generally be obtained from the dam owners.

8-4.5 Physical Characteristics of the Drainage Basin

Some of the parameters commonly used to define a watershed's runoff characteristics include area, elevation, basin slope, land use, basin orientation, and slope of the major watercourse. Most of these parameters can be estimated using topographic maps published by the USGS. Current and past aerial photographs can be very useful in assessing land use.

Caution: Accuracy in determination of these parameters can be a significant source of variability when developing PMF hydrographs.

Information on soils classification within the basin is desirable for use in estimation of applicable infiltration rates and can be determined from soil survey maps for the area as published by the SCS. These STATSGO data are available or in preparation in digital form for all 50 states from the SCS National Cartographic and GIS Center in Fort Worth, Texas. Land use data can be obtained from local government agencies, the USFS, or the United States Bureau of Land Management if federal land is involved. Future land use plans should be obtained and considered in the runoff analysis if it is apparent that potential changes will have a significant effect on runoff characteristics.

8-4.6 Snowpack Water Equivalent and Temperature Data

For sites where snowmelt contribution to extreme floods is common, snowpack water equivalent and temperature data must be obtained. Locations of snow courses, snow pillows, and weather stations in and near the project basin need to be identified and the altitude and period of record for these stations noted. If snowmelt must be considered in the development of a unit hydrograph, both the snowpack water equivalent and hourly and daily temperature data should be obtained for the periods preceding and concurrent with the major historic floods identified in Sections 8-2.1 and 8-4.3. These data may also be necessary to develop snowpack and temperature sequences to be used in computing PMF runoff.

Aerial photographs showing the snowcover pattern throughout the winter and spring seasons are desirable for periods preceding the major historic floods identified in Sections 8-2.1 and 8-4.3, since it will be necessary to define the extent of snowcover for the runoff analysis. The NWS has used aerial photographs to identify the extent of snow-covered areas in some of the north-central states.

Snowpack water equivalent data, as well as SCS SNOTEL data, may be obtained from SCS district or state offices or state water resources agencies. Temperature data is available from the NWS NCDC.

8-4.7 Data on Existing Reservoirs, Spillways, Outlet Works, and Operation Policy

For an existing project, reservoir water levels, spillway gate operation, turbine releases, and tailwater elevations recorded during passage of the identified historic floods should be obtained—particularly if reverse-reservoir routing will be required to obtain inflow hydrographs. The operating policies for passage of extreme floods, which were in force when the historic floods occurred, should also be obtained. To route the inflow PMF through the reservoir, reservoir area-capacity data, rating curves for spillways and outlet works, and flood-operation policy must be obtained from the dam's owner.

The rate of sediment deposition in the reservoir should be assessed, to determine whether the flood-storage capacity of the reservoir has been reduced. Historical information on sediment deposition may also be used to predict loss of active storage in the future if sediment accumulation has been significant.

Caution: It is important to note the date when this information was developed, since changes in active reservoir storage capacity or modifications to spillway and outlet works may have occurred in intervening years.

REVIEW AND ASSESSMENT OF DATA

PMF GUIDELINES SECTION 8-5

Monday 10:20 a.m.

8-5 Review and Assessment of Data

Before using the data obtained in Section 8-4 to develop the PMF for the project basin, the data must be reviewed for accuracy and adequacy. The selection of antecedent conditions as addressed in Section 8-3 will be assessed in relation to data collected in Section 8-4 and applied in Section 8-10. This section discusses the review processes and acceptance criteria.

8-5.1 Unit Hydrographs

Any unit hydrograph available from a previous study for the project basin or from a regional study must be reviewed and tested for its ability to reproduce major flood hydrographs. The best means of proving applicability of the unit hydrograph is to use it to reconstitute the largest of the historic flood hydrographs chosen for review.

- If the reconstituted flood hydrograph agrees well with the historic flood hydrograph, the unit hydrograph can normally be accepted without adjustment. Acceptance will depend on the historic flood magnitude and is further discussed in Section 8-10.

- If the available unit hydrograph does not reasonably reproduce major floods or is judged not to do so due to changes in basin characteristics or error in the assumed time distribution of rainfall excess, a new unit hydrograph will be required. Unit-hydrograph development is discussed further in Sections 8-8 and 8-9.

Caution: It is important to determine the magnitude and importance of the flood hydrographs that were used in producing the unit hydrograph. If the floods used were not of major significance, the unit hydrograph may not accurately predict the peak and timing of major floods. Compensating for such nonlinear effects is considered in Section 8-10.

8-5.2 Flood Data

The first task in the review of the flood data is to ensure that the historic floods used are the largest for which records are available. They should be the maximum floods of record and should preferably have occurred during the season of the critical PMP.

- It is important to note the cause of the floods (e.g., thunderstorm, general storm, hurricane, snowmelt, or rain-on-snow).

Caution: Floods caused by ice jams, debris blockage, or dam break should not be used in unit-hydrograph analysis.

Flood data must be reviewed for accuracy. The flood hydrographs should be plotted to detect discontinuities and suspicious peaks or lows in the recorded flow. Historical ratings, including methods used to extend the range for extreme floods, should be reviewed to make certain that the conversion of recorded stage to discharge was done correctly. Original stage records can usually be obtained from the local USGS district office or the gage owner if questions arise regarding accuracy of recorded flood flows.

- If a slope-area method was originally used to extend the rating curve, a check should be made to ensure that control did not shift to another location during the flood. This may require a computed water surface profile for the reach.

- If questionable aspects of the flood data cannot be resolved, the data should not be used further in unit-hydrograph development.

- If changes in watershed characteristics have occurred since the time of the historic flood, adjustments may be necessary to adequately model the new situation. For example, if the percentage of a watershed's impervious area has changed, the input to the runoff model can be adjusted to reflect the new percentage. Clearcutting of large areas of forests may require changes in both initial abstractions and infiltration rates to reflect changes. Such land use changes will affect the unit hydrograph as well as losses.

If no floods have been recorded within the basin of interest, flood records from other basins in the region will need to be evaluated for applicability to unit-hydrograph development. This procedure has been covered separately in Section 8-9.

- Ideally, unit hydrographs should not be developed from storms that produced less than 1 inch of runoff.

  Caution: Noncontributing areas may cause average runoff over the total drainage area to be less than 1 inch. Special studies may be required to develop an appropriate unit hydrograph.

8-5.3 Precipitation Data

Hyetographs for each storm at each recording rain gage should be plotted and examined for consistency, continuity, accuracy, and completeness. Storm totals and the time distributions for all rain-gage records should be compared to detect obvious inconsistencies. Gaps in records can usually be filled by using regression and correlation analysis with records from nearby gages. An isohyetal map of total rainfall for the storms of interest should be prepared using all acceptable rain-gage records. The location of individual isohyets, for zones obviously influenced by orographic effects, can be drawn parallel to elevation contours when the density of rain gages is insufficient to clearly define the rainfall pattern throughout the area. The general pattern should be compared to mean annual or 100-year isohyetal patterns, which can be obtained from Technical Paper 40 or National Oceanic and Atmospheric Administration Atlas II, published for individual states by the NWS.

Comparisons of the hyetographs and the flood hydrographs should be made to identify suspicious differences in timing between a storm's beginning and end and the rise, recession, and peak of the flood hydrographs.

- If a major timing difference is noted, additional study of the original recorded data records should be performed.

- The hyetographs from nearby rain gages should be checked to determine if the timing difference is due to a clock problem with the rain gage or the stage recorder.

- Rainfall records at the gage should also be analyzed to detect any trends that may coincide with changes in locations of gages or in conditions around them.

- Double-mass analysis or regression methods may be used to adjust rain-gage records to remove spurious trends and produce a homogeneous rainfall record.

Caution: Timing adjustments should not be made to the records unless the irregularity is minor or the source of the error can be positively identified.

The lag time should be measured as the elapsed time between the centroid of the hyetograph and the peak of the flood hydrograph. Other definitions of the lag time are often used and some are included in the Glossary.

Because most rain gage records will be available only as daily totals, the records from the most appropriate recording gage(s)—usually the nearest gage with a complete record—should be used in disaggregating daily records to the required temporal distribution. In assembling daily records it is important to note the time at which each daily gage was read, so that all daily totals can be adjusted to a common daily total.

8-5.4 Snowpack Data

Snowpack data will be required for those basins where snowmelt has been or may be a contributing factor to major floods. The required snowpack-related data include the portion of the basin covered by snow, water equivalent of the snow depth, and hourly or maximum average daily temperatures.

8-5.4.1 Water-Equivalent Data

Snowpack water-equivalent data for snowcover that existed during historic storms should be reviewed for completeness, consistency, and adequacy. Adequacy is determined by plotting the recorded snowpack water-equivalent depths against elevation. It is necessary to decide if data are sufficient to define an altitude-depth relationship for the basin, including the lowest elevation of snowcover for mountainous regions.

- If data are available from only one snow course in the basin, which is often the case, data from other basins with a similar orientation and exposure should be obtained.

- If applicable data from other snow courses are not available in sufficient quantity at different altitudes, undefined portions of the altitude-snowpack estimate can be proportioned in accordance with the isohyetal maps for annual basin rainfall.

- It is possible to reconstitute snowpack data for historic floods through the use of runoff models such as the Hydrological Simulation Program-Fortran [Crawford and Linsley 1966] or the Sacramento Model [Burnash, et al. 1973]. If no snowpack is available, but is required to study the historic floods, such a procedure may be necessary.

8-5.4.2 Temperature Data

Temperature data should be reviewed for accuracy and for applicability in analyzing historical snowmelt.

8-5.5 Data on Reservoir Volume, Spillway and Outlet-Works Capacity, and Operation Policy

Data on the operating history and performance characteristics of the spillway and outlet works, as well as on the reservoir storage volume, are required. Knowledge of operating policies during extreme floods will also be required for routing the inflow PMF hydrograph.

8-5.5.1 Reservoir Volume

Data for reservoir area and volume should be reviewed for accuracy and possible changes occurring since the relationship was formulated.

- Available data on sediment deposition in the active storage of the reservoir should be reviewed to assess the need for adjustment of the reservoir area and volume characteristics.

- If measured data are not available, visual observations of the reservoir's upper reaches should be made.

- If deposition in the active storage area at the head of the reservoir appears to be significant, an estimate of the deposited volume should be made using whatever data can be readily assembled.

- Unless the volume of deposition is large, its effects on the PMF hydrograph will not be important. However, if the reduction in active storage volume appears to be 5 percent or greater, a survey of sediment deposited in the active storage volume and the development of new reservoir area-elevation-capacity curves should be considered.

- If it appears that deposition exceeds one percent per year, an allowance should be made for future deposition between the time the survey is made and the time the next inspection is due.

8-5.5.2 Spillway and Outlet Works Capacity

The relationships for capacity of spillways and outlet works should be checked in accordance with available discharge coefficients for tested hydraulic structures, such as those given in the COE Hydraulic Design Criteria [COE 1989]. For unusual spillway crest shapes, the USBR publication "Discharge Coefficients for Irregular Overfall Spillways" [Bradley 1952] and the "Handbook of Hydraulics" [King and Brater 1954] provide additional guidance. Because approach conditions and site-specific geometry can affect the magnitude of discharge coefficients, precise agreement should not be expected.

- If differences of 10 percent or more are apparent, the source of the original discharge-capacity estimates should be reviewed.

- If adequate physical model studies have been made for the structures to experimentally determine the discharge relationships, they can be accepted.

- If such studies have not been made, values from verified references of discharge coefficient should be used for routing of the PMF inflow. Checks should also be made to determine if any structural modifications that could have produced a change have been made.

- A check should be made to ascertain that a common datum has been used for elevations of reservoir levels and the dam's appurtenances.

8-5.5.3 Operation History and Policy

Data on historical operation should be reviewed for correctness, especially if these data will be required to determine historical inflow floods by reverse-reservoir routing. The location of the reservoir stage recorder should be evaluated to ensure that measured stages are not influenced by drawdown due to spillway or outlet works operation or wind-generated waves.

- If stage records are available for any other location on the reservoir, the records should be compared to detect any inconsistencies, which will also aid in assessing the degree to which the reservoir surface is sloped during passage of extreme floods.

It is necessary to review operation policy and procedures for the passage of extreme floods to develop criteria to be used in routing the inflow PMF.

- If it is possible for operators to be present at the project and to perform the required operations during the PMF, and if redundant operation systems exist, assume that gates and valves that have been tested under head can be operated as proposed during flood passage.

- If gates and valves that would be operated during passage of an extreme flood have not been tested under head to ensure their operation, it will be necessary to make a detailed evaluation of their condition and reliability. Assumptions on the operation of the gates during passage of a PMF should then be made based on that evaluation.

- If the gates are operated remotely, it is necessary to assess the reliability of operation that can be expected during an extreme flood. Operations during historical floods should be reviewed to determine whether the operational policies have been consistently applied.

Spillways equipped with flashboards or stoplogs must be reviewed to determine the operation policy relative to their installation and removal. In addition, if the flashboards are designed to fail or collapse, it will be necessary to obtain detailed information on their structural design. The head at which the flashboards will fail or collapse must be checked.

- If the flashboards are designed to be tripped, the tripping operation should be reviewed to ensure that it can be accomplished at the planned time during passage of an extreme flood.

- If the spillway is sometimes blocked with stoplogs that must be removed manually, it will be necessary to determine if sufficient warning time and the needed equipment would logically be available to allow for removal.

- It is important to consider the possibility that a spillway or outlet works may be at least partially blocked by debris. The degree to which debris has been handled successfully during past major floods should be assessed. If a debris-handling operation plan that has worked successfully in the past is in place, it is acceptable to assume that blockage will be insignificant during passage of the PMF.

SUBDIVISION AND DRAINAGE AREA

PMF GUIDELINES SECTION 8-6

Monday 10:40 a.m.

8-6 Subdivision of Drainage Area

To accurately simulate runoff from the drainage area using unit-hydrograph theory to ultimately obtain a PMF inflow hydrograph, it may be necessary to:

- Subdivide the drainage area.
- Develop unit hydrographs for each subbasin.
- Perform runoff calculations for each subbasin.
- Appropriately route or combine the resulting flood hydrographs from each subbasin.

This section provides guidance for deciding if drainage area subdivision is necessary and for defining a subbasin. Subdivision may be necessary for large basins that are not hydrologically homogeneous or are drained by more than one major tributary. When records for the identified historic floods of interest are available for more than one stream gage in the basin, subdivision is usually advisable.

8-6.1 Evaluation of the Need to Subdivide

In the process of developing a unit hydrograph, and ultimately a PMF inflow hydrograph, the calculations are made using average lumped conditions for the area. If parts of the drainage area have hydrologic conditions that differ significantly from the basin average, subdivision should be considered. In such cases, separate analysis of the subbasins can improve the ultimate accuracy in developing a PMF inflow hydrograph. Subdivision of large basins is also required to properly simulate the effects of spatial distribution of precipitation.

Caution: Improvements in accuracy are possible only if the available data are adequate to define the hydrologic characteristics of each subbasin and to facilitate the required flood routing calculations.

Subdivision should be considered if there are subbasins in the drainage area that:

- Possess hydrologic characteristics obviously different from the average characteristics of the total basin. Examples include shape; large urban sections in an otherwise undeveloped drainage area; areas of unusually high infiltration rates such as those of fractured basalt; closed subbasins; large areas of dense or managed forest in an otherwise clear drainage area. Such hydrologic characteristics can be identified from examination of soil maps, geological maps, topographic maps, and aerial photos and from field visits.
- May contribute to delays in flood passage such as marshes, lakes, or high-altitude meadows.

- Experience significantly greater or less rainfall than the basin average due to orographic effects or to spatial characteristics of local storms. Such areas are best identified through study of isohyetal maps for individual storms and average-annual rainfall.

- Are controlled by large natural constrictions that can act as dams by restricting cross-sectional area and attenuating water flow.

- Are upstream of dams with sufficient storage to affect the peak flow rate and the timing of floods at the point of interest. Subdivision should definitely be considered if operational and streamflow records exist for the upstream dam for the historic floods of interest.

- Have a total drainage area large enough that it may not be covered by a single storm.

- Do not contribute to runoff from the basin.

- Have significantly steeper or flatter slopes than is typical for the basin.

If the reservoir area is large, subdivision may be advisable to allow consideration of direct precipitation on the reservoir surface.

8-6.2 Subdivision for Snowmelt

When snowmelt is known to be important for both historic floods and the PMF, the area covered by snowpack may need to be considered as a subbasin. The subbasin covered by snow may have different infiltration rates than the rest of the drainage area.

APPROACH TO TASKS FOR PMF DEVELOPMENT

PMF GUIDELINES SECTION 8-7

Monday 11:00 a.m.

8-7 Approach to Tasks for Probable Maximum Flood Development

The approach and identification of tasks for PMF development depend on whether available hydrologic and meteorologic data records for stations within the basin are sufficient to provide for confidence in developing the PMF hydrograph. If not, the existing records must be supplemented with data or unit-hydrograph information from other sources. The basin hydrometeorologic and runoff characteristics also have a role in defining the types of analysis required for the PMF development. The choice of procedures is governed by data availability and an understanding of the hydrologic processes of the project basin developed through interpretation of the data collected (Section 8-4) and reviewed (Section 8-5).

As was noted earlier, unit-hydrograph theory is recommended for use in developing the PMF hydrograph. As was pointed out in Section 8-6, it may be desirable to subdivide the basin to adequately treat hydrologic differences within the basin. Some, or all, of the required subbasins may not have stream gage records at their outlets, which can be used to develop a unit hydrograph for the subbasin. For cases where the basin is subdivided, a runoff model must be developed that will incorporate the unit hydrographs constructed for each subbasin, as well as the computations necessary to route and combine flood flow from the subbasins to produce the required PMF hydrograph.

8-7.1 Definitions of Gaged and Ungaged Sites

For the purposes of this chapter, "gaged" and "ungaged" sites are defined as follows:

- Gaged Site: One for which available hydrologic data, recorded at stations within the basin, are sufficient in quantity and quality to provide confidence in development of an inflow PMF hydrograph.

- Ungaged Site: One for which available hydrologic data, recorded at stations within the basin, are insufficient in quantity and quality to provide confidence in development of an inflow PMF hydrograph.

It will be necessary to assess available data and determine whether a site can be considered "gaged" or "ungaged" to establish the recommended methodology to be used in computing the inflow PMF hydrograph.

8-7.1.1 Gaged Site (Sufficient Data Available from Gages Within the Basin)

For the purposes of this chapter, a gaged site should meet the following requirements:

- At least one stream gage, with available flood records, should be located within the basin, preferably at the inlet to the reservoir. If the gage is located downstream of the dam, sufficient historical operational data must be available to allow reverse-reservoir routing to develop inflow hydrographs for each recorded historic flood.

- At least one rain gage—preferably a recording gage—with complete, correct, and consistent data, should be located within the project basin.

- If records for only one rain gage are available, the catch of that gage should be representative of average basin rainfall.

- There should be concurrent records of runoff and basin rainfall for at least three, but preferably five, severe storms.

- Preferably, the historic storms should be for the season of the critical PMF.

- The storm and flood records available for PMF development should have the following characteristics:

  — All runoff-producing parts of the watershed should have contributed runoff.

  — The floods selected for analysis should preferably not be snowmelt dominated, unless it is apparent that the PMF will also be dominated by snowmelt.

  — The storms should have generated substantial runoff. Ideally, the flood hydrograph should have at least one inch of runoff from the contributing area, since there may be nonlinear effects that violate unit-hydrograph theory. However, this condition will frequently not be satisfied. Adjustments of unit hydrographs for nonlinear effects are discussed in Section 8-10.

8-7.1.2 Ungaged Site (No or Limited Data Available from Gages Within the Basin)

For the purposes of this chapter, a basin should be treated as "ungaged" if it does not meet the criteria given in Section 8-7.1.1.

- If available data include less than the desired number of storms and corresponding flood hydrographs, all available data from within the basin should still be used to the extent possible in the unit-hydrograph development and supplemented, as justifiable, with data from other drainage basins in the region. The general rule is that all site-specific data are potentially valuable and should be evaluated for use.

- If no rain gages are located within the basin but flood data are available, rainfall data from nearby stations can be used if a review indicates that the data—and the results of their use in reconstituting a historic flood hydrograph—are acceptable.

8-7.2 Definition of Approach and Identification of Tasks

Once the basin is judged as "gaged" or "ungaged," the approach to developing the PMF inflow hydrograph will be defined accordingly. However, there will be different degrees to which available data within the basin can be used. The following briefly describes the approach, depending on available data:

- Sufficient streamflow and rainfall data of satisfactory quality are available for confidence in developing a unit hydrograph (gaged site). In this case, the approach will be to subdivide the basin as necessary and to use available data to develop the necessary unit hydrographs and the PMF inflow hydrograph. Details of the approach are described in Section 8-8.

- Stream gage records for major historic floods are available, but available rainfall data are insufficient to develop a unit hydrograph. In this case, applicable rainfall data from an adjoining basin should be used. The test for applicability of this transposed rainfall data will be whether or not it allows satisfactory reconstitution of a historical flood hydrograph.

- Available streamflow data are insufficient to provide confidence in developing a unit hydrograph satisfactory to develop a PMF inflow hydrograph. In this case, it will be necessary to follow the guidelines for "ungaged" sites as described in Section 8-9. If any data for major historic floods are available in the basin (e.g., gages, flood marks, informal flood records), they may be valuable in verifying the unit hydrograph's applicability.

- In some cases, where the "ungaged" approach is indicated, it may be possible to use a unit hydrograph developed in other studies or generalized unit-hydrograph parameters developed in regional studies. This possibility is discussed in Section 8-9.

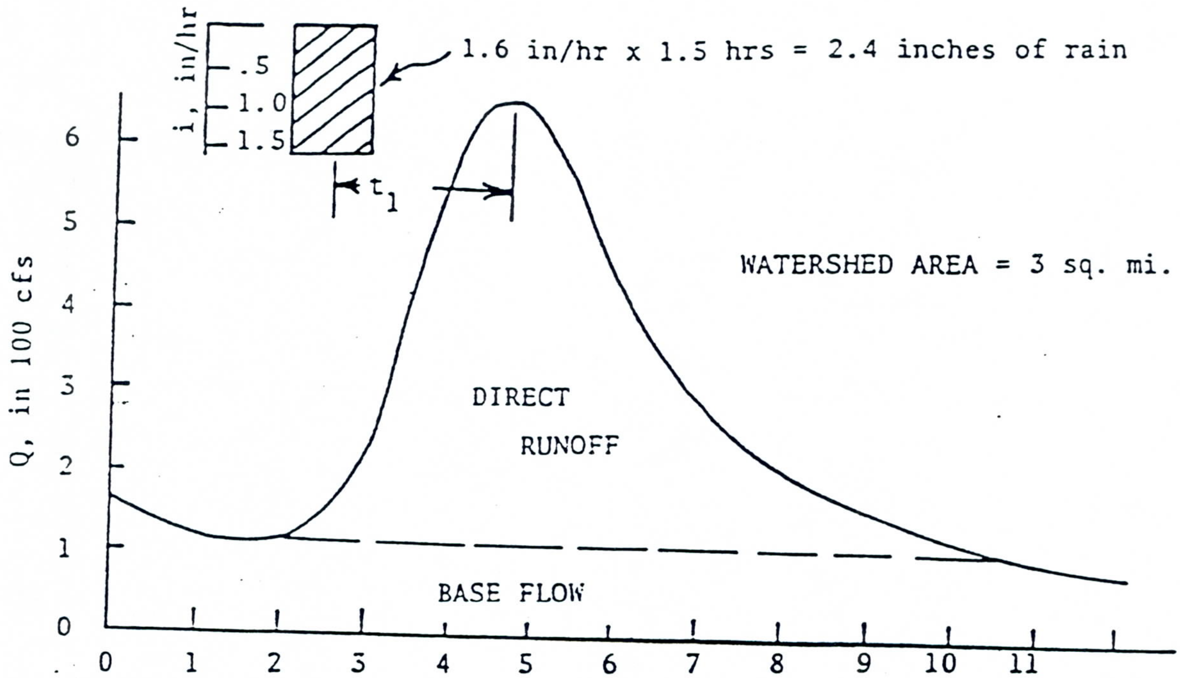
RELATION OF UNIT HYDROGRAPHS

TO S-CURVE HYDROGRAPHS

TIME IN HOURS	COMPUTATION OF S-CURVE HYDROGRAPH FROM KNOWN 12-HOUR UNIT HYDROGRAPH			COMPUTATION OF 6-HOUR UNIT HYDROGRAPH FROM 12-HOUR S-CURVE HYDROGRAPH		
	12-HOUR UNIT HYDROGRAPH (3) PLATE I IN C.F.S.	12-HOUR S-CURVE HYDROGRAPH (2) PLATE I IN C.F.S.	12-HOUR S-CURVE HYDROGRAPH (1) PLATE I IN C.F.S.	12-HOUR S-CURVE HYDROGRAPH (1) SHIFTED SIX HOURS	RUNOFF FROM 0.5 INCH R_e IN 6 HOURS (COL. 4-COL.5)	6-HOUR UNIT HYDROGRAPH [2(COL.6)] IN C.F.S.
1	2	3	4	5	6	7
6	900		900		900	1,800
12	3,400		3,400	900	2,500	5,000
18	6,900 +	900	7,800	3,400	4,400	8,800
24	10,100	3,400	13,500	7,800	5,700	11,400
30	12,300 +	7,800	20,100	13,500	6,600	13,200
36	13,600	13,500	27,100	20,100	7,000	14,000
42	13,900	20,100	34,000	27,100	6,900	13,800
48	13,200	27,100	40,300	34,000	6,300	12,600
54	11,800	34,000	45,800	40,300	5,500	11,000
60	10,300	40,300	50,600	45,800	4,800	9,600
66	8,950	45,800	54,750	50,600	4,150	8,300
72	7,650	50,600	58,250	54,750	3,500	7,000
78	6,400	54,750	61,150	58,250	2,900	5,800
84	5,250	58,250	63,500	61,150	2,350	4,700
90	4,200	61,150	65,350	63,500	1,850	3,700
96	3,200	63,500	66,700	65,350	1,350	2,700
102	2,280	65,350	67,630	66,700	930	1,860
108	1,580	66,700	68,280	67,630	650	1,300
114	1,100	67,630	68,730	68,280	450	900
120	750	68,280	69,030	68,730	300	600
126	500	68,730	69,230	69,030	200	400
132	300	69,030	69,330	69,230	100	200
138	150	69,230	69,380	69,330	50	100
144	50	69,330	69,380	69,380	0	0

*All discharges are instantaneous values at the end of the hour designated in Column 1. Drainage area equals 1290 square miles. See, also, Plate No. I

TECHNIQUES FOR DERIVING UNIT HYDROGRAPH



UNIT HYDROGRAPH DERIVATION

TIME UNITS	Q cfs	BASE FLOW cfs	DIRECT RUNOFF cfs	U.H. = DR ÷ P_e cfs/in	REMARKS
1	110	110	0	0	
2	122	122	0	0	Start of Rain
3	230	120	110	79	
4	578	118	460	328	
5	645	116	529	378	
6	434	114	320	229	
7	293	112	181	129	
8	202	110	92	66	
9	160	108	52	37	
10	117	106	11	8	
11	90	90	0	0	
12	80	80	0	0	
			1755		

Direct runoff volume -- 1755 cfs x 1.5 hrs = 2632 cfs hrs = 2632 ac·in

Precipitation Excess, P_e = Direct runoff depth = $\dfrac{2632 \text{ ac·in}}{3 \text{ sq.mi.} \times 640 \text{ ac/mi}^2}$

P_e = 1.37 inches = 1.4 inches

Uniform rainfall excess intensity = 1.37/1.5 = 0.9 in/hr

Rainfall loss rate = ϕ index = 1.6 - 0.9 = 0.7 in/hr

LAGGING AND S-CURVES

LAGGING METHOD -- (Simpler but limited)

1. Used only to lengthen time·base of Unit Hydrograph
2. Length of new time·base (t) must be integer multiple of original time base (D)

   therefore $t = n \times D$ where n = integer

3. Lag n Unit Hydrographs by D-hrs (where D = time·base of original unit hydrograph) to get new (t-hr) Unit Hydrograph

   therefore

$$\text{Ordinate of t-hr U.H.} = \frac{\sum_{i=1}^{n} \text{lagged D-hr U.H.}}{n}$$

S-CURVE METHOD -- (Flexible but more complex)

1. Used to lengthen or shorten time·base of Unit Hydrograph (Again want to go from D-hr U.H. to t-hr U.H.
2. Form D-hr S-Curve by cumulative addition of D-hr U.H. at time intervals equal to D-hrs. ** A-Curve **
3. Lag a second D-hr S-Curve by t-hrs ** B-Curve **
4. Ordinates of new t-hr U.H. = $(A-B) \times (D/t)$

   where A and B are ordinates of the two A and B S-Curves

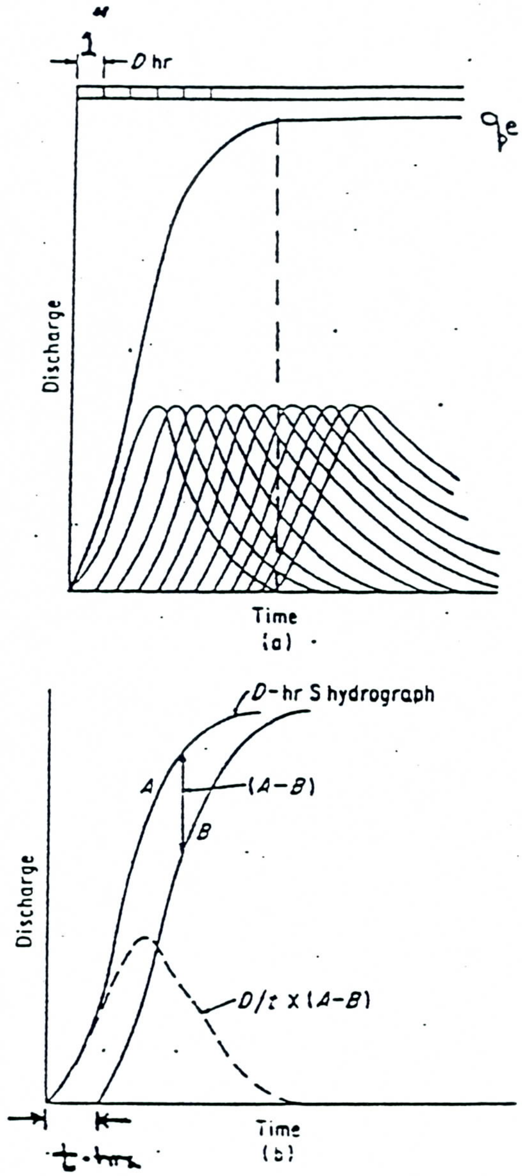
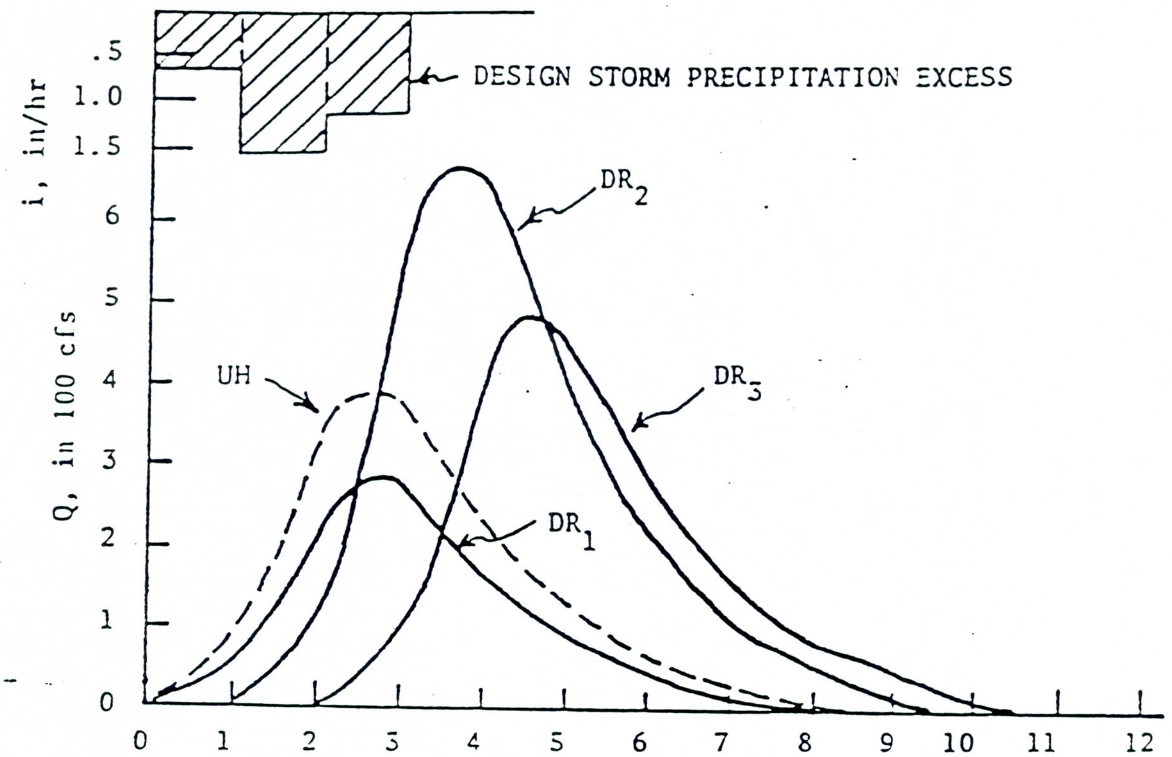


FIG. 3-14. S-hydrograph method.

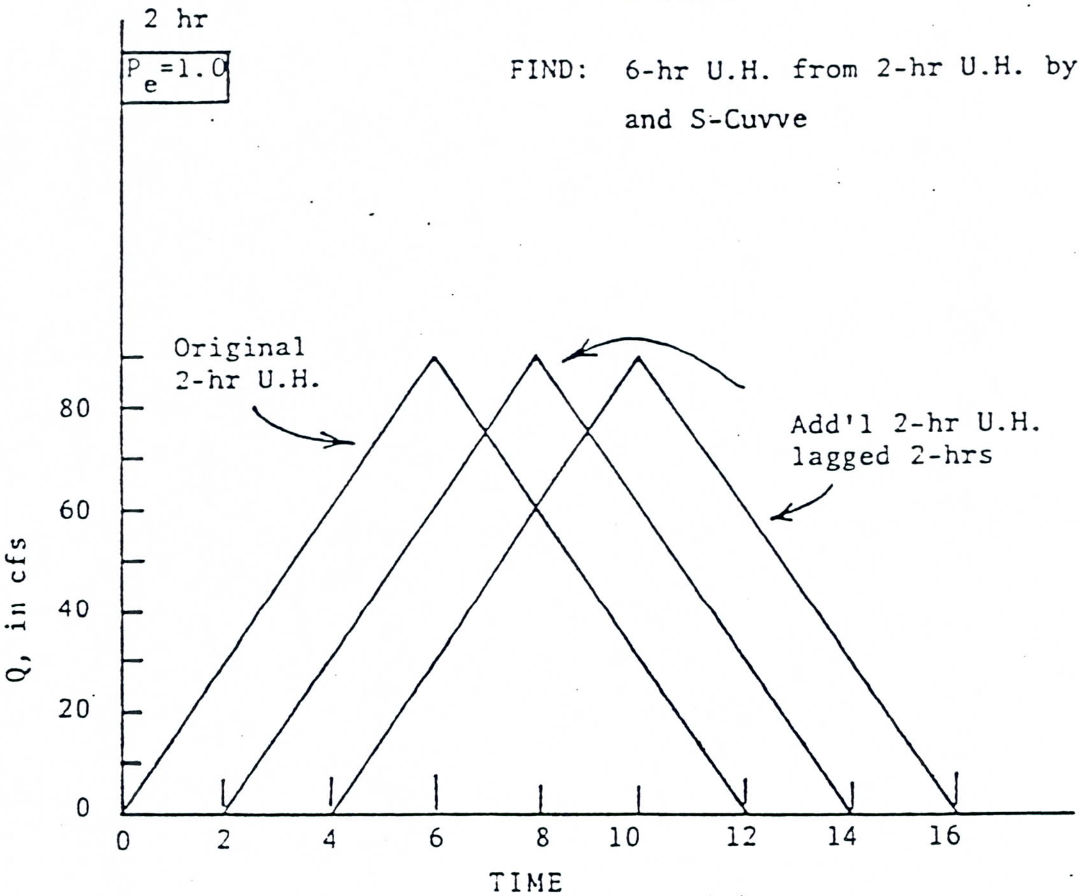
UNIT HYDROGRAPH APPLICATION TO A COMPLEX STORM



TIME UNITS	RAIN P_e, in	U. H. cfs/in	Direct Runoff, cfs Storm Segements, in 0.7	1.7	1.2	BASE FLOW cfs	TOTAL Q, cfs
0	0.7	0	0	0	0	120	120
1	1.7	79	55	0	0		175
2	1.2	328	230	134	0		484
3		378	265	558	95		1038
4		229	160	643	394		1317
5		129	90	389	454		1053
6		66	46	219	275		660
7		37	26	112	155		413
8		8	6	63	79		268
9		0	0	14	44		178
10		0	0	0	10		130
11		0	0	0	0	120	120
12		0	0	0	0	110	110

COMPARISON OF LAGGING VS S-CURVE

FIND: 6-hr U.H. from 2-hr U.H. by Lagging and S-Cuvve



TIME hr	2-hr	ΣU.H.÷ 3 (lagged 2-hrs)	2-hr S-Curve	6-hr lagged S-Curve	6-hr U.H. S-Curve ΔY x 2/6 6-hr U.H.
0	0	0	0		0
1	15	5	15		5
2	30	10	30		10
3	45	20	60		20
4	60	30	90		30
5	75	45	135		45
6	90	60	180	0	60
7	75	65	210	15	65
8	60	70	240	30	70
9	45	65	255	60	65
10	30	60	270	90	60
11	15	45	270	135	45
12	0	30	270	180	30
13		20	270	210	20
14		10	270	240	10
15		5	270	255	5
16		0	270	270	0

Example of Unit
Hydrograph

UNIT GRAPH DERIVATION AND HYDROGRAPH RECONSTRUCTION

WORKSHOP NO. 2

PROBLEM STATEMENT

UNIT GRAPH DERIVATION AND HYDROGRAPH

RECONSTRUCTION

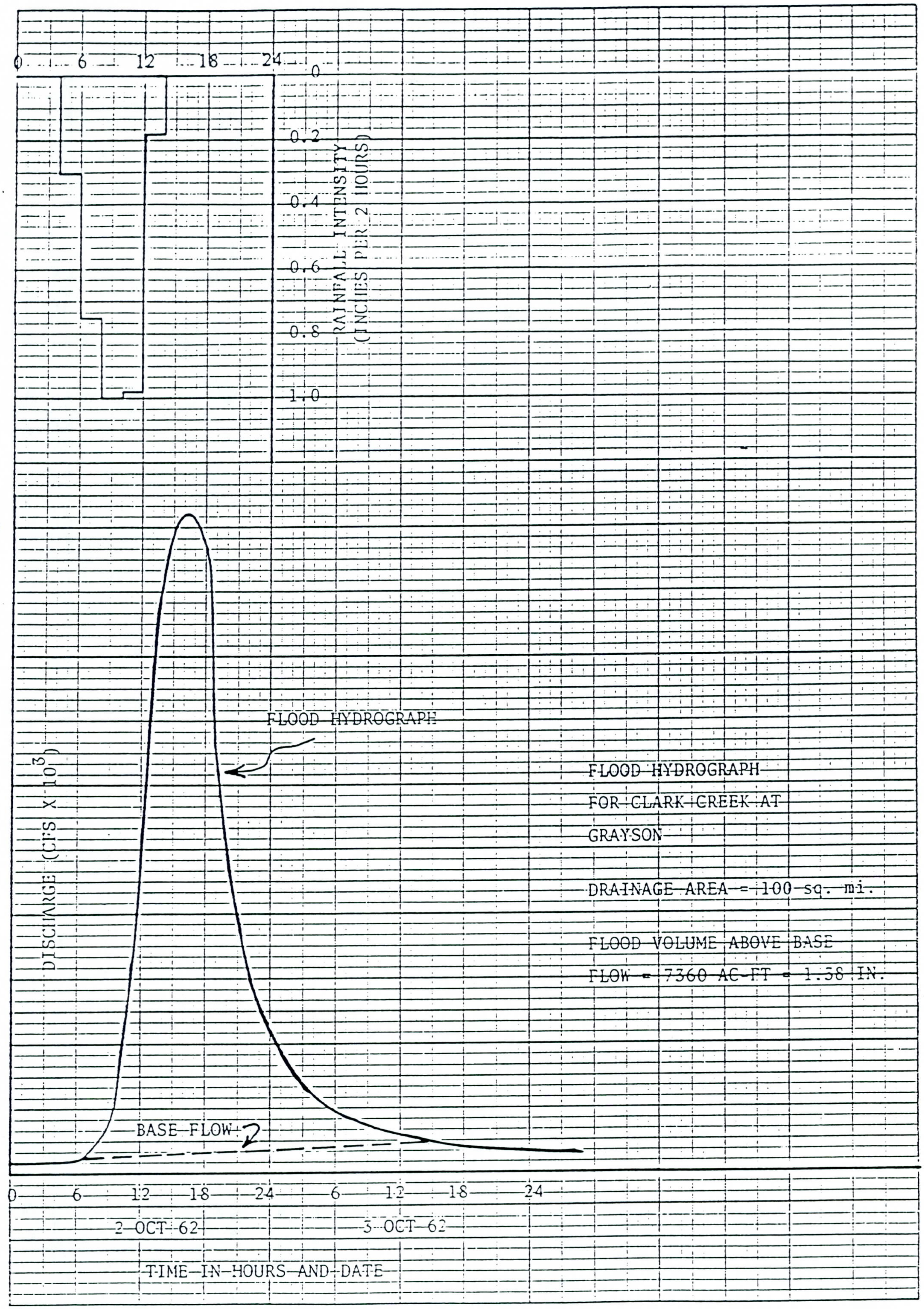
PROBLEM STATEMENT

Recorded hydrographs for the October 1962 and December 1941 floods on Clark Creek at Grayson are given in the following pages. The basin average hyetograph for the October 1962 and 1941 floods are given in figures 1 and 2 respectively.

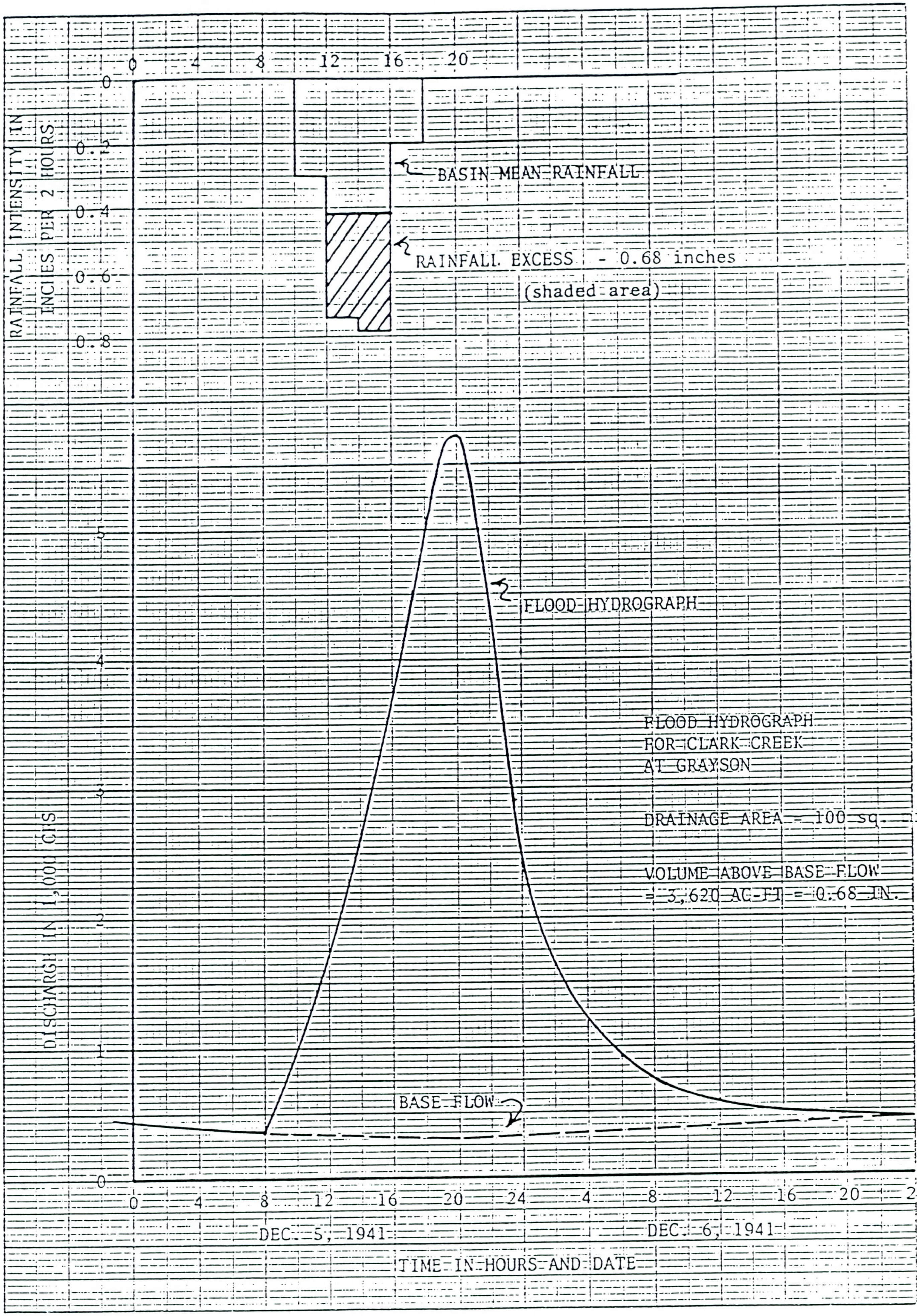
PROBLEM

1. Using the rainfall excesses and the flood hydrograph for the october 1962 flood, derive a unit hydrograph representing 1 inch of excess. Assume that the rainfall excess is uniform over its duration. What is the duration of your unit hydrograph?

2. Given the rainfall excess of 0.68 inches for the December 1941 flood, how would you apply your unit hydrograph in order to reconstitute the December 1941 flood? Outline in detail the steps you would take. Do not go through the calculations to actually reconstitute this flood.



0 6 12 18 24
0
0.2
0.4
0.6
0.8
1.0
RAINFALL INTENSITY
(INCHES PER 2 HOURS)
10
8
6
4
2
0
DISCHARGE (CFS X 10^3)
FLOOD HYDROGRAPH
BASE FLOW
FLOOD HYDROGRAPH
FOR CLARK CREEK AT
GRAYSON
DRAINAGE AREA = 100 sq. mi.
FLOOD VOLUME ABOVE BASE
FLOW = 7360 AC-FT = 1.38 IN.
0 6 12 18 24 6 12 18 24
2 OCT 62
3 OCT 62
TIME IN HOURS AND DATE



RAINFALL INTENSITY IN INCIES PER 2 HOURS
0
4
8
12
16
20
0.2
0.4
0.6
0.8
BASIN MEAN RAINFALL
RAINFALL EXCESS - 0.68 inches
(shaded area)
FLOOD HYDROGRAPH
FLOOD HYDROGRAPH
FOR CLARK CREEK
AT GRAYSON
DRAINAGE AREA - 100 sq. mi.
VOLUME ABOVE BASE FLOW
= 3,620 AC-FT = 0.68 IN.
DISCHARGE IN 1,000 CFS
5
4
3
2
1
0
BASE FLOW
24
DEC. 5, 1941
DEC. 6, 1941
TIME IN HOURS AND DATE

UNIT HYDROGRAPH DERIVATION

DAY	TIME ENDING (HRS)	RAINFALL EXCESS (inches)	TOTAL FLOW (cfs)	BASE FLOW (cfs)	DIRECT RUNOFF (cfs)	6-HOUR UNIT GRAPH (cfs) *
10/2	0600		200	200	0	0
	0800		600	200	400	290
	1000					
	1200					
	1400					
	1600					
	1800					
	2000					
	2200					
	2400					
10/3	0200					
	0400					
	0600					
	0800					
	1000					
	1200					
	1400					
	1600					

* Rounded off to nearest 10 cfs

DIRECT RUNOFF VOLUME = ____________________ ft^3

AREA OF WATERSHED = ____________________ ft^2

DEPTH OF RAINFALL = ____________________ inches

UNIT HYDROGRAPH ORDINATES = Q/DEPTH OF RUNOFF (in inches)

Solution of Example

Unit Hydrograph

UNIT GRAPH DERIVATION AND HYDROGRAPH RECONSTRUCTION

WORKSHOP NO. 2

PROBLEM SOLUTION

PRELIMINARY SCREENING FOR SUITABLE STORMS FOR

UNIT HYDROGRAPH FORMATION

1. Duration of rainfall event should be approximately 10 to 30% of the drainage area lag time.

2. Direct runoff for the selected storm should range from 0.5 to 1.75 inches for flood flow frequencies less than 100-year and greater than 1.5 inches for PMF development.

3. A suitable number of storms should be analyzed to obtain an average of the ordinates for a selected unit hydrograph duration. Modifications may be made to adjust the unit hydrographs by means of S-hydrographs or lagging procedures.

4. Direct runoff ordinates for each storm ordinates for each storm should be reduced so that each event represents 1.0 inches of direct runoff.

5. The final unit hydrograph of a specific duration for the watershed is obtained by averaging the ordinates of selected events and adjusting the result to obtain 1.0 inches of direct runoff.

Note: The above rules of thumb are from Introduction of Hydrology, by Viessman, Knapp, and Lewis, 3rd Edition, 1988. It is emphasized that they are suggestions and NOT rules.

RECONSTRUCTION OF DECEMBER 1941 FLOOD

(1) DERIVATION OF 4-HOUR U.G. ORDINATES BY THE S-CURVE METHOD

TIME IN HOURS	6-HOUR U.G. ord. (CFS)	S-HYDROGRAPH ADDITIONS (CFS)	ROUGH S-CURVE (CFS)	SMOOTHED S-CURVE (CFS)	4-HOUR LAGGED S-CURVE (CFS)	4-HOUR* UNIT HYDROGRAPH (CFS)
1	2	3	4	5	6	7
0	0		0	0		0
2	290		290	290		430
4	1380		1380	1380		2070
6	4130		4130	4130	290	5750
8	6450	290	6470	6470	1380	7620
10	7100	1380	8480	8480	4130	6510
12	5710	4130	9840	9380	6470	4360
14	2800	6740	9540	9820	8480	2010
16	1620	8480	10100	10200	9380	1230
18	1060	9840	10900	10400	9820	870
20	720	9540	10260	10550	10200	520
22	460	10100	10560	10600	10400	300
24	300	10900	11200	10650	10550	150
26	170	10260	10430	10680	10600	120
28	90	10560	10650	10700	10650	75
30	40	11200	11240	10710	10680	45
32	10	10430	10440	10710	10700	20
34	0	10650	10650	10710	10710	0
36						

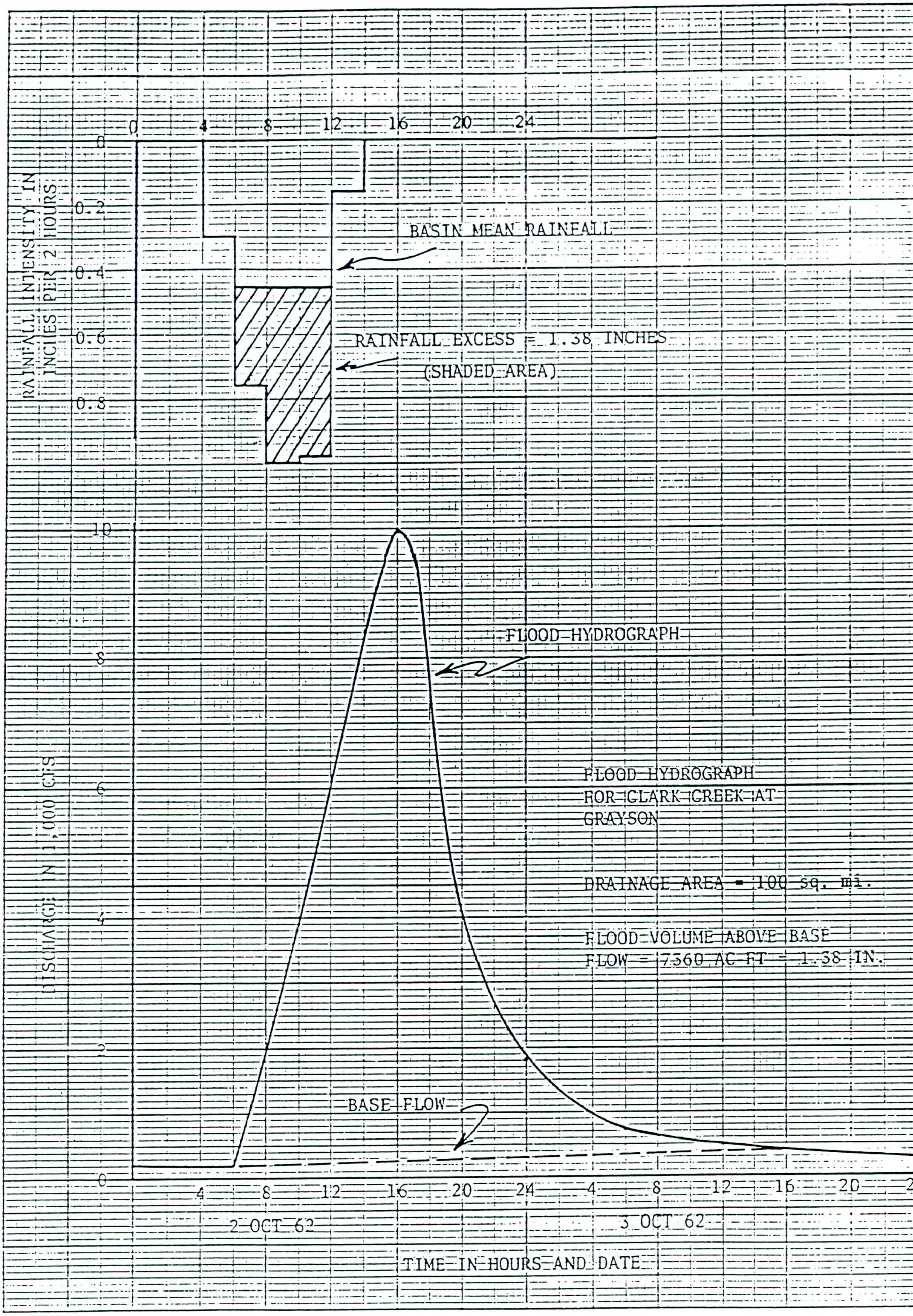
* 4-HOUR UNIT HYDROGRAPH ORDINATES = $\Delta Y/IT_i$ = $\Delta Y/(.167)(4)$

or (Column 5 - Column 6) ÷ (.167)(4)

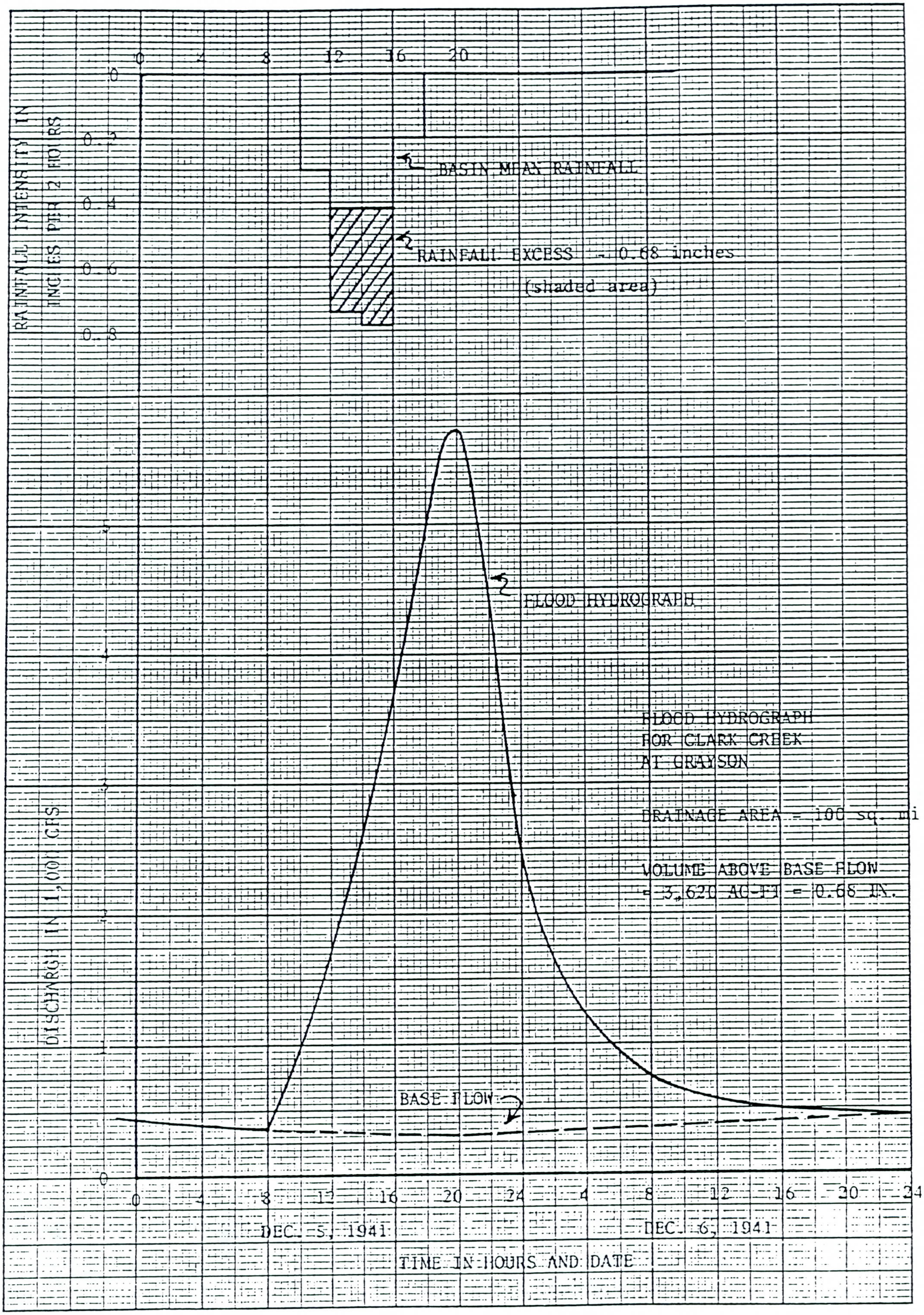
(2) APPLICATION OF 4-HOUR U.G. TO RECONSTRUCT THE DECEMBER 1941 FLOOD.

DATE	TIME (HRS)	4-HOUR U.G. ORDINATES (CFS)	RAINFALL EXCESS INCHES	DIRECT RUNOFF* (CFS)	BASE FLOW (CFS)	TOTAL FLOW (CFS)
12/5/41	1200	0	0.68	0	350	350
	1400	430		290	330	620
	1600	2070		1410	310	1720
	1800	5750		3900	300	4200
	2000	7620		5180	300	5480
	2200	6510		4430	310	4740
	2400	4360		2960	320	3280
12/6/41	0200	2010		1370	330	1700
	0400	1230		840	340	1180
	0600	870		590	350	940
	0800	520		350	360	710
	1000	300		200	370	570
	1200	150		100	380	480
	1400	120		80	390	470
	1600	75		50	400	450
	1800	45		30	410	440
	2000	20		10	420	430
	2200	0		0	430	430
	2400					

* DIRECT RUNOFF = U.G. ORDINATAES * 0.68, and are rounded off to the nearest 10 cfs.



RAINFALL INTENSITY IN INCHES PER 2 HOURS
0
4
8
12
16
20
24
0.2
0.4
0.6
0.8
BASIN MEAN RAINFALL
RAINFALL EXCESS = 1.38 INCHES
(SHADED AREA)
10
8
6
4
2
0
DISCHARGE IN 1,000 CFS
FLOOD HYDROGRAPH
FLOOD HYDROGRAPH
FOR CLARK CREEK AT
GRAYSON
DRAINAGE AREA = 100 sq. mi.
FLOOD VOLUME ABOVE BASE
FLOW = 7360 AC FT = 1.38 IN.
BASE FLOW
2 OCT 62
3 OCT 62
TIME IN HOURS AND DATE



RAINFALL INTENSITY IN INCHES PER 2 HOURS
BASIN MEAN RAINFALL
RAINFALL EXCESS - 0.68 inches
(shaded area)
FLOOD HYDROGRAPH
FLOOD HYDROGRAPH
FOR CLARK CREEK
AT GRAYSON
DRAINAGE AREA = 100 sq. mi.
VOLUME ABOVE BASE FLOW
= 3,620 AC-FT = 0.68 IN.
BASE FLOW
DISCHARGE IN 1,000 CFS
DEC. 5, 1941
DEC. 6, 1941
TIME IN HOURS AND DATE

4. UNIT GRAPH DERIVATION

DAY	TIME ENDING (HRS)	RAINFALL EXCESS (IN)	TOTAL FLOW (CFS)	BASE FLOW (CFS)	DIRECT RUNOFF (CFS)	6-HOUR UNIT GRAPH (CFS)*
10/2	0600	0.0	200	200	0	0
	0800	0.30	600	200	400	290
	1000	0.55	2100	200	1900	1380
	1200	0.53	5900	200	5700	4130
	1400	0.0	9100	200	8900	6450
	1600		10000	200	9800	7100
	1800		8100	220	7880	5710
	2000		4100	240	3860	2800
	2200		2500	260	2240	1620
	2400		1750	280	1470	1060
10/3	0200		1300	300	1000	720
	0400		950	320	630	460
	0600		750	340	410	300
	0800		600	360	240	170
	1000		500	380	120	90
	1200		460	400	60	40
	1400		430	420	10	10
	1600		400	400	0	0

* Rounded off to nearest 10 cfs

DIRECT RUNOFF VOLUME = 1.38 inches (given)

Therefore Unit Graph ordinates = Q/1.38

Assuming that the rainfall excess is uniformly distributed, the above ordinates are for a 6-hour unit hydrograph.

5. In order to apply the 6-hour unit hydrograph to the December 1941 flood, the following steps would be necessary:

   a. Determine the rainfall excess that contributed to the December 1941 flood (0.68 inches, given in the problem)

   b. Select a duration over which a portion or all of the rainfall excess is uniformly distributed. In this case, the rainfall excess of 0.68 inches was relatively uniform in its distribution over the 4-hour period.

   c. The S-curve method must be applied to convert the 6-hour unit graph to a 4-hour unit graph by the following procedure:

      (1) Develop an S-curve from the 6-hour unit graph, and smooth out the curve as needed.

      (2) Offset the S-curve by 4 hours and tabulate the difference between the ordinates of the original S-curve and those of the offset curve (ΔY).

      (3) The tabulated differences between the S-curves (ΔY) must then be adjusted to 1 inch of direct runoff. Since the 6-hour unit graph corresponds to 1 inch total or 0.17 inches per hour, the unit graph ordinates for a 1-inch excess in 4 hours are calculated as follows:

$$\text{U. G. ORD.} = \Delta Y \;/\; 0.17 \text{ inches per hour} \times 4 \text{ hours}$$

$$= \Delta Y \;/\; 0.68 \text{ inches}$$

   d. Apply the rainfall excess of 0.68 inches to the 4-hour unit graph ordinates to obtain the direct runoff component to flow (multiply ordinate by 0.68). Then add the base flow component to obtain the reconstructed hydrograph for the December 1941 flood.

UNIT HYDROGRAPH THEORY - THEORY OF UNIT HYDROGRAPH FOR GAGED WATERSHEDS - ASSUMPTIONS AND LIMITATIONS

PMF GUIDELINES SECTION 8-8

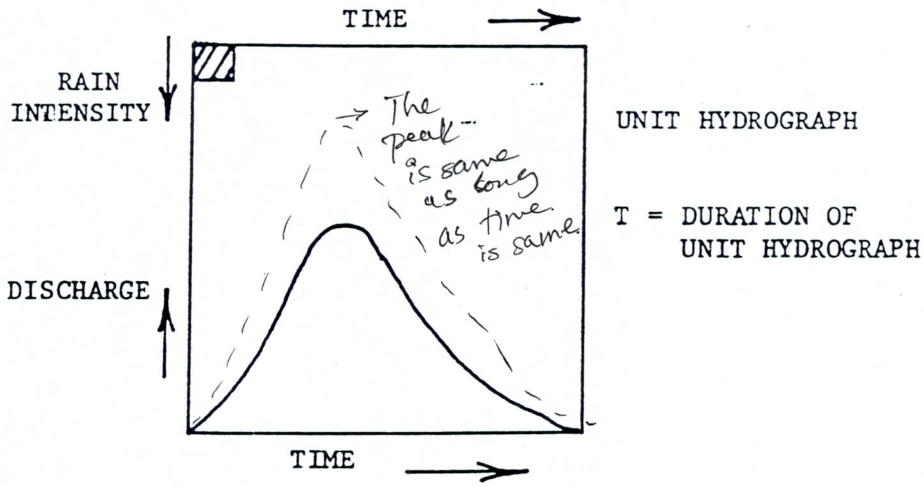
Monday 11:20 a.m.

UNIT HYDROGRAPH THEORY

Unit Hydrograph Theory

a. Definition of the unit hydrograph (UH)

A hydrograph representing 1 unit (e.g., 1 inch) of direct runoff from a rainfall excess of some unit duration and specific areal distribution



The volume of effective rain and the volume of direct runoff for the unit hydrograph are both equal to one inch.

b. Assumptions

(1) Unvarying spatial distribution of effective rainfall and loss rates (effective rainfall and loss rates are 'lumped')

(2) The ordinates of a direct runoff hydrograph corresponding to an effective rainfall of a given duration are directly proportional to the volume of effective rainfall (principle of linearity)

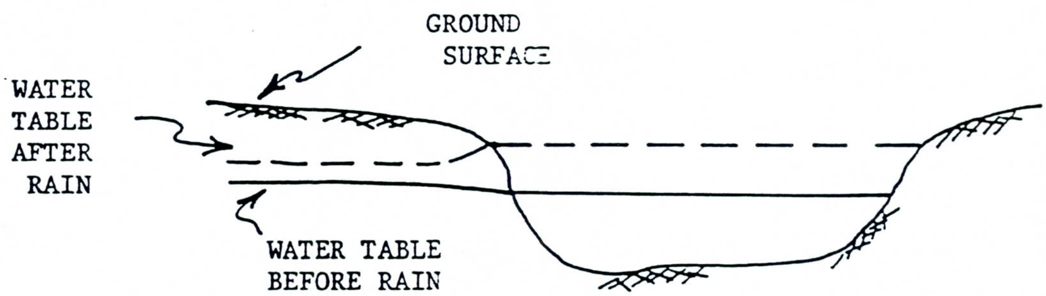
(3) The direct runoff hydrograph resulting from a given pattern of effective rainfall does not depend on the time of occurrence of the effective rainfall (principle of time invariance).

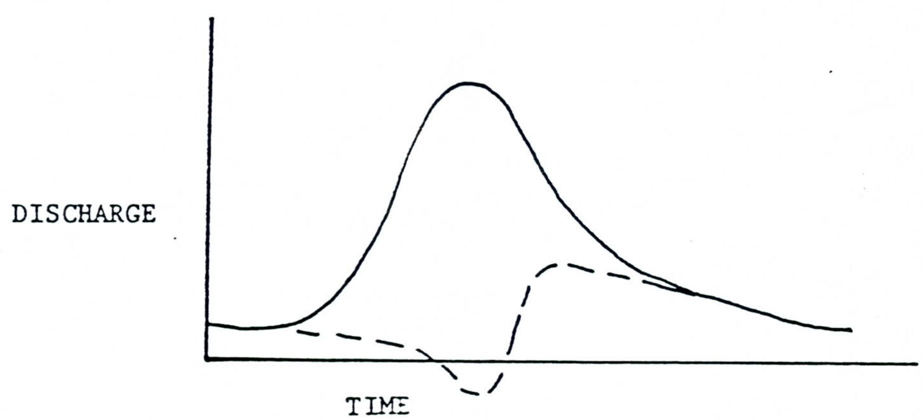
Depends on season. Different unit hydrograph for different season

Also, the time of occurance shall be checked.

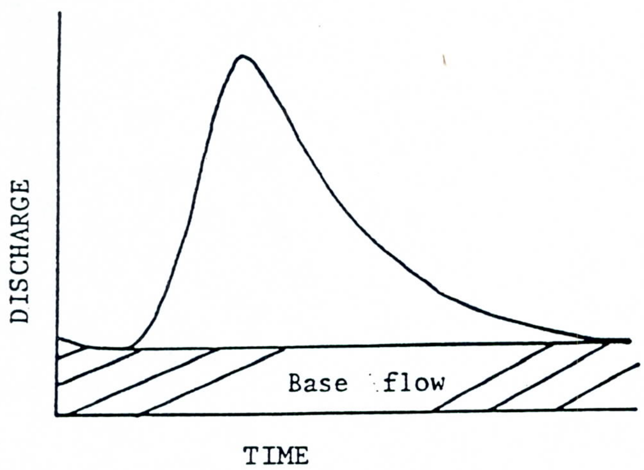
Base Flow

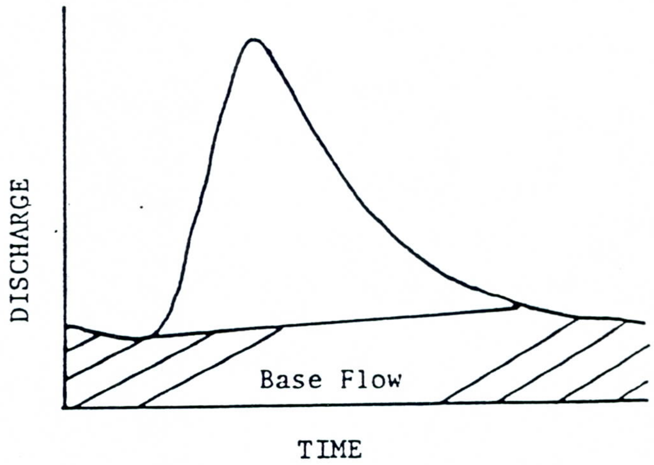
Base flow represents runoff due to antecedent rainfall events. It therefore represents water coming out of storage in the basin. Consider how base flow could typically vary during a period of rainfall and the subsequent rise of a stream:

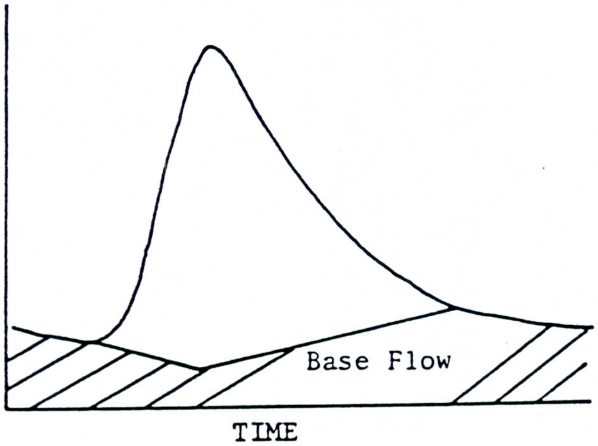




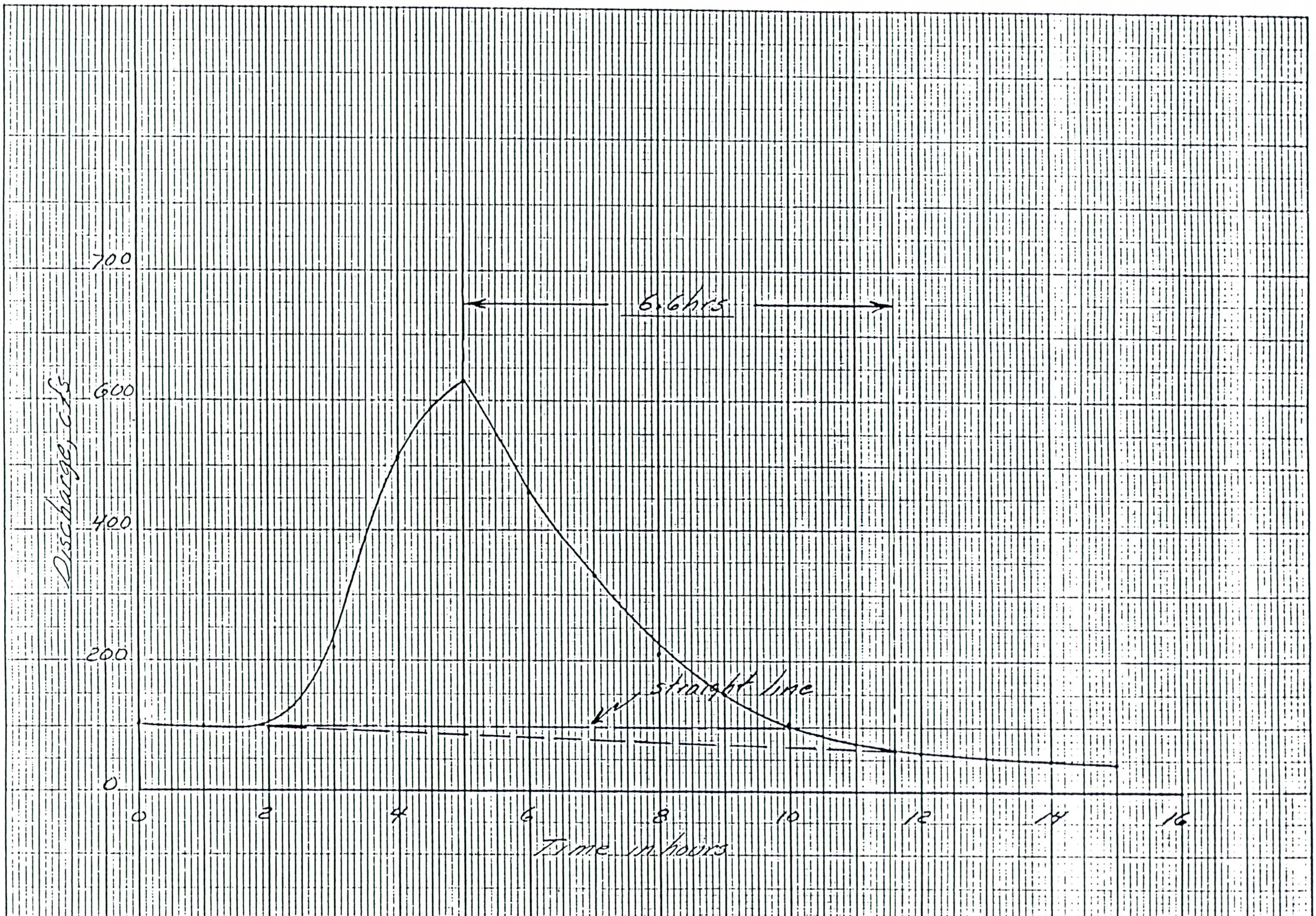
Base flow separation techniques:



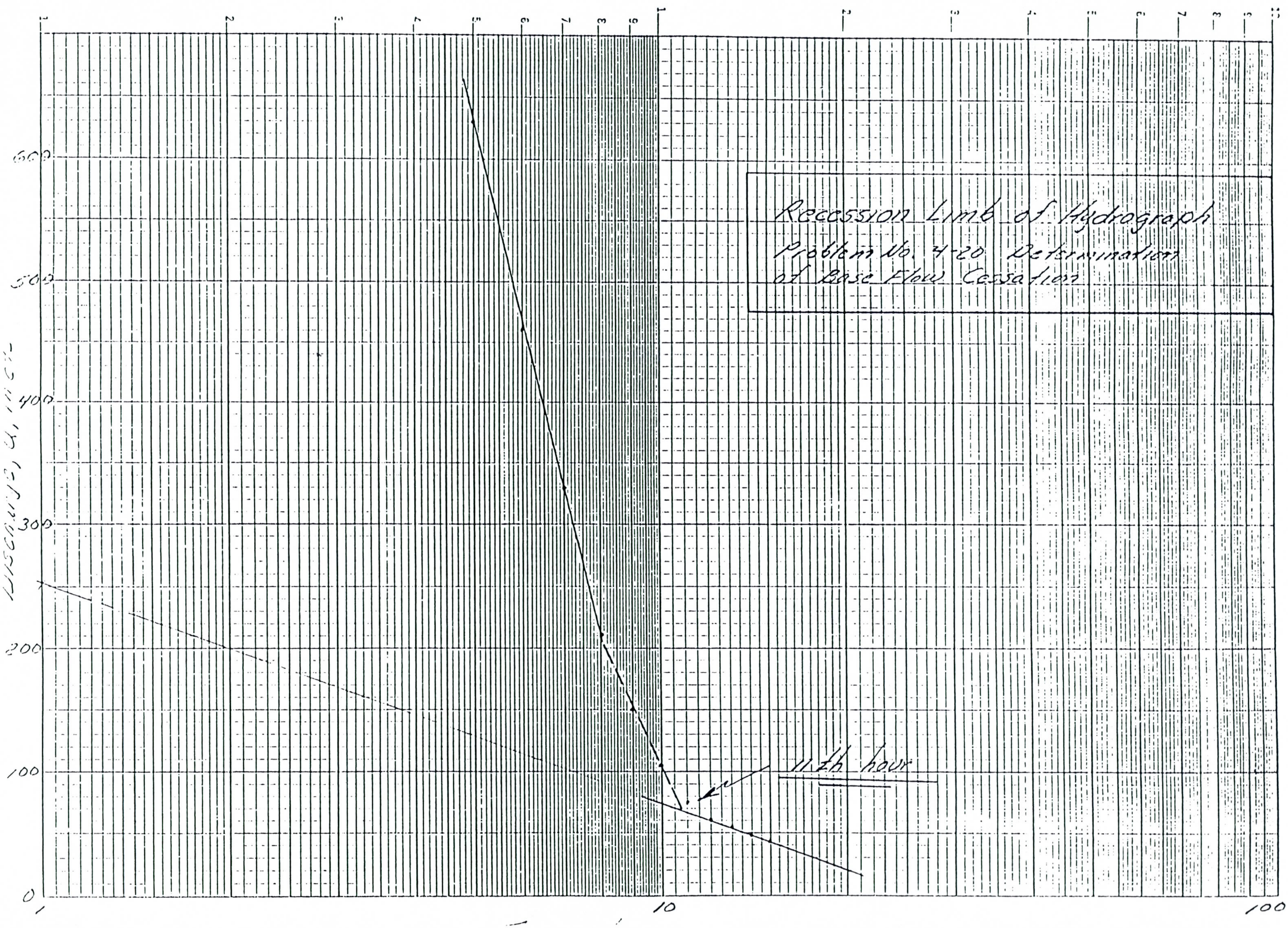




Estimation of base flow is generally arbitrary. For major floods, however, the volume of base flow is often a small percentage (say less than 10%) of total runoff.



700
600
400
200
0
Discharge, cfs
6.6hrs
straight line
0
2
4
6
8
10
12
14
16
Time in hours


Recession Limb of Hydrograph
Problem No. 4-20 Determination
of Base Flow Cessation
11th hour
600
500
400
300
200
100
0
1
10
100

6. Unit Hydrograph Derivation by the Isolated Storm Method

a. General Procedure

(1) Examine precipitation and streamflow records and extract data for isolated storms.

(2) Determine direct runoff component of hydrographs from isolated storms by separating the base flow component.

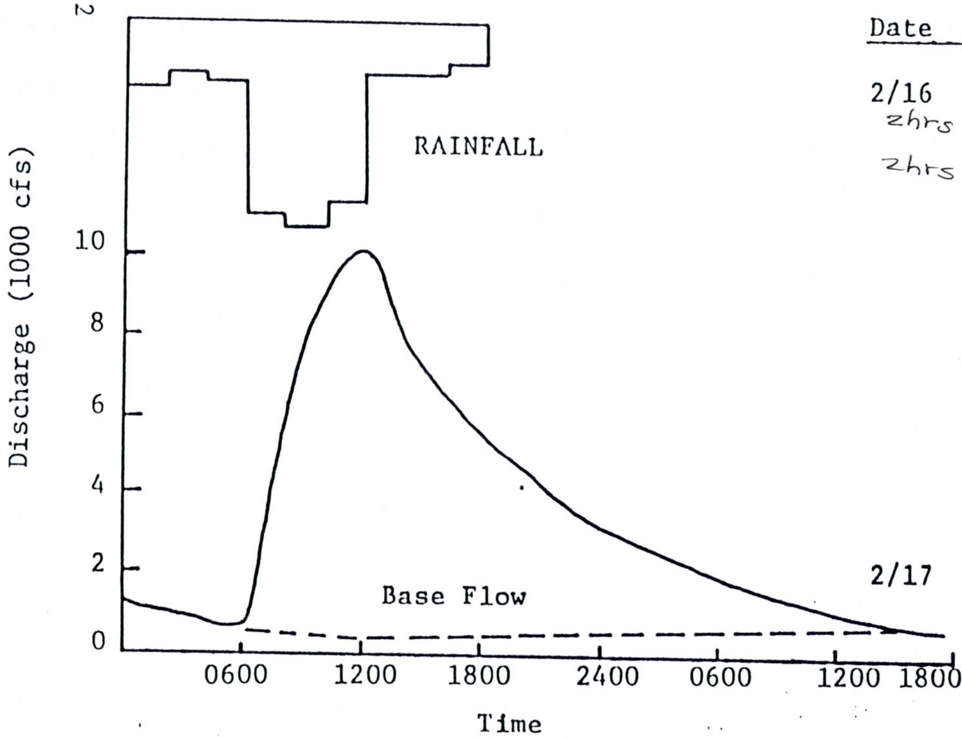
(3) Compute the effective rainfall (or rainfall excess) which corresponds to the direct runoff volume for each storm. This effective rainfall should be of a relatively uniform rate for the duration of the unit hydrograph.

(4) Divide the ordinates of the observed hydrograph by the basin runoff volume (in inches) for the storm. The resulting ordinates are for 1 inch direct runoff.

b. Example of Isolated Storm Method

L-65-2

ISOLATED STORM METHOD



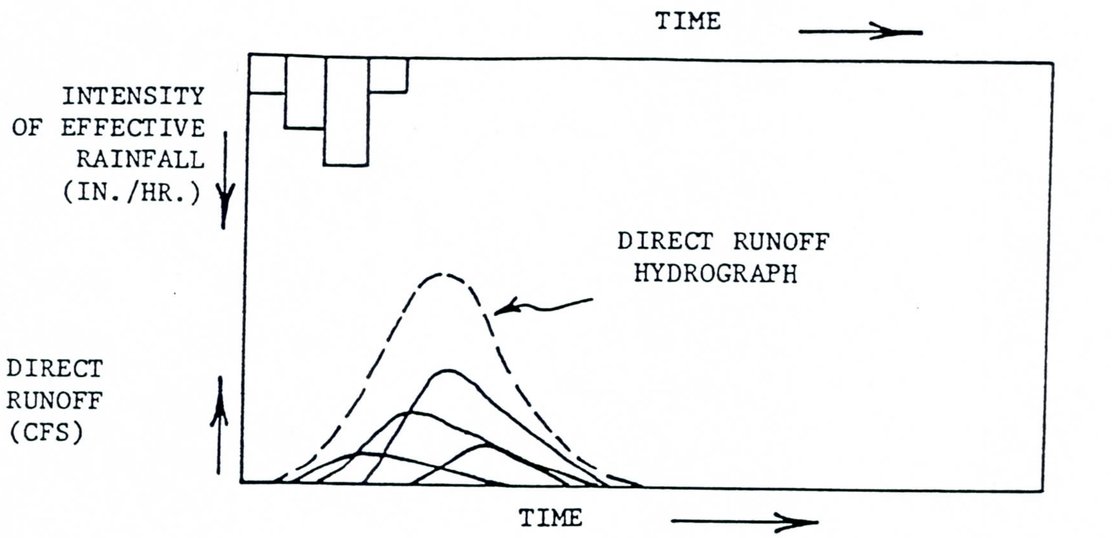
DRAINAGE AREA = 40 sq. mi.

Date	Time	Total Flow	Base Flow	Direct R.O.	U.G. Ord.	Hrs.
2/16	0600	500	500	0	0	0
2hrs	0800	5600	450	5150	1120	2
2hrs	1000	9200	400	8800	1915	4
	1200	10100	400	9700	2110	6
	1400	7800	450	7350	1600	8
	1600	6600	450	6150	1340	10
	1800	5550	500	5050	1100	12
	2000	4700	550	4150	900	14
	2200	4000	600	3400	740	16
	2400	3300	600	2700	590	18
2/17	0200	2700	600	2100	460	20
	0400	2300	650	1650	360	22
	0600	1950	650	1300	280	24
	0800	1650	700	950	210	26
	1000	1400	700	700	150	28
	1200	1200	750	450	100	30
	1400	1000	750	250	50	32
	1600	800	800	0	0	34
			TOTAL	59850	2hr-cfs	

$$\text{Direct Runoff} = \left(59{,}850\ \frac{2\text{hr ft}^3}{\text{sec}}\right)\left(\frac{1}{40\text{mi}^2}\right)\left(\frac{12\text{ in}}{\text{ft}}\right)\left(\frac{\text{mi}^2}{5280^2\ \text{mi}^2}\right)\left(\frac{2 \times 3600\text{ sec}}{2\text{ hr}}\right) = 4.63\text{ inches}$$

Assumption: This is an independent flood event produced by an isolated storm of approximately uniform rainfall excess.

Application of the Unit Hydrograph

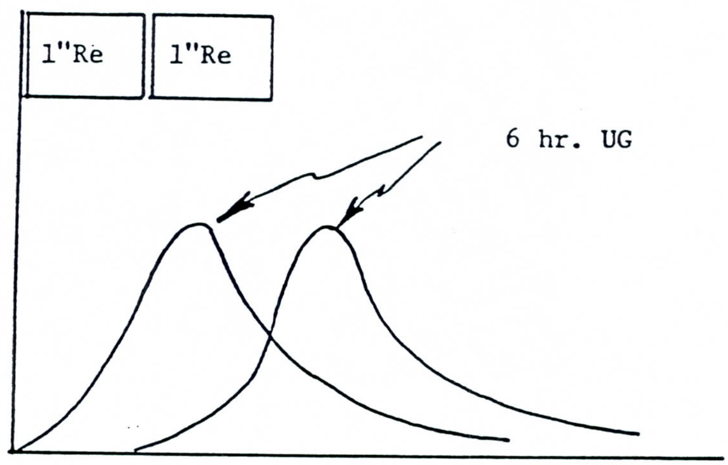


Time (Hours)	1-HR UHG (cfs)	Effective Rainfall (in)	Direct Runoff (cfs)					Base Flow cfs	Total Discharge cfs
			(a)	(b)	(c)	(d)	Sub-Total		
0	0								
	10	1.0	10				10	10	20
	100	2.0	100	20			120	10	130
	200	3.0	200	200	30		430	10	440
	150	1.0	150	400	300	10	860	10	870
	100		100	300	600	100	1100	10	1110
	50		50	200	450	200	900	10	910
	0		0	100	300	150	550	10	560
				0	150	100	250	10	260
					0	50	50	10	60
						0	0	10	10
Totals	610	7.0	610	1220	1830	610	4270	100	4370

Note that the unit duration associated with a unit hydrograph must be the same as the duration of effective rainfall increments. Techniques are available for changing the unit duration of a unit hydrograph, for example for converting a 3-hour unit hydrograph to a 6-hour unit hydrograph.

Changing the Unit Duration of a Unit Hydrograph

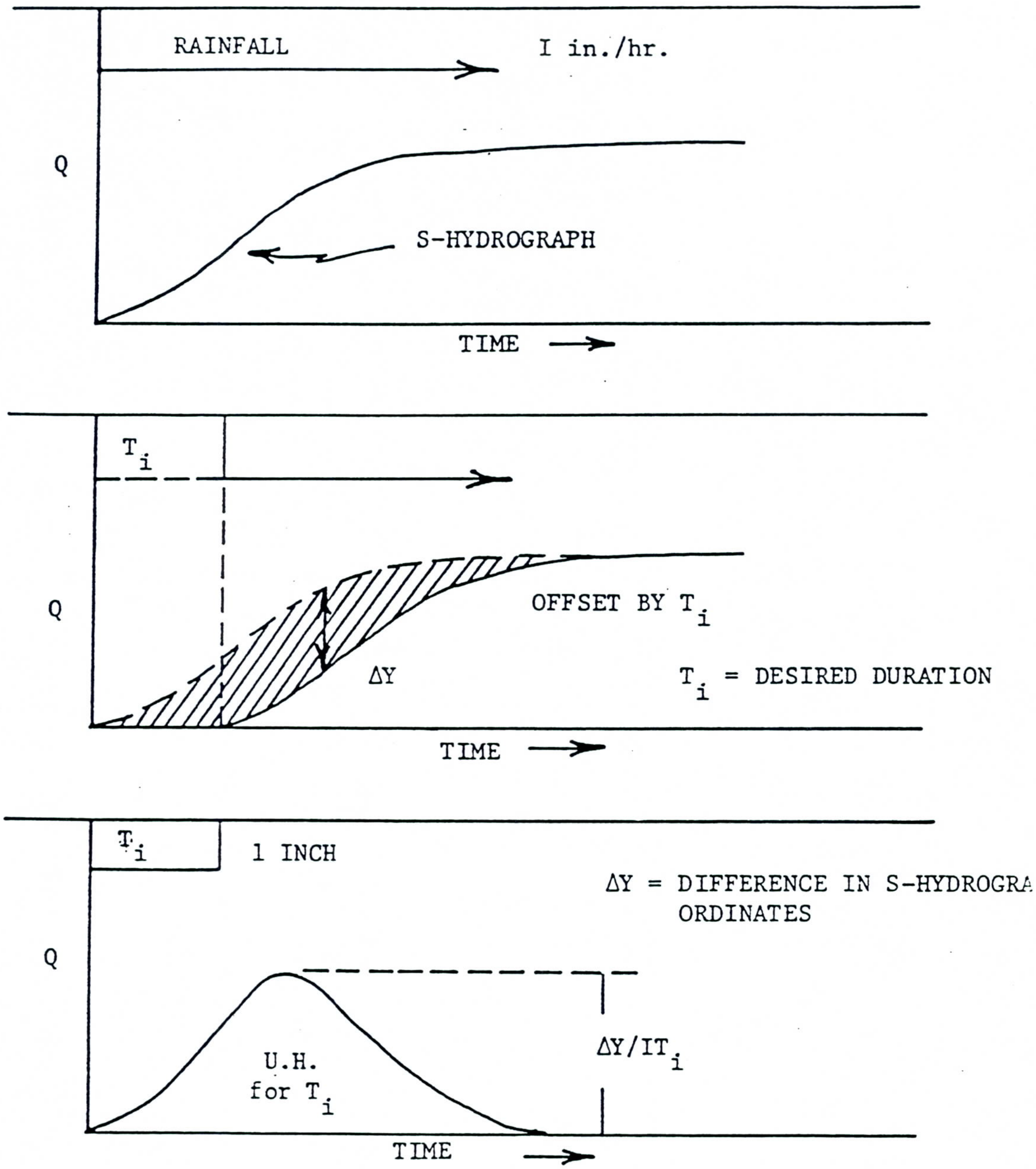
a. Simplified method from a unit hydrograph of duration equal to any integral multiple of T can be derived by the superposition principle.

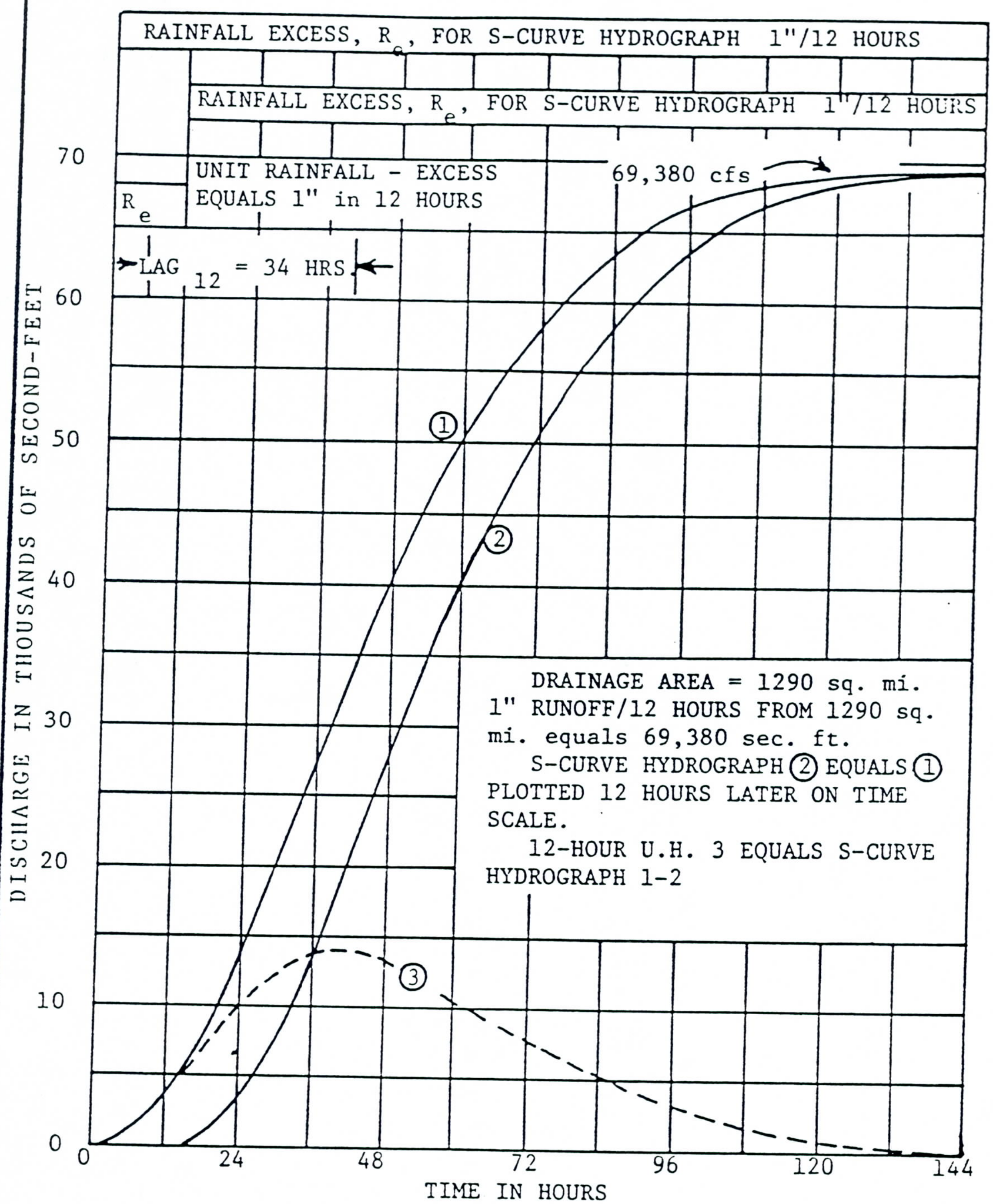


Time	6-hr. UG	lagged 6 hr. UG	2" hydrograph	12 hr. UG
3	2		2	1.0
6	5		5	2.5
9	7	2	9	4.5
12	5	5	10	5.0
15	3	7	10	5.0
18	1	5	6	3.0

Time	Re	6 hr. UG	12 hr. UG
3		2	(2 X .5 = 1.0
6	.5	5	(5 X .5) = 2.5
9		7	(7 X .5) + (2 X .5) = 4.5
12	.5	5	(5 X .5) + (5 X .5) = 5.0
15		3	(3 X .5) + (7 X .5) = 5.0
18		1	(1 X .5) + (5 X .5) = 3.0

b. The S-Curve Method. Provides a generalized procedure for changing the duration of a unit hydrograph to any other duration.





RELATION OF UNIT HYDROGRAPH
TO S-CURVE HYDROGRAPH

PLATE I

METHODS OF CALCULATING INFILTRATION

Monday 1:00 p.m.

US Army Corps
of Engineers

The Hydrologic
Engineering Center

Infiltration and Soil Moisture Redistribution in HEC-1

by

Arlen D. Feldman

David M. Goldman

Technical Paper No. 95

January 1984

Infiltration and Soil Moisture Redistribution in HEC-1*

Arlen D. Feldman
David M. Goldman
Hydrologic Engineering Center, Davis, California

INTRODUCTION

The U.S. Army Corps of Engineers' water resource modeling efforts have been motivated by the civil works needs of the Corps field offices. The main responsibilities of the Corps of Engineers have been in flood control and navigation, and thus the models were developed to meet those needs. Hydrologic analyses for flood control typically involved flood frequency and duration, spillway discharge, reservoir storage, channel and floodway capacity, water surface elevations, flow velocity, and flooded area computations.

Because of this primary interest in flood control and, therefore, the larger, damaging flood events, the Corps' Hydrologic Engineering Center (HEC) chose to simulate flood hydrographs with a so-called single-event watershed model. The "HEC-1 Flood Hydrograph Package" (Corps, 1981) simulates single flood events, although that one event may occur for many days or months in a complex river system. No soil mositure accounting is made between flood events.

More recently, however, HEC-1 is used for design flood simulation and flood forecasts. In the flood forecast mode, HEC-1F (forecast version) uses a feedback loop to update current soil moisture conditions as the flood event progresses. The update methodology is a parameter fitting process which mininizes the differences between the observed and computed runoff. The primary parameter fitted in this manner is the initial soil moisture deficiency.

SOIL MOISTURE'S PLACE IN A RIVER BASIN MODEL

What are the major factors which bring about the shape and size of a hydrograph? How important are these factors? Which factors does one have the most confidence in estimating? These are questions the hydrologic modeler must ask in the effort to simulate the occurrence of a flood event.

There are four main factors which determine the size and shape of a hydrograph.

1) Precipitation rates and spatial distribution.

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2) Interception/infiltration rates and spatial distribution.

3) Transformation of rainfall/snowmelt areal excess into stream runoff, and

4) Routing the runoff through rivers.

The volume of runoff is determined by the first two factors while all four contribute to the shape of a hydrograph.

Streamflow is probably the best known (measured) component of the rainfall-to-runoff process. Less is known about rainfall and catchment loss rates. Rainfall studies indicate that there is potential for larger errors in point measurement of intensity and that the spatial variability of the process can be quite large. For example, Neff (1977) indicates that measurement of rainfall intensity may differ by as much as 70% between surface and pit gages (the difference attributed to wind effects) and Woodley et al. (1977) indicates that rain gages only a few miles apart have known to differ as much as fifty percent in their measuremet of total storm precipitation.

Catchment loss rates are a function of both surface conditions (initial abstraction and depression storage) and soil hydraulic properties (infiltration capacity). Smith (1982) discusses the need to characterize the effects of rooted plants, crusting and cracking on infiltration processes and Woolhiser (1982) indicates the need for additional research to characterize depression storage. Although much work has been done theoretically to describe infiltration into a homogeneous soil, field measurement indicate that the soil hydraulic properties which control the infiltration process demonstrate a great deal of spatial variability. For example, Nielsen and Warwick (1980) summarize recent field investigations which indicate that the hydraulic conductivity at natural saturation and the unsaturated hydraulic conductivity have coefficients of variation on the order of 100%.

Our knowledge of the hydraulics of open channel flow make the routing process relatively well known. Although the flow is anything but what is assumed in the theory, the one-dimensional river process is easier to simulate than the wide spatial variation of the rainfall or interception/infiltration process. The rainfall excess transformation by unit graph or kinematic wave is difficult to estimate for large areas. But, if smaller subbasins are used, these factors become less important and more importance is placed on the better known channel routing hydraulics.

The hydrologic modelers' task is to put these processes together to reproduce observed runoff in a river basin. Then, more importantly, to use that same model to predict runoff in ungaged areas. To understand these processes, and the relative importance of one versus another during any particular flood event, one must be a hydrologic detective. The storm track, spatial variation in rainfall and infiltration rates and hydraulic regime of natural and man-made features of the watershed must all be considered. Too often the hydrologic modeler just specializes in understanding one of the factors contributing to the hydrograph. Very simplifying assumptions are made about the complex processes occurring on either side of the one where the expertise is being applied. Elegant mathematical formulations are made for homogeneous, isotropic representations of the physical process. Then those formulations are applied to heterogeneous, anisotropic conditions with poorly defined input and little concern for the next step with the output.

Thus, the infiltration processes discussed in the following section should always be kept in perspective with respect to the other parts of the hydrograph formation process. The rainfall excess is the desired result of this part of the process. That excess can be changed by varying the incoming rainfall and/or the interception/infiltration. However it is accomplished, the volume of the various surface, subsurface and ground water excesses must be equal to the observed hydrograph less previous base flow.

The following discussion describes the interception/infiltration, soil moisture redistribution, soil evaporation and aquifer recharge component of these hydrologic processes. In defining this part of the process, let us keep in mind how well we know (measure) the spatial and temporal distribution of precipitation and the heterogeneous mixture of land cover and soil types we have in a natural and/or man-influenced watershed.

HEC-1 INFILTRATION PROCESSES

The main purpose of the "HEC-1 Flood Hydrograph Package" (HEC, 1981) is to simulate the hydrologic processes during flood events. The precipitation (rainfall, snowfall/melt) to runoff process can be simulated for large complex watersheds. 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. 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:

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.

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 determines subbasin runoff parameters by a univariate search procedure. The unit hydrograph and interception/infiltration rates (hereafter referred to as precipitation loss rates) may be determined for individual storm events based on observed precipitation and streamflow data for a single subbasin. Streamflow routing parameters may also be determined 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 the headwaters of complex river basins. HEC-1 does not take into account the effect of downstream boundary conditions. This limitation may be overcome by using hydraulics models to provide the flood routing relationships for HEC-1. Keeping this limitation in mind, 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.

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 accuracy desired in the computational results, the allowable modeling efforts, project budget, and the available data.

Two important factors should be noted about the precipitation loss computation in the model. First, precipitation which does not contribute to the runoff process is considered to be lost from the system. Second, the equations used to compute the losses do not provide for soil moisture or surface storage recovery. (The Holtan loss rate option is an exception in that soil moisture recovery occurs by percolation out of the soil moisture storage.) This fact dictates that the HEC-1 program is a single event oriented model.

The precipitation loss computations can be used with either the unit hydrograph or kinematic wave model components. In the case of the unit hydrograph component, the precipitation loss is considered to be a subbasin average (uniformly distributed over an entire subbasin). On the other hand, separate precipitation losses can be specified for each overland flow plane in the kinematic wave component. The losses are assumed to be uniformly distributed over each overland flow plane.

In some instances, there are negligible precipitation losses for a portion of a subbasin. This would be true for an area containing a lake, reservoir or impervious area. In this case, precipitation losses will not be computed for a specified percentage of the area labeled as impervious.

There are four methods (Table I) that can be used to calculate the precipitation loss. Using any one of the methods, an average precipitation loss is determined for a computation interval and subtracted from the rainfall/snowmelt hyetograph. The resulting precipitation excess is used to compute an outflow hydrograph for a subbasin.

TABLE I

HEC-1 INTERCEPTION/INFILTRATION METHODS

Method	Parameters	Description
Initial and Constant	Initial volume loss and a constant infiltration rate	Initial loss 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 from land use and hydrologic soil type	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.

Initial and Constant Loss Rate Method

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 Δt, in hours, is:

$$L = \begin{cases} P & \text{if } L < I \\ C\Delta t & \text{if } L > I \end{cases} \qquad (1)$$

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 Ø index (if I is zero) 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.

HEC Exponential Loss Rate Method

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 Figure 1. The parameters of the method represent the effects of depression storage, D, infiltration rates, S and R, and the nonlinearity in the loss rate precess, E. The effects of soil moisture conditions are accounted for my adjusting the interception and infiltration rates by the accumulated loss C, resulting in two loss rate factors DK + AK. The loss rate factors are combined with the effect of precipitation intensity to obtain the following los rate function.

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

where $A = Ak + Dk$ and the precipitation intensity, P, is exponentiated by the nonlinearity parameter E. Note that a simple exponential decay to a constant loss S may be obtained by setting E = 0 and R = 1.

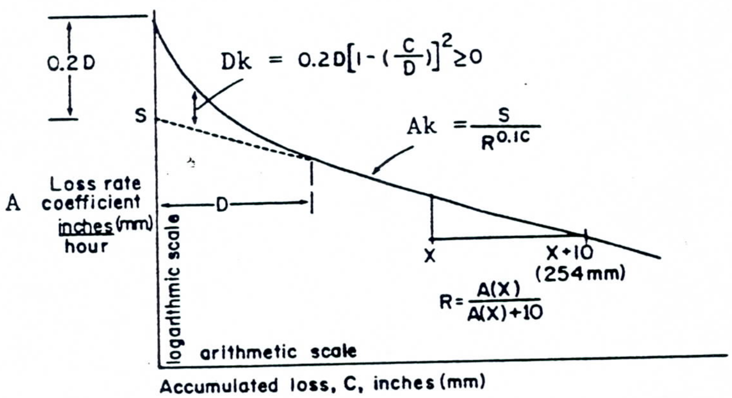

FIG. 1 The HEC exponential loss rate function. *S* is the loss rate for average soil moisture conditions; *D*, initial amount of loss for which the loss rate coefficient is increased to represent antecedent soil moisture conditions; *R*, rate of change of loss rate coefficient as soil moisture increases. (Feldman, 1981)

The HEC exponential loss rate equation is a function of the soil moisture accumulation; however, it is an empirical function whose parameters, S, D, E and R 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 types and land covers (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.

Curve Number Loss Rate Method

The Soil Conservation Service (SCS), U.S. Department of Agriculture, has instituted a soil classification system for use in soil survey maps across the country. Based on experimentation and experience, the agency has been able to relate the drainage characteristics of soil groups to a curve number, CN (SCS, 1972 and 1975). The SCS provides information on relating soil group type to the curve number as a function of soil cover, land use type and antecedent moisture conditions.

Precipitation loss is calculated based on CN and IA (where IA is an initial surface moisture storage capacity in units of depth). CN and IA are related to a total runoff depth for a storm by the standard SCS Method. The SCS method gives total excess for a storm. Thus, incremental excess (the difference between rainfall and precipitation loss) for a time period is computed as the difference between the accumulated excess at the end of the current period and the accumulated excess at the end of the previous period.

The SCS method has been the only method available for estimating loss rates based on the physical characteristics of the catchment. This is of immense practical importance when creating a physically based model in an ungaged watershed. However, the SCS method was developed primarily to evaluate the effect of land use change and not for the simulation of individual events (Rallison and Miller, 1982). In application to individual events the method suffers from theoretical defficiencies (Morel-Seytoux, 1981) and has had some difficulty in reproducing observed events (Rallison and Miller, 1982). To overcome this problem, the method has been developed (Rawls, et al., 1980) for using soil survey information to estimate the parameters of the Greem and Ampt equation. The Hydrologic Engineering Center plans to incorporate this methodology into HEC-1 (as discussed under future plans).

Holtan Loss Rate Method

H. Holtan of 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 may be derived directly from the soil water infiltration characteristics of the watershed.

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

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

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

Because the parameters of this method may be derived from the watershed's physical characteristics, there is potential for including this method in a physically based watershed model (see for example Li et al., 1977). However, as a basis for future investigations, the Green and Ampt equation seems more promising considering the recent efforts made to relate its parameters to readily available soil survey data.

Impervious Areas

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 percent 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.

Future Plans

The HEC is presently participating in a field investigation in Dry Creek Minnesota (near Jeffers) to determine the efficacy of using remote sensing to determine soil moisture. Data being obtained includes basic hydrometeorologic data; precipitation, wind speed, temperature, streamflow, and soil moisture data. Soil moisture data include point data (gravimetric, neutron probe and microwave) and remotely sensed data by aerial photography (passive microwave, infrared and gamma spectrums).

Among the intended uses for this data is to determine how best to include the various types of soil moisture data collected at different scales (point and remotely sensed measurements) in hydrologic models. Hopefully, inclusion of this data will produce better model predictions. The problem of how to combine soil moisture from various sources has been discussed extensively by Johnson et al. (1982) and the scale at which this data can be used is discussed by Wilkening and Ragan (1982).

Of prime interest to the HEC, is the potential advantage that this new source of soil moisture information has over antecedent precipitation index (API) in determining the initial conditions to be used in an event oriented watershed model, such as HEC-1. To include this information into HEC-1, a physically based and currently popular infiltration method of Green and Ampt (see Mein and Larson (1973)) will be included in HEC-1.

The Green and Ampt method expresses the relationship between cumulative infiltration, F, and infiltration rate, f, as:

$$F = \frac{\psi_f(\phi-\theta_i)}{(f/_k - 1)} \qquad f>k \tag{4}$$

where, k is the soil hydraulic conductivity at natural saturation, ψ_f, the average suction at the wetting front, ϕ, total porosity or volumetric water content at saturation and θ_i, initial water content. This method gives a direct means for including the initial soil moisture condition through the parameters ψ_f and θ_i.

The major stumbling blocks to this method are in applying the above relationship to actual rainfall amounts and estimation of the parameters of the method. The first stumbling block results because surface ponding must occur for the Green and Ampt equation to be valid. Mein and Larson (1973) for constant rainfall rates and Morel-Seytoux (1981) for variable rainfall rates describe a methodology for calculating a "time to ponding" (the time to ponding is essentially calculated as the time from the beginning of the storm at which the average rainfall intensity is equal to the infiltration rate). After this time, the Green and Ampt equation can be used as long as the rainfall rate exceeds the hydraulic conductivity. Of course, if the rainfall rate becomes less than the hydraulic conductivity then a soil moisture recovery will occur. During major storm events, this is unlikely to be a significant problem.

Parameters of the Green and Ampt equation can be estimated either by calibration or from information available from soil survey data. Rawls et al. (1982) have developed relationships between Green and Ampt parameters and readily available soil survey data. Their results were derived by making an extensive review of published soil water retention curves for different soil texture classes. The Green and Ampt parameters were calculated from the soil water retention relations by first parameterizing these relations with the Brooks and Corey (1964) equation,

$$S_e = \frac{\theta - \theta_r}{\phi - \theta_r} = (\psi_b / \psi)^{\lambda} \qquad (5)$$

where S_e equals the effective saturation, θ_r is the residual water content, ψ_b is the air entry or bubbling pressure and λ is the pore size distribution. Using this relationship and a technique recommended by Morel-Seytoux and Kahnji (1974), the average suction at the wetting front, ψ_f, was calculated. Note that ψ_f is dependent upon the assumed initial water content which in this case is the residual water content.

Table 2 displays the relationship between the Brooks and Corey, Green and Ampt, and soil texture class. Also listed is the variation that is expected in estimates of the Green and Ampt parameters based on texture class. Note that values given for hydraulic conductivity are only representative values and that, according to Rawls et al. (1982), hydraulic conductivity cannot be determined soley on the basis of texture class. These researchers found that greater confidence could be placed in estimates of the Green and Ampt parameters if soil water retention characteristics from a particular soil are known.

TABLE 2. HYDROLOGIC SOIL PROPERTIES CLASSIFIED BY SOIL TEXTURE

Texture class	Sample size	Total porosity (ϕ), cm^3/cm^3	Residual saturation (θ_r), cm^3/cm^3	Effective porosity (θ_e), cm^3/cm^3	Bubbling pressure (ψ_b): Arithmetic, cm	Bubbling pressure (ψ_b): Geometric**, cm	Pore size distribution (λ): Arithmetic	Pore size distribution (λ): Geometric**	Water retained at -0.33 bar tension, cm^3/cm^3	Water retained at -15 bar tension, cm^3/cm^3	Saturated Hydraulic Conductivity‡ (K_s), cm/h
Sand	762	0.437* (0.374-0.500)	0.020 (0.001-0.039)	0.417 (0.354-0.480)	15.98 (0.24-31.72)	7.26 (1.36-38.74)	0.694 (0.298-1.090)	0.592 (0.334-1.051)	0.091 (0.018-0.164)	0.033 (0.007-0.059)	21.00
Loamy sand	338	0.437 (0.368-0.506)	0.035 (0.003-0.067)	0.401 (0.329-0.473)	20.58 (0.0-45.20)	8.69 (1.80-41.85)	0.553 (0.234-0.872)	0.474 (0.271-0.827)	0.125 (0.060-0.190)	0.055 (0.019-0.091)	6.11
Sandy loam	666	0.453 (0.351-0.555)	0.041 (0.0-0.106)	0.412 (0.283-0.541)	30.20 (0.0-64.01)	14.66 (3.45-62.24)	0.378 (0.140-0.616)	0.322 (0.186-0.558)	0.207 (0.126-0.288)	0.095 (0.031-0.159)	2.59
Loam	383	0.463 (0.375-0.551)	0.027 (0.0-0.074)	0.434 (0.334-0.534)	40.12 (0.0-100.3)	11.15 (1.63-76.40)	0.252 (0.086-0.418)	0.220 (0.137-0.355)	0.270 (0.195-0.345)	0.117 (0.069-0.165)	1.32
Silt loam	1206	0.501 (0.420-0.582)	0.015 (0.0-0.058)	0.486 (0.394-0.578)	50.87 (0.0-109.4)	20.76 (3.58-120.4)	0.234 (0.105-0.363)	0.211 (0.136-0.326)	0.330 (0.258-0.402)	0.133 (0.078-0.188)	0.68
Sandy clay loam	498	0.398 (0.332-0.464)	0.068 (0.0-0.137)	0.330 (0.235-0.425)	59.41 (0.0-123.4)	28.08 (5.57-141.5)	0.319 (0.079-0.559)	0.250 (0.125-0.502)	0.255 (0.186-0.324)	0.148 (0.085-0.211)	0.43
Clay loam	366	0.464 (0.409-0.519)	0.075 (0.0-0.174)	0.390 (0.279-0.501)	56.43 (0.0-124.3)	25.89 (5.80-115.7)	0.242 (0.070-0.414)	0.194 (0.100-0.377)	0.318 (0.250-0.386)	0.197 (0.115-0.279)	0.23
Silty clay loam	689	0.471 (0.418-0.524)	0.040 (0.0-0.118)	0.432 (0.347-0.517)	70.33 (0.0-143.9)	32.56 (6.68-158.7)	0.177 (0.039-0.315)	0.151 (0.090-0.253)	0.366 (0.304-0.428)	0.208 (0.136-0.278)	0.15
Sandy clay	45	0.430 (0.370-0.490)	0.109 (0.0-0.205)	0.321 (0.207-0.435)	79.48 (0.0-179.1)	29.17 (4.96-171.6)	0.223 (0.048-0.398)	0.168 (0.078-0.364)	0.339 (0.245-0.433)	0.239 (0.162-0.316)	0.12
Silty clay	127	0.479 (0.425-0.533)	0.056 (0.0-0.136)	0.423 (0.334-0.512)	76.54 (0.0-159.6)	34.19 (7.04-166.2)	0.150 (0.040-0.260)	0.127 (0.074-0.219)	0.387 (0.332-0.442)	0.250 (0.193-0.307)	0.09
Clay	291	0.475 (0.427-0.523)	0.090 (0.0-0.195)	0.385 (0.269-0.501)	85.60 (0.0-176.1)	37.30 (7.43-187.2)	0.165 (0.037-0.293)	0.131 (0.068-0.253)	0.396 (0.326-0.466)	0.272 (0.208-0.336)	0.06

* First line is the mean value.
Second line is ± one standard deviation about the mean.
** Antilog of the log mean.
‡ Obtained from Fig. 2

(Rawls et al., 1982)

Initially this method will be tested on data currently available for small agricultural watersheds. Soil moisture parameters will probably be estimated based on an antecedent precipitation index. As data becomes available from the Dry Creek Project, soil moisture calculated from remotely sensed data will be used directly in the Green and Ampt equation.

LUMPED VERSUS DISTRIBUTED PARAMETER MODELS

HEC-1 calculates hydrologic responses which are average over specified increments of time and space. This is known as a "lumped" representation of the process. The real physical process varies widely in time and space. The lumped models account for spatial variation by allowing the user to specify various sizes of the process components (subbasin and routing reaches). The sizes are chosen (engineering judgment) to obtain the best definition of the runoff which is in keeping with the study objectives and budget. The time increment for the simulation is chosen likewise. Thus, virtually any spatial and temporal definition of the runoff can be obtained.

Work is currently underway at HEC to develop a terrain-based hydrologic model. The terrain is described by a grid of irregular triangular elements which follow slope, soil, land cover, etc., breaks in the watershed. The hydrologic process will be carried out on each of these finite elements. Streamflow will occur along rivlets and streams defined by the slopes/intersectons of the terrain elements.

SUMMARY

The major factors which determine the shape and size of a hydrograph were presented to set the stage for the infiltration process. The HEC-1 methodology for representing that infiltration process was described. Modelers were cautioned not to over emphasize one aspect of the runoff process at the expense of the components before and after it. Finally, the spatial and temporal definition of the runoff process by the models was discussed.

Hydrologic investigations most always result in the analysis of ungaged areas. Analysts are forced to extrapolate the calibrations made on gaged basins to areas where few data are available. The extrapolation process must rely on the hydrologist's ability to relate the parameters of the runoff process to the physical characteristics of the gaged and ungaged basins. In some models when the functions are primarily mathematical fits to the process, this can only be accomplished through the users experience with the model. Other models make use of readily measurable geographic characteristics of a watershed. Their parameters are much more easily transferred from gaged to ungaged areas. Thus, modelers of the hydrologic process should strive to describe that process with functions whose parameters are based on the physical characteristics of the watershed. Those functions must also be based on a sound theory of the physics of the process and still be practical for the intended applications of the model.

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The major factors which determine the shape and size of a hydrograph are presented to set the stage for the infiltration process. The HEC-1 methodology for representing that infiltration process is described. Modelers are cautioned not to over emphasize one aspect of the runoff process at the expense of the components before and after it. Finally, the spatial and temporal definition of the runoff process by the models was discussed.

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Appendix VIII-C

Loss Rates for Subbasins
Detailed Method (8-10.3.2)

The U.S. Department of Agriculture, Soil Conservation Service (SCS) has developed the following soil geographic databases:

- National Soil Geographic Database (NATSGO)[1]
- State Soil Geographic Database (STATSGO)[1]
- Soil Survey Geographic Database (SSURGO)[2]

Components of map units in each database are generally phases of soil series that enable the most precise interpretation. Interpretations are displayed differently for each geographic database to be consistent with differing levels of detail. The soil interpretations record database contains physical and chemical soil properties for about 18,000 soil series recognized in the United States.

Permeability

A soils property found in these databases and useful in PMF hydrologic studies is permeability.

- ***Definition***. Soil permeability is the quality of the soil that enables water or air to move through it. Accepted as a measure of this quality is the rate at which a saturated soil transmits water. This rate is the "saturated hydraulic conductivity" of soil physics and is expressed in inches per hour.

- ***Classes***. Soil permeability classes are listed below:

Permeability Class	Inches/Hour	Geometric Mean (Inches/Hour)
Extremely slow	0.0 - 0.01	0.0032
Very slow	0.01 - 0.06	0.0245
Slow	0.06 - 0.2	0.1095
Moderately slow	0.2 - 0.6	0.3464
Moderate	0.6 - 2.0	1.0954
Moderately rapid	2.0 - 6.0	3.4641
Rapid	6.0 - 20.0	10.9545
Very rapid	20.0 - 100.0	44.7200

1 Digital database complete.

2 Digital database to be developed over the next 10 years.

The STATSGO database (Exhibit 1) can be used to develop a detailed estimate of a basin's or subbasin's permeability (saturated hydraulic conductivity), a procedure of particular use when many soil types and associated infiltration rates exist within a basin. A discussion of the advantages of this procedure is given in Exhibit 2. For each soil series in the STATSGO database, the geometric mean permeability of the limiting (least permeable) soil layer should be used as the representative infiltration rate. The following steps provide a means to estimate excess precipitation while taking into account the variation of infiltration within a basin:

1. Calculate PMP rainfall in hourly increments (Exhibit 3).

2. Use a basin (subbasin) delineation (Exhibit 4) to identify the area, the STATSGO database to determine the percentage of the basin covered by each soil association identified within the basin (Exhibit 5), and Land Use and Land Cover maps to identify forested and wetland areas (database, Exhibit 6).

3. Use the STATSGO database to determine, for each soil association, the soil series percentage composition of each soil association unit (Exhibit 5).

4. Use the STATSGO database to identify the soil profile layer in each soil series with the minimum geometric mean value (i.e., the limiting layer) (Exhibit 7), and use that layer's range geometric mean permeability to represent that soil series' infiltration rate. A typical STATSGO soils association unit (e.g., MI131) is described by several different soil series, each potentially having several layers, each layer with its own range of permeability rates.[3] The STATSGO database will generally provide data to a depth of 5 feet, which will be adequate in most cases. In rapidly draining soils, an investigation of deeper depths may be necessary. A comparison of the available water capacity in the soil and the total depth of rainfall infiltrated during the critical hours of the PMP will provide guidance on the relevant soil profile depth to consider.

5. Use the results of steps 2, 3, and 4 to calculate the total area of the basin represented by each limiting geometric mean permeability, and formulate values of percent of total basin area with limiting geometric mean permeability values (Exhibit 8).

6. For each hour of the PMP, calculate the depth of excess rainfall for each limiting geometric mean infiltration rate category separately (Exhibit 8), multiply by the appropriate percentage of basin area (Exhibit 8), and sum the volumes by hour—thus calculating basin runoff for each storm hour (Exhibit 9) from each soil series' limiting layer infiltration rate.

7. Use the results of step 6 as the rainfall input, and set the loss function to zero in HEC-1.

Hydrologic model analyses of historic floods should be conducted using the infiltration rates from STATSGO, where possible. Some studies have found good agreement in cases where the watershed was saturated before the storm. When a dry period precedes the historic storm, the STATSGO method should overestimate the actual historic storm. If it does not, we would suspect that some physical feature other than soil permeability (high water table, shallow bedrock) is limiting the rate of infiltration.

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[3] The MI131 soil association consists of Lupton, Carbondale, Markey, Tawas, Cathro, Roscommon, Au Gres, Loxley, and Croswell. See Exhibit 7 for two examples of soil series within this association.

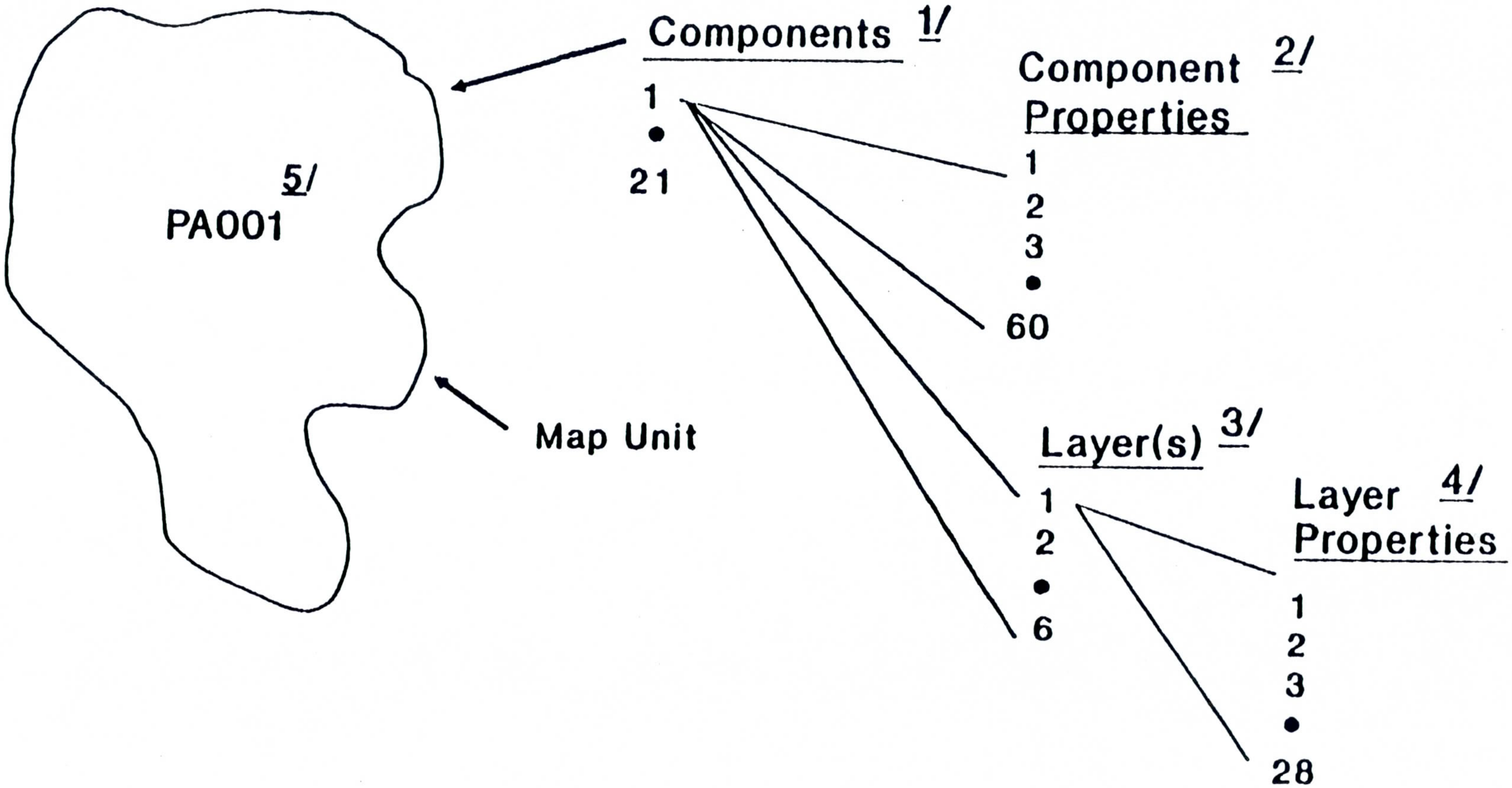
Exhibit 10 presents an excerpt from an actual report and a data plot from a PMF analysis on the river on which the project is located. In this case there was an antecedent condition of watershed saturation. Using the STATSGO approach in recreating a 1993 flood, a good correlation occurred with observed data.

EXHIBIT 1[4]

SCS STATSGO Database

[4] State Soil Geographic Data Base (STATSGO), Data Users Guide, U.S. Department of Agriculture, Soil Conservation Service, Miscellaneous Publication Number 1492.

Figure 1.—STATSGO MAP UNIT



1/ STATSGO map units consist of 1 to 21 components.
2/ For each component, there are 60 soil properties and interpretations in 84 different data elements (component tables); for example, flooding.
3/ There are 1-6 soil layers for each component.
4/ There are 28 soil properties for each layer; for example, percent clay.
5/ A symbol created by concatenation of the two-character State FIPS code and a three-digit Arabic number. It uniquely identifies a map unit within a State.

Distribution

Source

National Cartographic Center
U.S. Department of Agriculture
Soil Conservation Service
P.O. Box 6567
Fort Worth, TX 76115
(817) 334-5559

Format

The STATSGO spatial data are available in USGS Digital Line Graph (DLG-3) optional format. Map unit symbols (e.g., PA001) are not normally carried within the DLG-3 Optional formatted data; however, these map symbols are made available as a separate and unique ASCII file.

The STATSGO attribute data are stored in a relational data base format which is a nonfixed-length, tab-delimited ASCII file.

The SCS National Cartographic Center (NCC) operates a Geographic Resource Analysis Support System (GRASS) Geographic Information System (GIS) and an ARC/INFO GIS. SCS-GRASS and other formats may be made available by mutual agreement.

The STATSGO spatial and attribute data are distributed as one data set and are stored by USGS 1:250,000 1- by 2-degree quadrangle and distributed for a full State.

Medium

The distribution medium for spatial and attribute data will normally be 9-track magnetic tape at 1600 bits per inch, but may be cartridge tape by mutual agreement.

Ordering

Before ordering STATSGO data, the user needs to identify the State(s) of interest and may wish to consult a USGS index to the 1:250,000 base map series to ensure coverage. Additional information and costs may be obtained from NCC.

The STATSGO data are periodically updated, data files are dated, and users are responsible for obtaining the latest version.

EXHIBIT 2

"Lumped" Versus Distributed Loss Rate Parameters on a Watershed with Diverse Soil Permeabilities

September 1994

EXHIBIT 2

"Lumped" Versus Distributed Loss Rate Parameters on a Watershed with Diverse Soil Permeabilities

The use of spatially "lumped" (rather than distributed) infiltration parameters can produce misleading model results when applied to a watershed with highly variable soil infiltration characteristics. This problem became apparent in a December 1991 study, *Addendum to Runoff Curve Number Determination*. This report included a PMF estimate using the Green-Ampt infiltration equation, in which the most important variable is the saturated hydraulic conductivity (permeability) of the soils. That study yielded what appeared to be an unrealistically low estimate of the PMF. Upon inspection of the model computations, it was apparent that the area-weighted average infiltration rates exceeded the maximum PMP rainfall rate everywhere. Therefore, the model only predicted runoff from the impervious surfaces of the basin.

The purpose of this exhibit is to demonstrate with a quantitative example why, for a simplified basin with both contributing and highly permeable (essentially non-contributing) areas, both model calibration and model prediction can be in error when the infiltration characteristics of the two types of areas are spatially averaged. Although the example is very simple, the conclusions can be extended to more complex, real watersheds.

EXAMPLE:

1. Assume: **100-square-mile basin, of which:**
90 square miles have infiltration rate of **6 inches/hour**
10 square miles have infiltration rate of **0 inches/hour**

The calibration storm's peak 1-hour period produced **2 inches per hour**;
The maximum PMP increment is **4 inches per hour**.

2. **Calculate:** The basin average infiltration (loss) rate, based on the given infiltration rates, equals:

(10 square miles X 0 inches/hr + 90 square miles X 6 inches/hr)/(100 sq. mi) = 5.4 inches/hour.

3. **Consider:** Only the peak hour of each storm. Then a total of **2 inches** fell in the calibration storm, and a total of **4 inches** will fall in the PMP.

4. **Calculate:** The volume of runoff produced for:

(1) the calibration storm **(2 inches per hour)**:

90 square miles of the basin produced **0 inches** of runoff,
because 2 inches/hour < infiltration rate;
10 square miles of the basin produced **2 inches** of runoff.

Then the area average runoff is:
(10 square miles X 2 inches + 90 sq. mi X 0 inches)/100 sq. mi = 0.2 inches

And the calibrated loss rate is
(2 inches-0.2 inches)/ 1 hour = 1.8 inches/hour.

(2) the PMP **(4 inches per hour)**.
Note that if the basin average of 5.4 inches/hour is applied to the PMP, the model will predict 0 runoff. But if the areas are considered separately,

90 square miles produces 0 inches of runoff, because the PMP is still less than the infiltration rate, and
10 square miles produces 4 inches of runoff.

Then the area average runoff is:
(10 square miles X 4 inches + 90 square miles X 0 inches)/100 square miles = 0.4 inches.

And the average loss rate is **(4 inches - 0.4 inches)/1 hour = 3.6 inches per hour.**

(3) a larger PMP of **6 inches per hour**.

90 square miles produces **0 inches** of runoff;
10 square miles produces **6 inches** of runoff.

The area average runoff is:
(10 square miles X 6 inches + 90 square miles X 0 inches)/100 square miles = 0.6 inches

and the area average loss rate is:
(6 inches - 0.6 inches)/1 hour = 5.4 inches/hour, which is equal to the basin average loss rate estimated just from soil data.

(4) Finally, consider an even larger PMP of **8 inches per hour.**

90 square miles produces **2 inches** of runoff;
10 square miles produces **8 inches** of runoff.

The area average runoff is:
(90 square miles X 2 inches + 10 square miles X 8 inches)/(100 square miles) = 2.6 inches.

and the area average loss rate is
(8 inches - 2.6 inches)/1 hour = 5.4 inches per hour.

Conclusions to be drawn from the above example are as follows:

(1) The observed infiltration rate for a given storm increases with increasing rainfall intensity, until the rainfall intensity equals the maximum soil infiltration rate. For rainfall intensities equal to or higher than the maximum infiltration rate, the basin weighted average infiltration rate (from soil data) is appropriate.

(2) If a loss rate is calibrated on a historic storm (which is smaller than the PMP), the use of the calibrated loss rate for the PMP will tend to **overestimate** runoff from the PMP.

(3) If only infiltration rates given in soils literature are used, and the PMP modeled is less than the infiltration rate of the most permeable soil, the model will tend to **underestimate** runoff due to the PMP.

This problem is likely to be much less severe on most other basins, where published soil infiltration rates are less than, or close to, the PMP. However, in some cases, the above example makes clear why characterizing the basin (or any subbasin) by a single infiltration rate will produce highly inconsistent and unreliable results.

EXHIBIT 3

Precipitation in Hourly Increments

–Warm Season
–Cool Season

September 1994

Warm Season

TIME	DATE	BORD WS PRECIP-INC INCHES INST-VAL
0100	01AUG1999	.043
0200	01AUG1999	.043
0300	01AUG1999	.043
0400	01AUG1999	.043
0500	01AUG1999	.043
0600	01AUG1999	.043
0700	01AUG1999	.053
0800	01AUG1999	.053
0900	01AUG1999	.053
1000	01AUG1999	.053
1100	01AUG1999	.053
1200	01AUG1999	.053
1300	01AUG1999	.067
1400	01AUG1999	.067
1500	01AUG1999	.067
1600	01AUG1999	.067
1700	01AUG1999	.067
1800	01AUG1999	.067
1900	01AUG1999	.093
2000	01AUG1999	.093
2100	01AUG1999	.093
2200	01AUG1999	.093
2300	01AUG1999	.093
2400	01AUG1999	.093
0100	02AUG1999	.133
0200	02AUG1999	.138
0300	02AUG1999	.145
0400	02AUG1999	.153
0500	02AUG1999	.161
0600	02AUG1999	.171
0700	02AUG1999	.283
0800	02AUG1999	.306
0900	02AUG1999	.341
1000	02AUG1999	.390
1100	02AUG1999	.450
1200	02AUG1999	.524
1300	02AUG1999	.798
1400	02AUG1999	1.305
1500	02AUG1999	2.017
1600	02AUG1999	4.876
1700	02AUG1999	1.711
1800	02AUG1999	1.142
1900	02AUG1999	.262
2000	02AUG1999	.238
2100	02AUG1999	.218
2200	02AUG1999	.202
2300	02AUG1999	.189
2400	02AUG1999	.180
0100	03AUG1999	.115
0200	03AUG1999	.115

TIME	DATE	BCRD WS PRECIP-INC INCHES INST-VAL
0300	03AUG1999	.115
0400	03AUG1999	.115
0500	03AUG1999	.115
0600	03AUG1999	.115
0700	03AUG1999	.078
0800	03AUG1999	.078
0900	03AUG1999	.078
1000	03AUG1999	.078
1100	03AUG1999	.078
1200	03AUG1999	.078
1300	03AUG1999	.059
1400	03AUG1999	.059
1500	03AUG1999	.059
1600	03AUG1999	.059
1700	03AUG1999	.059
1800	03AUG1999	.059
1900	03AUG1999	.048
2000	03AUG1999	.048
2100	03AUG1999	.048
2200	03AUG1999	.048
2300	03AUG1999	.048
2400	03AUG1999	.048

Cool Season

TIME DATE	BCRD CS PRECIP-INC INCHES INST-VAL	BCRD CS SNOW-INC INCHES INST-VAL	BCRD CS PCP_SNOW-INC INCHES INST-VAL
0100 01AUG1999	.027	.027	.054
0200 01AUG1999	.027	.027	.054
0300 01AUG1999	.027	.027	.054
0400 01AUG1999	.027	.027	.054
0500 01AUG1999	.027	.027	.054
0600 01AUG1999	.027	.027	.054
0700 01AUG1999	.033	.027	.060
0800 01AUG1999	.033	.027	.060
0900 01AUG1999	.033	.027	.060
1000 01AUG1999	.033	.027	.060
1100 01AUG1999	.033	.027	.060
1200 01AUG1999	.033	.027	.060
1300 01AUG1999	.042	.027	.069
1400 01AUG1999	.042	.027	.069
1500 01AUG1999	.042	.027	.069
1600 01AUG1999	.042	.027	.069
1700 01AUG1999	.042	.027	.069
1800 01AUG1999	.042	.027	.069
1900 01AUG1999	.057	.027	.084
2000 01AUG1999	.057	.027	.084
2100 01AUG1999	.057	.027	.084
2200 01AUG1999	.057	.027	.084
2300 01AUG1999	.057	.027	.084
24[illegible]AUG1999	.057	.027	.084
[illegible]02AUG1999	.081	.038	.119
0200 02AUG1999	.084	.038	.122
0300 02AUG1999	.088	.038	.126
0400 02AUG1999	.092	.038	.130
0500 02AUG1999	.098	.038	.136
0600 02AUG1999	.104	.038	.142
0700 02AUG1999	.170	.038	.208
0800 02AUG1999	.185	.038	.223
0900 02AUG1999	.205	.038	.243
1000 02AUG1999	.230	.038	.268
1100 02AUG1999	.261	.038	.299
1200 02AUG1999	.297	.038	.335
1300 02AUG1999	.416	.038	.454
1400 02AUG1999	.632	.038	.670
1500 02AUG1999	.923	.038	.961
1600 02AUG1999	1.890	.038	1.928
1700 02AUG1999	.797	.038	.835
1800 02AUG1999	.562	.038	.600
1900 02AUG1999	.157	.038	.195
2000 02AUG1999	.143	.038	.181
2100 02AUG1999	.131	.038	.169
2200 02AUG1999	.122	.038	.160
2300 02AUG1999	.114	.038	.152
2400 02AUG1999	.109	.038	.147
0100 03AUG1999	.070	.035	.105
0200 03AUG1999	.070	.035	.105

TIME	DATE	BCRD CS PRECIP-INC INCHES INST-VAL	BCRD CS SNOW-INC INCHES INST-VAL	BCRD CS PCP_SNOW-INC INCHES INST-VAL
0300	03AUG1999	.070	.035	.105
0400	03AUG1999	.070	.035	.105
0500	03AUG1999	.070	.035	.105
0600	03AUG1999	.070	.035	.105
0700	03AUG1999	.048	.035	.083
0800	03AUG1999	.048	.035	.083
0900	03AUG1999	.048	.035	.083
1000	03AUG1999	.048	.035	.083
1100	03AUG1999	.048	.035	.083
1200	03AUG1999	.048	.035	.083
1300	03AUG1999	.037	.035	.072
1400	03AUG1999	.037	.035	.072
1500	03AUG1999	.037	.035	.072
1600	03AUG1999	.037	.035	.072
1700	03AUG1999	.037	.035	.072
1800	03AUG1999	.037	.035	.072
1900	03AUG1999	.030	.035	.065
2000	03AUG1999	.030	.035	.065
2100	03AUG1999	.030	.035	.065
2200	03AUG1999	.030	.035	.065
2300	03AUG1999	.030	.035	.065
2400	03AUG1999	.030	.035	.065
0100	04AUG1999		.016	.016
0200	04AUG1999		.016	.016
0300	04AUG1999		.016	.016
0400	04AUG1999		.016	.016
0500	04AUG1999		.016	.016
0600	04AUG1999		.016	.016
0700	04AUG1999		.016	.016
0800	04AUG1999		.016	.016
0900	04AUG1999		.016	.016
1000	04AUG1999		.016	.016
1100	04AUG1999		.016	.016
1200	04AUG1999		.016	.016
1300	04AUG1999		.016	.016
1400	04AUG1999		.016	.016
1500	04AUG1999		.016	.016
1600	04AUG1999		.016	.016
1700	04AUG1999		.016	.016
1800	04AUG1999		.016	.016
1900	04AUG1999		.016	.016
2000	04AUG1999		.016	.016
2100	04AUG1999		.016	.016
2200	04AUG1999		.016	.016
2300	04AUG1999		.016	.016
2400	04AUG1999		.016	.016
0100	05AUG1999		.000	.000
0200	05AUG1999		.000	.000
0300	05AUG1999		.000	.000
0400	05AUG1999		.000	.000

TIME	DATE	BCRD CS PRECIP-INC INCHES INST-VAL	BCRD CS SNOW-INC INCHES INST-VAL	BCRD CS PCP_SNOW-INC INCHES INST-VAL
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0600	05AUG1999		.000	.000
0700	05AUG1999		.000	.000
0800	05AUG1999		.000	.000
0900	05AUG1999		.000	.000
1000	05AUG1999		.000	.000
1100	05AUG1999		.000	.000
1200	05AUG1999		.000	.000
1300	05AUG1999		.000	.000
1400	05AUG1999		.000	.000
1500	05AUG1999		.000	.000
1600	05AUG1999		.000	.000
1700	05AUG1999		.000	.000
1800	05AUG1999		.000	.000
1900	05AUG1999		.000	.000
2000	05AUG1999		.000	.000
2100	05AUG1999		.000	.000
2200	05AUG1999		.000	.000
2300	05AUG1999		.000	.000
2400	05AUG1999		.000	.000
0100	06AUG1999		.000	.000
0200	06AUG1999		.000	.000
0300	06AUG1999		.000	.000
04[illegible]	[illegible]6AUG1999		.000	.000
05[illegible]	[illegible]6AUG1999		.000	.000
0600	06AUG1999		.000	.000
0700	06AUG1999		.000	.000
0800	06AUG1999		.000	.000
0900	06AUG1999		.000	.000
1000	06AUG1999		.000	.000
1100	06AUG1999		.000	.000
1200	06AUG1999		.000	.000
1300	06AUG1999		.000	.000
1400	06AUG1999		.000	.000
1500	06AUG1999		.000	.000
1600	06AUG1999		.000	.000
1700	06AUG1999		.000	.000
1800	06AUG1999		.000	.000
1900	06AUG1999		.000	.000
2000	06AUG1999		.000	.000
2100	06AUG1999		.000	.000
2200	06AUG1999		.000	.000
2300	06AUG1999		.000	.000
2400	06AUG1999		.000	.000
0100	07AUG1999		.014	.014
0200	07AUG1999		.014	.014
0300	07AUG1999		.014	.014
0400	07AUG1999		.014	.014
0500	07AUG1999		.014	.014
0600	07AUG1999		.014	.014

TIME	DATE	BCRD CS PRECIP-INC INCHES INST-VAL	BCRD CS SNOW-INC INCHES INST-VAL	BCRD CS PCP_SNOW-INC INCHES INST-VAL
0700	07AUG1999		.014	.014
0800	07AUG1999		.014	.014
0900	07AUG1999		.014	.014
1000	07AUG1999		.014	.014
1100	07AUG1999		.014	.014
1200	07AUG1999		.014	.014
1300	07AUG1999		.014	.014
1400	07AUG1999		.014	.014
1500	07AUG1999		.014	.014
1600	07AUG1999		.014	.014
1700	07AUG1999		.014	.014
1800	07AUG1999		.014	.014
1900	07AUG1999		.014	.014
2000	07AUG1999		.014	.014
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0400	08AUG1999		.011	.011
0500	08AUG1999		.011	.011
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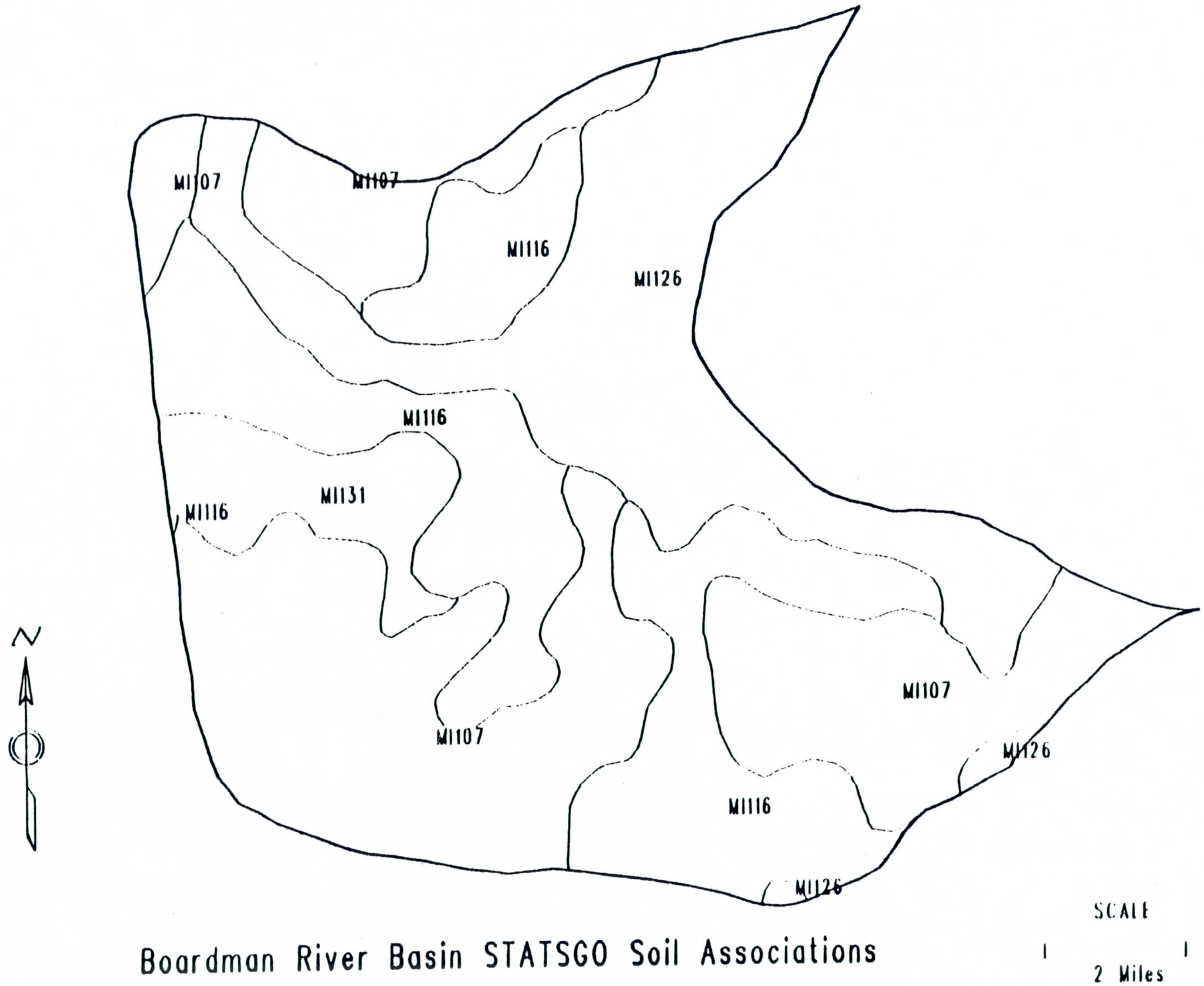
0800	08AUG1999		.011	.011
0900	08AUG1999		.011	.011
1000	08AUG1999		.011	.011
1100	08AUG1999		.011	.011
1200	08AUG1999		.011	.011
1300	08AUG1999		.011	.011
1400	08AUG1999		.011	.011
1500	08AUG1999		.011	.011
1600	08AUG1999		.011	.011
1700	08AUG1999		.011	.011
1800	08AUG1999		.011	.011
1900	08AUG1999		.011	.011
2000	08AUG1999		.011	.011
2100	08AUG1999		.011	.011
2200	08AUG1999		.011	.011
2300	08AUG1999		.011	.011
2400	08AUG1999		.011	.011
0100	09AUG1999		.002	.002
0200	09AUG1999		.002	.002
0300	09AUG1999		.002	.002
0400	09AUG1999		.002	.002
0500	09AUG1999		.002	.002
0600	09AUG1999		.002	.002
0700	09AUG1999		.002	.002
0800	09AUG1999		.002	.002

TIME	DATE	BCRD CS PRECIP-INC INCHES INST-VAL	BCRD CS SNOW-INC INCHES INST-VAL	BCRD CS PCP_SNOW-INC INCHES INST-VAL
0900	09AUG1999		.002	.002
1000	09AUG1999		.002	.002
1100	09AUG1999		.002	.002
1200	09AUG1999		.002	.002
1300	09AUG1999		.002	.002
1400	09AUG1999		.002	.002
1500	09AUG1999		.002	.002
1600	09AUG1999		.002	.002
1700	09AUG1999		.002	.002
1800	09AUG1999		.002	.002
1900	09AUG1999		.002	.002
2000	09AUG1999		.002	.002
2100	09AUG1999		.002	.002
2200	09AUG1999		.002	.002
2300	09AUG1999		.002	.002
2400	09AUG1999		.002	.002
0100	10AUG1999		.010	.010
0200	10AUG1999		.010	.010
0300	10AUG1999		.010	.010
0400	10AUG1999		.010	.010
0500	10AUG1999		.010	.010
0600	10AUG1999		.010	.010
0700	10AUG1999		.010	.010
08[illegible]	[illegible]AUG1999		.010	.010
0900	10AUG1999		.010	.010
1000	10AUG1999		.010	.010
1100	10AUG1999		.010	.010
1200	10AUG1999		.010	.010
1300	10AUG1999		.010	.010
1400	10AUG1999		.010	.010
1500	10AUG1999		.010	.010
1600	10AUG1999		.010	.010
1700	10AUG1999		.010	.010
1800	10AUG1999		.010	.010
1900	10AUG1999		.010	.010
2000	10AUG1999		.010	.010
2100	10AUG1999		.010	.010
2200	10AUG1999		.010	.010
2300	10AUG1999		.010	.010
2400	10AUG1999		.010	.010

EXHIBIT 4

Basin (Subbasin) and Soil Association Delineations

September 1994



Boardman River Basin STATSGO Soil Associations

EXHIBIT 5

Percentage of Area of Soil Series Within the Basin (Subbasin)

–60-inch Permeability Values
–24-inch Permeability Values

SAU		Soil Series	% in Assoc.	Perm	Perm
A5,3X,I2,1X,A22,F2.0,1X,F5.2,1X,F5.2)					
MI107	1	EMMET	11	1.30	4.00
MI107	2	EMMET	14	1.30	4.00
MI107	3	EMMET	9	1.30	4.00
MI107	4	EMMET	6	1.30	4.00
MI107	5	EMMET	7	1.30	4.00
MI107	6	MONTCALM	7	4.00	4.00
MI107	7	MONTCALM	9	4.00	4.00
MI107	8	MONTCALM	10	4.00	4.00
MI107	9	MONTCALM	5	4.00	4.00
MI107	10	MONTCALM	4	4.00	4.00
MI107	11	LEELANAU	1	4.00	13.00
MI107	12	LEELANAU	1	4.00	13.00
MI107	13	LEELANAU	2	4.00	13.00
MI107	14	KALKASKA	2	13.00	13.00
MI107	15	KALKASKA	5	13.00	13.00
MI107	16	OMENA	2	1.30	1.30
MI107	17	OMENA	1	1.30	1.30
MI107	18	OMENA	2	1.30	1.30
MI107	19	RUBICON	1	13.00	13.00
MI107	20	CHARLEVOIX	1	3.30	3.30
MI116	1	KALKASKA	8	13.00	13.00
MI116	2	KALKASKA	24	13.00	13.00
MI116	3	KALKASKA	8	13.00	13.00
MI116	4	KALKASKA	6	13.00	13.00
MI116	5	KALKASKA	7	13.00	13.00
MI116	6	LEELANAU	5	4.00	13.00
MI116	7	LEELANAU	5	4.00	13.00
MI116	8	LEELANAU	2	4.00	13.00
MI116	9	LEELANAU	5	4.00	13.00
MI116	10	MONTCALM	5	4.00	4.00
MI116	11	MONTCALM	4	4.00	4.00
MI116	12	EMMET	3	1.30	4.00
MI116	13	EMMET	2	1.30	4.00
MI116	14	EMMET	7	1.30	4.00
MI116	15	EAST LAKE	1	13.00	13.00
MI116	16	EAST LAKE	1	13.00	13.00
MI116	17	MANCELONA	2	4.00	4.00
MI116	18	MANCELONA	2	4.00	4.00
MI116	19	RUBICON	1	13.00	13.00
MI116	20	BELDING	1	0.40	4.00
MI116	21	BRECKENRIDGE	1	0.40	3.30
MI126	1	RUBICON	48	13.00	13.00
MI126	2	GRAYLING	46	13.00	13.00
MI126	3	CROSWELL	2	13.00	13.00
MI126	4	AU GRES	2	13.00	13.00
MI126	5	ROSCOMMON	2	13.00	13.00
MI131	1	LUPTON	35	3.10	3.10
MI131	2	CARBONDALE	13	3.10	3.10
MI131	3	MARKEY	5	3.10	3.10
MI131	4	TAWAS	13	3.10	3.10
MI131	5	CATHRO	7	1.10	1.10
MI131	6	ROSCOMMON	8	13.00	13.00
MI131	7	AU GRES	12	13.00	13.00
MI131	8	LOXLEY	2	3.10	3.10
MI131	9	CROSWELL	5	13.00	13.00

← Soil Association Unit Example (SAU)

-X,A5,4X,A2,7X,F6.0)

MI107	11	54
MI107	12	21
MI107	14	17
'07	17	7
07	21	15014
MI107	22	351
MI107	41	3459
MI107	42	2142
MI107	43	5588
MI107	52	63
MI107	61	177
MI107	75	145
MI116	0	5
MI116	11	237
MI116	12	40
MI116	16	36
MI116	21	5428
MI116	22	87
MI116	41	3963
MI116	42	3161
MI116	43	7034
MI116	52	39
MI116	61	3
MI126	11	604
MI126	12	3
MI126	21	1184
MI126	41	1847
MI126	42	2382
MI126	43	10252
MI126	52	859
MI126	53	243
MI126	61	83
.26	75	42
MI131	21	104
MI131	41	934
MI131	43	1808
MI131	61	1237
MI131	75	36

EXHIBIT 6

Land Use and Land Cover Digital Data from 1:250,000– and 1:100,000–Scale Maps[5]

[5] *U.S. Geological Survey, National Mapping Program Technical Instructions, Data Users Guide 4.*

Table 1.--U.S. Geological Survey Land Use and Land Cover Classification System for Use with Remote Sensor Data

LEVEL 1		LEVEL II	
1	Urban or Built-up Land	11	Residential
		12	Commercial and Services
		13	Industrial
		14	Transportation, Communications and Utilities
		15	Industrial and Commercial Complexes
		16	Mixed Urban or Built-up Land
		17	Other Urban or Built-up Land
2	Agricultural Land	21	Cropland and Pasture
		22	Orchards, Groves, Vineyards, Nurseries, and Ornamental Horticultural Areas
		23	Confined Feeding Operations
		24	Other Agricultural Land
3	Rangeland	31	Herbaceous Rangeland
		32	Shrub and Brush Rangeland
		33	Mixed Rangeland
4	Forest Land	41	Deciduous Forest Land
		42	Evergreen Forest Land
		43	Mixed Forest Land
5	Water	51	Streams and Canals
		52	Lakes
		53	Reservoirs
		54	Bays and Estuaries
6	Wetland	61	Forested Wetland
		62	Nonforested Wetland
7	Barren Land	71	Dry Salt Flats
		72	Beaches
		73	Sandy Areas Other than Beaches
		74	Bare Exposed Rock
		75	Strip Mines, Quarries, and Gravel Pits
		76	Transitional Areas
		77	Mixed Barren Land
8	Tundra	81	Shrub and Brush Tundra
		82	Herbaceous Tundra
		83	Bare Ground
		84	Wet Tundra
		85	Mixed Tundra
9	Perennial Snow or Ice	91	Perennial Snowfields
		92	Glaciers

Hydrologic Unit Map

The hydrologic unit map is based on the Hydrologic Unit Maps published by the USGS Office of Water Data Coordination, together with the list "Boundary descriptions and name of region, subregion, accounting units, and cataloging unit" or USGS Circular 878-A, Codes for the Identification of Hydrologic Units in the United States and the Caribbean Outlying Areas (U.S. Geological Survey, 1982). The hydrologic units are encoded with an eight-digit number that indicates the

EXHIBIT 7

Soil Series Interpretations Record
Lupton and Carbondale Series

September 1994

MLRA(S): 90, 91, 92, 93, 94A, 94B, 96, 142, 144B, 145
REV. LWB, 6-88
TYPIC BOROSAPRISTS, EUIC

LUPTON SERIES

THE LUPTON SERIES CONSISTS OF VERY POORLY DRAINED SOILS FORMED IN WOODY ORGANIC DEPOSITS IN
LAKE PLAINS, OUTWASH PLAINS AND TILL PLAINS. THE SURFACE LAYER IS BLACK SAPRIC MATERIAL ...
IS DARK REDDISH BROWN AND VERY DARK BROWN SAPRIC MATERIAL. SLOPES ARE LESS THAN 2 PERCENT.
WOODLAND.

ESTIMATED SOIL PROPERTIES (A)

DEPTH (IN.)	USDA TEXTURE	UNIFIED	AASHTO	FRACT >3 IN (PCT)	PERCENT OF MA[TERIAL LESS] THAN 3" PASS[ING SIEVE NO.] 4	10
0-10	SP, MUCK	PT	A-8	0		
0-10	HM, MPT	PT	A-8	0		
10-65	SP	PT	A-8	0		

DEPTH (IN.)	CLAY (PCT)	MOIST BULK DENSITY (G/CM3)	PERMEABILITY (IN/HR)	AVAILABLE WATER CAPACITY (IN/IN)	SOIL REACTION (PH)	SALINITY (MMHOS/CM)	SHRINK-SWELL POTENTIAL	EROSION FACTORS K	T	WIND EROD. GROUP	ORGANIC MATTER (PCT)	CORROSIVITY STEEL	CONCRETE
0-10		0.10-0.35	0.2-6.0	0.35-0.45	5.6-7.8	-		-	5	2	70-90	HIGH	LOW
0-10		0.20-0.35	0.6-6.0	0.45-0.55	5.6-7.8	-		-	5	5	70-90		
10-65		0.10-0.35	0.2-6.0	0.35-0.45	5.6-7.8	-		-					

...[FLOODING]	HIGH WATER TABLE DEPTH (FT)	KIND	MONTHS	CEMENTED PAN DEPTH (IN)	HARDNESS	BEDROCK DEPTH (IN)	HARDNESS	SUBSIDENCE INIT. (IN)	TOTAL (IN)	HYD GRP	POTENTIAL FROST ACTION
	+1-1.0	APPARENT	SEP-MAY	-		>60			50-55	A/D	HIGH

...TIES

	...NDING, PERCS SLOWLY
SEWAGE LAGOON AREAS	...ESS HUMUS, PONDING
SANITARY LANDFILL (TRENCH)	SEVERE-PONDING, EXCESS HUMUS
SANITARY LANDFILL (AREA)	SEVERE-SEEPAGE, PONDING
DAILY COVER FOR LANDFILL	POOR-PONDING, EXCESS HUMUS

CONSTRUCTION MATERIAL

ROADFILL	POOR-WETNESS, LOW STRENGTH
SAND	IMPROBABLE-EXCESS HUMUS
GRAVEL	IMPROBABLE-EXCESS HUMUS
TOPSOIL	POOR-EXCESS HUMUS, WETNESS

BUILDING SITE DEVELOPMENT

SHALLOW EXCAVATIONS	SEVERE-EXCESS HUMUS, PONDING
DWELLINGS WITHOUT BASEMENTS	SEVERE-SUBSIDES, PONDING, LOW STRENGTH
DWELLINGS WITH BASEMENTS	SEVERE-SUBSIDES, PONDING, LOW STRENGTH
SMALL COMMERCIAL BUILDINGS	SEVERE-SUBSIDES, PONDING, LOW STRENGTH
LOCAL ROADS AND STREETS	SEVERE-PONDING, FROST ACTION, SUBSIDES
LAWNS, LANDSCAPING AND GOLF FAIRWAYS	SEVERE-PONDING, EXCESS HUMUS

WATER MANAGEMENT

POND RESERVOIR AREA	SEVERE-SEEPAGE
EMBANKMENTS DIKES AND LEVEES	SEVERE-EXCESS HUMUS, PONDING
EXCAVATED PONDS AQUIFER FED	SEVERE-SLOW REFILL
DRAINAGE	PONDING, SUBSIDES, FROST ACTION
IRRIGATION	HM, MPT: PONDING SP, MUCK: PONDING, SOIL BLOWING
TERRACES AND DIVERSIONS	HM, MPT: PONDING SP, MUCK: PONDING, SOIL BLOWING
GRASSED WATERWAYS	WETNESS

REGIONAL INTERPRETATIONS

MLRA(S): 95A, 90, 93, 94A, 94B, 142, 143, 96, 147
REV. SHE, 4-85
HEMIC BOROSAPRISTS, EUIC

CARBONDALE SERIES

THE CARBONDALE SERIES CONSISTS OF VERY POORLY DRAINED SOILS FORMED IN HERBACEOUS ORGANIC DEPOSITS IN DEPRESSIONS ON TILL PLAINS, OUTWASH PLAINS AND LAKE PLAINS. THE SURFACE LAYER IS VERY DARK GRAY MUCK 3 INCHES THICK. THE SUBSTRATUM IS 36 INCHES OF VERY DARK BROWN AND BLACK MUCK OVER DARK BROWN MUCKY PEAT. SLOPES ARE 0 TO 2 PERCENT. WOODLAND IS THE MAIN USE.

ESTIMATED SOIL PROPERTIES

DEPTH (IN.)	USDA TEXTURE	UNIFIED	AASHTO	FRACT >3 IN (PCT)	PERCENT OF MATERIAL LESS THAN 3" PASSING SIEVE NO. 4	10	40	200	LIQUID LIMIT	PLASTICITY INDEX
0-39	SP	PT	A-8	0						
39-60	MM	PT	A-8	0						

DEPTH (IN.)	CLAY (PCT)	MOIST BULK DENSITY (G/CM3)	PERMEABILITY (IN/HR)	AVAILABLE WATER CAPACITY (IN/IN)	SOIL REACTION (PH)	SALINITY (MMHOS/CM)	SHRINK-SWELL POTENTIAL	EROSION FACTORS K	T	WIND EROD. GROUP	ORGANIC MATTER (PCT)	CORROSIVITY STEEL	CONCRETE
0-39	-	0.15-0.40	0.2-6.0	0.35-0.45	5.6-7.8	-		-	2	2	40-65	HIGH	MODERATE
39-60	-	0.10-0.20	0.6-6.0	0.45-0.55	5.6-7.8	-		-					

FLOODING FREQUENCY	DURATION	MONTHS	HIGH WATER TABLE DEPTH (FT)	KIND	MONTHS	CEMENTED PAN DEPTH (IN)	HARDNESS	BEDROCK DEPTH (IN)	HARDNESS	SUBSIDENCE INIT. (IN)	TOTAL (IN)	HYD GRP	POTENTIAL FROST ACTION
NONE			+1-1.0	APPARENT	SEP-MAY	-		>60			45-54	A/D	HIGH

SANITARY FACILITIES

SEPTIC TANK ABSORPTION FIELDS	SEVERE-SUBSIDES,PONDING,PERCS SLOWLY
SEWAGE LAGOON AREAS	SEVERE-SEEPAGE,EXCESS HUMUS,PONDING
SANITARY LANDFILL (TRENCH)	SEVERE-SEEPAGE,PONDING,EXCESS HUMUS
SANITARY LANDFILL (AREA)	SEVERE-SEEPAGE,PONDING
DAILY COVER FOR LANDFILL	POOR-PONDING,EXCESS HUMUS

CONSTRUCTION MATERIAL

ROADFILL	POOR-WETNESS
SAND	IMPROBABLE-EXCESS HUMUS
GRAVEL	IMPROBABLE-EXCESS HUMUS
TOPSOIL	POOR-EXCESS HUMUS,WETNESS

BUILDING SITE DEVELOPMENT

SHALLOW EXCAVATIONS	SEVERE-EXCESS HUMUS,PONDING
DWELLINGS WITHOUT BASEMENTS	SEVERE-SUBSIDES,PONDING,LOW STRENGTH
DWELLINGS WITH BASEMENTS	SEVERE-SUBSIDES,PONDING,LOW STRENGTH
SMALL COMMERCIAL BUILDINGS	SEVERE-SUBSIDES,PONDING,LOW STRENGTH
LOCAL ROADS AND STREETS	SEVERE-SUBSIDES,PONDING,FROST ACTION
LAWNS, LANDSCAPING AND GOLF FAIRWAYS	SEVERE-PONDING,EXCESS HUMUS

WATER MANAGEMENT

POND RESERVOIR AREA	SEVERE-SEEPAGE
EMBANKMENTS DIKES AND LEVEES	SEVERE-EXCESS HUMUS,PONDING
EXCAVATED PONDS AQUIFER FED	SEVERE-SLOW REFILL
DRAINAGE	PONDING,SUBSIDES,FROST ACTION
IRRIGATION	PONDING,SOIL BLOWING
TERRACES AND DIVERSIONS	PONDING,SOIL BLOWING
GRASSED WATERWAYS	WETNESS

REGIONAL INTERPRETATIONS

EXHIBIT 8

Percentage of Basin Area Versus Infiltration Rate

–Warm Season
–Cool Season

September 1994

THIS OUTPUT CREATED BY PROGRAM "RUNOFFQ.EXE ", DATED 2/10/94

DSS FILE = TRAVWARM .DSS
S[illegible] LE = TRAVWARM .SOI

BCRDBCRD.OWS

AREA FILE = BCRD.ARE
PATHNAME = /BCRD/BCRD/PRECIP-INC//1HOUR/WS/

WARM SEASON CONDITIONS

TOTAL AREA IN BASIN = 68699.000000

PERCENT FOREST =61.97%
PERCENT STORAGE = 2.18%

% IN BASIN	INF. RATE
23.7500	1.30
23.9500	4.00
45.7500	13.00
.3900	3.30
2.1900	.00
.5800	.40
2.3600	3.10
.2900	1.10

SUM OF PERCENTAGES = 99.76

```
RRRRR   U    U   N     N    OOOO   FFFFF  FFFFF
R   R   U    U   N N   N   O    O  F      F
RRRRR   U    U   N  N  N   O    O  FFFF   FFFF
R       U    U   N   N N   O    O  F      F
R       U    U   N    NN   O    O  F      F
R    R   UUUU    N     N    OOOO   F      F
```

THIS OUTPUT CREATED BY PROGRAM "RUNOFFQ.EXE ", DATED 2/10/94

DSS FILE = travcool .DSS
SOILFILE = travcool .SOI

BCRDBCRD.OCS

AREA FILE = BCRD.ARE
PATHNAME = /BCRD/BCRD/PRECIP-INC//1HOUR/CS/

COOL SEASON CONDITIONS

INF.RATE BELOW WHICH THE SOILS FREEZE = 10.000000

TEMPERATURES USED FOR SNOWMELT 45.0 50.0 49.0 39.5 32.0 32.0 39.0 37.5 33.0 37.0

TOTAL AREA IN BASIN = 68699.000000

PERCENT FOREST =61.97%
PERCENT STORAGE = 2.18%

% BASIN	INF. RATE
2- .500	.00
14.3700	4.00
45.7500	13.00
10.9100	1.30
.1600	3.30
.4200	.40
2.7200	3.10
.2800	1.10

SUM OF PERCENTAGES = 99.76

EXHIBIT 9

Precipitation and Rainfall Excess
Hourly Increments

–Warm Season
–Cool Season

September 1994

EXHIBIT 10

Summer 1993 Historic Flood
Unit Hydrograph Verification

September 1994

V. Unit Hydrograph Verification

A. Flood of 1993

Flow Data

In the early summer of 1993, an extended period of rainfall—including several storms of very high intensity—produced severe flooding in the Upper Mississippi River Valley. Although the drainage was not in the most severely affected area, it experienced unusually high flows in June 1993, partially as a result of a condition of extreme watershed saturation due to previous rains. A 3-inch storm on June 21 produced a flood peaking at 16,500 at the only gaging station on River 1 about 30 miles downstream of the project. In addition, the flood hydrograph at the project was reconstructed from records of headwater, tailwater, generation, and spillway gate openings. The two hydrographs agreed very well, given the additional drainage area and lag time at the gage.

To focus on the portion of the model upstream of the project, the project hydrograph, rather than the gage hydrograph, was used to collectively verify the HEC-1 models for subbasins 2 and 3, plus the UNET routing model. No further verification on Subbasin 1 was attempted. Instead, recorded outflows from the reservoir for the period of interest were entered as an inflow hydrograph to the upstream end of the UNET model.

Rainfall Data

Rainfall data for June 1993 were obtained from the Wisconsin State Climatologist for another gage. In addition, daily rainfall totals for the same period were recorded at the project. These daily totals were considerably greater than the daily total at the gage, indicating that precipitation was not uniform over the drainage basin. The total rain depth at the gage for this period was 3 inches, while 4.3 inches were recorded at the project. Although this situation is less than optimal for unit hydrograph analysis, it was not considered sufficient justification for disregarding the 1993 event. The rainfall was partitioned over subbasins 2 and 3 in accordance with distance to each rain gage. Both subbasins were divided at a point midway between the two rain gage locations (which, for each basin, happened to coincide with the location of discontinued stream gages Nos. 99999999 and 00000000, respectively. The part of each subbasin closest to the project was designated *a* and the part closest to the project was designated *b*. Subbasins 2a and 3a were assumed to receive the hourly rainfalls recorded at the gage. Subbasins 2b and 3b were assumed to receive the daily totals recorded at the project, distributed by hour as recorded at the gage. (In a sensitivity analysis, runs were made using data from each rain gage exclusively.) This procedure follows directions in the FERC *Engineering Guidelines* to use hourly data from only one gage for a given basin, rather than weighted or averaged records.

Assumed Loss Rates for Verification Flood

Basin loss rates for the flood of 1993 were the same loss rates (based on STATSGO soils analysis) used to estimate the warm-season PMF.

Flood Routing

Due to the additional subdivision of subbasin 3 for the analysis of the 1993 flood, it was necessary to route flows from Subbasins 3a through the 13-mile reach of the south fork of River 1 below the subdivision point. (Subbasin 2 was also divided, but the division point was on the main stem of the River 1 and routing was accomplished in the UNET model.) The flows generated by subbasin No. 3a were routed in the HEC-1 model to the main stem of the river. The Muskingum routing method was used assuming a typical x value of 0.2 and a routing time of 4 hours (assuming average channel velocities of about 5 feet per second).

Unit Hydrograph Analysis for Verification Flood

Unit hydrograph parameters were calculated for subbasins 2a, 2b, 3a and 3b in four different ways. The first (considered the base case in this analysis) is consistent with all the parameters or methods determined to be best during development of the unit hydrograph. The second, third, and fourth varied these methods to evaluate possible adjustments to improve the fit of the verification hydrograph. The analysis of the 1993 flood should be seen as a test of *methods*, rather than specific unit hydrograph parameters, because the subbasins used in this analysis are smaller than those ultimately used to model the PMF. This approach was necessary to account for the nonuniformity of the recorded precipitation.

These methods are summarized in Table 10, as Trials B - E. Trial A is not included in this discussion, because it was a preliminary analysis that did not consider the analysis of hourly flows on subbasin 1 for the flood of 1974. The letter designations B - E were maintained for this report to be consistent with the letter designations in the documentation of the analyses.

TABLE 10
Unit Hydrograph Methods
Evaluated for 1993 Flood

Trial	Subbasin 2a	Subbasin 2b	Subbasin 3a	Subbasin 3b
B	Snyder parameters based on SEI (Reference 2) T_p = 46; C_p =.73	Snyder parameters based on SEI (Reference 2) T_p = 28; C_p = .73	Calibrated T_c and R from 1974 flood T_c = 70; R = 90	Snyder parameters based on SEI(Reference 2) T_p = 18; C_p = .73
C	Snyder parameters based on SEI (Reference 2) T_p = 46; C_p = .73	Snyder parameters based on SEI (Reference 2) T_p = 28; C_p = .73	Snyder parameters based on SEI (Reference 2) T_p = 39; C_p = .73	Snyder parameters based on SEI (Reference 2) T_p = 18; C_p = .73
D	Snyder parameters based on SEI (Reference 2) T_p = 46; C_p = .73	Snyder parameters based on SEI (Reference 2) T_p = 28; C_p = .73	River 2 Regional Equations T_c = 46; R = 73	Snyder parameters based on SEI (Reference 2) T_p = 18; C_p = .73
E	River 2 Regional Equations T_c = 52; R = 63	River 2 Regional Equations T_c = 28; R = 40	River 2 Regional Equations T_c = 46; R = 73	River 2 Regional Equations T_c = 21; R = 36

Results of Verification

Each set of t_c and R values or T_p and C_p values was used in the HEC-1 model to compute subbasin hydrographs for routing in the UNET model. The two trials that achieved the best fit to the observed outflow values are Trials B and D. Both trials yielded approximately the same peak flow (13,600 cfs), but Trial D matched the recession limb slightly better than Trial B. The subbasin 3 t_c used for Trial D, 46 hours, is similar in magnitude to the equivalent t_c obtained by Snyder's synthetic method (Trial C - t_c=45 hours) and the 1974 calibration value (Trial B - t_c=40 hours), lending confidence in a t_c of 46 hours.

Trial E was initiated after reviewing the results of Trial D, to check whether the River 2 regression equations might give the best fit for all subbasins. The fit produced by Trial E, however, was relatively poor, confirming the assumption stated above that Subbasin 2 is not sufficiently similar to the basins used in the River 2 studies.

Finally, as an additional check, the Trial D (River 2 regional equation) parameters for subbasin 3a were applied to the south fork flood of 1974, to ensure that the fit for this flood was essentially as good as the parameters originally calibrated for that event (those used in Trial B).

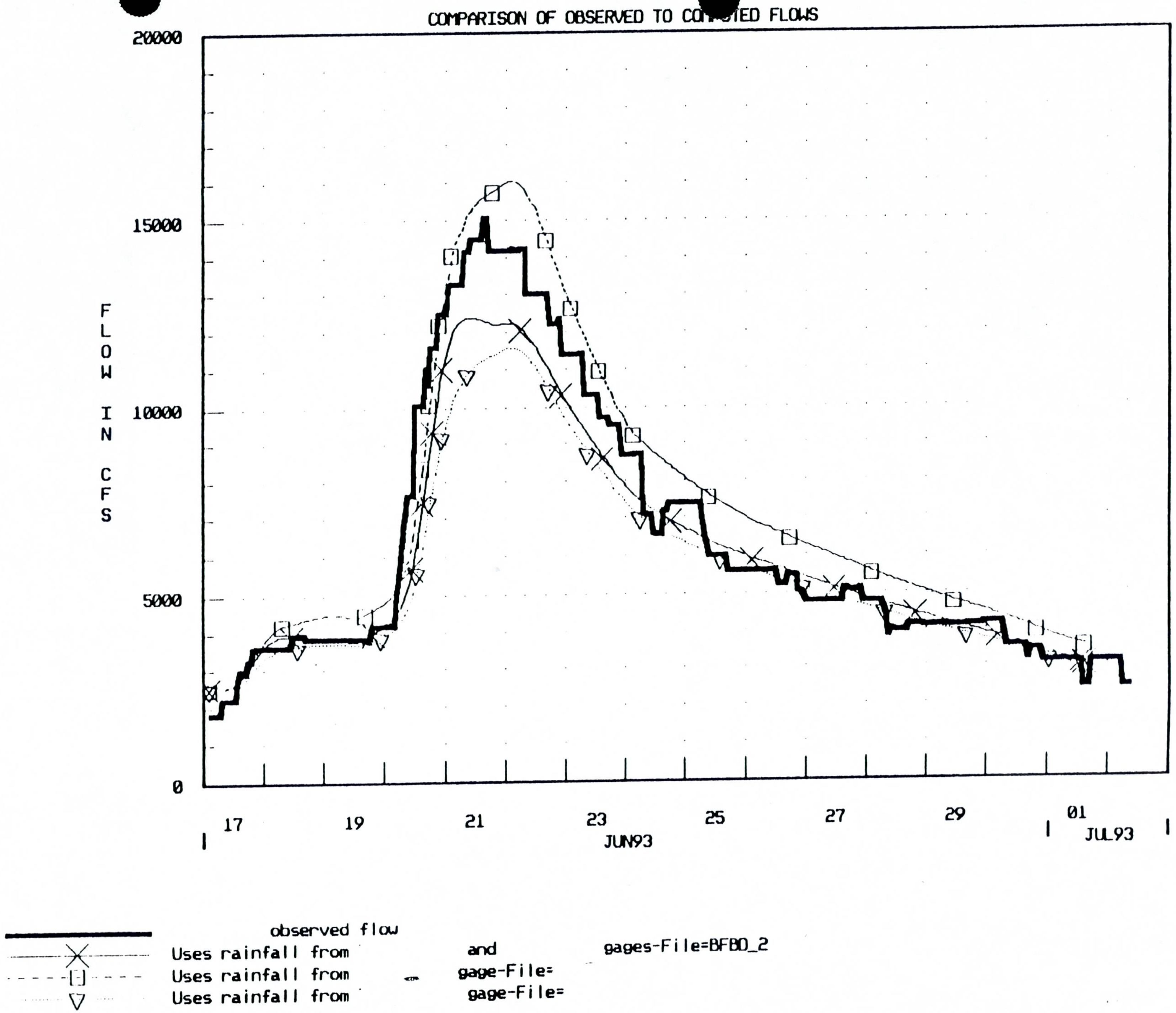
Warm Season

[illegible] 09:25:56 PAGE

TIME	DATE	BCRD WS PRECIP-INC INCHES INST-VAL	BCRD WS RUNOFF-INC INCHES INST-VAL
0100	01AUG1999	.043	.001
0200	01AUG1999	.043	.001
0300	01AUG1999	.043	.001
0400	01AUG1999	.043	.001
0500	01AUG1999	.043	.001
0600	01AUG1999	.043	.001
0700	01AUG1999	.053	.001
0800	01AUG1999	.053	.001
0900	01AUG1999	.053	.001
1000	01AUG1999	.053	.001
1100	01AUG1999	.053	.001
1200	01AUG1999	.053	.001
1300	01AUG1999	.067	.001
1400	01AUG1999	.067	.001
1500	01AUG1999	.067	.001
1600	01AUG1999	.067	.001
1700	01AUG1999	.067	.001
1800	01AUG1999	.067	.001
1900	01AUG1999	.093	.002
2000	01AUG1999	.093	.002
2100	01AUG1999	.093	.002
2200	01AUG1999	.093	.002
2300	01AUG1999	.093	.002
2400	[illegible]AUG1999	.093	.002
0100	02AUG1999	.133	.003
0200	02AUG1999	.138	.003
0300	02AUG1999	.145	.003
0400	02AUG1999	.153	.003
0500	02AUG1999	.161	.004
0600	02AUG1999	.171	.004
0700	02AUG1999	.283	.006
0800	02AUG1999	.306	.007
0900	02AUG1999	.341	.007
1000	02AUG1999	.390	.009
1100	02AUG1999	.450	.010
1200	02AUG1999	.524	.012
1300	02AUG1999	.798	.020
1400	02AUG1999	1.305	.036
1500	02AUG1999	2.017	.226
1600	02AUG1999	4.376	1.260
1700	02AUG1999	1.711	.144
1800	02AUG1999	1.142	.029
1900	02AUG1999	.262	.006
2000	02AUG1999	.238	.005
2100	02AUG1999	.218	.005
2200	02AUG1999	.202	.004
2300	02AUG1999	.189	.004
2400	02AUG1999	.180	.004
0100	03AUG1999	.115	.003
0200	03AUG1999	.115	.003

Cool Season

TIME DATE	BCRD CS PRECIP-INC INCHES INST-VAL	BCRD CS SNOW-INC INCHES INST-VAL	BCRD CS PCP_SNOW-INC INCHES INST-VAL	BCRD CS RUNOFF-INC INCHES INST-VAL
0100 01AUG1999	.027	.027	.054	.013
0200 01AUG1999	.027	.027	.054	.013
0300 01AUG1999	.027	.027	.054	.013
0400 01AUG1999	.027	.027	.054	.013
0500 01AUG1999	.027	.027	.054	.013
0600 01AUG1999	.027	.027	.054	.013
0700 01AUG1999	.033	.027	.060	.015
0800 01AUG1999	.033	.027	.060	.015
0900 01AUG1999	.033	.027	.060	.015
1000 01AUG1999	.033	.027	.060	.015
1100 01AUG1999	.033	.027	.060	.015
1200 01AUG1999	.033	.027	.060	.015
1300 01AUG1999	.042	.027	.069	.017
1400 01AUG1999	.042	.027	.069	.017
1500 01AUG1999	.042	.027	.069	.017
1600 01AUG1999	.042	.027	.069	.017
1700 01AUG1999	.042	.027	.069	.017
1800 01AUG1999	.042	.027	.069	.017
1900 01AUG1999	.057	.027	.084	.021
2000 01AUG1999	.057	.027	.084	.021
2100 01AUG1999	.057	.027	.084	.021
2200 01AUG1999	.057	.027	.084	.021
2300 01AUG1999	.057	.027	.084	.021
2400 01AUG1999	.057	.027	.084	.021
0100 02AUG1999	.081	.038	.119	.029
0200 02AUG1999	.084	.038	.122	.030
0300 02AUG1999	.088	.038	.126	.031
0400 02AUG1999	.092	.038	.130	.032
0500 02AUG1999	.098	.038	.136	.033
0600 02AUG1999	.104	.038	.142	.035
0700 02AUG1999	.170	.038	.208	.051
0800 02AUG1999	.185	.038	.223	.055
0900 02AUG1999	.205	.038	.243	.060
1000 02AUG1999	.230	.038	.268	.066
1100 02AUG1999	.261	.038	.299	.074
1200 02AUG1999	.297	.038	.335	.082
1300 02AUG1999	.416	.038	.454	.112
1400 02AUG1999	.632	.038	.670	.166
1500 02AUG1999	.923	.038	.961	.239
1600 02AUG1999	1.890	.038	1.928	.552
1700 02AUG1999	.797	.038	.835	.208
1800 02AUG1999	.562	.038	.600	.149
1900 02AUG1999	.157	.038	.195	.048
2000 02AUG1999	.143	.038	.181	.044
2100 02AUG1999	.131	.038	.169	.042
2200 02AUG1999	.122	.038	.160	.039
2300 02AUG1999	.114	.038	.152	.037
2400 02AUG1999	.109	.038	.147	.036
0100 03AUG1999	.070	.035	.105	.026
0200 03AUG1999	.070	.035	.105	.026



COMPARISON OF OBSERVED TO COMPUTED FLOWS
20000
15000
10000
5000
0
FLOW IN CFS
17
19
21
23
25
27
29
01
JUN93
JUL93
observed flow
Uses rainfall from and gages-File=BFBO_2
Uses rainfall from gage-File=
Uses rainfall from gage-File=

TIME OF CONCENTRATION

(T_c)

(SEC. 8-8.5 of FERC Guidelines on the Determination of Probable Maximum Flood)

Materials under TAB 14 in Volume 1 of the Notebook

This is part of the Guidelines related to the development of unit hydrograph for basins with adequate data - Sec 8.8 of the FERC Guidelines

DEFINITIONS

1) the time that it takes for runoff to travel from the hydraulically most remote part of the watershed to the basin outlet during a storm

2) the time elapsed between the end of rainfall excess to the point of inflection of the runoff hydrograph

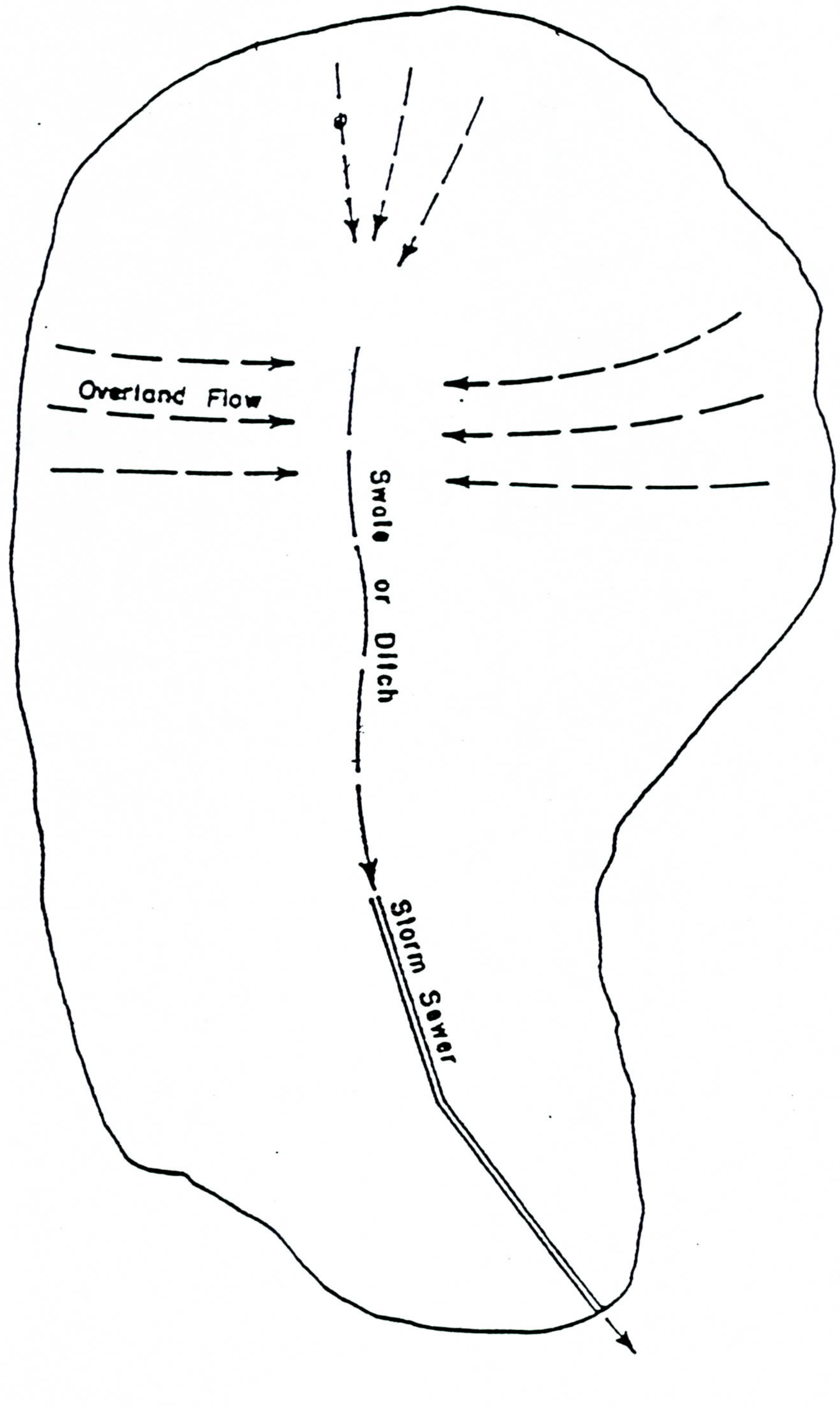
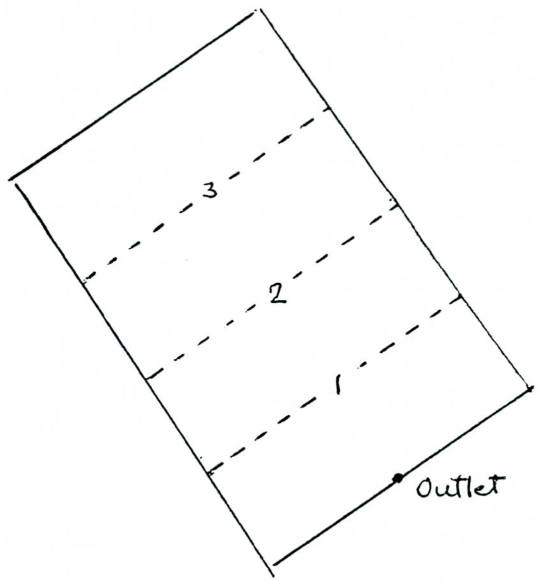
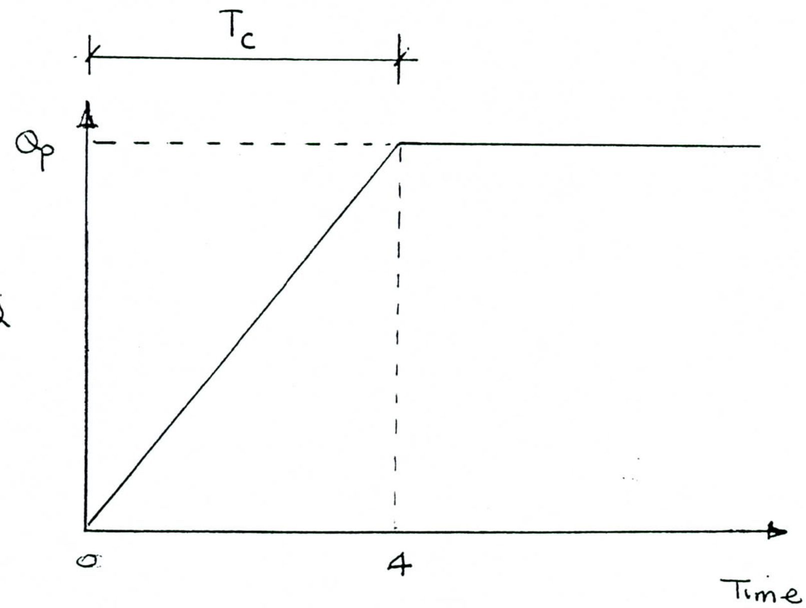


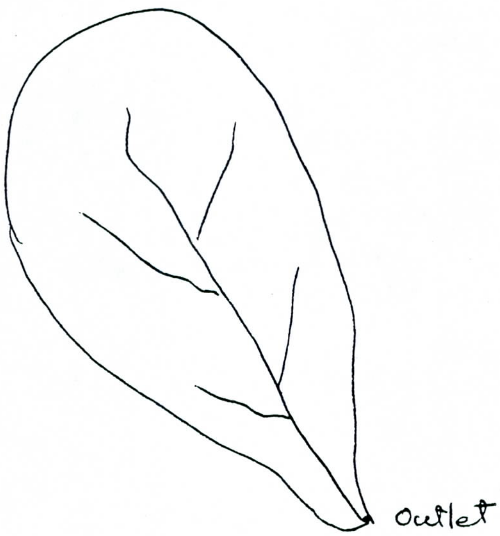
Figure 1. Small watershed with overland flow, swale flow, and storm sewer flow.

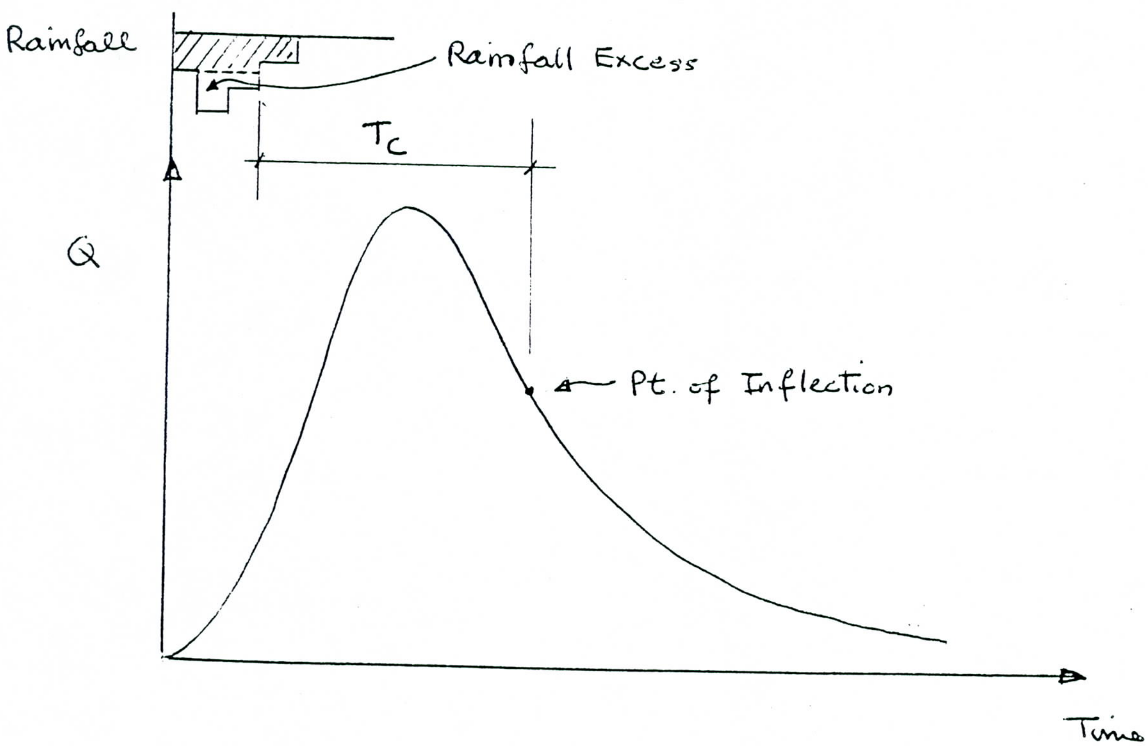
- Physical understanding of the T_c definition
  - Idealized condition - rectangular parking lot (impervious) with an uniform rainfall intensity for an indefinite period





- **Typical watershed - rainfall and corresponding flood hydrograph**

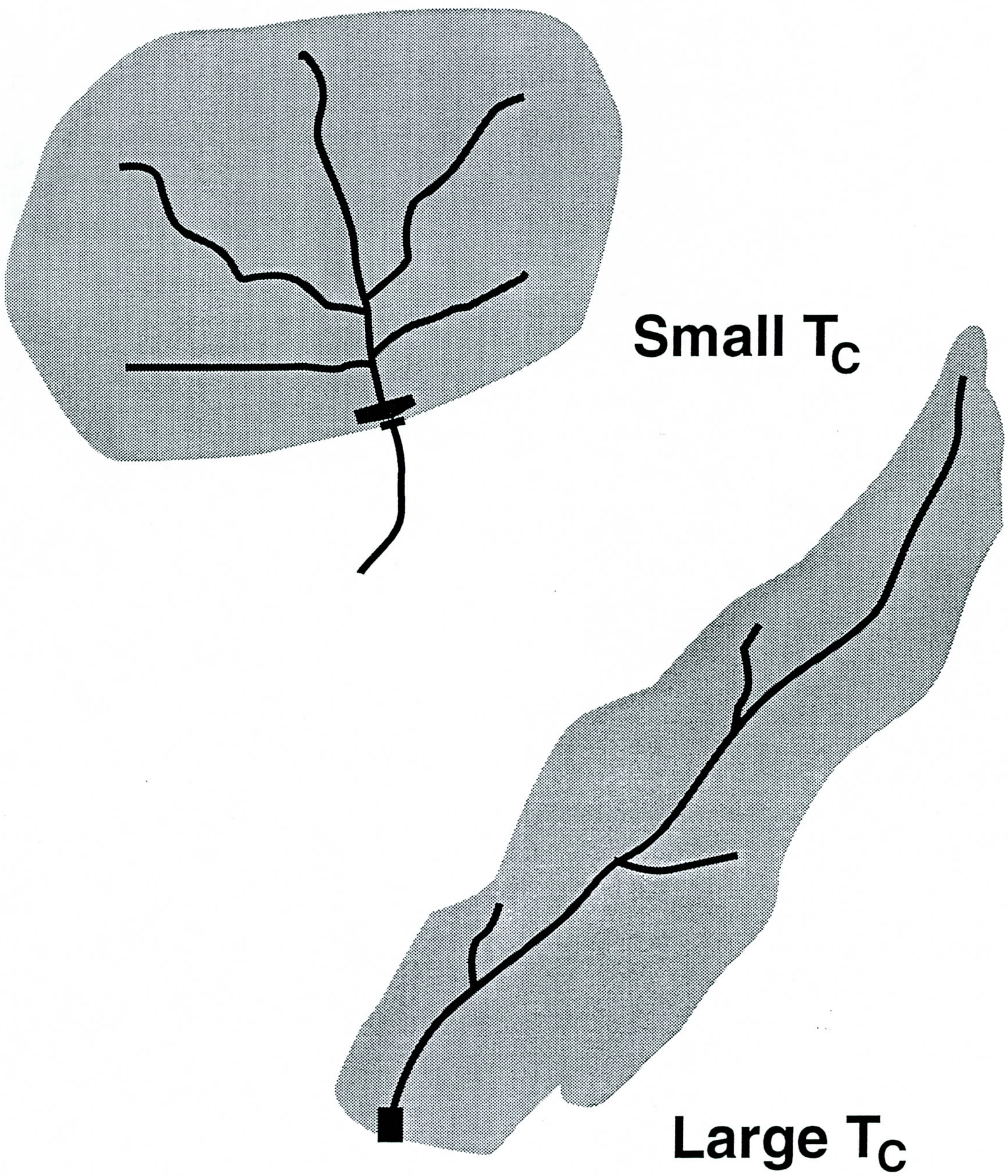




TIME OF CONCENTRATION (T_C)

- an extremely important parameter in estimating flood runoff from a watershed
- a measure of the response time of the watershed to rainfall
- the shorter the T_c, the quicker the response time of the watershed to rainfall and the higher the peak of the unit hydrograph
- varies with basin geometry, stream density, and basin soil and cover characteristics
- also varies with storm pattern. T_c will be longer for a storm moving up river than for one moving downstream
- T_c = 1.67 Lag time (SCS definition)

EFFECT OF BASIN CHARACTERISTICS ON T_C



METHODS TO COMPUTE T_C

- REGRESSION METHODS (EQUATIONS)
- HYDRAULIC CALCULATIONS
- HYDROGRAPH METHOD

REGRESSION METHODS

- Kirpich Equation (1940) - rural watersheds; channel flow condition only
- Izzard Equation (1946) - laboratory experiments; overland flow condition for larger areas like parking lots and bare land.
- Kinematic Wave Equation by Morgali (1965) - overland flow condition; developed areas
- Federal Aviation Agency Equation (1970) - overland flow condition
- Old SCS Lag Equation (1975) - agricultural watersheds
- New SCS Sheet Flow Equation (1986) - urban watersheds

Table 1. Summary of Time of Concentration Methods.

METHOD AND DATE	FORMULA FOR T_c (minutes)	REMARKS
Kirpich (1940)	$T_c = 0.0078 \left[\frac{L^2}{S}\right]^{0.385}$ L = length of channel/ditch from headwater to outlet, feet S = average gully slope, ft/ft.	Developed from SCS data for 7 rural basins in Tennessee with well-defined channel and steep slopes (3% to 10%). For overland flow on concrete or asphalt surfaces, multiply T_c by 0.4. For concrete channels, multiply by 0.2. No adjustment for overland flow on bare soil or flow in roadside ditches. Reference: Civil Engineering, Vol. 10, No. 6, June 1940.
Izzard (1946)	$T_c = \frac{41.025\ (0.0007\ i + c)\ L^{0.33}}{S^{0.333}\ i^{0.667}}$ i = rainfall intensity, in/hr c = retardance coef. L = length of flow path, feet S = slope of flow path, ft/ft	Developed in laboratory experiments by Bureau of Public Roads for overland flow on roadway and turf surface. Values of the retardance coef. range from 0.0070 for very smooth pavement; c = 0.012 for concrete pavement; c = 0.06 for dense turf. Solution is extremely tedious and requires iteration. Product i times 1 should be ≤ 500. Reference: Proc. Highway Research Board, Vol. 26, pp. 129-146, 1946.
Fed. Aviation Agency (1970)	$T_c = \frac{1.8(1.1 - C)L^{0.50}}{S^{0.333}}$ c = rational method runoff coeff. L = length of overland flow, ft S = average overland slope, %	Developed from air field drainage data assembled by the Corps of Engineers. Method is intended for use on airfield drainage problems, but has been used frequently for overland flow in urban basins. Reference: "Airport Drainage," Federal Aviation Agency, Dept. of Transportation Advisory Circular, A/C 150-5320-5B, Washington, D.C., 1970.

L

METHOD AND DATE	FORMULA FOR T_c (minutes)	REMARKS
Kinematic Wave Formulas Morgali (1965)	$T_c = \dfrac{0.94\, L^{0.6}\, n^{0.6}}{i^{0.4}\, S^{0.3}}$ L = length of overland flow, feet n = Manning roughness coef. i = rainfall intensity, in/hr S = average overland slope, ft/ft	Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces. method requires iteration since both i (rainfall intensity) and T_c are unknown. Reference: Morgali and R. K. Linsley, "Computer Simulation of Overland Flow," J. Hyd. Div. ASCE, Vol. 91, No. 81, May 1965.
Old SCS Lag Equation (1975)	$T_L = \dfrac{L^{0.8}\left[\dfrac{1000}{CN} - 9\right]^{0.7}}{190\, S^{0.5}}$ L = hydraulic length of watershed (longest flow path), feet CN = SCS runoff curve number S = average watershed slope, ft/ft	Equation developed by SCS from agricultural watershed data. It has been adapted to small urban basins under 2000 acres. Found generally good where area is completely paved. For mixed areas, it tends to overestimate. Adjustment factors are applied to correct for channel improvement and impervious area. The equation assumes that T_c = 1.67 x basin lag. Reference: Soil Conservation Service Tech. Release No. 55, U.S. Department of Agriculture, January 1975.
New SCS Sheet Flow Equation	$T_c = \dfrac{0.42\,(nL)^{0.8}}{P_2^{0.5}\, S^{0.4}}$ L = overland flow length, ft n = Manning n for sheet flow P_2 = 2 yr. - 24 hr. rainfall, inches S = overland slope, ft/ft	Equation developed by SCS for Revised TR 55, June 1986.

These equations all have two common parameters:

- a "length" term - the length of the flow path
- a "slope" term - the <u>average</u> slope of the flow path
- there are many approaches used in estimating the "average" slope of the flow path
  - Weighted average approach (Taylor and Schwarz (1952, Trans AGU Vol 33, No.2)
  - USGS 10/85 approach
  - Equivalent area approach

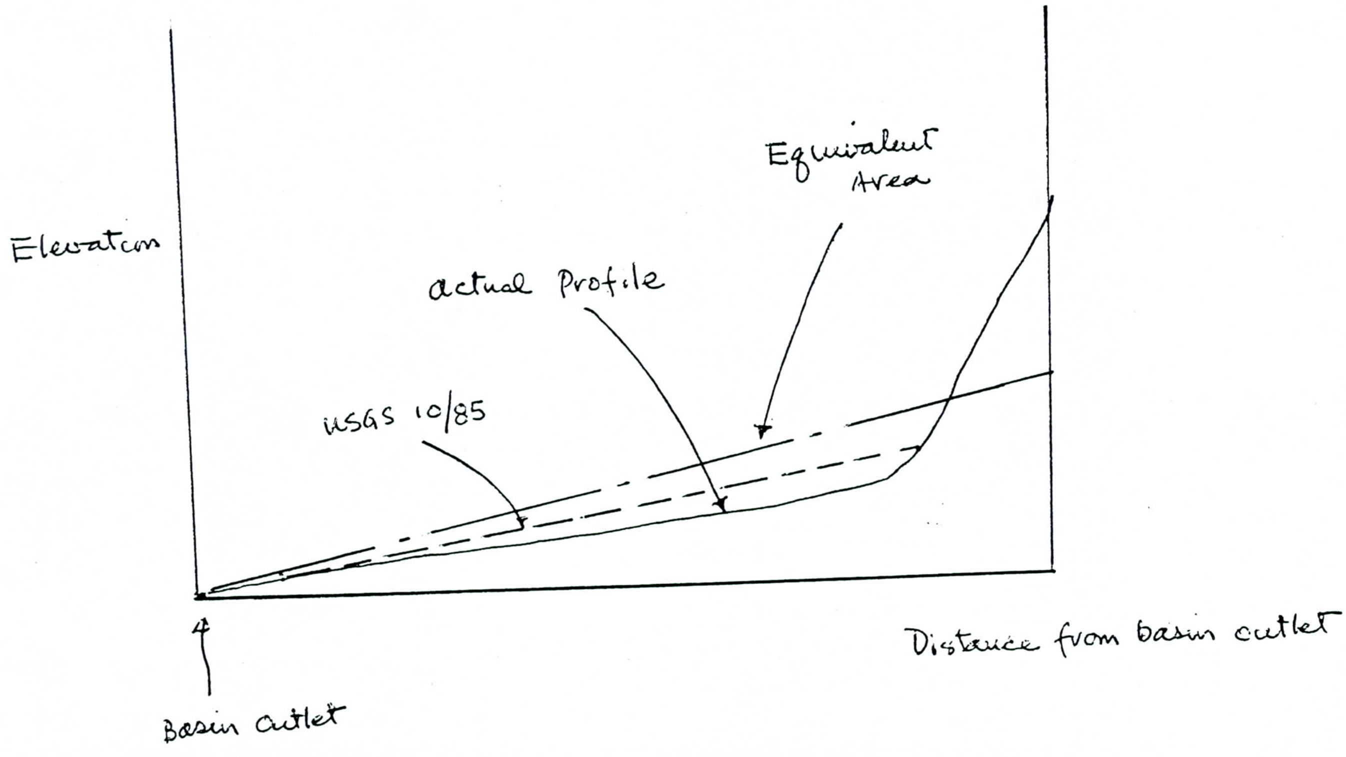


Table 2. Example T_c Calculations for Subarea F in the Calder Alley Watershed.

METHOD	T(grass), min	T(gutter), min	T(pipe), min	T_c(total), min	T_c(comp)[d], min	Design i_b, in./hr	Q_{25}[e], cfs
Kirpich	4.0	1.0	3.2	8.2	9.6	5.3	117
SCS lag	---	---	---	---	12.4	4.9	108
SCS velocity charts	2.8	1.4	2.9[c]	7.1	12.0	5.0	110
Federal Aviation Agency	18.0	3.2	2.9[c]	24.1	36.0	2.8	62
Izzard[a,b]	---	---	---	---	11.1	5.1	113
Kinematic[b] wave	18.0	3.2	2.9[c]	24.1	18.0	4.0	88

Given (1) Subarea F in Calder Alley drainage system; (2) single family residential, area = 52.6 acres, 20% impervious; (3) longest flow distance to nearest major inlet = 1800 ft (200 ft grass, 300 ft gutter, 1300 ft 12-in. storm drain; (4) overland slope and pipe slope = 3%; (5) average runoff coefficient = 0.42, SCS CN = 80; (6) Manning n: grass = 0.20, gutter = pipe = 0.014; and (7) 1-hour, 25-year rainfall (1^P25 = 2.06 in. Find T_c to major inlet under 25-year storm conditions.

[a]Izzard (1946) calculations are iterative and too tedious to be used in design analysis.

[b]Penna. intensity-duration-frequency curves are those developed by NWS; these are applied to 1^P25 to get design i as well as T_c in Izzard and kinematic wage methods; design i is for T_c (comp).

[c]Manning equation under pipe-full flow.

[d]Composite T_c computed for length = 1800 ft using weighted average retardance-roughness coefficients.

[e]Q_{25} = c i A = (0.42) (52.6) i.

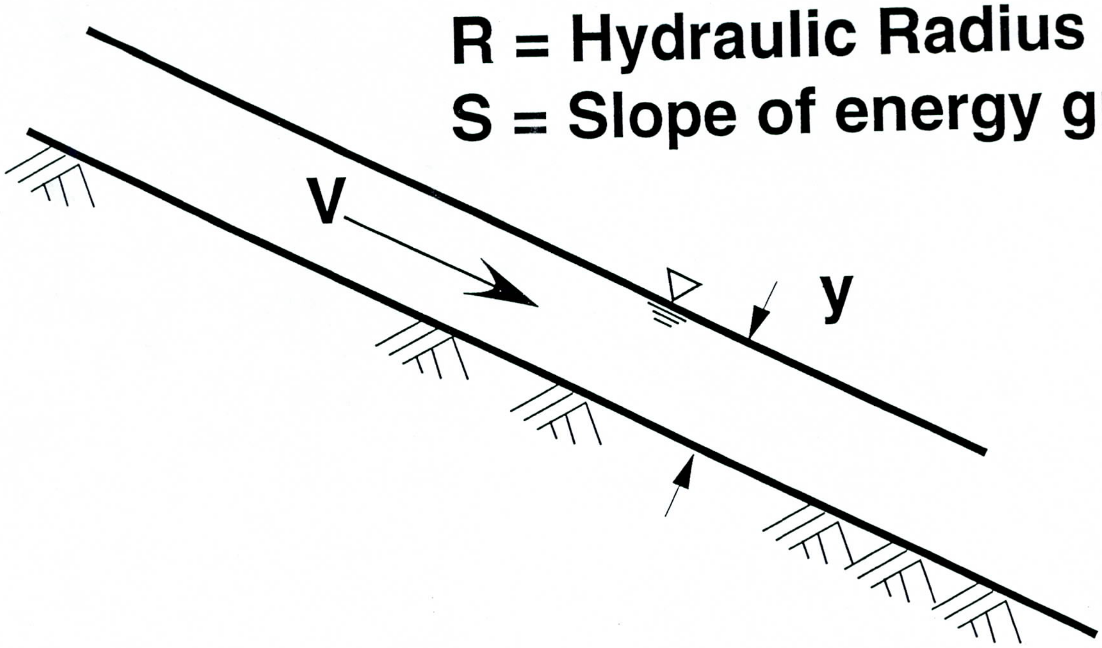
HYDRAULIC CALCULATIONS

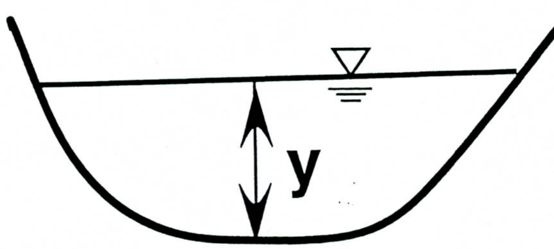
- Manning Equation
- SCS Average Velocity Charts in SCS National Engineering Handbook, Chapter 4 and SCS TR-55
- Segmental Approach

MANNING EQUATION FLOW IN CHANNELS AND CONDUITS

$$V = \frac{1.5}{n} R^{2/3} S^{1/2}$$

R = Hydraulic Radius
S = Slope of energy gradient





$$R = \frac{A}{P}$$

For "Overland Flow" Only

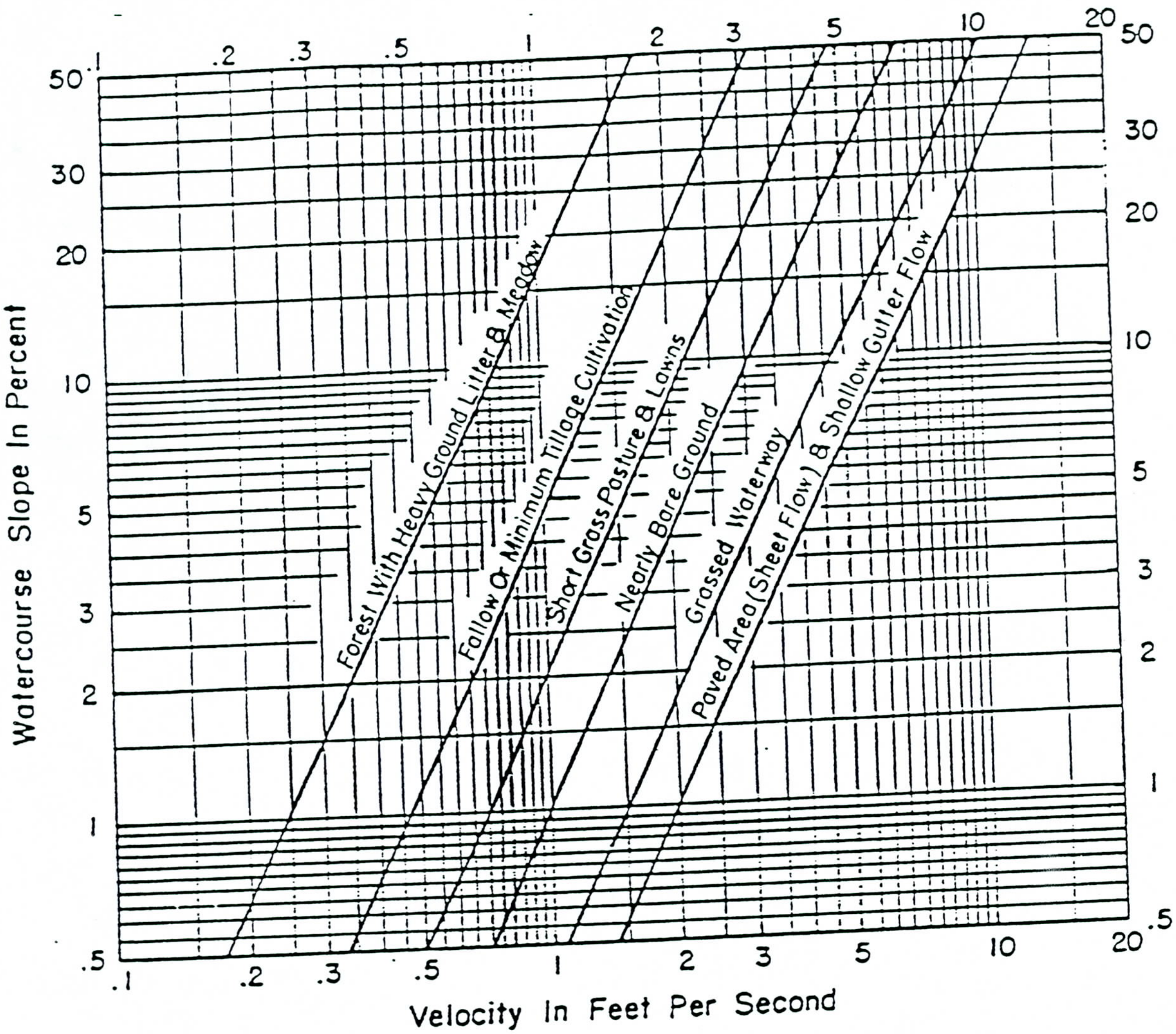
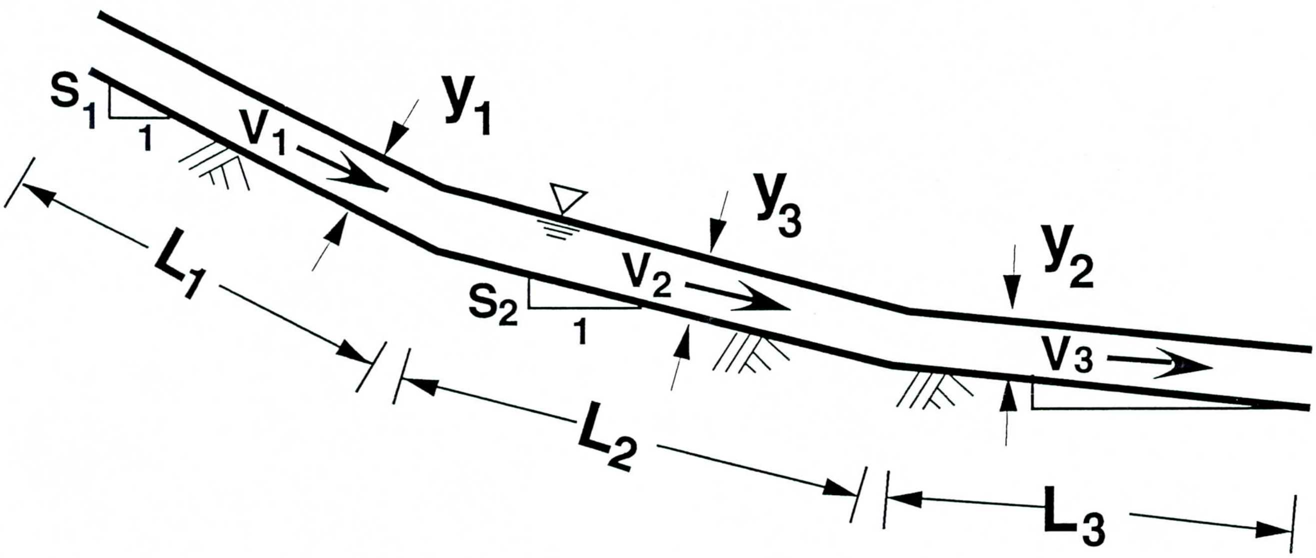


Figure 2. Average velocities for estimating travel time for overland flow. [After SCS TR-55, 1975].

from Soil Cons. Ser.

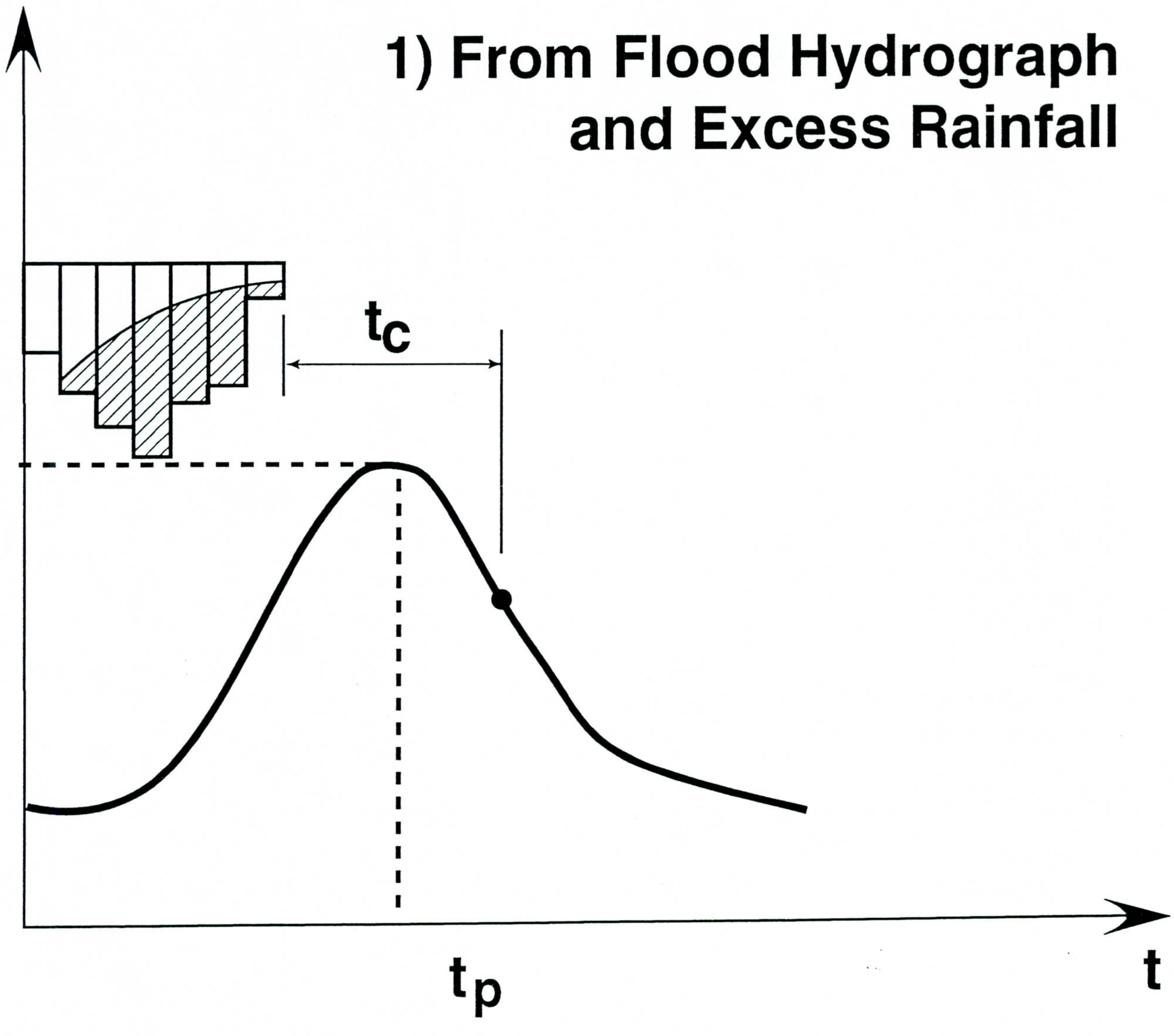
ESTIMATING TIME OF CONCENTRATION

2) Segmental Approach



$$\text{Time of Travel} = \frac{L_1}{V_1} + \frac{L_2}{V_2} + \frac{L_3}{V_3}$$

ESTIMATING TIME OF CONCENTRATION



HYDROGRAPH METHOD

- From observed hyetographs and corresponding flood hydrographs in the watershed of interest

- T_c is defined as the time elapsed between the end of the rainfall excess and the point of inflection in the corresponding flood hydrograph

  (the point of inflection is defined as the time in the flood hydrograph when the contribution from surface runoff has ceased and observed flows are primarily of sub-surface flow origin.)

- the determination is fairly subjective. It involves the determinations of the rainfall excess and the point of inflection in the flood hydrograph

- the point of inflection in the flood hydrograph can be estimated by plotting the recession limb of the hydrograph, Q versus T, on the semi ~~log~~-log paper. The break in the curve would be the point of inflection

- It most likely varies from storm to storm due to different storm and runoff characteristics

- one would have to select the value most appropriate for its application

REMARKS

- Hydrograph Method is the preferred approach for estimating T_c

- This is the way how T_c is defined in unit hydrograph applications

- Regression Method must be used with caution. The origins and application limitations of these equations should be clearly understood. Often, the units and definition of the various parameters in the equations vary from one to the other

- The use of hydraulic calculations appears to be well founded. However, it may not be necessarily consistent with the way T_c is defined in unit hydrograph applications

entire ws is contributing the run-off

TIME OF CONCENTRATION

Monday 1:45 p.m.

TRAVEL TIME ESTIMATIONS, T_c

Presently there are numerous methods available for estimating T_c. See Tables 1 and 2.

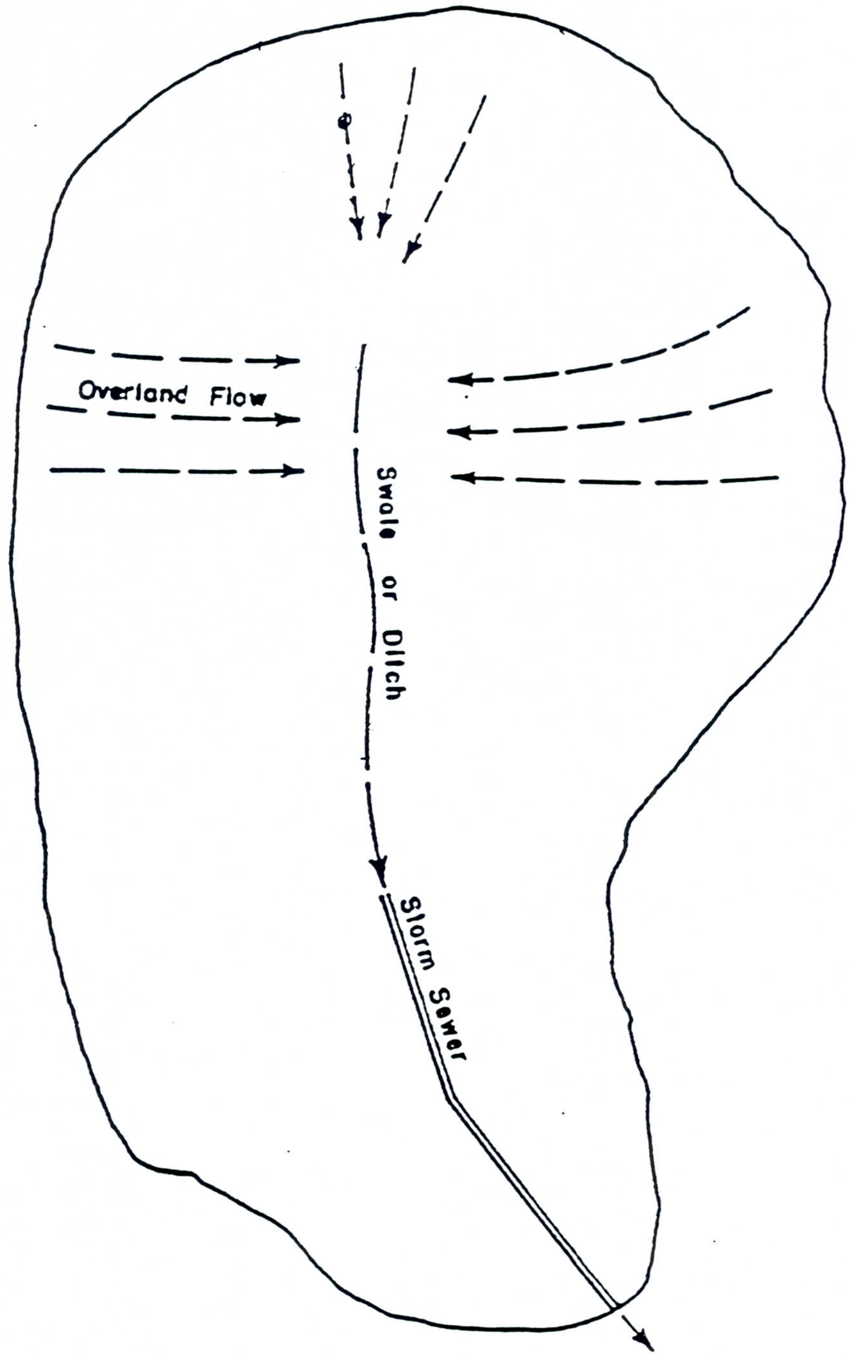
1. Segmental Approach (see Fig. 1)

2. Kinematic Wave Equation

3. SCS Average Velocity Chart in TR-55 (see Fig. 2)

4. SCS Lag Equation TR-55

5. Manning Equation for Flow Velocity in Channels/Pipes



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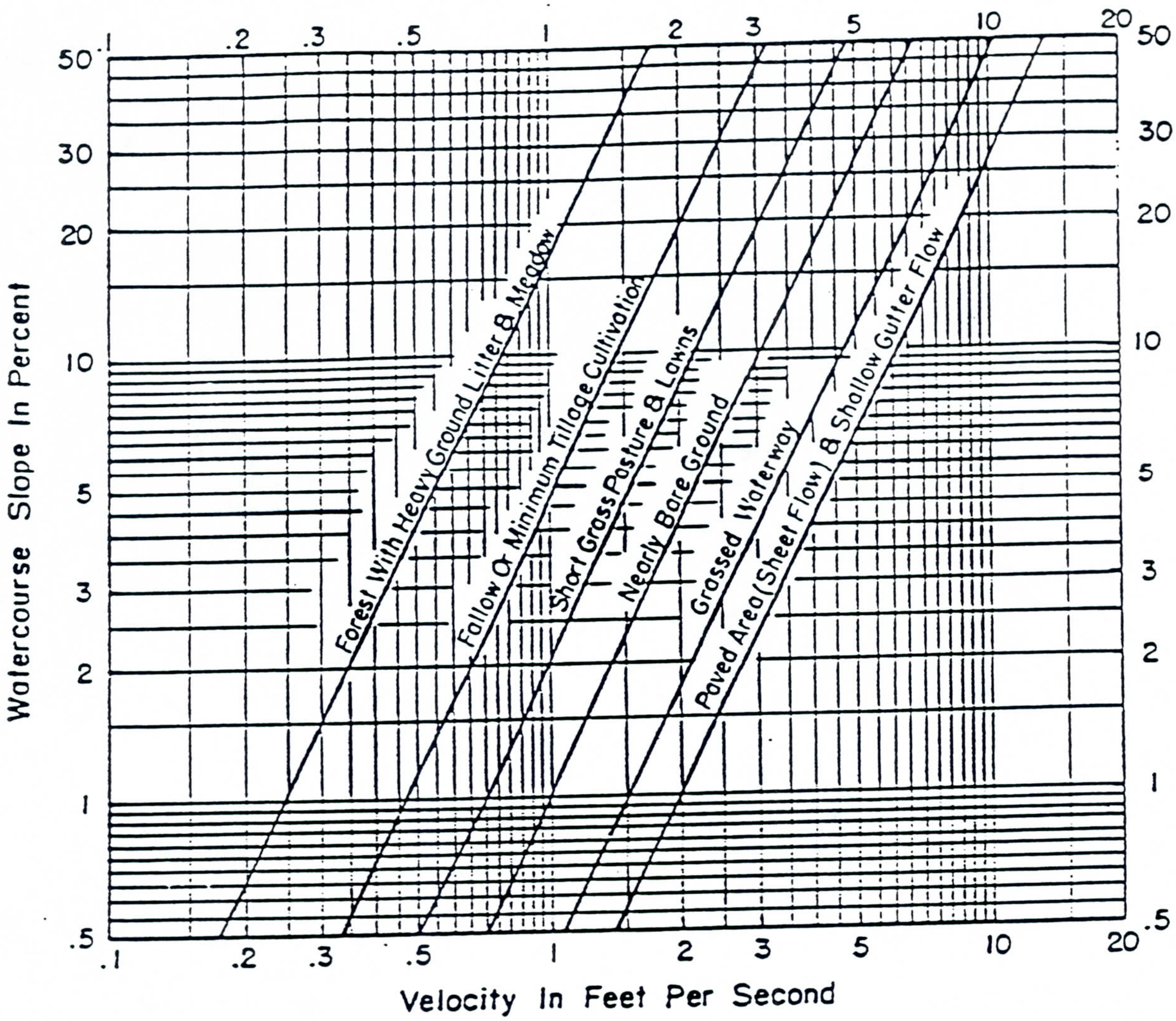

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References

1. American Geophysical Union, Water Resources Monograph No. 7, "Urban Stormwater Hydrology," edited by D. F. Kibler, 271 pp., 1982.

2. ASCE/EFC Proceedings, "Stormwater Detention Facilities," edited by W. DeGroot, Henniker, NH, 1982.

3. Espey, W. H. and D. G. Altman, "Nomographs for Ten-Minute Unit Hydrograph for Small Urban Watersheds," Addendum 3 of Urban Runoff Control Planning, Report EPA-6009-78-035, Environmental Protection Agency, Washington, D.C., 1978.

4. Kibler, D. F., G. Aron, G. Osei-Kwado, K. A. Riley, and E. L. White, "Recommended Hydrologic Procedures for Computing Urban Runoff from Small Watersheds in Pennsylvania," Research Completion Report by Institute for Research on Land and Water Resources, Penn State University, 1982.

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8. U.S. Department of Agriculture, "Soil Conservation Service," Urban Hydrology for Small Watersheds," Tech. Release 55, Washington, D.C., 1975.

9. U.S. Department of Agriculture, Soil Conservation Service, "Hydraulic Design of Two-Stage Risers by Shortcut Method," Tech. Note Engrg., No. 28, Broomall, PA, 1982, 13 pp.

10. U.S. Environmental Protection Agency, "Area-wide Assessment Procedures Manual," Vol. 1, Report EPA-600/9-76-004, Washington, D.C., 1976.

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Kinematic Wave Formulas Morgali (1965)	$T_c = \frac{0.94\ L^{0.6}\ n^{0.6}}{i^{0.4}\ S^{0.3}}$ L = length of overland flow, feet n = Manning roughness coef. i = rainfall intensity, in/hr S = average overland slope, ft/ft	Overland flow equation developed from kinematic wave analysis of surface runoff from developed surfaces. method requires iteration since both i (rainfall intensity) and T_c are unknown. Reference: Morgali and R. K. Linsley, "Computer Simulation of Overland Flow," J. Hyd. Div. ASCE, Vol. 91, No. 81, May 1965.
Old SCS Lag Equation (1975)	$T_c = \frac{L^{0.8}\left[\frac{1000}{CN} - 9\right]^{0.7}}{190\ S^{0.5}}$ L = hydraulic length of watershed (longest flow path), feet CN = SCS runoff curve number S = average watershed slope, ft/ft	Equation developed by SCS from agricultural watershed data. It has been adapted to small urban basins under 2000 acres. Found generally good where area is completely paved. For mixed areas, it tends to overestimate. Adjustment factors are applied to correct for channel improvement and impervious area. The equation assumes that T_c = 1.67 x basin lag. Reference: Soil Conservation Service Tech. Release No. 55, U.S. Department of Agriculture, January 1975.
New SCS Sheet Flow Equation	$T_c = \frac{0.42\ (nL)^{0.8}}{P_2^{0.5}\ S^{0.4}}$ L = overland flow length, ft n = Manning n for sheet flow P_2 = 2 yr. - 24 hr. rainfall, inches S = overland slope, ft/ft	Equation developed by SCS for Revised TR 55, June 1986.

Table 2. Example T_c Calculations for Subarea F in the Calder Alley Watershed.

METHOD	T(grass), min	T(gutter), min	T(pipe), min	T_c(total), min	T_c(comp)[d], min	Design i_b, in./hr	Q_{25}[e], cfs
Kirpich	4.0	1.0	3.2	8.2	9.6	5.3	117
SCS lag ·	---	---	---	---	12.4	4.9	108
SCS velocity charts	2.8	1.4	2.9[c]	7.1	12.0	5.0	110
Federal Aviation Agency	18.0	3.2	2.9[c]	24.1	36.0	2.8	62
Izzard[a,b]	---	---	---	---	11.1	5.1	113
Kinematic[b] wave	18.0	3.2	2.9[c]	24.1	18.0	4.0	88

Given (1) Subarea F in Calder Alley drainage system; (2) single family residential, area = 52.6 acres, 20% impervious; (3) longest flow distance to nearest major inlet = 1800 ft (200 ft grass, 300 ft gutter, 1300 ft 12-in. storm drain; (4) overland slope and pipe slope = 3%; (5) average runoff coefficient = 0.42, SCS CN = 80; (6) Manning n: grass = 0.20, gutter = pipe = 0.014; and (7) 1-hour, 25-year rainfall ($1^{P}25$ = 2.06 in. Find T_c to major inlet under 25-year storm conditions.

[a]Izzard (1946) calculations are iterative and too tedious to be used in design analysis.

[b]Penna. intensity-duration-frequency curves are those developed by NWS; these are applied to $1^{P}25$ to get design i as well as T_c in Izzard and kinematic wage methods; design i is for T_c (comp).

[c]Manning equation under pipe-full flow.

[d]Composite T_c computed for length = 1800 ft using weighted average retardance-roughness coefficients.

[e]Q_{25} = c i A = (0.42) (52.6) i.

CLARK UNIT HYDROGRAPH

(SEC. 8-8.5 of FERC Guidelines on the Determination of Probable Maximum Flood)

Materials under TAB 15 in Volume 1 of the Notebook

This is part of the Guidelines related to the development of unit hydrograph for basins with adequate data - Sec 8.8 of the FERC Guidelines

CLARK UNIT HYDROGRAPH

CONCEPT

- Clark Unit Hydrograph is different from other unit hydrographs
- It is an Instantaneous Unit Hydrograph (IUH)
- IUH is an Unit Hydrograph when the unit rainfall excess is instantaneously applied to the watershed of interest
- Clark's IUH is developed by routing the Time-Discharge Histogram of the watershed of interest (Translation Hydrograph) through a hypothetical linear reservoir
- The Translation Hydrograph can be developed from the Travel Time-Basin Area relationship for the watershed of interest
- The reservoir routing is performed using the equation

  $$O_i = C\, I_i + (1-C)\, O_{i-1}$$

  where C is defined as $[2\Delta t/(2R+\Delta t)]$ and R is the storage coefficient of the watershed of interest

- The basin storage coefficient R is defined as:

  $$R = -Q / (dQ/dt) \text{ at the point of inflection}$$

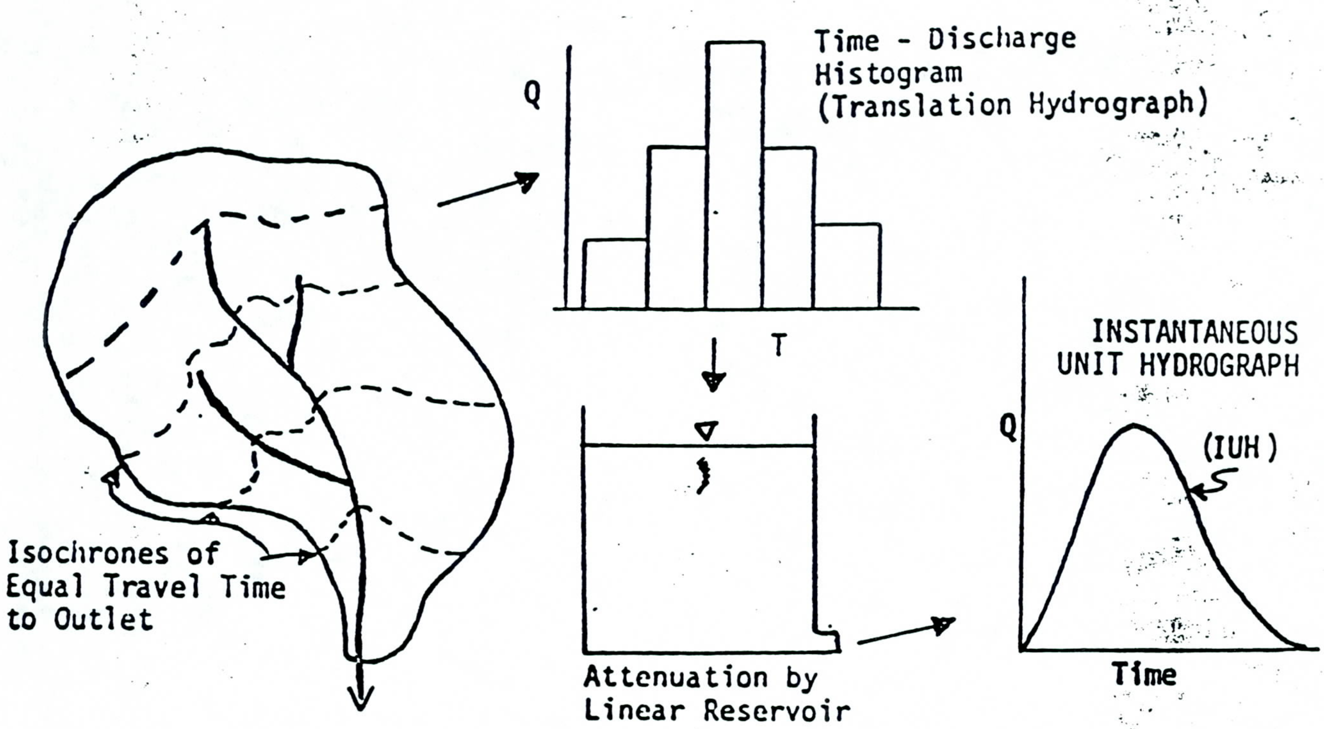


Figure 1. Clark Conceptual Model

- there are three elements in the Clark Unit Hydrograph:

  - Time of concentration "T_c"

  - Translation Hydrograph

  - basin storage coefficient "R" $k = \frac{R}{t_c + R}$ almost const for homogenous w.s.

- the most tedious part of the Clark Unit Hydrograph is the development of the translation hydrograph - very time consuming

- for many basins, the Clark Unit Hydrograph is not very sensitive to the shape of the translation hydrograph due to the storage effect of the hypothetical linear reservoir, except for those basins not having a substantial amount of natural storage, such as a steep urban basin

- a standard basin shape, such as an ellipse, can generally be used to develop translation hydrograph for use in Clark Unit Hydrograph development

SYNTHETIC TIME-AREA CURVE USED IN THE DEVELOPMENT OF CLARK UNIT HYDROGRAPH

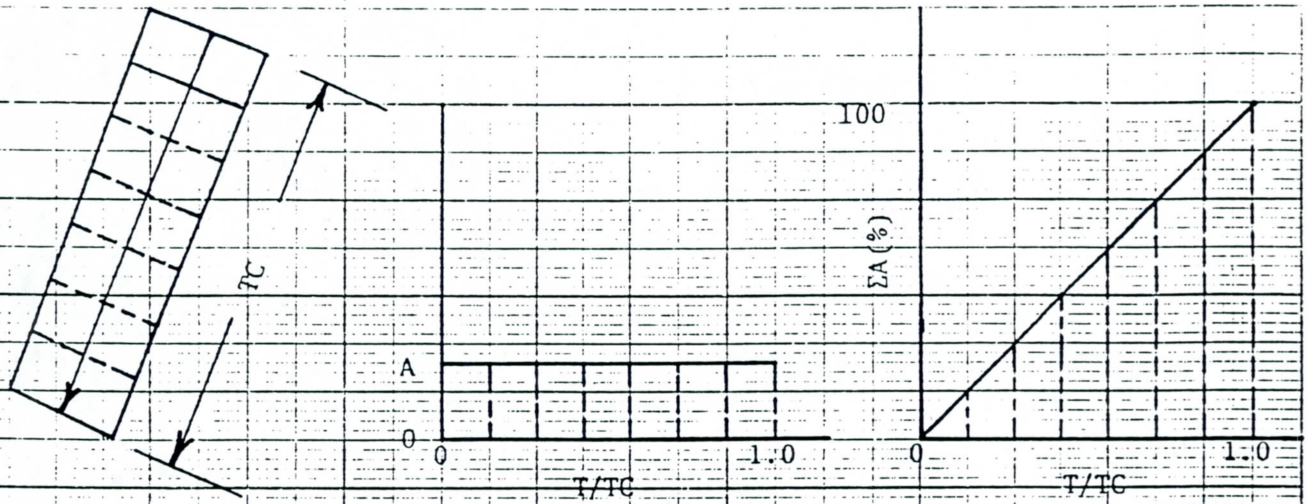


FIG. 3 RECTANGULAR SHAPE WATERSHED, n=1

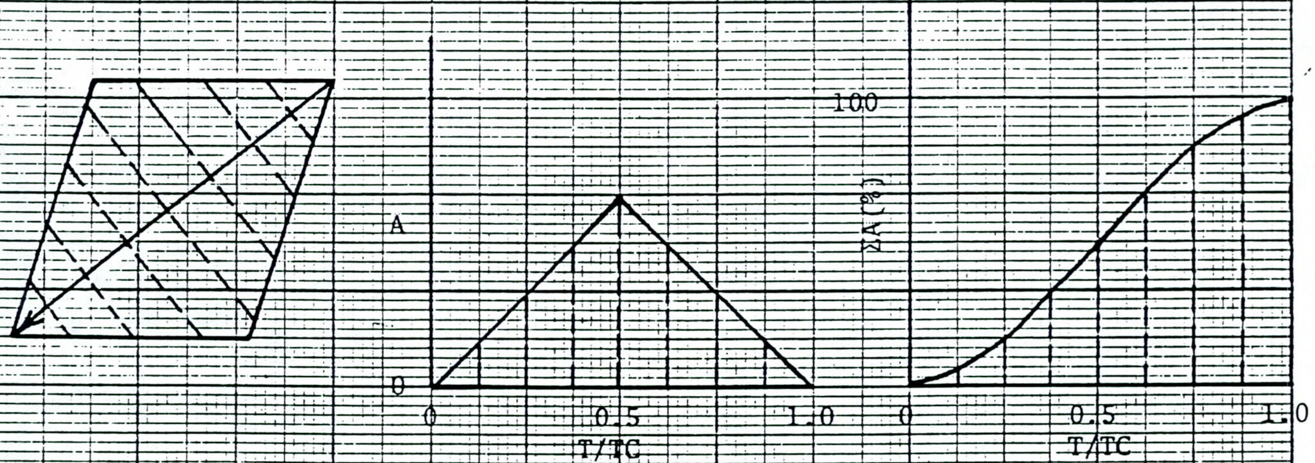
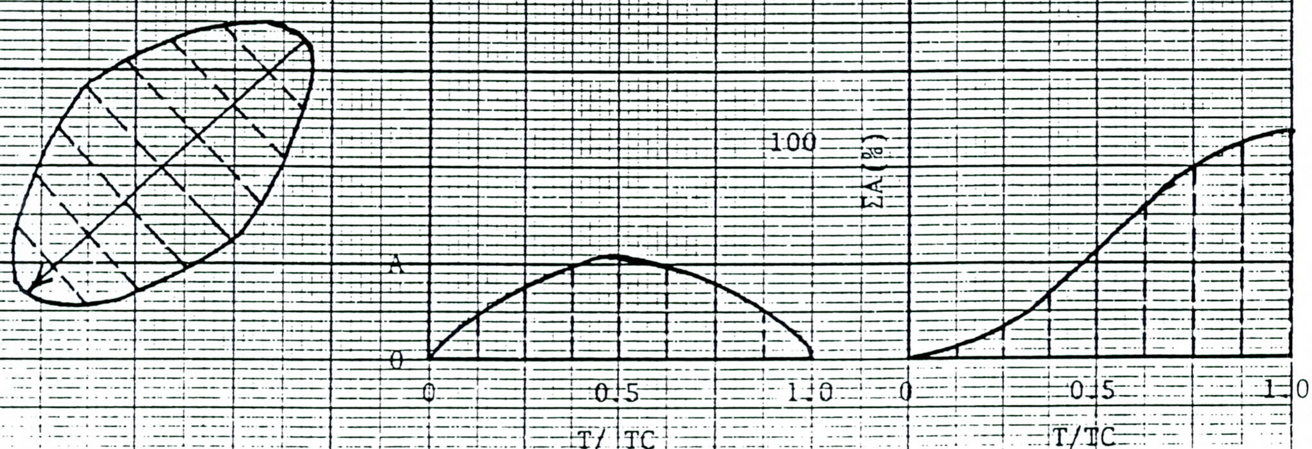


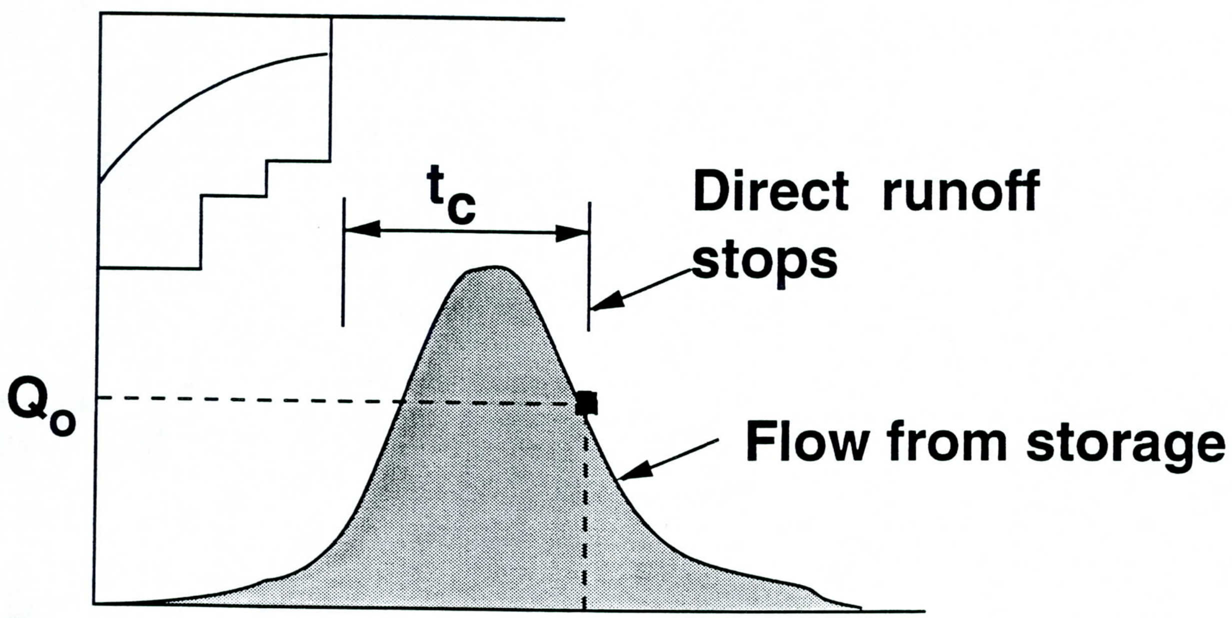
FIG. 4 DIAMOND SHAPE WATERSHED, n=2



HEC-I Default W.S. Shape

FIG. 5 GENERALIZED SHAPE WATERSHED, n=1.5

TO DERIVE THE CLARK'S T_C AND R COEFFICIENTS



neg.

$$R = -\frac{Q_o}{\Delta Q/\Delta t}$$

- Choose Δt properly, in general

$$\Delta t \leq \frac{T_c}{5}$$

CLARK UNIT GRAPH

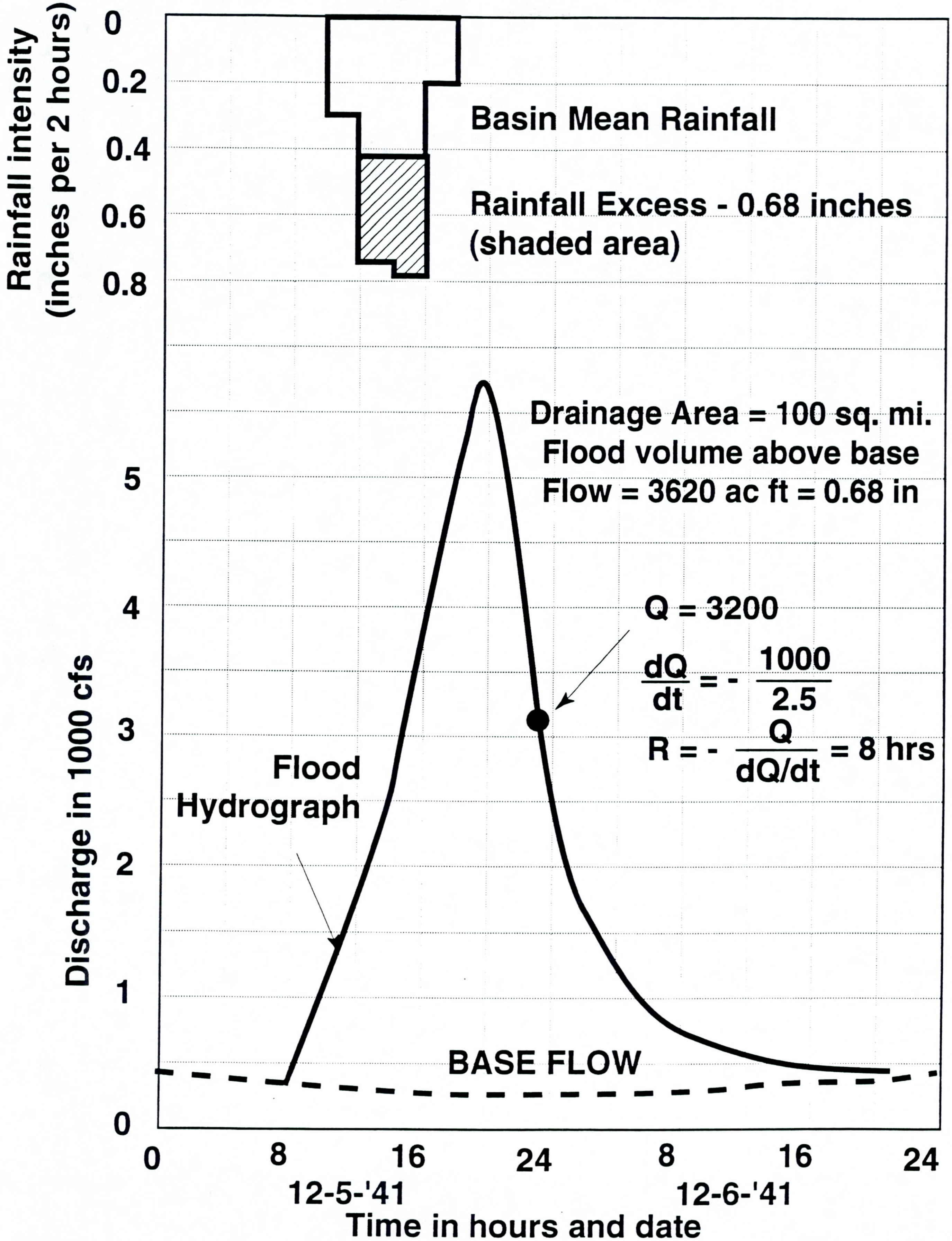
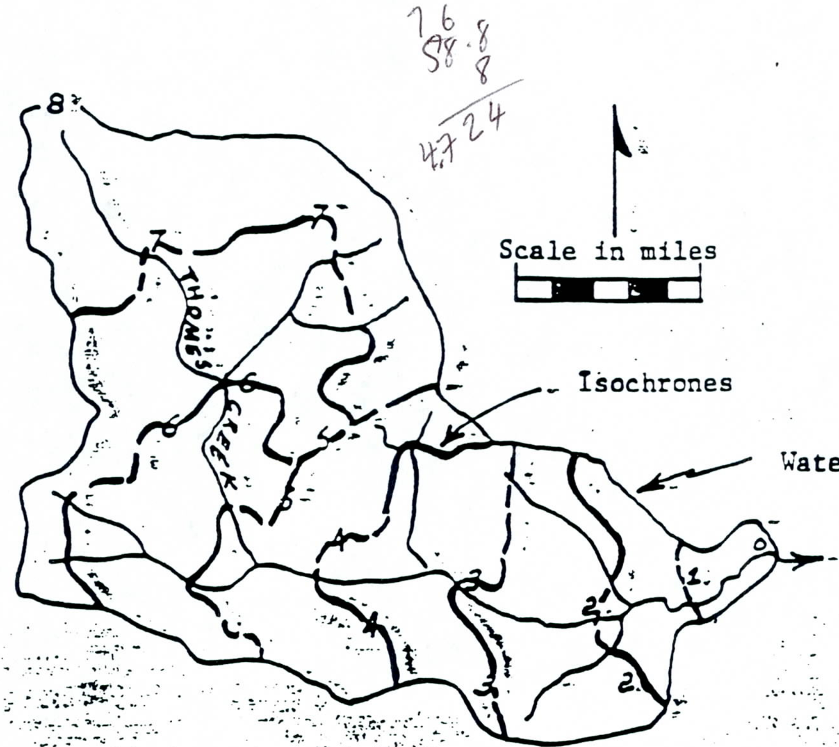


Figure 2. Computation of the Time-Area Relation



Travel Time from "8" to Gage is 8.0 Hours for the 32 Miles

Map Area Number (1)	Planimeter Values from Map Incremental units (2)	Accumulated units (3)	Accumulated area (sq. mi.) (Col 3) · (58.8) (4)	Travel Time in Percent [(1/8)·(100)] (5)
1	0.08	0.08	5	1/8 12.5
2	0.15	0.23	14	2/8 25.0
3	0.40	0.63	37	37.5
4	0.36	0.99	58	50.0
5	0.45	1.44	85	62.5
6	0.45	1.89	111	75.0
7	0.66	2.55	150	87.5
8	0.68	3.23	190	100.0
Total	3.23			

Sq. mi./Planimeter unit = 190/3.23 = 58.8

Drainage Area = 190 square miles

Figure 3. Watershed Time-Area Relation

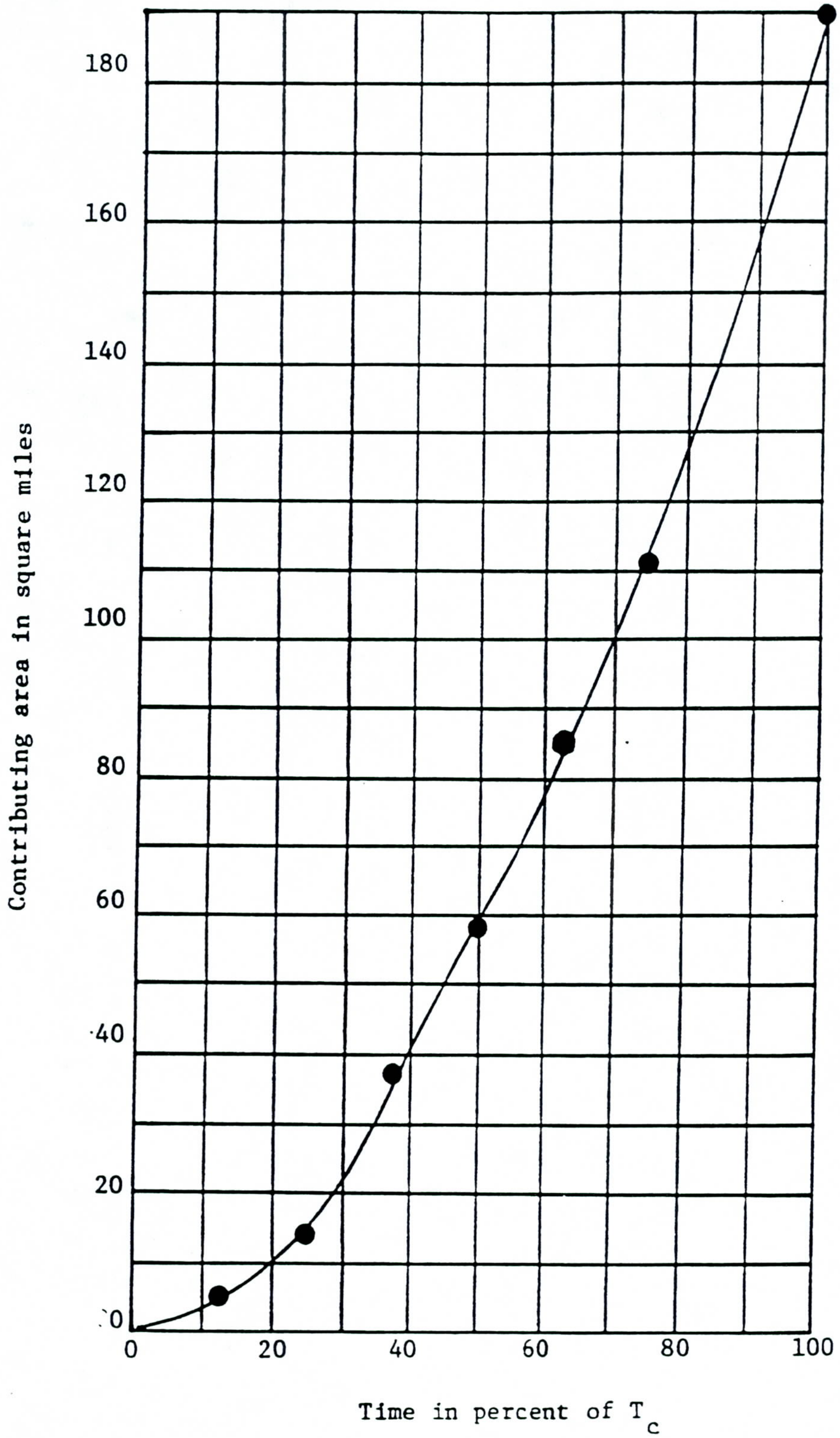


Table 1. Unit Graph Computation - Clark Method

(Thomas Creek at Paskenta, California)

Drainage Area = 190 square miles
Time of Concentration (T_c) = 8.0 hours (see Figure 1)
Attenuation Value (R) = 5.5 hours (see Figure 1)
Time Interval (Δt) = 2.0 hours

measured from run-off hydrograph

Equations (Subscript i refers to current period)

$$I_i = a_i 645/\Delta t$$

a · 1" = 14.1=14

$$C = \Delta t/(R + .5\Delta t) = 0.308$$

$a_i \times \frac{645}{2} = 4515$ cfs

$$O_i = CI_i + (1 - C)O_{i-1}$$

$$Q_i = .5(O_{i-1} + O_i)$$

TIME	INFLOW (Fig. 2)		INSTANTANEOUS UNIT GRAPH	2-HOUR UNIT GRAPH
hr	a_i sq.mi.-in.	I_i cfs	Q_i cfs	Q_i cfs
(1)	(2)	(3)	(4)	(5)
0	0	0	0	0
2	14	4,515	1,391 0	700
4	44	14,190	5,333 1391	3,360
6	53	17,093	8,955 5333	7,150
8	79	25,478	14,043	11,500
10	0	0	9,717	11,880
12			6,724	8,220
14			4,653	5,690
16			3,220	3,940
18			2,228	2,720
20			1,542	1,890
22			1,067	1,300
24			738	900
26			510	630
28			352	430
30			242	300
32			168	200
34			116	140
36			81	100
38			55	70
40			39	50
42			26	30
44			19	20
46			13 19	20

13

CLARK METHOD FOR DERIVING UNIT HYDROGRAPHS

Monday 2:05 p.m.

CLARK METHOD FOR DERIVING UNIT HYDROGRAPHS

1. Conceptual Models of the Unit Hydrograph

   a. Conceptual models are abstractions used to simulate the runoff response of a watershed. For example, watershed response can be simulated by passing rainfall through a series of reservoirs.

   b. For a conceptual model to be a useful tool for synthesizing unit hydrographs,

      (1) The model should provide a convenient method for predicting the shape of the unit hydrograph.

      (2) It should be possible to relate the model parameters to watershed characteristics.

2. Concept of the Instantaneous Unit Hydrograph (IUH)

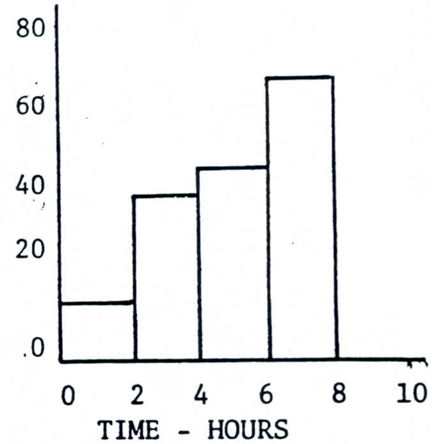
   a. The IUH is the direct runoff hydrograph resulting from one mm (or inch) of rainfall excess occurring instantaneously over the whole watershed.

b. In developing the translation hydrograph, it is convenient to first construct a time-accumulated area curve. This is illustrated on the following two pages. For many applications, satisfactory results can be achieved with a synthetic time-area curve.

c. The translation hydrograph can be obtained from the time-accumulated area curve. For example,

Time (Hours)	Q (Sq Mi/In)
0	
	14
2	
	44
4	
	53
6	
	79
8	

$\frac{\text{Sq Mi-In}}{\text{2 Hours}}$



5. Routing the Translation Hydrograph Through a Linear Reservoir

a. Muskingum Routing

$$O_i = C_0 I_i + C_1 I_{i-1} + C_2 O_{i-1}$$

where

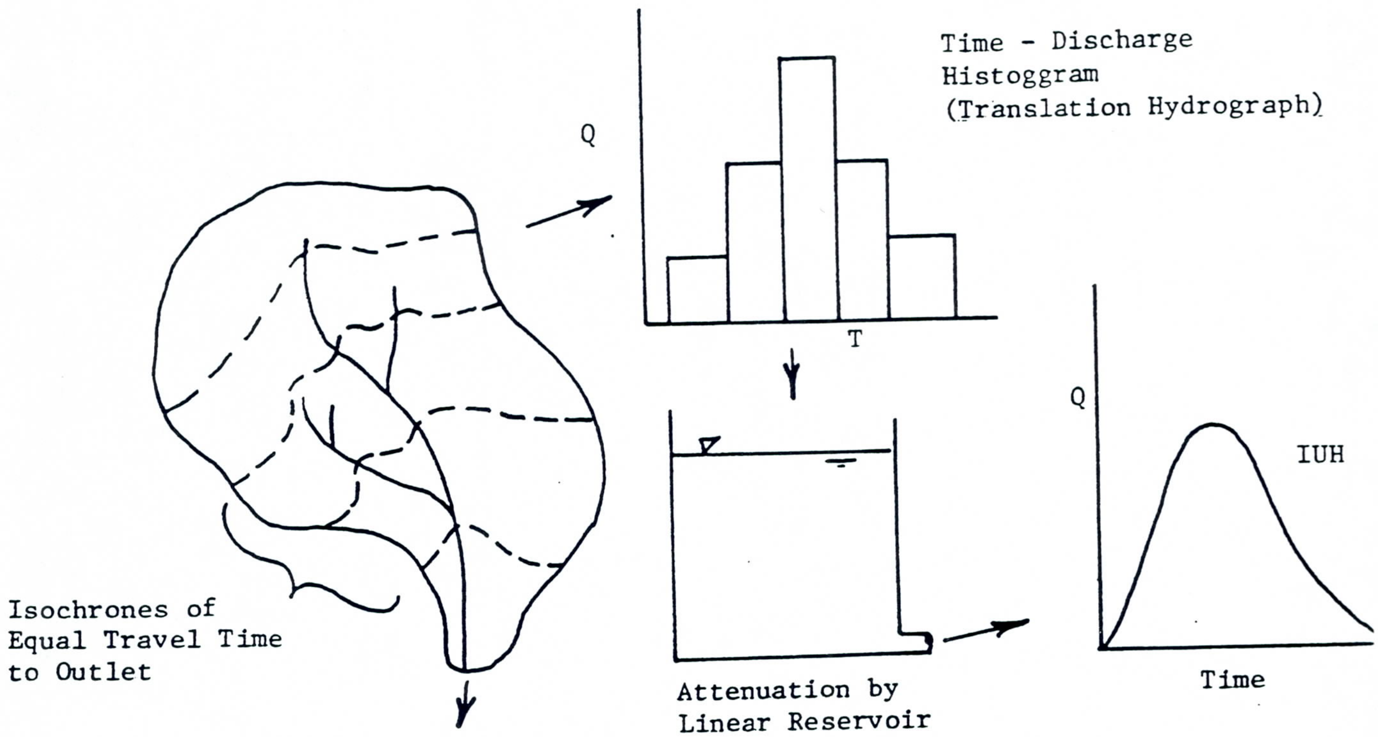
I = Rate of inflow

O = Rate of outflow

C_0, C_1, C_2 = Routing coefficients

Subscripts i and i-1 indicate the beginning and end, respectively, of a time interval.

3. The Clark Conceptual Model of the IUH



The parameters of the Clark model are the translation hydrograph and the storage coefficient, R.

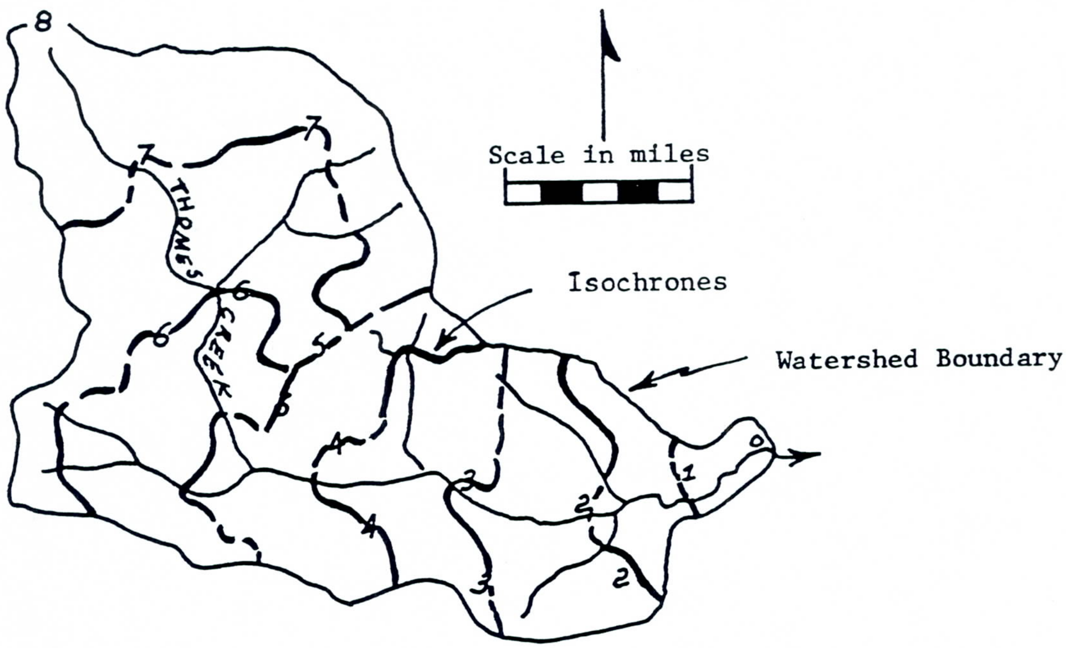
4. Development of the Translation Hydrograph

a. Definitions

(1) Translation hydrograph - Hydrograph obtained by translating rainfall excess from subareas of a basin to the outlet in time intervals proportional to the travel time from the basin outlet.

(2) Time of concentration, T_c - The time required for rainfall excess to travel from the most remote point in the basin to the basin outlet.

Figure 2. Computation of the Time-Area Relation



Travel Time from "8" to Gage is 8.0 Hours for the 32 Miles

Map Area Number (1)	Planimeter Values from Map: Incremental units (2)	Accumulated units (3)	Accumulated area (sq. mi.) (Col 3) · (58.8) (4)	Travel Time in Percent [(1/8)·(100)] (5)
1	0.08	0.08	5	12.5
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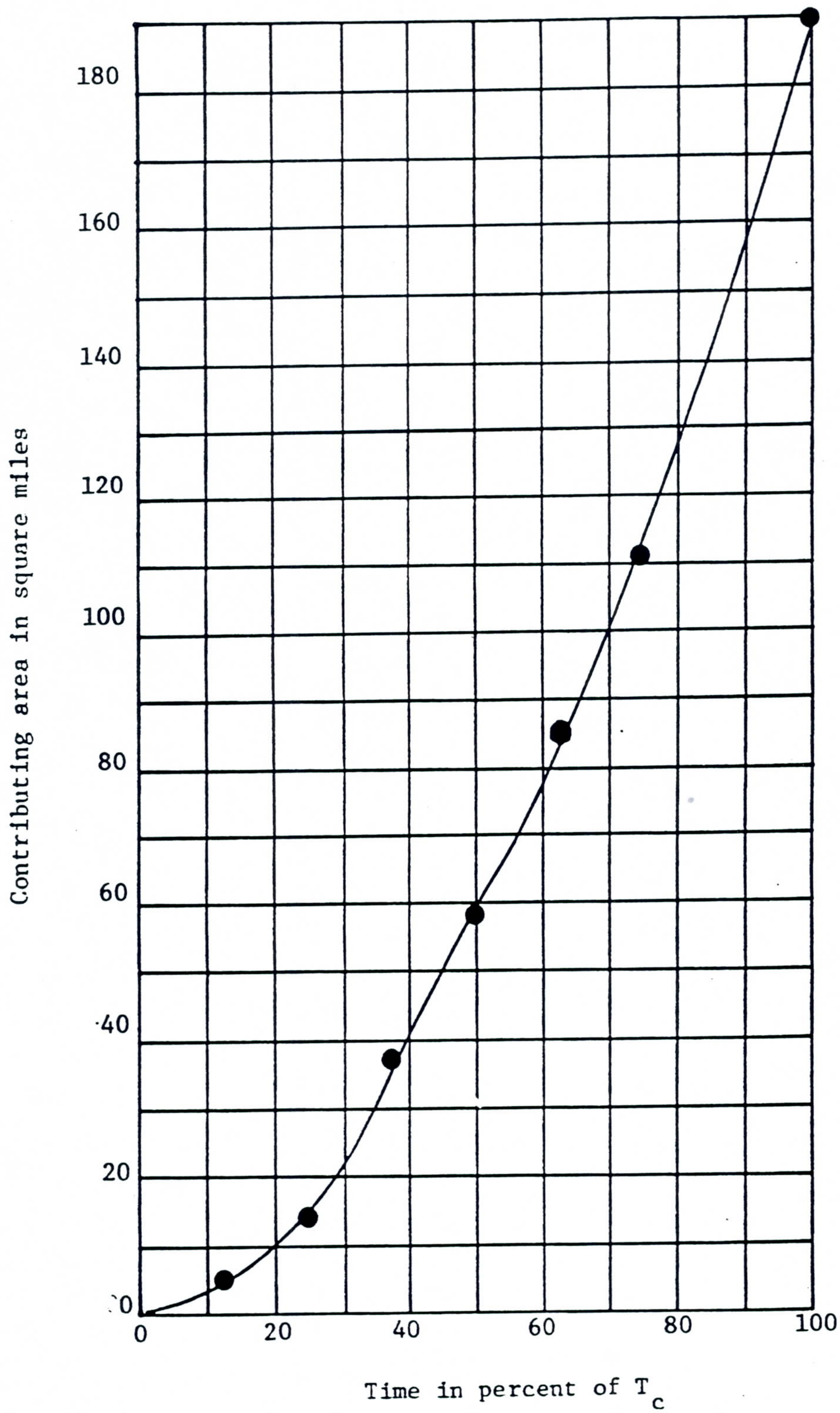
4	0.36	0.99	58	50.0
5	0.45	1.44	85	62.5
6	0.45	1.89	111	75.0
7	0.66	2.55	150	87.5
8	0.68	3.23	190	100.0
Total	3.23			

Sq. mi./Planimeter unit = 190/3.23 = 58.8

Drainage Area = 190 square miles

Figure 3. Watershed Time-Area Relation



For reservoir-type storage for which the Muskingum $X = 0$,

$$C_0 = C_1 = \frac{\Delta t}{2R + \Delta t}$$

$$C_2 = \frac{2R - \Delta t}{2R + \Delta t}$$

where R = storage coefficient of linear reservoir.

If the translation hydrograph is described by a histogram so that $I_i = I_{i-1}$, the routing equation becomes

$$O_i = \frac{2\Delta t}{(2R + \Delta t)} I_i + \frac{(2R - \Delta t)}{(2R + \Delta t)} O_{i-1}$$

let $C = \dfrac{2\Delta t}{(2R + \Delta t)}$

then $(1 - C) = \dfrac{2R - \Delta t}{2R + \Delta t}$

The routing equation can then be stated as

$$\boxed{O_1 = C\, I_i + (1 - C)\, O_{i-1}}$$

b. Application of the routing equation is illustrated on the following page.

6. Conversion of IUH to a Unit Hydrograph of Duration Δt

Average two IUH's spaced an interval Δt apart. This is illustrated in columns (4) and (5) of the example problem.

Table 1. Unit Graph Computation - Clark Method

(Thomas Creek at Paskenta, California)

Drainage Area = 190 square miles
Time of Concentration (T_c) = 8.0 hours (see Figure 1)
Attenuation Value (R) = 5.5 hours (see Figure 1)
Time Interval (Δt) = 2.0 hours

Equations (Subscript i refers to current period)

$$I_i = a_i 645/\Delta t$$

$$C = \Delta t/(R + .5\Delta t) = 0.308$$

$$O_i = CI_i + (1 - C)O_{i-1}$$

$$Q_i = .5(O_{i-1} + O_i)$$

TIME	INFLOW (Fig. 2)		INSTANTANEOUS UNIT GRAPH	2-HOUR UNIT GRAPH
hr (1)	a_i sq.mi.-in. (2)	I_i cfs (3)	Q_i cfs (4)	Q_i cfs (5)
0	0	0	0	0
2	14	4,515	1,391	700
4	44	14,190	5,333	3,360
6	53	17,093	8,955	7,150
8	79	25,478	14,043	11,500
10	0	0	9,717	11,880
12			6,724	8,220
14			4,653	5,690
16			3,220	3,940
18			2,228	2,720
20			1,542	1,890
22			1,067	1,300
24			738	900
26			510	630
28			352	430
30			242	300
32			168	200
34			116	140
36			81	100
38			55	70
40			39	50
42			26	30
44			19	20
46			13	20

7. Determination of T_c and R for a Gaged Basin

a. T_c

Time from the end of a burst of rainfall excess to the inflection point on the recession limb of the resulting direct runoff hydrograph.

b. R

The discharge at the inflection point on the recession limb of the direct runoff hydrograph divided by the slope of the recession limb at that point.

That is:

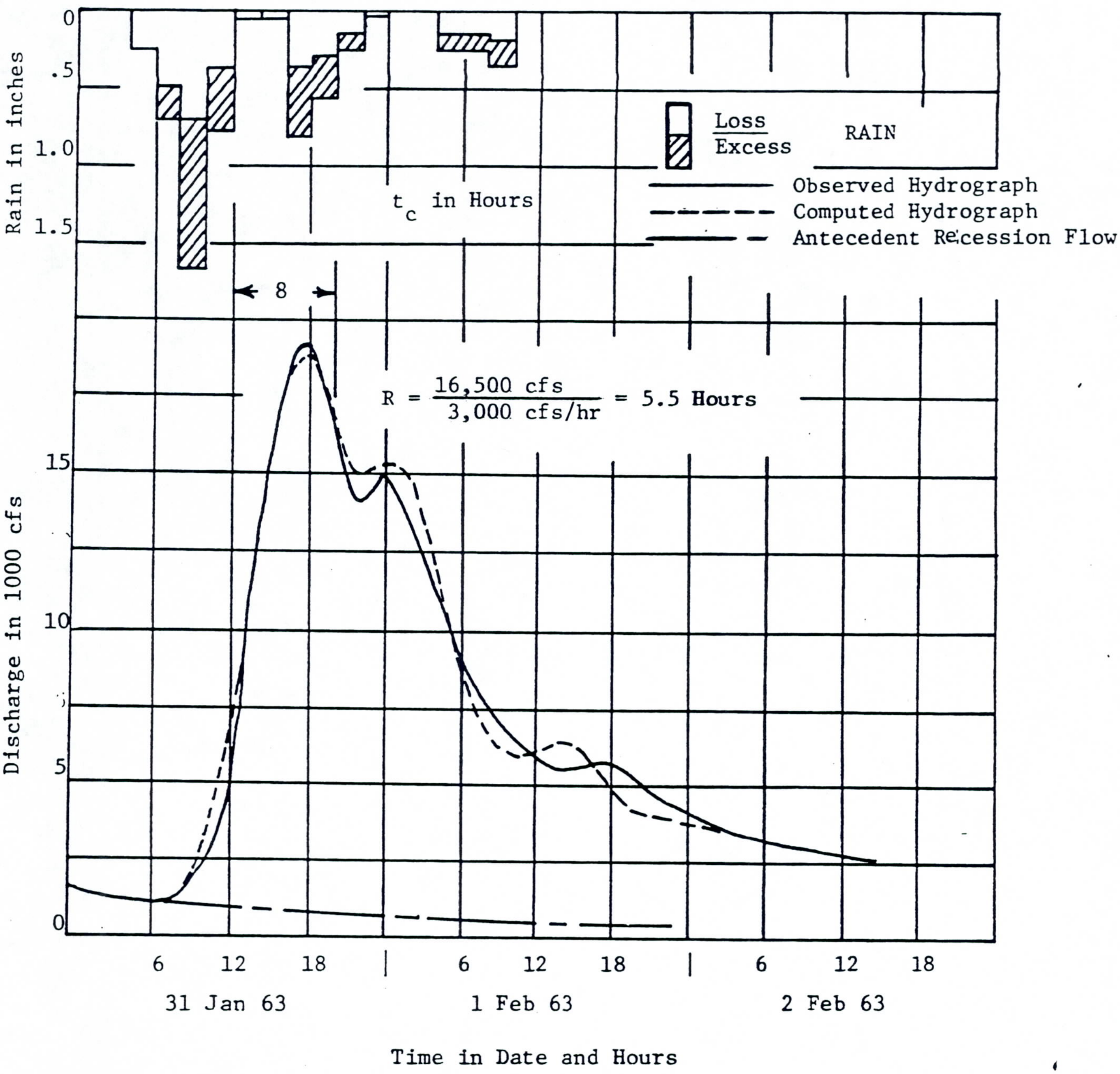
$$R = -\frac{Q}{dQ/dt}$$

where Q and dQ/dt are determined at the point of inflection. This is illustrated on the following page.

c. Values obtained in this fashion for T_c and R are only first approximations; the parameters should be adjusted to give optimum reproduction of historical events.

d. Where data is not available for calibrating Clark parameters, use regression analysis to relate parameters to watershed characteristics.

Figure 1. Determination of Clark Coefficients and Flood Reconstitution



Drainage Area: 190 sq. mi.

8. Advantage of Clark Method

Aside from the application of the Clark method as the basis for developing synthetic unit hydrographs, a major advantage of the method is that it provides a complete unit hydrograph in terms of two parameters. This is particularly useful for computer applications.

9. Bibliography

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g. Dawdy, D. R., Lichty, R. W., and Bergmann, J. M., "A Rainfall-Runoff Simulation Model for Estimation of Flood Peaks for Small Drainage Basins," U.S. Geological Survey Professional Paper 506-B, 1972.

CLARK'S UNIT HYDROGRAPH PHILOSOPHY

1. Instantaneous UH theory hypothesizes that 1 inch of rainfall excess is dumped instantaneously on the entire watershed.

2. The total watershed travel time is estimated from an observed hydrograph by postulating that beyond the deflection point on the receding side of the hydrograph, the flow is recession flow derived from water in storage without further rainfall. In the HEC-I example, the time of concentration t_τ between the end of rainfall excess and the deflection point is 8 hours.

3. Each point on the watershed has a travel time to the outlet. If storage would not delay the runoff, some sort of "piston" flow would cause the entire inch of rainfall excess to arrive at the outlet at its time t_τ. For simplicity, the watershed is divided into subareas, which will deliver water at a uniform rate during the corresponding time interval. For example, the subarea between contours 2 and 3 of area 3 will deliver a rainfall volume

$$V_3 = 1 \text{ in.} \times a_3 \tag{1}$$

over a time interval t = 1 hour, at a rate

$$I_3 = \frac{V_3}{\Delta t} = \frac{1 \text{ in} \times 23 \text{ mi}^2 \times 5280^2 \text{ft}^2/\text{mi}^2}{12 \text{ in/ft} \times 1 \text{ hr} \times 3600 \text{ sec/hr}} = 645 \frac{a_3}{\Delta t} = 14800 \text{ cfs}$$

Only 4 subareas, travel times 0 to 2, 2 to 4, 4 to 6, and 6 to 8 hours, are considered to save, space, resulting in 4 consecutive hypothetical piston flows 4515, 14190, 17093, and 25478 cfs, which are labeled "inflows" because they will be routed through hypothetical reservoir representing the storage in the watershed.

4. The inflows into the hypothetical reservoir are now routed through the reservoir using the equation

$$Q_i = cI + (1 - c)Q_{i-1} \tag{2}$$

where Q_i is the IUH outflow at the end of step i, and c is a coefficient, to be determined. To assign a calibration value to the coefficient c, the inflection point is chosen because beyond this point, I = O.

5. In the recession flow, it is assumed that the rate of flow decline, dQ/dt, is proportional to Q. Thus a storage coefficient

$$R = - \frac{Q}{dQ/dt} = - \frac{Q}{dQ/dt} \tag{3}$$

is found from the tangent to the hydrograph at this point. R should be a constant. The receding flow can now be described in steps as

$$Q_i = Q_{i-1} + \frac{dQ}{dt} \Delta t = Q_{i-1} - \frac{Q_i + Q_{i-1}}{2} \frac{\Delta t}{R} \tag{4}$$

Combining terms, we get

$$Q_i \left(1 + \frac{\Delta t}{2R}\right) = Q_{i-1} \left(1 - \frac{\Delta t}{2R}\right) \tag{5}$$

or

$$Q_i = Q_{i-1} \frac{2R - \Delta t}{2R + \Delta t} \tag{6}$$

Comparing with equation (2)

$$\frac{2R - \Delta t}{2R + \Delta t} = 1 - c \tag{7}$$

so that

$$C = 1 - \frac{2R - \Delta t}{2R + \Delta t} = \frac{2R + \Delta t - 2R + \Delta t}{2R + \Delta t} = \frac{\Delta t}{R + \Delta t/2}$$

6. The coefficient C can now be evaluated, and the IUH computed as tabulated on page 7.

7. Alternative Solution for R

It has been found that the hydrograph slope at the point of inflection on the recession side is often very steep and irregular, making it very difficult to determine dQ/dt with any degree of confidence. An alternate equation is derived below which results in a more stable estimate of the parameter R.

$$R = - \frac{Q}{dQ/dt}$$

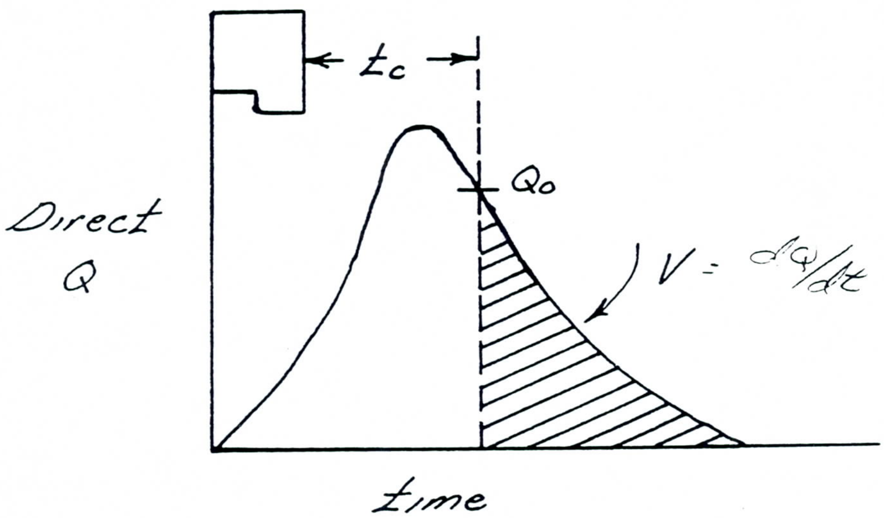
$$\frac{dQ}{Q} = d \ln Q = - \frac{dt}{R}$$

$$\ln \frac{Q}{Q_o} = \frac{t - t_o}{R} \qquad t_o = 0$$

$$Q = Q_o e^{-t/R}$$

$$\text{Vol } V = \int Q dt = Q_o R e^{-t/R} \Big|_{t=\infty}^{t=0} = Q_o R$$

$$R = V/Q_o$$



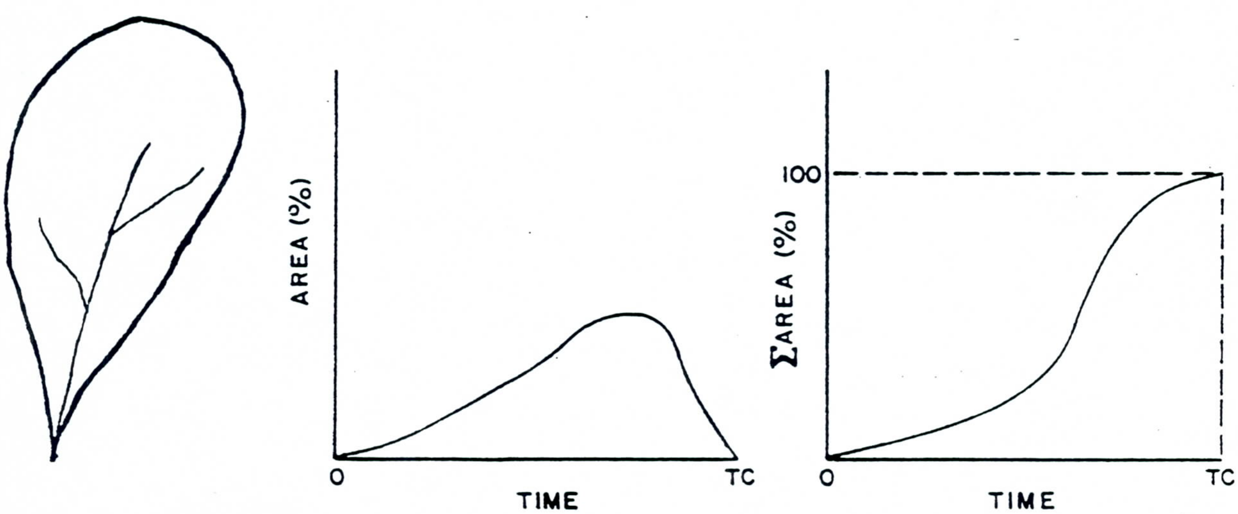
SYNTHETIC TIME-AREA CURVE

In the application of Clark's unit hydrograph procedure and other methods of runoff transformation, it is often necessary to utilize a time-area curve for the basin under study.

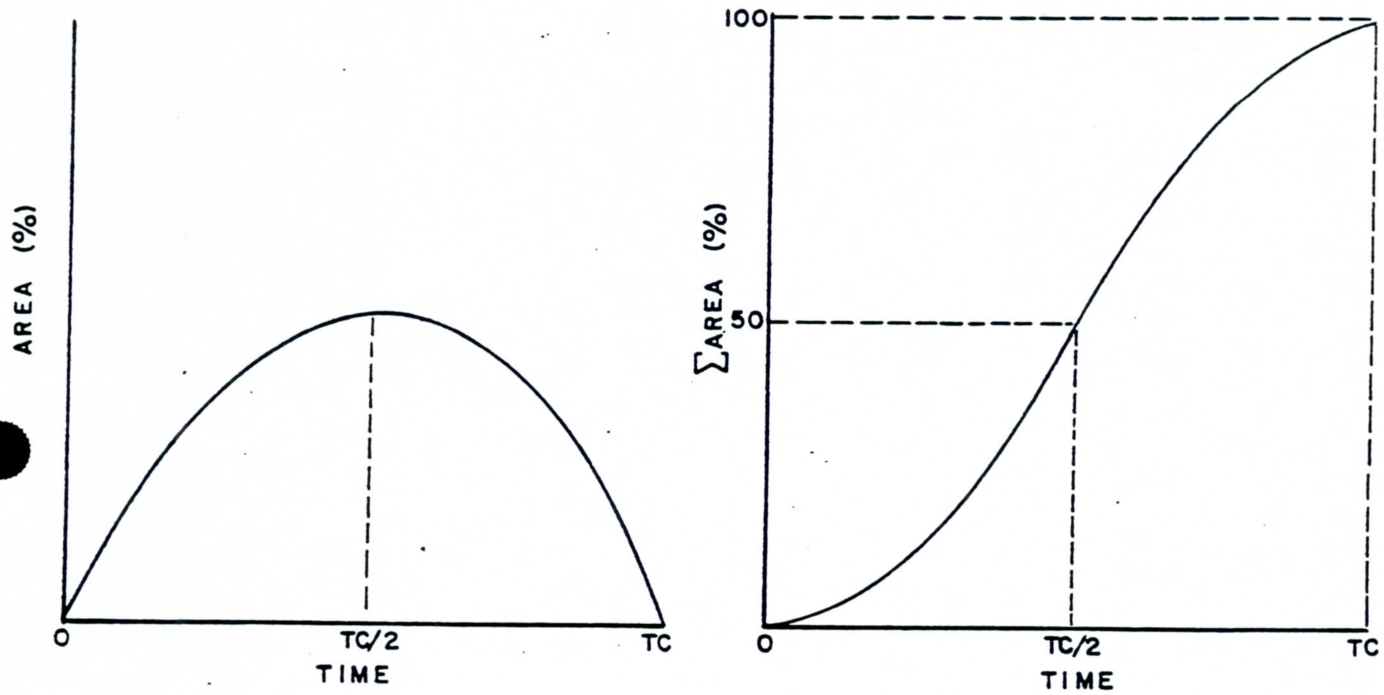
This time-area curve may be obtained in a rigorous fashion using measured or calculated flow travel times from various locations in the basin. An expedient approximation to the time-area curve is the distance-area curve. When the distance-area curve is used instead of the time-area curve, the assumption is made that flow time from a given location is proportional only to the travel distance from the basin outlet.

In many studies, it has been found that it is not necessary to use the actual distance-area curve in the analysis. Instead, a distance-area curve of general shape is used to represent the time-area curve of the basin. This generalized distance-area curve is referred to as a synthetic time-area curve.

Figure 1 shows a typical basin, its time-area curve, and its time-accumulated area curve.



It has been found that a smooth function can easily be fit to the time accumulated-area curve, and that the time-area curve may be taken as symmetrical about TC/2. A symmetrical time-area curve yields a time accumulated-area curve of the form shown in Figure 2.



It is convenient to use a function of the form

$$\text{Acc Area} = C\ T^n$$

to represent the first half of the time accumulated-area curve

$(0 > T > \frac{TC}{2})$ and $(1\text{-Acc Area}) = C\ (1-T)^n$ for the second half of the curve

$(\frac{TC}{2} < T < TC)$.

Such a function, with n = 1, represents a rectangular watershed, with a rectangular synthetic time-area curve and a triangular synthetic time accumulated-area curve, as shown in Figure 3. With n = 2, the function represents a diamond-shaped area, as shown in Figure 4. In HEC-1, the exponent "n" has been set to 1.5, which was found to yield a shape representative of a common watershed configuration. With n = 1.5, the function represents the basin shape shown in Figure 5.

The Synthetic time-area curve in HEC-1 may be used for most watersheds. However, for basins that deviate substantially from the generalized shape, a real time-area curve should be used.

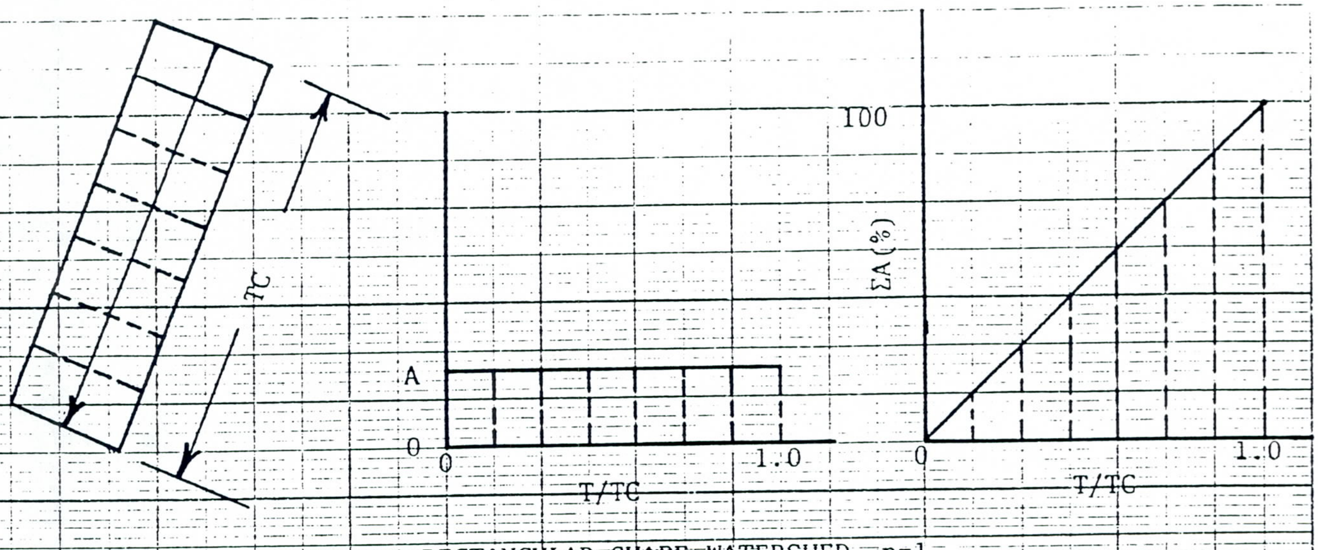


FIG. 3 RECTANGULAR SHAPE WATERSHED, n=1

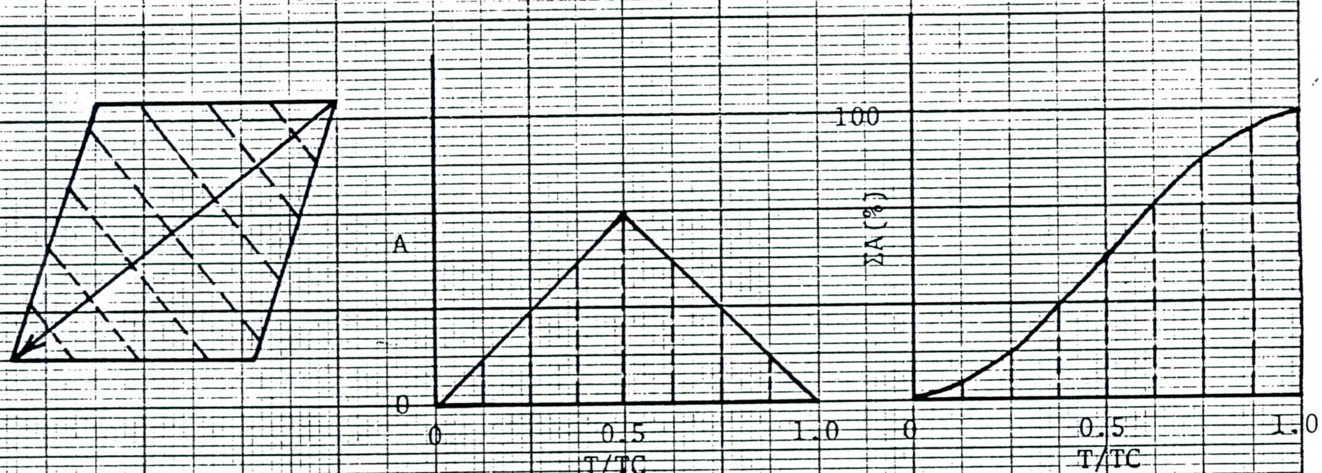


FIG. 4 DIAMOND SHAPE WATERSHED, n=2

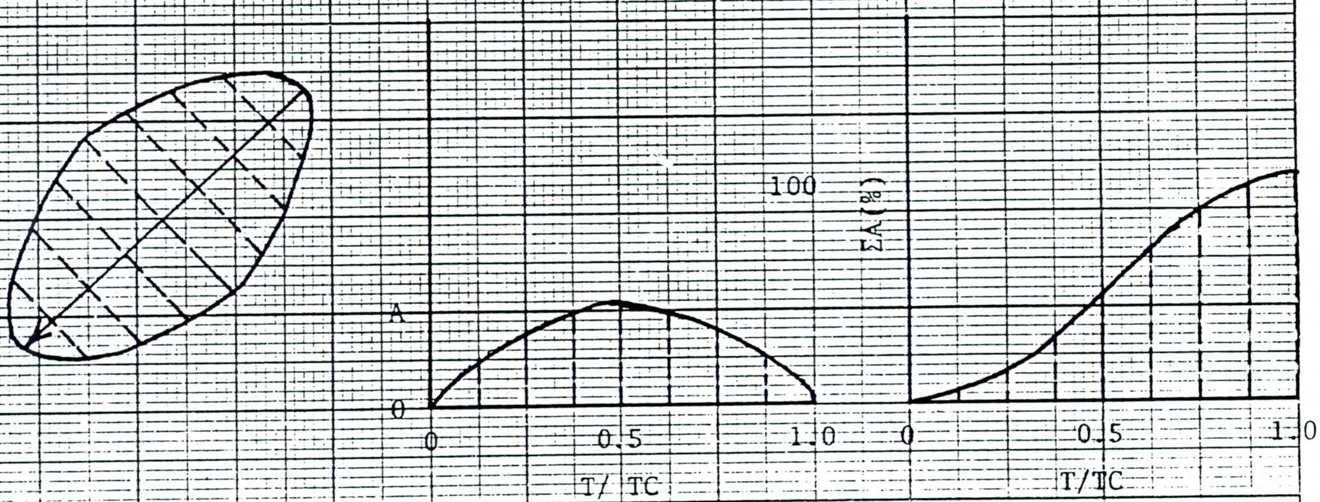


FIG. 5 GENERALIZED SHAPE WATERSHED, n=1.5

WORKSHOP NO. 4

FLOOD HYDROGRAPH USING CLARK UNIT HYDROGRAPH

PROBLEM STATEMENT

CLARK UNIT GRAPH

PROBLEM DESCRIPTION

Recorded flood hydrographs for the October 1962 and December 1941 floods on Clark Creek at Grayson are given on the following pages. Coordinates of the time-area relation for the basin are as follows:

% of Time of Concentration	Contributing Area Sq. Mi.
0	0.0
10	3.0
20	7.5
30	13.5
40	20.5
50	28.0
60	37.0
70	48.0
80	64.0
90	84.0
100	100.0

PROBLEM

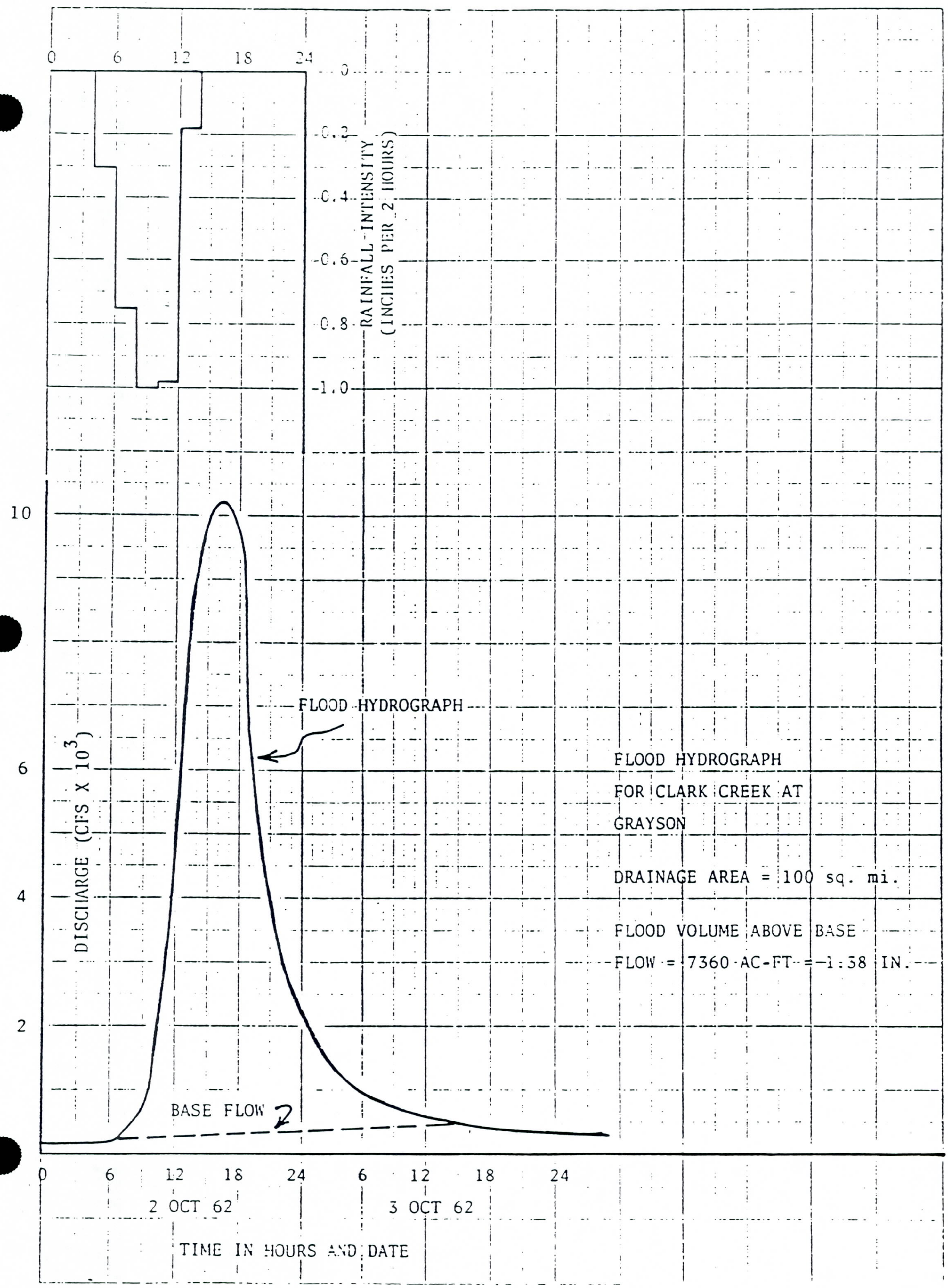
a. Use the Clark method to develop a 2-hour unit graph for 100 sq. mi. Clark Creek Basin.

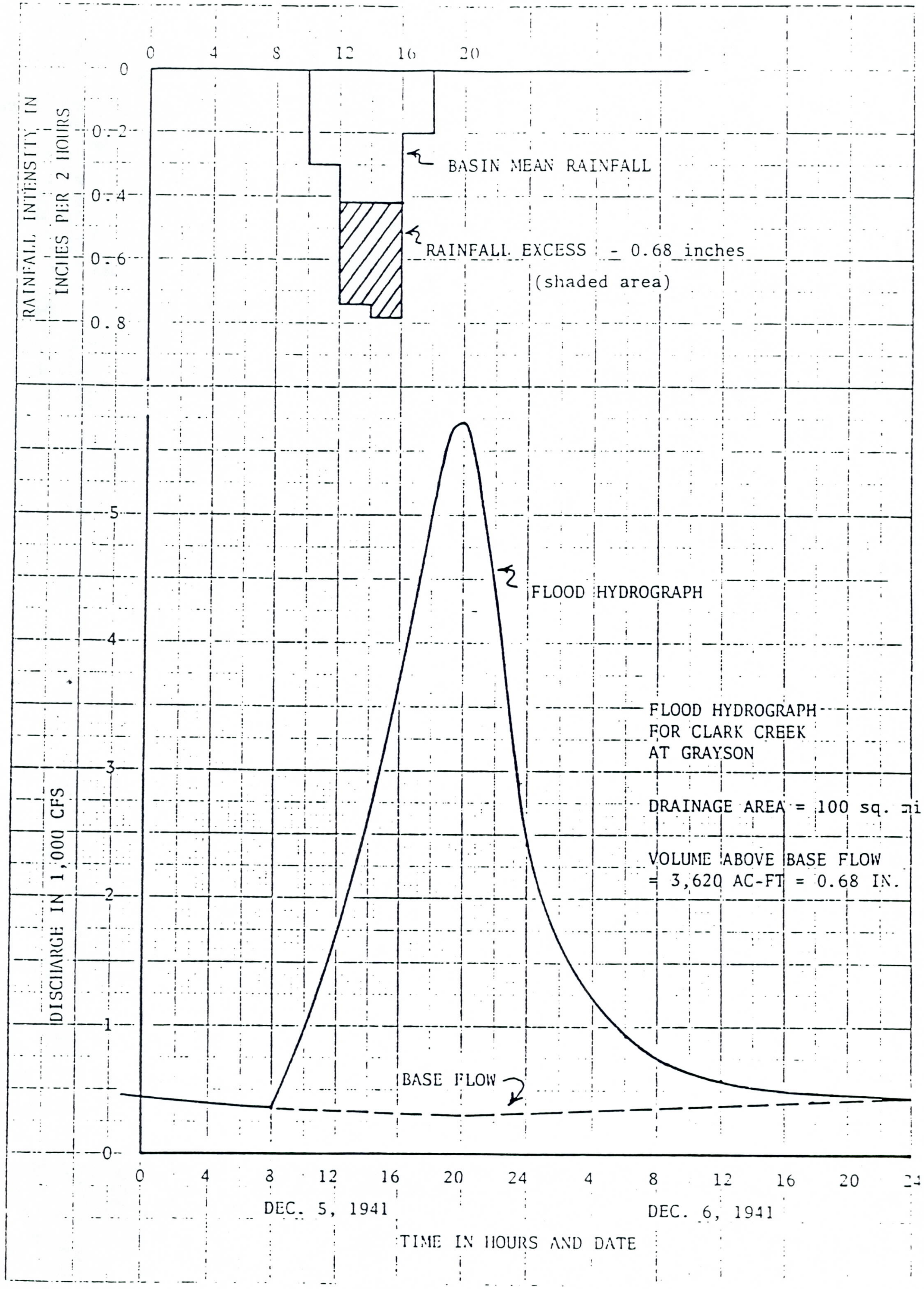
b. Use the unit graph derived in "a" above and the rainfall excess given below to reconstruct the December flood hydrograph.

December 5, 1941

Time Ending (hrs)	Rainfall (in)	Loss (in)	Excess (in)
1200	0.30	0.30	0
1400	0.74	0.42*	0.32
1600	0.78	0.42	0.36
1800	0.20	0.20	0

* Resulting uniform loss rate


0
6
12
18
24
0
0.2
0.4
0.6
0.8
1.0
RAINFALL INTENSITY
(INCHES PER 2 HOURS)
10
6
4
2
DISCHARGE (CFS X 10³)
FLOOD HYDROGRAPH
BASE FLOW
FLOOD HYDROGRAPH
FOR CLARK CREEK AT
GRAYSON
DRAINAGE AREA = 100 sq. mi.
FLOOD VOLUME ABOVE BASE
FLOW = 7360 AC-FT = 1.38 IN.
0
6
12
18
24
6
12
18
24
2 OCT 62
3 OCT 62
TIME IN HOURS AND DATE



RAINFALL INTENSITY IN INCHES PER 2 HOURS
0
0.2
0.4
0.6
0.8
0 4 8 12 16 20
BASIN MEAN RAINFALL
RAINFALL EXCESS - 0.68 inches
(shaded area)
FLOOD HYDROGRAPH
FLOOD HYDROGRAPH FOR CLARK CREEK AT GRAYSON
DRAINAGE AREA = 100 sq. mi.
VOLUME ABOVE BASE FLOW = 3,620 AC-FT = 0.68 IN.
BASE FLOW
DISCHARGE IN 1,000 CFS
0
1
2
3
4
5
0 4 8 12 16 20 24 4 8 12 16 20 24
DEC. 5, 1941
DEC. 6, 1941
TIME IN HOURS AND DATE

WORKSHOP NO. 4

FLOOD HYDROGRAPH USING CLARK UNIT HYDROGRAPH

PROBLEM SOLUTION

CLARK UNIT GRAPH

Development of Clark Unit Graph for Clark Creek Basin from October 1962 and December 1941 hydrographs:

Using t_c = 6 hours, R = 2.2 hours, Δt = 1 hour

$$C = \frac{\Delta t}{R + 0.5\Delta t} = \frac{1}{2.7} = 0.37; \qquad 1 - C = 0.63$$

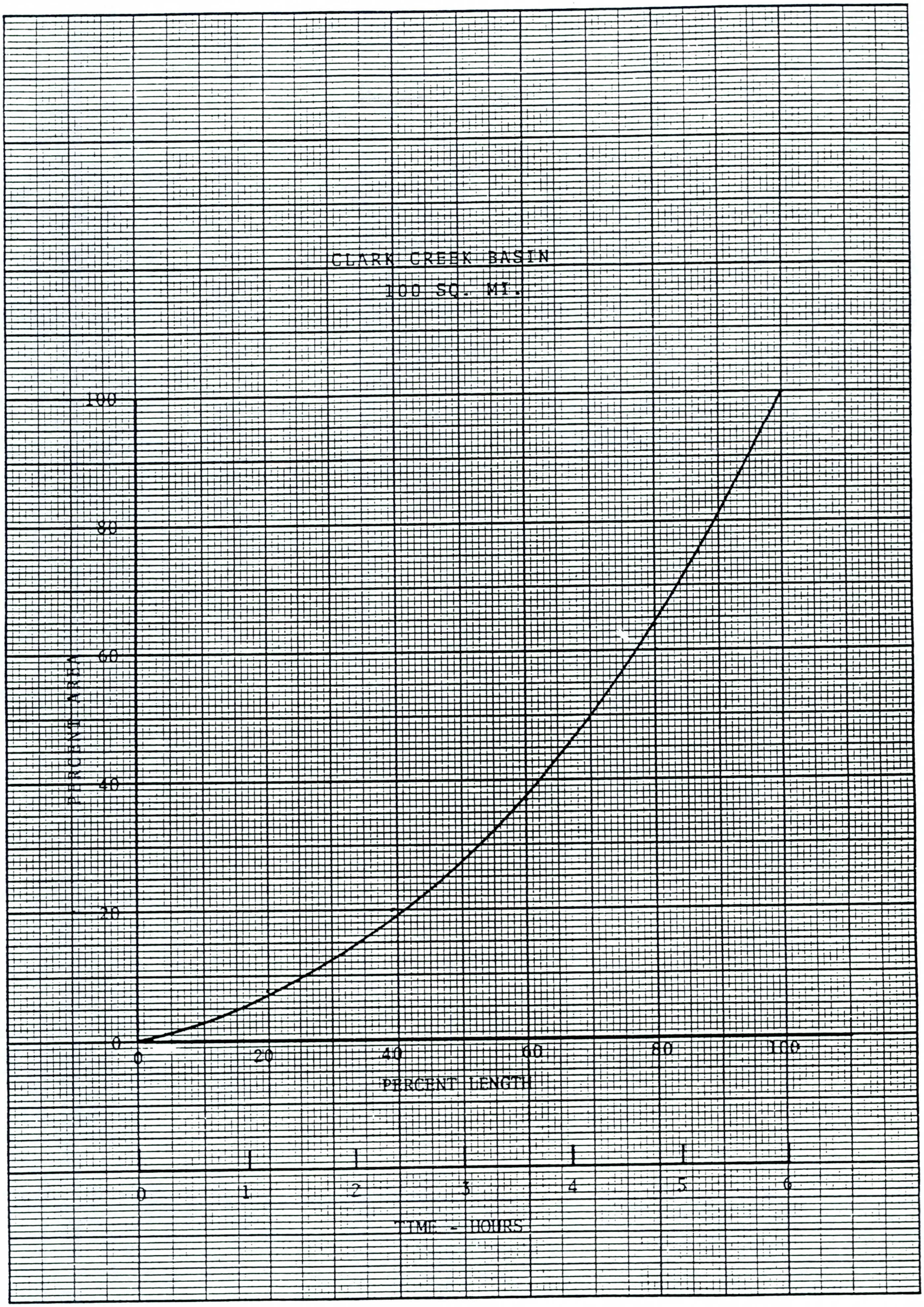
$$I_i = 645 a_i / \Delta t \; ; \qquad O_i = C I_i + (1 - C)\, O_{i-1}$$

TIME	a_i (mi^2/in)	Δa_i (mi^2/in)	I_i (cfs)	CI_i (cfs)	$(1-C)O_{i-1}$ (cfs)	O_i (cfs)	1 HR UG (cfs)	2 HR UG (cfs)
0	0	0	0	0	0	0		
1	5.5	5.5	3547	1312	0	1312	656	
2	15.5	10.0	6450	2386	827	3213	2262	
3	28.0	12.5	8062	2983	2024	5007	4110	3186
4	43.5	15.5	9998	3699	3154	6853	5930	5020
5	70.5	27.0	17415	6444	4318	10762	8807	7368
6	100.0	29.5	19027	7040	6780	13820	12291	10549
7		0	0	0	8706	8706	11263	11777
8						5485	7095	9179
9						3456	4470	5782
10						2177	2816	3643
11						1371	1774	2295
12						864	1117	1445
13						544	704	910
14						343	443	573
15						216	280	361
16						136	175	228
17						86	111	143
18						54	70	90
19						34	44	57
20						21	27	36
21						13	17	22
22						8	10	13

$$\left\{ 100\,mi^2 \times 1\,in. \times \frac{1}{12}\frac{ft}{in} \times (5280)^2 \frac{ft^2}{mi^2} \times \frac{1}{3600}\frac{hr}{sec} = 64{,}500\ cfs\text{-}hrs \right\}$$

COMPUTATION OF THE HYDROGRAPH:

	TIME	DIRECT RUNOFF (CFS)	BASE FLOW (CFS)	TOTAL FLOW (CFS)
DEC 5	1300	105	330	435
	1400	468	325	793
	1500	1138	320	1458
	1600	2132	315	2447
	1700	3505	310	3815
	1800	5183	305	5488
	1900	6423	300	6723
	2000	6737	295	7031
	2100	6091	300	6391
	2200	4471	305	4776
	2300	2817	310	3127
	2400	1775	315	2090
DEC 6	0100	1119	320	1439
	0200	704	325	1029
	0300	443	330	773
	0400	278	335	613
	0500	175	340	515



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8-8 Unit Hydrograph for Basins with Adequate Data

For the purposes of this chapter, the methods described in the following paragraphs describe the preferred methodology. Other methods may be applicable, and if used must be fully described, justified, and documented.

In this chapter, a gaged basin is one for which sufficient rainfall and streamflow data are available from gages within the basin to allow development of the PMF hydrograph without resorting to data from outside the basin. In some cases, rainfall data from a gage near, but outside, the basin may be used if the storm track clearly included the basin and that gage. The COE-developed computer program HEC-1, Flood Hydrograph Package, (the most recent version is dated 1990) is recommended for use in developing unit hydrographs for gaged basins and PMF inflow hydrographs [COE 1990]. HEC-1, with added user-friendly input and output routines, is available from some private software vendors. Other programs may be used but must be fully documented and verified.

Programs with capabilities similar to HEC-1 have been produced by other agencies; some have unique capabilities, or they incorporate data or relationships applicable to particular regions of the United States. The Tennessee River Basin, for example, has been extensively studied by the Tennessee Valley Authority (TVA), where extensive hydrologic information for that region, as well as applicable computer programs, are available. Similarly, the Los Angeles District of the COE has developed a preprocessor program for HEC-1, which incorporates unit hydrographs for the district's entire region.

- If regional studies that have produced accepted results are available, the methods presented in those studies may be used, if justified. Use of the regional unit hydrograph in developing the PMF inflow hydrograph is described in Section 8-10.

Caution: Deviations from any of the recommended methods should be fully justified, described, and documented.

8-8.1 Historic Floods for Calibration and Verification

Data from historic storms and the resulting floods that are available from systematic gaging should be considered for use in developing unit hydrographs. Although those for which the rainfall sequence is the most uniform, and for which the flood hydrograph is single-peaked, are the most desirable for use in unit-hydrograph computation, HEC-1 provides the means to satisfactorily analyze flood hydrographs that are not single-peaked. It is always necessary to be able to identify the runoff-contributing area for each flood used. The more storms and floods that can be used, the greater will be the confidence in the PMF computation. If data from at least three historic floods are available, two should be used for calibration of the unit hydrograph and one for verification. Ideally, the calibration floods should have occurred during the season when the critical PMP would occur. For calibration, unit-hydrograph parameters are computed by analyzing the largest floods with the best data to develop a representative unit hydrograph; the degree to which the representative unit hydrograph provides for duplication of the verification floods is then assessed. Actually, the verification process affects calibration; if the representative unit hydrograph does not produce a good duplication of the verification flood, the unit-hydrograph parameters must be reviewed and modified to improve the fit for most events.

- Single extreme rainfall events should be used to develop the unit hydrograph. A unit hydrograph developed from a complex storm (i.e., multiple events occurring back-to-back) can be in error and is very difficult to compute, primarily because of baseflow separation.

Caution: It will be difficult to develop a unit hydrograph that generally reproduces all portions of all historic flood hydrographs. The adopted unit hydrograph should be the one judged to best predict the magnitude, shape, and timing of the PMF. Normally, the adopted unit hydrograph should be the one that most faithfully reproduces the largest floods of record.

In choosing the floods to be used for calibration and verification, the distinction between rain-on-snow and rainfall-generated floods should be kept in mind. For the same basin, a rain-on-snow flood will exhibit a longer lag time than an equal event produced by rainfall alone.

- If the critical PMP will occur during a month when a significant part of the basin will be covered by snow, the calibration floods should include historical floods generated by rain on snow.

- If the critical PMP will occur during summer months when snow cover is unlikely, the calibration floods should be selected from rainfall-dominated floods.

- In analyzing major floods that occurred during a cold season, it will be desirable to judge whether or not the ground was frozen, since frozen ground may have caused reduced infiltration rates.

If only historic flood peak discharge and time-to-peak data are available, it may be advisable to attempt calibration to that data, assuming a triangular-shaped hydrograph. This may be appropriate if application of historic rainfall with synthetic unit-hydrograph parameters does not provide a good match with these data.

8-8.2 Determination of Basin Average Rainfall

Basin average rainfall must be determined for each storm used in developing a unit hydrograph. The method to be used in determining basin average rainfall depends on whether orographic effects exist in the basin.

- If orographic effects are not important, either the Thiessen polygon or the distance-averaging method can be used to calculate the basin average precipitation using recorded rainfall at each gage.

- For basins where orographic effects are important, an isohyetal map provides the best means to determine basin average rainfall. The basin average rainfall for each subbasin is determined by integrating the areas between isohyets in the subbasin.

You don't want HEC-I to do time average if you have two gages on WS with diff. hydrographs.

HEC-1 will compute basin average precipitation from individual gage records, if a weighting factor is entered for each rain gage. When multiplied by the recorded rainfall depth at the gage, the weighting factors yield the portion of the basin average (or subbasin average) rainfall contributed by the gage reading. The weighting factors must be externally computed from the results of either the Thiessen polygon or isohyetal methods.

* *Caution: Separate weights are required to (1) determine total storm volume and (2) develop a temporal distribution of the rainfall.*

8-8.3 Cold Season Considerations

It should be determined if at least part of the basin had snowpack or ground subject to frost during historic floods.

8-8.3.1 Snowmelt Considerations

If the basin is one for which at least part of the drainage area is subject to snowpack and if the historical floods were influenced by snowmelt, snowmelt calculations must be included in the rainfall-runoff simulation process. The area covered by snow at the time of the flood-producing storm must be determined from the data acquired.

To use the snowmelt function of HEC-1, the temperature at the base elevation of the snowpack is required along with a temperature-lapse rate. For mountainous areas, the elevation is usually taken as increments of 1,000 feet and the lapse rates are given in increments of degree change per 1,000 feet.

- If sufficient temperature information is not available to construct a lapse rate for each storm, a rate of 3°F per 1,000 feet may be used.

- The degree-day method of snowmelt computation is recommended; alternative methods exist and may be used if documented. Recommended values of "degree-day" factors for use in snowmelt calculations can be obtained from the U.S. Army Corps of Engineers Snowmelt Manual EM 1110-2-1406 [COE 1960].

- Precipitation should be assumed to fall as snow above the elevation at which it is 34°F. HEC-1 makes this assumption.

Snowmelt from large, relatively flat areas—such as the northern midwest—are calculated by HEC-1 in the same manner as for mountainous areas, but temperature will be more uniform across the area. Areas covered by forests, which will be covered by humus beneath the snow cover, will tend to have higher retention and infiltration rates. HEC-1 provides the capability to consider snowmelt in up to 10 zones of equal increments of elevation.

8-8.3.2 Infiltration Characteristics of Potentially Frozen Soils

It is well understood that the structure type of soil frost has a strong influence on the rate of infiltration of soil [Trimble, et al. 1987]. Because of different vegetation cover and surface soil characteristics, soils will respond differently to freezing, producing different types of soil frost structures. These structures are most commonly classified as either concrete or granular frost. Soils with concrete frost are identified by dense thin ice lenses and ice crystals. Soils with concrete frost allow very little infiltration. Granular frost, typically found in woodland soils, consists of small frost particles intermingled with soil particles. Typically, soils classified as having granular frost have higher infiltration rates than the same soil unfrozen [Blackburn and Wood 1990].

Frost structures are related to the moisture content of the frozen soil [Post and Dreibelbis 1942]. Soils frozen at low moisture content may become granulated and provide little impediment to infiltration. Conversely, soils frozen at high moisture contents often freeze into massive, dense, concrete-like structures that are nearly impermeable to water [Zuzel and Pikul 1987].

Reduced levels of moisture content are found in forested areas because of interception and evapotranspiration [Kane and Stein 1983]. These low moisture contents result in granular frost structures in the winter.

Many researchers have identified the effects of soil freezing on the infiltration capability of soils. Type of frost, soil structure and antecedent soil moisture content have all been noted as factors influencing frozen soil infiltration.

In Engelmark's set of laboratory experiments [Engelmark 1987], infiltration rates were measured in a fine sand. The grain-size curve of the fine sand indicated 84 percent passing a #40 sieve and 5 percent passing a #200 sieve. Infiltration rates obtained for this soil in the frozen state were between 1-2 mm/min. (2.4-4.7 in/hr).

Another experiment executed by Blackburn and Wood [Blackburn and Wood 1990] provided a range of infiltration rates of 0.42-1.08 mm/min (1-2.4 in/hr), depending on the type of frost that existed. This experiment was performed on a sandy soil of the Larimer series.

When the soil type is combined with the vegetation, a low soil moisture content can be predicted. Even during the PMP, the rainfall rate may not exceed the rate of infiltration in soils and they will not be saturated. With these conditions, a granular soil frost will predominate in the winter. Granular soil frost is far from impervious; it typically has infiltration rates the same as, or higher than, the soil in an unfrozen condition [Blackburn and Wood 1990].

- Wetlands should be modeled as impervious elements. These soils, although sandy, may intersect the seasonal high water table and thus have a higher potential to produce a concrete type of frost.

- Infiltration rates for granular soils, such as sand and sandy loam, should be assumed equal to the unfrozen condition.

- Soils with high silt content associated with high groundwater tables should be assumed to be impervious.

- Clays should also be assumed to be impervious.

- Forested soils or soils with a minimum 4-inch humus depth should have unfrozen condition infiltration rates applied [Kane and Stein 1983].

- Non-forested soils, other than sands or sandy loams, should be considered impervious when they occur within the historical maximum frost depth.

It will be difficult to determine an infiltration rate with confidence for a historical storm. This is usually due to lack of data, whether it relates to combination floods (rainfall-snowmelt), non-uniform snowpack water equivalent with the basin, lack of snowpack water equivalent information for spatial distribution, lack of temperature data both temporally and spatially, and unknown conditions of frozen ground within the basin before and during the historic event. HEC-1 can be used to determine an optimized infiltration rate for each subarea of a basin in analyzing a historic event. In assessing the reasonableness of this optimized infiltration rate, the hydrologic engineer will need to consider the character of the soils in the basin, whether or not they were frozen at the time of the runoff, and the quality and adequacy of the data indicated above. Unless the definition of these data is optimal for the historic event, the optimized infiltration rate should not be used in the determination of the runoff hydrograph for the PMF, rather refer to 8-10.3.

8-8.4 Base-Flow Separation

Separation of base flow from runoff in unit-hydrograph analysis has been done in several different ways, none of which are exact. This chapter suggests that the procedure specified for use in HEC-1 parameter estimation be used. Three parameters must be determined from the recorded flood data and used as input to separate direct runoff and baseflow:

- The flow rate at the beginning of runoff simulation STRTQ.
- The value of flow at which direct runoff ceases QRCSN.
- The recession characteristic RTIOR.

As an aid in calculating these parameters, logarithms of recorded flows during the hydrograph recession should be plotted against the time at one-hour intervals (semilog plot). QRCSN is taken as the flow rate at which the plot of the recession deviates from a straight line and RTIOR is taken as the slope of the straight line portion of the plot.

Caution: Choosing QRCSN can have an important effect on the ordinates of the unit hydrograph and will involve judgment, since the plots are not always smooth and the deviation is often gradual. Figure 8-8.1 shows the way in which baseflow and surface runoff are separated in HEC-1.

8-8.5 Time of Concentration and Clark's Storage Coefficient for Each Subbasin

HEC-1 will calculate values of the time of concentration T_c, and a storage coefficient R to provide a unit hydrograph which yields, by transform, an optimized fit to a recorded flood hydrograph [HEC 1990]. R is a coefficient reflecting effect of storage in the basin and is described in Clark's original paper [Clark 1943].

- The time of concentration is defined as the time required for runoff or water to travel from the most remote point in the watershed to the outlet or point of consideration.

The ratio $R/(T_c + R)$ tends to be approximately constant for hydrologically similar drainage basins in a region. Values for R and T_c can be computed for input into HEC-1. Using the optimization capability of HEC-1, rainfall and resulting flood flows can be input to the program and values of $R/(T_c + R)$ and T_c computed so that the unit-hydrograph shape is optimized to produce a best fit between recorded and simulated flood flow. HEC-1 also computes separate values for R and T_c, which should be checked against those estimated from drainage basin characteristics.

- If the agreement is good, the value of $R/(T_c + R)$ should be kept constant in the hydrograph analysis.

To check HEC-1-derived values for the time of concentration T_c, the time between the end of rainfall excess and the point of inflection as plotted on the recession hydrograph should be scaled for each storm and related flood hydrograph.

Caution: If values computed by HEC-1 differ significantly from the scaled value, both should be reviewed and the calculations verified. The scaled value should control, unless a clock-synchronization error is found in either rainfall or streamflow records.

In addition, a check can be made by calculating a time of concentration using hydraulic theory. This is done by dividing the watercourse from the basin outlet to the top of the basin into segments of approximately uniform slope; USGS quadrangle maps are adequate for this purpose. The time of travel through the various portions of the flow path can be estimated using methods developed by SCS [SCS 1986]. Average velocity of flow through each channel reach can be estimated using the Manning equation. Appropriate flow depths can be assumed and Manning's "n" values can be estimated using Barnes [Barnes 1967]. Time of travel in each reach is calculated as the length of the reach divided by the average velocity in the reach.

For a hydrograph, the time of concentration represents the time required for a kinematic wave to travel the total length of the longest flow path for the drainage area. The celerity of a kinematic wave is approximately 1.5 times the average velocity, so the time of concentration for the basin can be computed as the sum of the travel times in all reaches divided by 1.5.

A value for R, the storage coefficient in Clark's hydrograph, can be calculated by examination of the observed flood hydrograph as illustrated in Figure 8-8.2. This value of R is not required for the unit-hydrograph determination but should be estimated for comparison with the value calculated by HEC-1 after the unit hydrograph has been optimized using the constant value of $R/(T_c + R)$.

8-8.6 Rainfall Sequence for Recorded Storms

The maximum time increment of rainfall to be used in the unit-hydrograph analysis is calculated as $T_L/4$ rounded down to an even number. This limitation will generally ensure numerical accuracy in the construction of the unit hydrograph and the flood hydrographs. T_L is lag time.

With computer program it's not a big deal to calc. this

~~Caution:~~ Sensitivity studies on the effect of this time ~~increment ($T_L/4$)~~ on computational ~~accuracy should be performed if there~~ is any indication that a shorter ~~time increment would~~ result in a higher peak or a more accurate lag time.

Temporal distribution of the basin-average rainfall must be developed for input to HEC-1. This should be done by distributing the calculated basin average rainfall in accordance with records from the nearest recording gage.

Caution: For basins where there may be more than one recording gage, it may be appropriate to subdivide the basin and use the temporal distributions for each gage as input to HEC-1 for the respective subbasin. Averaging recording gage readings is usually not appropriate and must be justified.

8-8.7 Infiltration for Unit-Hydrograph Development

The initial-abstraction and uniform-loss-rate method of simulating infiltration is recommended since it is easy to use, approximates an exponential loss function, and provides sufficient precision. The value of uniform infiltration calculated by HEC-1 for the historical floods should be used as a guide when estimating infiltration rates for PMF runoff calculations. This value can be checked against the range of permeability established for each soil type and weighted by the percentage of soil-type areas.

The values of infiltration determined by HEC-1 should be checked against those accepted for the soil types in the basin. This check will provide an indication as to whether the values determined by HEC-1, in the unit-hydrograph optimization process, are consistent with the basin characteristics. Justification of selected loss values is necessary.

Caution: Assigning values of uniform loss rate should take into account soil infiltration characteristics, depth-to-bedrock, geologic considerations (e.g. fractured bedrock), and groundwater table elevation in relation to ground surface.

8-8.8 Streamflow Sequence for Historic Floods

Available streamflow records are used to develop the sequence of streamflows to be input to HEC-1 for each historic flood to be studied.

8-8.9 Calibrate Unit Hydrographs

Unit hydrographs must be generated for each historic flood chosen for calibration. The way in which this is accomplished will depend on whether or not the basin is subdivided and the number of stream gages present in the basin.

Caution: There are several sources of error that can affect the acceptability of a unit hydrograph. A major potential source of error is the estimate of time distribution of precipitation excess. This estimate depends on the validity of the assumption of basin average rainfall, the estimated time distribution of rainfall, and selection and variability of the infiltration rate. The adopted time and/or spatial distribution of rainfall may be in error because of clock-synchronization errors, or because of an insufficient number of rain gages to allow for accurate assessment. Given the time distribution of rainfall, estimates of the precipitation excess for a given time depend on the selection of infiltration rate for that period. All these assumptions may make it difficult or impossible to develop a unit hydrograph that satisfactorily reconstitutes a major historic flood hydrograph that then may be verified by reproducing another historic flood. The hydrologic engineer needs to be alert to such problems.

8-8.9.1 Cases Where a Single Basin Unit Hydrograph is Sufficient (No Subdivision)

The rainfall input sequences, as calculated in Section 8-8.6, should be used with the corresponding streamflow sequence assembled in Section 8-8.8 and the hydrograph parameters computed in Sections 8-8.3, 8-8.4, and 8-8.5. The value $R/(T_c + R)$ is calculated from the estimated values of T_c and R or adapted on the basis of available regional values. The parameter can be fixed or allowed to vary when using HEC-1 to develop unit hydrographs. HEC-1 should be programmed to optimize all parameters [HEC 1990a, Section 5] of the hydrographs. For each calculated unit hydrograph, check the HEC-1-calculated values for T_c, R, and the uniform infiltration rate with the values estimated.

Caution: Since HEC-1 makes only a limited number of iterations in this optimization process, more than one trial may be necessary to enable the program to reach an optimum fit. The value of $R/(T_c + R)$ produced by HEC-1 should be input to subsequent runs to ensure that a best fit, in terms of HEC-1 capabilities, has been obtained.

Caution: If the values of T_c or R differ substantially from those calculated in Section 8-8.5, review the calculation of those values. Calculated values for T_c, because of its physical relevance, should be a particular guide to the final value of $R/(T_c + R)$ chosen as correct for the unit hydrograph.

If the reconstituted historic hydrographs compare well with the recorded hydrographs, no further adjustment of the unit-hydrograph parameters will be necessary. However, if the peak is too low, the hydrograph shape is poor, or the calculated values of R or T_c differ greatly from the original estimates, the input parameters should be revised and HEC-1 should be rerun to compute a new hydrograph. This process should be repeated until the fit between reconstituted and recorded hydrographs can no longer be improved.

A representative unit hydrograph should be prepared using the individual unit hydrographs developed with HEC-1. In general, the representative unit hydrograph should be based on the largest historic flood that occurred during the season of the critical PMP. The representative unit hydrograph can be obtained by adopting appropriate values (T_c and R) from calibrations as opposed to manual adjustment of unit-hydrograph ordinates.

Caution: If adjustments to the representative unit-hydrograph peak and base are made, the ordinates of the unit hydrograph will need to be adjusted to preserve a runoff volume of 1 inch.

8-8.9.2 Unit Hydrographs for Subbasins and Channel Routing

If the drainage area is to be subdivided, it will be necessary to compute runoff from each subbasin and to route and combine runoff from subbasins in the downstream direction to develop the hydrograph at the basin outlet.

- If streamflow records are available for each subbasin, the entire process of optimizing the unit hydrograph for each subbasin is the same as described in Section 8-8.9.1.

- If streamflow records are not available at the outlet of all subbasins, it will be necessary to estimate unit hydrographs for each subbasin. This can usually be done with sufficient accuracy by using SCS dimensionless, synthetic unit hydrographs, which require only an estimate of lag time for the subbasin.

- A unit hydrograph for an ungaged subbasin can be developed following the procedures described in Section 8-9.

The Muskingum-Cunge method of routing, as incorporated in HEC-1, is recommended for channel routing of outflow from each subbasin. Channel cross sections required for the routing can usually be obtained with sufficient accuracy by scaling measurements and elevations from 7½-minute USGS quadrangle maps. Manning's roughness coefficients, required as input to the routing process, must be estimated on the basis of field observations of the streams. The USGS publications "Roughness Characteristics of Natural Channels" [Barnes 1967] and Water Supply Paper No. 2339 "Guide For Selecting Roughness Coefficients for Natural Channels and Flood Plains" can be used to aid in evaluating roughness coefficients. Also, Ven Te Chow's "Open-Channel Hydraulic" textbook provides Manning's "n" value. HEC-1 includes the capability to combine hydrographs in the downstream direction. The combination of the unit hydrographs and the routing and combining of hydrographs forms a single-event runoff model for the basin.

Caution: Muskingum-Cunge uses a single (representative) cross section defined by eight coordinate points for each routing reach. The method cannot accommodate for backwater effects and should not be used when attenuation of the hydrograph is expected. An example of where this technique might be used is to translate the hydrograph from gages downstream. Where the intention is to properly model the attenuation of the hydrograph, the dynamic wave routing is the preferred method (e.g., when the river is expanding or contracting or where there is natural storage).

Calibration with the historic outflow hydrograph is accomplished differently when routing is involved, because the runoff from each subbasin must be routed and/or combined in the downstream direction to produce the total inflow hydrograph. The agreement between the recorded and reconstituted hydrograph should be examined; if differences are unacceptable, adjustments must be made to the assumed routing parameters and/or the unit hydrographs for each subbasin. The unit hydrograph for each subbasin, if the subbasin is gaged, is also calibrated by checking the accuracy with which its use reproduces the recorded historic floods.

Caution: The unit hydrograph for a subbasin, if the subbasin is gaged is calibrated by applying the historic rainfall excess to the unit hydrograph and checking for the reproducibility of recorded historic floods.

- If a regional value for $R/(T_c + R)$ is available, it can be used to estimate T_c at the outlet of each subbasin.

8-8.10 Verification Hydrographs

Once calibration of unit hydrographs has been completed, the representative unit hydrograph (or the runoff model consisting of the subbasin unit hydrographs and routing calculations) is used with the corresponding basin average rainfall in an attempt to reproduce the historic flood or floods chosen for verification. If the historic hydrographs are duplicated well, the representative unit hydrograph can be accepted. Checking between the historic hydrograph and the generated verification hydrograph can be done automatically with HEC-1 in terms of statistical differences.

- Plotting the hydrographs for visual comparison is necessary.
- For the case where a single representative unit hydrograph is involved, only adjustments to the unit-hydrograph parameters will be required.
- For subdivided basins, where the hydrograph generation involves a runoff model, adjustments to achieve better agreement with the historical flood hydrograph may involve adjustments to unit-hydrograph parameters and routing data as well.

The verification process must be continued until an acceptable fit is achieved.

Caution: It is important to be certain that any adjustments to unit hydrographs or other runoff-model parameters do not decrease the degree of fit achieved in Section 8-8.9 for the historic flood hydrographs.

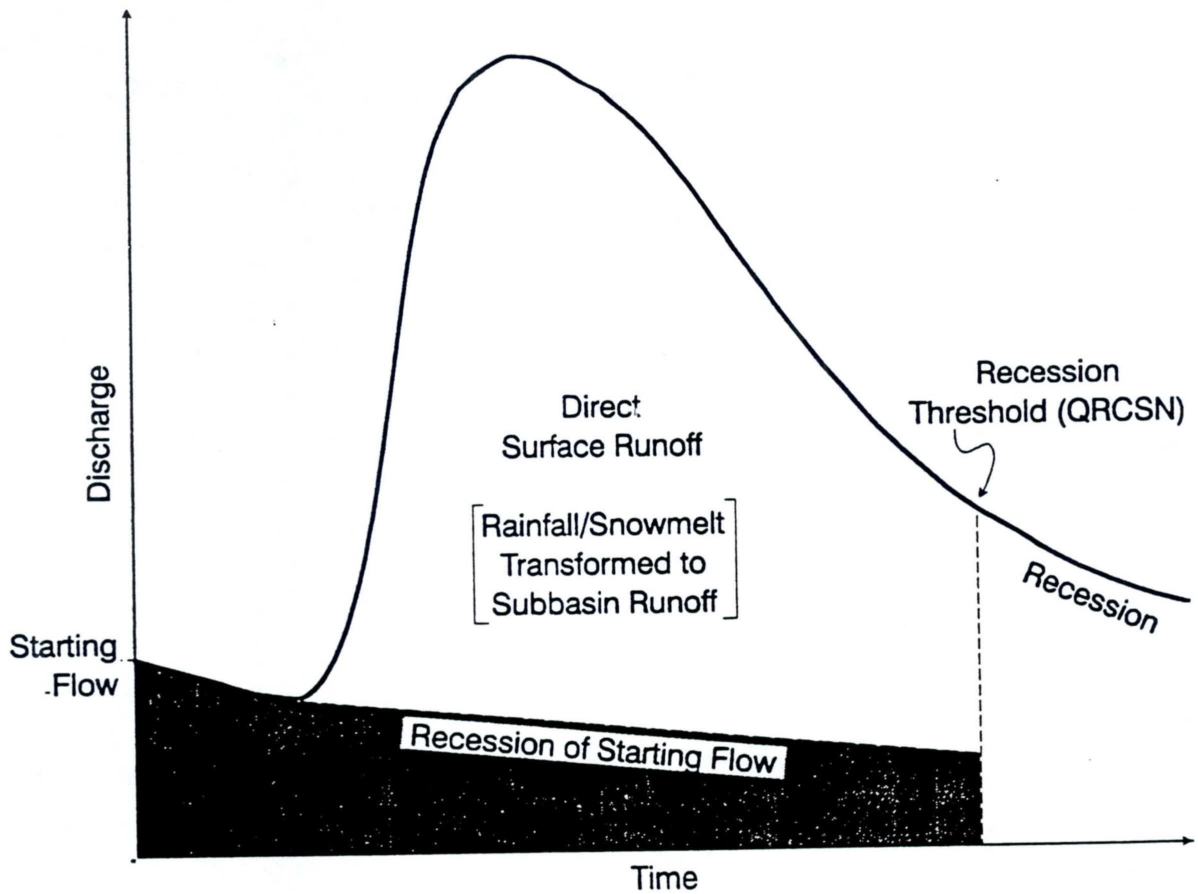

Figure 8-8.1 Baseflow Simulation in HEC-1
[HEC 1990a]

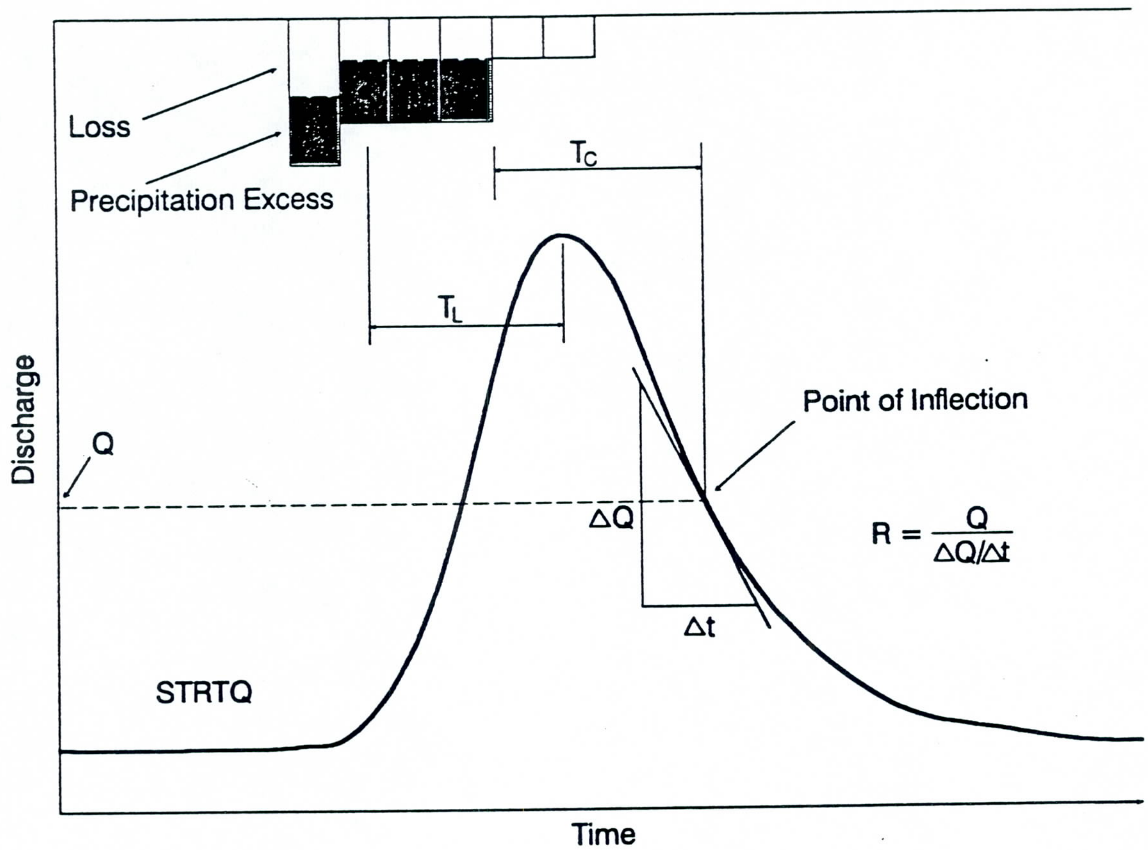

Figure 8-8.2 Estimation of Clark Unit-Hydrograph Parameters
[HEC 1982]

FLOOD HYDROGRAPH FOR GAGED WATERSHED
SABRINA EXAMPLE

Monday 3:05 p.m.

Probable Maximum Flood Studies

Example 1: Sabrina Dam

March 1994

Purpose: *To illustrate a PMF calculation for a gaged basin with "detailed method" loss rate calculations.*

Summary

Subbasin Division:	Single-basin analysis
Routing:	No channel routing, modified Puls in reservoir
Unit Hydrograph Analysis:	Calibration of Clark parameters at gage upstream of project; transfer to project
Loss Rates:	Detailed method using STATSGO data
Initial Reservoir Level:	Annual maximum normal operating level at run-of-river project
Snowpack:	Insufficient data to determine snowpack or water equivalent; assumed unlimited relative to melt temperatures
Snowmelt:	Three-day high melt temperatures retrieved from climate data
Sensitivity Analysis:	Sensitivity to transfer of unit hydrograph parameters from gaged site

Example 1
Sabrina River Probable Maximum Flood

March 1994

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Example 1
Sabrina River Probable Maximum Flood

Summary

This study was performed to estimate the Probable Maximum Flood (PMF) at the Sabrina Hydroelectric Project, Federal Energy Regulatory Commission (FERC) Project No. AAAA. The study follows the recommended procedures for a gaged basin in Chapter VIII of the FERC *Engineering Guidelines for the Evaluation of Hydropower Projects* (Reference 1). The estimated PMF inflow to the project is 5,760 cubic feet per second (cfs), and the estimated PMF outflow is 5,690 cfs. The PMF is caused by a 72-hour warm-season Probable Maximum Storm (PMS). During the PMF, the reservoir would rise to a maximum level of 994.3 feet National Geodetic Vertical Datum (NGVD).

I. Project Description

A. Project Data

The Sabrina Hydroelectric Project is located on the Sabrina River in Wolf County, Anystate. The 30-foot-high dam, constructed on a natural bedrock falls, impounds an 85-acre reservoir at normal pool elevation 992.6 feet. From left to right looking downstream, the project structures include a 46-foot-long masonry nonoverflow dam; a 125-foot-wide overflow spillway; a gate section with two 15-foot-wide by 12-foot-high tainter gates; a trash sluice; and a penstock intake structure. The powerhouse is located on the right riverbank approximately 350 feet downstream from the dam (Reference 2).

All project and basin elevations in this report are given in feet NGVD. Normal pool is maintained at elevation 992.6, one-half foot below the overflow spillway crest elevation of 993.1 feet. The project is operated as a run-of-river project. The gate operation plan during floods is to begin opening the tainter gates as needed when flow begins to overtop the overflow spillway, maintaining the pool at this level as long as possible. The annual maximum normal operating level is equal to the 993.1-foot spillway crest elevation.

The plant is visited twice daily by an operator and is monitored from the licensee's operations center in Bassett. During times of high flow, the plant is attended continuously.

B. Basin Hydrologic Data

The Sabrina River at the Sabrina Hydroelectric Project drains a 305-square-mile area. The basin is approximately 30 miles long, trending in a northwest-southeast direction. Land cover is almost entirely managed and natural forest. Approximately 25 percent of the basin is heavily forested wetland (Reference 3). Relief is moderate, with elevations ranging from 1230 feet at the upper basin divide to 993 feet at the project headwater.

A stream gage [U.S. Geological Survey (USGS) No. 44651000] is located on the Sabrina River near Baldwin City, at a drainage area of 237 square miles and approximately 17 miles upstream of the project. The period of record at the gage (hereafter called the Baldwin City gage) is 1948 to the present. In addition to the Baldwin City gage, the USGS operated stream gage No. 44658220 on the Skunk River tributary to the Sabrina River (drainage area 44 square miles) from 1978 to 1983. This gage, however, was only a crest-stage recorder and provides no data on timing of flows (Reference 4).

There are no recording rain gages in the basin. However, two hourly recording gages are located just outside the basin boundaries—the Iron River Falls and St. John gages. The periods of record at the two gages are 1948 to the present and 1953 to the present, respectively.

A basin map showing the location of the project, the Baldwin City stream gage, and the recording rain gages is included as Exhibit 1.

C. Upstream Dams

There are no dams upstream of the Sabrina Hydroelectric Project.

D. Field Visit

The hydrologist visited the project site during the independent consultant's 5-year inspection. Operators were interviewed for information on flood operations and recent storm events. In addition, the project hydrologist conducted aerial and driving surveys to verify land cover and the absence of upstream dams.

The operators interviewed were both present during a high-flow condition in the spring of 1989, during which both tainter gates were opened to a height of approximately 3 feet, and approximately 0.5 foot of water passed over the overflow spillway. The estimated discharge during this event was 1,490 cfs. Although the operators reported significant accumulation of debris along the waterline at the tainter gates, there was no apparent interference with spillway flow. The gates were opened with a traveling electric hoist, and no difficulties with gate opening were encountered.

The aerial reconnaissance of the watershed confirmed that the basin is almost entirely covered in conifer forest. Logging activities were noted at three separate locations, but the disturbed areas were limited in extent and selectively logged, with smaller trees left in place.

The driving reconnaissance included stops at three river crossings, from which general properties of the channel were observed. Typical channel width varied from 25 feet (at a drainage area of 110 square miles) to 40 feet (near the Baldwin City gage). Substrate consisted of sand to cobble-size particles. The immediate banks were heavily vegetated, and fallen trees were commonly seen in the channel.

E. Previous Studies

A *Report on Inspection* (Reference 2) for the Sabrina Hydroelectric Project was prepared in 1983 under Part 12 of the Code of Federal Regulation (FERC). The report included a determination of the PMF at the project. In the 1983 study, the PMF was estimated to be 8,800 cfs, using a Snyder unit hydrograph transferred from the nearby Pine River, the Soil Conservation Service (SCS) runoff curve number method of estimating losses, and Hydrometeorological Report No. 51 for estimating the Probable Maximum Precipitation (PMP). The Snyder unit hydrograph parameters Tp and Cp were, respectively, 46 hours and 0.55. An SCS runoff curve number of 69 was determined from statewide soil and land use maps. The 72-hour basin average rainfall derived from HMR51 was 20.2 inches, and the total estimated basin runoff from this storm was 4.4 inches. The 1983 study did not include a separate cool-season PMF estimate.

II. Watershed Model and Subdivision

A. Watershed Model Methodology

The U.S. Army Corps of Engineers (COE) "HEC-1" watershed model was used to generate the calibrated, verified, and predicted flood hydrographs.

B. Subbasin Definition

Without further subdivision, the Sabrina River basin was modeled as one basin. This was considered adequate to represent the basin hydrology, as land cover, soil distributions, and topography are homogeneous over the 305-square-mile area, and there are no upstream impoundments affecting flood peaks.

C. Channel Routing Method

Since the unit hydrograph was transferred from the gage site to the project, no channel routing was required.

III. Historic Flood Records

A. Stream Gages

The only stream gage available for unit hydrograph studies was the Baldwin City gage at a drainage area of 237 square miles. There are no other continuous-recording stream gages in the basin. The period of record at the Baldwin City gage is 1948 to the present. Stages have been recorded hourly since 1958. Prior to that time only daily average flows were recorded (Reference 4). Potential calibration events at the gage were identified through a review of USGS-published daily flows and rainfall data for the same dates. Based on this review, hourly stage data and the gage rating table current at the time of the flood were requested from the USGS. Hourly flood flows were reconstructed from these data.

B. Historic Floods

Three historic floods (including the flood of record) were found to be suitable for calibration and verification. For this study, the following criteria were used to select a flood for calibration:

- Exceptionally high flows—preferably out-of-bank—to account for nonlinearities in unit hydrograph peak (Reference 1, *Section 8-10.7.2*).

- A single-peaked hydrograph clearly related to a distinct precipitation or snowmelt event, to avoid problems in separating multipeak events.

- Uniform precipitation or snowmelt over the basin, as estimated from hourly and daily rain gage data, to ensure that the unit hydrograph represents an even distribution of runoff over the basin.

The flood of record at the Baldwin City gage occurred in May 1960, peaking at 2,570 cfs. The state Bureau of Land and Water Resources (BLWR) has developed exponential relationships for relating flood peaks for various drainage areas (Reference 5). Applying the BLWR's recommended regional exponent of 0.65 to the flood of record, the corresponding flood at the Sabrina Hydroelectric Project would be 3,030 cfs. Other large floods chosen for unit hydrograph analysis were the flood of May 1965, which peaked at 1,810 cfs at the Baldwin City gage, and the flood of June 1968, which peaked at 1,700 cfs. Observed hydrographs for these events are included in the HEC-1 data in Exhibit 2.

The historical floods used for calibration and verification are summarized in Table 1. The largest and smallest of the three were used for calibration, while the intermediate flood was reserved for verification.

TABLE 1
Floods Used for Calibration and Verification

Date of Flood	Peak Flow (cfs) Baldwin City Gage	Rainfall Associated With Flood (in.)[1]
April – May 1960	2,570 (calibration)	3.01
May 1965	1,810 (verification)	2.04
June 1968	1,700 (calibration)	3.51

1 *See Section III.C.*

Although several large floods have occurred during the snowmelt months of March and April (including the seasonal maximum of 2,100 cfs in April 1961), all were associated with an extended period of snowmelt and intermittent rain and do not lend themselves to calibration. Therefore, it was necessary to use the unit hydrograph parameters developed from warm-season floods for all seasonal conditions. This procedure is conservative, as warm-season floods typically peak more quickly than rain-on-snow floods, and the use of warm-season parameters is most likely to overestimate the cool-season hydrograph peak. Since the storage at the Sabrina Hydroelectric Project is negligible, peak discharge, rather than volume, is critical in evaluating spillway capacity.

Based on a log Pearson analysis of the Baldwin City gage record, the 100-year flood at the gage is 2,350 cfs. The corresponding 100-year flood estimate at the project would be 2,770 cfs, based on the BLWR equations.

C. Precipitation Associated with Historic Floods

The basin average precipitation producing the April–May 1960, May 1965, and June 1968 floods was estimated by averaging rainfalls recorded at the Iron River Falls and St. John rain gages. Basin average total precipitation associated with the three floods was 3.01, 2.04, and 3.51 inches, respectively. Hourly precipitation data were estimated by distributing the basin average rainfall in accordance with the hourly record at Iron River Falls for the 1960 and 1968 events. Because it is slightly closer to the basin centroid, Iron River Falls was chosen over St. John for the hourly precipitation distribution. For the 1965 event, the Iron River Falls data were found to be questionable, lagging other gages by several hours. Therefore, the St. John hourly data were used instead. Hourly precipitation data are included in the HEC-1 model input files presented in Exhibit 2.

Hourly precipitation for these events was discontinuous, with several peaks separated by as much as 24 hours. Although this is not optimal for unit hydrograph calibration, the storms were used because:

- Each storm had one maximum rainfall peak that could be assumed to produce the majority of runoff.

- The resulting floods were single-peaked and the best available for calibration.

D. Snowpack and Snowmelt During Historic Floods

As stated above, several long, multiple-peaked floods occurred during snowmelt periods. However, due to the extended periods of rain, snowmelt, and high baseflow causing these floods, they were not suitable for unit hydrograph parameter calibration. The floods used to calibrate unit hydrograph parameters were caused entirely by rain, and no estimate of snowmelt contributions is required. A discussion of snowpack historically occurring during snowmelt flood months is presented in *Section VIII.C*.

IV. Unit Hydrograph Development

A. Discussion of Approach and Tasks

The Clark unit hydrograph method was used for this analysis. Clark parameters were calibrated at the Baldwin City gage and transferred to the project site. The time-area curve required in the Clark method was calculated from topographic maps, assuming an equal rate of travel throughout the channel network. The Sabrina basin is considered "gaged" under the FERC PMF guidelines (Reference 1, *Section 8-7.1*), because existing hydrologic data from within the basin are adequate to calibrate and verify unit hydrograph parameters. Exhibit 2 shows HEC-1 model runs used to calibrate and verify the unit hydrograph parameters, including precipitation data, observed hydrographs, and calibration statistics.

B. Baseflow Separation

The HEC-1 model's required baseflow parameters STRTQ, QRSCN, and RTIOR were determined by inspection of the plotted hydrographs, as shown in Exhibit 3. STRTQ was taken as the observed flow at the beginning of runoff simulation. Preliminary estimates of QRCSN and RTIOR were obtained by plotting the flow hydrograph on a semi-logarithmic scale and identifying the point of deviation from a straight line (QRCSN) and the slope of the line (RTIOR). These estimates were somewhat subjective, as the entire falling limb of the observed hydrographs tended to be fairly straight. This may be a result of discontinuous rainfall during the calibration storms. These estimates of QRCSN and RTIOR were then readjusted during calibration to obtain a fit with the recession limbs of the observed hydrographs. Final baseflow parameters for the calibration floods are shown in Table 2.

TABLE 2
Baseflow Parameters for Calibration and Verification Floods
Baldwin City Gage

Date of Flood	STRTQ[1]	RTIOR[2]	QRCSN[3]
April - May 1960	1020	1.008	1050
May 1965	720	1.008	1250
June 1968	555	1.008	1250

[1] *Starting flow (cfs)*
[2] *Baseflow recession constant*
[3] *Flow at which baseflow recession begins*

C. Preliminary Estimates of Clark Parameters

A preliminary estimate of time of concentration (T_c) was made for each hydrograph by estimating the time between the end of effective rainfall and the end of direct runoff (indicated by the inflection point in the observed hydrograph's recession limb), which ranged from 54 to 68 hours for the three floods analyzed. The estimates are presented graphically in Exhibit 4. These T_c values (54, 60, and 68 hours for the 1960, 1965, and 1968 floods, respectively) were used as starting values in the calibration and verification runs of the HEC-1 model.

As a second predictor of T_c, travel time along the maximum channel length (to the Baldwin City gage) of 44 miles was estimated assuming a 1.9-foot/second flow velocity, giving a T_c of 35 hours. The flow velocity was estimated from Manning's equation using a generalized 30-foot-wide by 3-foot-deep rectangular section, an average map slope of 4.1 feet/mile (0.0008 ft/ft), and estimated Manning's *n* of 0.04. The typical section and Manning's *n* were estimated based on field observations.

A preliminary estimate of R was made for the floods of 1960, 1965, and 1968 using the method described in *Figure 8-8.2* of Reference 1. The estimated R values ranged from 94 to 144 (see Exhibit 4).

D. Estimate of Infiltration During Historic Floods

The HEC-1 model was allowed to optimize infiltration rate for the two calibration floods, using the initial and constant loss rate function. For both events, optimized initial and constant loss rates were very low—for the 1968 storm, 0.54 inches and 0.04 inches per hour, respectively, and for the 1960 storm, 0.04 inches and 0.01 inches per hour, respectively. For the verification storm (May 1965), initial and constant loss rates were 0.02 and 0.0 inches per hour, respectively. These rates, however, reflect the optimized *average* loss rate during each 3-hour computational time step and may not represent the actual infiltration rate during peak rainfall. Additionally, for the calibration floods the rainfall distribution is known only from two gage sites, neither of which is in the basin. Therefore, for computation of the PMF, a loss function based on saturated soil infiltration rates, rather than on calibration flood values, was adopted (see *Section VII*).

E. Subbasin Unit Hydrograph Parameters

Clark unit hydrograph parameters T_c and R were estimated from the 1960 and 1968 flood events. The time-area curve required in the Clark method was derived by measurement of main channel and tributary lengths, assuming equal rates of travel through the main channel and tributaries (Reference 6). Initial optimization runs for the two storms resulted in the optimized parameters shown in Table 3.

TABLE 3
Optimized Clark Parameters

Storm Starting Date	Optimized T_c	Optimized R
April 27, 1960	38	117
June 26, 1968	43	127

Subsequent runs of the HEC-1 model were made to determine final calibration values that produced a good fit for both calibration floods. Based on these analyses, the basin at the Baldwin City gage was assigned a T_c of 40 hours and R of 125. No adjustment for nonlinearity was made to the peak of the unit hydrograph, because both calibration floods were clearly overbank events, with the larger of the two being greater than the 100-year flood.

Final calibration runs are included in Exhibit 2. The HEC-1 model output includes precipitation, optimized loss rate, and predicted and observed hydrographs.

V. Unit Hydrograph Verification

The May 1965 storm was used to verify the Clark parameters estimated above. The HEC-1 model was again allowed to optimize loss rates but used fixed Clark T_c and R of 40 hours and 125, respectively. The observed and predicted hydrographs at the Baldwin City gage are shown in the HEC-1 printout in Exhibit 2, as are the statistics used to evaluate the fit between the two hydrographs. Based on these statistics (peak flow underestimated by 3 percent; lag overestimated by 0.4 percent), the calibrated Clark parameters were accepted for use in predicting the PMF. Table 4 summarizes the unit hydrograph parameter calibration and verification.

The unit hydrograph parameters were then transferred downstream to the Sabrina Dam site. Time of concentration (T_c) was determined at the dam site by estimating travel time from the gage to the dam site. A 1.9-foot/second velocity was again assumed, as this had proved to be a reasonable estimator for T_c at the Baldwin City gage. This assumption yielded a T_c of 55 hours at the dam site. The storage coefficient R at the dam was estimated by assuming constant $R/(T_c + R)$, giving a value of 172.

TABLE 10
Model Sensitivity to Incremental T_c
Between Calibration Gage and Project

Incremental T_c (hrs.)	Total T_c (hrs.)	Warm-Season Synoptic Storm PMF Peak Inflow (cfs)
7.5	47.5	5,870
15	55	5,760
22.5	62.5	5,630

A 50 percent change in the incremental T_c between the gage and the project site corresponds to a 14 percent change in the total T_c. This change, in turn, results in only a 1 to 2 percent change in the PMF peak inflow. Therefore, the calculated PMF peak is not sensitive to the assumptions used in transferring T_c to the project site.

Loss Rates. Loss rates were estimated conservatively by assuming no initial loss and assigning all wetland areas a loss rate of zero. All other areas were assigned the saturated permeability of their least permeable layer, which is equivalent to assuming soils are saturated before the PMS begins. No sensitivity analysis was performed.

Snowmelt. Snowmelt was again estimated conservatively by assuming an unlimited snowpack for the rainfall runoff period.

Precipitation. The worst-case precipitation pattern was determined within the modified HMR52 computer model. This model centers the storm over the basin centroid and evaluates various storm sizes and orientations to identify the condition that produces the greatest basin average precipitation. Since the basin was not subdivided, the maximum average rainfall will produce the greatest predicted runoff for each type of storm analyzed.

C. Reservoir Outflow PMF

The inflow hydrograph was routed through the project reservoir using the Modified Puls routing option in the HEC-1 model. The reservoir storage is very small and proved to be negligible, with reservoir outflows essentially equal to inflows. The outflow flood peaks are shown in Table 9.

TABLE 4
Summary of Calibration/Verification Data

Storm Starting Date	April 27, 1960	May 5, 1965	June 26, 1968
Calibration or Verification	Calibration	Verification	Calibration
Historical Data			
Total precipitation depth (in.)	3.01	2.04	3.51
Total runoff depth (in.)	2.35	2.03	1.80
Peak flow (cfs)	2,570	1,810	1,700
Preliminary estimate of T_c (hrs.)	45	52	62
Optimized Parameters			
T_c	38	40	43
R	117	125	127
RTIOR	1.008	1.008	1.008
QRCSN	1,050	1,250	1,250
STRTQ	1,020	720	555
Initial loss rate	0.04	0.02	0.54
Constant loss rate	0.01	0.0	0.04

VI. Probable Maximum Precipitation

A. Probable Maximum Precipitation Data

Probable Maximum Precipitation data were obtained from a regional PMP study completed in 1992 (Reference 7). Points selected from depth-area-duration maps in that report were plotted to show depth as a function of area and duration and smoothed to ensure consistency with PMP logic. A cool-season storm and two warm-season storms (24-hour mesoscale convective and 72-hour synoptic) were considered, as directed in the PMP study report. Also, synoptic-storm depths for 6-hour durations and 100 through 1,000 square miles were reduced by 10 percent as directed in the report. Depth-area-duration values, in inches, for the three types of storms analyzed are shown in Table 5. The original and smoothed data are plotted in Exhibit 5.

TABLE 5
Probable Maximum Precipitation Data (inches)
Sabrina River Basin

Warm-Season – Mesoscale Convective Storm					
Area (mi^2)	6-hour	12-hour	24-hour	48-hour	72-hour
100	15.2	16.8	17.8		
200	14.0	15.5	16.5		
500	12.4	13.9	14.9		
1,000	11.4	12.7	13.7		
5,000					
10,000					
Warm-Season – Synoptic Storm					
Area (mi^2)	6-hour	12-hour	24-hour	48-hour	72-hour
100	13.7	16.8	17.8	19.8	21.2
200	12.5	15.6	16.6	18.6	20.0
500	11.2	14.2	15.2	17.2	18.6
1,000	10.3	12.7	13.7	15.7	17.1
5,000	6.8	9.2	10.2	12.2	13.6
10,000	5.7	8.1	9.1	11.1	12.3
Cool-Season Storm					
Area (mi^2)	6-hour	12-hour	24-hour	48-hour	72-hour
100	5.3	6.8	7.8	9.0	10.4
200	5.0	6.5	7.5	8.5	9.9
500	4.7	6.0	7.0	8.0	9.3
1,000	4.1	5.4	6.4	7.4	8.7
5,000	3.4	4.7	5.7	6.7	8.0
10,000	2.5	3.8	4.8	5.8	7.1

B. Candidate Storms for the PMF

Because the Sabrina River basin was modeled with no further subbasin division, only one storm centering (over the entire basin) was considered. A modified version of the COE HMR52 program, adapted to specific considerations in the regional PMP study, was used to evaluate possible storm sizes and orientations. For each storm type and season analyzed, the storm size and orientation producing the greatest basin average precipitation depth was selected. Table 6 shows the PMS sequences adopted for HEC-1 modeling.

TABLE 6
Probable Maximum Storms
Mesoscale, Synoptic, and Cool-Season

6-hour increment	6-hour Basin Average Precipitation Depth (in.)		
	Mesoscale System	Synoptic Storm	Cool-Season
1		.20	.16
2		.25	.20
3		.32	.25
4		.44	.34
5	.69	.75	.52
6	1.16	2.50	1.09
7	13.02	11.66	4.80
8	.86	1.14	.71
9		.56	.41
10		.37	.29
11		.28	.22
12		.22	.18

The precipitation rates shown in 6-hour increments in Table 6 were actually modeled in 1-hour increments, as discussed in the following section. The peak 1-hour precipitation for the mesoscale, synoptic, and cool-season storms were 4.72, 4.53, and 1.85 inches, respectively. Isohyetal patterns for each storm are mapped in Exhibit 6, and the PMP program output is shown in Exhibit 7.

VII. Loss Rates

A. Discussion of Loss Rate Methodology

The detailed method of estimating loss rates (as defined in Reference 1, *Section 8-10.3*) was adopted for this study, using data from the STATSGO soils database for the state. The detailed method includes evaluating minimum seasonal infiltration rates for each soil type occurring in the basin, adjusting for land cover, and summing hourly runoff from each soil unit in the basin. This method conservatively assumes that soils are saturated at the beginning of the rainfall period, and any other initial losses (such as depression storage or leaf interception) are already satisfied. The method also assumes that infiltration occurs at a rate equal to the saturated permeability of the least permeable layer in the soil horizon, which is defined in the STATSGO database down to 60 inches in depth.

The basin boundaries were superimposed on the STATSGO map using geographic information system (GIS) software, and the area of the basin occupied by each soil unit was determined within the program. Soils having the same minimum permeability were aggregated, and the hourly rainfall excess from each soil class was determined by subtracting infiltration rate from hourly precipitation. These hourly excesses were then weighted by the percentage of the basin area occupied by each infiltration class and summed to give a basinwide average runoff for each hour of the PMS. Using this method, rather than a lumped average loss rate for the entire basin, produces some runoff for every increment of precipitation—no matter how small—because a fraction of the basin is impervious.

B. Warm-Season

Each soil unit was assigned an infiltration rate equal to the saturated permeability of the least permeable layer in the unit, based on the STATSGO data. In addition, all wetlands and open water (accounting for 25 percent of the basin area) were assumed to be completely impervious. The assumption that all wetlands are impervious implies a condition where previous precipitation or snowmelt has caused the water table to reach seasonal high levels, and wetlands are thus hydrologically equivalent to standing water. Initial losses were assumed to be zero, also reflecting wet antecedent conditions. A map showing the distribution of soil units, with associated infiltration rates, is included as Exhibit 8. Table 7 shows infiltration rate versus basin area percent.

TABLE 7
Distribution of Soil Infiltration Rates
Warm and Cool Seasons

Infiltration Rate (in./hr.)	% of Basin	
	Warm-Season	Cool-Season
0.00	25.3	32.2
0.03	1.8	1.6
0.04	0.03	0.03
0.13	0.03	0.03
0.40	0.4	0.3
1.10	0.7	0.7
1.30	5.6	4.9
3.10	18.5	16.0
3.30	13.5	11.7
4.00	12.0	10.4
13.00	22.0	22.0

To estimate the runoff hydrograph from the watershed, hourly increments of rainfall were applied to each soil unit separately to determine that unit's hourly contribution to the runoff hydrograph. The hourly rates of runoff were then weighted by the area of the basin occupied by each soil unit and added together to produce the hourly amount of runoff generated by the basin as a whole.

Table 8 shows the basin average runoff depth for each 6-hour increment of the PMS. This sequence was then entered in the basin HEC-1 model (with loss rates set to zero, as losses were already subtracted from precipitation). The calculations to determine rainfall excess are shown in Exhibit 9.

The estimated average depth of runoff was 5.22 inches for the 24-hour mesoscale convective storm and 5.84 inches for the synoptic storm.

TABLE 8
Probable Maximum Storm Runoff Sequence

6-hour increment	6-hour Basin Average Runoff Depth (inches)		
	Mesoscale System	Synoptic Storm	Cool-Season[1]
1		0.05	0.17
2		0.07	0.18
3		0.09	0.20
4		0.13	0.24
5	0.20	0.25	0.33
6	0.48	0.83	0.58
7	4.36	3.81	1.73
8	0.19	0.27	0.35
9		0.14	0.26
10		0.09	0.22
11		0.07	0.20
12		0.05	0.16

[1] *Includes runoff from snowmelt.*

C. Cool-Season

The method for calculating cool-season runoff was similar to that used for the warm season. However, for the cool season, precipitation was combined with estimated snowmelt before subtracting the adjusted infiltration rate for each soil type. Cool-season infiltration rates were estimated by assuming that all soils with the potential to be imperviously frozen were, in fact, completely impervious. The maximum assumed frost depth of 24 inches was derived from regional studies (Reference 8). For this study, frozen soils were identified by the following criteria, based on the FERC PMF guidelines (Reference 1, *Section 8-8.3.2*):

- All soils except sands, loamy sands, and heavily forested soils.

- All wetland soils, regardless of forestation or unfrozen infiltration rate.

A criterion of 6 inches/hour permeability for freezing soils was used to separate sands and loamy sands from other, potentially more freezable soils. This limit was selected based on soils classifications accompanying the STATSGO database.

Wetland and forest areas were identified using digitized land use maps developed by the Bassett Area Planners Office (Reference 3). Only 10 percent of the basin is nonforested, while 25 percent is wetland and open water. All wetland areas are also forested. Using the criteria stated above, approximately 32 percent of the basin was assumed to be imperviously frozen, as shown in Table 7. The resulting total depth of runoff from the basin was 4.62 inches, including runoff from snowmelt. (Melt rate estimates are discussed in *Section VIII.*) Cool-season runoff calculations are shown in Exhibit 9. The time distribution of this runoff, in 6-hour increments, is shown in Table 8.

VIII. Coincident Hydrometeorological and Hydrological Conditions for the Probable Maximum Flood

A. Reservoir Level

The starting reservoir level was assumeo to be at the annual maximum normal operating level of 993.1 feet (the overflow spillway crest). There is no seasonal headwater fluctuation at this project, and the crest elevation of the uncontrolled spillway limits the stage to which the pool can rise under all but the most extreme flood conditions.

B. Baseflow

Starting baseflow was set equal to the highest seasonal average monthly flows at the Baldwin City gage, which were June flows for the warm-season condition and April flows for the cool-season condition (Reference 5). The Baldwin City gage monthly flows were increased by the ratio of drainage area at the project to that at the gage. For baseflows, a straight drainage area ratio was used, rather than the exponential ratio used for flood peaks. This resulted in an assumed cool-season baseflow of 900 cfs and warm-season baseflow of 600 cfs.

C. Snowpack

The critical cool-season event was assumed to occur in March or April, as these months have the highest probability of combining snowpack and frozen ground with melting temperatures and extreme precipitation. Therefore, both March and April data were searched to identify critical cool-season conditions.

Snowpack was recorded at the St. John station only from 1968 to 1984, and the record is incomplete. However, a review of the data indicated maximum March snowpacks of 20 to 30 inches (water equivalent not recorded) and maximum April snowpacks of 12 to 20 inches. A preliminary estimate of melt was made by assuming three 60-degree days (an approximate seasonal high for April) and a forest-area melt coefficient of 0.05 inches/degree day above 84°F (Reference 9), resulting in a total melt of 3.9 inches. Conservatively assuming a water equivalent of 30 percent (Reference 10) indicates that any snowpack greater than approximately 13 inches will be unlimiting, and the total melt will be controlled by temperature rather than snowpack availability.

D. Snowmelt

The daily temperature sequence assumed to coincide with the PMP was determined by inspection of daily climate data at the St. John station. Temperature and climate data were searched to identify the warmest three-day temperature sequence occurring coincidentally with rain in March or April. Due to the incompleteness of the snow data, it was not possible to determine which sequences also coincided with snow on the ground. The maximum March/April three-day temperature sequence for the period of record was as follows: 59°, 64.5°, and 62.5°F. Since it is probable that these temperatures did not coincide with snow on the ground, this sequence is expected to yield a conservative snowmelt estimate. The sequence was positioned in the PMF watershed model so that the warmest day coincided with the peak PMS rainfall.

Snowmelt was modeled using the degree-day method. Snowmelt was estimated outside of the HEC-1 model simultaneously with the cool-season runoff estimation procedure. A melt coefficient of 0.05 inches/day/degree F was assumed. This is the recommended melt coefficient to be used with daily average temperature data for forested regions (Reference 9). This sequence resulted in a total (water equivalent) snowmelt depth of 4.2 inches. Again assuming a 30 percent water equivalent, this would require a 14-inch snow depth before the beginning of the melt period. As this is within the range of historical snow depths for March and April, the assumption of unlimiting snowpack is reasonable.

The application of the melt equation to unlimiting snowpack yielded a daily melt rate, which was added to the cool-season rainfall sequence and applied in hourly increments to the watershed runoff spreadsheet as precipitation. In this way, the total computed runoff was the excess of the combined precipitation and snowmelt. Runoff calculations are shown in Exhibit 9.

IX. PMF Hydrographs

Table 9 summarizes the HEC-1 input and output data used to estimate the PMF for each of the three storm conditions.

TABLE 9
Summary of HEC-1 Input and Output Data

Parameter	Warm-Season Mesoscale Convective Storm	Warm-Season Synoptic Storm	Cool-Season Storm
Total rain depth[1] (in.)	18.81	18.69	9.17
Total snowmelt depth[2] (in.)	n/a	n/a	4.2
Total runoff depth[2] (in.)	5.25	5.84	4.63
Clark T_c	55	55	55
Clark R	173	173	173
Starting baseflow STRTQ	600	600	900
Baseflow recession constant QRCSN	1.008	1.008	1.008
Inflow hydrograph peak (cfs)	5390	5760	4470
Outflow hydrograph peak (cfs)	5340	5690	4470
Peak stage at project	993.9	994.3	993.2

1 *Shown in PMS calculations (Exhibit 7)*
2 *Shown in RUNOFF calculations (Exhibit 9)*

PMF hydrographs for both the warm- and cool-season conditions were computed by the above methods. Because of the small storage in the reservoir and the run-of-river project operation, peak flow rather than volume would produce the critical PMF at the Sabrina project. PMF inflow and outflow hydrographs are plotted in Exhibit 11.

A. Inflow PMF Hydrograph

The PMF inflow hydrograph peak for each condition is shown in Table 9. The warm-season synoptic condition produces the largest inflow peak, which is 5,760 cfs.

B. Sensitivity Analysis

Note: Sensitivity analyses should be conducted on parameters that are very uncertain, unverifiable, or not conservatively estimated.

Unit Hydrograph Parameters. The unit hydrograph parameters were derived from some of the largest floods on record (including the flood of record), and calibration achieved close agreement between simulated and observed hydrographs. All three floods used in calibration and verification are significantly greater than the 2-year flood of 1,600 cfs, and, based on the USGS gage rating, would overtop the riverbanks. The April to May 1960 flood used for calibration was the flood of record and is greater than the 100-year flood. The May 1965 flood used for verification is approximately equal to the 10-year flood at the gage site.

However, since the Baldwin City gage used for calibration is not at the project and the method used to transfer the calibrated unit hydrograph parameters could not be verified, the unit hydrograph parameters at the project could be in error. Therefore, an analysis was conducted to evaluate the sensitivity of the estimated PMF peak to the assumptions used to transfer the unit hydrograph parameters. The incremental T_c between the Baldwin City gage and the project (15 hours) was varied by ± 50 percent. The effects of these changes on the final PMF peak are shown in Table 10. Sensitivity analysis runs are shown in Exhibit 10.

TABLE 10
Model Sensitivity to Incremental T_c
Between Calibration Gage and Project

Incremental T_c (hrs.)	Total T_c (hrs.)	Warm-Season Synoptic Storm PMF Peak Inflow (cfs)
7.5	47.5	5,870
15	55	5,760
22.5	62.5	5,630

A 50 percent change in the incremental T_c between the gage and the project site corresponds to a 14 percent change in the total T_c. This change, in turn, results in only a 1 to 2 percent change in the PMF peak inflow. Therefore, the calculated PMF peak is not sensitive to the assumptions used in transferring T_c to the project site.

Loss Rates. Loss rates were estimated conservatively by assuming no initial loss and assigning all wetland areas a loss rate of zero. All other areas were assigned the saturated permeability of their least permeable layer, which is equivalent to assuming soils are saturated before the PMS begins. No sensitivity analysis was performed.

Snowmelt. Snowmelt was again estimated conservatively by assuming an unlimited snowpack for the rainfall runoff period.

Precipitation. The worst-case precipitation pattern was determined within the modified HMR52 computer model. This model centers the storm over the basin centroid and evaluates various storm sizes and orientations to identify the condition that produces the greatest basin average precipitation. Since the basin was not subdivided, the maximum average rainfall will produce the greatest predicted runoff for each type of storm analyzed.

C. Reservoir Outflow PMF

The inflow hydrograph was routed through the project reservoir using the Modified Puls routing option in the HEC-1 model. The reservoir storage is very small and proved to be negligible, with reservoir outflows essentially equal to inflows. The outflow flood peaks are shown in Table 9.

Based on these analyses, the PMF is caused by the warm-season synoptic storm. The PMF inflow peak is equal to 5,760 cfs, and the PMF outflow peak is equal to 5,690 cfs. With the tainter gates fully open, the PMF can be passed with the pool rising to elevation 994.3. The peak pool level is 1.2 feet above the crest of the overflow spillway (which is equal to the annual normal maximum operating level).

PMF hydrograph calculations are shown in Exhibit 10. The stage and outflow hydrographs are plotted in Exhibit 11. The inflow hydrograph is not plotted, because it is virtually identical to the outflow hydrograph.

References

1. Federal Energy Regulatory Commission, 1993, *Engineering Guidelines for the Evaluation of Hydropower Projects*. Chapter VIII, "Determination of the Probable Maximum Flood."

2. WaterWays Engineering, Inc., 1983. Report on Inspection, Sabrina River Hydroelectric Project.

3. Basset Area Planners Office, 1988. Land Cover Maps for Eastern Wolf County.

4. U.S. Geological Survey Water Data Report, 1983. Water Resources Data, Anystate, Water Year 1983, Vol. 1.

5. Anystate Bureau of Land and Water Resources, 1989. Methods for Estimating Flood Peaks at Ungaged Sites in Anystate. Technical Bulletin 890028.

6. U.S. Army Corps of Engineers, Hydrologic Engineering Center, 1982. Hydrologic Analysis of Ungaged Watersheds Using HEC-1, Training Document No. 15.

7. National Weather Service, 1992. Probable Maximum Precipitation Study for Anystate Region.

8. Anystate Office of Climatology, 1988. Technical Report 88-0022. Snow and Frost Depth Data for Experimental Farms in Anystate.

9. Chow, V. T., 1964. *Handbook of Applied Hydrology.* McGraw-Hill, New York.

10. Gray, Don M. and Terry D. Prowse, 1992. "Snow and Floating Ice," Chapter 7 in *Handbook of Hydrology*, David Maidment, Ed., McGraw-Hill Publishers.

EXHIBIT 1

Basin Map

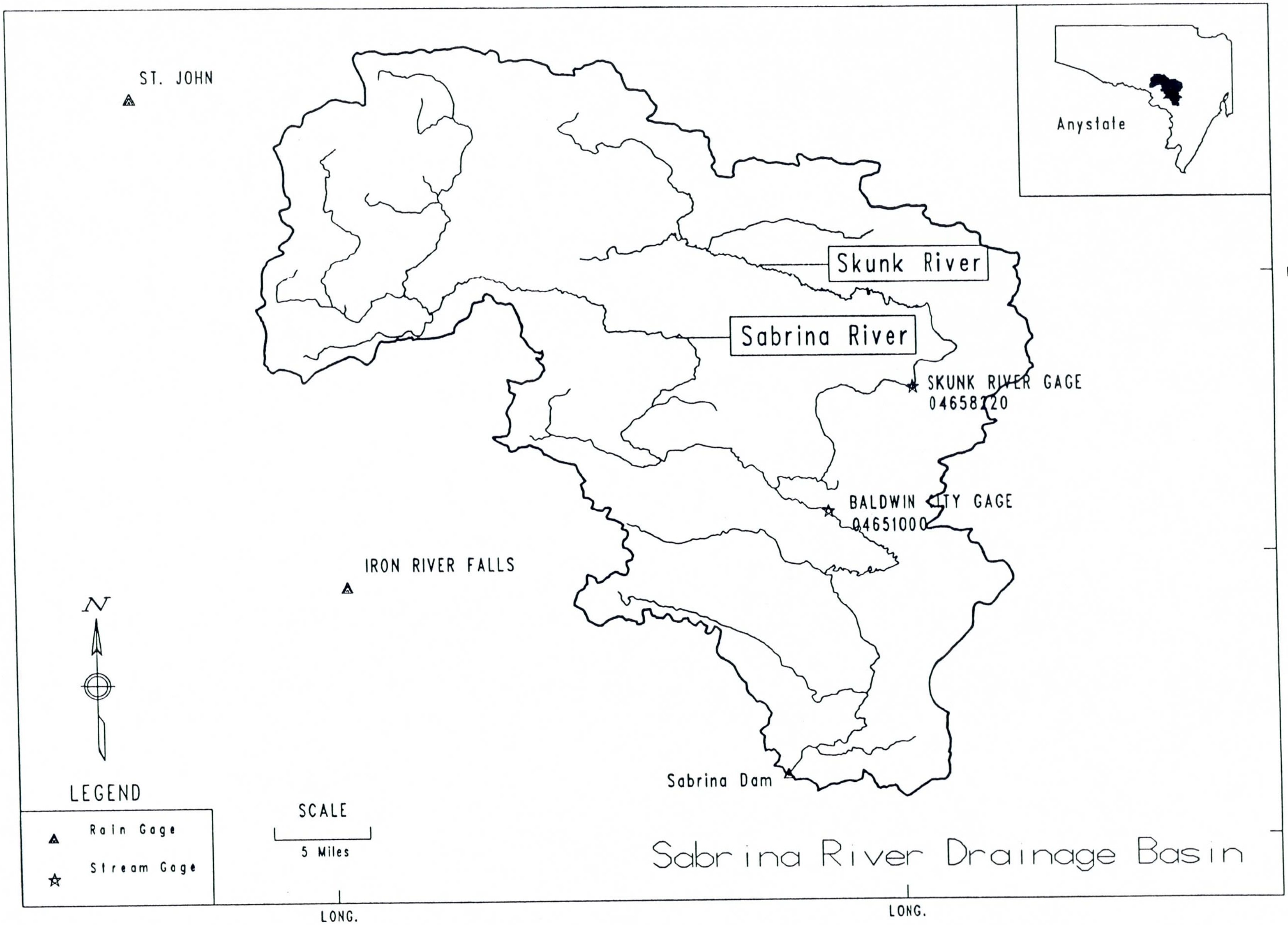
ST. JOHN
Anystate
Skunk River
Sabrina River
SKUNK RIVER GAGE
04658220
BALDWIN CITY GAGE
04651000
IRON RIVER FALLS
N
Sabrina Dam
LEGEND
Rain Gage
Stream Gage
SCALE
5 Miles
LAT.
LAT.
LAT.
LONG.
LONG.
EXHIBIT 1
Sabrina River Drainage Basin

EXHIBIT 2

Unit Hydrograph Calibrations
HEC-1 Output

Note: Complete HEC-1 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

UNIT HYDROGRAPH CALIBRATION

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*           VERSION 4.0.1 (LOCAL)       *
*                                       *
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*                                       *
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```

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*    HYDROLOGIC ENGINEERING CENTER      *
*          609 SECOND STREET            *
*       DAVIS, CALIFORNIA 95616         *
*           (916) 551-1748              *
*                                       *
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X     X  X        X     X       XX
X     X  X        X              X
XXXXXXX  XXXX     X      XXXXX   X
X     X  X        X              X
X     X  X        X     X        X
X     X  XXXXXXX   XXXXX        XXX
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THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM

1

HEC-1 INPUT


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  1         ID   SABRINA DAM - PROJECT # W24-92K
  2         ID   HYDROGRAPH SIMULATION, STORM OF JUNE, 1968
  3         ID   CALIBRATION OF WATERSHED MODEL
  4         ID   BALDWIN CITY GAGE
  5         ID   RUNOFF DATA READ FROM THE GAGE DATA
  6         ID   RAINFALL DATA FROM NCDC
  7         ID   CLARK'S UNIT HYDROGRAPH DATA OPTIMIZED
            *DIAGRAM
*** FREE ***
*** LIST ***
  8         IT     180 26JUN68    0000     110
  9         IO       0       2
 10         OU
 11         IN      60 25JUN68    0300
 12         PG    IRON
 13         PI     .02     .00     .01     .01     .01     .01     .00     .00     .00     .00
 14         PI     .00     .00     .00     .02     .03     .00     .00     .03     .00     .00
 15         PI     .00     .07     .08     .10     .06     .03     .04     .03     .01     .02
 16         PI     .00     .01     .07     .10     .20     .07     .02     .01     .21     .09
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 24         PI     .00     .00     .00     .00     .00     .00     .00     .00     .00     .00
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 28          PG  STJOHN
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 32          PI     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
 33          PI     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
 34          PI     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
 35          PI     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
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 39          PI     .00      .00      .00      .00      .00      .00      .00      .00      .00      .00
 40          PI     .04      .49

 41          KK BALDWINUSGS GAGE SITE
 42          KM   CLARK'S UNIT HYDROGRAPH PARAMETERS TO BE OPTIMIZED
 43          BA    237.
 44          PR    IRON   STJOHN
 45          PW      1.       00
 46          PT    IRON   STJOHN
 47          PW      .5       .5
 48          BF    555.    1250.    1.008
 49          UC     40.     125.
 50          UA       0     4.97    17.15    36.01    45.12    56.10    78.92    94.75
 51          LU    -.54     -.04
1                                    HEC-1 INPUT                                         PAGE  2

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 55          QO     815      840      845      851      873      870      852      832      810      793
 56          QO     780      750      731      715      704      707      759      795      820      906
 57          QO     970     1030     1083     1161     1215     1292     1405     1506     1602     1640
 58          QO    1700     1690     1680     1675     1665     1640     1601     1570     1526     1493
 59          QO    1400     1380     1340     1320     1250     1220     1200     1191     1185     1159
 60          QO    1131     1104     1075     1049     1024      999      974      950      941      917
 61          QO     882      860      840      822      810      789      765      741      722      710
 62          QO     695      670      661      638      624      607      592      582      569      549
 63          QO     538      529      510      497      487      474      463      452      441      435
 64          QO     425      411      401      392
 65          ZZ
1
         SCHEMATIC DIAGRAM OF STREAM NETWORK
INPUT
 LINE     (V) ROUTING          (--->) DIVERSION OR PUMP FLOW

  NO.     (.) CONNECTOR        (<---) RETURN OF DIVERTED OR PUMPED FLOW

  41     BALDWIN

(***) RUNOFF ALSO COMPUTED AT THIS LOCATION
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*         VERSION 4.0.1 (LOCAL)           *
*                                         *
* RUN DATE  09/24/93  TIME  11:32:54      *
*                                         *
*******************************************

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*                                         *
*     U.S. ARMY CORPS OF ENGINEERS        *
*    HYDROLOGIC ENGINEERING CENTER        *
*          609 SECOND STREET              *
*       DAVIS, CALIFORNIA 95616           *
*           (916) 551-1748                *
*                                         *
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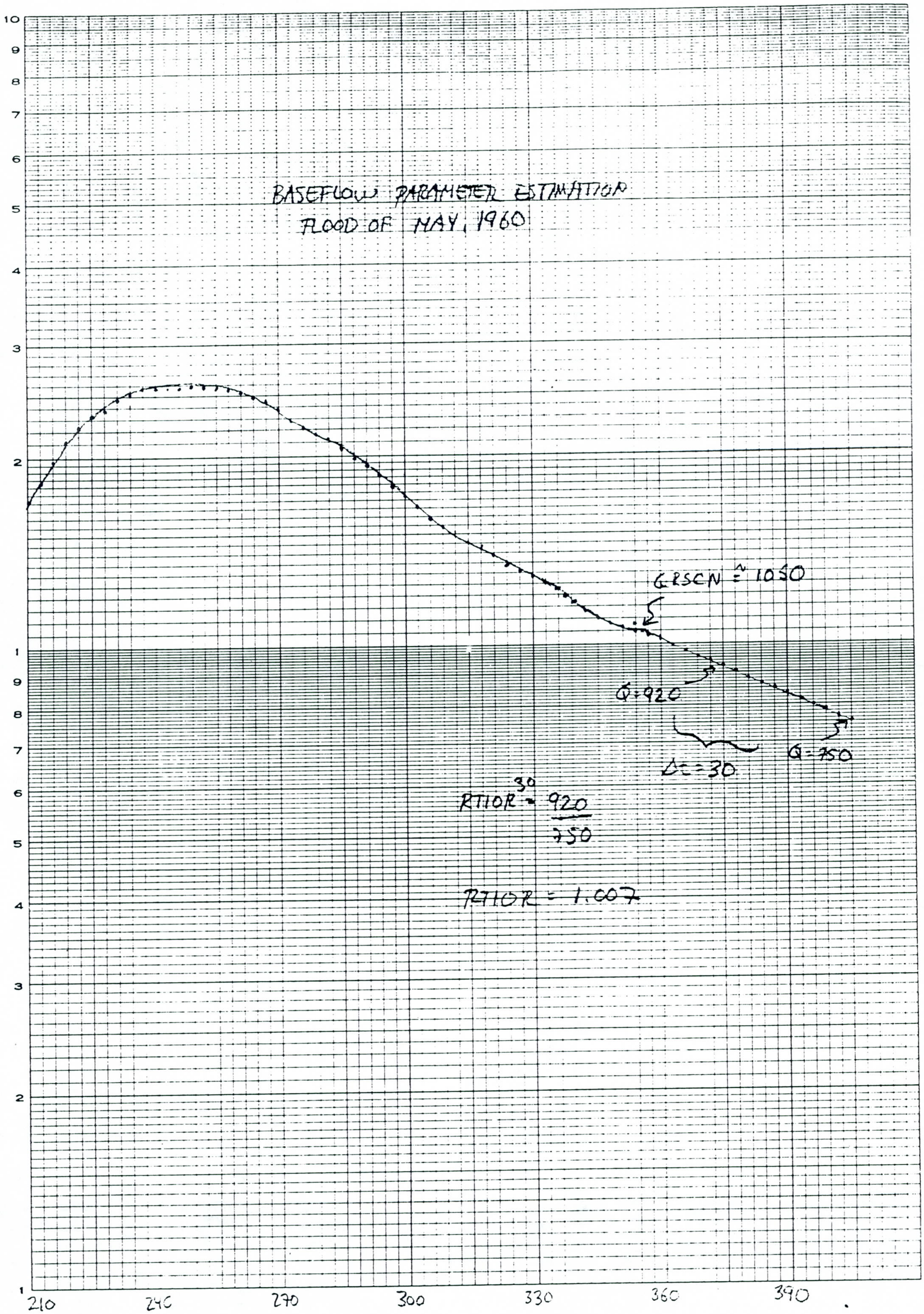
EXHIBIT 3

Calibration Hydrographs: Baseflow Separation Plots


BASEFLOW PARAMETER ESTIMATION
FLOOD OF MAY, 1960
GRSCN ≈ 1050
Q=920
Q=750
Δt=30
RTIOR^30 = 920/750
RTIOR = 1.007
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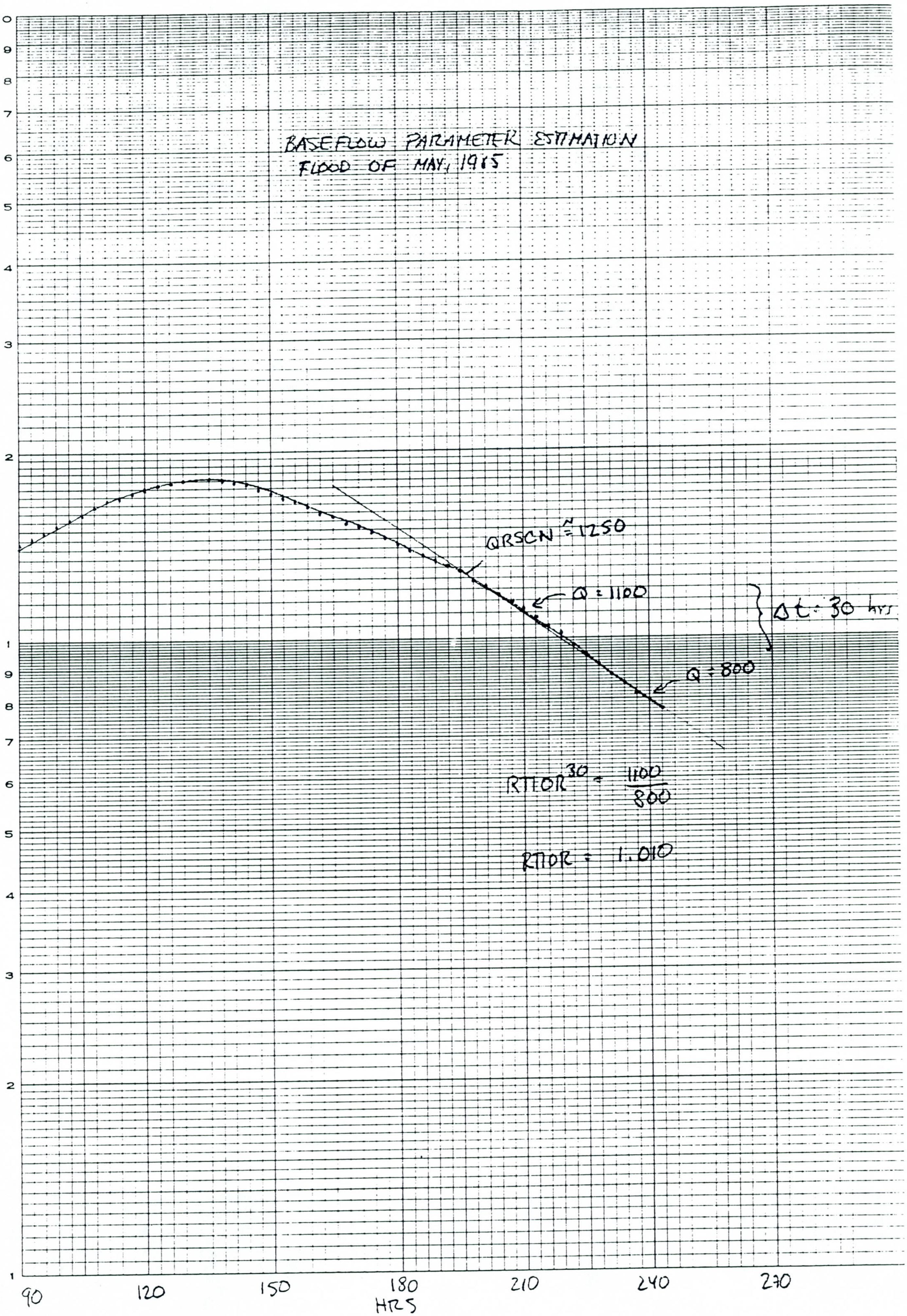
9
8
7
6
5
4
3
2
1
210
240
270
300
330
360
390


BASEFLOW PARAMETER ESTIMATION
FLOOD OF MAY, 1985
QRSCN ≅ 1250
Q = 1100
Δt = 30 hrs
Q = 800
RTIOR^{30} = 1100/800
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90
120
150
180
210
240
270
HRS
10
9
8
7
6
5
4
3
2
1

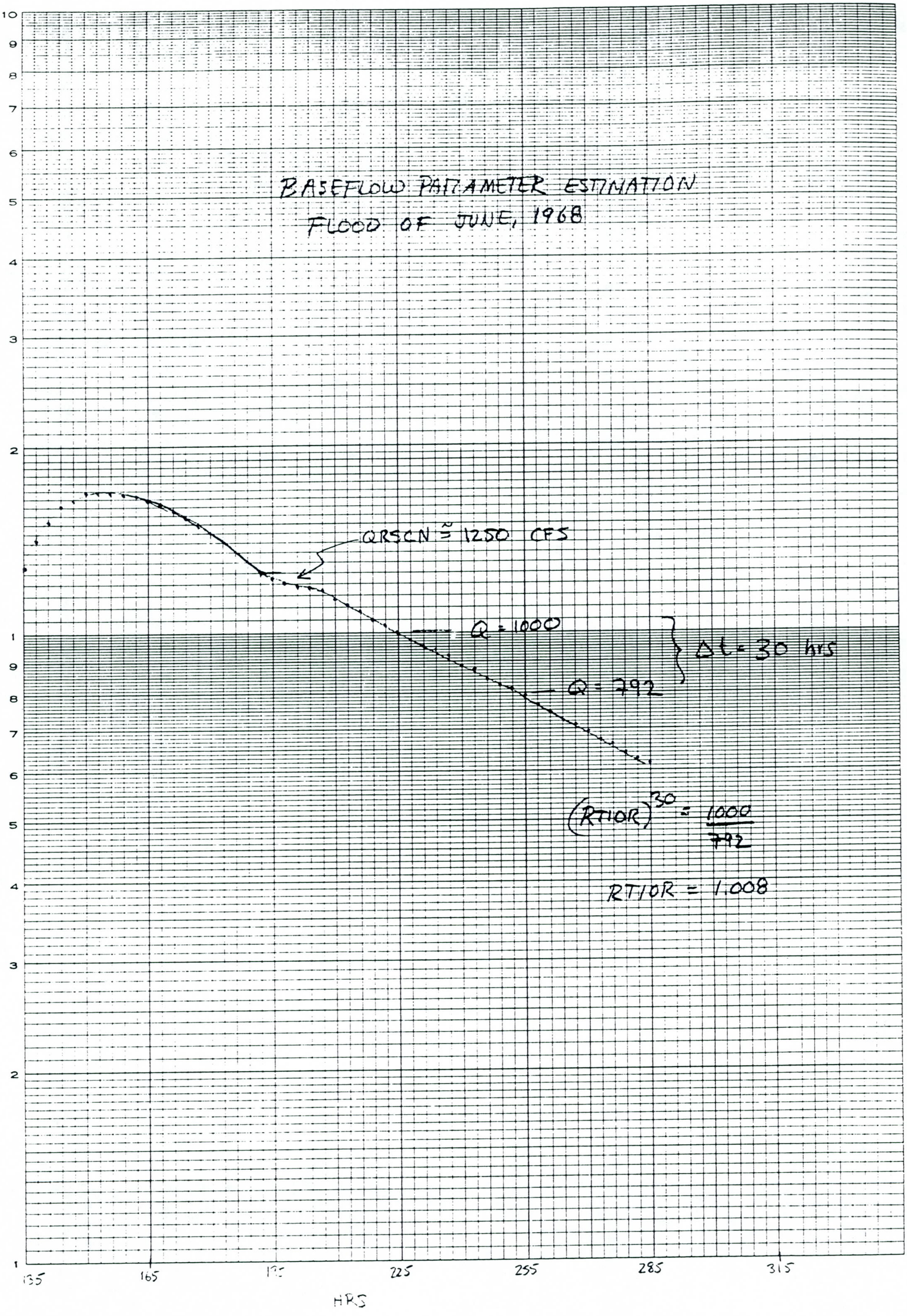
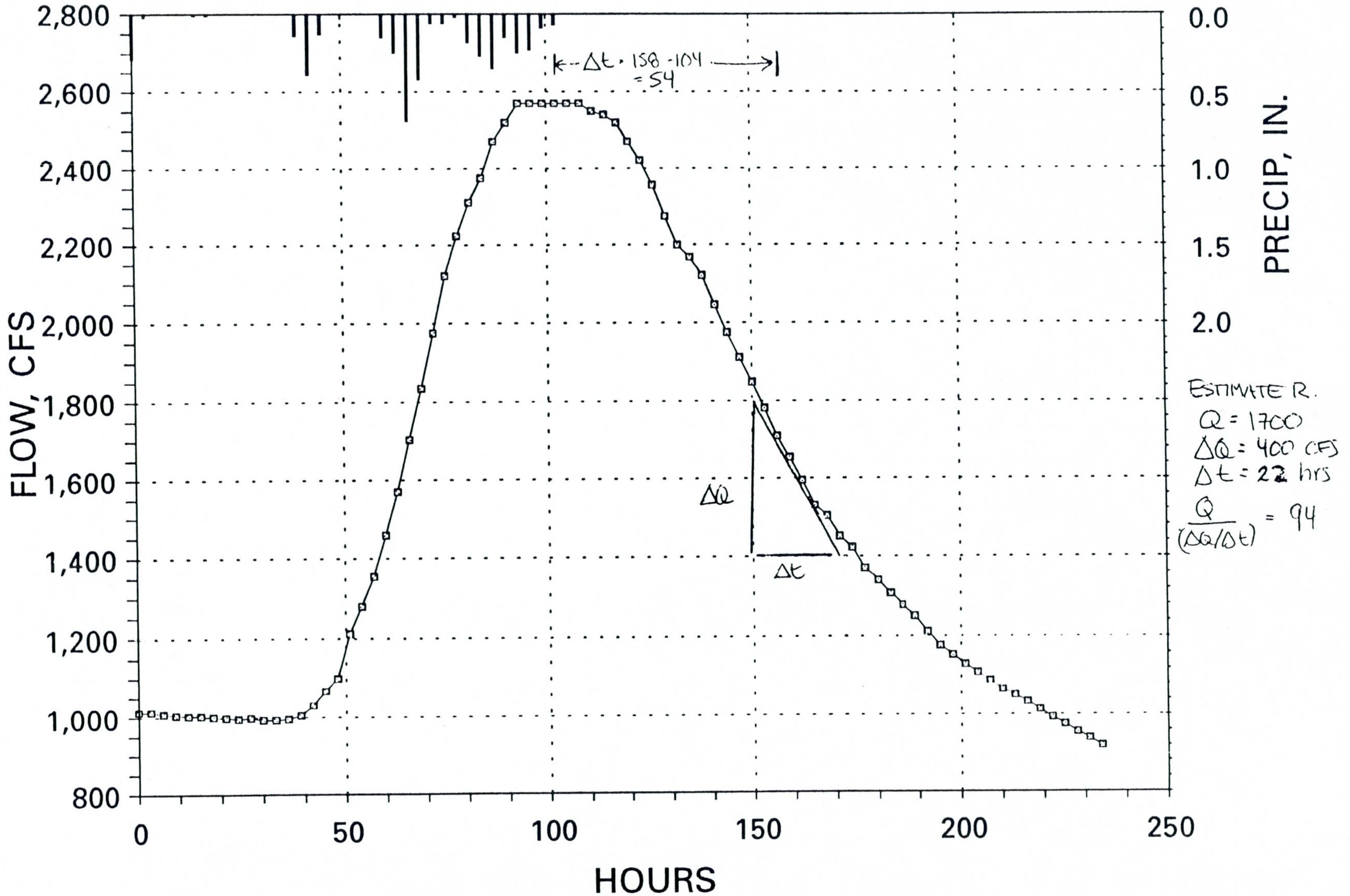

BASEFLOW PARAMETER ESTIMATION
FLOOD OF JUNE, 1968
QRSCN ≅ 1250 CFS
Q = 1000
Q = 792
Δt = 30 hrs
$(RTIOR)^{30} = \frac{1000}{792}$
RTIOR = 1.008
10
9
8
7
6
5
4
3
2
1
9
8
7
6
5
4
3
2
1
135
165
225
255
285
315
HRS

EXHIBIT 4

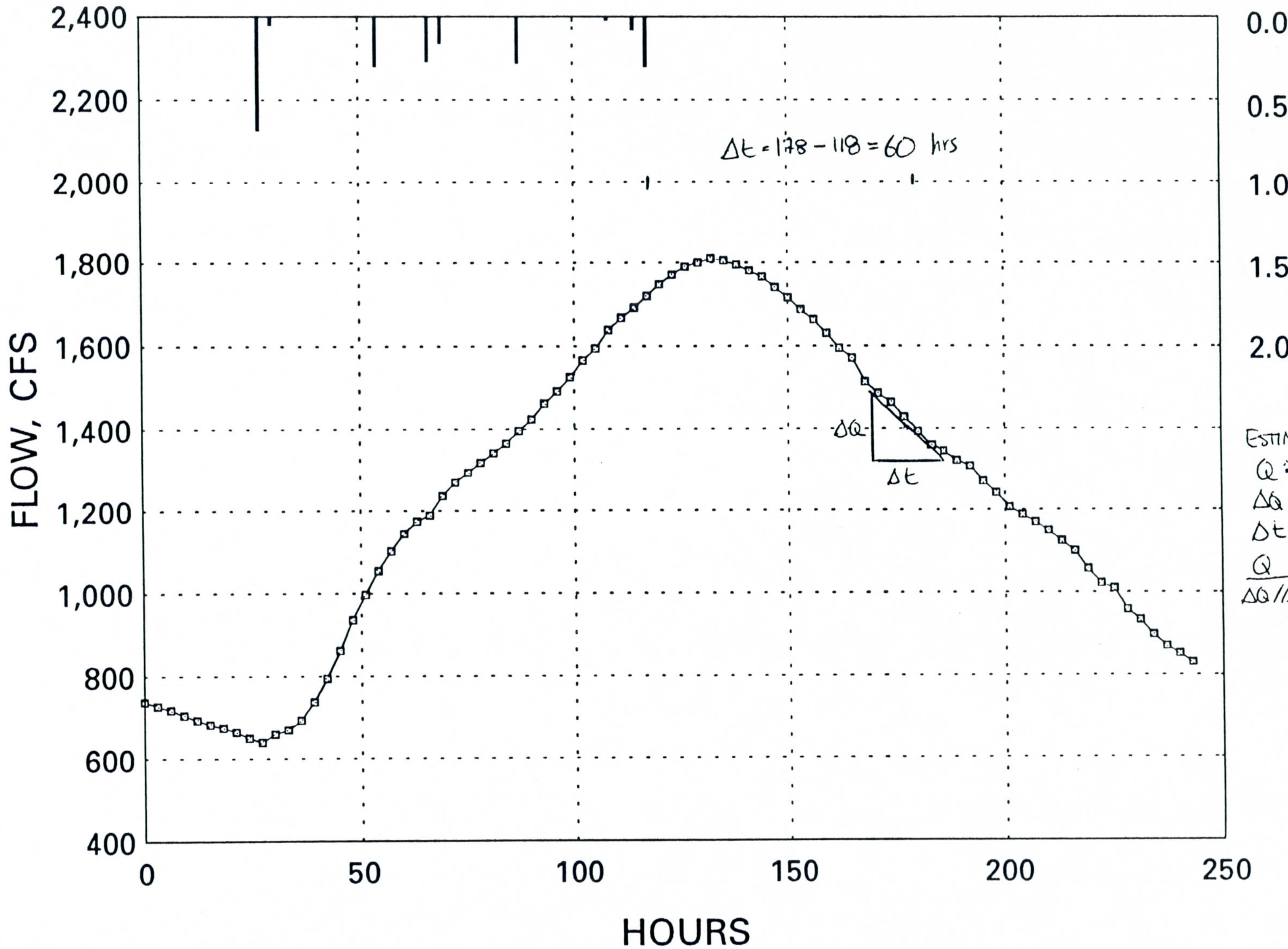
Calibration Hydrographs: Preliminary Estimates of Clark Parameters

MAY, 1960
PRELIMINARY ESTIMATE OF TC AND R



MAY, 1965

PRELIMINARY ESTIMATE OF TC AND R



ESTIMATE R:

$Q \doteq 1400$ cfs

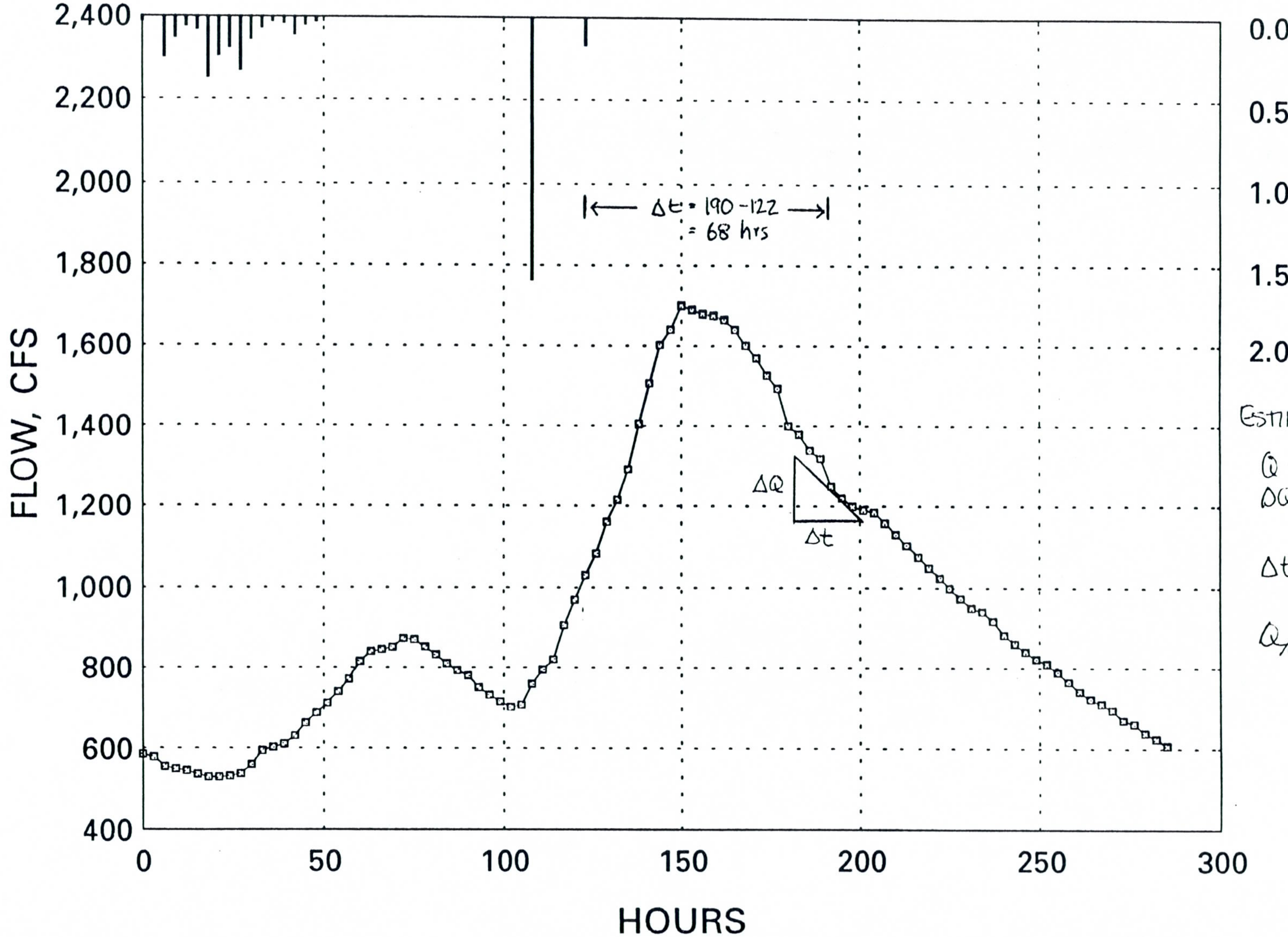
$\Delta Q = 1500 - 1300 = 200$

$\Delta t = 187 - 170 = 17$

$\dfrac{Q}{\Delta Q / \Delta t} = 119$

JUNE, 1968

PRELIMINARY ESTIMATE OF TC AND R



ESTIMATE R:

Q = 1220

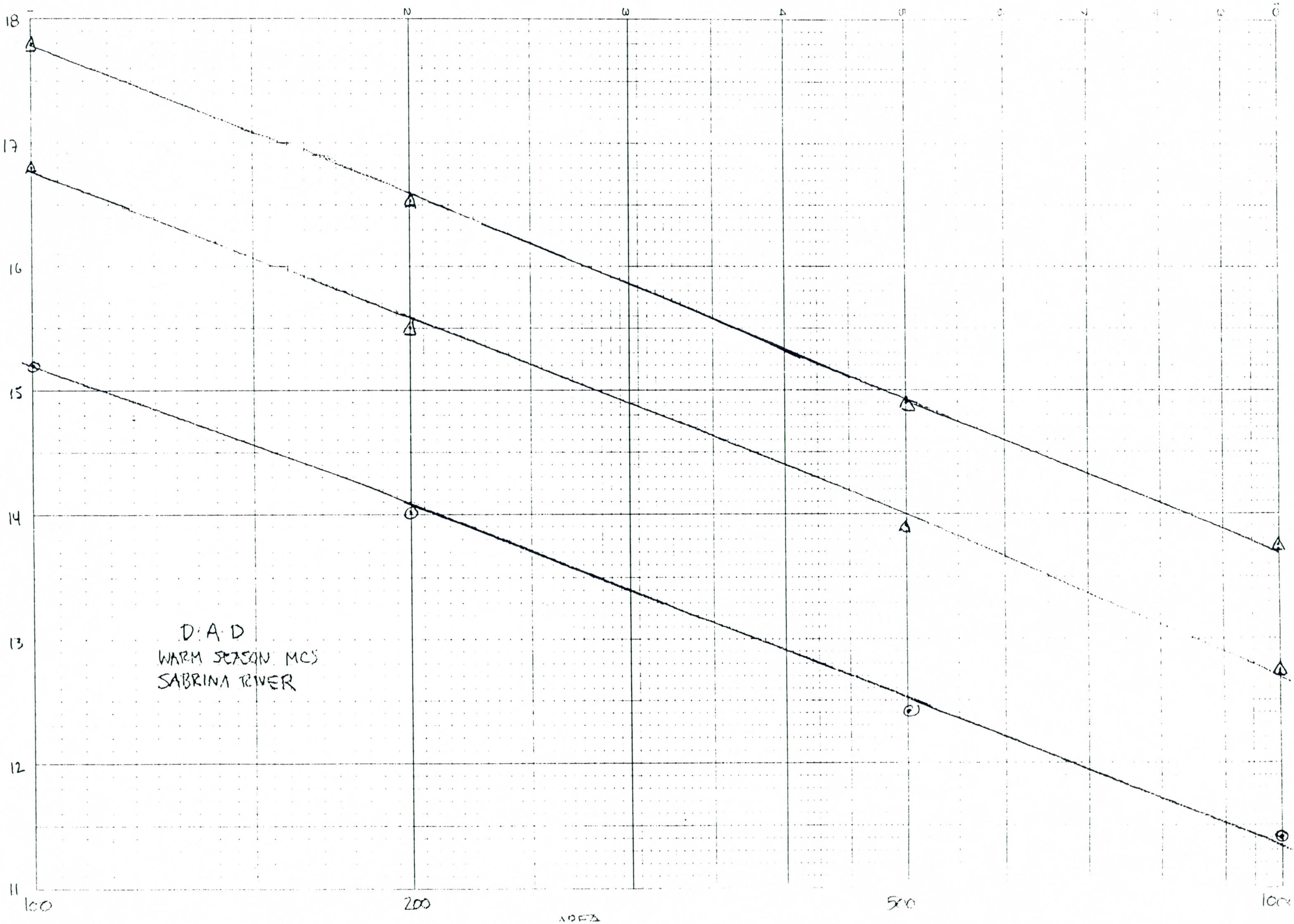
ΔQ = 1330 - 1160
= 170

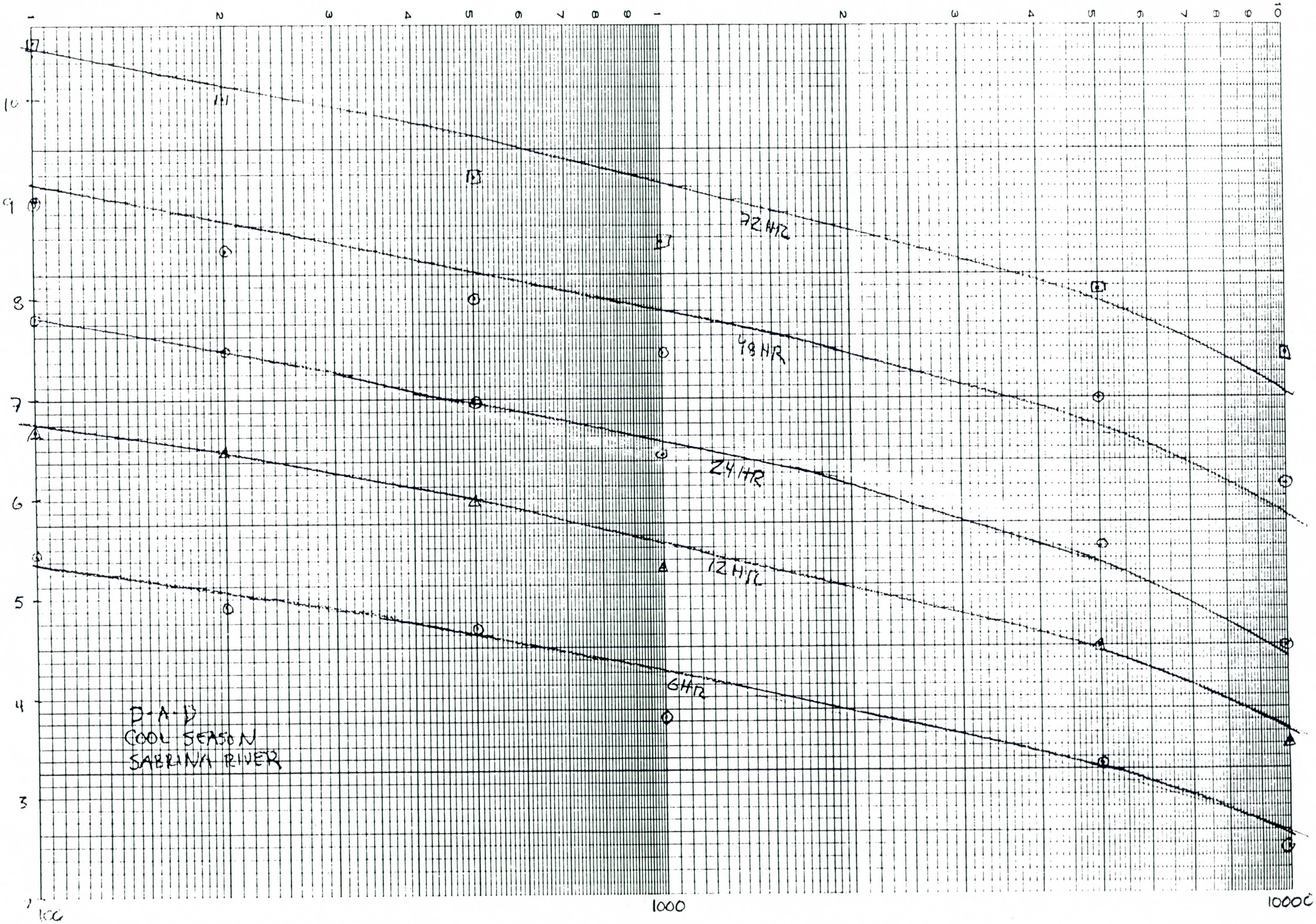
Δt = 200 - 180
= 20

Q/(ΔQ/Δt) =
144

EXHIBIT 5

Probable Maximum Precipitation Depth-Area-Duration Curves


18
17
16
15
14
13
12
11
100
200
500
1000
AREA
D.A.D
WARM SEASON MCS
SABRINA RIVER


72 HR
48 HR
24 HR
12 HR
6 HR
D-A-D
COOL SEASON
SABRINA RIVER
10
9
8
7
6
5
4
3
100
1000
10000

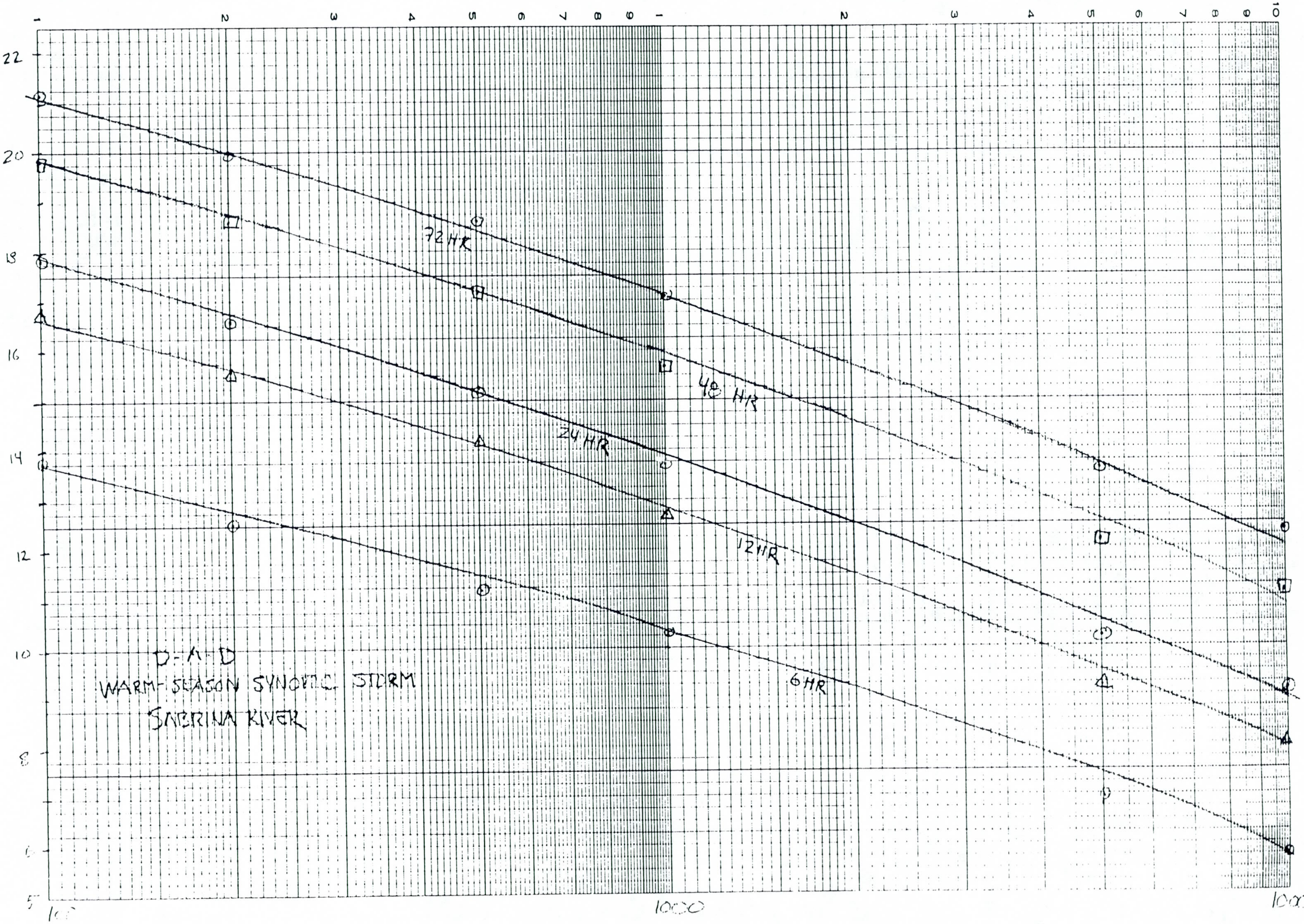
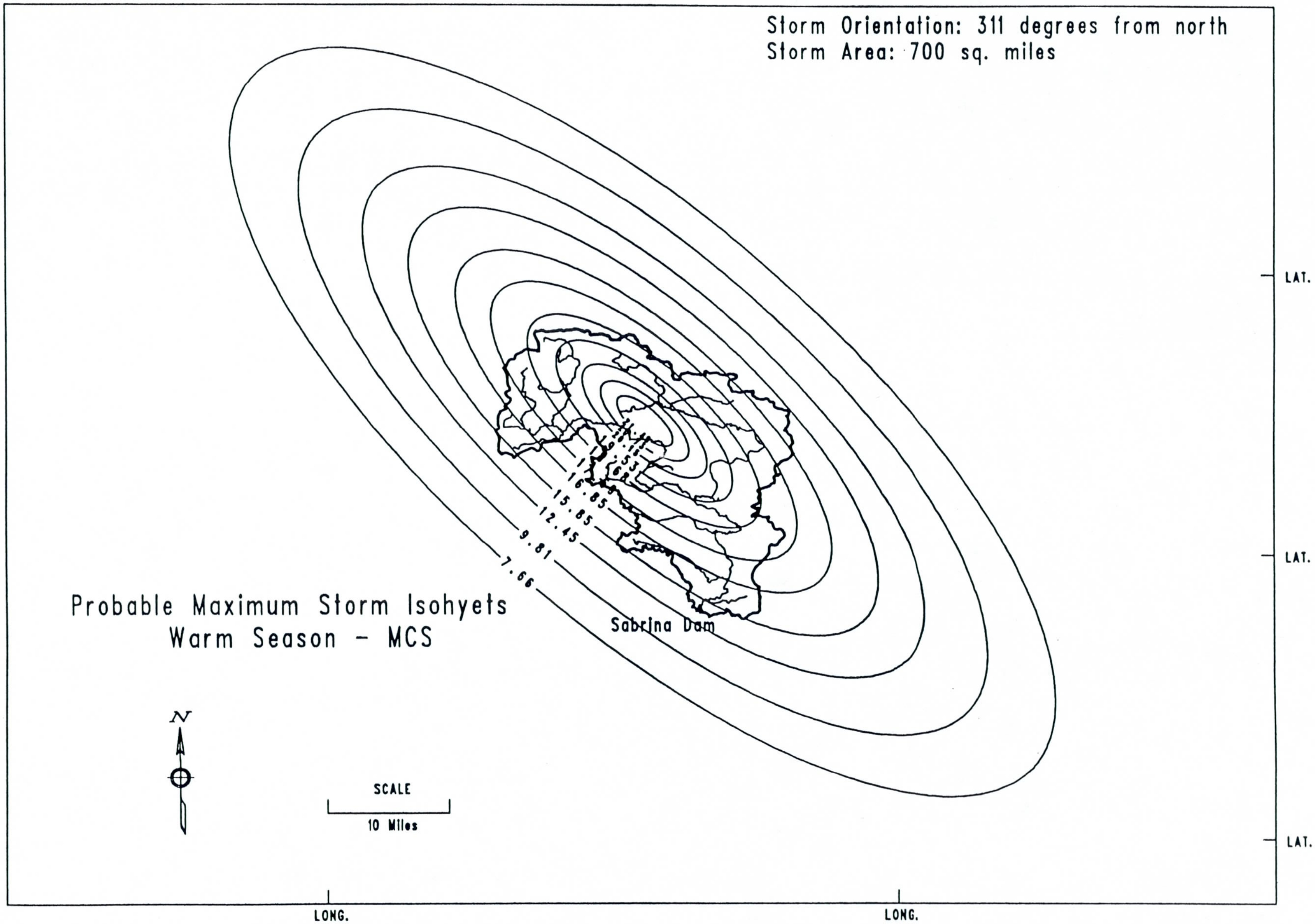
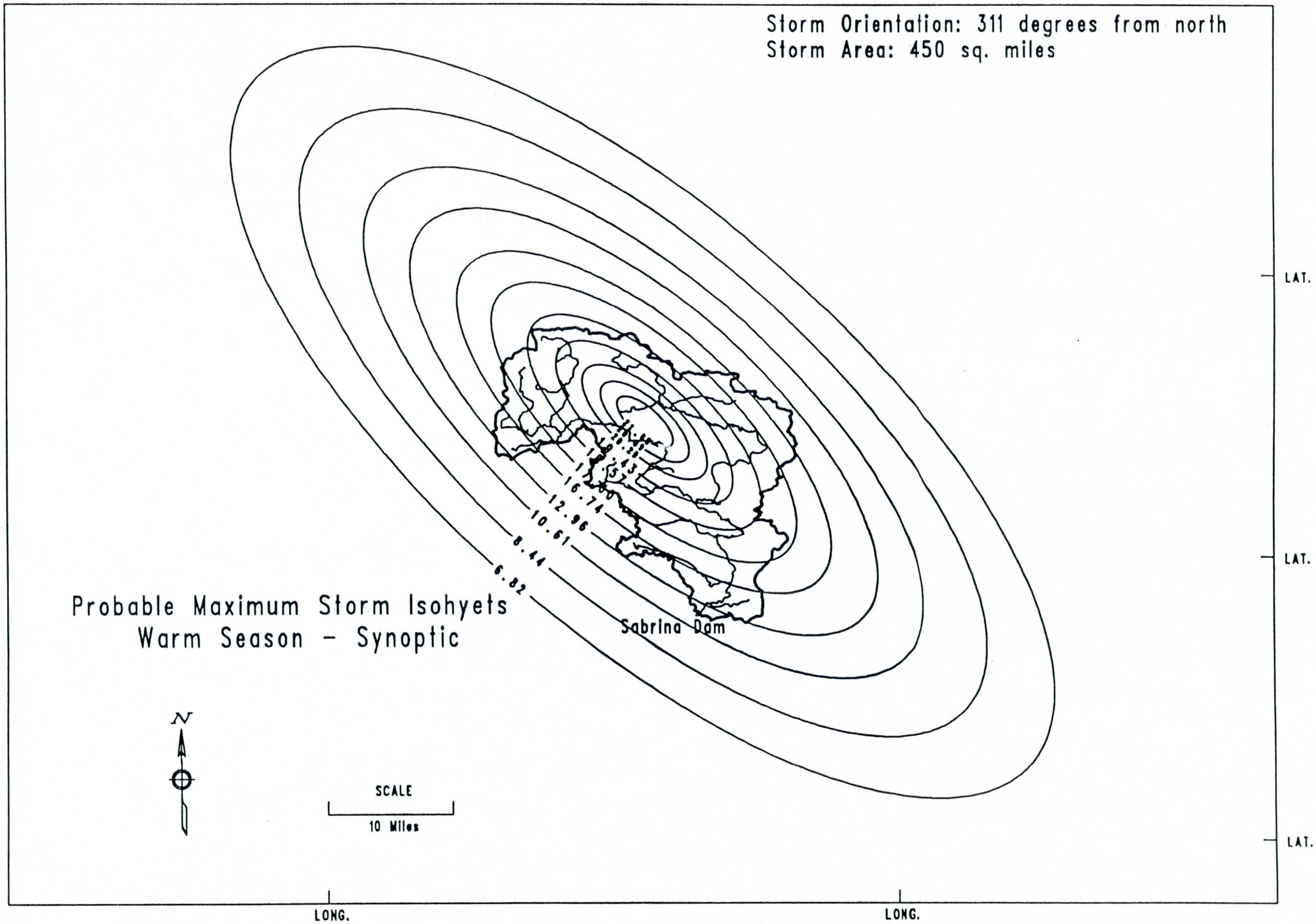
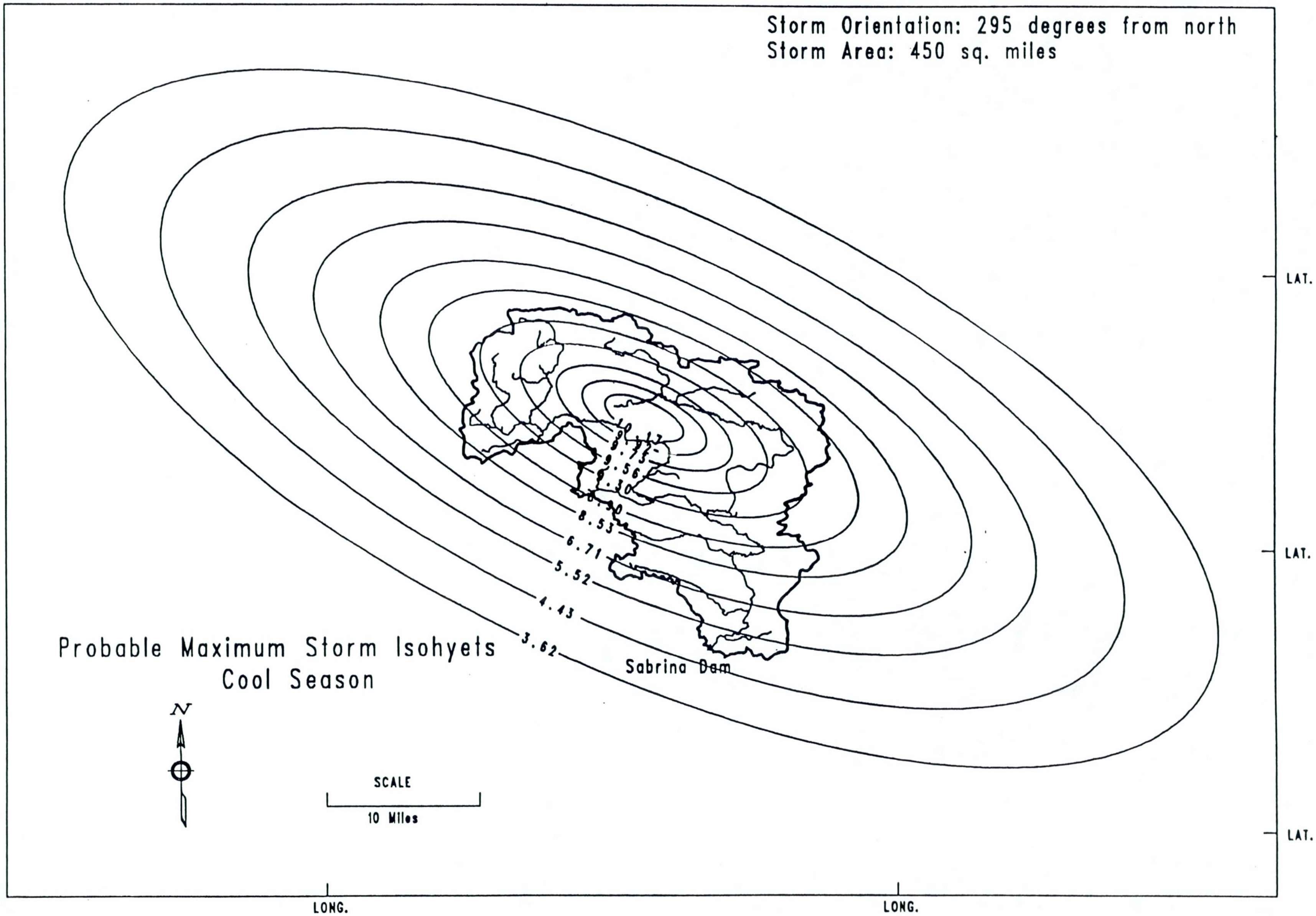

D-A-D
WARM-SEASON SYNOPTIC STORM
SABRINA RIVER
72 HR
48 HR
24 HR
12 HR
6 HR
22
20
18
16
14
12
10
8
6
100
1000
1000

EXHIBIT 6

Probable Maximum Storm Isohyetal Maps


Storm Orientation: 311 degrees from north
Storm Area: 700 sq. miles
LAT.
LAT.
LAT.
16.85
15.85
12.45
9.81
7.66
Sabrina Dam
Probable Maximum Storm Isohyets
Warm Season - MCS
N
SCALE
10 Miles
LONG.
LONG.


Storm Orientation: 311 degrees from north
Storm Area: 450 sq. miles
Probable Maximum Storm Isohyets
Warm Season - Synoptic
Sabrina Dam
6.82
8.44
10.61
12.96
16.74
SCALE
10 Miles
N
LAT.
LAT.
LAT.
LONG.
LONG.



Probable Maximum Storm Isohyets
Cool Season

EXHIBIT 7

Probable Maximum Storm Output Data

Note: Complete program outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

```
LINE            ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

   1            ID
   2            ID     PROBABLE MAXIMUM PRECIPITATION OVER SABRINA RIVER WATERSHED
   3            ID
   4            ID                     WARM SEASON - SYNOPTIC
*** FREE ***
   5            PU        HY

   6            BN     SABR
   7            BS   0.6215
   8            BX     47.4     40.2     34.9     29.3     28.5     29.2     31.3     31.2     28.5     25.0
   9            BX     25.5     27.1     25.9     24.6     22.5     18.8     18.2     13.3     13.0     12.9
  10            BX     14.1     14.2     16.6     18.2     17.7     23.7     28.8     30.5     36.5     37.3
  11            BX     44.5     43.3     44.2     46.5     51.3     52.0     49.7     49.7     47.7     46.8
  12            BX     48.7     50.9     46.4     47.3
  13            BY     70.3     70.0     78.3     78.5     79.3     80.6     80.8     82.5     86.3     87.5
  14            BY     90.7     91.2     94.2     94.9     93.2     92.6     91.8     91.0     94.4     96.7
  15            BY     97.2    103.4    104.8    104.7    107.0    107.4    106.2    106.8    100.8    102.0
  16            BY    101.5    100.6     99.3     99.3     96.8     91.1     89.6     87.4     86.6     82.9
  17            BY     82.8     80.5     75.5     73.6
  18            BA      310
                **
                **
                **
  19            HO      255     WARM
  20            HP      100     13.7     16.8     17.8     19.8     21.2
  21            HP      200     12.5     15.6     16.6     18.6     20.0
  22            HP      500     11.2     14.2     15.2     17.2     18.6
  23            HP     1000     10.3     12.7     13.7     15.7     17.1
  24            HP     5000      6.8      9.2     10.2     12.2     13.6
  25            HP    10000      5.7      8.1      9.1     11.1     12.3
  26            SA        0        0
  27            ST       60     .305
  28            ZZ
```

EXHIBIT 8

Soil Infiltration Rates

Map Unit	Seq.	% of Map Unit	Minimum Permeability 60"	Minimum Permeability 24"
177	[illegible]	9	4.00	4.00
177	2	13	4.00	4.00
177	3	17	1.30	1.30
177	4	15	1.10	1.10
177	5	12	1.30	1.30
177	6	9	13.00	13.00
177	[illegible]	3	13.00	13.00
177	8	7	3.10	3.10
177	9	3	11.00	11.00
177	10	3	13.00	13.00
177	11	2	1.30	1.30
177	12	2	13.00	13.00
177	13	2	4.00	4.00
177	14	2	1.30	1.30
179	1	14	1.30	4.00
179	2	8	1.30	4.00
179	3	19	3.10	3.10
179	4	9	3.10	3.10
179	5	9	1.30	1.30
179	6	[illegible]	1.30	1.30
179	[illegible]	7	13.00	13.00
179	8	7	3.10	3.10
179	9	4	3.30	3.30
179	10	4	4.00	4.00
179	11	3	1.30	13.00
179	12	[illegible]	1.30	13.00
179	13	3	13.00	13.00
179	14	2	1.30	1.30
179	15	2	1.30	1.30
179	16	2	4.00	4.00
179	17	2	1.30	1.30
179	18	[illegible]	11.00	11.00
179	19	[illegible]	0.33	0.33
179	20	[illegible]	13.00	13.00
182	[illegible]	27	3.30	3.30
182	2	2	3.30	3.30
182	3	24	0.40	4.00
182	4	13	3.10	3.10
182	5	10	1.10	1.10
182	6	7	4.00	4.00
182	[illegible]	[illegible]	4.00	4.00
182	8	3	1.30	1.30
182	9	[illegible]	1.30	1.30
182	10	5	0.04	0.04
182	11	3	0.00	0.00
182	12	3	0.00	3.10
204	[illegible]	19	1.10	1.10
204	2	11	1.30	4.00

Map Unit	Seq.	% of Map Unit	Minimum Permeability 60"	Minimum Permeability 24"
234	3	3	1.30	4.00
234	4	1	1.30	4.00
234	5	9	0.40	0.40
234	6	5	0.40	0.40
234	7	9	3.10	3.10
234	8	8	3.10	3.10
234	9	5	1.30	1.30
234	10	3	1.30	1.30
234	11	7	3.30	3.30
234	12	5	1.30	1.30
234	13	3	1.30	1.30
234	14	3	1.30	1.30
234	15	2	0.40	0.40
234	16	2	1.10	3.10
234	17	1	1.30	1.30
234	18	1	0.33	0.33
234	19	1	0.33	1.30
234	20	1	0.33	0.33
234	21	1	1.30	1.30
237	1	5	4.00	4.00
237	2	14	4.00	4.00
237	3	12	4.00	4.00
237	4	4	13.00	13.00
237	5	6	13.00	13.00
237	6	12	13.00	13.00
237	7	6	13.00	13.00
237	8	11	13.00	13.00
237	9	2	3.30	4.00
237	10	4	3.30	4.00
237	11	6	3.30	3.30
237	12	4	3.10	3.10
237	13	4	1.30	3.10
237	14	5	3.10	3.10
237	15	2	3.10	3.10
237	16	2	4.00	4.00
237	17	1	13.00	13.00
239	1	29	3.10	3.10
239	2	26	1.10	1.10
239	3	5	13.00	13.00
239	4	5	13.00	13.00
239	5	9	1.30	1.30
239	6	8	13.00	13.00
239	7	5	11.00	11.00
239	8	5	1.30	1.30
239	9	4	13.00	13.00
239	10	2	13.00	13.00
239	11	2	13.00	13.00

SABRINA BASIN STATSGO SOIL ASSOCIATIONS AND MINIMUM PERMEABILITY

EXHIBIT 9

Determination of Rainfall Excess

SABRMCS.OUT (MCS)

THIS OUTPUT CREATED BY PROGRAM "RUNOFFC.EXE ", DATED 9/7/93

DSS FILE = SABRMCSH.DSS
PATHNAME = /SABR/SABR/PRECIP-INC//1HOUR/MS/
AREA FILE = SABR.ARE
SOILFILE = SABRMCSH.SOI

WARM SEASON CONDITIONS

TOTAL AREA IN BASIN = 194999 acres

% IN BASIN	INFILTRATION RATE (in/hr)
22.0000	13.00
12.0300	4.00
13.4800	3.30
18.4700	3.10
5.6300	1.30
.7400	1.10
.3900	.40
.0300	.13
.0300	.04
1.8300	.03
25.2500	.00

HOUR	RAIN (in.)	RAINFALL EXCESS (in.)
1.	.106	.028
2.	.109	.029
3.	.112	.030
4.	.116	.031
5.	.121	.032
6.	.125	.033
7.	.154	.041
8.	.144	.039
9.	.154	.041
10.	.182	.049
11.	.229	.062
12.	.296	.080
13.	.775	.211
14.	1.610	.462
15.	2.380	.723
16.	4.722	2.095
17.	2.145	.644
18.	1.391	.388
19.	.161	.043
20.	.153	.041
21.	.146	.039
22.	.140	.037
23.	.135	.036
24.	.130	.035

TOTAL RAINFALL EXCESS DEPTH =
5.249669 inches

SABRWS.OUT (Synoptic)

THIS OUTPUT CREATED BY PROGRAM "RUNOFFC.EXE ", DATED 9/7/93

DSS FILE = SABRWARM.DSS
PATHNAME = /SABR/SABR/PRECIP-INC//1HOUR/WS/
AREA FILE = SABR.ARE
SOILFILE = SABRWARM.SOI

WARM SEASON CONDITIONS

TOTAL AREA IN BASIN = 194999 acres

% IN BASIN	INFILTRATION RATE (in/hr)
22.0000	13.00
12.0300	4.00
13.4800	3.30
13.4800	3.30
5.6300	1.30
.7400	1.10
.3900	.40
.0300	.13
.0300	.04
1.8300	.03
25.2500	.00

HOUR	RAIN (in.)	RAINFALL EXCESS (in.)
1.	.033	.009
2.	.033	.009
3.	.033	.009
4.	.033	.009
5.	.033	.009
6.	.033	.009
7.	.041	.011
8.	.041	.011
9.	.041	.011
10.	.041	.011
11.	.041	.011
12.	.041	.011
13.	.053	.014
14.	.053	.014
15.	.053	.014
16.	.053	.014
17.	.053	.014
18.	.053	.014
19.	.074	.020
20.	.074	.020
21.	.074	.020
22.	.074	.020
23.	.074	.020
24.	.074	.020
25.	.109	.029
26.	.114	.030
27.	.120	.032

28.	.127	.034
29.	.136	.036
30.	.146	.039
31.	.283	.076
32.	.328	.089
33.	.380	.103
34.	.438	.118
35.	.502	.136
36.	.571	.155
37.	.840	.229
38.	1.344	.372
39.	2.027	.604
40.	4.526	1.943
41.	1.735	.505
42.	1.183	.324
43.	.246	.066
44.	.217	.058
45.	.193	.052
46.	.175	.047
47.	.161	.043
48.	.153	.041
49.	.093	.025
50.	.093	.025
51.	.093	.025
52.	.093	.025
53.	.093	.025
54.	.093	.025
55.	.062	.016
56.	.062	.016
57.	.062	.016
58.	.062	.016
59.	.062	.016
60.	.062	.016
61.	.046	.012
62.	.046	.012
63.	.046	.012
64.	.046	.012
65.	.046	.012
66.	.046	.012
67.	.037	.009
68.	.037	.009
69.	.037	.009
70.	.037	.009
71.	.037	.009
72.	.037	.009

TOTAL RAINFALL EXCESS DEPTH =
5.847570 (inches)

SABRCS.OUT

THIS OUTPUT CREATED BY PROGRAM "RUNOFFC.EXE ", DATED 9/7/93

DSS FILE = SABRCOOL.DSS
PATHNAME = /SABR/SABR/PRECIP-INC//1HOUR/CS/
AREA FILE = SABR.ARE
SOILFILE = SABRCOOL.SOI

COOL SEASON CONDITIONS

INF.RATE BELOW WHICH THE SOILS FREEZE = 10.000000

TEMPERATURES USED FOR SNOWMELT 59.0 64.5 62.5

TOTAL AREA IN BASIN = 194999 acres

% IN BASIN	INFILTRATION RATE (in/hr)
22.0000	13.00
10.4300	4.00
11.6900	3.30
16.0100	3.10
4.8900	1.30
.6500	1.10
.3400	.40
.0300	.13
.0300	.04
1.5800	.03
32.2300	.00

HOUR	RAIN (in.)	RAINFALL EXCESS (in.)
1.	.084	.028
2.	.084	.028
3.	.084	.028
4.	.084	.028
5.	.084	.028
6.	.084	.028
7.	.089	.030
8.	.089	.030
9.	.089	.030
10.	.089	.030
11.	.089	.030
12.	.089	.030
13.	.098	.033
14.	.098	.033
15.	.098	.033
16.	.098	.033
17.	.098	.033
18.	.098	.033
19.	.113	.038
20.	.113	.038
21.	.113	.038
22.	.113	.038
23.	.113	.038

24.	.113	.038
25.	.145	.049
26.	.148	.050
27.	.152	.051
28.	.156	.052
29.	.160	.054
30.	.165	.055
31.	.212	.071
32.	.218	.073
33.	.231	.078
34.	.251	.084
35.	.277	.093
36.	.311	.105
37.	.421	.142
38.	.622	.211
39.	.901	.306
40.	1.921	.691
41.	.780	.265
42.	.557	.189
43.	.207	.069
44.	.197	.066
45.	.189	.063
46.	.181	.061
47.	.175	.059
48.	.170	.057
49.	.132	.044
50.	.132	.044
51.	.132	.044
52.	.132	.044
53.	.132	.044
54.	.132	.044
55.	.111	.037
56.	.111	.037
57.	.111	.037
58.	.111	.037
59.	.111	.037
60.	.111	.037
61.	.100	.034
62.	.100	.034
63.	.100	.034
64.	.100	.034
65.	.100	.034
66.	.100	.034
67.	.094	.031
68.	.094	.031
69.	.094	.031
70.	.094	.031
71.	.094	.031
72.	.094	.031

TOTAL RAINFALL EXCESS DEPTH =
4.639219 (in.)

EXHIBIT 10

Probable Maximum Flood Hydrograph Determination and Sensitivity Analysis – HEC-1 Output

A. Warm-Season Synoptic Storm
B. Warm-Season MCS
C. Cool-Season Storm With Snowmelt
D. Sensitivity to T_c Transfer

Note: Complete HEC-1 outputs have been omitted in this example to conserve space and paper. Actual study submittals should contain hard copy input and output data and a 3.5-inch diskette containing complete input and output data.

```
1*****************************************                ****************************************
 *                                       *                *                                      *
 *   FLOOD HYDROGRAPH PACKAGE  (HEC-1)   *                *   U.S. ARMY CORPS OF ENGINEERS       *
 *            FEBRUARY  1991             *                *   HYDROLOGIC ENGINEERING CENTER      *
 *         VERSION 4.0.1 (LOCAL)         *                *         609 SECOND STREET            *
 *                                       *                *      DAVIS, CALIFORNIA 95616         *
 * RUN DATE  11/11/93  TIME  14:42:36    *                *          (916) 551-1748              *
 *                                       *                *                                      *
 *****************************************                ****************************************
```

```
X     X  XXXXXXX   XXXXX           X
X     X  X        X     X         XX
X     X  X        X                X
XXXXXXX  XXXX     X         XXXXX  X
X     X  X        X                X
X     X  X        X     X          X
X     X  XXXXXXX   XXXXX          XXX
```

```
THIS PROGRAM REPLACES ALL PREVIOUS VERSIONS OF HEC-1 KNOWN AS HEC1 (JAN 73), HEC1GS, HEC1DB, AND HEC1KW.

THE DEFINITIONS OF VARIABLES -RTIMP- AND -RTIOR- HAVE CHANGED FROM THOSE USED WITH THE 1973-STYLE INPUT STRUCTURE.
THE DEFINITION OF -AMSKK- ON RM-CARD WAS CHANGED WITH REVISIONS DATED 28 SEP 81. THIS IS THE FORTRAN77 VERSION
NEW OPTIONS: DAMBREAK OUTFLOW SUBMERGENCE , SINGLE EVENT DAMAGE CALCULATION, DSS:WRITE STAGE FREQUENCY,
DSS:READ TIME SERIES AT DESIRED CALCULATION INTERVAL   LOSS RATE:GREEN AND AMPT INFILTRATION
KINEMATIC WAVE: NEW FINITE DIFFERENCE ALGORITHM
```

```
1                                       HEC-1 INPUT                                        PAGE  1

        LINE        ID.......1.......2.......3.......4.......5.......6.......7.......8.......9......10

          1         ID   SABRINA RIVER
          2         ID   WARM SEASON PROBABLE MAXIMUM FLOOD
          3         ID   WARM SEASON PMP CONVERTED TO EXCESS RAINFALL
          4         ID   EXCESS RAINFALL INPUT
          5         ID   STRTQ EQUAL TO NORMAL FLOW FOR WARM SEASON
          6         ID   USE CLARK'S CALIBRATED UNIT HYDROGRAPH
          7         ID
          8         ID          WARM-SEASON MCS
*** FREE ***
          9         IT     360 01AUG99    0100      120
         10         IN      60
         11         IO       0       2       0

         12         KK SABRINARIVER
         13         BA    305.
         14         ZR   =PI A=SABR B=SABR C=RUNOFF-CUM D=01AUG1999 E=1HOUR F=MS
         15         BF     600    1250   1.008
         16         UC      55     173
         17         UA    0.14    5.50   15.50   28.15   33.12   45.30   64.16   73.27   84.25  107.07
         18         UA  122.90
         19         LU      0.      .0     100
         20         ZW    A=INFLOW C=FLOW
```

```
21          KK  ROUTED
22          KM   ROUTE THROUGH RESERVOIR
23          KO       4       1
24          RS       1    ELEV   993.1
25          SA      76      80      91
26          SE     991   993.1     995
27          SQ       0     600    4800    5450    6520    8860
28          SE   992.6   993.1   993.2     994     995     997
29          ZW   A=OUTFLOW C=FLOW
30          ZZ
```

```
1*****************************************
*                                         *
*   FLOOD HYDROGRAPH PACKAGE  (HEC-1)     *
*            FEBRUARY  1991               *
*           VERSION 4.0.1 (LOCAL)         *
*                                         *
* RUN DATE  11/11/93  TIME  14:42:36      *
*                                         *
*****************************************

*****************************************
*                                         *
*   U.S. ARMY CORPS OF ENGINEERS          *
*   HYDROLOGIC ENGINEERING CENTER         *
*        609 SECOND STREET                *
*     DAVIS, CALIFORNIA 95616             *
*         (916) 551-1748                  *
*                                         *
*****************************************
```

```
                    SABRINA RIVER
                    WARM SEASON PROBABLE MAXIMUM FLOOD
                    WARM SEASON PMP CONVERTED TO EXCESS RAINFALL
                    EXCESS RAINFALL INPUT
                    STRTQ EQUAL TO NORMAL FLOW FOR WARM SEASON
                    USE CLARK'S CALIBRATED UNIT HYDROGRAPH

                          WARM-SEASON MCS

 11 IO        OUTPUT CONTROL VARIABLES
                    IPRNT          0  PRINT CONTROL
                    IPLOT          2  PLOT CONTROL
                    QSCAL         0.  HYDROGRAPH PLOT SCALE

   IT         HYDROGRAPH TIME DATA
                    NMIN         360  MINUTES IN COMPUTATION INTERVAL
                   IDATE      1AUG99  STARTING DATE
                   ITIME        0100  STARTING TIME
                      NQ         120  NUMBER OF HYDROGRAPH ORDINATES
                  NDDATE     30AUG99  ENDING DATE
                  NDTIME        1900  ENDING TIME
                   ICENT          19  CENTURY MARK

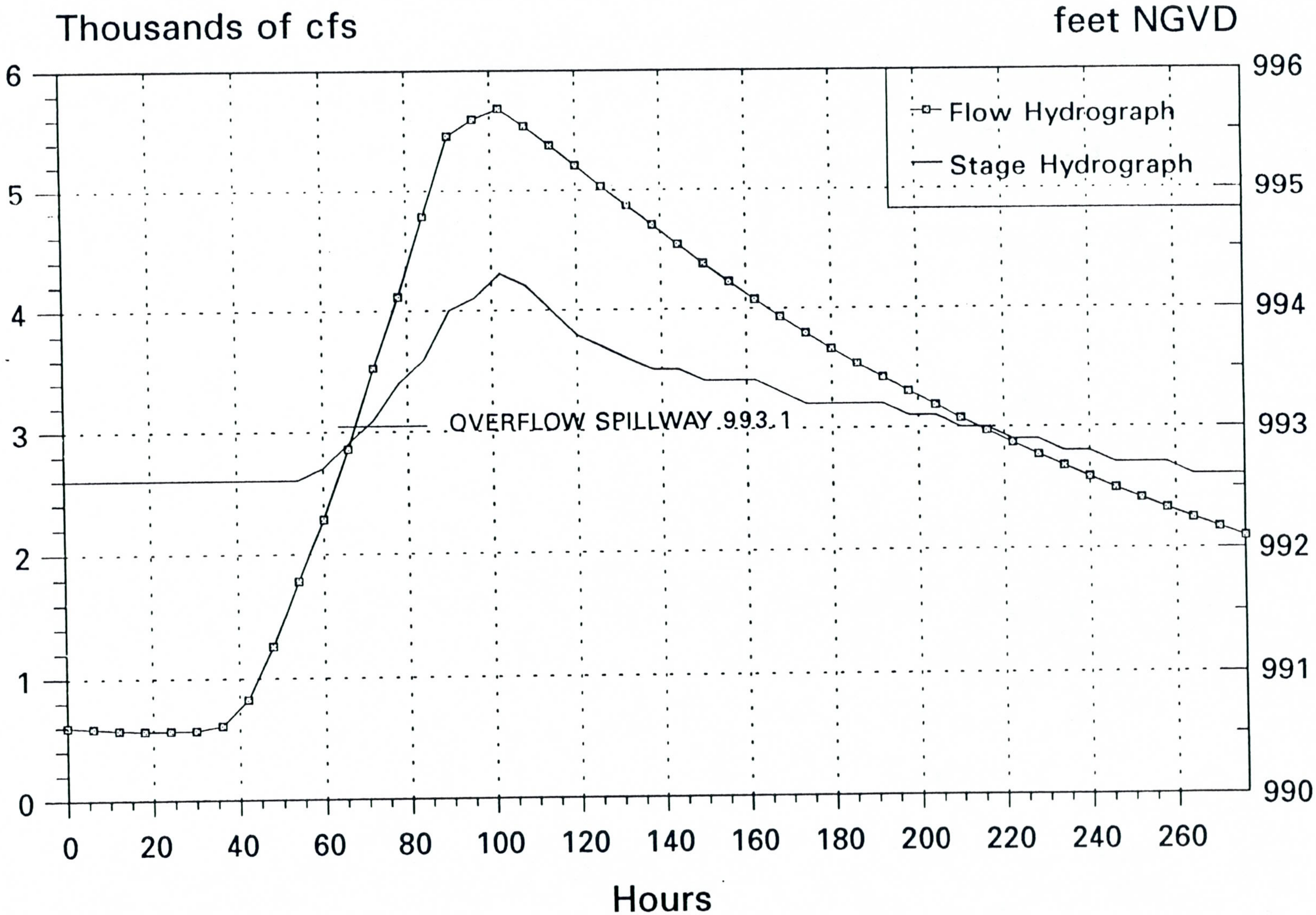
              COMPUTATION INTERVAL    6.00 HOURS
                   TOTAL TIME BASE  714.00 HOURS

      ENGLISH UNITS
          DRAINAGE AREA          SQUARE MILES
          PRECIPITATION DEPTH    INCHES
          LENGTH, ELEVATION      FEET
          FLOW                   CUBIC FEET PER SECOND
          STORAGE VOLUME         ACRE-FEET
          SURFACE AREA           ACRES
          TEMPERATURE            DEGREES FAHRENHEIT
```

EXHIBIT 11

Probable Maximum Flood Hydrograph Plots

SABRINA HYDROELECTRIC PROJECT
OUTFLOW PMF



REVIEW AND QUESTIONS

Tuesday 8:00 a.m.

SYNTHETIC UNIT HYDROGRAPHY THEORY FOR UNGAGED WATERSHEDS

Tuesday 8:30 a.m.

DEVELOPING WATERSHED PARAMETERS FOR UNGAGED WATERSHEDS

Tuesday 8:30 a.m.

SYNTHETIC UNIT HYDROGRAPHS

(SEC. 8-9 of FERC Guidelines on the Determination of Probable Maximum Flood)

Materials under TAB 18 in Volume 1 of the Notebook

This is part of the Guidelines related to the development of unit hydrograph for basins with limited data - Sec 8.9 of the FERC Guidelines

UNGAGED BASINS

DEFINITION

An <u>ungaged basin</u> defined as a basin for which no hydrologic data have been recorded within the basin, or available data are insufficient to enable the determination of a precipitation-runoff model for the basin

- one must use synthetic unit hydrographs derived on the basis of data from other "hydro-meteorologically similar" basins
- three synthetic unit hydrographs are recommended in the Guidelines:
    - Snyder Unit Hydrograph
    - SCS Dimensionless Unit Hydrograph
    - Clark Unit Hydrograph

Brief Review of the Unit Hydrograph Concept

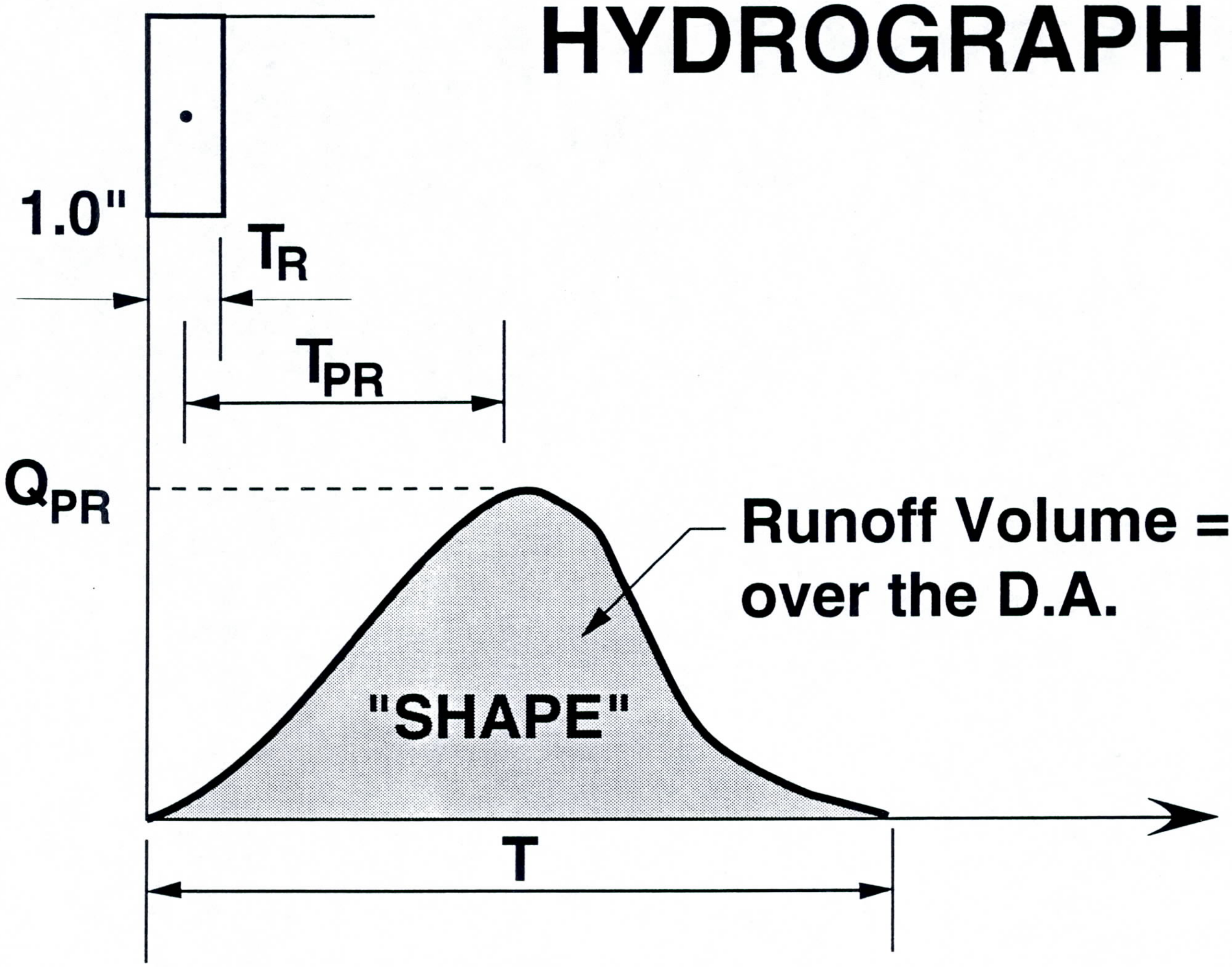
- volume of the unit hydrograph equals to <u>one unit of rainfall excess</u> from the watershed of interest

- the one unit of rainfall excess is <u>distributed uniformly over the entire watershed</u>

- the one unit of rainfall excess is applied to the watershed over <u>a specified time period</u>

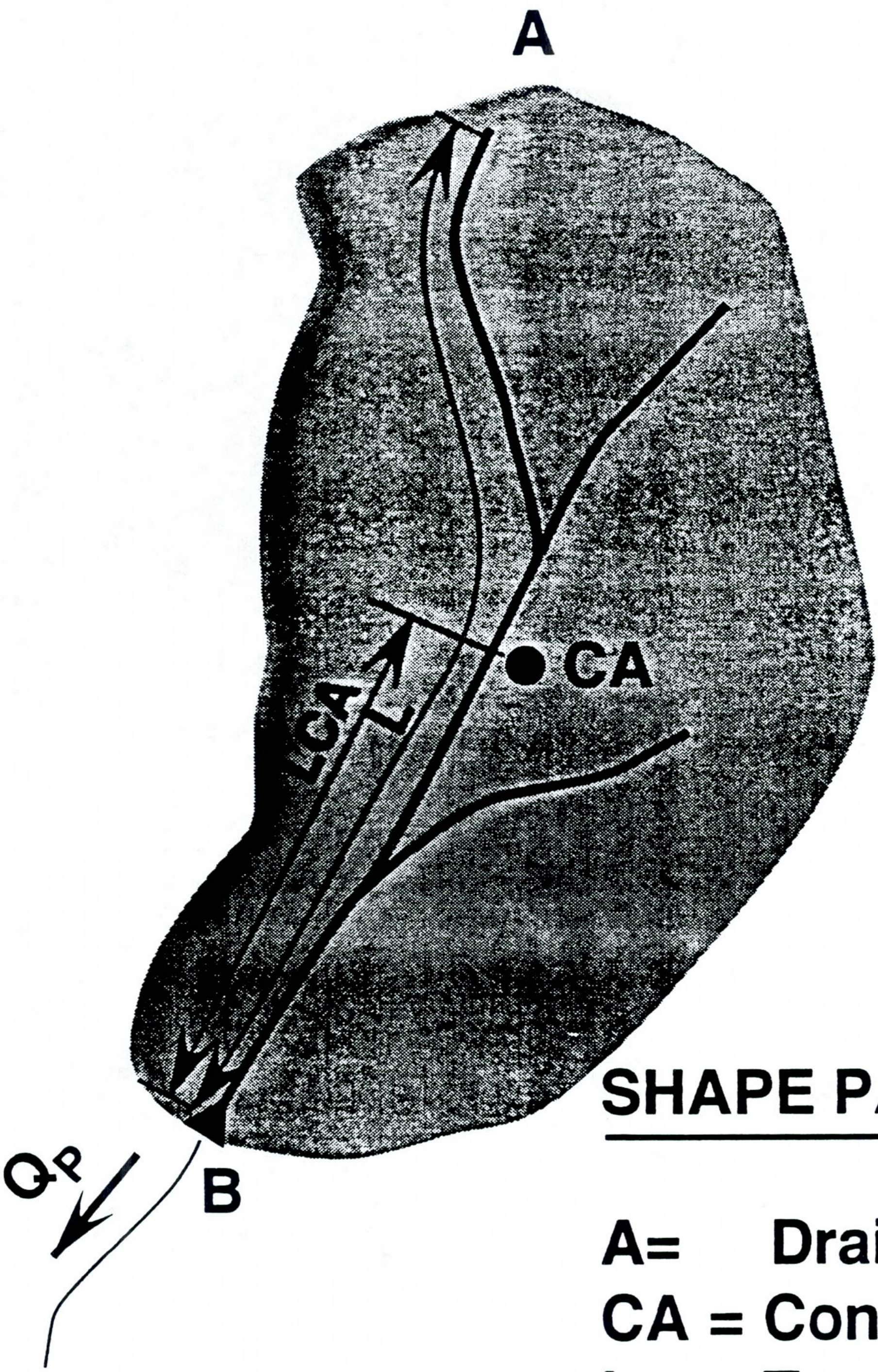
- the unit hydrograph theory is a <u>linear</u> theory, and the principle of superposition applies

A GENERAL UNIT HYDROGRAPH



Basin Characteristics

- Valley and Channel Storage
- Basin Configuration
- Topographical Features
- Meteorological Conditions



SHAPE PARAMETERS

A= Drainage Area

CA = Controid Location

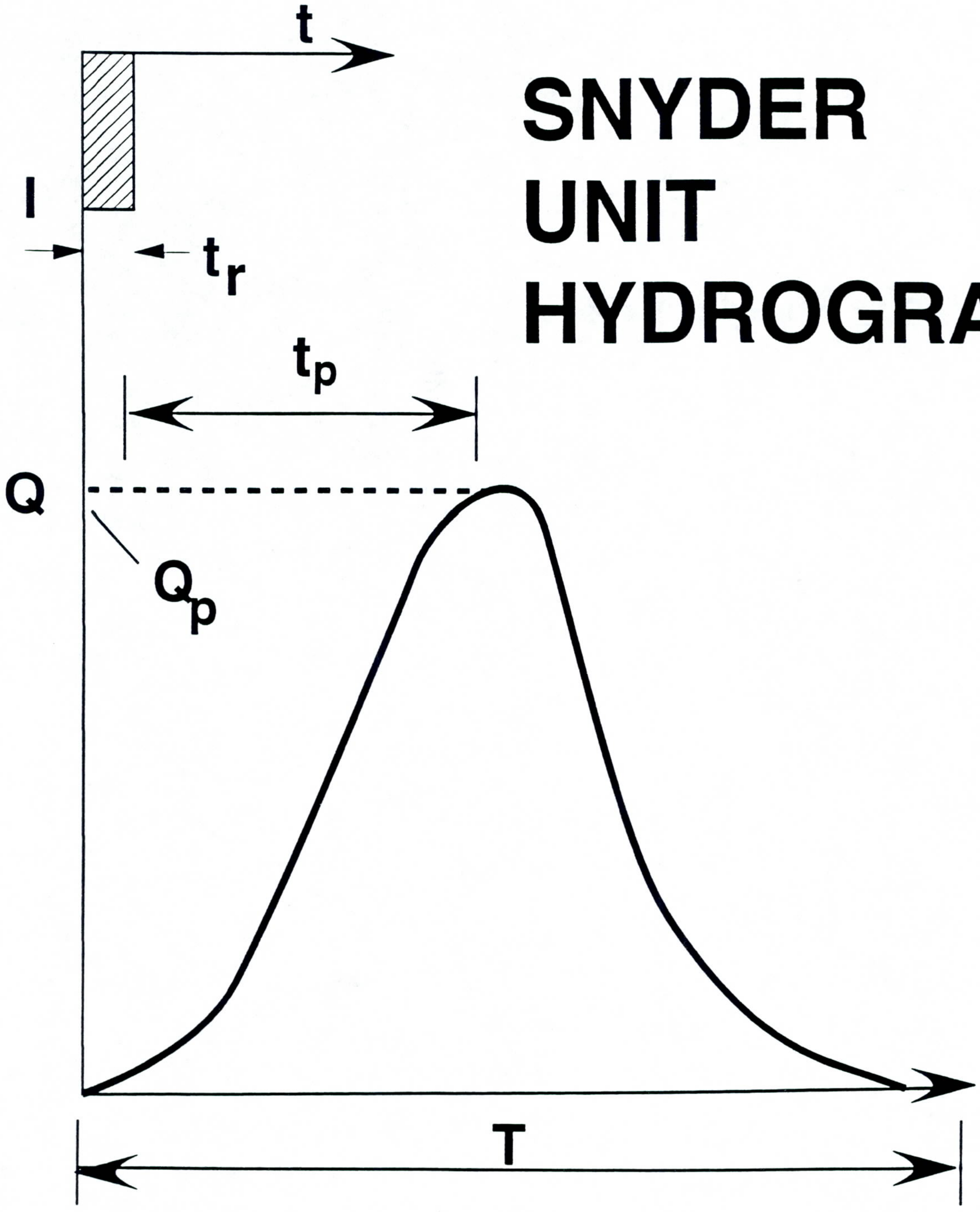
L = Total Length of Drainage Path

L_{CA}= Length of Drainage Path to Point Opposite CA

S = Slope

$$S_T = \frac{El.A - El.B}{L}$$

SNYDER UNIT HYDROGRAPH



$$t_p = C_t \, (L \cdot L_{CA})^{0.3}$$

$$Q = \frac{640 \, C_p A}{t_p}$$

$$t_r = t_p \, / \, 5.5$$

for very steep slope.

for very flat slope.

$$0.4 \leq C_t \leq 8.0$$

for very flat slope

for very steep slope

$$200 \leq C_p \leq 600$$

Question

- How do you chose C_t and C_p?

Answer

- You can't unless you have have information from other studies,

or

- You perform a regional study.

Source of C_t and C_p

- U.S. Army Corps of Engineers Report on Civil Works Investigation Project No. CW-153; Unit Hydrograph Compilations, Volumes 1 through 4. Volumes 1, 2 and 3 were published in 1949 and Volume 4 in 1954 by the Office of the District Engineer, Washington District

  Snyder's C_t and C_p for 146 watersheds were developed. These basins are in

  - New England Division
  - North Atlantic Division
  - South Atlantic Division
  - Lower Mississippi Valley Division
  - Southwestern Division
  - Great Lake Division
  - Ohio River Division
  - Upper Mississippi Valley Division
  - Missouri River Division

- District Offices of the U.S. Army Corps of Engineers

Note that with Snyder's C_t and C_p known, one can estimate the lag time and the peak of the unit hydrograph. However, the shape of the unit hydrograph is still undefined.

- Additional data are developed by the U.S. Army Corps of Engineer to help defining the shape of the unit hydrograph given in the form of a chart.

- This chart gives the W_{75} and W_{50} of the unit hydrograph in hours expressed as a function of the unit peak discharge of the unit hydrograph in cfs/sq mi.

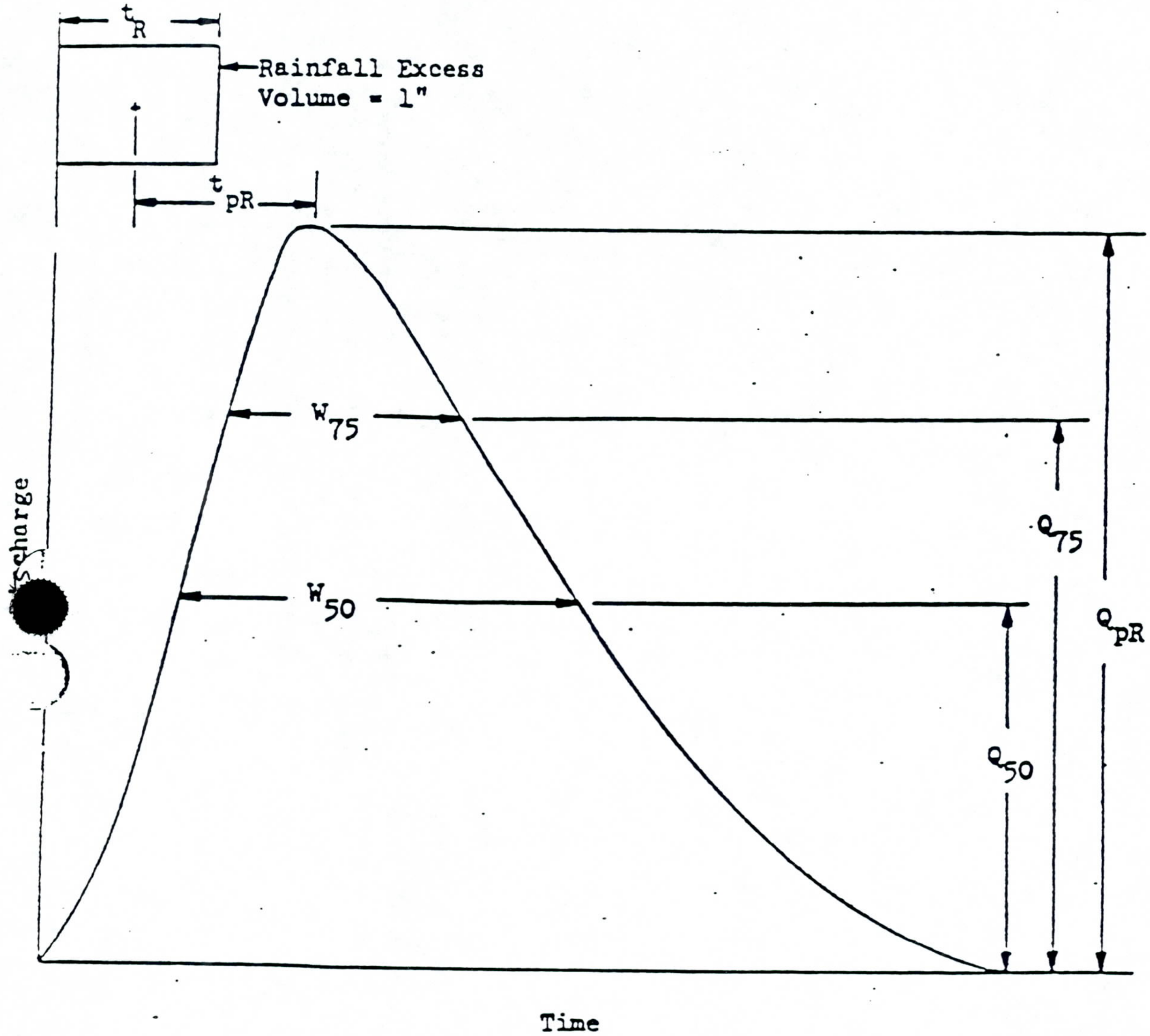


Fig. 1 - Definition of Unit Hydrograph Properties

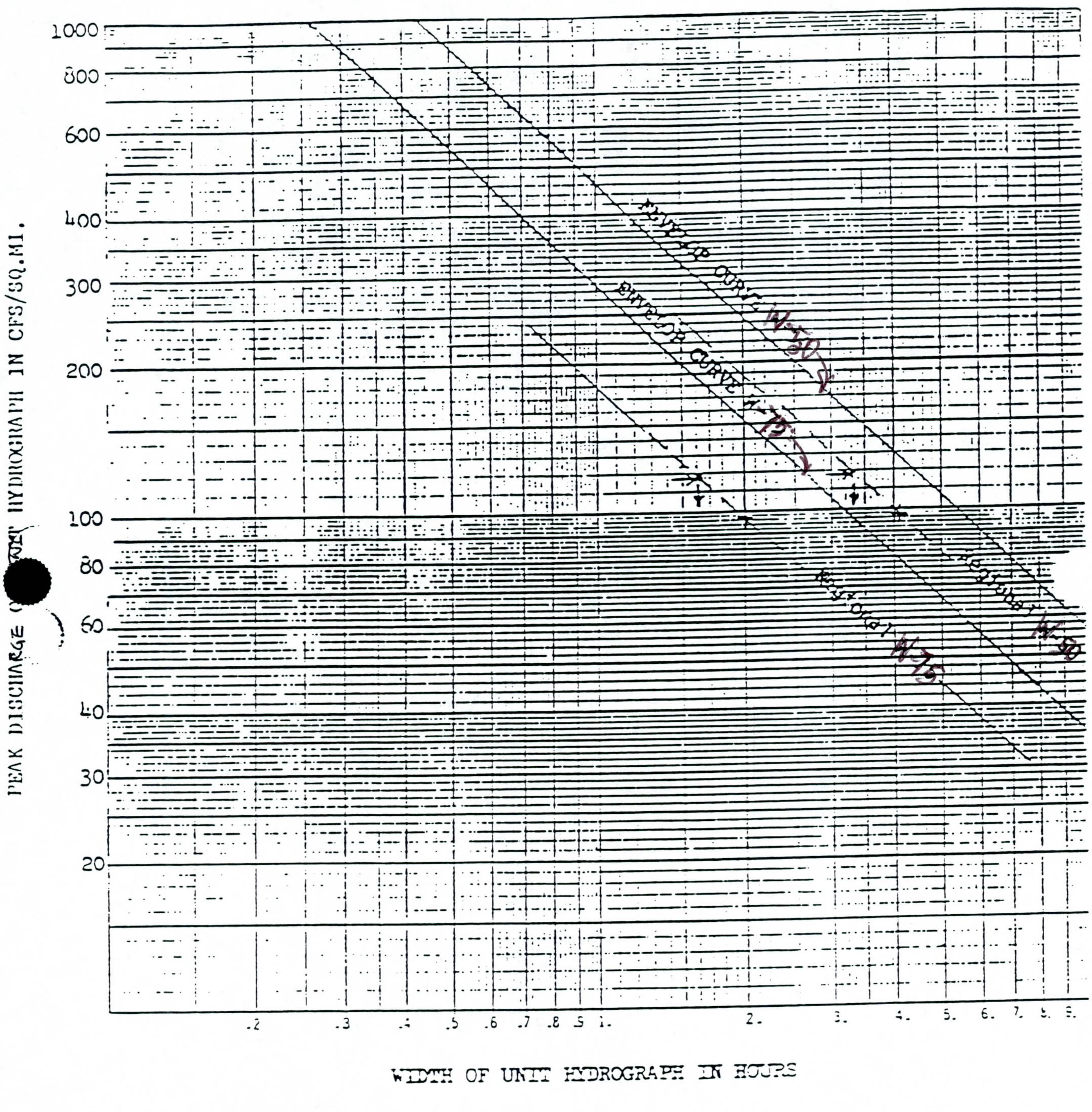


Fig. 2 - Width of Unit Hydrograph

SELECTION OF SNYDER'S STANDARD DURATION (t_r)

- t_r is defined as equal to (t_p / 5.5)
- It is a property of the Snyder Unit Hydrograph
- It is also synonymous with unit rainfall duration
- U.S. Army Corps of Engineers recommended values for t_r:

t_p (hr)	t_r (hr)
4 - 6	1
6 - 12	2
12 - 16	3
> 16	6

Snyder Lag Equation (1938)

$$Lag = C_t \, (L \, L_{ca})^{0.3}$$

where lag is in hours, L and L_{ca} are in miles and C_t is the Snyder coefficient

Modified Snyder Lag Equation

- U.S. Corps of Engineers Los Angeles District (1944)

$$Lag = 20 \, n \, [\, L \, L_{ca}/S^{0.5}]^{0.38}$$

where Lag is in hours, L & L_{ca} are in miles, S is the overall slope in feet per mile and n is the basin roughness coefficient.

→ Not a "manning's n", corp's own parameter.

- U.S Bureau of Reclamation (1989)

$$Lag = 26 \, K_n \, [\, L \, L_{ca}/S^{0.5}]^{0.33}$$

where Lag is in hours, L & L_{ca} are in miles, S is the overall slope in feet per mile and K_n is the weighted average Manning's n value for the principal watercourse.

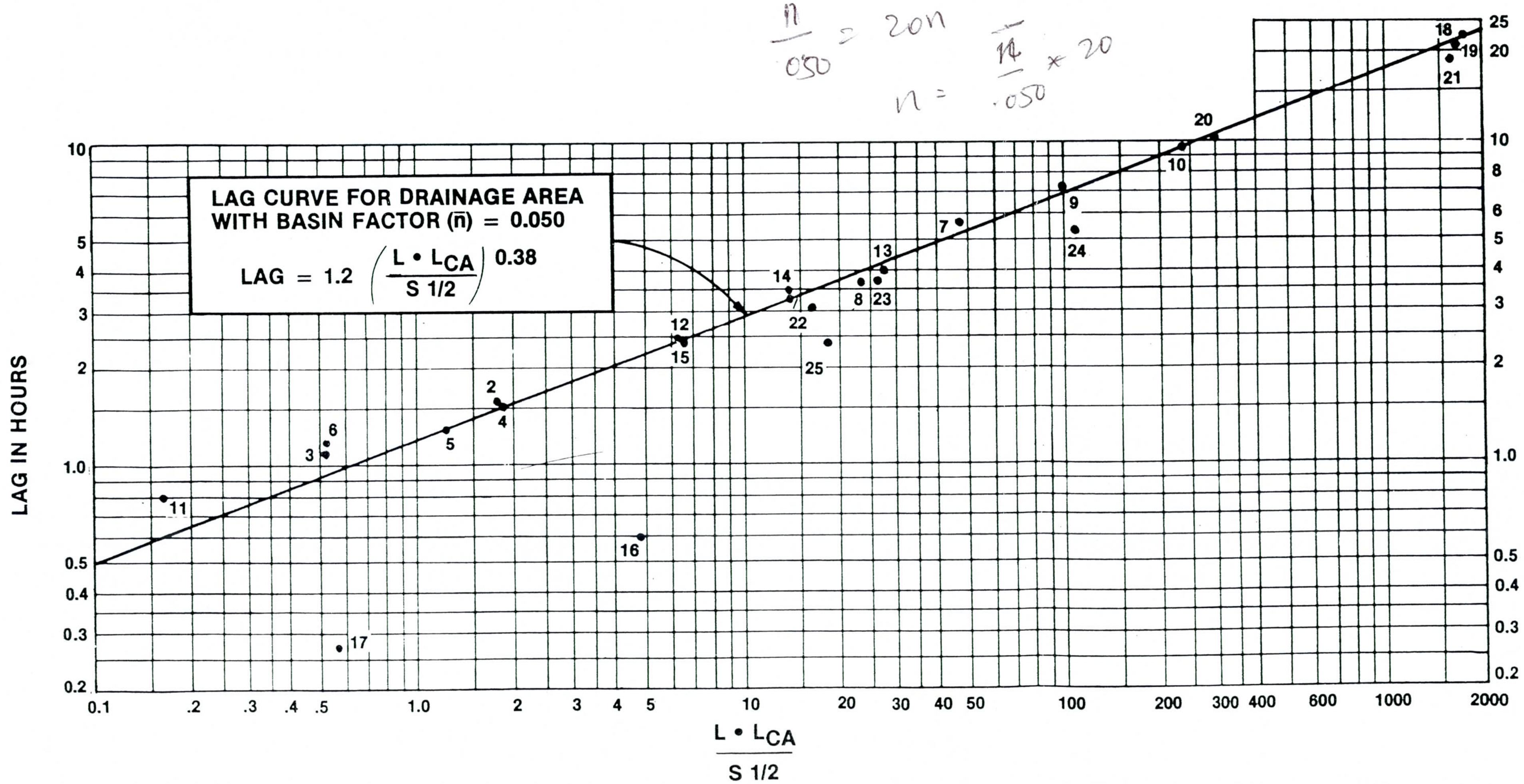
Note that in these two modified Snyder equations, the lag time is defined as the time elapsed from the centroid of the unit rainfall excess to the centroid of the unit hydrograph.

This is different from the original definition by Snyder in which lag time is defined as the time elapsed from the centroid of the unit rainfall excess to the peak of the unit hydrograph.

	CONTRIBUTING AREA SQ. MI.	L MILES	L_{cg} MILES	S FT./MI.	LAG HOURS	ESTIMATED $\bar{n}$
1. SAN GABRIEL RIVER AT SAN GABRIEL DAM, CALIF.	162.0	23.2	11.6	350	3.3	0.050
2. WEST FORT SAN GABRIEL RIVER AT COGSWELL DAM, CALIF.	40.4	9.3	4.3	450	1.6	.050
3. SAN ANITA CREEK AT SANTA ANITA DAM, CALIF.	10.8	5.8	2.5	690	1.1	.050
4. SAN DIMAS CREEK AT SAN DIMAS DAM, CALIF.	16.2	8.6	4.8	440	1.5	.050
5. EATON WASH AT EATON WASH DAM, CALIF.	9.5	7.3	4.4	600	1.3	.050
6. SAN ANTONIO CREEK NEAR CLAREMONT, CALIF.	16.9	5.9	3.0	1017	1.2	.055
7. SANTA CLARA RIVER NEAR SAUGUS, CALIF.	355.0	36.0	15.8	140	5.6	.050
8. TEMECULA CREEK AT PAUBA CANYON, CALIF.	168.0	26.0	11.3	150	3.7	.050
9. SANTA MARGARITA RIVER NEAR FALLBROOK, CALIF.	645.0	46.0	22.0	105	7.3	.055
10. SANTA MARGARITA RIVER AT YSIDORA, CALIF.	740.0	61.2	34.3	85	9.5	.055
11. LIVE OAK CREEK AT LIVE OAK DAM, CALIF.	2.3	2.9	1.5	700	0.8	.070
12. TUJUNGA CREEK AT BIG TUJUNGA DAM, CALIF.	81.4	15.1	7.3	290	2.5	.050
13. MURRIETA CREEK AT TEMECULA, CALIF.	220.0	27.2	10.3	95	4.0	.050
14. LOS ANGELES RIVER AT SEPULVEDA DAM, CALIF.	152.0	19.0	9.0	145	3.5	.050
15. PACOIMA WASH AT PACOIMA DAM, CALIF.	27.8	15.0	8.0	315	2.4	.050
16. ALHAMBRA WASH ABOVE SHORT STREET, CALIF.	14.0	9.5	4.6	85	0.6	.015
17. BROADWAY DRAIN ABOVE RAYMOND DIKE, CALIF.	2.5	3.4	1.7	100	0.28	.015
18. GILA RIVER AT CONNOR NO. 4 DAM SITE, ARIZ.	2840.0	131.0	71.0	29	21.5	.050
19. SAN FRANCISCO RIVER AT JUNCTION WITH BLUE RIVER, ARIZ.	2000.0	30.0	74.0	32	20.6	.050
20. BLUE RIVER NEAR CLIFTON, ARIZ.	790.0	77.0	37.0	65	10.3	.050
21. SALT RIVER NEAR ROOSEVELT, ARIZ.	4310.0	160.0	66.0	45	18.6	.050
22. NEW RIVER AT ROCK SPRINGS, ARIZ.	67.3	20.2	9.7	141	3.1	.045
23. NEW RIVER AT NEW RIVER, ARIZ.	85.7	23.2	13.6	145	3.7	.045
24. NEW RIVER AT BELL ROAD, ARIZ.	187.0	47.6	20.7	83	5.3	.037
25. SKUNK CREEK NEAR PHOENIX, ARIZ.	64.6	17.6	10.0	89	2.4	.033

$\frac{\bar{n}}{.050} = 20n$

$n = \frac{\bar{n}}{.050} \times 20$



(SOURCE: REF. 2)

GUIDE FOR ESTIMATING BASIN FACTOR ($\bar{n}$)

$\bar{n}$ = 0.200: DRAINAGE AREA HAS COMPARATIVELY UNIFORM SLOPES AND SURFACE CHARACTERISTICS SUCH THAT CHANNELIZATION DOES NOT OCCUR. GROUND COVER CONSISTS OF CULTIVATED CROPS OR SUBSTANTIAL GROWTHS OF GRASS AND FAIRLY DENSE SMALL SHRUBS, CACTI, OR SIMILAR VEGETATION. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$ = 0.050: DRAINAGE AREA IS QUITE RUGGED, WITH SHARP RIDGES AND NARROW, STEEP CANYONS THROUGH WHICH WATERCOURSES MEANDER AROUND SHARP BENDS, OVER LARGE BOULDERS, AND CONSIDERABLE DEBRIS OBSTRUCTION. THE GROUND COVER, EXCLUDING SMALL AREAS OF ROCK OUTCROPS, INCLUDES MANY TREES AND CONSIDERABLE UNDERBRUSH, NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$ = 0.030: DRAINAGE AREA IS GENERALY ROLLING, WITH ROUNDED RIDGES AND MODERATE SIDE SLOPES, WATERCOURSES MEANDER IN FAIRLY STRAIGHT, UNIMPROVED CHANNELS WITH SOME BOULDERS AND LODGED DEBRIS. GROUND COVER INCLUDES SCATTERED BRUSH AND GRASSES. NO DRAINAGE IMPROVEMENTS EXIST IN THE AREA.

$\bar{n}$ = 0.015: DRAINAGE AREA HAS FAIRLY UNIFORM GENTLE SLOPES WITH MOST WATERCOURSES EITHER IMPROVED OR ALONG PAVED STREETS. GROUND COVER CONSISTS OF SOME GRASSES WITH APPRECIABLE AREAS DEVELOPED TO THE EXTENT THAT A LARGE PERCENTAGE OF THE AREA IS IMPERVIOUS.

TERMINOLOGY

L = LENGTH OF LONGEST WATERCOURSE.

L_{ca} = LENGTH ALONG LONGEST WATERCOURSE, MEASURED UPSTREAM TO POINT OPPOSITE CENTER CENTER OF AREA.

S = OVER-ALL SLOPE OF LONGEST WATERCOURSE BETWEEN HEADWATER AND COLLECTION POINT.

LAG = ELAPSED TIME FROM BEGINNING OF UNIT PRECIPITATION TO INSTANT THAT SUMMATION HYDROGRAPH REACHES 50% OF ULTIMATE DISCHARGE.

$\bar{n}$ = VISUALLY ESTIMATED MEAN OF THE n (MANNING'S FORMULA) VALUES OF ALL THE CHANNELS WITHIN AN AREA.

NOTE:
TO OBTAIN THE LAG (IN HOURS) FOR ANY AREA, MULTIPLY THE LAG OBTAINED FROM THE CURVE BY:

$\frac{\bar{n}}{0.050}$ OR 20n

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, AZ

LAG RELATIONSHIPS

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

DIMENSIONLESS UNIT HYDROGRAPHS FOR MODIFIED SNYDER LAG EQUATIONS

Los Angles District - U.S. Army Corps of Engineers

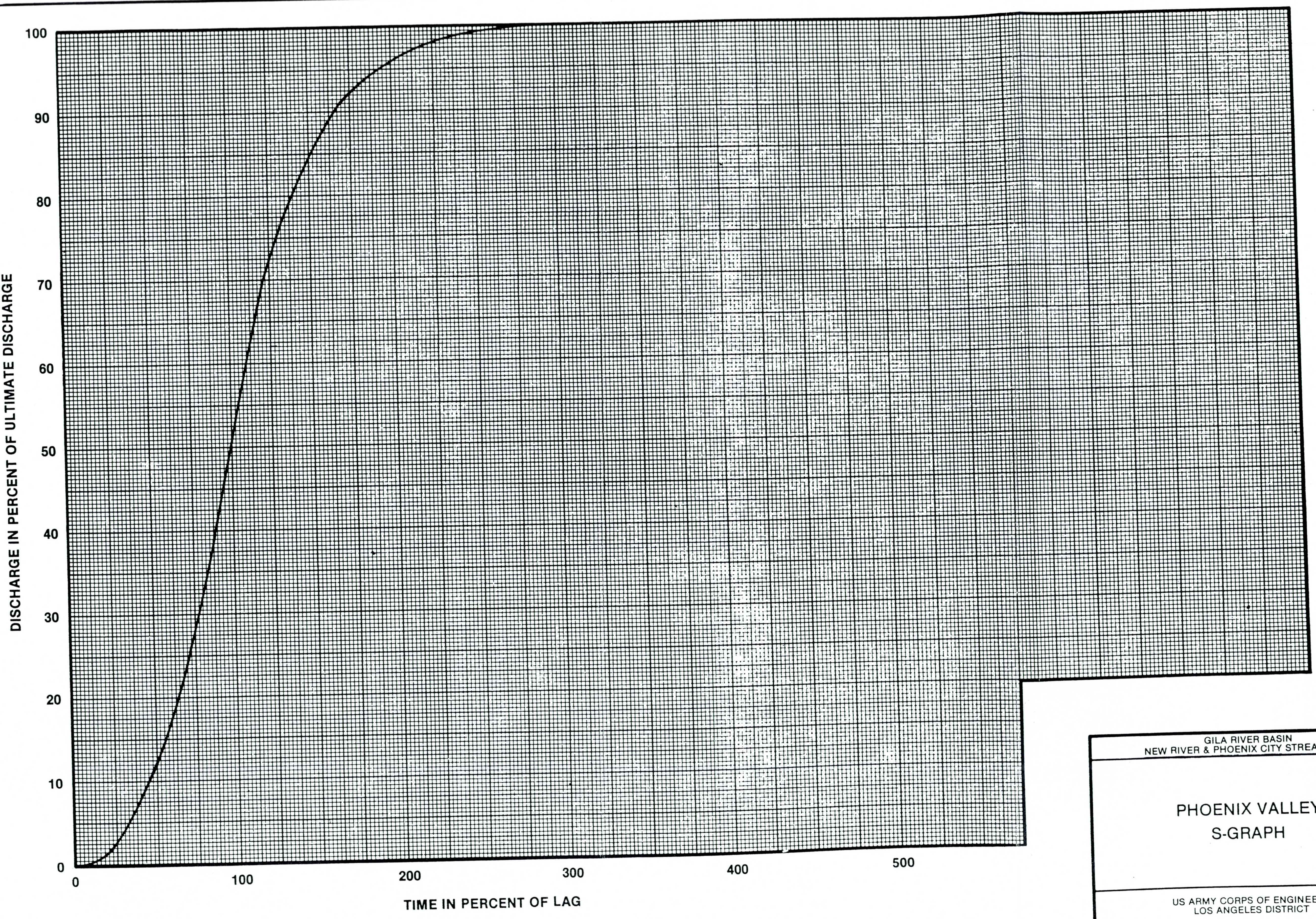
- Dimensionless S-graph data are available for the areas under the jurisdiction of the Los Angeles District in Arizona, California, Colorado, Nevada and New Mexico

U.S. Bureau of Reclamation

- Dimensionless unit hydrographs and S-graph data are available for the various areas under the USBR jurisdiction. They are for the areas in

    - the Great Plains
    - the Rocky Mountains
    - Southwest Desert, Great Basin and Colorado Plateau
    - Sierra Nevada, Coast and Cascade Ranges
    - urban basins

* "Flood Hydrology Manual", 1989 by USBR.

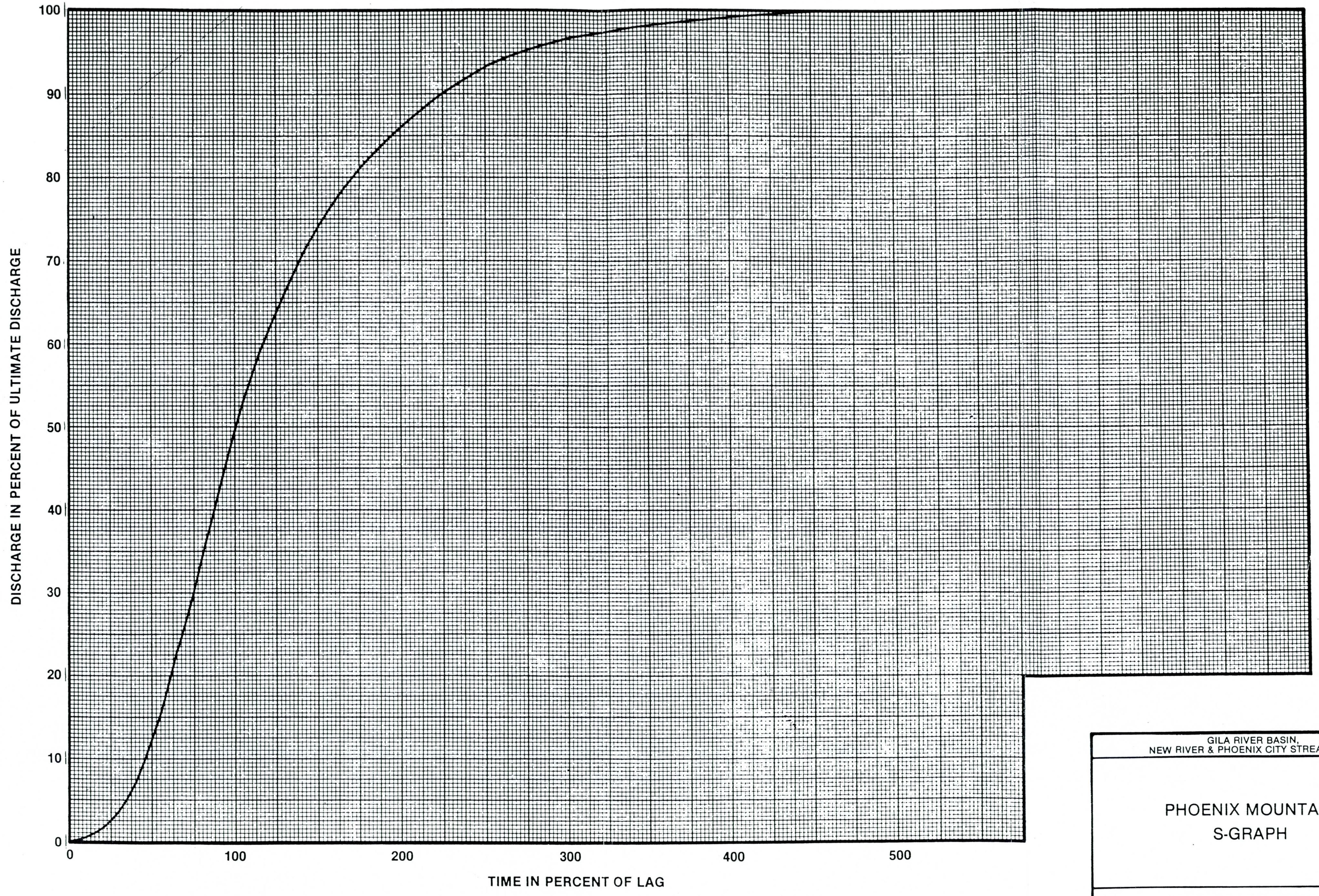


(SOURCE: REF. 2)

GILA RIVER BASIN
NEW RIVER & PHOENIX CITY STREAMS, AZ.

PHOENIX VALLEY
S-GRAPH

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT



(SOURCE: REF. 2)

GILA RIVER BASIN,
NEW RIVER & PHOENIX CITY STREAMS, AZ.

PHOENIX MOUNTAIN
S-GRAPH

US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

DERIVATION OF SNYDER'S C_t & C_p FROM OBSERVED FLOOD HYDROGRAPHS

- Snyder's Equations:

  $t_p = C_t (L\ L_{ca})^{0.3}$ and $Q_p = (640 C_p A) / t_p$

- the unit hydrograph developed using these equations is associated with a unit duration (t_r) equal to ($t_p/5.5$)

- t_p and t_r are defined as the standard lag and standard duration of the Snyder Unit Hydrograph, respectively

- develop unit hydrographs from observed hyetographs and correponding flood hydrographs

- from each unit hydrograph, measure the lag time ($t_{p,R}$) and note the duration of the unit rainfall excess (t_R) From observed hydrograph

- for the same basin, the measured basin lag time is a function of the duration of the rainfall excess. Therefore, the measured lag time would not necessarily be the same

- to convert the measured lag time to the standard lag time (t_p), Snyder suggested that

$$t_p = t_{p,R} + 0.25\ (t_r - t_R)$$

Since $\quad t_r = (t_p/5.5)$

Therefore,

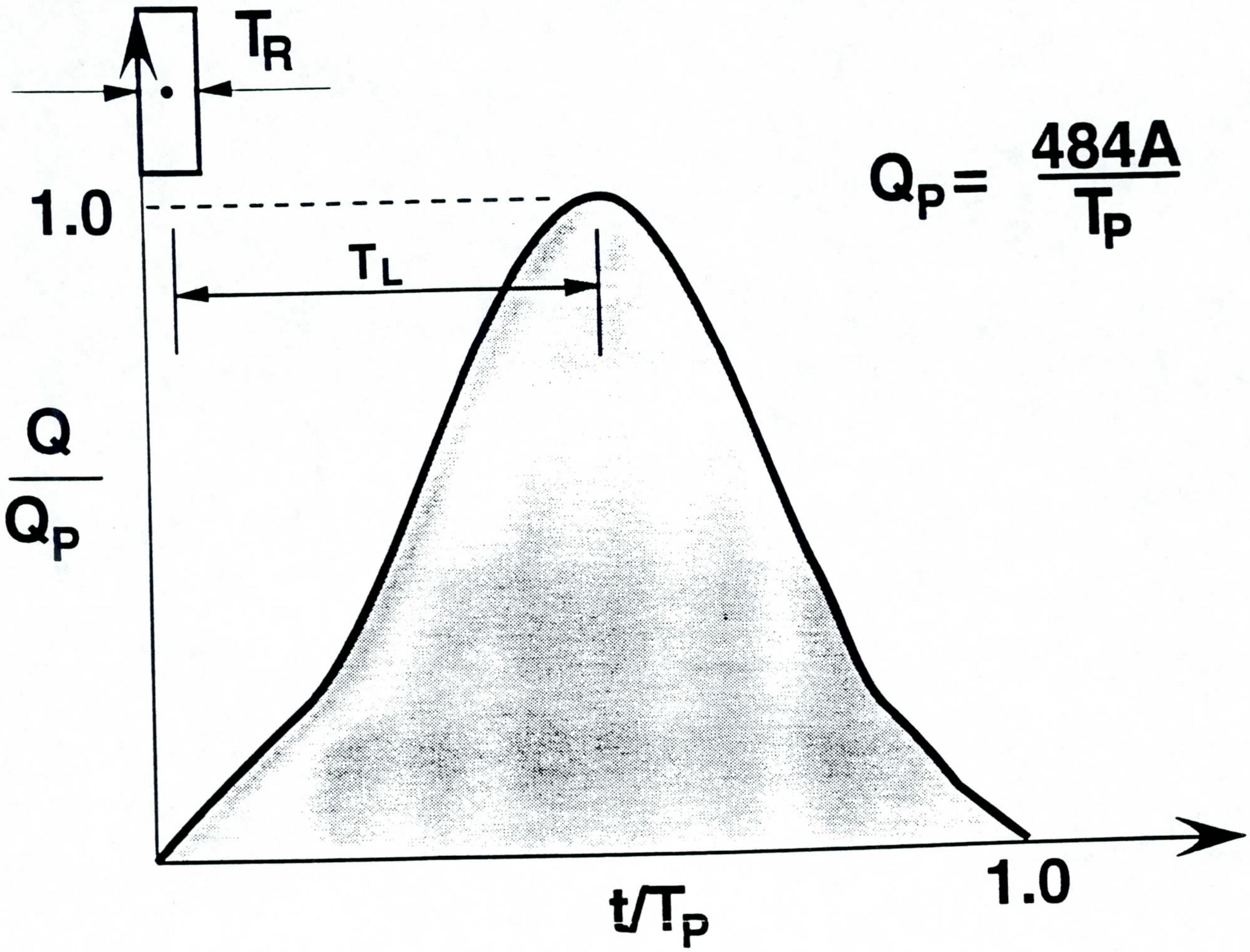
$$t_p = (t_{p,R} - 0.25\ t_R) / 0.955$$

Since $\quad C_t = t_p / (L\ L_{ca})^{0.3}$

$$C_t = (t_{p,R} - 0.25\ t_R) / [\ 0.955\ (\ L\ L_{ca}\ )^{0.3}]$$

$$640\ C_p = (Q_{p,R}\ \ t_{p,R}) / A$$

SCS DIMENSIONLESS UNIT HYDROGRAPH



$$Q_P = \frac{484A}{T_P}$$

$$T_P = T_L + 0.5\,T_R$$

$$T_L = \text{Lag Time} = \frac{L^{0.8}(S+1)^{0.7}}{1900\,Y^{0.5}}$$

A ≤ 2000 Acre

L (ft)

$$S = \frac{1000}{CN} - 10$$

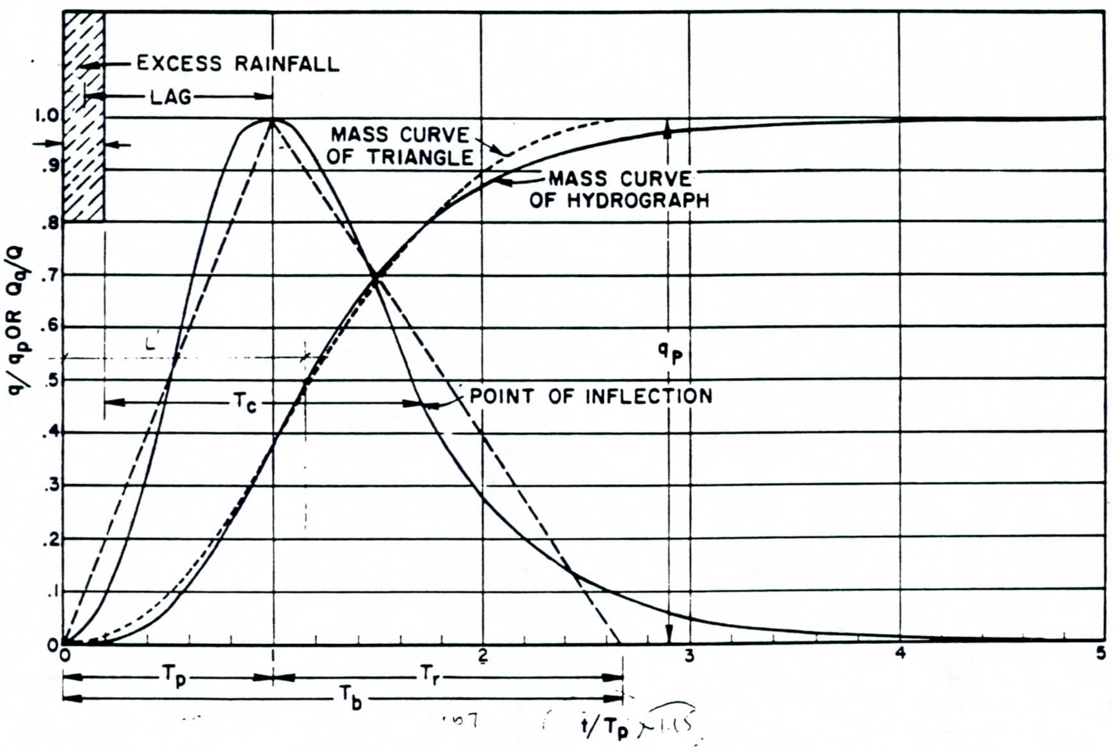
Y = SLOPE (%)

SCS DIMENSIONLESS UNIT HYDROGRAPH

Concept

- 37.5% of the total volume of the SCS dimensionless unit hydrograph is under the rising limb of the unit hydrograph
- has a point of inflection approximately 1.7 times the time-to-peak (T_p)

defined from begining of rainfall



$$0.5\ (q_p\ T_b) = 1.0$$

$(0.5\ q_p\ T_p)\ T_b = T_p$

since $T_p + T_r = T_b$

and $(0.5\ q_p\ T_p) = 0.375$ by definition

$0.375\ (T_p + T_r) = T_p$

$T_r = (0.625\ T_p) / 0.375 = 1.67\ T_p$

$Q = 0.5\ (q_p\ T_p) + 0.5\ (q_p\ T_r) = 0.5\ q_p(T_p + T_r)$

$q_p = 2Q / (T_p + T_r)$

let $K = 2 / (1 + T_r/T_p)$

therefore, $q_p = KQ/T_p$

Converting Equation $q_p = KQ/T_p$ to units commonly used in hydrology, such as time unit in hour, area unit in square miles, flow unit in cubic feet per second,

$$q_p = (645.33\ K\ A\ Q) / T_p$$

since $T_r = 1.67\ T_p$, $K = 0.75$ and

$$q_p = (484\ A\ Q) / T_p$$

- Note that the constant "484", the peak rate factor, is valid only for the shape of the dimensionless unit hydrograph assumed by SCS. It is known to vary from 300 to 600.

For flat land or very steep land do not use SCS Method

Defining $\quad Lag = 0.6\ T_c$

and $\quad$ the point of inflection $(T_c + \Delta D) = 1.7\ T_p$ T_R

where $\quad \Delta D$ is the duration of the unit rainfall excess associated with the unit hydrograph in question, or the unit duration of the unit hydrograph

Since $\quad 0.5\ (\Delta D) + Lag = T_p$ (see figure)

Therefore, $\quad T_c + \Delta D = 1.7\ \{0.5\ (\Delta D) + Lag\}$

and $\quad T_c + \Delta D = 1.7\ \{0.5\ (\Delta D) + 0.6\ T_c\}$

$\Delta D = 0.133\ T_c$ or 13.3% of T_c

or $\quad \Delta D = 0.22\ Lag$ or 22% of Lag. or Lag shorter than T_c.

- to preserve the accuracy of the flood hydrograph developed, the selection of ΔD must follow this criterion. A smaller variation in ΔD is permissible, however, it should not be greater than 0.25 Lag or smaller than 0.2 Lag

- for practical applications, ΔD should not be less than 5 minutes

- SCS suggests that for areas less than 2,000 acres:

$$Lag = \{L^{0.8}\ (S+1)^{0.7}\}\ /\ \{1900\ Y\}^{0.5}$$

where $\quad$ L is in feet, $S = \{(1000/CN) - 10\}$ and Y is the average basin slope in %

REMARKS

- need to be aware that an identical term used in the various time of concentration and lag equations may be defined differently from one equation to the other

- adjustments may have to be made if we use the term out of context

    - for example, using a lag time defined by the modified Snyder equation suggested by the Los Angeles District of the U.S. Army Corps of Engineers and use it to compute unit hydrograph by the SCS method

    - we would have to reduce the lag time so estimated by about 11% to account for the different definition of lag time in these two approaches

Table 16.1 Ratios for dimensionless unit hydrograph and mass curve.

Time Ratios (t/T_p)	Discharge Ratios (q/q_p)	Mass Curve Ratios (Qa/Q)
0	.000	.000
.1	.030	.001
.2	.100	.006
.3	.190	.012
.4	.310	.035
.5	.470	.065
.6	.660	.107
.7	.820	.163
.8	.930	.228
.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000

SNYDER METHOD FOR DERIVING SYNTHETIC UNIT HYDROGRAPHS

1. Need for Synthetic Unit Hydrographs

2. Properties of Unit Hydrographs (observed runoff characteristics)

   a. Magnitude of peak

   b. Lag time between rainfall and peak or other definitive points such as centroid of hydrograph or inflection point on recession limb of hydrograph

   c. Shape - rising and recession limbs of hydrograph

   d. Length of base

3. Basin Characteristics

   a. Valley and channel storage

   b. Basin configuration

   c. Topographical features

   d. Meteorological conditions

4. Synthetic Unit Hydrograph Procedures

   a. Snyder Method

b. Clark Method

c. Other

5. Snyder Unit Hydrograph Equations (see attached handout H-41-4)

$$t_p = C_t \left(LL_{ca}\right)^{0.3} \qquad C_t = \frac{T_{pR} - .25\, t_R}{.955\left(L \cdot L_{ca}\right)^{.3}}$$

$$t_r = \frac{t_p}{5.5} \qquad \left[640C_p\right] = \frac{Q_{pR}\, t_{pR}}{A}$$

$$Q_p = \frac{\left[640C_p\right]A}{t_p} \qquad t_{pR}^{*} = C_t(.955)\left(L^{*} \cdot L_{ca}^{*}\right)^{.3} + .25t_R^{*}$$

$$t_{pR} = t_p + .25\left(t_R - t_r\right) \qquad Q_{pR}^{*} = \frac{\left[640C_p\right] A^{*}}{t_{pR}^{*}}$$

6. A mathematical formula for determining the ordinates of the unit hydrograph (refer to Paper No. 9 of reference c).

7. Synthetic Unit Hydrograph Slope

8. Relationships Between Unit Hydrograph and Basin Characteristics

a. Drainage area

b. Stream slope - S

c. Equivalent stream slope - S_{st} (refer to Appendix C of reference b)

d. Stream Length - L or L_{ca}

e. Length / $\sqrt{S}$ or length / $\sqrt{S_{st}}$

f. $L \cdot L_{ca} / \sqrt{S}$ (Van Sickle)

g. $C\,(L/\sqrt{S})^K$ (Chow)

References:

a. Engineer Manual, EM 1110-2-1405, Flood Hydrograph Analyses and Computations

b. Civil Works Investigations Project 152, Report on Unit Hydrographs, Part 1, Principles and Determinations

c. Proceedings of a Seminar on Urban Hydrology

SNYDER SYNTHETIC UNIT HYDROGRAPH

The unit hydrograph has been shown to be a practical tool for converting rainfall excess to direct runoff. The purpose of this handout is to describe a technique for deriving unit hydrographs for basins for which streamflow records are not available. To do this requires a relationship between the physical characteristics of a drainage basin and the basin's unit hydrograph.

Method

Key parameters that can be used to define a unit hydrograph are peak flow, timing of the peak, and the degree of concentration of runoff near the peak. The unit hydrograph can be sketched in once the above parameters have been determined. In the Snyder method, the timing index is the basin lag, which is defined as the time between the centroid of the unit rainfall excess and the unit hydrograph peak.

Snyder's method is usually employed in regions where gaging records exist on only some of the streams of interest. The solution to this type of problem can be accomplished using the following method:

A. From examination of the drainage basin maps, the following values are determined for both the gaged and the ungaged basin:

(1) L, the main channel length from the outflow point to the upstream watershed boundary, in river miles,

(2) L_{ca}, the main channel length from outflow point to a point opposite the center of gravity of the river basin, in river miles, and

(3) A, the drainage area in square miles.

B. Using a known unit hydrograph for the gaged basin, the following values shown in Figure 1 are determined as follows:

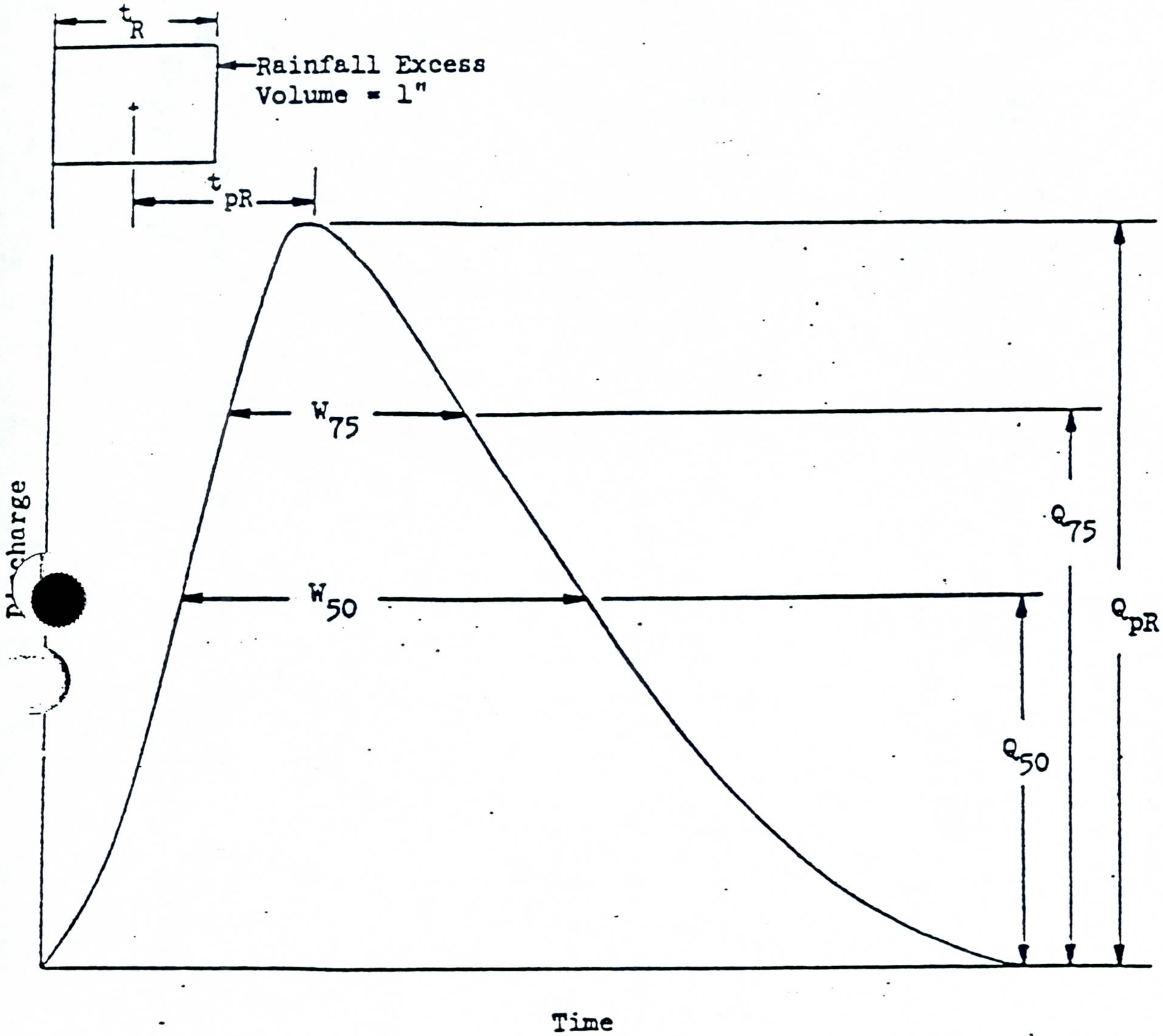


Fig. 1 - Definition of Unit Hydrograph Properties

(1) t_R, the unit rainfall duration in hours,

(2) t_{pR}, the time lag between mid-point of unit rainfall excess and the unit hydrograph peak in hours, and

(3) Q_{pR}, the peak rate of discharge of the unit hydrograph in cfs.

C. Snyder found that the lag (t_{pR}) of a basin measured from unit hydrographs was not constant. The measured basin lag depended on the duration of the rainfall excess as well as physical characteristics of the basin. Thus, in a given basin, if several unit hydrographs were available with different durations, each one would have a different lag (t_{pR}). Snyder adopted a "standard" lag which would be characteristic of a given basin. This lag, t_p, was defined as the lag of a unit hydrograph for which the ratio of t_p to t_r is 5.5, that is,

$$t_r = \frac{t_p}{5.5} \tag{1}$$

where

t_r = Snyder's standard duration, hrs.
t_p = Snyder's standard lag, hrs.

D. To reduce the measured unit hydrograph lag (t_{pR}) to the basin standard lag (t_p), the following relation was given:

$$t_p = t_{pR} + .25\ (t_r - t_R) \tag{3}$$

where

t_{pR} = measured basin lag from unit hydrograph
t_R = duration of unit hydrograph

However, at the standard lag, equation (1) applies, substituting (1) in (2) yields

$$t_p = \frac{(t_{pR} - .25\ t_R)}{.955} \tag{3}$$

Therefore, we may take any measured lag (t_{pR}) for a unit hydrograph of duration (t_R) and determine Snyder's standard lag for the basin.

E. Snyder found that the standard lag (t_p) for a basin could then be related to basin characteristics as:

$$t_p = C_t\,(L \times L_{ca})^{.3} \tag{4}$$

where

C_t = a regional coefficient dependent upon basin slopes, stream patterns, shape, and other properties.

F. It is then possible to solve equation (4) for C_t.

$$C_t = \frac{t_p}{(L \times L_{ca})^{.3}} \tag{5}$$

Thus, regional values of C_t may be determined from known unit hydrographs. In practice, it is convenient to combine equations (3) and (5), yielding:

$$C_t = \frac{(t_{pR} - .25\ t_R)}{.955\ (L \times L_{ca})^{.3}} \tag{6}$$

Equation (6) may be used to determine C_t from a given unit hydrograph of duration (t_R) and lag (t_{pR}).

G. Snyder also found that the peak discharge of a unit hydrograph (Q_{pR}) could be expressed as a function of its lag (t_{pR}) as follows:

$$Q_{pR} = \frac{[640C_p]\ A}{t_{pR}} \qquad (7)$$

where

$[640C_p]$ = a regional coefficient dependent on basin slopes, stream patterns, shape, and other properties

H. Solving equation (7) for $640C_p$:

$$640C_p = \frac{Q_{pR}\ t_{pR}}{A} \qquad (8)$$

Equation (8) may be used to determine regional values of $640C_p$ by fitting to a known unit hydrograph peak discharge (Q_{pR}), lag (t_{pR}) and basin area (A).

I. With coefficients C_t and $640C_p$ available, we may then determine unit hydrographs for watersheds with similar characteristics (i.e., basin slopes, stream patterns, etc.). Applying equations (4) and (1) directly will yield Snyder's standard unit hydrograph lag (t_p^*) and duration (t_r^*). (NOTE: The asterisk indicates values that apply to the new or ungaged basin.) However, only by chance would the duration (t_r^*) be equal to the duration required in a particular study. It is convenient to combine equations (4), (1), and (2), yielding:

$$t_{pR}^* = C_t\ (.955)\ \left(L^* \times L_{ca}^*\right)^{.3} + .25\ t_R^* \qquad (9)$$

Thus, equation (9) would give the lag (t_{pR}^*) for an ungaged area with lengths L^* and L_{ca}^* for a specified duration (t_R^*). Requirements of a given study may dictate what duration (t_R^*) to use. In order to provide sufficient points on the rising limb of the unit hydrograph, it is suggested that the values of t_{pR}^* and t_R^* should be in agreement with those shown in Table 1. (Ref. EM-1110-2-1411, Plate #11)

Table 1

t_{pR}^* (hr)	t_R^* (hr)
4-6	1
6-12	2
12-16	3
16-up	6

Equation (10) would then give the unit hydrograph peak discharge (Q_{pR}^*) for the ungaged area.

$$Q_{pR}^* = \frac{640 C_p A^*}{t_{pR}^*} \tag{10}$$

J. An adjunct to the Snyder method developed by the Corps of Engineers involves a technique to determine the width of the unit hydrograph in hours at 50 and 75 percent of the peak flow, as shown in Figure 1. The widths are represented by W_{50} and W_{75}, respectively.

A point can be plotted on Figure 2 for W_{50} versus Q_{pR}/A for the gaged basin. Draw a straight line parallel to the envelope curve shown on Figure 2 and through the point determined above. Using the straight line as a regional relation and the Q_{pR}^*/A^* for the ungaged basin, the W_{50}^* for the ungaged basin can be determined. By repeating step (7) using the W_{75} from the gaged basin, the W_{75}^* for the ungaged basin can be determined.

The proportioning of W_{50}^* and W_{75}^* relative to the location of the peak should be similar to that of the unit hydrograph of the gaged basin.

Snyder's method can also be used in areas of no hydrologic data by assuming values of C_t and $640C_p$ and using Step I for unit hydrograph determination. C_t has been found to vary from .4 to 8.0. C_p ranges from 200 to 600. In step J, the envelope curves may be used for approximate solutions. Often an indication of t_{pR}^* can be found from a quick field examination of the basin lag during a rainstorm.

The distance L_{ca} may be taken as the mean distance off of an accumulated-area vs. distance curve, or it may be measured along the principal stream channel to a point approximately opposite the center of gravity of the basin. The location of the center of gravity of the drainage area may be determined by suspending a cardboard outline of the drainage area by means of a pin near the edge, drawing a vertical line, and then rotating the cardboard approximately 90° and drawing a second vertical line. The intersection of the two lines is the center of gravity of the area.

Example

Using the above method, the following problem can be solved as shown.

Given data for basin A:

t_{pR} = 6 hours	A = 98 sq. miles	Q_{pR} = 9550 cfs
L = 24 miles	L_{ca} = 10 miles	t_R = 2 hours
W_{75} = 2 hours	W_{50} = 4.1 hours	

Given data for basin B:

A^* = 62 sq. miles	L^* = 15.6 miles	L_{ca}^* = 9.4 miles

Assuming basin B is hydrologically and meteorologically similar to basin A, calculate a synthetic one hour ($t_R^* = 1$) unit hydrograph for basin B.

Solution:

1. Applying equations (6) and (8) using the data for basin A

$$C_t = \frac{(t_{pR} - .25t_R)}{.955(L \times L_{ca})^{.3}}$$

$$= \frac{(6 - .25(2))}{.955(24 \times 10)^{.3}}$$

$$= 1.11$$

$$[640C_p] = \frac{(Q_{pR})(t_{pR})}{A}$$

$$= \frac{9550\ (6)}{98}$$

$$= 585$$

2. Using these derived coefficients for the ungaged area, B, equations (9) and (10) yield:

$$t_{pR}^* = C_t\ (.955)\left(L^* \times L_{ca}^*\right)^{.3} + .25t_R^*$$

$$= 1.11(.955)(15.6 \times 9.4)^{.3} + .25(1)$$

$$= \underline{4.98\ \text{hr.}}$$

$$Q_{pR}^* = \frac{640C_p\ A^*}{t_{pR}^*}$$

$$= \frac{585\ (62)}{4.98}$$

$$= \underline{7150\ \text{cfs}}$$

Using Figure 2 and the unit hydrograph widths from basin A:

$$q_{pR} = \frac{Q_{pR}}{A} = \frac{9550}{98} = 95.5 \text{ cfs/sq mi.}$$

The widths at 50 and 75 percent height of the given unit hydrograph (4.1 and 2 hours respectively) are plotted on Figure 2, and a regional curve is determined parallel to the envelope curve.

Using the unit peak discharge of the ungaged area:

$$q^*_{pR} = \frac{Q^*_{pR}}{A^*} = \frac{7150}{62} = 115 \text{ cfs/sq mi.}$$

Values of W^*_{50} and W^*_{75} may be read from Figure 2.

$W^*_{75} = 1.62$ hours

$W^*_{50} = 3.35$ hours

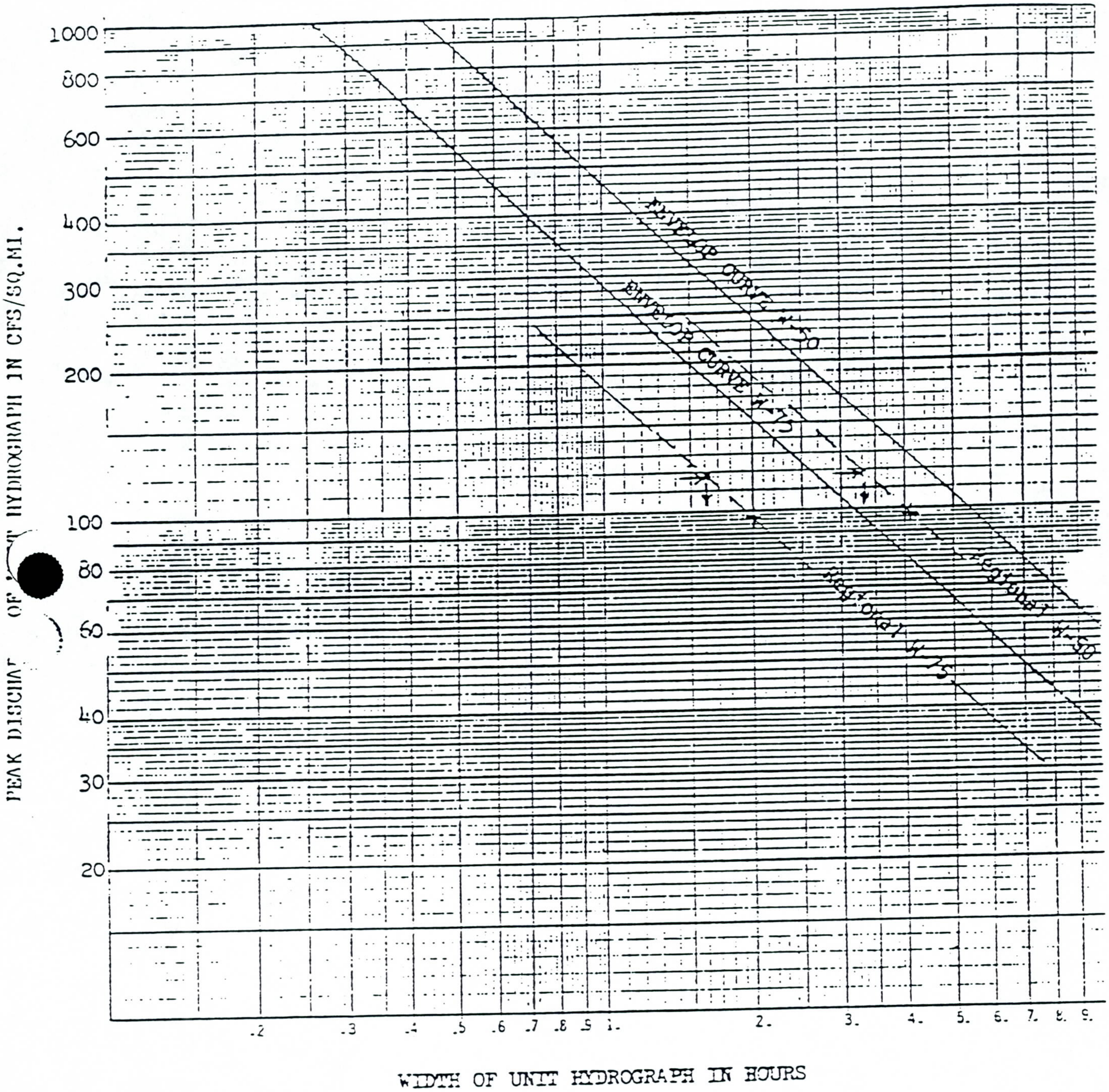


Fig. 2 - Width of Unit Hydrograph

SYNTHETIC UNIT HYDROGRAPH SLOPE

This method can be used to determine the required recession slope of the unit hydrograph when "n" ordinates to the point of inflextion are known.

The method is based upon the assumption that the recession of a unit hydrograph will be a straight line on semilog paper (see figure 1). The slope of the curve can be calculated as follows:

$$\text{slope} = K = \frac{\Delta UG}{\Delta t} = \frac{\log UG_{n+1} - \log UG_{n}}{\Delta t} = \frac{UG_{n+1}/UG_{n}}{\Delta t}$$

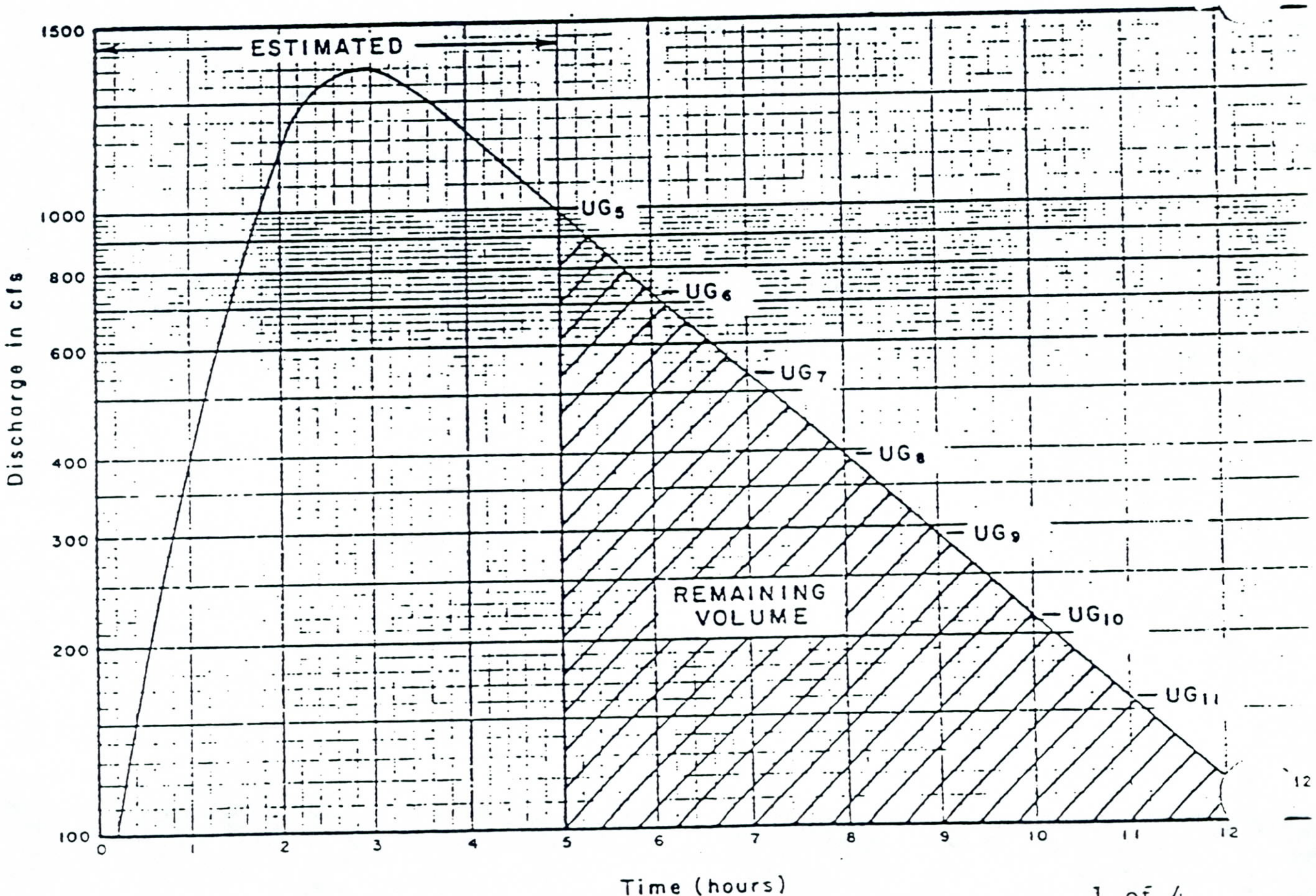
If Δt equals one tabulation interval, then the slope per tabulation interval can be calculated from the following:

$$K = UG_{n+1}/UG_{n} \qquad (1)$$

As shown in the proof on page 4, K can also be calculated as follows:

$$K = 1 - (UG_{n}/(UG_{n} + \text{remaining volume})) \qquad (2)$$

where: "remaining volume" is that portion of the total volume under the unit hydrograph after time "n" in cfs-hours.



Example

For a given basin having a drainage area of 10 sq. miles, the unit hydrograph ordinates calculated by the "Direct Solution Method" are as follows:

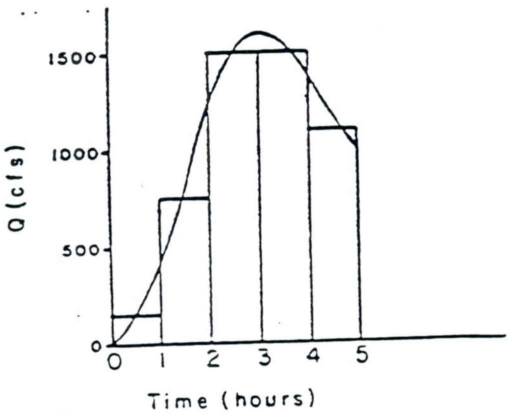
U.G. Ordinate number	Discharge (cfs)
1	400
2	1250
3	1600
4	1350
5	1000

The volume of one inch of rainfall excess over 10 square miles can be calculated as follows:

$$\text{Total Volume} = 1 \text{ inch} \times 10 \text{ sq. miles} \times \frac{\text{ft.}}{12 \text{ in.}} \times \frac{(5280)^2 \text{ft}^2}{\text{sq. mi.}} \times \frac{1 \text{ hr}}{3600 \text{ sec}}$$

$$= 6453 \text{ one-hour cfs}$$

By plotting the ordinates from the Direct Solution Method and summing the interpolated average period ordinates, the volume under the unit hydrograph up to the point of inflextion can be calculated as shown:



Period	Average Ordinate (cfs)
0-1	150
1-2	750
2-3	1500
3-4	1500
4-5	1100
Volume	5000 one-hr cfs

Remaining Volume = Total Volume - Volume up to inflextion point
= 6453 - 5000 = 1453 one-hr cfs

From equation 2: K = 1 - (1000/(1000 + 1453)) = .6

Therefore, the calculation of the recession ordinates using equation 1 results in the following

U.G. Ordinate Number	Discharge (cfs)
UG_6	.6 (1000) = 600
UG_7	.6 (600) = 360
UG_8	.6 (360) = 216
UG_9	.6 (216) = 130
UG_{10}	.6 (130) = 78

This method can be used to force fit the recession of the unit hydrograph. The method is based on a unit hydrograph volume of 1 inch and an assumed expotential recession curve. The unit hydrograph ordinates prior to the point of inflextion can be obtained by methods such as the "Direct Solution Method" or "Snyder's Synthetic Unit Hydrograph Method."

Proof of Formula Used in Synthetic Unit Hydrograph Slope

1. Basic Formulas

   a. By Definition $K = UG_{n+1}/UG_n$

   b. TO BE PROVED $K = 1 - UG_n/(UG_n + R_v)$, where:

   UG_n = Discharge on hydrograph at time n.

   UG_{n+1} = Discharge on hydrograph at time n+1

   Rv = Remaining volume under hydrograph from time n expressed in cfs-hrs.

2. Proof by identity technique:

   a. Assuming $K = 1 - UG_{n/(UG}n + Rv)$

   b. Substituting $UG_n + Rv\backslash(UG_n + Rv)$ for 1

   $$K = \frac{UG_n + Rv - UG_n}{UG_n + Rv} = \frac{Rv}{UG_n + Rv}$$

   c. Using the definition of $Rv = UG_{n+1} + UG_{n+2} + --$

   $$K = \frac{UG_{n+1} + UG_{n+2} + ---}{UG_n + UG_{n+1} + UG_{n+2} + ---}$$

   d. Simplifying

   $K(UG_n) + K(UG_{n+1}) + K(UG_{n+2}) + -- = UG_{n+1} + UG_{n+2} + --$

   e. Since $K = UG_{n+1}/UG_n = UG n+2/UG_{n+1} ---$

   $UG_{n+1} + UG_{n+2} + --- = UG_{n+1} + UG_{n+2} + ---$

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CALIBRATION OF SNYDER COEFFICIENTS FOR PENNSYLVANIA[1]

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ABSTRACT: Unit hydrograph theory is one of the most widely used techniques to predict surface runoff. The present study is concerned with the Snyder unit hydrograph and the calibration of the Snyder coefficients for Pennsylvania.

Twenty-seven study basins were selected, located randomly across the state. With the rainfall and runoff recorded for several events for each basin (more than 500 events were analyzed) unit hydrographs were calculated and the Snyder coefficients determined. A map of the coefficients was drawn to illustrate the variability in the coefficients and two equations using multiple regression theory were developed. The unexplained variability of the coefficients suggests that upper and lower bounds on the peak flow might be placed on storm hydrographs developed for ungaged watersheds.

(KEY TERMS: unit hydrograph; Snyder coefficients; rainfall runoff; ungaged watersheds.)

INTRODUCTION

There are a number of methods that have been developed to generate a hydrograph, given excess rainfall data (Viessman, *et al.*, 1977). One of the most widely used and accepted techniques is that of unit hydrograph theory. The Clark, Snyder and the Soil Conservation Service methods are probably the three most recognized unitgraph theories in the literature (Clark, 1945; Snyder, 1938; Mockus, 1957).

The unit hydrograph combined with excess rainfall can provide a single storm hydrograph for any basin. However, in most drainage basins the unit hydrograph must be a synthetic unitgraph, since most watersheds have no recorded rainfall or runoff data.

The present study is concerned with the Snyder unitgraph. The Snyder method was selected because it is a procedure used by many engineers to predict hydrologic events. The method is also included in the U.S. Army Corps of Engineers Hydrologic Engineering Center's HEC-1 computer program (1973). This program is used in many hydrologic studies; however, sometimes the users of the program do not fully understand the limitations of the methods inherent in the program.

The Snyder synthetic unitgraph is dependent upon two parameters, the lag factor (C_t) and the peak flow factor (C_p). If regional values of these parameters could be found, then design storm hydrographs for any ungaged basin could be computed.

BACKGROUND DATA

For twenty-seven gaged watersheds throughout Pennsylvania, mean values of the Snyder coefficients, C_t and C_p, have been determined from this study. The basins, which were in close proximity to raingage stations, were located across the entire state (Figure 1). For each watershed, if the data from the primary recording raingage were missing or incomplete, an alternate raingage was selected based on its proximity to the basin as well as the influencing topographic features, such as a mountain range location. No basins with major regulation effects were used, and very little urbanization is evident with any of the watersheds studied.

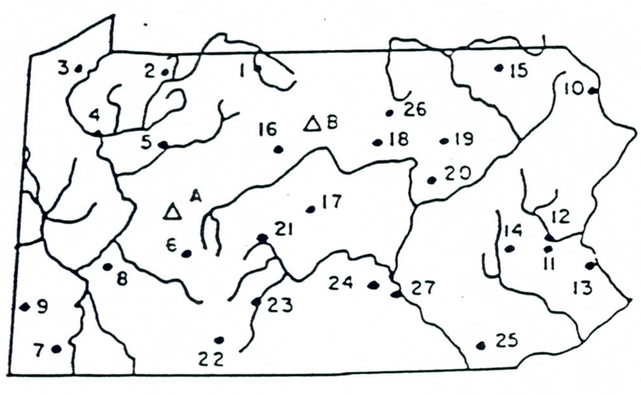

Figure 1. Location of Study Basins (1-27) and Test Basins (A and E).

For each basin, data for 20 to 25 representative events were available and mean values of C_p and C_t were calculated. Most of the streamflow data were taken from hourly and bi-hourly gage data recorded by the United States Geological Survey

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(USGS). Storms were picked from the records where there appeared to be significant rises in stage as a result of storm runoff. In addition to the readings at regular time intervals, the USGS data also contained the peak stage reading and the corresponding time of peak for each day.

In addition to the USGS data, hydrographs from previous research projects at The Pennsylvania State University (PSU, 1970) were utilized. The corresponding rainfall values for these events were taken from the Hourly Precipitation Data, collected and published by the National Oceanographic and Atmospheric Administration (NOAA), at Asheville, North Carolina.

The rainfall and streamflow data were input into the U.S. Army Corps of Engineers HEC-1 computer program, the Flood Hydrograph Package. In order to reproduce the original hydrograph as closely as possible, the program's optimization routine was employed to optimize the infiltration or loss rate parameters. For each storm event, a unit hydrograph was derived and the Snyder's coefficients were computed. In order to generate a unit hydrograph, the direct runoff must be separated from the base flow. In this study, the discharge just prior to storm runoff was used as a constant base flow for the entire storm hydrograph.

RESULTS OF INVESTIGATION

The computed mean coefficient values and corresponding standard deviations of C_p and C_t for each of the study basins are listed in Table 1. The frequency histograms of C_t and C_p values were fairly close in shape to the standard bell curve, or normal distribution in most cases. Some typical histograms are shown in Figures 2a and 2b. The plots of Figure 2a are typical of the majority of the histograms, while the plots of Figure 2b are representative of some of the more irregular results. For the statistical analysis and further application it was assumed that all distributions were approximately normal.

Regional trends for each of the coefficients are evident upon inspection of the data. Because of the size limitation of the fitures, the values of C_p and C_t could not be included on the map. The only results that appear to be inconsistent are those of basins 01-4515 and 01-4520 (Numbers 11 and 12 in Figure 1), near Allentown. The two adjacent basins have C_p values of 0.25 and 0.67, respectively, and C_t values of 1.01 and 2.47. These differences are relatively high in magnitude. The drainage areas are similar (80.8 and 75.8 square miles), but other basin characteristics are different, which may account for the variation in coefficients. Basin 01-4515 has approximately 50 percent less total stream length, which may account for the lower peak flow factor (C_p). The lower lag time coefficient (C_t), which results in early peaking hydrographs, might be explained by the fact that the slope of the main stream of basin 01-4515 is steeper than that of basin 01-4520. Also, there are some urban areas near the outlet of 01-4515, contributing to early peaking hydrographs. Other than these two basins, regional trends can be observed and the engineer should be able to pick representative values from the data and feel relatively confident in these selections. For a more detailed description of the basins see Spaeder (1979).

TABLE 1. Mean Snyder Coefficients and Standard Deviations for 27 Study Basins.

No.	Basin	$\bar{C}_p$	s*	$\bar{C}_t$	s	n**
1	03-0080	0.45	0.24	2.51	1.60	22
2	03-0152.8	0.28	0.21	1.59	1.18	20
3	03-0217	0.67	0.17	4.14	2.16	22
4	03-0252	0.38	0.28	2.22	1.36	22
5	03-0294	0.45	0.30	3.99	2.93	18
6	03-0422	0.32	0.17	2.14	1.31	21
7	03-0730	0.55	0.21	2.43	1.11	20
8	03-0840	0.24	0.13	1.30	0.59	13
9	03-111.5	0.35	0.22	2.40	1.83	26
10	01-4276.5	0.53	0.20	4.33	2.69	17
11	01-4515	0.25	0.16	1.01	0.62	26
12	01-4520	0.67	0.16	2.47	0.91	21
13	01-4595***	0.46	0.20	2.08	0.94	19
14	01-4707.2	0.35	0.23	1.93	1.57	24
15	01-5328.5	0.36	0.18	1.79	0.73	23
16	01-5435	0.59	0.18	4.00	1.55	23
17	01-5465	0.50	0.19	2.03	0.80	27
18	01-5497.8	0.40	0.23	2.41	1.49	19
19	01-5525	0.34	0.17	1.67	0.87	20
20	01-5536	0.49	0.21	2.77	2.06	25
21	01-5575	0.40	0.19	1.61	1.13	14
22	01-5597	0.23	0.12	1.78	0.67	18
23	01-5620	0.59	0.28	3.06	1.89	19
24	01-5680	0.52	0.18	2.15	0.99	23
25	01-5784	0.41	0.14	1.26	0.61	24
26	Liberty	0.44	0.20	1.07	0.70	15
27	Enola-1	0.56	0.18	3.09	1.27	12

*s represents the standard deviation of the sample.
**n represents the number of observations in the sample.
***Now subject to effects of regulation.

Variation of Coefficients Within Individual Basins

Theoretically, for any given basin, the calculated Snyder coefficients, C_p and C_t, should be approximately constant for all hydrographs. This was not found to be the case for any of the studied watersheds. The values were in most instances normally distributed and large variations from the mean were observed in some cases. In an attempt to explain these differences, the variations of the coefficients within three of the basins were examined.

Plots and multiple regression analyses were performed in order to determine how C_t and C_p varied with storm parameters. Seven parameters, including peak flow, initial flow, total storm rainfall, total rainfall excess, and duration of rainfall excess were chosen for the analysis. The MINITAB II program of The Pennsylvania State University Computation Center was used for the statistical analysis (Ryan, *et al.*, 1976). For each event, the seven predictor variables, as well as the values of C_p and C_t, were known. For each study basin, individual plots were made between C_t and each storm parameter, and between C_p and each storm parameter. Simple linear regressions were made corresponding to each plot, using the least squares criteria. There was no indication of a relationship, whatsoever, for any plot indicating that the variation could not be explained by the storm data.

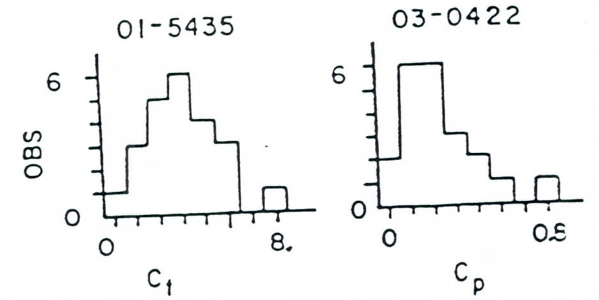

Figure 2a. Typical Distribution of Snyder Coefficients.

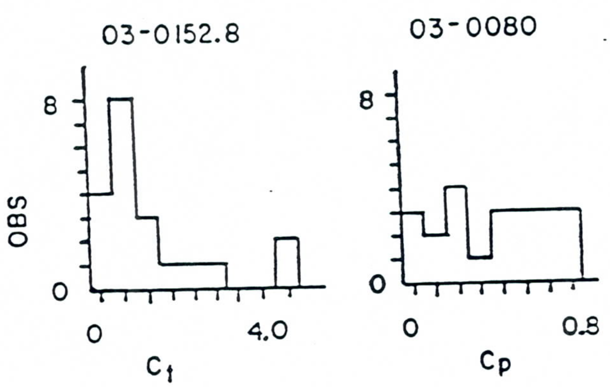

Figure 2b. Less Typical Distribution of Snyder Coefficients.

Description of the Regression Study

Table 1 lists the mean Snyder coefficients for the 27 study basins located through the State of Pennsylvania (Figure 1). From the map and Table 1, the values of C_t and C_p for any location in the State can be estimated. In order to aid further in the estimation of these coefficients, a regression analysis was conducted, from which C_t and C_p could be predicted based on watershed characteristics.

First, the appropriate watershed characteristics were selected. There were two criteria used in the selection of these basin parameters, or predictor variables. Only parameters that could logically affect the shape of the hydrograph, have a cause-effect relationship, were chosen, and the numerical values for these parameters had to be fairly easy to obtain or to compute.

The variables investigated include the drainage area (A), length of main channel (L), length of main channel to a point opposite the basin centroid (L_{CA}), the drainage density (DD), and the stream slope (S_1). A second channel slope term (SCE), is defined as the difference in elevation between the point on the basin divide and the basin outlet, divided by the length parameter LEXT, which is the length of the longest collector extended perpendicularly to a point on the basin divide. Other stream length parameters used included LSFOB, total length of first order streams, and LSGFOB, total length of streams greater than first order. The parameter LMAX is defined the maximum dimension of the watershed in any direction. All of these length and slope parameters were determined from USGS topographic maps. The percent wooded area (PWA) for each basin was determined by planimetering the forested areas denoted on the USGS topographic maps. Because the values of this parameter do not remain constant over the years, they were rounded to the nearest 5 percent for each basin, to allow for uncertainty. Since the exact descriptions of the various land uses of each basin were not known, approximations were made of the runoff curve numbers (CNs). For each watershed, a composite runoff curve number was calculated, based on the percentage of each soil group. A basin shape parameter (Shape) was created, which separated the basins into three categories, Shapes 1, 2, and 3. These numbers refer to shapes that are wide, circular, and long, respectively, with regard to the main stream. Pennsylvania is divided into three physiographic provinces: the Appalachian Plateau, the Ridge-and-Valley Region, and the Piedmont Province. The possibility of any relationships existing between the Snyder coefficients and the provinces was investigated. Upon initial inspection of the data there did not appear to be any relationships between physiographic province and C_p or C_t. However, in case any relationships did exist, the provinces were included in the regression study in the form of indicator variables.

Three computer programs were used in the regression analysis. They included MINITAB II and the Statistical Package (STPAC) (Stein, 1968), both of The Pennsylvania State University Computation Center, and the Statistical Analysis System (SAS) (Barr, *et al.*, 1976), of the SAS Institute, Raleigh, North Carolina. For each of the Snyder coefficients, C_p and C_t, the same procedure was followed. Initially, by the use of the MINITAB II program, the coefficient was plotted and regressed against each of the predictor variables, in order to show any possible relationships. (The variables were used both in natural forms and as log transforms.) Then, multiple regressions were made, using several combinations of the variables. It was found that for each coefficient a four to six variable model would be most appropriate, and so several candidate models were initially selected. Next, the STPAC program was employed. The "UPREG" option was used, which involves a step-up regression procedure. (This means that variables are added to the regression equation one at a time, depending on how they improved the "fit" of the regression line. They may also be deleted from the equation, if found to be not significant.) Again, several equations were found containing four to six predictor variables. Finally, the R^2 values for all possible regression equations (with four, five, and six variables) were determined. The SAS program, "RSQUARE" option, was employed here providing a good check on the equations found from the other methods.

Results of Regression Study

First, the coefficient C_p was investigated. When its average value, $\bar{C}_p$ was plotted and regressed against each of the predictor variables separately, there was modest correlation between this coefficient and the area, channel slope, and stream length parameters. From multiple regression, several predictive equations were found. The equation selected as "best" was:

$$\bar{C}_p = 0.907 + 0.0020(L \cdot L_{CA}) - 0.130(DD)$$

$$- 0.0613(SCE) - 0.0352(LEXT).$$

The fraction of explained variation, or R^2, was 0.75 for this equation. The equation was significant at the 99.9 percent level. Each coefficient of the equation was significant at the 95 percent level. One "outlier" basin, 03-0152.8, was deleted in the development of the above equations.

Next, the coefficient C_t was analyzed. The only parameters that showed even slight correlation with the average value of C_t, $\bar{C}_t$, were the slope terms (S_1, SCE) and physiographic province. Multiple regression equations were then searched for, again by the use of the three computer programs. The results were not as good as those of the C_p regressions, but were still reasonable. The selected predictive equation for the lag factor was:

$$\bar{C}_t = 18.6 + 0.0108(L \cdot L_{CA}) - 1.29(DD) - 0.464(SCE)$$

$$- 0.468(LMAX) - 0.150(CN).$$

The value of R^2 for this equation was 0.67. The coefficients were all significant at the 95 percent confidence level, and the equation itself at the 99.5 percent level. However, in the development of this equation, it was necessary to delete two outlier basins, 01-4276.5 and 01-5784.

The best overall predictive equation for C_t was a four-parameter model using $\bar{C}_p$, A, S_1, and CN, in which the most influential parameter was the $\bar{C}_p$ coefficient. The fraction of explained variance (R^2) was 0.78, with no basins discarded. The other parameters in this model were area (A), stream slope (S_1), and curve number (CN). However, the use of this model is questionable, since the main predictor variable itself comes from a regression equation.

Other investigators have found that the most important parameters in determining hydrograph shapes were area, drainage density, percent wooded area, stream slope, and a factor similar to the runoff curve number (e.g., Heerdegen, 1973; McSparran, 1968; Wolf, 1975). In this study, agreement was found with the use of the drainage density, slope, and curve number parameters, but variables LEXT, LMAX, and $L \cdot L_{CA}$ were found significant also. These last three parameters each correlated very highly with area. Also, percent wooded area, found to be important in the other studies, was found to correlate modestly with the curve number.

Only 22 of the study basins were used in the regression analysis. The two smallest basins (0.83 and 1.05 square miles) and the three largest (200, 685, and 756 square miles) were not included because of the limitation of the data available. These suggest a restriction on the regression equations base on drainage area: only basins ranging in size from 3.6 to 18 square miles are covered by the models. However, most basin would fall within this range.

From this regression study two models were developed fo estimating values of C_p and C_t for drainage basins. Howeve there did not seem to be any physical basis for either of thes equations.

Results of Tests on Other Basins

Two basins, not included in the original 27, were selecte according to the same criteria as the original basins, to test th study results. The basins were USGS watersheds 03-0345, A Little Mahoning Creek at McCormick, Pennsylvania; an 01-5445, B, Kittle Creek at Cross Fork, Pennsylvania. Thei locations are shown in Figure 1. For these watersheds, severa rainstorms were chosen, where the streamflow hydrograph were known. Hydrographs for these storm events were calcu lated according to the Snyder method, where the coefficient were determined by both the developed equations and the dat from Table 1. To test the validity of the results of this study the predicted hydrographs were compared to the actual hydrograph.

First, the Snyder coefficients for each basin were estimate using the map and Table 1. Second, the coefficients wer determined from the equations, after the required basin parameters were evaluated. Thus there were two different syntheti hydrographs developed for each event to be compared to th actual hydrograph.

Initially, the loss rate parameters were determined from th optimization routine of the HEC-1 program, in order to ac count for infiltration and other losses. They were then used i the development of the synthetic hydrographs, again by th use of the HEC-1 program.

The estimated Snyder values are listed in Table 2, alon with a summary of the results obtained from using them. Fc basin 03-0345, the results were slightly better for the regres sion equation estimates of C_t and C_p than for the map esti mates in three of the four cases. The map values yielded hy drographs that had peaks ranging from 0 to 46 percent belo the actual peaks. The hydrographs from the regression equa tion estimates had peaks ranging from 19 percent above to 34 percent below the actual peaks. The timing of the predicte peaks was relatively good in all cases. Figure 3 displays bot predicted hydrographs and the actual hydrograph for on event in watershed 03-0345.

For basin 01-5445, the C_t-value obtained from the regres sion equation was 0.10, which is unreasonably low. The co responding value of C_p was 0.32. Using these numbers, it not possible to construct a workable unit hydrograph. Thu the regression approach is not applicable for this basin.

The results from the regression approach highlight some c the limitations of the statistical analysis. The regression equa tions were developed for specific sites. When these equation are applied to other basins they may or may not be valid de pending upon the similarity of the basin with those used in th

TABLE 2. Summary of Test Results.

Date of Storm	Map Estimates: Peak Flow[a]	Map Estimates: Time of Peak[b]	Regression Equation Estimates: Peak Flow	Regression Equation Estimates: Time of Peak
	$C_p = 0.38, C_t = 2.5, t_p = 13.3$		$C_p = 0.29, C_t = 1.5, t_p = 8.0$	
BASIN 03-0345				
6-24-57	−30	−1	−13	−7
10-08-59	−46	+4	−34	−1
7-02-63	−40	−2	−28	−2
8-19-68	0	0	+19	−2
	$C_p = 0.52, C_t = 3.5, t_p = 18.5$		$C_p = 0.32, C_t - 0.10, t_p = 0.53$	
Date of Storm	Peak Flow	Time of Peak	Peak Flow	Time of Peak
BASIN 01-5445				
5-22-53	+28	+10	---[c]	---
10-13-55	+11	+6	---	---
7-28-58	+14	+10	---	---

[a]Percentage by which the predicted peak overestimates (+) or underestimates (−) the actual hydrograph peak flow.

[b]Time in hours by which the predicted peak lags (+) or precedes (−) the actual hydrograph peak.

[c]Not applicable; coefficients do not yield a workable unit hydrograph.

development of the equations. For basin 01-5445, the predicted peaks ranged from 11 to 28 percent above the actual peaks, and were 6 to 10 hours behind the true peaks.

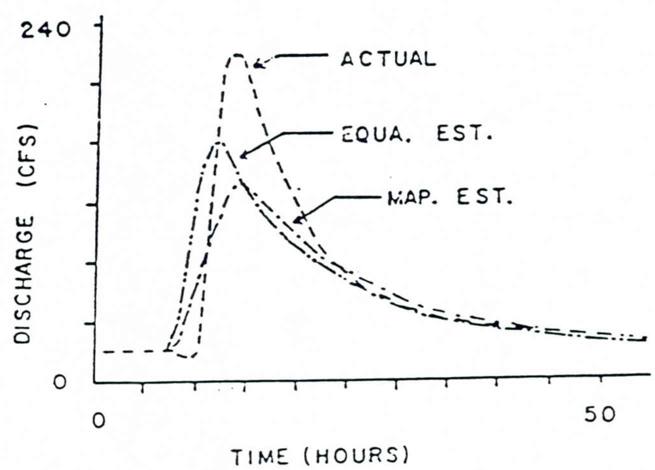

Figure 3. Plots of Actual and Estimated Hydrographs for Basin 03-0345, Storm of July 2, 1963.

The overall results for basin 03-0345 are slightly in favor of the regression equation approach. But from the results of the tests on basin 01-5445, it was found that the regression method can yield impractical numbers. It would have to be concluded that the map estimate approach, which provides the more e-liable numbers, would be the better of the two methods to use. The regression equations, which seem to have no physical basis, could be used as a check on the estimates for the coefficients.

SUMMARY AND CONCLUSIONS

The purpose of this study was to determine mean values of Snyder's unit hydrograph coefficients, C_t and C_p, for several basins across Pennsylvania, thus providing a means of computing design hydrographs for ungaged basins. The Snyder method was chosen because it is used in many hydrologic investigations and there are not much data available to estimate C_p and C_t. For the ungaged basins, Snyder's coefficients can be estimated from the maps or from the regression equations developed, relating C_p and C_t to basin characteristics. The design hydrographs would come from applying design rain-storm (computed from methods such as that of Kerr, *et al.*, 1970), to the synthetic unit hydrographs obtained with the estimated coefficients.

Twenty-seven study basins were selected, located randomly across the State. Watersheds were selected near recording rain-gages of the U.S. Weather Bureau. Hourly and bi-hourly streamflow values at stream gaging stations were recorded by the USGS. With the rainfall and runoff recorded for several events for each basin, unit hydrographs could be calculated and the Snyder coefficients determined. This was done with the aid of the HEC-1 computer program. Mean values of the two coefficients were then determined for each watershed. Regional trends were observed to some extent, although it appears that many more values would be needed in order to draw any definite conclusions.

There were variations in the coefficients from storm to storm within the individual basins, some of which were quite great. The variations were investigated for three of the watersheds, in order to relate C_t or C_p to storm variables, such as rainfall excess and antecedent precipitation. No strong relationship were found, although total rainfall, excess rainfall, and duration of excess rainfall did show some correlation with the coefficients. Sensitivity tests were made in order to examine the effects of using various values and combinations of values of the Snyder coefficients for a given basin. The coefficients were raised and lowered by 25 and 50 percent in various combinations. The greatest variations in unit hydrograph peaks were found to occur when Cp is overestimated and C_t is underestimated.

As a further aid in the estimation of the Snyder coefficients for ungaged watersheds, regression equations were found relating C_p and C_t to basin characteristics. Two equations were selected as the best equations for predicting C_p and C_t. There seemed to be no physical basis for either equation, however, which must be taken into account when applying them.

Finally, two test basins were selected and the Snyder coefficients were estimated according to two methods: from the regression equation sand from the coefficient data and map. Seven events were selected for these basins, where the rainfall and runoff data were known. Two synthetic hydrographs were computed for each storm, one using the regression values and one using the map estimated values, and compared to the actual hydrograph. For the hydrographs obtained from the map estimated coefficients, peak flows ranged from 46 percent below the actual value to 28 percent above. For the hydrographs obtained from the regression equation estimates, peak flows ranged from 34 percent below the actual peak to 19 percent above. However, for one of the two basins, an unreasonably low value of C_t was obtained from the equation, and thus the Snyder method could not be applied. From these tests, it would have to be concluded that the map estimate approach, which provides fairly good results, is more reliable than the regression method. The regression approach could be used as a check on the map estimates.

For any gaged basin, it can be concluded that the coefficients should be calculated in the conventional manner from several hydrographs. For an ungaged basin, the map estimates are more reliable than the regression method; however, the regression approach does give a second check.

Possibly the most significant conclusion concerns the accuracy of the technique. No one should be so unwise as to presume that the predicted hydrograph for an ungaged watershed is totally accurate. However, we can determine upper and lower bounds for the discharge values of the hydrographs and appreciate the complexity of the hydrologic process that we are trying to model.

ACKNOWLEDGMENTS

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THE SCS UNIT HYDROGRAPH

The first SCS concept of a synthetic unit hydrograph was a triangle as shown in Fig. H-1, with a specified rainfall excess duration D, runoff peak q_p, time to peak t_p and a base time $t_b = 8/3\ t_p$.

With all time terms expressed in hours, and the unit hydrograph defined as the runoff from 1 inch of precipitation excess from a drainage area A measured in square miles, a mass balance yields the equation

Fig. H-1. SCS Triang. Unit Hydrograph

$$1/2\ q_p\ t_b = 4/3\ q_p \left[\frac{\mathrm{ft}^3}{\mathrm{sec}}\right] t_p\ [\mathrm{hrs}] = \frac{1\ \mathrm{in} \times A\ [\mathrm{mi}^2] \times 5280^2\ \mathrm{ft}^2/\mathrm{mi}^2}{12\ \mathrm{in/ft} \times 3600\ \mathrm{sec/hr}}$$

or

$$q_p = \frac{484\,A}{t_p} \qquad (1)$$

The time to peak is defined as the sum of the lag time t_L (from the storm center to the runoff peak) and one-half the rainfall duration. The lag time had been found empirically to be approximated by the equation

$$t_L = \frac{L^{0.8}(s + 1)^{0.7}}{20\ Y^{0.5}} \qquad t_p = t_L + \frac{D}{2} \qquad (2)$$

in which t_L is expressed in hours,m L = length of drainage path in miles, Y = drainage path slope in ft/ft, and S = the SCS soil water storage capacity in inches, computed as a function of the curve number CN, namely

$$S = \frac{1000}{CN} - 10 \qquad (3)$$

The refined and finally accepted SCS unit hydrograph consists of a gamma distribution fitted over the original triangular unit hydrograph in such a way as to preserve the property that 3/8 of the runoff occurred prior to the time of peak t_p. The equation of the gamma distribution is

$$q = \frac{645\ A\ t^{\alpha}\ e^{-t/\beta}}{B^{\alpha+1}\ \Gamma\alpha\ +\ 1} \qquad (4)$$

Fig. H-2. SCS Synthetic Unit Hydrograph

in which q is the unit hydrograph ordinate in cfs,

A	=	drainage area in sq. mi.,
t	=	any time measured from the beginning of the unit hydrograph,
e	=	the base of the natural logarithm,
α, β	=	two parameters which determine the shape of the unit hydrograph, with the property α and $\beta = t_p$, and
$\Gamma\alpha+1$	=	the gamma function of $(\alpha+1)$ (equal to $\alpha!$ when α is an integer).

Expressing time in the dimensionless form $\tau = t/t_p$ and substituting eq. (1) into eq. (4), the latter reduces to the relative discharge equation

Hymo = Jimmie Williams

$$q/qp = \left[\tau\ e^{(1-\tau)}\right]^{\alpha} \tag{5}$$

To satisfy the requirement that 3/8 of the runoff occur prior to the time to peak t_p, the exponent α takes the value 3.70.

Using equations (1) to (5), the entire unit hydrograph can be constructed as demonstrated in the example below.

Example

A watershed has a drainage area of 15 mi^2 with a drainage path of length L = 6 mi and average slope Y = 0.04 ft/ft. For a curve number CN = 70, construct a 1-hr unit hydrograph.

1) Find the soil water storage capacity

$$S = \frac{1000}{70} - 10 = 4.29 \text{ inches}$$

2) Find the lag time

$$t_L = \frac{6^{0.8}\ 5.29^{0.7}}{20 \times 0.04^{0.5}} = 3.36 \text{ hrs.}$$

3) Find the time to peak.

$$t_p = t_L + D/2 = 3.36 + 0.5 = 3.86 \text{ hrs.}$$

4) Find the peak discharge rate.

$$q_p = \frac{484 \times 15}{3.86} = 1800 \text{ cfs}$$

5) Find the unit hydrograph ordinates at 1-hr. intervals, by using eq. 5 as shown in Table H-1.

Table H-1. SCS Unit Hydrograph for the Sample Watershed

t hrs	τ t/t_p	q/q_p	q cfs
0	0	0	0
1	0.26	0.106	199
2	0.52	0.525	988
3	0.78	0.900	1692
3.86	1.00	1.00	1880
4	1.04	0.997	1875
5	1.30	0.870	1636
6	1.55	0.661	1243
7	1.81	0.449	843
8	2.07	0.282	530
9	2.33	0.167	313
10	2.59	0.094	177
12	3.11	0.027	51
15	3.89	0.003	6.5
20	5.18	0.00008	0.16

<u>SCS Method for Rural Watersheds</u>

The SCS Unit Hydrograph was developed in the dimensionless form described in Fig. 16.1 (SCS National Engineering Handbook) and Table 16.1, with time expressed as a ratio of time to peak t_p and discharge as a ratio of peak discharge Q_p.

As in the Snyder method, the time to peak

$$t_p = t_l + 0.5\ t_R$$

in which

t_ℓ = t_l = lag time = $\dfrac{L^{0.8}\ (S\ +\ 1)^{0.7}}{1900\ Y^{0.5}}$, in hours

L = length of drainage path, in feet

Y = slope of drainage path, in percent

S = soil water storage capacity, in inches, as computed as a function of SCS curve number, CN, as follows:

$$S = \frac{1000}{CN} - 10$$

The peak discharge rate of the unit hydrograph, in cfs/inch, is computed by the equation

$$Q_p = \frac{484\ A}{t_p}$$

in which A = the watershed are in sq. mi., and t_p = time to peak.

An example of the SCS unit hydrograph construction is presented below and in Table 16.2, for a watershed of drainage area A = 4.58 sq. mi.

Let L = 14,000 feet, CN = 80, t_R = 0.5 hours, and Y = 4%. From the above equations,

S = 2.5 inches

t_l = 1.28 hours

t_p = 1.53 hours

$Q_p \quad = \quad 1450$ cfs/in.

The values of t_p and Q_p form the basis of the unit hydrograph, and the hydrograph ordinates in Table 16.2 are computed from the dimensionless variables t/t_p and Q/q_p, listed in Table 16.1.

Table 16.1. Ratios for dimensionless unit hydrograph and mass curve.

Time Ratios (t/T_p)	Discharge Ratios (q/qp)	Mass Curve Ratios (Qa/Q)
0	.000	.000
.1	.030	.001
.2	.100	.006
.3	.190	.012
.4	.310	.035
.5	.470	.065
.6	.660	.107
.7	.820	.163
.8	.930	.228
.9	.990	.300
1.0	1.000	.375
1.1	.990	.450
1.2	.930	.522
1.3	.860	.589
1.4	.780	.650
1.5	.680	.700
1.6	.560	.751
1.7	.460	.790
1.8	.390	.822
1.9	.330	.849
2.0	.280	.871
2.2	.207	.908
2.4	.147	.934
2.6	.107	.953
2.8	.077	.967
3.0	.055	.977
3.2	.040	.984
3.4	.029	.989
3.6	.021	.993
3.8	.015	.995
4.0	.011	.997
4.5	.005	.999
5.0	.000	1.000

Table 16.2. Computation of coordinates for unit hydrograph for use in Example 1.

(1) Time Ratios (table 16.1) (t/T_p)	(2) Time (col. 1 x 1.53) (hours)	(3) Discharge Ratios (Table 16.1) (q/q_p)	(4) Discharges (col 3 x 1450) (cfs)
.0	0	0	0
.1	.15	.030	44
.2	.31	.100	145
.3	.46	.190	276
.4	.61	.310	450
.5	.76	.470	682
.6	.92	.660	957
.7	1.07	.820	1189
.8	1.22	.930	1349
.9	1.38	.990	1435
1.0	1.53	1.000	1450
1.1	1.68	.990	1435
1.2	1.84	.930	1349
1.3	1.99	.860	1247
1.4	2.14	.780	1131
1.5	2.29	.680	986
1.6	2.45	.560	812
1.7	2.60	.460	667
1.8	2.75	.390	565
1.9	2.91	.330	479
2.0	3.06	.280	406
2.2	3.37	.207	300
2.4	3.67	.147	213
2.6	3.98	.107	155
2.8	4.28	.077	112
3.0	4.59	.055	80
3.2	4.90	.040	58
3.4	5.20	.029	42
3.6	5.51	.021	30
3.8	5.81	.015	22
4.0	6.12	.011	16
4.5	6.89	.005	7
5.0	7.65	0	0

WORKSHOP NO. 3

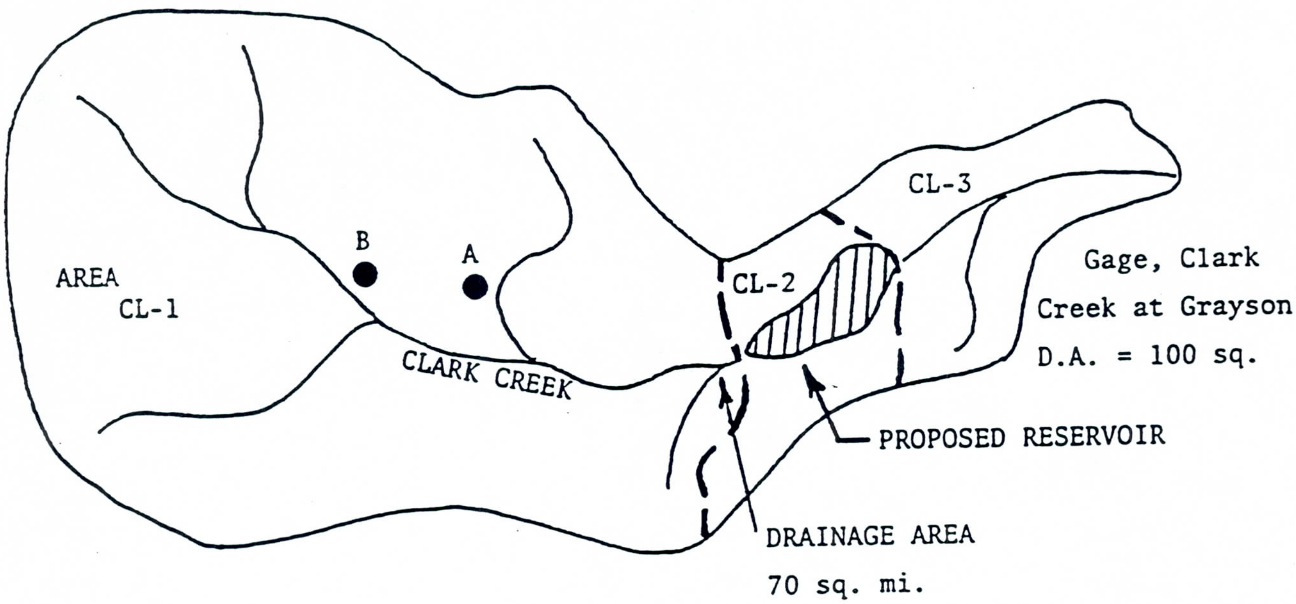
FLOOD HYDROGRAPH USING SNYDER UNIT HYDROGRAPH

PROBLEM STATEMENT

SNYDER UNIT GRAPHS

PROBLEM DESCRIPTION

A 2 hour unit graph is required for the area above the proposed reservoir on Clark Creek. Characteristics of the Clark Creek Basin are shown below.



Point A = Center of gravity of total area above gage. (100 sq. mi.)

L = 20 miles

L_{ca} = 12 miles

Point B = Center of gravity of area above reservoir. (70 sq. mi.)

L^* = 12.6 miles

L^*_{ca} = 7.4 miles

Snyder Synthetic Unit Hydrograph is to be used to derive a synthetic unit graph.

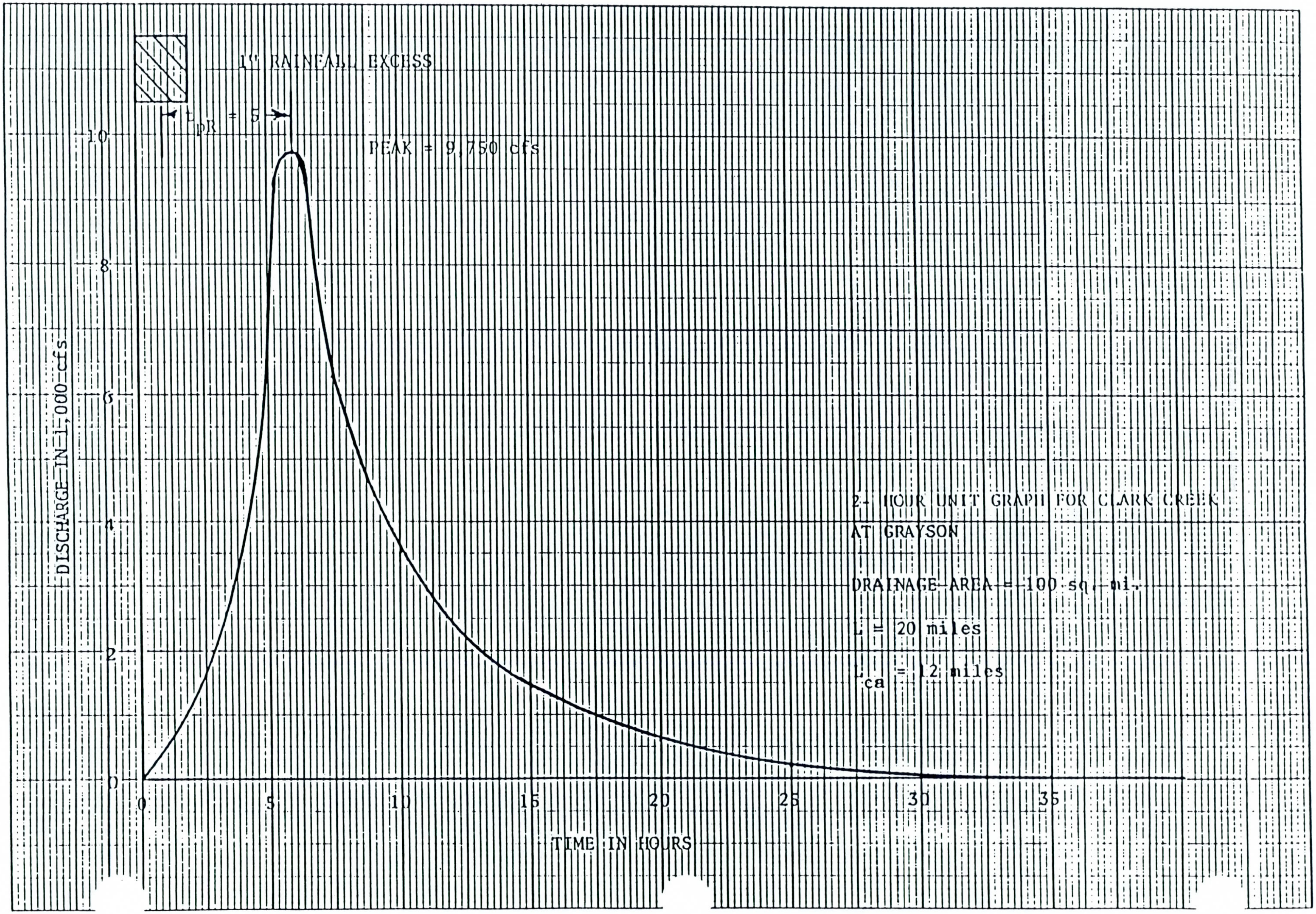
PROBLEM

a. Compute the coefficients C_t and 640 C_p using the unit graph results shown on the following page and the characteristics of the 100 sq. mile area.

b. From the results in "a", determine the t^*_{pR} and Q^*_{pR} values for the synthetic unit graph (70 sq. mile area).

NOTE: Use t^*_R = 2 hours. It should be mentioned that 2 hour periods were used to allow later comparisons of results. The recommended t^*_R in table 1 of Snyder Synthetic Unit Hydrograph would normally be used.

c. Determine widths of the synthetic unit graph at 75% and 50% of peak flow using unit graph data shown on the following page. Plot the calculated points on the following page and sketch that portion of the hydrograph. The remainder of the recession limb may be determined by the method of "Synthetic Unit Hydrograph Slope", using 1-hour increments to achieve adequate definition of this 2-hour UHG.


1" RAINFALL EXCESS
t_{pR} = 5
PEAK = 9,750 cfs
DISCHARGE IN 1,000 cfs
0
2
4
6
8
10
0
5
10
15
20
25
30
35
TIME IN HOURS
2- HOUR UNIT GRAPH FOR CLARK CREEK
AT GRAYSON
DRAINAGE AREA = 100 sq. mi.
L = 20 miles
L_{ca} = 12 miles

WORKSHOP NO. 3

FLOOD HYDROGRAPH USING SNYDER UNIT HYDROGRAPH

PROBLEM SOLUTION

SYNDER UNIT GRAPHS

Problem Solution

a. Given unit hydrograph at Grayson:

$A = 100$ sq. miles

$L = 20$ miles

$t_{pR} = 5$ hours

$L_{ca} = 12$ miles

$t_R = 2$ hours

$Q_{pR} = 9750$ cfs

Determine local coefficients

$$C_t = \frac{t_{pR} - 0.25t_R}{0.955(L \times L_{ca})^{.3}}$$

$$= \frac{5 - 0.25(2)}{0.955(20 \times 12)^{.3}} = \frac{5 - .5}{.955(240)^{.3}} = \frac{4.5}{.955(5.18)}$$

$$\underline{C_t = 0.91}$$

$$640C_p = \frac{Q_{pR}\, t_{pR}}{A} = \frac{9750(5)}{100}$$

$$\underline{640C_p = 487}$$

b. Determine 2 hour unit hydrograph above reservoir

$A^* = 70$ sq. miles

$L_{ca} = 7.4$ miles

$L^* = 12.6$ miles

$$t^*_{pR} = C_t(0.955)(L^* \times L^*_{ca})^{.3} + 0.25t^*_R$$

$$= .91(.955)(12.6 \times 7.4)^{.3} + .25(2)$$

$$\underline{t_{pR} = 3.89 \text{ hours}}$$

$$Q^*_{pR} = \frac{640C_p\ A^*}{t^*_{pR}}$$

$$= \frac{487(70)}{3.89}$$

$$\underline{\underline{Q^*_{pR} = 8760 \text{ cfs}}}$$

c. Widths from the given unit graph for gage site are as follows:

W^*_{75} = 2.0 hours

W^*_{50} = 4.2 hours

Plot these points on the grid at q^*_{pR} = 97.5 cfs/sq.mi. and construct curves parallel to envelope curves for respective widths. The resulting values read from the constructed curves for the synthetic unit hydrograph (q^*_{pR} = 125.4 cfs/sq. mi.) are as follows:

W^*_{75} = 1.5 hours

W^*_{50} = 3.2 hours

Using these values and Q^*_{pR} = 8760 cfs, a synthetic unit hydrograph can be sketched except for the recession limb. For remaining volume calculations, the following values are tabulated from the sketched curve:

Period (hours)	Average Ordinate (cfs)
0-1	300
1-2	1100
2-3	2200
3-4	4700
4-5	8100
5-6	6900
6-7	4600

Ordinates = 27,900 one hr-cfs

$$\text{Total UHG Vol.} = \frac{70 \text{ sq. mi.}(640 \text{ Ac/sq. mi.})}{(12 \text{ in/ft})} = 3733 \text{ AF}$$

$$= \frac{(3733 \text{ AF})(24 \text{ hr period/day})}{(1.9835 \text{ AF/cfs})} = 45{,}173 \text{ one hr-cfs}$$

Remaining Vol. = 45,173 - 27,900 = 17,273 one hr-cfs

$$K = 1 - \frac{4000}{4000 + 17{,}273} = 1 - 0.188 = 0.812$$

Computation of the recession limb ordinates

Time Ending (hrs)	2-Hr Unit Graph (cfs)	
1	650	
2	1600	
3	3200	
4	6500	27,900
5	8780	
6	5500	
7	4000	
8	3250	
9	2640	
10	2140	
11	1740	
12	1410	
13	1140	
14	930	
15	760	
16	620	
17	500	
18	410	
19	330	

Where:

$UG_n = K \times Q_{prev.}$

$UG_8 = 0.812 \times 4000$

$UG_9 = 0.812 \times 3250$

$UG_{10} = 0.812 \times 2640$